EFFECT OF LOAD PATH ON DAMAGE TO CONCRETE BRIDGE PIERS

by

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ABSTRACT

In earthquake engineering studies of reinforced concrete (RC) bridge columns, a controlled, cyclic lateral load pattern with gradually increasing amplitude has traditionally been applied to laboratory test specimens. However, in actual earthquakes bridge columns are exposed to random cyclic lateral loading patterns, which are much different from typical laboratory loading patterns. Current American Association of State Highway and Transportation Officials (AASHTO) and California Department of Transportation (Caltrans) design provisions are based almost entirely on tests in which traditional, controlled laboratory loading patterns have been applied. The differences in the effects of these types of loading have never been explored systematically. In this study both types of loading (controlled, cyclic lateral loads, and random earthquake type loads) were applied to a series of twelve nominally identical, one-fourth scale circular, cantilever columns, and the differences in observed damage were studied. In this paper the experimental results are briefly summarized, and preliminary findings are discussed.

KEYWORDS: bridge columns; building technology; damage modeling; earthquake engineering; laboratory testing; random loading; reinforced concrete

1. INTRODUCTION

The purpose of this study is to explore both experimentally and analytically the relationship between load path and the inelastic damage induced in reinforced concrete bridge columns. Here "load path" is meant to describe the series of lateral displacements applied at the top of the column. If cyclic lateral loading causes inelastic behavior of a bridge column, the type and extent of damage induced in the column will depend on the order in which the displacements are applied. That is, damage induced by inelastic deformations is path-dependent. In this study a variety of load paths were applied to reinforced concrete bridge columns to investigate the nature of the dependency of damage on load path.

Over the last 25 years, in laboratories around the world, a large number of tests have been performed in which cyclic lateral displacements were applied to the top of cantilever reinforced concrete columns. Although the purpose of these tests was to study the seismic response of the columns, the displacement pattern applied to the columns in most cases did not resemble the displacements induced by an earthquake. The displacement patterns were usually in the form of a sawtooth wave, often with gradually increasing amplitude, as illustrated in Figure 1. However, earthquakes impart displacement patterns with random amplitudes, as shown in Figure 2. The seismic design provisions of the American Association of State Highway and Transportation Officials (AASHTO) and of the California Department of Transportation (Caltrans) are based almost entirely on tests in which the applied displacement pattern had a form similar to that shown in Figure 1. The tests conducted in this study are intended to investigate the relationship between regular, sawtooth displacement patterns widely applied in the laboratory (Figure 1), and the random displacement patterns induced by earthquakes (Figure 2).

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The experimental portion of this study consisted of laboratory tests of 12 nominally identical bridge columns which were subjected to a variety of cyclic lateral displacement patterns. These displacement patterns were in the form of sawtooth waves of constant amplitude, sawtooth waves of gradually increasing amplitude, and random displacement patterns which simulated earthquakes of various intensities. Observations were made of the progress of damage, both visually and with instrumentation. When a specimen was subjected to earthquake-type loads, several simulated earthquakes were applied sequentially to study the effects of multiple earthquakes on the same bridge column (Figure 2). The earthquake displacement patterns applied to the columns were determined from prior nonlinear dynamic analyses of the columns, using recorded earthquake accelerograms as input to the analysis. This process is explained further in Section 3.

The analytical portion of the study includes the formulation and verification of damage model hypotheses. The key feature sought in a damage model is that it apply equally to damage incurred either from a sawtooth wave displacement pattern, or a random displacement pattern. The data gathered from the 12 specimens in this study enable the testing of damage model hypotheses against both types of displacement histories.

In this paper the results of the experimental program are briefly summarized, and observations relevant to damage modeling are discussed. Since analysis of the laboratory data is still ongoing, preliminary findings are presented here. Detailed descriptions of both the experiments and analytical study will be presented in a forthcoming report.

2. EXPERIMENTAL PROGRAM

Figure 3 shows a test specimen and the loading apparatus. The specimens had circular cross sections and were nominally identical. The scale of the specimens was 1:4, and the full scale pier on which the design was based conformed to California Department of Transportation (Caltrans) specifications (Caltrans, 1992). The specimens were cast in an inverted position with an integral base block. Figure 4 shows the arrangement of reinforcement in the specimen and base block. The column confining reinforcement was a continuous smooth spiral wire, which extended through the depth of the specimen base block. Longitudinal reinforcement consisted of 21 deformed bars distributed around the perimeter of the spiral, extending into the base block and terminating in hooks. The measured strength of the concrete (using 150 mm by 300 mm cylinders) was 38 MPa. The maximum concrete aggregate size was 13 mm. The yield strength of the longitudinal reinforcement was 414 MPa, and the yield strength of the spiral reinforcement was 427 MPa.

As shown in Figure 3, additional re-usable base block sections were joined to the specimen base block with post tensioning rods, and the entire base block assembly was post-tensioned to the laboratory strong floor, forming a rigid base. A constant vertical load of 10 percent of the axial capacity of the gross column section was applied through a 200 kN capacity vertical servo-hydraulic actuator. A 670 kN capacity actuator was used to apply cyclic lateral displacements. This second actuator was controlled by a computer and servo-hydraulic system, which were programmed to apply specific lateral displacement patterns to the top of the column. The measured lateral load was corrected to account for the lateral component of the vertical actuator. A system of horizontal guide rails and ball bearing rollers, located at the top of the column, maintained the in-plane alignment of the column during testing. These rails and rollers are not shown in Figure 3 for clarity. Instrumentation consisted of load cells and displacement transducers (LVDT's) for each hydraulic actuator, an array of horizontal LVDT's to measure lateral displacement over the height of the specimen, strain gages on selected longitudinal reinforcing bars near the base of the column, clip-on external strain gages to measure curvature near the base of the column, and inclinometers to measure rotation near the base of the column.

3. DISPLACEMENT HISTORIES

Two types of displacement histories were applied to the specimens in this study: regular, controlled 2-D sawtooth patterns, which have historically been applied in most laboratory tests; and random 2-D patterns, which simulate earthquake loading. Both types of displacement histories were applied slowly, in a pseudo-dynamic mode.

The load paths of the first six specimens are illustrated in Figure 5. Specimen 1 was subjected to monotonically increasing lateral displacement to failure. Since all specimens are nominally identical, this test established basic parameters, such as initial lateral stiffness, displacement at first yield, and ultimate lateral strength, which were used in planning subsequent tests. Specimen 2 was subjected to a "standard" laboratory displacement history, detailed in Figure 1. While there is in fact no commonly accepted "standard" displacement history, many researchers have used a displacement pattern similar to that shown in Figure 1. Several cycles are applied initially at a displacement Δy, which causes first yielding of the column longitudinal reinforcement. These cycles are followed by a single cycle of small amplitude which is used to measure the change in initial tangent stiffness of the column. As shown in Figure 1, subsequent groups of cycles are applied with increasing amplitudes. This continues until substantial degradation of the lateral load carrying capacity of the column is observed, usually defined as the point at which the maximum lateral load in a cycle is only 75 percent to 80 percent of the peak lateral load capacity measured during the test. Specimens 3 to 6 were subjected to constant-amplitude cycles (of ±2Δy, ±3Δy, ±4Δy, and ±5Δy, respectively) until severe deterioration of the column was observed. These tests provided data for calibration of analytical damage models which reflect damage accumulated during constant amplitude cycling.

A typical displacement history for Specimens 7 to 12 is shown in Figure 2. The details of the displacement histories for these specimens are shown in Table 1. These last six specimens were used to study the effects of real earthquake displacement histories on accumulation of damage, particularly over a series of events of varying intensity. The displacement patterns applied to Specimens 7 to 12 were derived from measured earthquake acceleration records, using the following process. Given an earthquake acceleration record, the deformation history at the top of the column was computed using the inelastic dynamic analysis program IDARC (Kunnath et al. 1992). IDARC computes the response of the column by taking into account the degradation in stiffness, lateral strength and energy dissipation capacity (pinching of the hysteresis loops) which occur as inelastic deformations progress over the course of the earthquake. The degree of stiffness degradation, strength degradation and pinching are controlled by three parameters α, β, and γ. These parameters are determined *a priori*, either empirically, based on past experience with tests on similar members (Kunnath et al. 1992), or analytically using correlation rules which relate α, β and γ to the geometric and material properties of the column cross section (Stone and Taylor, 1991, 1993).

The earthquake records used in tests 7 to 12 were selected from a pool of available strong ground motion records recorded at bedrock sites in the state of California, and one record from Mexico City. Each specimen was subjected to a series of up to five earthquakes. Table 1 shows that earthquakes of a range of intensities were applied, and the order in which they were applied was varied. The pool of available earthquake records was subdivided into two main categories: minor events and major events. Minor events were intended to simulate either isolated low-intensity earthquakes, or low intensity aftershocks following a major earthquake. Minor events would cause little or no damage to the specimens in this test program, as determined from the results of the constant amplitude tests (Specimens 3 to 6). Based on observations from these tests, minor events were defined as those which caused three to five excursions into the inelastic range, with ductility demands no greater than 3Δy. Major events were divided into two types: damaging earthquakes and maximum credible events. Again, based on the results of the earlier constant-amplitude tests, damaging events were defined as those which caused multiple excursions into the inelastic range, with ductility demands greater than 3Δy, but generally less than 5Δy. The earlier tests showed that this level of ductility demand caused deterioration of the column, but that the column retained much of its strength and stiffness. Maximum credible events were those which caused cycles with ductility demands greater than 5Δy. The earlier tests also showed that just a few excursions greater than 5Δy resulted in rapid deterioration of strength and stiffness. In some cases, where acceleration records were not available which met the desired criteria, the records were modified by simple amplitude scaling. These scale factors are reported in Table 1.

4. OBSERVATIONS AND DAMAGE MODELING

Preliminary observations from the experimental program are presented here. The experimental data is currently being analyzed, and detailed descriptions of the experimental results will be presented in a forthcoming report. The experiments are discussed in terms of two categories: the constant amplitude tests and the random amplitude (earthquake loading) tests.

4.1 Constant Amplitude Tests

It was observed that repeated cycling of Specimen 3 at a displacement amplitude of $\pm 2\Delta y$ caused almost no degradation of stiffness and strength of the column. After the first full cycle at $\pm 2\Delta y$ (during which initial yielding and cracking occurred) the hysteresis loops remained extremely stable, lying nearly on top of one another for 150 cycles. The test was stopped at 150 cycles not because the column failed, but because it was believed that no further useful information could be obtained by continuing the test. At the other extreme, Specimen 6 was subjected to cycles of $\pm 5\Delta y$ and exhibited a rapid decrease in strength and stiffness, nearly completely losing lateral stiffness and load capacity after only 5 cycles. This rapid deterioration is illustrated in Figure 6. Specimen 5, which was cycled at $\pm 4\Delta y$, exhibited a gradual decrease in strength and stiffness as cycling progressed (Figure 7), but the decrease was not nearly as rapid as for Specimen 6. This is illustrated in Figure 7.

Thus, for the specimens tested in this program there appears to be a threshold ductility level, of about $4\Delta y$, above which deterioration is rapid and severe. It could be surmised that earthquakes which induce displacements of less than $4\Delta y$ in the columns tested in this program would cause much less damage than those which induce displacements greater than $4\Delta y$. Indeed, subsequent testing under earthquake displacement patterns (Specimens 7 to 12) confirmed this observation: a series of several minor events, which caused few excursions greater than $2\Delta y$, would result in very little damage to a column, while a single earthquake with a few excursions greater than $4\Delta y$ would result in rapid deterioration of strength and stiffness. The apparent threshold of $4\Delta y$ applies only to the columns tested in this program. It could be expected that similarlydesigned circular columns, with high ratios of confining reinforcement (such as those designed under the Caltrans specifications), and which were dominated by flexural rather than shearing deformations, would also exhibit such a threshold, but the level of the threshold would not necessarily be $4\Delta y$.

Another significant observation from the constant amplitude tests was that cumulative dissipated energy (the area contained within the hysteresis loops) is not good predictor of column failure. It was found that the cumulative dissipated energy at failure depended on the amplitude of the sawtooth wave. Figure 8 shows the accumulation of energy to failure for Specimens 2, 4, 5 and 6 ("standard" displacement pattern, $\pm 3\Delta y$, $\pm 4\Delta y$, and $\pm 5\Delta y$, respectively). Results for Specimen 3 are not plotted because the specimen did not deteriorate significantly under 150 cycles at $\pm 2\Delta y$, so a failure state was not achieved. In Figure 8 it can be seen that dissipated energy at failure is strongly dependent on the displacement history. Therefore, because of the highly variable nature of earthquake-induced displacement histories, it is apparent that cumulative energy alone is not an acceptable measure of column damage.

4.2 Earthquake Loading Tests

A typical lateral load-displacement plot for one of the earthquake loading tests is shown in Figure 9. The two main observations made in these tests are described below.

First, it was observed that "minor" events (Table 1) had little or no effect on the ultimate failure of a column. That is, it made no difference if minor events occurred before or after a damaging event: the level of damage following the damaging event was essentially the same in both cases. This has important implications for the management of earthquake damage to bridge columns. It would appear that minor seismic events should be considered to have little effect on well-confined, circular, flexure-dominated bridge columns. It remains for transportation authorities to define precisely what a "minor" event is, but it appears acceptable to not discount the strength and stiffness of well-confined columns which have been subjected to a minor earthquake.

Second, it was observed that failure of the columns could be classified into two general types: failure due to low cycle fatigue of the longitudinal reinforcement, called here a "low cycle fatigue failure"; and failure due to rupture of confining reinforcement, or "confinement failure." Confinement failures were observed to occur more frequently than low cycle fatigue failures. While these two classes of failure have been observed and reported by others, it is interesting to recall that the only variable in these tests was the displacement history. Thus displacement history, rather than the column configuration, determined the failure mode. This illustrates the importance of conducting nonlinear dynamic analyses of bridge columns whenever feasible, and considering a range of possible ground motions at the bridge site, rather than permitting a single earthquake record to determine the design.

With regard to analytical damage modeling, preliminary analyses of the test results indicate that cumulative fatigue models, based on variations of the classic Miner's rule (Miner 1945) for metals, can successfully track damage in columns which fail due to low cycle fatigue of the longitudinal reinforcement. However, this same approach does not apply well to columns which exhibit confinement failure. Other methods of modeling confinement failure are currently being investigated. Furthermore, the columns tested in this study were dominated by flexural behavior, and none of the findings stated here necessarily apply to short, shear-dominated columns. A similar experimental investigation of shear-dominated columns is planned.

5. CONCLUSIONS

The main findings of this study to date are summarized below. Since interpretation of the test data is still ongoing, these findings must be considered preliminary.

1) There appears to be a threshold ductility level for well-confined, circular, flexure-dominated columns above which degradation of stiffness and strength is rapid. For the columns tested in this study this threshold ductility level was about $4\Delta y$. Below this threshold degradation was more gradual, and for ductility demands of $2\Delta y$ or less degradation was minimal.

2) The constant amplitude tests in this study confirm that cumulative dissipated energy at failure is strongly path dependent. Therefore cumulative energy by itself is not a reliable measure of column damage.

3) Minor earthquakes appear to have very little influence on the damage exhibited by well-confined, circular, flexure-dominated bridge columns in subsequent major events. While the precise definition of a "minor" event is open to interpretation, this study indicates that the strength and stiffness of a well-confined, circular, flexure-dominated bridge column should not be discounted because the column has been subjected to minor events.

4) The random loading tests indicate that failure mode is path dependent. Nominally identical columns failed either due to low-cycle fatigue of longitudinal reinforcement, or due to rupture of confining reinforcement. The only variable between tests was the load path. The relationship between load path and failure mode is currently under study.

5) Cumulative fatigue methods, based on Miner's rule for fatigue of metals, appear to predict well the progress of damage in columns which failed due to low cycle fatigue of longitudinal bars. However, Miner's rule does not apply to those columns which failed due to rupture of the confining reinforcement. Other damage models are being investigated which may apply to confinement failure. Furthermore, the findings of this study apply only to well-confined, circular, flexure-dominated columns. A future study is planned to investigate well-confined, circular, shear-dominated columns.

6. ACKNOWLEDGMENTS

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Table 1: Earthquake records applied to Specimens 7 to 12

Spec. No.	Event No.	Description: Event Severity and Number	Purpose of Applying Event to Column	Earthquake Record	Scale Factor	Scaled PGA g's
7	1	Damaging No. 1	Significant damage	Loma Prieta 1989, Presidio	12.0	1.20
	2	Minor No. 1	Aftershock	Imperial Valley 1979, Superstition Mtn.	1.8	0.34
	3	Minor No. 2	Second aftershock	San Fernando 1971, 2011 Zonal Avenue	1.2	0.10
	4	Damaging No. 2	Failure of column	San Fernando 1971, 455 S. Figueroa St.	3.6	0.54
8	1	Minor No. 1	Minor damage	Imperial Valley 1979, Superstition Mtn.	1.8	0.34
	2	Minor No. 2	Minor damage	San Fernando 1971, 2011 Zonal Avenue	1.2	0.10
	3	Damaging No. 1	Significant damage	Loma Prieta 1989, Presidio	12.0	1.20
	4	Damaging No. 2	Failure of column	San Fernando 1971, 455 S. Figueroa St.	3.6	0.54
9	1	Damaging No. 3	Significant damage	San Fernando 1971, Orion Boulevard	3.25	1.43
	2	Minor No. 2	Aftershock	San Fernando 1971, 2011 Zonal Avenue	1.2	0.10
	3	Damaging No. 4	Moderate damage	El Centro 1940	1.0	0.35
	4	Minor No. 3	Aftershock	San Fernando 1971, 455 S. Figueroa St.	1.0	0.15
	5	Damaging No. 3	Failure of column	San Fernando 1971, Orion Boulevard	3.25	1.43
10	1	Minor No. 2	Minor damage	San Fernando 1971, 2011 Zonal Avenue	1.2	0.10
	2	Damaging No. 4	Moderate damage	El Centro 1940	1.0	0.35
	3	Minor No. 3	Aftershock	San Fernando 1971, 455 S. Figueroa St.	1.0	0.15
	4	Damaging No. 3	Significant damage	San Fernando 1971, Orion Boulevard	3.25	1.43
	5	Damaging No. 3	Failure of column	San Fernando 1971, Orion Boulevard	3.25	1.43
11	1	Damaging No. 4	Significant damage	Northridge 1994, VA Hospital	1.0	0.42
	2	Minor No. 4	Aftershock	Northridge 1994, Griffith Observatory	1.0	0.26
	3	Minor No. 5	Minor damage	Taft 1952	1.0	0.36
	4	Damaging No. 5	Failure of column	Mexico City 1985, SCT	1.0	0.17
12	1	Minor No. 4	Minor damage	Northridge 1994, Griffith Observatory	1.0	0.26
	2	Minor No. 5	Minor damage	Taft 1952	1.0	0.36
	3	Damaging No. 4	Significant damage	Northridge 1994, VA Hospital	1.0	0.42
	4	Damaging No. 5	Failure of column	Mexico City 1985, SCT	1.0	0.17

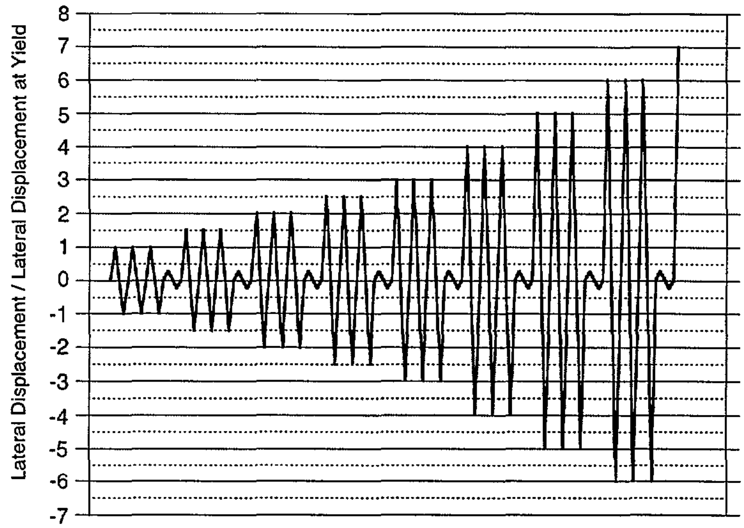

Figure 1: Typical laboratory displacement history, applied to Specimen 2

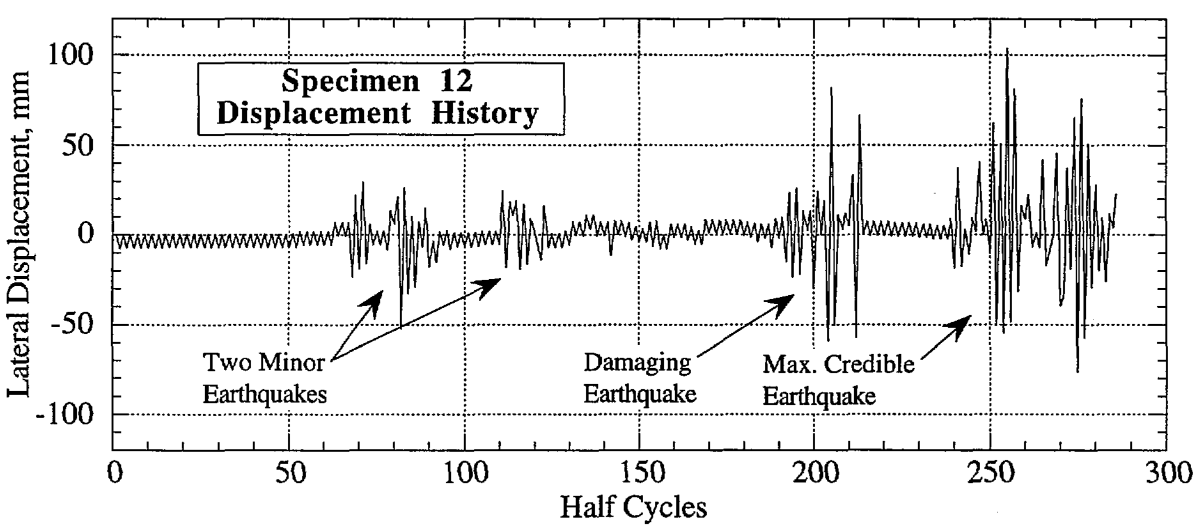

Figure 2: Typical earthquake displacement histories, applied to Specimen 12

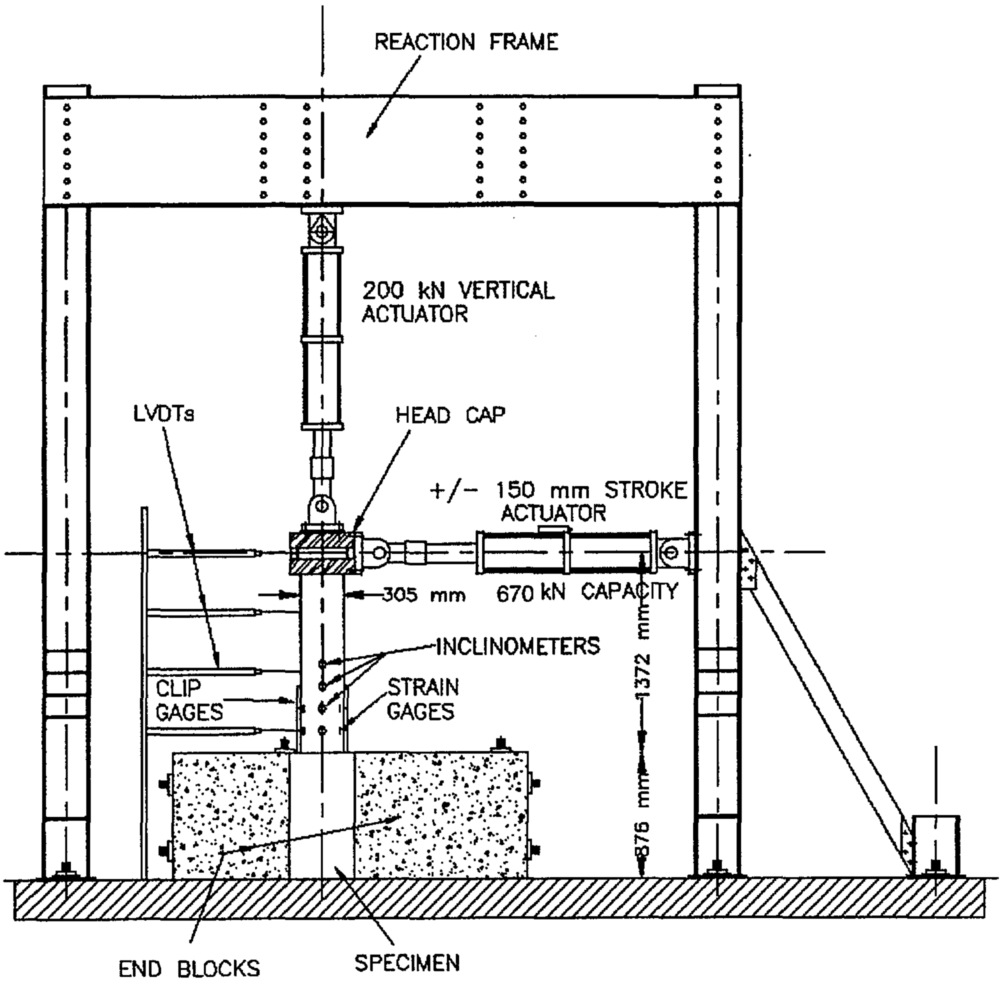

Figure 3: Load frame and instrumentation for cyclic lateral load tests

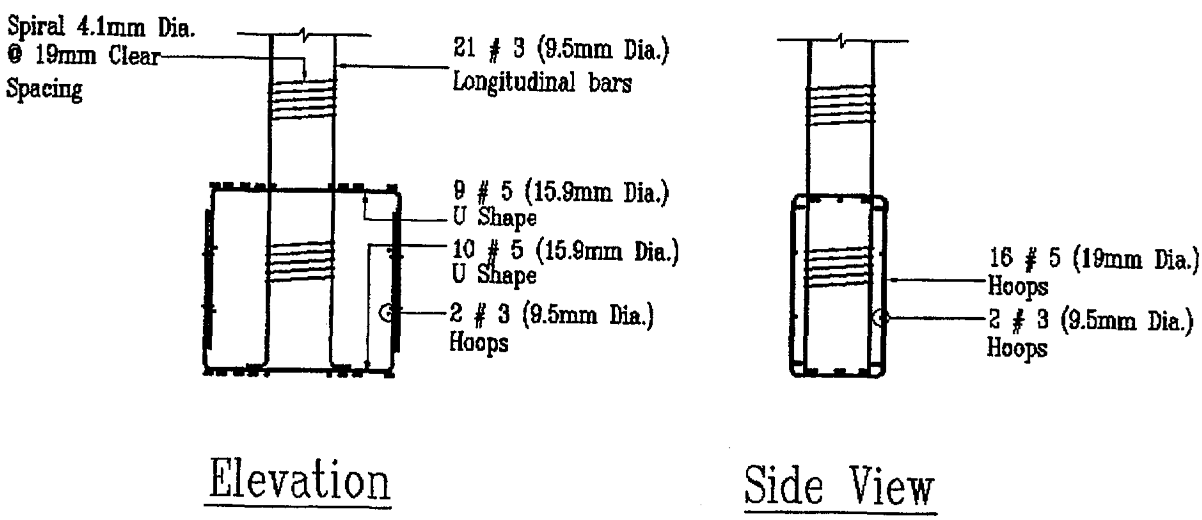

Figure 4: Reinforcement details for column specimen and base block

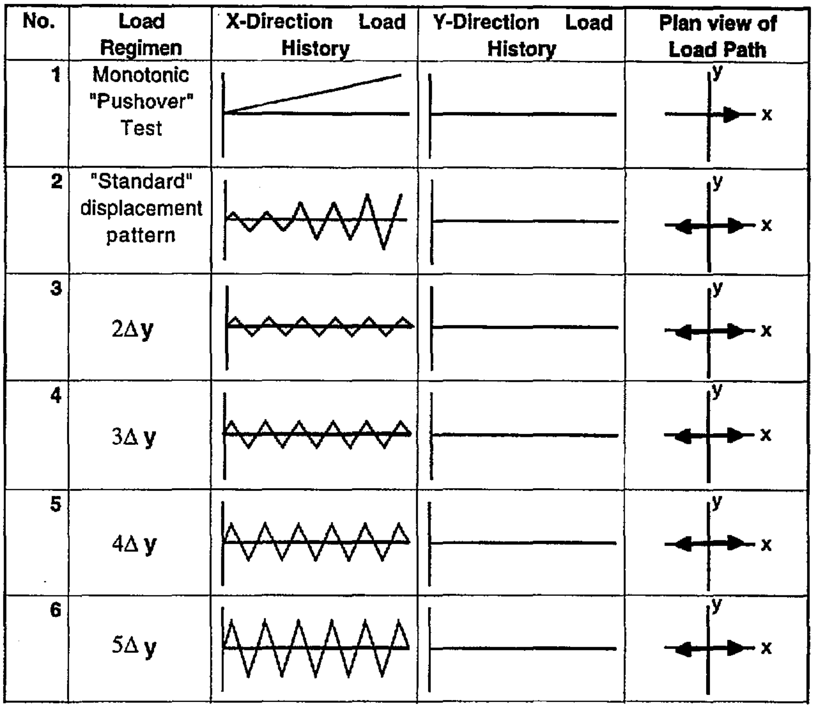

Figure 5: Displacement histories of Specimens 1 to 6

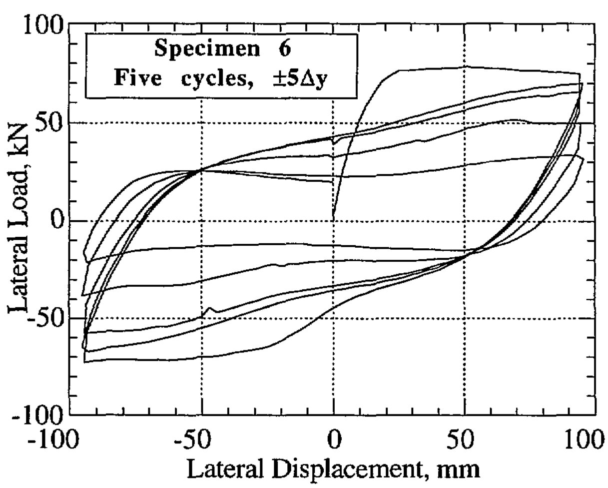

Figure 6: Lateral load-displacement plot, Spec. 6

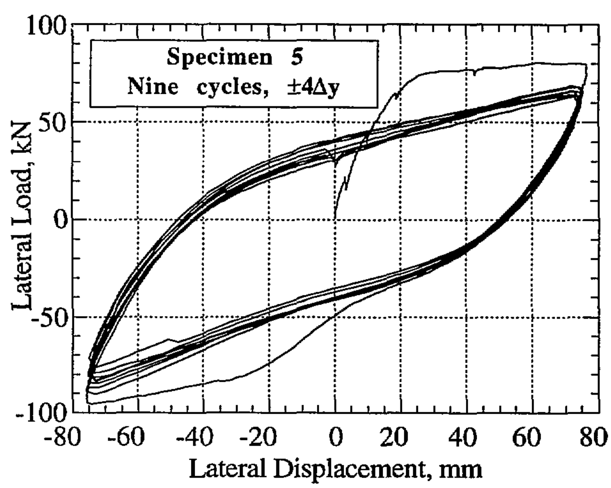

Figure 7: Lateral load-displacement plot, Spec. 5

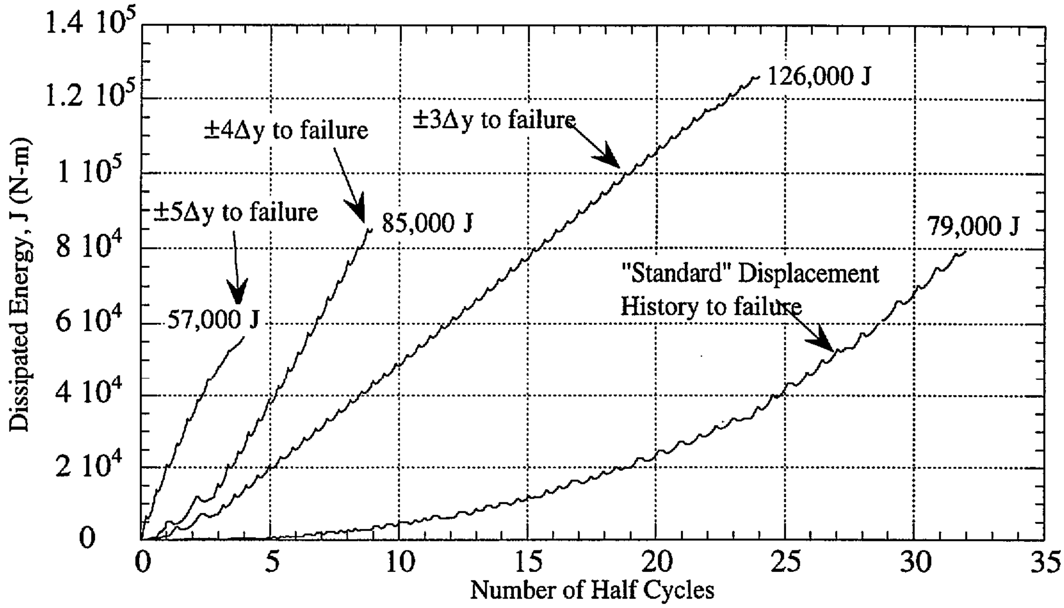

Figure 8: Cumulative energy to failure for Specimens 2, 4, 5 and 6

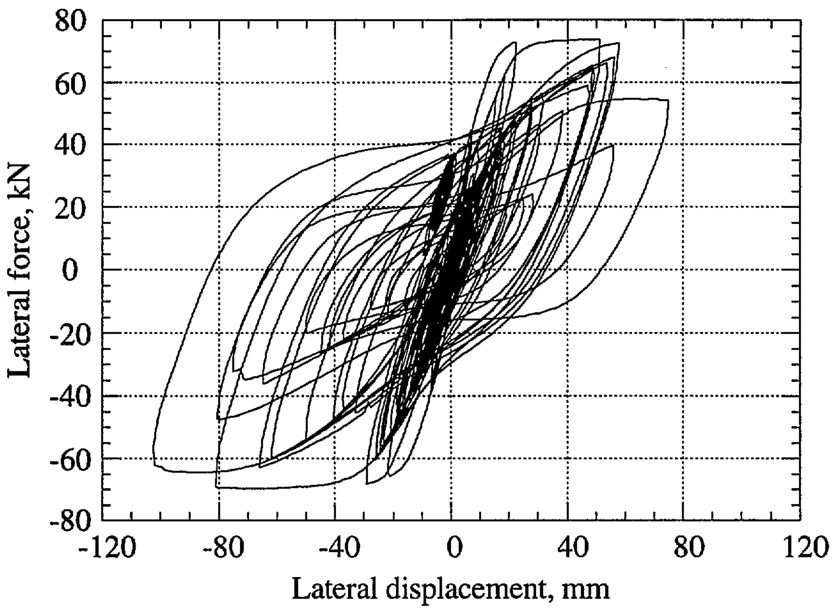

Figure 9: Lateral load-displacement plot for Specimen 12