

Seismic Behavior and Modeling of Reinforced Concrete Building Columns

by

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B.S. (Middle East Technical University, Ankara, Turkey) 1993

M.S. (Cornell University, Ithaca, New York) 1996

A dissertation submitted in partial satisfaction of the requirements for the degree of

Doctor of Philosophy

in

Engineering - Civil and Environmental Engineering

in the

GRADUATE DIVISION

of the

UNIVERSITY OF CALIFORNIA, BERKELEY

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Fall 2002

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Abstract

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Doctor of Philosophy in Engineering - Civil Engineering

University of California, Berkeley

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Post-earthquake reconnaissance and experimental research indicate that columns with insufficient transverse reinforcement and poor seismic details are vulnerable to earthquake damage in existing buildings. One of the major objectives of this research was to identify main factors contributing to shear failure and gravity load collapse of lightly reinforced concrete columns. Four full-scale column specimens were constructed and tested statically as part of the experimental investigation. The columns were tested in double bending under uni-directional lateral load.

Test results showed that the response of older columns with nominally identical properties depended largely on the magnitude and history of axial and lateral loads. Specimens with low axial load lost their lateral strength substantially at low displacement ductility, but sustained axial load at large displacements. Under the same flexural demand and very high axial load, lateral stiffness and strength increased at low

displacements, however, the specimen had a sudden shear and axial load failure. Under monotonic lateral load, and under very low compressive or tensile axial loads, the lateral strength degradation was less severe.

The analytical phase of this investigation included development of a model to predict the shear strength of columns, and comparison with the experimental results from a database of more than fifty columns with shear failure. Based on a statistical evaluation, a strength reduction factor is proposed for design and assessment purposes. The proposed shear strength model is also compared with other existing models.

The analytical investigation also included examination of the behavior of columns with significant stiffness and strength degradation due to shear failure after the flexural strength is attained. Based on test results, simplified analytical procedures are developed to model the flexure, longitudinal bar slip, and shear behavior under monotonic lateral load. The flexure and longitudinal bar slip models are based on fiber section moment-curvature analysis with uniaxial material properties. The shear model is developed using the data from experiments and previous theoretical investigations. A model with three springs in series is proposed to combine individual monotonic deformation models. The measured cyclic response of test columns compared well with the response calculated from the proposed spring model.

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ACKNOWLEDGMENTS

I would like to express my sincerest gratitude to my supervisor, Professor Jack Moehle for his continuing support and guidance throughout this research. I always wanted to learn more from him. I was impressed with his knowledge, professionalism and his friendly approach to his students.

I am grateful to many people for the truly inspiring, challenging, and enjoyable years I spent at Berkeley. First, I would like to thank Professor Anil Chopra for his support during my difficult first year. I would also like to thank my friends and mentors Professors Khalid Mosalam and Andrew Whittaker. I also enjoyed the stimulating and fruitful discussions with Professors Stephen Mahin and Vitelmo Bertero.

I should emphasize that the experimental work described in this thesis would not have been possible without the assistance of laboratory personnel at the Richmond Field Station, namely, Wes Neighbor, Don Clyde, and Changrui Yin. Thanks also to the personnel at the Richmond Field Station. Without them, the life at the Field Station would not be pleasant.

I would like to thank to my friend Mahmoud Hachem, who has been my classmate and officemate since I met him my first day at U.C. Berkeley. I would also like to thank many friends for their support and helping me over the course of my studies, especially, Ken Elwood, Abe Lynn, Charles Chadwell, Ayhan Irfanoglu, Chatpan Chintanapakdee, Janise Rodgers, Patxi Uriz, Keri Ryan, Clay Naito, Dawn Lehman, Michael Scott, Afsin Saritas, Silvia Mazzoni, Gilberto Mosqueda, Andy Espinoza, and Brian Buckman.

Finally, I would like to thank my parents, my wife and family. Without their love, patience and support, I could not have completed this work.

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1 INTRODUCTION

1.1 INTRODUCTORY REMARKS

The poor performance of some older reinforced concrete buildings during earthquakes has caused concern about the vulnerability of a class of older existing buildings to damage or collapse. Most of those buildings were designed and constructed in accordance with standards that do not meet current seismic code requirements. Post-earthquake reconnaissance and experimental research indicate that columns with light transverse reinforcement are most vulnerable to damage. Such details were permitted in regions of high seismicity in the US until the mid-1970s. Similar details still are used in regions of lower seismicity in the US and other parts of the world.

The research described in this study was initiated to examine the shear and gravity load failure of columns with insufficient and poorly detailed transverse reinforcement. The

research included both analytical and experimental investigation of behavior of such columns. Using information from an inventory of older buildings, recent experimental test data, and related analytical research results, four full-scale columns were designed in accordance with older building code standards, constructed in a laboratory, and tested under simulated gravity and seismic loading as part of the experimental investigation. The behavior of columns subjected to various levels of axial loads, and reversed cyclic and monotonic lateral loads were studied. As part of the analytical work, the strength and deformation capacity of the columns were investigated. New models were proposed to predict the load-deformation relations and shear strength of columns.

1.2 DETAILS FOUND IN EXISTING CONSTRUCTION

Two surveys conducted by the firm of Rutherford & Chekene (Oakland, CA) identified typical column details found in older existing construction. The survey results are provided in Lynn (2001). The surveys were done on buildings constructed on the west coast between 1919 and 1971. Typical column cross-sectional dimensions ranged from 14 in. to 48 in. The most common cross sections measured either 18 in. by 18 in. or 24 in. by 24 in. In general, the axial load ratio, $P/(A_g f'_c)$, was less than thirty percent (P = calculated service load including dead load and live load, f'_c = specified concrete compressive strength, A_g = gross cross-sectional area). Typical specified concrete compressive strength was about 3000 psi. The specified yield strength of the steel reinforcement varied between 33 ksi and 60 ksi, with 40 ksi and 60 ksi being the most common values. The longitudinal steel ratio, ρ_l , which is defined as the ratio of the longitudinal steel area to gross area, A_g , ranged from 0.5 percent to 4.3 percent.

Typically, No. 3 column ties were spaced at 12 inches over the midheight of the column, with smaller spacing sometimes used near column ends. Both 90-degree and 135-degree hooks were found at the end of column ties. Table 1.1 summarizes the detailing requirements found in recent and current building codes.

A survey conducted for the preliminary seismic evaluation of older reinforced concrete buildings at the University of California, Berkeley campus (Comerio 2000) showed that many of the existing campus buildings have structural components with poor seismic details. Figure 1.1 shows one building, constructed in 1961 and under retrofit in 2001, which includes gravity-load-carrying columns on the perimeter. The 79-ft-tall building is being retrofitted with walls at both ends of the building as shown in the figure. During the retrofit construction, the cover concrete was removed from selected columns. Figure 1.1 shows that widely spaced transverse reinforcement with 90-degree hooks were distributed uniformly over the height of a perimeter column. Behavior of columns having similar details is the primary subject of this study.

1.3 PERFORMANCE OF REINFORCED BUILDINGS IN PAST EARTHQUAKES

Summaries of the performance of reinforced concrete buildings in past earthquakes are provided in the literature (e.g., Moehle and Mahin 1991, and Bertero 2000). Lessons and prominent observations summarized in those documents and other earthquake reconnaissance reports (e.g., Sezen et al. 2001, and EERI 2000) indicate that damage in poorly detailed columns is a primary cause for significant structural damage including

excessive permanent drift and building collapse. Some examples are described in this section.

The Imperial County Services Building, which was severely damaged and nearly collapsed during the 1979 Imperial Valley, California earthquake, is a well-studied example with notable column failures (e.g., Kreger and Sozen 1983, Zeris and Altmann 1984, and Leeds 1980). As illustrated in Figure 1.2, the corner and end columns in the first story likely were subjected to significant shear, bending and axial forces transferred through a discontinued shear wall above the first story as well as through framing action in the orthogonal direction. As a result, these columns failed as shown in Figure 1.3. Exterior frames of the building were intended to carry gravity loads and were not designed to be ductile. The cross-sections of the exterior columns shown in Figure 1.3 were 24 in. square. The transverse reinforcement, which included No. 3 hoops and cross ties, was spaced at 12 in. on center over the height of the column between the ground floor slab line and 2 ft – 2 in. below the girder framing (where the spacing was smaller). The localized column failure shown Figure 1.3.a indicates that the amount of transverse reinforcement was not sufficient to ensure ductile response for the imposed loading.

Another well-studied example is the Olive View Hospital, which nearly collapsed during the 1971 San Fernando, California earthquake (Mahin et al. 1976, Ersoy et al. 1973, and Bertero and Collins 1973). Most of the first-story columns failed during this earthquake (Figure 1.4). The inset in the figure shows the transverse reinforcement detailing for the corner column in the figure. No. 3 column ties and cross ties were spaced at 18 inches uniformly over the height of the column. Column cross-sectional drawings indicate that

typically 90-degree (and sometimes 180-degree) hooks were used at the end of column ties. It should be noted that most of the column tie hooks opened up when the column was subjected to excessive lateral deformations as shown in Figure 1.4.

Figure 1.5 shows a seven-story building used as a hotel in Van Nuys, California. The building has been modeled and analyzed as a case study building in numerous studies since it was first damaged during the 1971 San Fernando earthquake (e.g., Blume 1973, Shimizu 1985, and Moehle et al. 1998). During the 1994 Northridge earthquake, the building sustained severe damage including column damage in the fourth story (Figure 1.5). Similar to the other examples presented above, typical tie spacing and transverse reinforcement configuration of the exterior columns did not meet the ductile detailing requirements of current building codes.

The performance of reinforced concrete buildings in recent earthquakes around the world demonstrates that column failure is one of the most common causes of the building damage and collapse in other parts of the world as well. Figure 1.6 shows three examples of column failures (out of countless column failures and often resulting building collapses) occurred during the 1999 Kocaeli, Turkey, earthquake (Sezen et al. 2001, and EERI 2000). It is striking that, even though the first-story perimeter column shown in Figure 1.7 apparently lost its lateral and axial-load-carrying capacity, the building did not collapse after the earthquake. It appears that in some buildings, the failure of an isolated column may not be catastrophic if the vertical and lateral loads can be redistributed to adjacent elements in the building.

1.4 RESEARCH SIGNIFICANCE AND IMPETUS

Figure 1.8 shows an idealization of typical building frame subjected to gravity and lateral earthquake loads. Performance of existing buildings in earthquakes indicates that the beams (component 1 in Figure 1.8) are less vulnerable to damage during earthquakes and their damage appears to be less critical to performance as compared with that of columns and beam-column joints (Moehle and Mahin, 1991, and Sezen et al. 2001). Post-earthquake reconnaissance and laboratory tests (e.g., Moehle et al. 1994) suggest that in older existing buildings with poorly detailed frame components, a preferable inelastic action in beams most likely would not take place. Considering that the weak beam-strong column design philosophy was not widely implemented before mid-1970s, and considering that gravity loading often dictated the member strengths, inelastic action under earthquake loading commonly is limited to the columns in older building frames.

Past earthquakes show that exterior and interior beam-column joints (components 2 and 3 in Figure 1.8, respectively) with poor details and proportions might be susceptible to significant damage with ensuing reduction in the strength and ductility of the joints or adjacent framing members. The behavior of beam-column joints with typical details found in older existing construction is the subject of other research reported in the literature (e.g., Moehle et al. 1994, and Lehman et al. 2002).

One of the major goals of this research is to analytically and experimentally investigate the lateral and axial behavior of interior and exterior columns (components 4 and 5 in Figure 1.8, respectively). Assuming that the column axial forces due to the vertical

component of the ground acceleration are relatively small compared with those from gravity loads, the interior columns of typical frames will be subjected to nearly constant gravity loads during an earthquake. In contrast, the exterior and corner columns will be subjected to significantly varying axial loads mainly due to overturning effects. Recognizing these differences, the research reported here considers columns under both constant axial load and varying axial load.

In cases where the column sustains inelastic action and extensive damage, it is of interest to sort the inelastic action mechanism into shear and flexure. It is also possible that a poorly detailed slender column could fail in shear following a flexure failure. A simple way to check for shear failure in a frame system with double-curvature columns and strong beams is to compare the column shear strength, V_n , with the maximum probable shear force required for the plastic hinge formation at column ends, $V_p (= 2M_p/L$, where M_p = maximum plastic moment capacity of the column, and L = clear height of the column). This type of behavior mode, where the inelastic response is initiated by shear (i.e., $V_n < V_p$), was not the focus of this research.

Alternatively, if the column shear strength is large enough, inelastic action mechanism initiates with flexural yielding in relatively tall columns. However, after the flexural yielding, if sufficient transverse reinforcement with seismic details is not provided at the column ends, the column still could fail in shear with a subsequent loss of lateral and/or axial-load-carrying capacity. One goal of this research is to predict the column failure after flexural yielding under various axial and lateral load histories.

Another motivation behind this research was to observe the effect of axial load on shear and gravity load failure mechanisms. In lightly reinforced columns after the shear failure, degradation of the core concrete may lead to loss of gravity-load-carrying capacity. A sudden loss of column axial capacity will lead to transfer of column gravity loads to neighboring frame members with ensuing dynamic redistribution of forces within those members, and a possible subsequent building collapse. The magnitude of the axial load and the variation in the axial load history are the key loading parameters in determining how and when the gravity load failure occurs. The focus of this study was to further the understanding of gravity load failure mechanism in relation to shear failure and the nature of the axial load.

1.5 OBJECTIVES AND SCOPE

The emphasis of this investigation is the behavior, evaluation, and modeling of older existing building columns that have widely spaced transverse reinforcement, and hence, are susceptible to shear and axial load failure. Existing analytical and experimental research on columns of this type is reviewed. Gaps in existing information are identified and used to guide development of an experimental and analytical study.

A main objective of the experimental investigation was to observe and study the effect of different axial load and lateral-load history on the lateral and axial-load failure mechanisms of columns. Two columns were tested under different constant axial loads with reversed cyclic lateral load. Another column examined the behavior under constant axial load and monotonically increasing lateral load. A fourth test column examined the

effect on behavior of varying axial load while lateral displacements were cycled. In all tests, loading was continued until specimens were no longer capable of supporting axial loads.

Analytical investigation included development of models to predict the shear strength of columns, and monotonic lateral load-deformation response including flexure, longitudinal bar slip, and shear components.

1.6 ORGANIZATION

A review of previous research related to primary parameters affecting the shear behavior of reinforced concrete columns is provided in Chapter 2. The main parameters studied are the shear, flexural and axial strength of columns with an emphasis on the influence of longitudinal and transverse reinforcement, and the magnitude and history of lateral and axial loads. Chapter 2 also presents an overview of the most widely used evaluation methods and design equations for the prediction of shear strength of existing columns as well as new columns. These equations were used to model the shear strength of the column specimens constructed and tested as part of this investigation before the experiments were carried out. A short summary and comparison of the shear strength equations and a summary of the available procedures to estimate the yield displacement of reinforced concrete columns are also included in this chapter.

Chapter 3 describes the test program including the test specimen details, material properties, test specimen construction, test setup, instrumentation, and the loading procedure.

Chapter 4 presents a detailed description of the behavior of each specimen at various stages during the experiments. Measured relations including descriptions of procedures used to reduce digital data are presented. Behavior of test specimens is compared quantitatively and qualitatively.

Chapter 5 compares measured responses due to flexure and longitudinal bar slip with results obtained from analytical models. Moment-curvature response of a typical test specimen is computed using a fiber cross-section model and uniaxial material properties. Effects of longitudinal bar slip from end beams are evaluated. Calculated and measured lateral load-displacement relations due to flexure and bar slip under monotonic lateral loading are compared.

Using an experimental database of lightly reinforced columns with relatively large aspect ratios, Chapter 6 evaluates the effectiveness of various column shear strength equations. Results from various existing models are examined in relation to the database results. The effects of several response parameters, such as displacement ductility, axial load, column aspect ratio, and transverse reinforcement, on the column shear response are evaluated. An alternative shear strength model is proposed.

In Chapter 7, the effects of shear deformations are evaluated for each test specimen. A monotonic response envelope is defined to represent the lateral load-shear deformation response of a lightly reinforced column. The contribution and effect of flexural, bar slip and shear deformation components on the total column response are evaluated, and a

model with three springs in series is proposed to represent the total monotonic lateral load-displacement relationship.

Chapter 8 presents conclusions and provides a summary of recommendations for future research.

Table 1.1 Column dimension and detailing requirements in recent US building codes

	1961 UBC	ACI 318-71	ACI 318-02
Minimum width, w	10 in.	none	12 in. or 0.4*D
Minimum depth, D	12 in.	none	12 in.
ρ (longitudinal)	$0.01 \leq \rho \leq 0.08$	$0.01 \leq \rho \leq 0.06$	$0.01 \leq \rho \leq 0.06$
Column middle zone: tie spacing	minimum of: 16*d _b , 48*d _{b-tie} , min. dimension	minimum of: 16*d _b , 48*d _{b-tie} , min. dimension minimum of: 12 in., d/2 (ductile columns)	minimum of: 6 in., 6*d _b
Column end zone: tie spacing		minimum of: 16*d _b , 48*d _{b-tie} , min. dimension d/4 (ductile columns)	minimum of: 4 in., w/2
Hooks	none	135° hooks	135° hooks

w: minimum member dimension (width); D: longer member dimension; ρ : longitudinal reinforcement ratio; d_b: longitudinal bar diameter; d_{b-tie}: transverse reinforcement diameter; d: distance from the extreme compression fiber to centroid of tension reinforcement (depth)

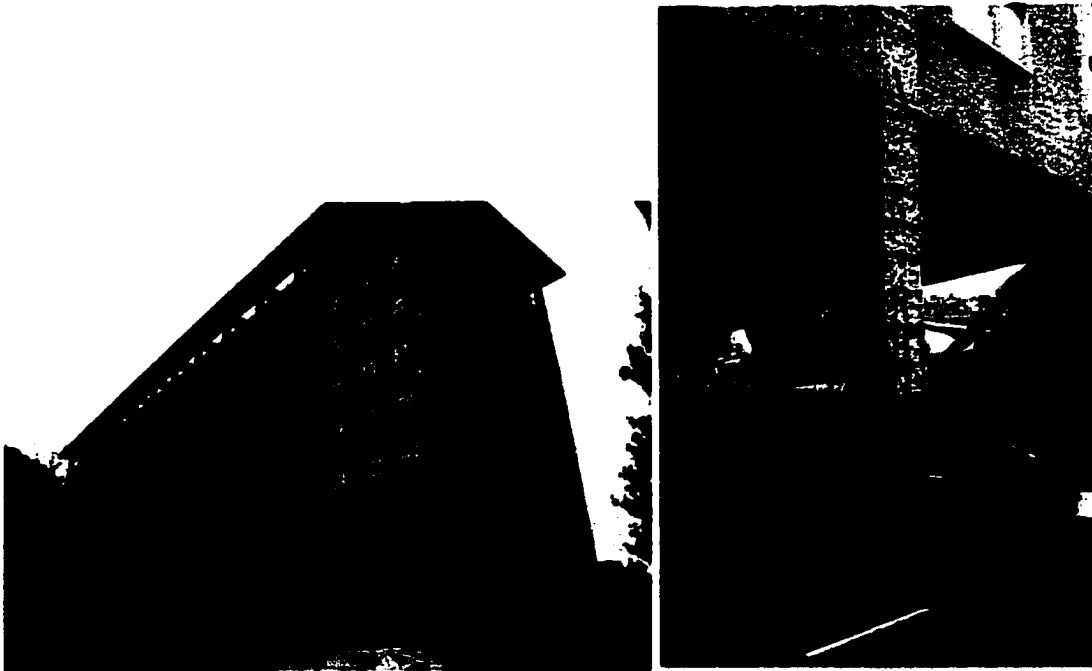


Figure 1.1 Older existing building under retrofit construction at the University of California, Berkeley campus and a close-up view of the first-story exterior column.



Figure 1.2 Idealized load paths in the Imperial County Services Building in the 1979 Imperial Valley Earthquake (Bertero, 2000)



Figure 1.3 Corner column and the first-story exterior column failures in the end of the Imperial County Services Building (Bertero V. V., NISEE-EQIIS Image Database)

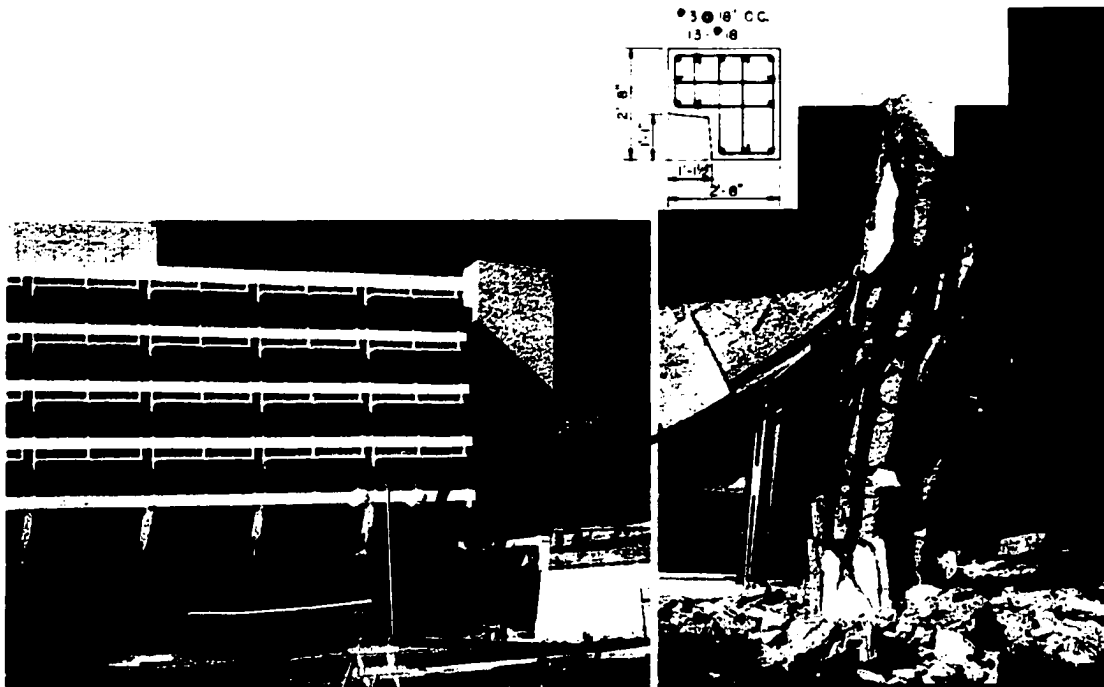


Figure 1.4 Damage to the Olive View Hospital in the 1971 San Fernando earthquake and damage to the corner column in the first story (Steinbrugge K. V., NISEE Database)

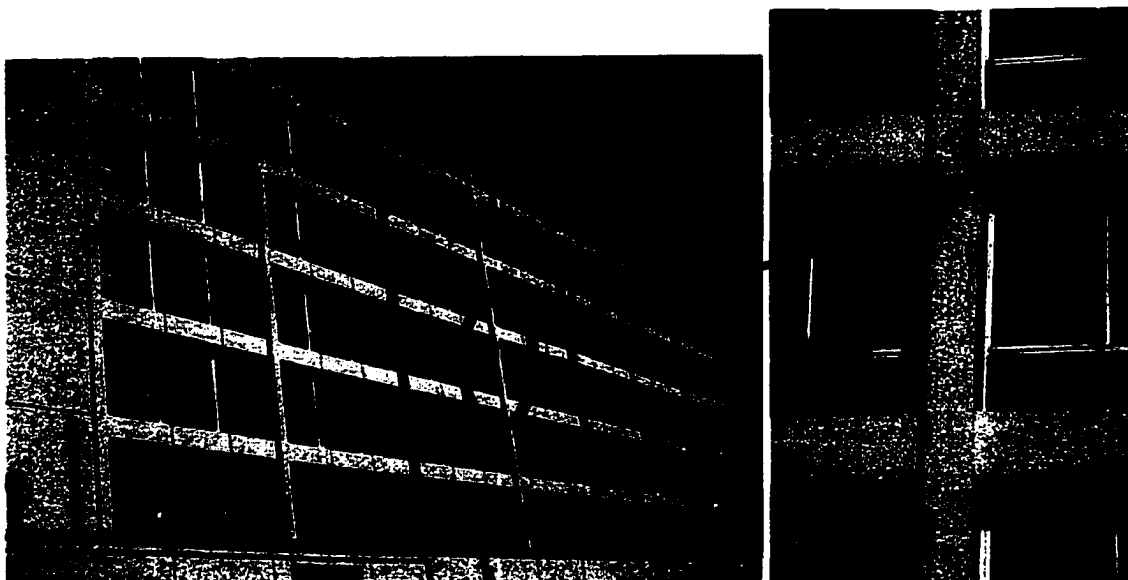


Figure 1.5 Column damage in the Van Nuys hotel in the 1994 Northridge earthquake, and a close-up of a failure in a fourth story-column (NISEE Database)



Figure 1.6 Column failures from the 1999, Kocaeli, Turkey earthquake



Figure 1.7 Loss of axial-load-carrying capacity due to column midheight failure (November 12, 1999, Duzce, Turkey earthquake)

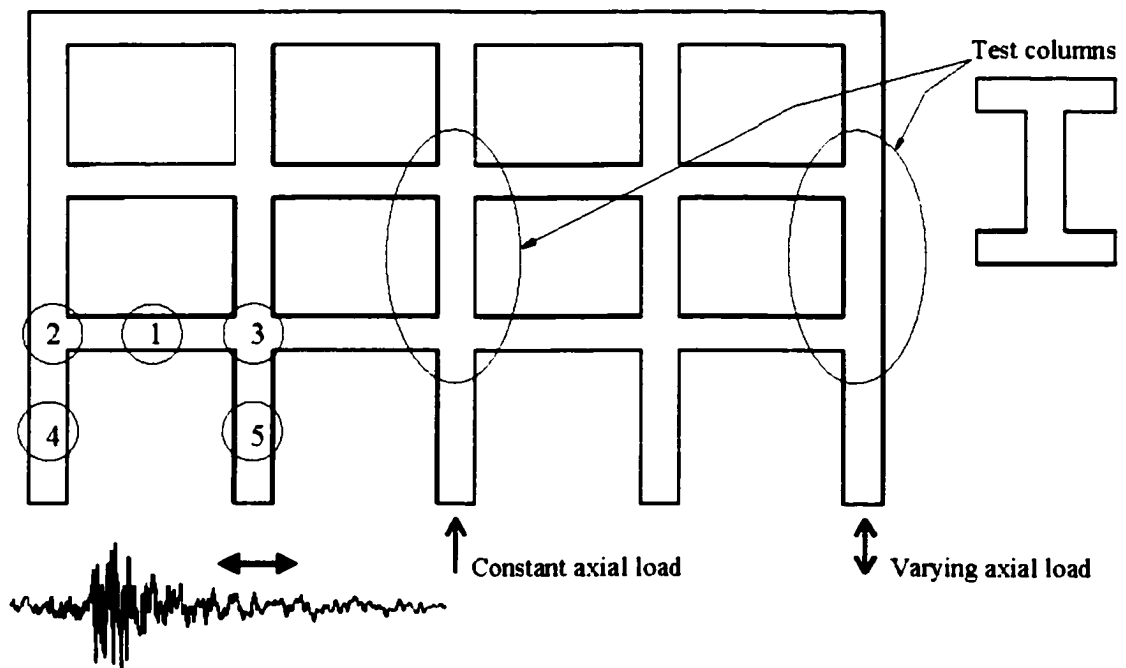


Figure 1.8 Idealized building frame subjected to lateral earthquake and gravity loads

2 PREVIOUS RESEARCH AND BACKGROUND

2.1 INTRODUCTION

This chapter provides an overview of existing experimental and analytical research on the shear behavior and deformation characteristics of reinforced concrete columns. Of particular interest were experimental studies that used test setups that had specimen configurations similar to the ones used in this investigation. The review was useful in defining test geometries and load histories that should be investigated in the present study. This chapter also presents a summary of available shear strength models and yield displacement prediction models that were used to predict the strength and displacement capacities of column specimens tested in this research.

2.2 EXPERIMENTAL STUDIES

The behavior of older shear-critical reinforced concrete building columns subjected to reversed cyclic deformations has been studied by several researchers. However, experimental research was mostly undertaken using smaller-scale or short aspect ratio columns. Tests of nearly full-scale columns with realistic loading and boundary conditions are very limited.

2.2.1 Test Geometries

As illustrated in Figure 2.1, mainly four types of column specimens were tested in the experimental investigations described in this chapter. In Figure 2.1, the clear length and the lateral displacement of an equivalent cantilever specimen are shown by L and Δ , respectively.

Many researchers believe that a double-curvature specimen with nearly rigid horizontal top and bottom beams (Type-A specimen in Figure 2.1) is the best model for a typical building column. However, very limited experimental work has been done using large or full-scale test columns in double curvature. The most common test configuration used to date is Type-B (cantilever) test setup.

In building columns susceptible to shear failure under lateral loads, major inclined shear cracks are expected to occur in the midheight column region. Figures 1.3 through 1.7 demonstrate examples of this kind of damage observed around midheight of lightly reinforced building columns. In cantilever column tests, it may not be possible to

observe similar damage including major diagonal shear cracks. This is because the tip of the cantilever specimen, which is assumed to be the inflection point in a building column, is restrained and sustains no damage.

For the deformation and shear strength prediction of building columns, one should be cautious when interpreting and comparing test results from specimens with single or double stubs. As illustrated in Figure 2.1, in most experiments, the stubs in such specimens are allowed to rotate freely, as opposed to fixed boundary conditions used at the end of double curvature and cantilever specimens. However, it is not unrealistic to have some rotation in the slabs above and below the columns in a typical frame building. Using a similar test setup (Type-C in Figure 2.1), Wight and Sozen (1975) minimized the rotation of the stub by clamping the middle joint by a pair of hydraulic actuators as illustrated in Figure A.16 in Appendix A.

The following sections present an overview of the previous experimental and analytical research studies related to certain aspects of the behavior of reinforced concrete columns.

2.2.2 Tests to Study Behavior in Shear

Lu and Chen (1992) tested ninety-four short columns under constant axial load and cyclic lateral loads. This extensive experimental investigation describes most of the potential damage modes with possible reasons for the type of damage observed. The longitudinal and transverse reinforcement ratio, shear span to depth ratio, and the axial load ratio were the primary parameters studied. The columns were tested in double

curvature. Fifty-two specimens had a shear span to depth ratio of 2.0, and the remainder had a ratio of 1.5. The longitudinal reinforcement ratio varied between 0.008 and 0.031, and the transverse reinforcement ratio ranged from 0 to 0.01. The axial load ratio varied between 0.2 and 1.0.

Five major failure modes were identified. In specimens with lower axial load and insufficient shear reinforcement, the column ties yielded or fractured after formation of diagonal cracks. This type of failure with limited ductility was called shear-tension failure. Shear-compression failure (Figure 2.2.a) occurred in specimens with moderate axial load and larger shear reinforcement ratio. High axial compression-shear failure was observed in specimens with axial load ratio larger than 0.6. Specimens with high axial load had lower displacement ductility and failed in the midheight region where major diagonal cracks formed and longitudinal bars buckled. Shear-bond failure (Figure 2.2.b) was observed mostly in specimens with larger longitudinal reinforcement ratio, large diameter bars, and low strength concrete. Shear-flexure failure (Figure 2.2.c) occurred in specimens with smaller longitudinal reinforcement ratio and lower axial load.

Lynn et al. (1996) tested eight full-scale columns with constant axial load and cyclic lateral load. The loading, boundary conditions, and geometrical properties of test specimens were very similar to those of the specimens in the testing program used in the present study. Figure 2.3 shows the details and overall dimensions of the column specimens. Grade 40 deformed longitudinal reinforcement and concrete with strength ranging between 3700 and 4800 psi were used. The applied constant axial load was

equal to either 112 kips ($= 0.12 f'_c A_g$) or 340 kips ($= 0.35 f'_c A_g$) in compression (f'_c = specified compressive concrete strength, and A_g = gross cross-sectional area).

Lynn presented the experimental data and compared the test results with estimated behavior from various evaluation methods. The recorded lateral load-displacement relations for the specimens are shown in Figure 2.4. Specimens 3CLH18 and 3SLH18 showed a typical column behavior with limited flexural ductility followed by the loss of lateral resistance due to shear failure. It should be noted that Specimen 3SLH18 with a constant axial load of $0.12 f'_c A_g$ sustained its axial-load-carrying capacity up to relatively large displacements indicating that columns with low axial load and inadequate shear resistance could sustain vertical loads after the loss of lateral resistance. Specimen 2CLH18, which was also subjected to low axial load, had moderate flexural ductility followed by the loss of lateral- and axial-load-carrying capacity. Specimens 3CMD12 and 3SMD12 with the same transverse reinforcement configuration as the specimens tested in this research program had very low flexural ductility and failed in shear.

Lynn evaluated column failure modes associated with bond, shear, flexure and axial load. The $20d_b$ lap splice length used for the longitudinal reinforcement of some of the columns was found to be inadequate to sustain bond strength under yielding cycles. The measured flexural and shear strengths were found to be consistent with the strengths calculated using the equations given in the ACI 318-95 building code.

2.2.3 Tests to Study Effect of Axial Load Variations

As illustrated in Figure 1.8, the exterior or corner columns of a typical building frame can be subjected to large axial load variations during an earthquake. This section provides a short summary of important results and findings from previous experimental and analytical investigations related to the behavior of columns subjected to varying axial load. Varying axial loads are categorized as proportional or non-proportional. For the proportional case, a proposed mathematical function is defined to relate the axial load to the lateral load or lateral displacement. Proportionally varying axial and lateral loads are applied simultaneously, and both axial and lateral loads reach their peak values at the same time. On the other hand, variations in non-proportionally varying axial load and lateral load are uncoupled. In other words, the axial and lateral loads are not related and applied independently.

Experimental work by Gilbertsen and Moehle (1980) was one of the few early studies that considered the variation in axial loads. Eight small-scale cantilever columns were subjected to constant axial load and varying axial load proportional to cyclic lateral load. The results showed that the lateral load-displacement loops were unsymmetrical about the axis of zero lateral load. The average of lateral strengths under tensile and high compressive axial loads for columns with varying axial load was comparable with the strengths of the columns with constant axial load. Abrams (1987) investigated the effect of axial load variation on the flexural behavior of reinforced concrete cantilever columns. Abrams concluded that the shape of the lateral load-displacement loops was

influenced by the range of axial load variation and the rate of change of axial load with lateral displacement.

In recent years, Li, Park and Tanaka (1991, 1995) have carried out an extensive experimental and analytical investigation at the University of Canterbury, New Zealand. Seventeen cantilever columns were tested under constant axial load, non-proportionally and proportionally varying axial load, and cyclic lateral load. It was concluded that proportionally varying axial load pattern resulted in significant shear strength degradation in non-ductile columns. The results showed that the variation in magnitude of the axial load had a significant effect on the stiffness, strength and deformation capacity of the columns. It was found that the column response was influenced mainly by the magnitude of the axial load rather than the frequency and phasing of the non-proportionally varying axial load.

Kreger and Linbeck (1986) reported results from the experimental investigation on three double-curvature specimens with various lateral and axial load variations. Two specimens were subjected to axial loads that were proportional to the lateral load. The other specimen was tested using uncoupled axial and lateral loads. The results indicated an increase in the lateral stiffness with increasing axial load. The energy dissipation characteristics of the columns were largely dependent on the axial load history.

Lejano et al. (1992) tested three identical double-curvature columns under cyclic lateral load, and constant and proportionally varying axial loads. The first two specimens were tested under constant axial loads of $0.74f'_c A_g$ in compression and $0.26f'_c A_g$ in tension. An

initial gravity load of approximately $0.25f_c A_g$ was assumed for the axial-lateral load relationship for the third test. The minimum and maximum axial loads were set equal to constant axial loads applied on the first two specimens (Figure 2.5). The measured lateral load-displacement relations and damage pattern for each specimen are shown in Figure 2.5.b. The tensile axial load appeared to produce larger displacements and horizontal cracks, whereas the compressive axial load caused a brittle failure. The displacement ductility increased substantially for the specimen subjected to varying axial load.

Saadeghvaziri (1997), and Saadeghvaziri and Foutch (1990) developed an analytical model to predict the flexural response of columns with proportionally and non-proportionally varying axial loads. They also presented a comprehensive review of different available analytical models. Based on analytical investigation of several types of non-proportional axial load histories, they concluded that energy dissipation capacity of the columns could be reduced significantly under uncoupled variations in axial and lateral loads.

2.3 MODELS FOR SHEAR STRENGTH

During the last few decades, several shear strength models have been proposed and used for the design and evaluation of reinforced concrete columns. Examination of these models shows differences in the approaches used to develop the equations, and in terms of parameters used in the models. According to most models, the shear strength can be computed as the sum of the strength contributions from concrete and transverse

reinforcement. However, effects of various parameters such as axial load, displacement ductility, and aspect ratio are represented either differently or not included.

As part of the experimental investigation carried out in this research, several available shear strength equations were used to predict the shear strength of the column specimens before the testing. In the design of test specimens, the shear strengths estimated from the following methods were used. All units are in lb, in., and psi.

2.3.1 ACI 318-02 (2002)

The shear strength equations provided in ACI 318-02 are design equations, however, they may be used to estimate the shear strength of existing reinforced concrete members. The shear strength, V_n , is calculated as the summation of contributions from the concrete, V_c , and the transverse reinforcement, V_s .

$$V_n = V_c + V_s \quad (2.1)$$

For members subjected to shear and axial compression, the concrete contribution to the shear strength is given by

$$V_c = 2 \left(1 + \frac{P}{2000 A_g} \right) \sqrt{f'_c} b d \quad (2.2)$$

where P is the axial load, which is positive for compression, A_g is the gross cross-sectional area, f'_c is the specified compressive concrete strength, and b and d are the web width and effective depth of the section, respectively.

The transverse reinforcement contribution is calculated as

$$V_s = \frac{A_{sw} f_{yw} d}{s} \quad (2.3)$$

where A_{sw} is the transverse reinforcement area within a spacing, s , in the loading direction, and f_{yw} is the yield strength of transverse reinforcement.

The Special Provisions for Seismic Design (Chapter 21) in ACI 318-02 stipulates that, at column ends or in the possible plastic-hinge regions, the concrete contribution, V_c , be taken equal to zero if the factored axial compressive force including earthquake effects is less than $f'_c A_g / 20$ and if the earthquake induced shear force is large.

2.3.2 ASCE-ACI Committee 426 Proposals (1973, 1977)

The ASCE-ACI Joint Committee 426 published a report on the shear strength of reinforced concrete members in 1973. The report was written partly in response to damage observed after the 1971 San Fernando earthquake. A revised version of the report was published in 1977. In the 1973 report, reasons for undesirable shear failure of reinforced concrete members were investigated. The report reviewed typical shear transfer mechanisms, design proposals, and existing research.

The report described the important shear transfer mechanisms as: (a) shear transfer by uncracked concrete; (b) interface shear transfer in the cracked concrete, i.e., aggregate interlock; (c) dowel shear carried by the longitudinal reinforcement; (d) arch action in deep members; and (e) shear transfer by the transverse reinforcement. For design

purposes, the most critical mechanisms were identified as the shear transfer by the transverse reinforcement and concrete.

For the calculation of shear strength, the ASCE-ACI Joint Committee 426 (1973) proposed an approach similar to the one provided in the ACI 318 code (i.e., Equation 2.1). The transverse reinforcement contribution, V_s , is the same as that given in ACI 318 (Equation 2.3). In members subjected to axial compression, the concrete contribution was calculated by

$$V_c = v_c \left(1 + \frac{3 P}{f'_c A_g} \right) b d \quad (2.4)$$

For members of normal weight concrete subjected to axial tension exceeding $0.5\sqrt{f'_c} A_g$, the shear force carried by the concrete was calculated by

$$V_c = v_c \left(1 + \frac{P}{6 \sqrt{f'_c} A_g} \right) b d \quad (2.5)$$

where v_c is the shear stress carried by concrete and given by

$$v_c = (0.8 + 100 \rho_l) \sqrt{f'_c} \leq 2.0 \sqrt{f'_c} \quad (2.6)$$

where ρ_l is the longitudinal reinforcement ratio ($\rho_l = A_s/(bd)$).

Another predictive equation was provided for the calculation of shear strength required to initiate flexure-shear cracks, V_{cr} .

$$V_c = v_c b d + \frac{M_u}{a} \quad (2.7)$$

For columns loaded in double curvature, the shear span, a , is set equal to $L/2$. M_u is

$$M_u = \frac{P}{A_g} \frac{I}{y_t} \quad (2.8)$$

Substituting for the second moment of inertia, I , A_g , and approximating y_t as one half of the section height ($= h/2$). Equation 2.7 becomes

$$V_c = v_c b d + 0.167 \frac{h P}{a} \quad (2.9)$$

The committee discussed in detail the factors affecting the shear strength including the effect of cross-sectional shape and size, the reinforcement strength and details, span-to-depth ratio, and type of loading. The committee recommended that for members with short span-to-depth (aspect) ratio, the concrete shear stress, v_c , should be reduced by the span-to-depth ratio. Note that the axial load contribution to shear carried by concrete (Equation 2.9) is already reduced by span-to-section height ratio, h/a .

2.3.3 SEAOC (1973)

The 1973 SEAOC *Recommended Lateral Force Requirements* included the ACI 318-71 column shear strength equations except that the concrete contribution was set equal to zero for axial stresses (P/A_g) smaller than $0.12f'_c$. The transverse reinforcement contribution was calculated from Equation 2.3. The concrete contribution, V_c , was

calculated from Equation 2.2 except that concrete area resisting shear was used instead of bd and furthermore the concrete contribution was limited to

$$V_c = 3.5 \sqrt{f'_c} \sqrt{1 + 0.002 \left(\frac{P}{A_g} \right)} A_c \quad (2.10)$$

where A_c is defined as the area of concrete resisting shear.

2.3.4 Aschheim and Moehle (1992)

The study by Aschheim and Moehle (1992) used laboratory data from cantilever bridge column tests. The data indicated that the column shear strength is a function of displacement ductility demand, μ_δ , the quantity of transverse reinforcement, and axial load. The shear strength is calculated as the summation of strength contributions from transverse reinforcement and concrete (Equation 2.1). The transverse reinforcement contribution is computed from Equation 2.3. The concrete contribution, V_c , is defined as

$$V_c = \alpha' \left(1 + \frac{P}{2000 A_g} \right) \sqrt{f'_c} b d \leq 3.5 \sqrt{f'_c} b d \quad (2.11)$$

For design and evaluation of rectangular hoop reinforced concrete columns

$$\alpha' = \frac{0.0060 \rho_w f_{yw}}{\mu_\delta} \quad (2.12)$$

where ρ_w is the transverse reinforcement ratio, $\rho_w = A_{sw}/(bs)$.

2.3.5 Caltrans (1995)

For the evaluation of shear strength of existing reinforced concrete columns, the California Department of Transportation (Caltrans) proposed Equation 2.1 to calculate the shear strength, V_n , in which the transverse reinforcement contribution, V_s , is obtained from Equation 2.3. The concrete contribution, V_c , is considered to be a function of the applied axial load, displacement ductility, and confinement.

$$V_c = (F_1)(F_2) \sqrt{f'_c} (0.8 A_g) \leq 4 \sqrt{f'_c} (0.8 A_g) \quad (2.13)$$

$$F_1 = \frac{\rho'' f_y}{150} + 3.67 - \mu_\delta \leq 3.0 \quad (\text{psi}) \quad (2.14)$$

where ρ'' is the ratio of volume of transverse reinforcement to volume of core concrete. Note that a volumetric transverse reinforcement ratio is used in Equation 2.14 instead of ratio of transverse reinforcement area to the concrete area ($\rho_w = A_{sw} / bs$). F_2 is an axial load factor varying between 1.0 for zero axial stress and 1.5 for a compressive stress of 1000 psi. Equation 2.13 was developed for the evaluation of prismatic reinforced concrete members.

2.3.6 Architectural Institute of Japan, Structural Design Guidelines (1994)

For the shear strength prediction of reinforced concrete members with rectangular cross sections, two design methods (Method-A and -B) were introduced in the *Architectural Institute of Japan (AIJ), Structural Design Guidelines*. Method-A, which was based on a more conservative model by Watanabe and Ichinose (1992), is presented here. As

illustrated in Figure 2.6, the column shear strength, V_n , is calculated from the superposition of truss and arch actions ($V_t + V_a$). The contribution of the truss mechanism to shear strength is calculated by

$$V_t = b j_t \rho_w f_{yw} \cot \phi \quad (2.15)$$

where j_t is the distance between top and bottom longitudinal bars, ρ_w is the shear reinforcement ratio, $\cot \phi$ is the minimum of 2.0, $j_t/D \tan \theta$, and $\sqrt{[v f_c' / (\rho_w f_y)] - 1}$, where D is the overall depth of the section, $\tan \theta$ is the tangent of the strut angle in the arch mechanism, given by $(\sqrt{L^2 + D^2} - L)/D$, and v is the effectiveness factor for the compressive strength of the concrete calculated from the relationship given in Figure 2.7. The parameter, v_n , is equal to $0.7 f_c' / 29,000$.

The contribution of arch mechanism is given by

$$V_a = \tan \theta (1 - \beta) b D v f_c' / 2 \quad (2.16)$$

where β is calculated as

$$\beta = \frac{\rho_w f_y (1 + \cot^2 \phi)}{v f_c'} \quad (2.17)$$

where $\cot \phi$ is the minimum of 2.0, $j_t/D \tan \theta$, $\sqrt{[v f_c' / (\rho_w f_y)] - 1}$, and the value determined from Figure 2.7.

Note that the effect of axial load on the shear strength is not considered in the AIJ guidelines. In the derivation of the shear strength equations, which is based on the assumption of ductile behavior, shear resistance from truss and arch mechanisms are superposed. For lightly confined members susceptible to shear failure, superposition of the two actions may not be appropriate.

Shear strength degradation is related to plastic hinge rotation through a reduction in the concrete contribution. The concrete contribution is reduced as much as seventy-five percent at large displacements (through ν in Figure 2.7). Similarly, the steel contribution is reduced as much as fifty percent at larger displacements (through $\cot\phi$ in Figure 2.7) by reducing the truss angle to 45 degrees.

2.3.7 Priestley et al. (1994)

Priestley et al. (1994) proposed to calculate the shear strength of columns under cyclic lateral loads as the summation of contributions from the concrete, V_c , a truss mechanism (or transverse reinforcement), V_s , and an arch mechanism associated with axial load, V_p , as follows:

$$V_n = V_c + V_s + V_p \quad (2.18)$$

The concrete component is given by

$$V_c = k \sqrt{f'_c} A_c \quad (2.19)$$

where $A_e = 0.80A_g$ and the parameter, k , depends on the member displacement ductility level as defined in Figure 2.8. As demonstrated in Figure 2.8, the concrete contribution is reduced as much as 66 percent with increasing displacement ductility.

The contribution of transverse reinforcement to shear strength is based on a truss mechanism using a 30-degree angle between the diagonal compression struts and the column longitudinal axis. For rectangular cross-section columns, the truss-mechanism component, V_s , is given by

$$V_s = \frac{A_{sv} f_{sv} D'}{s} \cot 30^\circ \quad (2.20)$$

where D' is the distance measured parallel to the applied shear between centers of the peripheral hoop.

The arch mechanism contribution is given by

$$V_p = P \tan \alpha = \frac{D-c}{2a} P \quad (2.21)$$

where α is the inclination of diagonal compression strut (Figure 2.9), c is the neutral axis depth, and D is the overall depth of the section.

The shear resistance mechanism shown in Figure 2.9 is similar to the arch-mechanism model defined by Watanabe and Ichinose (1992) and shown in Figure 2.6. However, the model included in the Japanese code is independent of the applied axial load. It should be noted that the neutral axis depth, c , varies with the curvature at the critical section and

hence with the displacement ductility. As the aspect ratio increases, the axial load contribution decreases. Also, as the axial load increases, the neutral axis depth increases such that the term V_p increases at a decreasing rate with increasing axial load. The effect of axial tensile load on the shear strength is not defined.

2.3.8 Kowalski et al. (1997)

The model proposed by Priestley et al. (1994) was revised by Kowalski et al. (1997) to include the effects of column aspect ratio and longitudinal reinforcement. The concrete contribution is given by

$$V_c = \alpha \beta K \sqrt{f_c'} 0.80 A_g \quad (2.22)$$

where α includes the effect of aspect ratio ($\alpha = (2 - u/h) + 1$) and cannot be smaller than 1.0 and larger than 1.5, and β accounts for the effect of longitudinal reinforcement ($\beta = (0.5 + 20\rho_l) \leq 1$). Note the similarity between the factor, β , and the shear stress recommended by ASCE-ACI Committee 426 (Equation 2.6). As shown in Figure 2.10, the strength degradation factor, K , is reduced at larger displacement ductilities. Figure 2.10 indicates that the reduction in the concrete contribution could be as much as 83 percent at large displacement ductilities.

While the axial load component, V_p , remains the same as in Equation 2.21, the truss-mechanism component, V_s , is modified slightly and is given by

$$V_s = \frac{A_{sw} f_{sw} (D' - c)}{s} \cot 30^\circ \quad (2.23)$$

2.3.9 Model Proposed by Konwinski (1996) and Konwinski et al. (1996)

The study by Konwinski (1996) and Konwinski et al. (1996), which was based on experimental data from ductile column tests, concluded that the column shear strength is independent of displacement ductility demand. The proposed method includes a concrete component and a transverse reinforcement component to calculate the shear strength of columns. The transverse reinforcement contribution, V_s , is reduced by fifteen percent compared with the similar expression in the ACI 318-02, and is calculated as

$$V_s = 0.85 \frac{A_{vt} f_{vt} h}{s} \quad (2.24)$$

The concrete component, V_c , is given by

$$V_c = \alpha \sqrt{f_c} \sqrt{1 + \frac{(P/A_g)}{12 \sqrt{f_c}}} 0.85 A_g \quad (2.25)$$

where $\alpha = 6 d/a$, but is limited by $2 \leq \alpha \leq 4$. In this method, for the calculation of shear strength, the deformation capacity of the member is not need to be known. However, Equation 2.25 is an indirect representation at the displacement ductility demand being beyond some moderate value; hence, a displacement ductility effect is included implicitly. In order to include the effect of strength degradation due to cyclic loading, the maximum nominal core concrete shear stress ($= V_u / A_{core}$) is set equal to $f_c / 8$. The method assumes that the cover concrete has spalled.

2.3.10 FEMA-273 (1997)

The column shear strength equations recommended in FEMA-273, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (1997), are based on a review of available experimental data for existing columns subjected to axial load and reversed cyclic lateral displacements. The primary source of data was columns with transverse reinforcement higher than is typical in older existing buildings. In the FEMA-273 document, ductility demands on the older columns is taken into account in a simple manner.

The transverse steel contribution to the shear strength is computed from Equation 2.3, given in the ACI 318-95. The concrete contribution is calculated as

$$V_c = 3.5\lambda \left(k + \frac{P}{2000 A_g} \right) \sqrt{f'_c} b d \quad (2.26)$$

where the ductility factor, k is taken as 1.0 for low ductility demand and 0.0 for moderate and high ductility demand, and λ is equal to 1.0 for normal weight concrete. A demand-to-capacity ratio of 2.0 (which is calculated from a linear analysis) or a displacement ductility of less than 2.0 is described as low ductility demand. The axial load, P , is equal to zero in tension.

2.4 YIELD DISPLACEMENT AND DEFORMATIONS

As described in the previous section, several recent shear strength prediction methods (e.g. Aschheim and Moehle 1992, Priestley et al. 1994, and Kowalski et al. 1997) relate the column shear strength to column displacement ductility, which is defined as the ratio

of ultimate displacement to yield displacement. To be able to use these proposed models, the yield displacement of the column must be calculated.

The specified displacement history used in the experimental investigation of this study was a function of yield displacement. Therefore, it was necessary to predict the yield displacement of the specimens before the tests. There appears to be a considerable discrepancy in the approaches in calculation of different displacement components using alternative procedures described below. However, the difference between the calculated total yield displacements from these methods was comparable.

In this study, yield displacement is calculated as the summation of three components: flexural, shear, and longitudinal bar slip displacements. Since the test specimens were tested in double curvature, rotation of the top and bottom beams was negligible, thus the displacements due to base rotation are not considered.

2.4.1 Conventional Methods of Mechanics of Materials

2.4.1.1 Flexural Displacement

The flexural displacement component at the tip of a cantilever column may be calculated by integrating the curvature over the length of the column. For practical purposes, a linear strain distribution may be assumed in the elastic range. Integration of a linear curvature distribution with zero curvature at the tip and yield curvature, ϕ_y , at the base gives:

$$\Delta_{flexure} = \frac{\phi_y L^2}{3} \quad (2.25)$$

where $\Delta_{flexure}$ is the flexural component of the yield displacement, and L is the length of the cantilever.

Theoretical calculations of flexural displacement with a more refined curvature distribution taking into account of cracked concrete give very similar results to the displacement calculated using Equation 2.25. It should be noted that the crack distribution in columns tested in double curvature would not be the same as the distribution assumed for cantilever columns. After the formation of cracks around midheight of the column, the flexural displacement of double-curvature columns would be larger than those estimated from two equivalent cantilever columns.

2.4.1.2 Shear Displacement

Assuming an uncracked elastic homogeneous material with a constant modulus of elasticity, and a constant shear strain along the length of the column, the shear displacement of a cantilever column with a rectangular cross section may be calculated as

$$\Delta_{shear} = \frac{6}{5} \frac{V_y L}{G A_g} \quad (2.26)$$

where V_y is the lateral load at yield, and G is the shear modulus that may be taken as 0.4 times the modulus of elasticity for concrete.

2.4.1.3 Bar Slip Displacement

Slip due to extension of longitudinal reinforcing bars near the column ends and possible slip of the longitudinal bars from the anchorage concrete may be estimated by assuming a uniform bond stress, u_b , along the bars within the development length inside the footing or beam-column joint. Then, the equilibrium of bar forces at yield gives

$$u_b (\pi d_b) l_d = \frac{\pi d_b^2}{4} f_y \quad (2.27)$$

where d_b is the bar diameter, and l_d is the development length over which the slip occurs.

Solving Equation 2.27 for l_d gives

$$l_d = \frac{d_b f_y}{4 u_b} \quad (2.28)$$

Assuming maximum strain at column ends and a linear variation of strain along the development length, sum of the area under the strain curve gives the total bar slip at the footing-column interface or beam-column interface:

$$slip = \frac{\epsilon_y l_d}{2} \quad (2.29)$$

where ϵ_y is the yield strain of the bar ($= f_y / E_s$). Substituting Equation 2.28 for l_d in Equation 2.29 gives

$$slip = \frac{\epsilon_y d_b f_y}{8 u_b} \quad (2.30)$$

Assuming that the cross section rotates about its neutral axis when slip takes place ($\phi_s = \epsilon_s / (d - c)$), the displacement related to bar slip at a point L distance from the column base is calculated as

$$\Delta_{slip} = \frac{\phi_s d_s f_s L}{8 u_h} \quad (2.31)$$

2.4.2 Procedure Proposed by Priestley et al. (1996)

Based on the column test data from more than fifty experiments, Priestley et al. (1996) developed a model to estimate the yield displacement of circular bridge columns including the effect of flexural and shear deformation components. Considering the analogy between rectangular and circular columns and the corresponding definitions provided in Priestley et al. (1994), the following equations were used for the calculation of yield displacement of rectangular columns.

2.4.2.1 Flexural Displacement

In the model proposed for cantilever columns, the flexural component of the yield displacement is calculated as:

$$\Delta_{flexure} = \frac{\phi_s L_{eff}^2}{3} \quad (2.32)$$

where the effective length, L_{eff} , is calculated from Equation 2.33.

$$L_{eff} = L + 0.00015 f_s d_h \quad (2.33)$$

The coefficient 0.00015 has units of 1/psi. The effective length includes the effect of longitudinal bar strains, that is, the bar slip displacement contribution to the yield displacement. Consequently, the bar slip displacement is included in the flexural displacement component implicitly by the second term in Equation 2.33.

2.4.2.2 Shear Displacement

Shear displacement is calculated as the sum of two components: Δ_{sc} due to shear carried by concrete, and Δ_{ss} due to shear carried by the transverse reinforcement:

$$\Delta_s = \Delta_{sc} + \Delta_{ss} \quad (2.34)$$

The concrete component of the shear displacement is

$$\Delta_{sc} = \frac{2 (V_c + V_p) L}{(0.4E) (0.8A_g)} \quad (2.35)$$

where the concrete and axial load components of the shear, V_c and V_p , were defined in Equations 2.19 and 2.20, respectively. The constant k given in Equation 2.18 was set equal to 3.5 for columns with low ductility (Figure 2.8). If the calculated sum, $V_c + V_p$ is larger than V_y , then $(V_c + V_p)$ must be replaced by V_y in Equation 2.35.

The shear displacement is related to transverse reinforcement as follows

$$\Delta_{ss} = \epsilon_t L \quad (2.36)$$

where ε_t is the average elastic strain in the transverse reinforcement. If the transverse reinforcing bars yield before the longitudinal reinforcement, Equation 2.36 will not give a good estimate of the displacement associated with the transverse reinforcement and should not be used. If the compression diagonal members in the truss model form a 45-degree angle at early stages of the test, ε_t may be obtained from Equation 2.20. Thus

$$\varepsilon_t = \frac{V_s s}{E_s A_s D} \quad (2.37)$$

where V_s is set equal to $V_y - (V_c + V_p) \geq 0$. Other variables were defined earlier in this chapter.

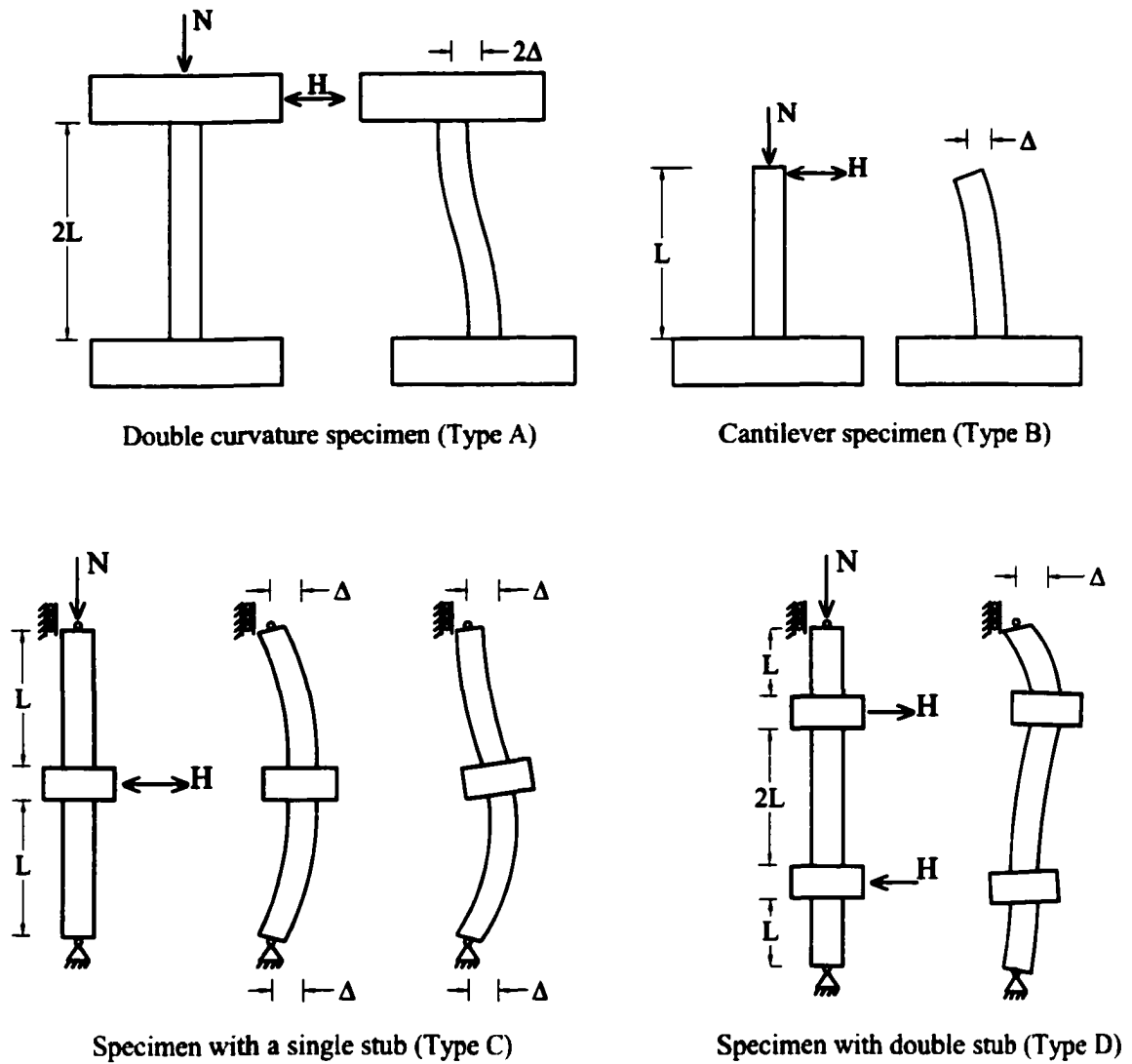


Figure 2.1 Schematic of test specimens with different boundary conditions

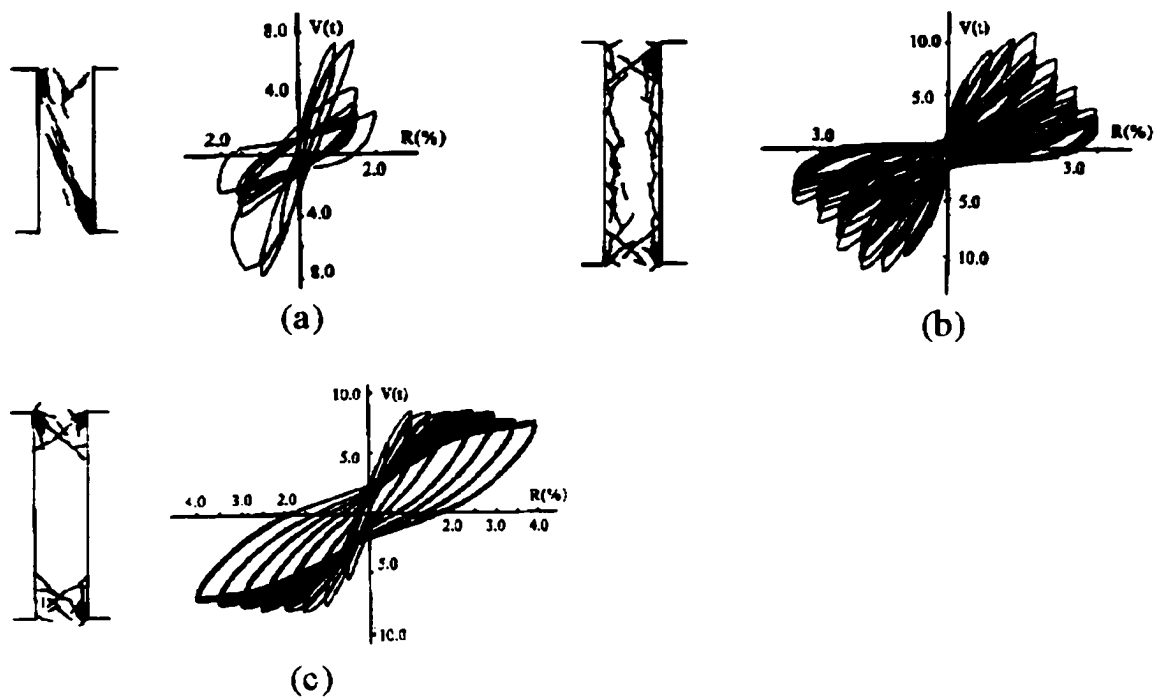


Figure 2.2 Typical failure modes and lateral load-displacement relations (Lu and Chen, 1992)

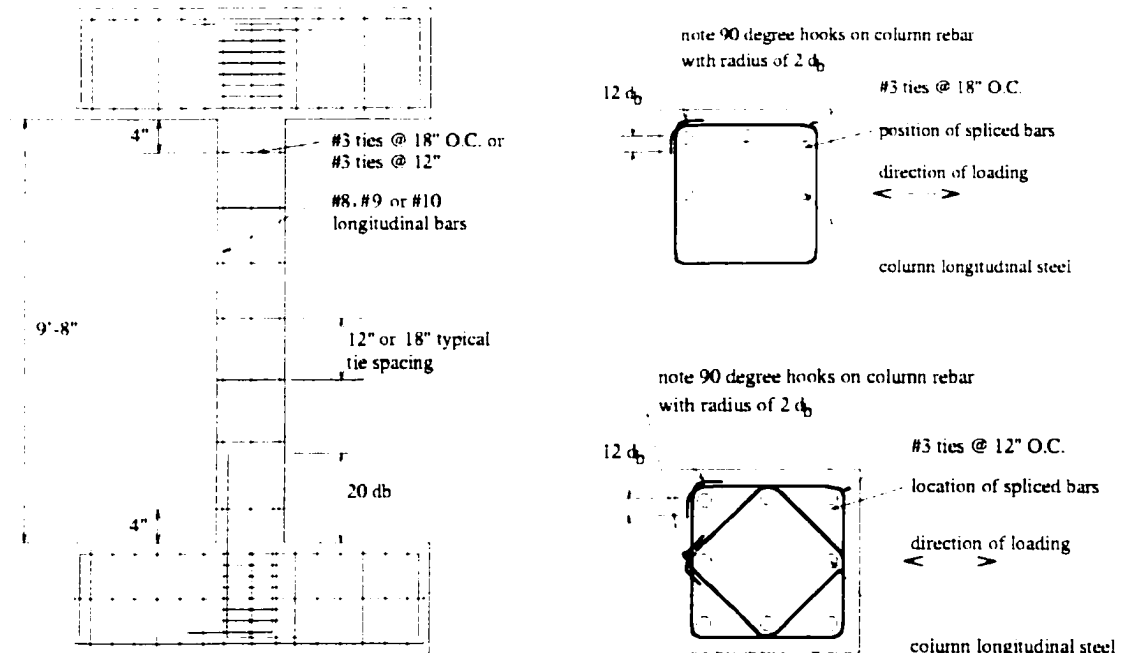


Figure 2.3 Typical test specimen and section details (Lynn et al. 1996)

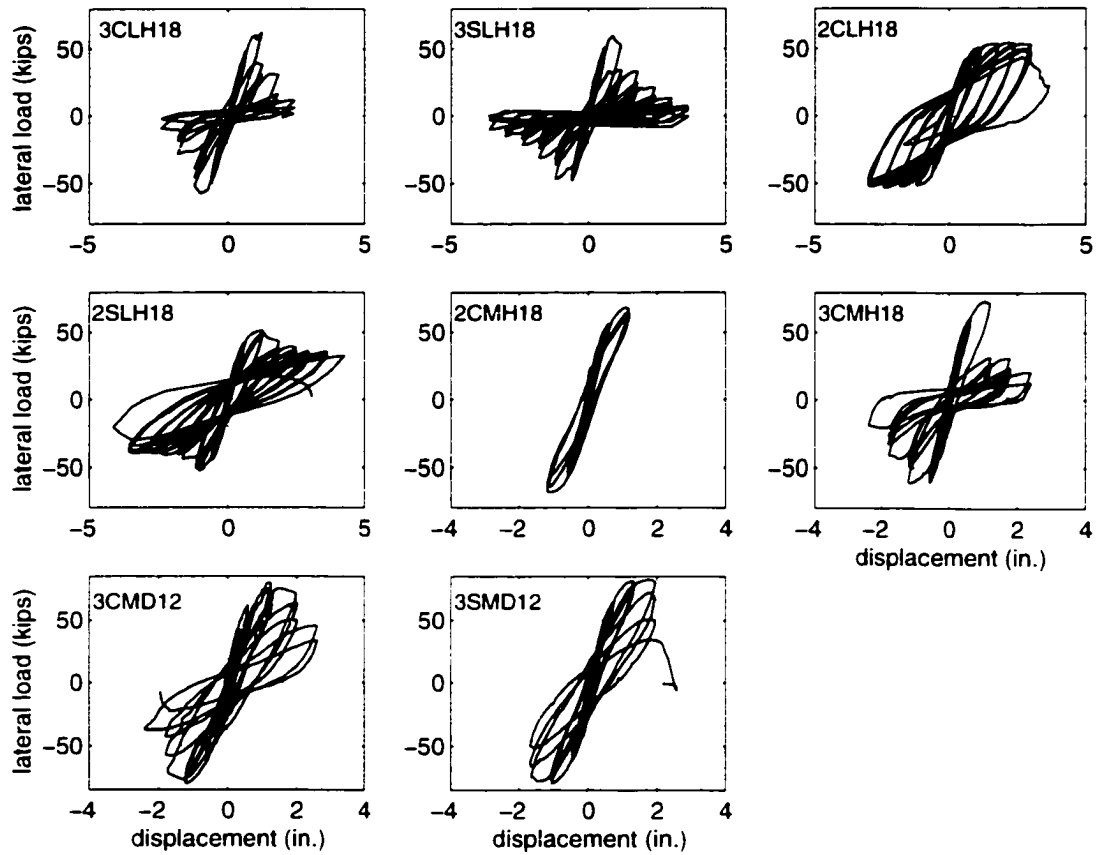


Figure 2.4 Measured lateral load-displacement relations (Lynn et al. 1996)

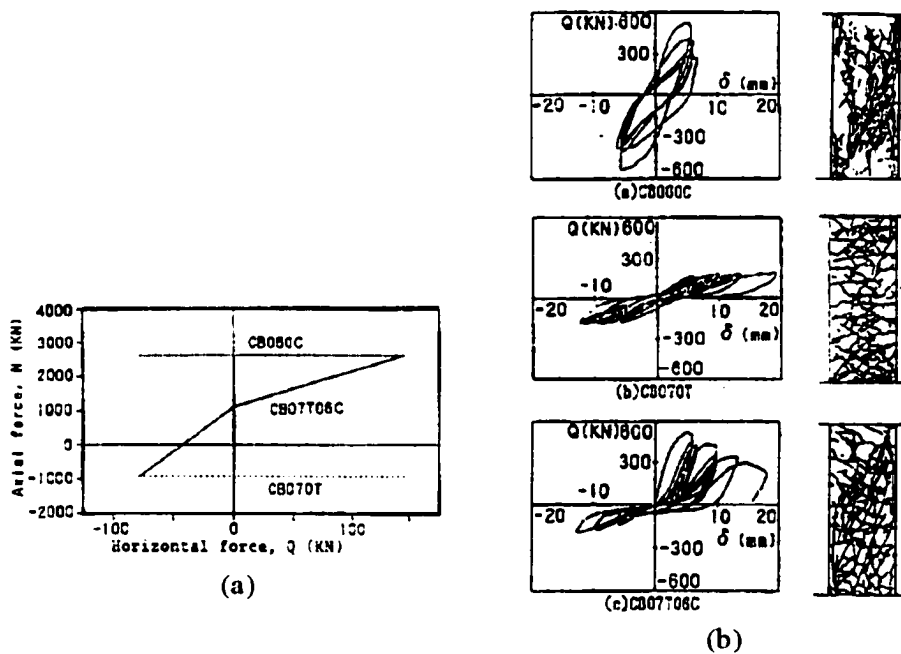


Figure 2.5 Axial and lateral load relations, damage pattern and lateral load-displacement plots (Lejano et al. 1992)

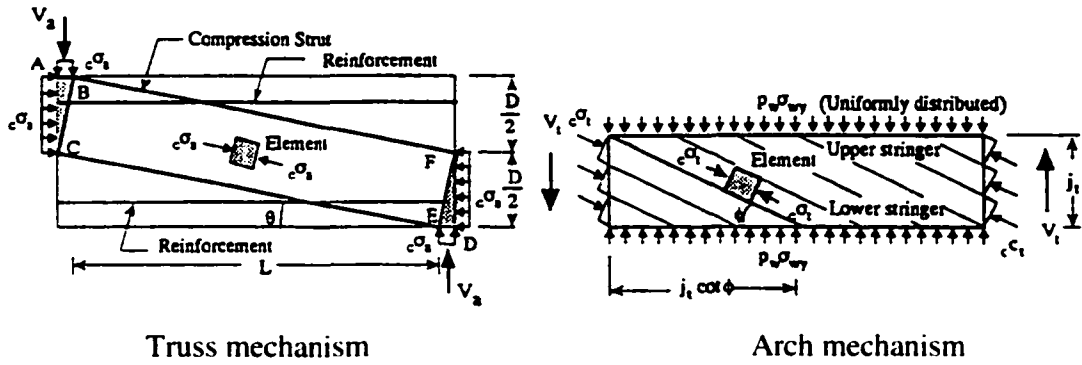


Figure 2.6 Shear force carried by truss and arch mechanisms (AIJ 1994)

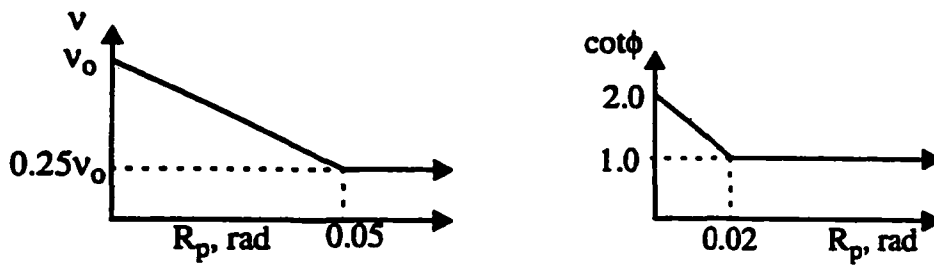


Figure 2.7 Relation between plastic hinge rotation, R_p , and effectiveness factor, v , and $\cot \phi$ (AIJ 1994)

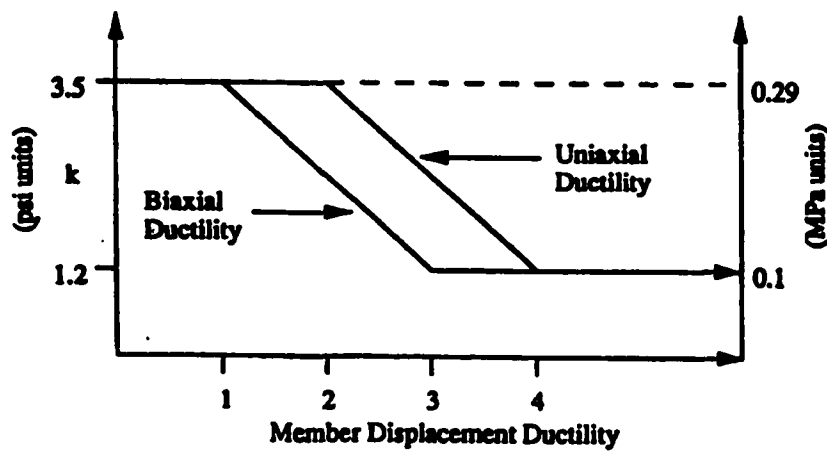


Figure 2.8 Concrete shear strength degradation with displacement ductility (Priestley et al. 1994)

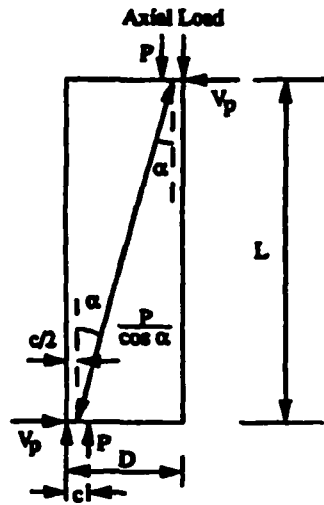


Figure 2.9 Axial load contribution to shear strength (Priestley et al. 1994)

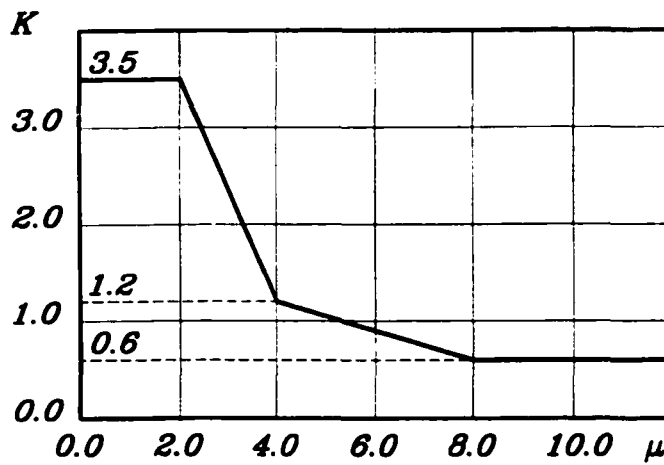


Figure 2.10 Concrete shear strength degradation with displacement ductility (Kowalski et al. 1997)

3 TEST PROGRAM

3.1 INTRODUCTION

Experimental simulation can be indispensable for gaining insight into fundamental behavior of structural components and systems. However, simulation of the effects of real events on actual structures usually requires idealization and simplification for most problems of interest. This chapter provides details of the experimental investigation of the behavior of full-scale building columns isolated from a complete building frame and subjected to gravity and earthquake loads. The column specimens and their boundary conditions are illustrated in Figure 1.8. In order to simulate the loads and keep the boundary conditions of the specimens as close to the real building environment as possible, a computer-controlled static testing system with multiple displacement and force control loops was used. A control scheme was developed to apply the varying axial load as a function of the lateral load while maintaining the specified lateral displacement and preventing rotation at the top of the specimen.

The experimental program included construction and testing of four columns with deficient transverse reinforcement under various axial and lateral load combinations. The test columns, which were connected to nearly rigid top and base beams, were tested in double curvature. The amount of reinforcement, details, and material properties were nominally identical in all specimens. The overall dimensions of the specimens and the test setup were chosen to be similar to the eight column specimens tested by Lynn et al. (1996). Thus, the results from a series of twelve column tests could be compared.

This chapter describes the test specimens, material properties, construction process, test setup, and instrumentation. It also describes the loading protocol and loading system.

3.2 TEST SPECIMEN DESIGN

In order to achieve the goals stated in Chapter 1, the following loading and design parameters were considered in the design of test columns.

3.2.1 Shear Strength and Flexure/Shear Demand

In 1983, the Applied Technology Council (ATC) recommended the model shown in Figure 3.1 for shear response of columns. In the model, the shear strength ($V_t(c)$ in Figure 3.1) was considered to degrade with inelastic displacements beyond displacement ductility of two. The shear demand (i.e., $V_u(d)$ in Figure 3.1, and V_p in Section 1.4) was calculated considering flexural response of the column, and was assumed constant after flexural yielding, i.e., beyond a displacement of ductility of one.

In the present investigation, the objective was to design the test columns in such a way that the initial shear strength ($V_i(c)$ in Figure 3.1) and the shear demand corresponding to the flexural yield ($V_u(d)$ in Figure 3.1) would be very close. Specifically, the columns were designed so that they would yield in flexure at a shear approaching the shear strength. The intent was to observe effects of flexural yield on shear failure. Even though the objective was to have a shear demand-capacity relation similar to "CASE B" in the ATC model, it would also be possible have a relation as in "CASE A" as the demands and capacities were very close. The shear strength ($V_i(c)$ or V_n) was calculated from the models presented in Section 2.3. The shear demand ($V_u(d)$ or V_p) was calculated from moment-curvature analysis using a fiber cross-section computer model and uniaxial material models (UCFyber, 1999).

3.2.2 Axial Loads and Flexure/Shear Demand

Three of the four test specimens tested in this research represent a typical interior column in a gravity-load-carrying frame in a building. Constant axial loads were applied to model the gravity loads on these columns. The applied axial loads and the calculated axial load-moment interaction diagram for the specimens are shown in Figure 3.2. The interaction diagram was calculated using a fiber cross-section of the column and uniaxial material properties (UCFyber, 1999).

As illustrated in Figure 3.2, the 150-kip axial load applied on the first and last columns, which is below the balance point, and the 600-kip axial load on the second column, which is above the balance point, correspond to approximately the same theoretical

flexural strength (i.e., moment demand = 3900 kip-in.) on the axial load-moment interaction diagram. Therefore, the columns tested under constant axial load have the same theoretical shear demand ($V_u(d)$ in Figure 3.1). Because of identical theoretical flexure and shear demands under constant axial loads, results from Test 1 and Test 2 should indicate primarily the effect of axial load on the behavior of columns. The third column was representative of an exterior column of a building frame. As demonstrated on the interaction diagram, this specimen was subjected to axial load varying between 600 kips in compression and 60 kips in tension.

3.3 SPECIMEN DESCRIPTION

The test columns had a cross section of 18 in. by 18 in. Clear height of the columns was 116 in. Stiff end beams at the top and bottom of the column had length, depth, and width of 96 in., 30 in., and 26 in., respectively. The base beam simulated a rigid floor system or a rigid foundation while the top beam simulated the rigid floor system. As illustrated in Figure 2.1 (double-curvature specimens), rotation of the top and base beams within the vertical plane were prevented during the tests. These beams were designed conservatively to avoid significant deformation in the beams during the tests. The dimensions and arrangement of longitudinal and transverse reinforcement in the column and beams are shown in Figure 3.3 through Figure 3.6.

Eight #9 deformed longitudinal bars were placed uniformly around the perimeter of the cross-section (Figure 3.5a) resulting a longitudinal reinforcement ratio, $\rho_{sl} = A_{sl}/A_g = 0.025$, where A_{sl} = total cross-sectional area of longitudinal reinforcement, A_g = gross

area of cross-section. Longitudinal bars were continuous without lap splices. As shown in Figure 3.5.a, Number 3 deformed bars were used for the transverse reinforcement. These were in the form of square hoops with 90-degree hooks with a $6d_b$ extension, where d_b is the bar diameter. The transverse reinforcement was spaced uniformly at 12 in. The nominal clear concrete cover over the longitudinal reinforcement was 2 in. for the column and beams. The column longitudinal bars were bent 90 degrees at the top and bottom inside the beams. The lead-in length of the longitudinal reinforcing bars within the beams was slightly shorter than the development length calculated using the ACI 318-02.

3.4 MATERIAL PROPERTIES

The specified concrete strength was 3000 psi. The concrete had normal weight aggregate with a maximum nominal aggregate size of 1.0 in. The concrete mix specifications and mix design properties are shown in Tables B.1 and B.2 in Appendix B, respectively. A local ready mix supplier, Right Away in Oakland, California, delivered concrete. The measured slump values for each truck varied between 5 and 6 in. At 28 days, standard cylinder tests showed that the compressive strength of the column concrete was below the target strength. Between 28 and 88 days, columns and cylinders that were cast using the column concrete were cured using a water-curing blanket, which was made of burlap and polyethylene. By the time of the column tests, the cylinders representing the column concrete strength had strength very close to the specified target strength of 3000 psi. Typical column concrete stress-strain relationships are shown in Figure B.2. Table 3.1 lists the average concrete strengths from cylinder tests done on the day of each test. The

failure modes for the cylinders (Figure B.1) and the concrete strengths obtained from cylinder tests are tabulated in Tables B.3 through B.5.

Grade 60 steel deformed bars from the same batch were used for all four specimens. Detailed descriptions of coupon tests and testing procedures are provided in Appendix B. The measured tensile stress-strain relationships for the reinforcing bars are plotted in Figures B.4 and B.5. The average yield strength of the Number 9 longitudinal bars was 63 ksi. The measured yield strength, ultimate strength, and yield strain of reinforcing bars are listed in Table B.7.

3.5 CONSTRUCTION OF SPECIMENS

Reinforcing bars were cut and bent by a local supplier. A total of 112 strain gages were attached on the longitudinal reinforcement in the laboratory in Davis Hall, University of California, Berkeley. A section of the bar approximately one-inch long was filed and cleaned before attaching a strain gage. After the strain gages attached and wired, they were sealed by three different coating agents. Then, the gages were wrapped by vinyl mastic to protect them from damage during concrete casting. Electrical resistance strain gages were produced by Tokyo Sokki Kenkyujo Co. Type YFLA-5 and YFLA-2 strain gages with 5 mm and 2 mm gage length were used on the longitudinal and transverse bars, respectively. The nominal limiting strain for the post-yield gages was 0.1 to 0.2 at room temperature.

The reinforcement cages were tied and the test specimens were constructed outside at the Richmond Field Station Structures Laboratory of the University of California, Berkeley.

The specimens were cast in the vertical position. First, the wood formwork for the bottom and top beams of the specimens was constructed and shoring was provided. Then, the reinforcement cages were tied inside the formwork. Typical reinforcement cages inside the bottom and top beams are shown in Figure 3.7 and Figure 3.8, respectively.

The specimens were cast in three stages, each being a continuous pour. First, the bottom beams of all specimens were cast from a single batch of concrete. Second, the columns were poured using one truckload of concrete. Third, the top beams were poured using the concrete delivered by another truck. The columns and top beams were cast on the same day, thirty days after the bottom beams were cast. As a result, a cold joint was formed between the columns and bottom beams. When the columns and top beams were cast, the 28-day compressive cylinder strength of the bottom beam concrete was known and it was close to target strength. Therefore, the same mix design specified in Tables B.1 and B.2 was used for the column concrete.

The casting of the bottom beams is shown in Figure 3.9. Shoot or shovel was used to place the concrete in the bottom beams. As shown in Figure 3.10, concrete was vibrated using mechanical vibration to reduce the presence of voids or honeycombs in the specimens. After casting, the top concrete surface of the beams, which is exposed to the air, was finished (Figure 3.10) and covered by plastic sheets.

Strain gages were placed on the transverse reinforcement after curing of the bottom beam concrete was completed (Figure 3.11). Fourteen strain gages were placed on the

transverse reinforcement in each specimen in the direction of lateral loading. Photographs of the strain gages on the longitudinal and transverse reinforcement are presented in Figure 3.12.

The construction joints between the bottom beams and columns were cleaned using abrasive blast methods to the extent that clear aggregate was exposed. The construction joints were then flushed with water and allowed to dry prior to placing concrete. As shown in Figure 3.13, concrete for columns and top beams was pumped from the truck. The column concrete was allowed to set before concrete was placed in the top beams. The forms were removed from the test models seven days after the concrete was cast. Figure 3.14 shows the specimens outside the laboratory before testing. The test specimens were moved into the laboratory 70 days after the columns were cast.

3.6 TEST SETUP

The test setup shown in Figure 3.15 was used to simulate gravity, and lateral and vertical earthquake loads. The top and bottom beams were post-tensioned to the loading frame and strong floor, respectively. The specimens were loaded axially using two 400-kip-capacity vertical hydraulic actuators (“A” and “B” in Figure 3.16). “Pin” connections were used at the end of the vertical actuators to minimize moment transfer to the “L” shaped steel loading frame.

The simulated lateral earthquake load was applied by a 500-kip-capacity horizontal hydraulic actuator (“C” in Figure 3.16) under displacement control. The maximum stroke of the horizontal actuator was 20 in. The horizontal actuator was attached to a

steel reaction frame and loading frame such that its loading axis passed through the column midheight (Figure 3.17). Appendix C shows the overall view of the test setup from different directions. Figure 3.18 shows the support frame used to prevent out-of-plane displacements. An 8-inch deep slot at the end of the support frame allowed horizontal movement of the loading frame only in the east-west direction. The loading frame was free to move in the vertical plane.

The actuators could be used under either load control or displacement control depending on the input command. The horizontal actuator was controlled by a prescribed horizontal displacement history. The two vertical actuators were controlled by both prescribed displacements and forces. For the specimen subjected to varying axial load, total axial load applied on the specimen was a function of lateral load. For the other specimens, axial load was maintained constant. The operation of vertical actuators and the horizontal actuator was related through control loops in the computer-controlled testing system. The operation of actuators and details of the loading system including multiple displacement and force control loops are illustrated in Appendix D.

3.7 LOADING CONSIDERATIONS

The loading conditions (lateral displacement and axial load histories) were varied for each column as shown in the test matrix in Figure 3.19. The first specimen is considered as the reference specimen with constant low axial load and standard displacement cyclic history. For each of the subsequent test specimen, only one loading condition was changed with respect to the reference test as illustrated in the test matrix.

The prescribed lateral displacement history was a function of the yield displacement. The yield displacement was using the methods presented in Chapter 2. Displacement due to flexure was obtained assuming the curvature varied linearly from the yield curvature at the ends to zero at midheight, yielding a value of 0.60 in. The contribution of bar slip and shear distortion at the yield load were calculated as 0.46 in. and 0.02 in., respectively. Based on these calculations, during the tests, a yield displacement Δ_y of 1.10 in. was used for the lateral displacement histories.

The displacement history was applied initially with three cycles each at one fourth and one half of the calculated yield displacement. Once the yield displacement was reached, the amplitude of the displacement cycles was increased incrementally, i.e., three cycles each at Δ_y , $2\Delta_y$, $3\Delta_y$, etc., until the specimen failed. The applied displacement history for each specimen is shown in Figure 3.20. The recording frequency was 1 Hz, and the rate of lateral loading varied between 0.002 in./sec for the low displacement cycles to 0.03 in./sec for the large displacement cycles.

To simulate the behavior of an exterior column, which would be subjected to varying axial load during an earthquake, typical relationship between the varying axial load, P , and the lateral earthquake load, V , can be defined as

$$P = P_G + c V \quad (3.1)$$

where c is a constant and P_G is the estimated initial gravity load on the column.

During the third test, the total vertical load, P ($= P_1 + P_2$, where P_1, P_2 = axial loads applied by the two vertical actuators) and the applied lateral load satisfied the following equation

$$\begin{aligned} P_1 + P_2 &= -250 + 5.83 V & \text{if } V > 0 \\ P_1 + P_2 &= P_G = -250 & \text{if } V = 0 \\ P_1 + P_2 &= -250 - 4.67 V & \text{if } V < 0 \end{aligned} \quad (3.2)$$

The initial total vertical (gravity) load on the third specimen was 250 kips ($\cong 0.25 A_g f'_c$) in compression. Plots of prescribed total vertical load-horizontal load relations for Specimen-3 are shown in Figure 3.21.

3.8 INSTRUMENTATION AND MEASUREMENT OF LOAD, STRAIN AND DISPLACEMENTS

During each test, one hundred twenty six channels of data were recorded at regular intervals. Three load cells, forty two strain gages, sixty two displacement potentiometers, one LVDT (Linear Voltage Displacement Transducer), eleven DCDTs (Direct Current Displacement Transducer), and seven wire potentiometers were used for the load, strain and displacement measurements.

As illustrated in Appendix D, the vertical actuators were operated under both force and displacement control, and the horizontal actuator was operated under displacement control. The vertical and lateral load recordings were a crucial part of the experimental test data. Three load cells were installed on the hydraulic actuators to monitor the lateral and vertical loads. On each actuator, a DCDT and a wire potentiometer were installed to

monitor and measure the displacements. The maximum stroke on each DCDT was 3 in. Wire potentiometers had a larger stroke compared with DCDTs, however, they were less precise at small displacements. Therefore, the displacement data smaller than 3 in. were obtained from the DCDT measurements.

Numbering and arrangement of strain gages on the longitudinal and transverse reinforcement are shown in Figure 3.22. Of the forty two strain gages, twenty eight were placed on the longitudinal reinforcing bars. Seven levels of strain gages were attached on the longitudinal reinforcement within the column height. As illustrated in the inset in Figure 3.22, at each level four strain gages were attached on the face of the longitudinal bars perpendicular to the lateral loading direction (i.e., Strain Gage-A, -B, -C, and -D). If the cross-section is subjected to moment only, of the four strain gages at each level, a pair of strain gages (e.g., Strain Gage-A and -B) will measure tensile strains while the other pair (e.g., Strain Gage-C and -D) will measure compressive strains. As shown in the inset, a pair of strain gages was placed on the outside face of the transverse reinforcement in the direction parallel to the lateral loading direction (i.e., Strain Gage-E and -F). For example, Strain Gage-E7 is located on the transverse reinforcement 4 in. below the top beam, and Strain Gage-A2 is located on the corner longitudinal bar 18 in. above the base beam.

As shown in Figure 3.23, linear displacement potentiometers were installed on the column to measure the diagonal, horizontal, and vertical displacements at various levels over the height of the column. Figure 3.24 shows an isometric view of the instrumentation frame and displacement potentiometer connections while Figure 3.25

shows a plan view. It was thought that any non-structural piece embedded in the concrete or mounted on the column would influence the structural response of the specimen during the test due to, for example, concrete crushing near the embedded piece. In addition, possible bending of a steel rod embedded in concrete would affect the local deformations recorded during the test. Therefore, for the purpose of displacement measurements, no steel rods were embedded into the column.

As shown in Figure 3.24, two 2 in. by 2 in. wood pieces were screwed and glued on both sides of the column surface perpendicular to the loading direction. Then, a 24-in. long aluminum tube section was mounted on the wood pieces on both sides. The aluminum tubes were connected by two large-diameter springs (Figure 3.25). These springs were strong enough to hold the instrumentation frame together and flexible enough to deform under expected local deformations. The displacement potentiometers were connected to the ends of these aluminum tubes. To prevent the sagging of the diagonal and horizontal potentiometers under their own weight, smaller springs were attached between the aluminum pieces and potentiometers (Figure 3.25). These springs had enough initial stiffness to hold the potentiometers in an effectively straight position and had relatively low stiffness to allow deformations during the tests. The numbering system and arrangement of the displacement potentiometers are shown in Figure 3.26. For example, the displacement potentiometer numbered 29 (LP-29) measures the relative diagonal displacement between the points at 16 in. and 40 in. above the base beam.

The total lateral displacement of the test specimens was measured by an LVDT. This LVDT was installed on a steel post attached to the strong floor about 5 ft away from the

specimen. As illustrated in Figure 3.27, two wire potentiometers and a DCDT were placed on the steel post and connected to the top beam. Considering the possibility of out-of-plane rotation of the top beam, displacements were measured on both sides of the top beam. For displacement measurements of up to 3 in., DCDTs were used. Wire potentiometers with a 15-in. stroke were used in the large-displacement range. Similarly, two DCDTs and two wire potentiometers were used to measure the displacements at the midheight of the column. Two DCDTs were mounted on the rigid wall for out-of-plane displacement measurements. These DCDTs were 6 ft apart and connected to the top beam. To measure the slippage between the base beam and the strong floor, another DCDT was installed between the base beam and the steel post. The total column vertical displacement was measured by two DCDTs installed between the top and base beams on both sides of the column. In addition, total vertical displacements were measured and monitored by the DCDTs and wire potentiometers installed on each vertical actuator, which were 6 ft away from the column.

Table 3.1 Average concrete compressive strength of specimens

Column	Test day (number of days after casting)	f_c' (psi)
Specimen-1	146	3060
Specimen-2	166	3060
Specimen-3	222	3030
Specimen-4	244	3160

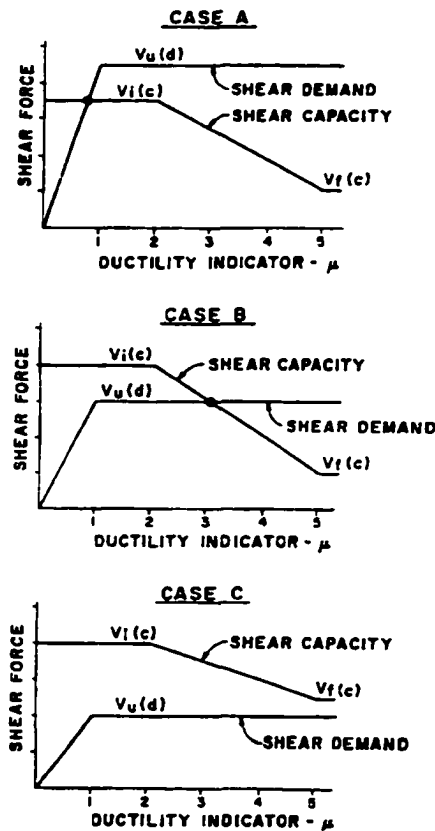


Figure 3.1 ATC model for shear demand-capacity relation

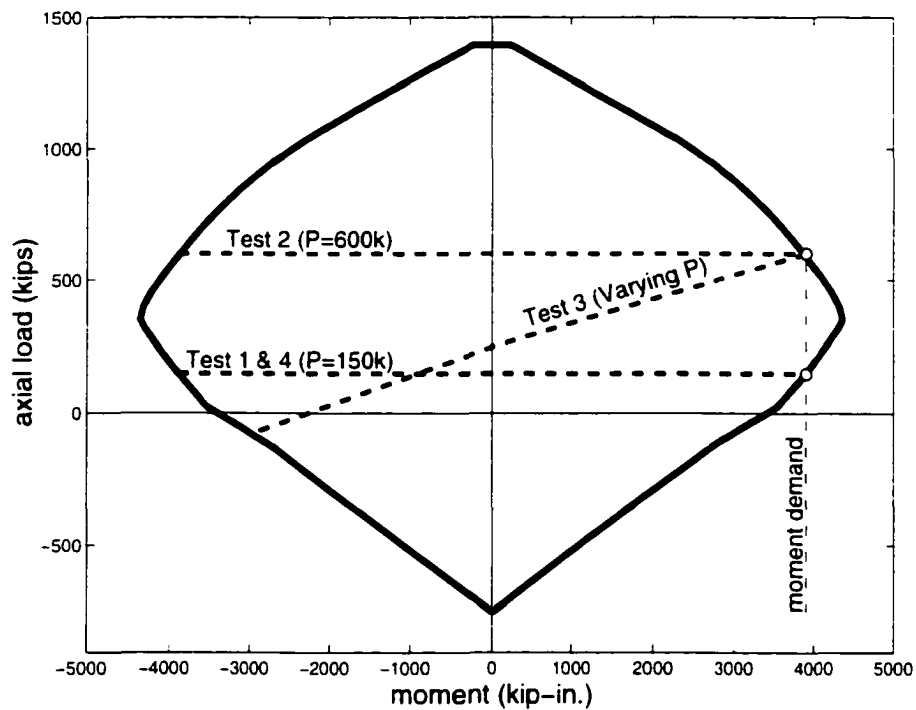


Figure 3.2 Axial load-moment interaction diagram

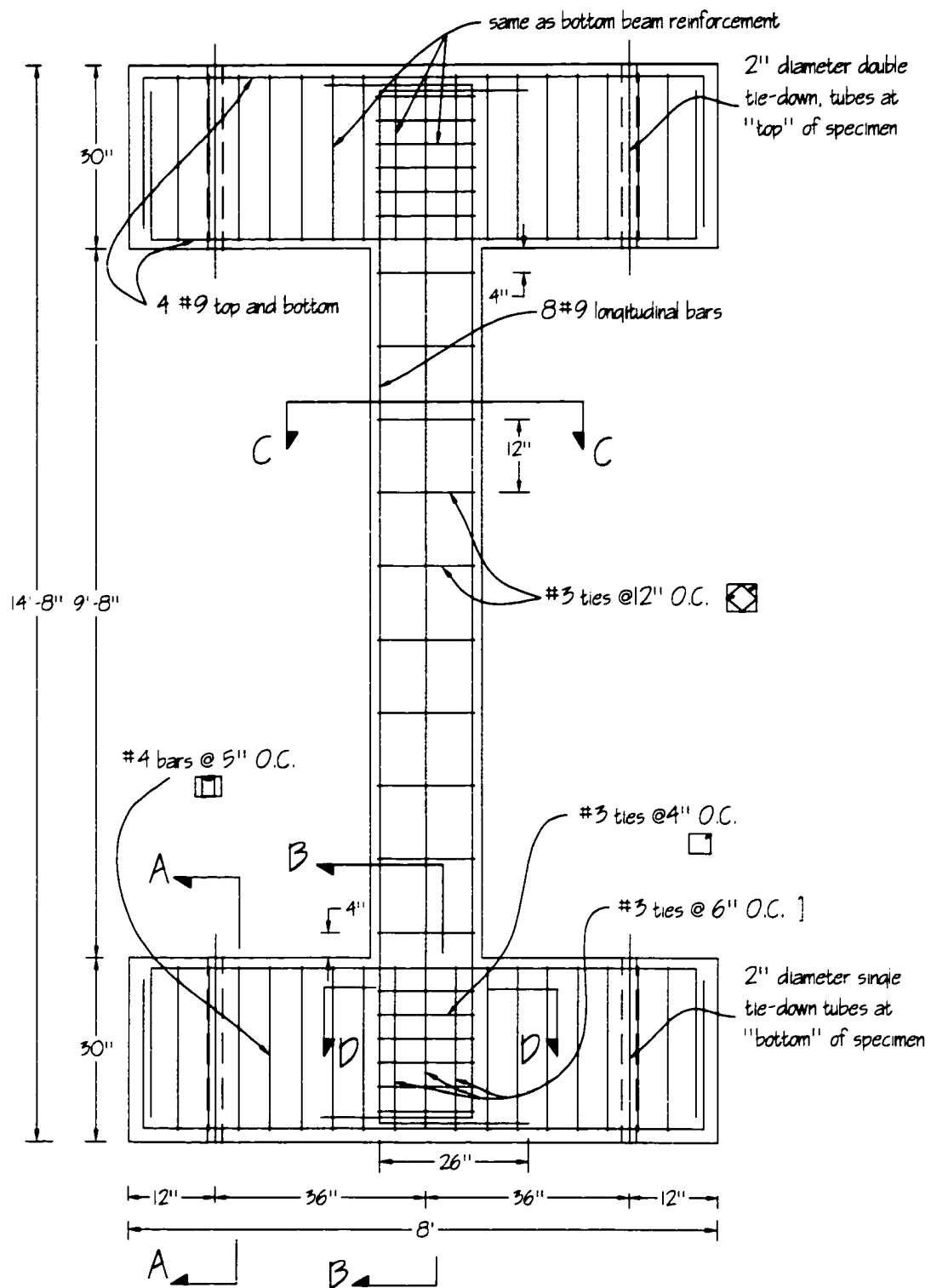


Figure 3.3 Elevation of specimens

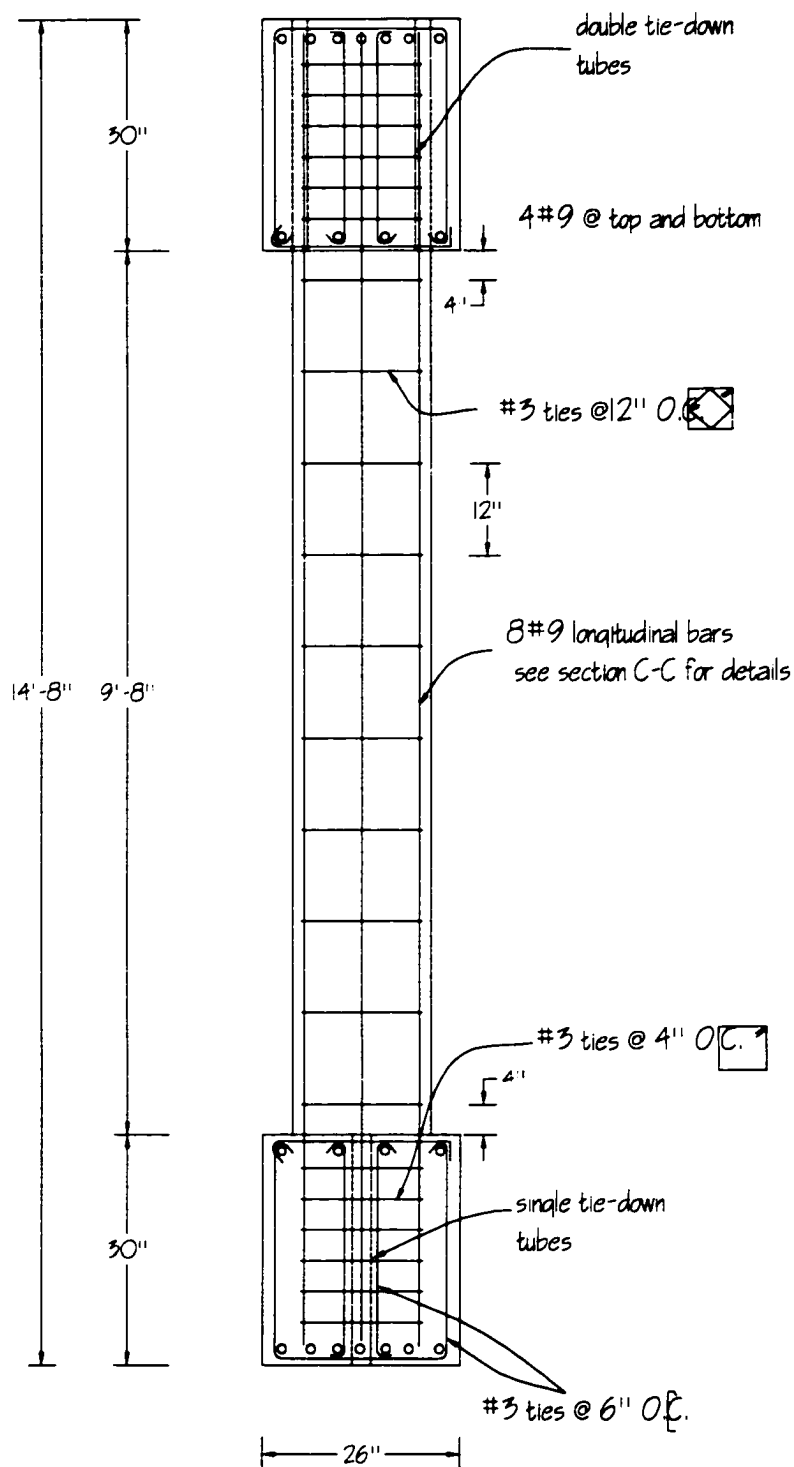


Figure 3.4 Face elevation of specimens

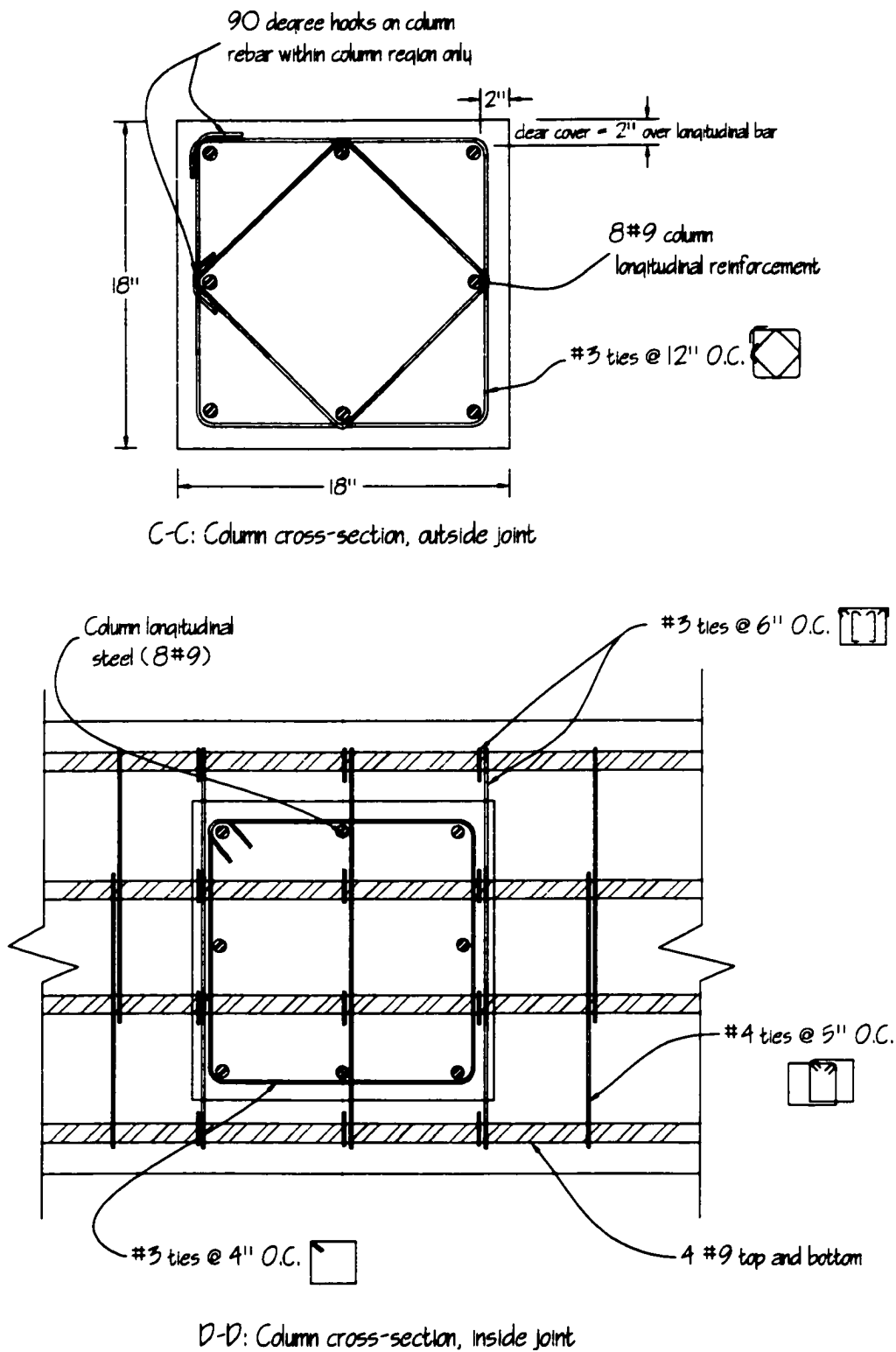


Figure 3.5 Column cross-section details

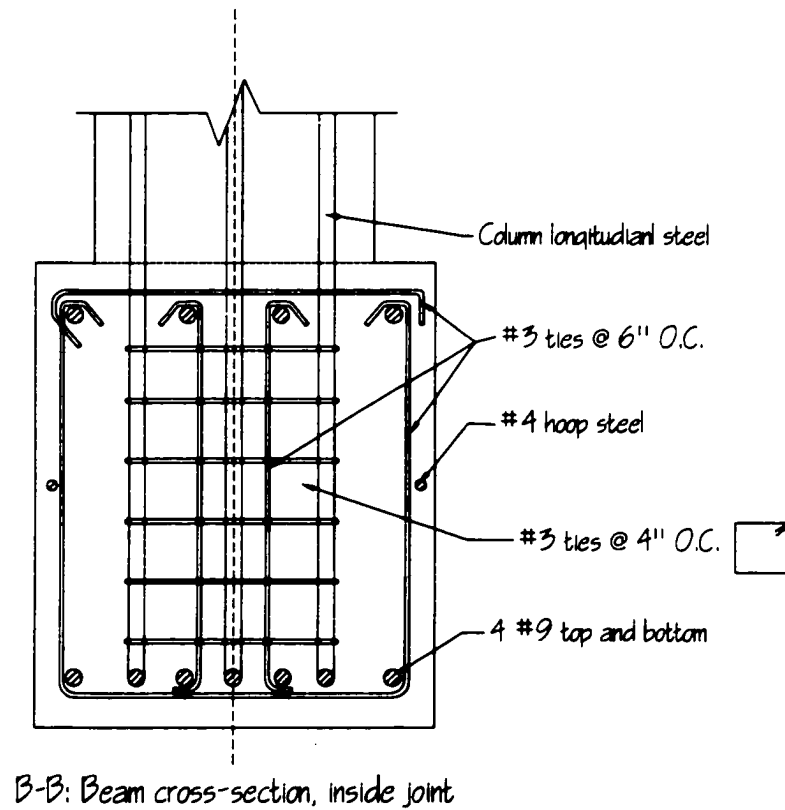
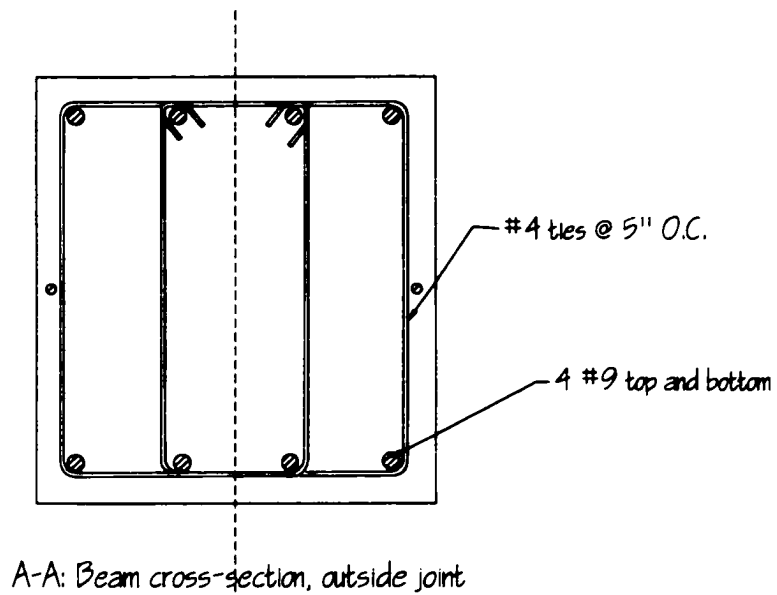


Figure 3.6 Beam cross-section details

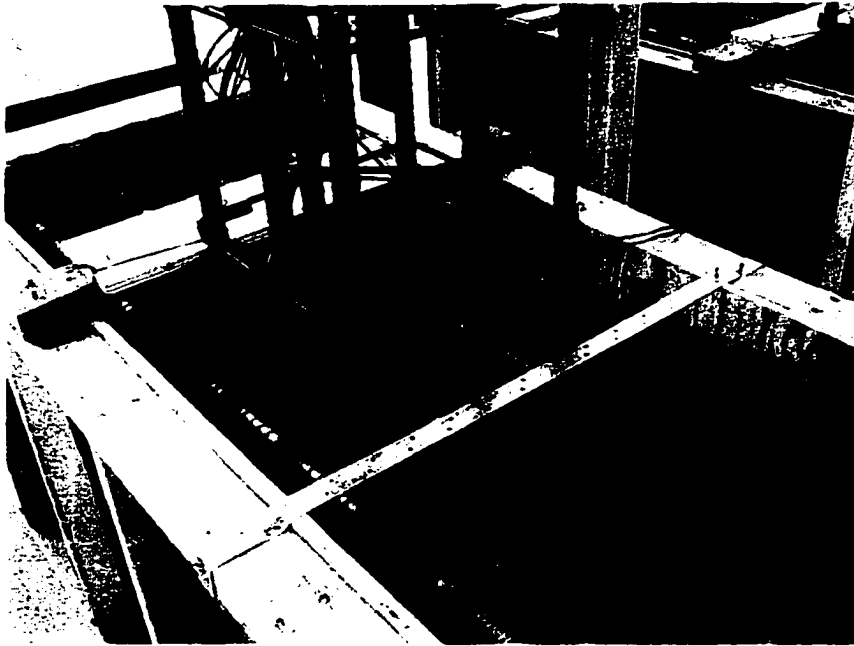


Figure 3.7 Typical reinforcement cage detail inside bottom beam

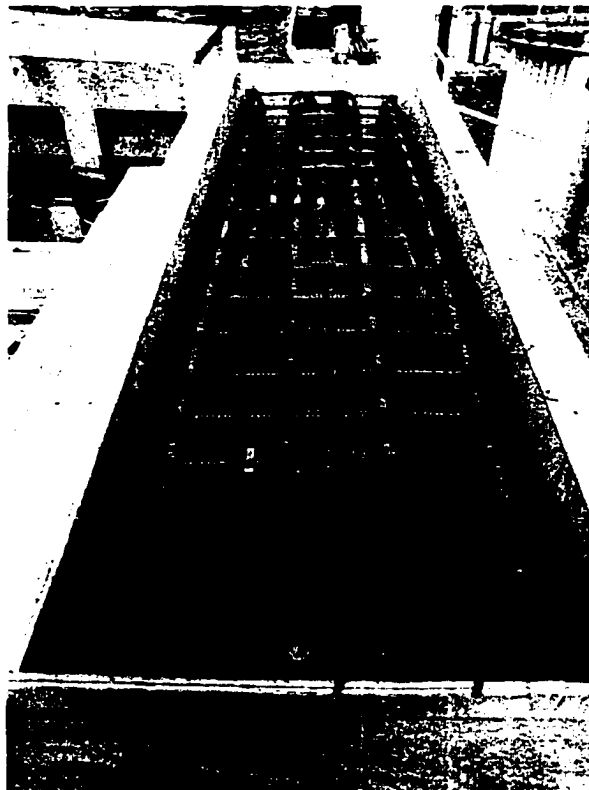


Figure 3.8 Typical reinforcement cage inside top beam



Figure 3.9 Concrete casting (base beam)

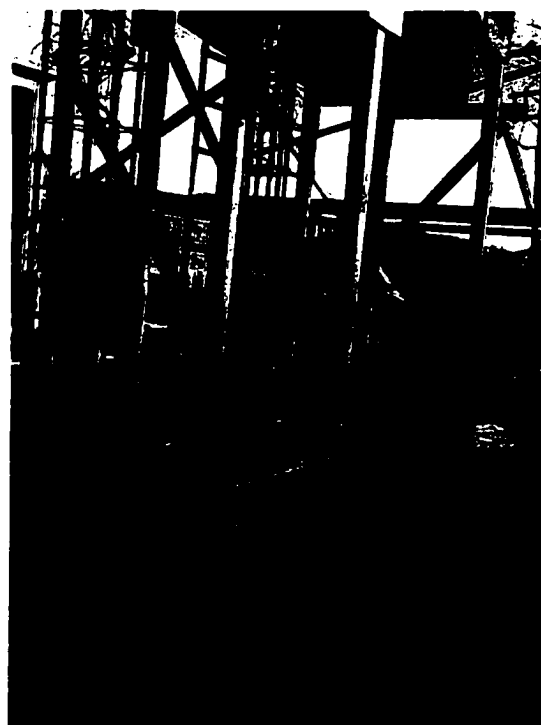


Figure 3.10 Concrete placement and vibration



Figure 3.11 Placement of strain gages on the transverse reinforcement

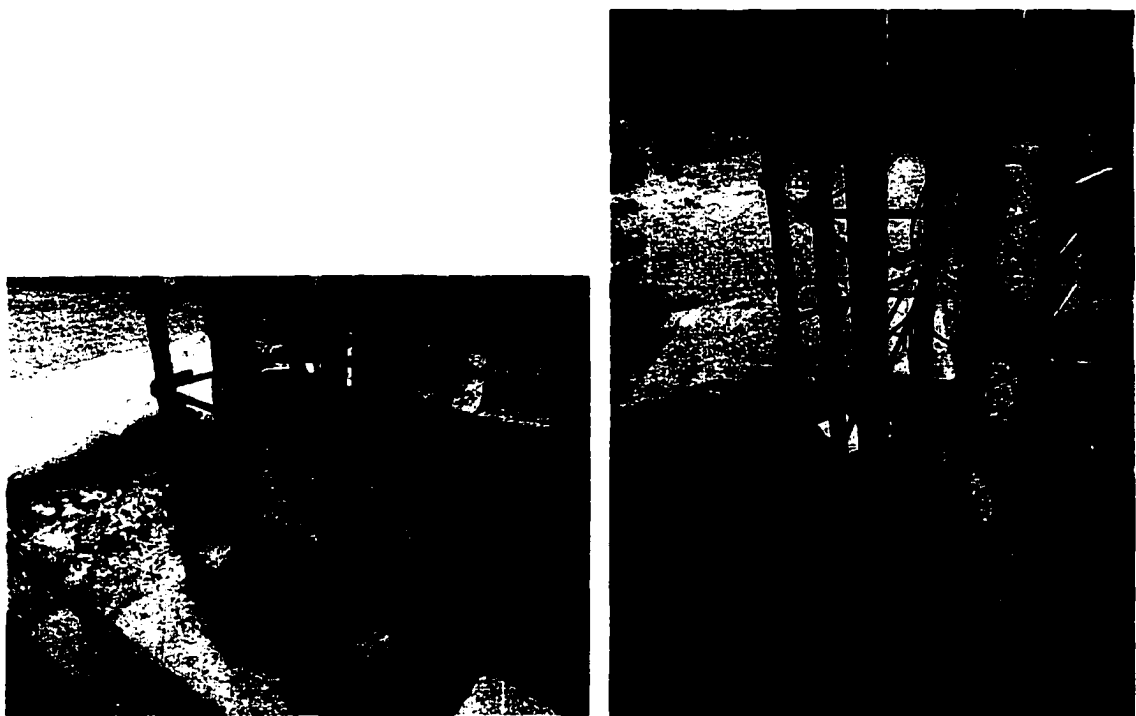


Figure 3.12 Strain gages were attached on column ties after base beams were cast



Figure 3.13 Casting of top beams and columns, and pumping and vibration of top beam concrete



Figure 3.14 Specimens outside the laboratory before testing

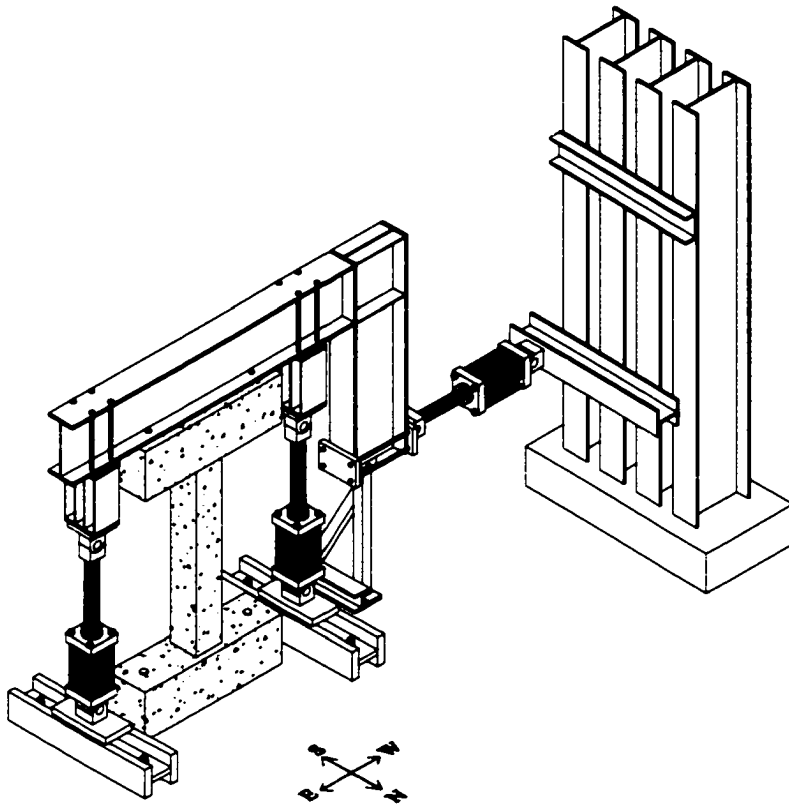


Figure 3.15 Test setup

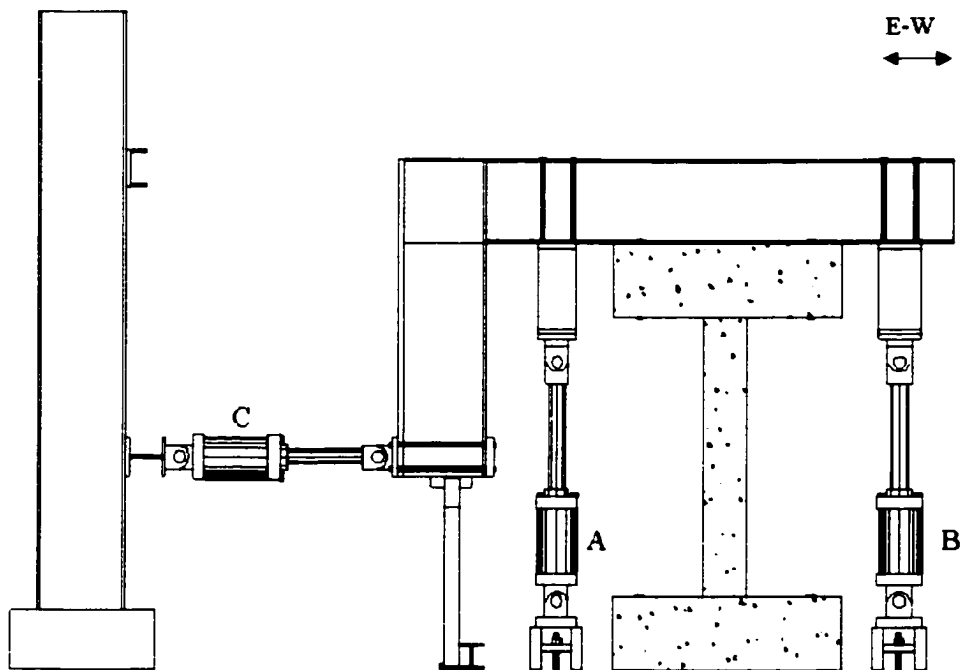


Figure 3.16 Loading frame elevation in the east-west direction



Figure 3.17 Overall view of test setup

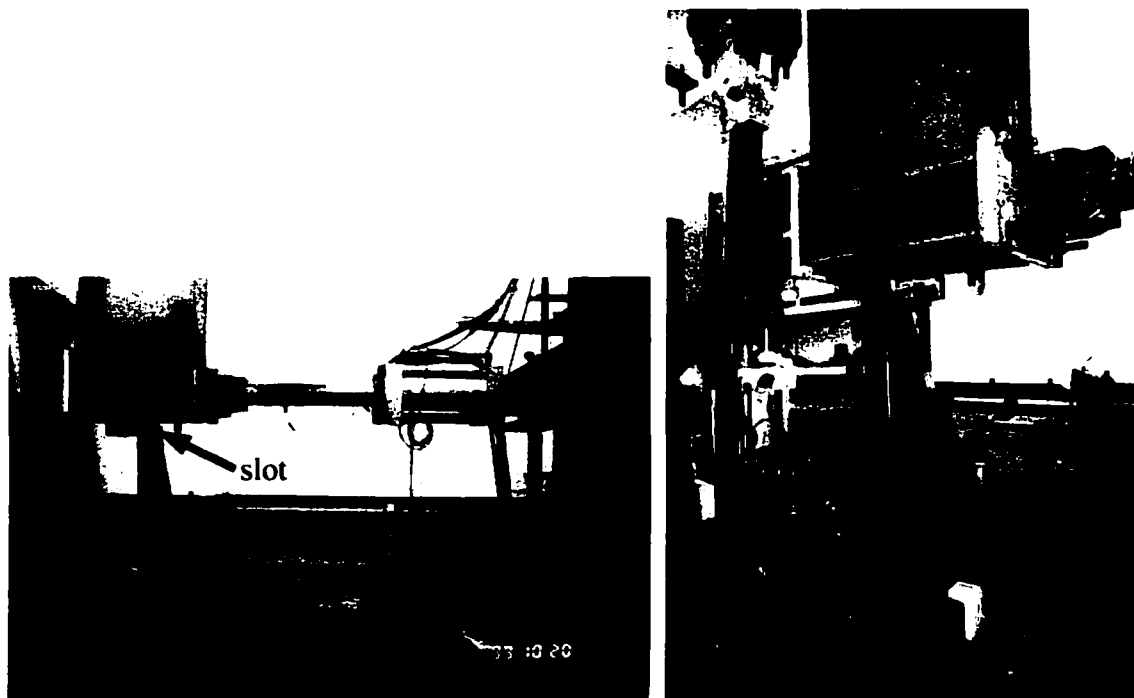


Figure 3.18 Horizontal actuator and the support mechanism used to prevent out-of-plane displacements

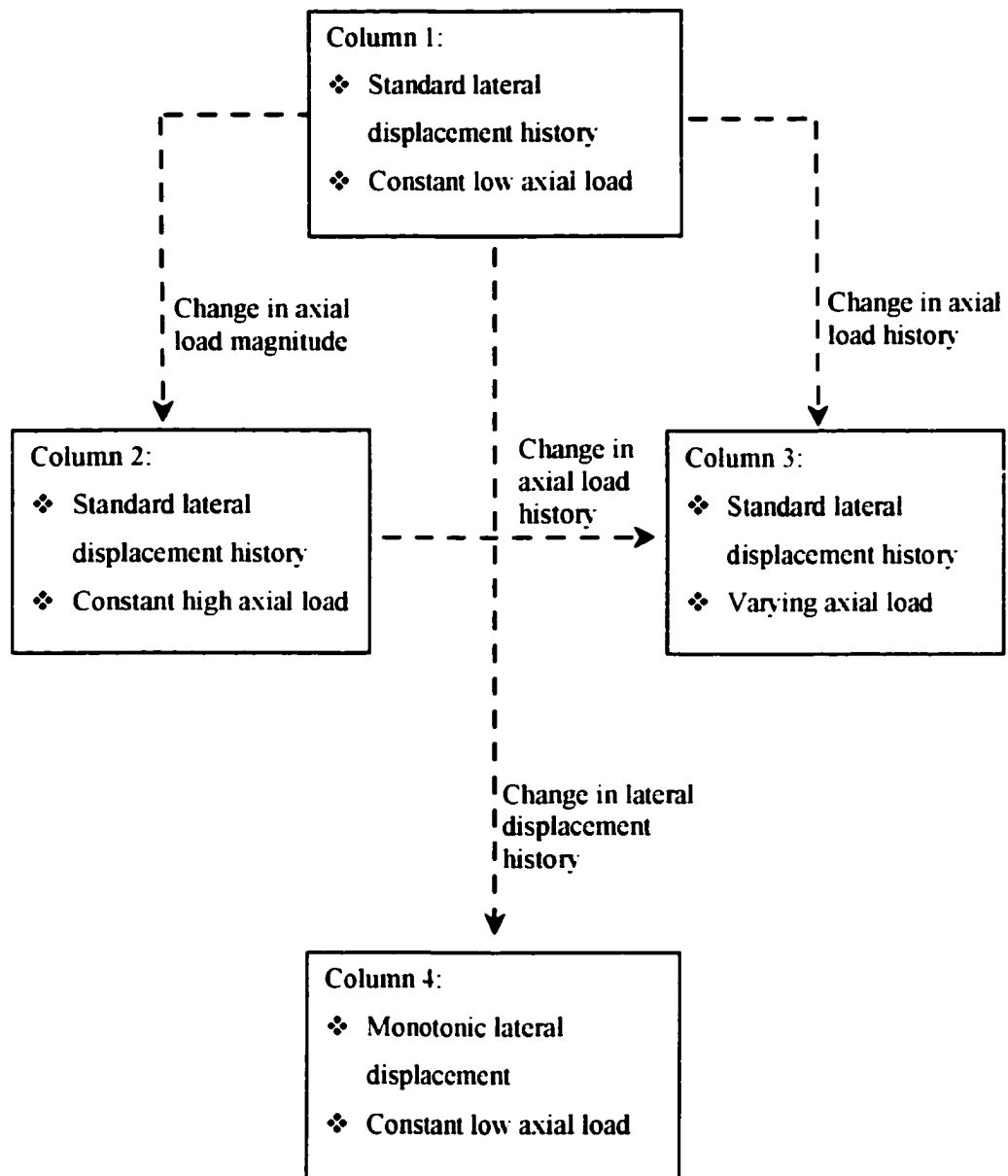


Figure 3.19 Test matrix for column specimens

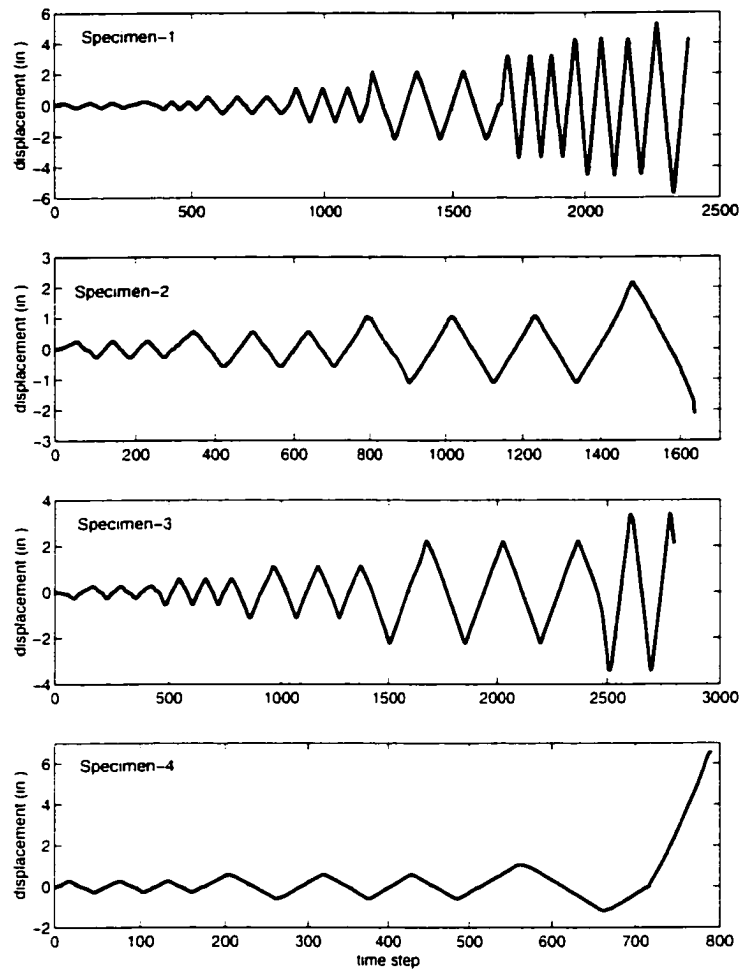


Figure 3.20 Applied displacement histories

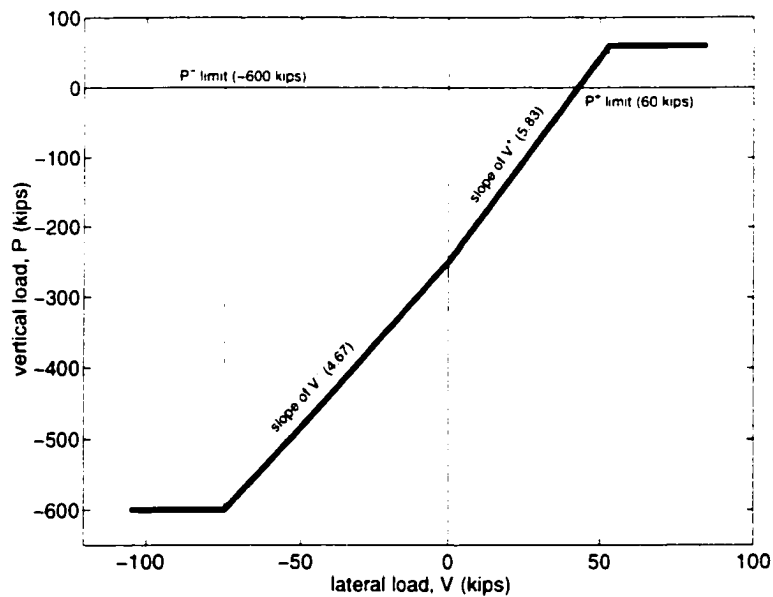


Figure 3.21 Prescribed axial-lateral load relations for Specimen-3

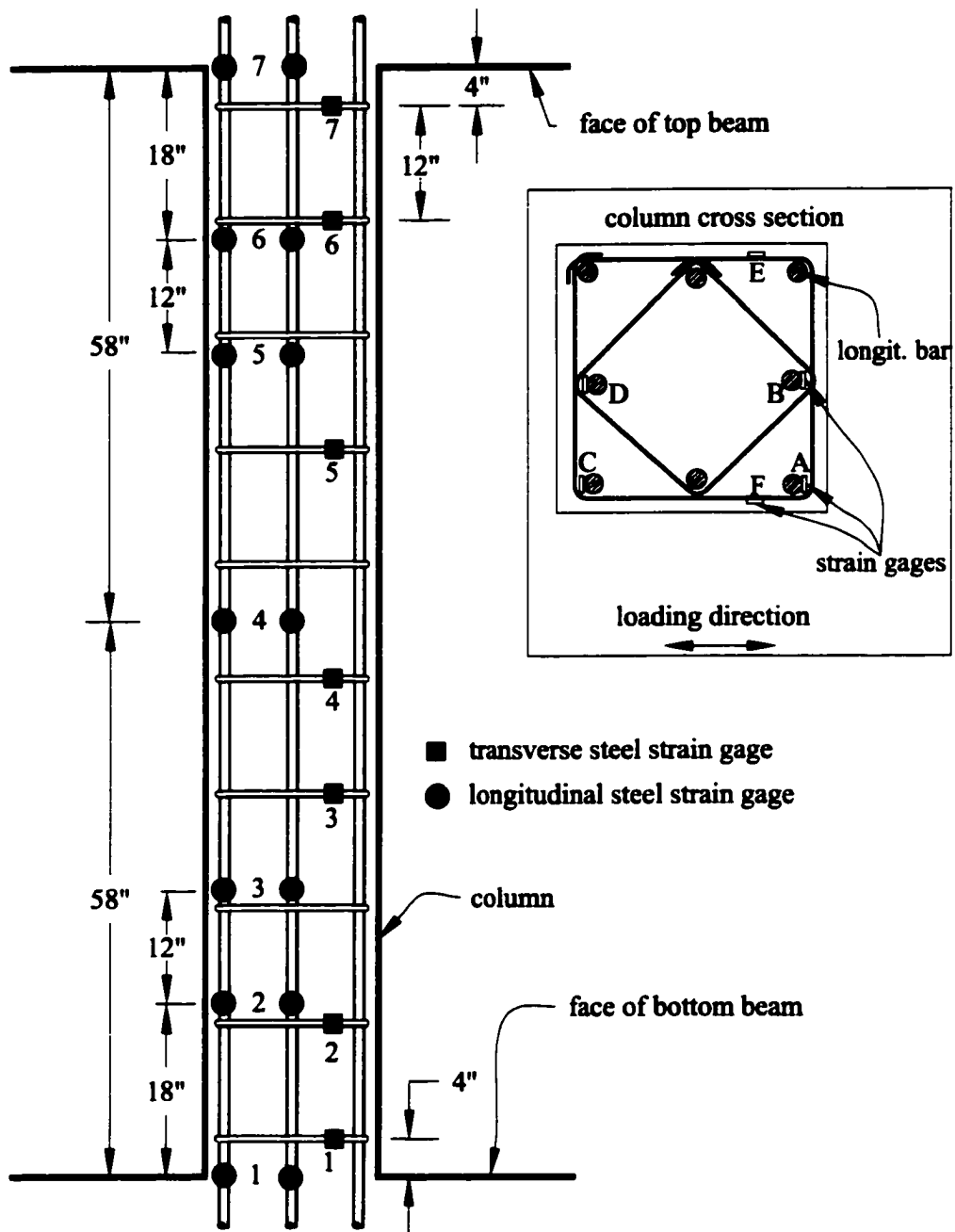


Figure 3.22 Arrangement and numbering of strain gages on the hoops and longitudinal bars

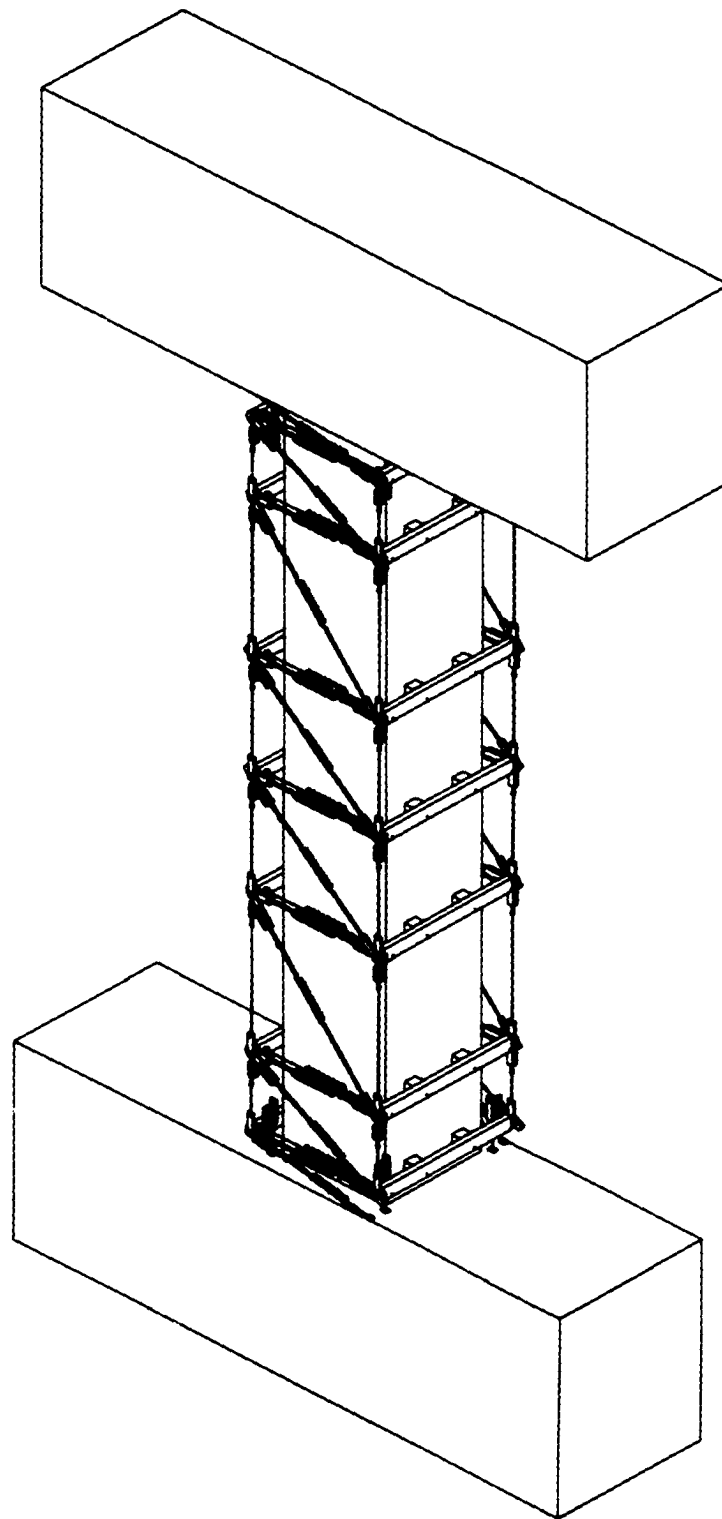


Figure 3.23 Arrangement of displacement potentiometers installed on the column

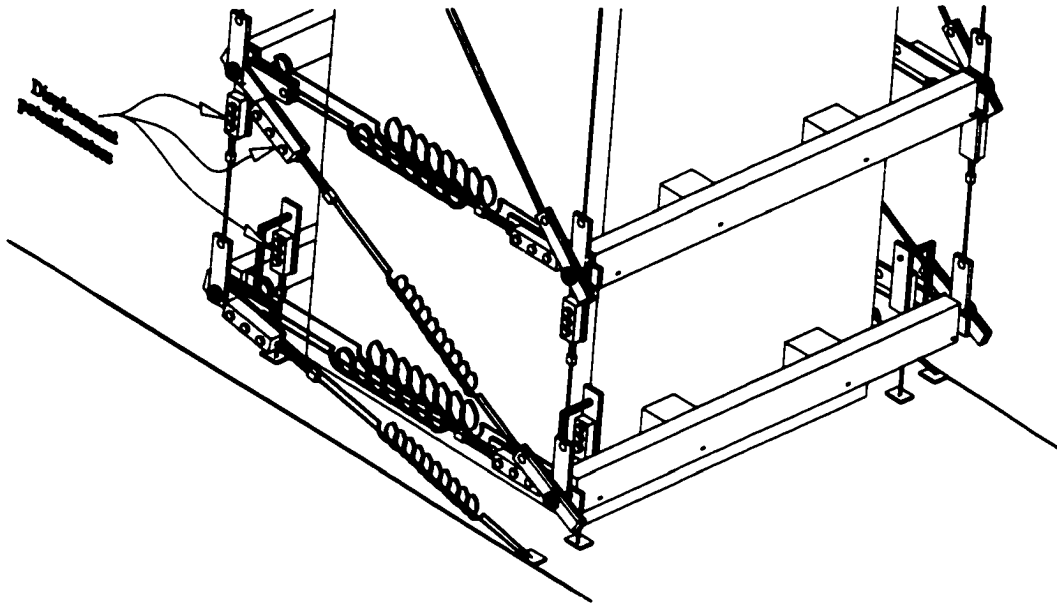


Figure 3.24 Close-up view of instrumentation setup

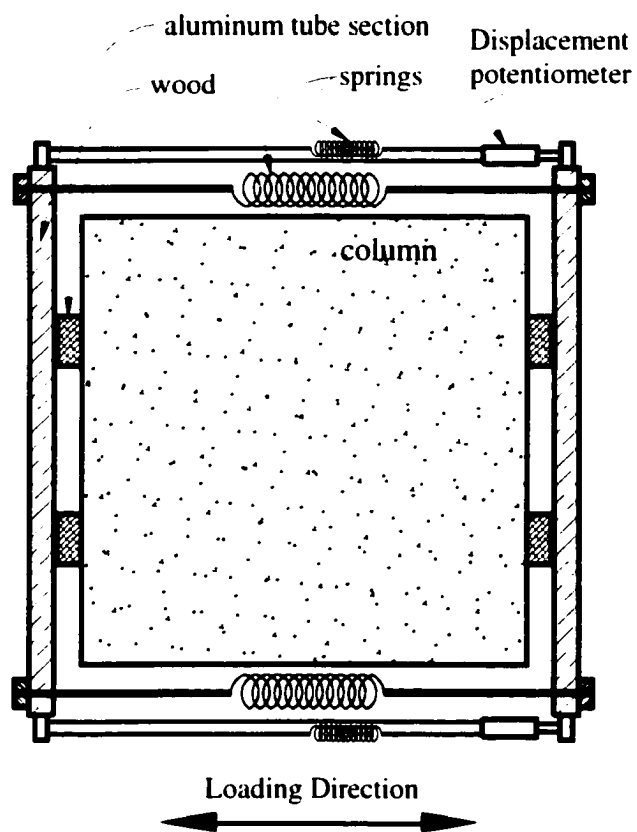


Figure 3.25 Top view of instrumentation setup

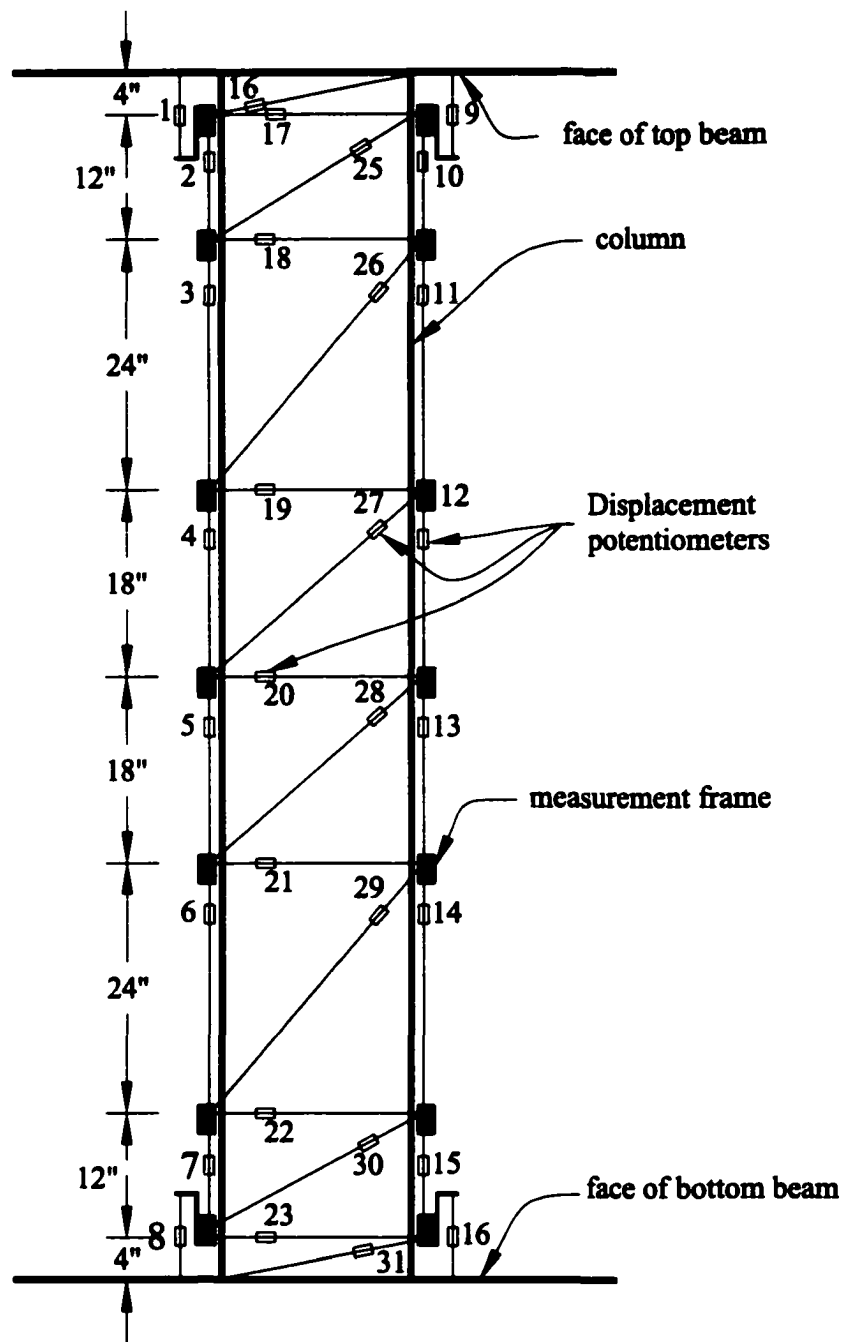


Figure 3.26 Arrangement of displacement potentiometers

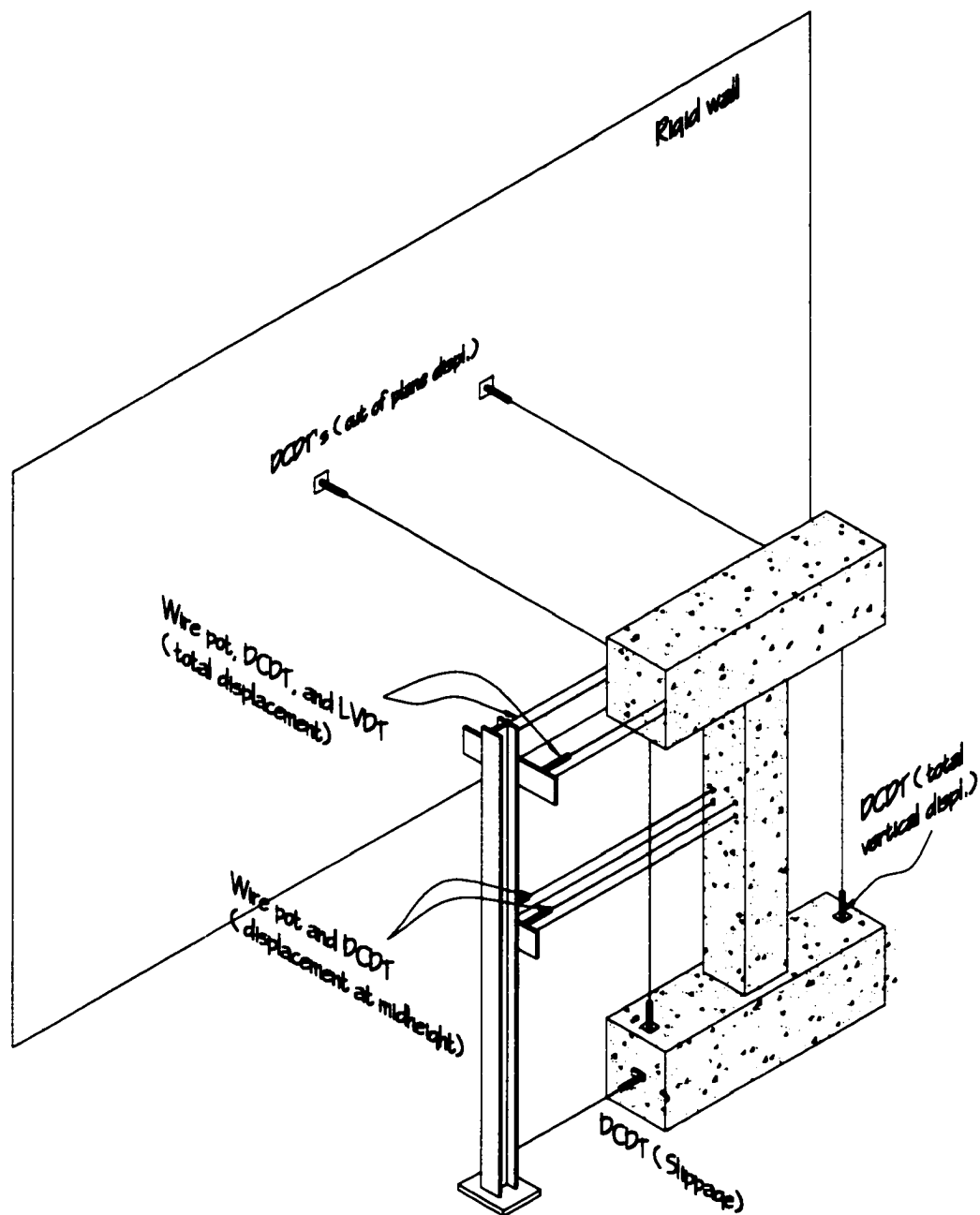


Figure 3.27 Setup for global deformation measurements

4 TEST RESULTS AND OBSERVATIONS

4.1 INTRODUCTION

This chapter provides a summary of the test results including damage description and test data measured during each test. This chapter also includes a brief description of the data reduction procedures, including calibrations and corrections of offsets, modification for second-order effects, and description of engineering quantities such as moment and average curvature.

Based on visual observations and recorded test data, the performance of each test specimen is discussed. For each specimen, the measured lateral load-displacement relations and plots of other important test parameters are presented. The damage description of specimens and their implications are discussed, and the measured

response is compared mainly in terms of applied displacement and load configurations described in the previous chapter. The following sections discuss the response of each specimen and observations made during each test.

4.2 SPECIMEN-1: CONSTANT LOW AXIAL LOAD

Specimen-1 was subjected to a constant compressive axial load of 150 kips ($0.15 f'_c A_g$, where f'_c = specified compressive concrete strength, and A_g = gross cross-sectional area) and cyclic increments of lateral displacements as described in Figure 3.20.

Before the test, horizontal hairline cracks apparently due to shrinkage were distributed relatively uniformly over the height of the column. During the initial displacement cycles to one fourth and one half of the nominal yield displacement, inclined hairline cracks (width of less than 0.005 in.) developed near the top and bottom of the column. At these displacement levels, no new cracks were observed around the midheight region of the column. The number of inclined cracks and the crack width on the faces in the lateral loading direction (i.e., on the north and south faces, Figure 3.15), increased as the number and magnitude of the displacement cycles increased. Small horizontal cracks on the faces perpendicular to the lateral loading direction (i.e., on the east and west faces), started to span the width of the column. These relatively straight continuous horizontal cracks opened and closed during each cycle. During displacement cycles to one half of the nominal yield displacement, vertical cracks were observed on the north face of the top and bottom beams within the joint regions. On the flexural tension side of the top beam-column interface and footing-column interface, relatively large crack openings

were observed suggesting slip of the longitudinal reinforcing bars from the beams. Figure 4.1 shows the 0.028-in. wide crack opened between the flexural tension side of the column base and the beam at peak lateral displacement.

During displacement cycles to the nominal yield displacement ($\Delta_y = 1.10$ in.), more inclined cracks appeared near the top and bottom of the column. The middle third of the column was still uncracked except for the pre-existing shrinkage cracks. On the column faces perpendicular to the loading direction, the width of the existing horizontal cracks increased and more cracks formed closer to the midheight of the column.

At the beginning of cycling at the displacement level of 2.2 in. ($2\Delta_y$), when the specimen was loaded the first time (push or eastward direction, Figure 3.15), spalling of the cover concrete was observed in the top northeast corner. In the other loading direction (pull or westward direction), wide inclined cracks appeared around the middle third of the column. In the flexural compression zones, at the top and bottom of the column, flaking and scaling of concrete were observed. When the lateral load was reversed (push or eastward direction), another wide inclined crack with an orientation opposite that noted previously and a width of approximately 0.008 in. appeared along the midheight of the column. As the number of cycles increased, some vertical cracks and more horizontal cracks appeared on the east and west faces in the midheight region of the column.

Figure 4.2 through Figure 4.4 show the change in the damage pattern as the number and amplitude of the displacement cycles increased. On the flexural compression side, concrete flaking and spalling increased at the top and bottom of the column. By the

displacement level of 3.3 in. (3 Δ_y), wide inclined cracks had merged with other vertical cracks that followed the longitudinal reinforcement on both sides. The instrumentation frame and the displacement potentiometers were dismantled after 4.4 in. (4 Δ_y) displacement cycles were completed (Figure 4.4). Even though the cover concrete spalled off over a wide region in the upper half of the column, no permanent deformations, buckling, or other significant distress were observed in the longitudinal reinforcement. Similarly, in the transverse reinforcement, no hook release or fracture was observed at this stage.

During the cycles to a maximum displacement of 5.5 in., in the upper half of the column, the 90-degree hooks at the ends of the perimeter hoops gradually opened. Figure 4.5 shows the damage in the column at a lateral displacement of 5.5 in. when the test ended. At this stage, the column was able to carry the applied constant axial load. Nevertheless, the testing had to be stopped, as the column had almost no lateral resistance due to significant deterioration in concrete, and vertical shortening was progressing with continued cycles.

Figure 4.6 shows the recorded lateral load-displacement relation for Specimen-1. The lateral load was corrected to include the effect of horizontal component of the load applied by the two vertical actuators. The maximum recorded lateral strength and displacement was 71 kips and 5.76 in., respectively. There was a significant reduction in the lateral strength after the displacement exceeded 2.2 in. However, the column was able to sustain its axial-load-carrying capacity at relatively large displacements until the

end of the test. More detailed description of the column behavior and implications of test results will be discussed in the following chapter.

4.3 SPECIMEN-2: CONSTANT HIGH AXIAL LOAD

Specimen-2 was subjected to constant compressive axial load of 600 kips ($0.6 f'_c A_g$) and displacement-controlled loading history as shown in Figure 3.20. During the low amplitude displacement cycles ($\Delta_y/4$ and $\Delta_y/2$), observed damage included extensions of existing horizontal shrinkage cracks and formation of several new hairline cracks. The number and size of the cracks increased with increasing displacement amplitude. By the displacement level of one fourth of nominal yield displacement, $\Delta_y/4$, the bar slip between the top and bottom beams and the column tension zones was visible as a relatively wide crack at the interface. The bar slip increased roughly in proportion with the increase in lateral displacement.

During the yield displacement cycles, some inclined cracks appeared within 36 inches from top and bottom of the column. Outside these regions, cracking was limited to opening and closing of initial shrinkage cracks. Flaking of the cover concrete was observed in the flexural compression zones near column ends.

During the first displacement cycle to a maximum displacement of 2.2 in. ($2\Delta_y$), the damage was quite extensive with the development of several vertical and inclined cracks near the ends of the column (Figure 4.7). Some of the inclined cracks were as wide as 0.18 in. The concrete cover spalled off near the column ends in the flexural compression zones. As the magnitude of the lateral displacement increased, a wide vertical crack

formed over the longitudinal reinforcement along the entire length of the column (Figure 4.8). After the formation of this vertical crack, a steep inclined crack formed within the upper half of the column. As shown in Figure 4.9 and Figure 4.10, damage associated with this inclined crack included opening of the column tie hooks and the buckling of longitudinal bars near this wide crack.

When the lateral displacement was reversed to 2.2 in., the column had sudden loss of both axial and lateral-load-carrying capacities. The test had to be terminated when the total vertical displacement exceeded 1.0 inch after the axial load failure. Figure 4.11 and Figure 4.12 show the failure plane and the crack pattern from different angles at the end of the test. Note that the longitudinal bars buckled in different directions with no consistent trend. Also, the length of the portion of the buckled bar varied widely.

The measured lateral load-displacement relation for Specimen-2 (Figure 4.13) shows a minor reduction in strength due to repeated cycling for displacements as small as 1.1 in. (nominal yield displacement). During the first displacement cycle to a maximum displacement of 2.2 in., the maximum lateral load was reached at 1.5 in. lateral displacement followed by loss in resistance. The loss in the lateral strength continued in the opposite direction after the lateral load was reversed. When testing was terminated, the bottom half of the column had limited damage while the upper portion was extensively damaged (Figure 4.11).

As described in Chapter 3 and illustrated in Figure D.4 in Appendix D, total axial load applied by the two vertical actuators was required to be constant during the test. As an

example of showing the performance of vertical load control mechanism, the axial loads applied on Specimen-2 are plotted in Figure 4.14. The figure shows that the axial load histories applied by the two vertical actuators were out of phase because of the unbalanced weight of the L-shaped steel loading beam and P- Δ effects. However, the total applied axial load was almost constant throughout the test.

4.4 SPECIMEN-3: VARYING AXIAL LOAD

Specimen-3 was tested under simultaneously varying axial load and cyclic lateral load following the axial load-lateral load relationship given in Chapter 3 (Equation 3.2). To simulate gravity load on the column, a compressive axial load of 250 kips ($0.25f'_c A_g$) was applied initially. When the column was laterally loaded in the positive loading direction, the axial load was increased linearly up to 600 kips. In the negative loading direction, the axial load was decreased linearly to a minimum axial tensile load of about 56 kips. Table 4.1 lists the maximum and minimum axial loads applied in each displacement cycle during the test. As the lateral load resistance decreased near the end of the test, the maximum compressive axial load was reduced from 600 kips to 530 kips. On the other hand, because lateral strength resistance increased progressively in the other direction, the tensile axial load increased after each lateral displacement increment.

During the 0.27-in. ($\Delta/4$) and 0.55-in. ($\Delta/2$) displacement cycles, well-distributed existing horizontal cracks combined with new inclined hairline cracks extended along the height of the column.

Flaking of the cover concrete was observed in the flexural compression zone at the column base during each cycle at the displacement level of 1.10 in. (Δ_y). At the end of the first cycle, the measured longitudinal bar slip was about 0.06 in. at the top beam-column interface. Mostly in the positive loading direction, i.e., under larger compressive axial loads, new inclined cracks developed in the joint regions within the beams. At this stage, numerous new inclined cracks appeared near the top and bottom of the column. Only a few inclined cracks formed around the midheight of the column at this displacement level. The number and width of the inclined cracks increased with increasing displacement cycles and amplitude.

During the 2.20-in. ($2\Delta_y$) displacement level, the maximum measured longitudinal bar slip at the beam-column interface was about 0.12 and 0.20 in. at the end of the first and second cycles, respectively. Figure 4.15 shows the crack pattern at the end of the first cycle to maximum 2.2 in. displacement. At this stage, new inclined cracks formed and older cracks opened and extended as the number of cycles increased. Some inclined cracks were as wide as 0.12 in. At the end of the second cycle, in the positive loading direction when the axial compressive load reached 600 kips, spalling and crushing of concrete was observed in the flexural compression zone at the bottom. At the end of displacement cycles to 2.2 in., crushing of concrete extended into the core concrete at the bottom.

At the end of the first 3.30-in. ($3\Delta_y$) displacement cycle, more crushing and spalling of concrete took place in the compression regions at the top and bottom. A wide-open continuous vertical crack formed along the height of the column over the longitudinal

reinforcing bars on the opposite side of the crushed concrete at the bottom. In the positive loading direction, under 600 kips compressive axial load, along with more concrete crushing and spalling, a longitudinal bar buckled in the corner between the two column ties near the column base (Figure 4.16). During the second 3.30-in. displacement cycle, when the lateral load was reversed, the buckled longitudinal bar straightened under tensile load. When the lateral load was reversed and the compressive axial load was applied, the column ties opened failing to confine the core concrete and longitudinal bars. Figure 4.17 and Figure 4.18 show the damage and crack pattern at the end of 3.30-in. displacement cycles. Figure 4.19 shows the specimen after the loss of lateral-load-carrying capacity and removal of instrumentation frames. As demonstrated in the figure, crushing and spalling of concrete just below the top beam was limited to the cover concrete only, and was not as significant as the damage in the bottom third of the column.

After the test was terminated, in order to obtain the residual axial-load-carrying capacity of the column, the test specimen was loaded axially at zero lateral displacement. Prior to loss of axial-load-carrying capacity, the column carried an axial load of 98 kips at zero lateral displacement. Figure 4.20 shows the bottom of the column after the loss of axial-load-carrying capacity.

In the measured lateral load-displacement relation shown in Figure 4.21, the negative lateral load was plotted on the tensile axial load side. The nonlinear response shown in the figure indicates a large decrease in the lateral stiffness in the negative loading direction under the low compressive axial load or tensile axial load. Figure 4.22 shows

the applied axial load-lateral load relationship for Specimen-3. The applied loads closely followed the specified relationship given in Figure 3.20 and Equation 3.2.

4.5 SPECIMEN-4: CONSTANT LOW AXIAL LOAD AND MONOTONIC LATERAL LOAD

Specimen-4 was subjected to constant compressive axial load of 150 kips and displacement-controlled lateral load history as shown in Figure 3.20. After the displacement cycles to a maximum displacement of one fourth and one half of the yield displacement were completed, the specimen was loaded laterally up to nominal yield displacement (1.10 in.) in the positive and negative directions only once. Then, the specimen was loaded monotonically until it lost its lateral-load-carrying capacity.

During the low-amplitude displacement cycles ($\Delta_y/4$ and $\Delta_y/2$), the cracks generally appeared to be horizontal hairline cracks or extensions of horizontal shrinkage cracks. New inclined cracks appeared and the crack length and width increased with increasing displacement amplitude. At yield displacement level, in the positive loading direction, the measured longitudinal bar slip was about 0.02 in. at the bottom beam-column interface. When the lateral load was reversed, the measured slip was about 0.03 in. on the other side of the interface. During the first cycle to yield displacement, the damage and new inclined cracks were concentrated within the upper and lower one third of the column (Figure 4.23). At this displacement level, a few inclined cracks were initiated in the joint regions inside the top and bottom beams.

Under monotonic lateral loading, the opening of one of the previously formed inclined cracks close to the column base led to the final failure. Figure 4.24 shows the propagation of damage including crack opening, longitudinal bar buckling, and column tie failure. Figure 4.25 shows the crack pattern after the loss of lateral-load-carrying capacity. At 6.5 in. lateral displacement, the column was unable to carry 150-kip axial load applied on it (Figure 4.25).

After the test was completed under monotonic loading, the specimen was pulled back to zero lateral displacement. Then, it was loaded axially until it lost its residual axial-load-carrying capacity. Prior to loss of axial-load-carrying capacity, the column carried an axial load of 145 kips at zero lateral displacement. Figure 4.26 compares the damage at the bottom of the column before and after the loss of axial-load-carrying capacity. Note that the column shown on the left had no lateral strength, but was able to carry the applied axial load.

Figure 4.27 shows the measured lateral load-displacement relation for Specimen-4. The test had to be terminated because of lateral load failure and relatively large vertical displacements leading to an unstable mechanism. However, as described above, the residual axial capacity at zero lateral displacement (145 kips) was almost equal to the original constant axial load applied on the column (150 kips).

4.6 PERFORMANCE DESCRIPTION AND LIMIT STATES

Figure 4.28 compares the measured lateral load-displacement relations for all specimens. The results indicate that the performance of columns with poor seismic details is

significantly influenced by the applied displacement history and the magnitude of the axial load. With increasing axial load, the lateral strength and stiffness increase and the displacement capacity decreases. Figure 4.28 indicates that in the negative lateral loading direction, the lateral strength and stiffness are the largest in Specimen-2, which was subjected to very high axial load. The response of Specimen-3 shows that the column behavior appears to be greatly affected by the magnitude of the applied axial load. Under high compressive loads, Specimen-3 failed in shear at a displacement smaller than the displacement Specimen-1 and Specimen -4 failed, and at a displacement larger than the displacement Specimen-2 failed indicating a relation between the axial load level and displacement capacity at shear failure. Unlike other specimens, in Specimen-3, no significant strength reduction was observed under tensile or low compressive axial load. Comparison of the lateral load-displacement response of Specimen-4 with others shows the reduction in the resistance and displacement capacity as a result of cyclic loading.

A qualitative description of damage progression and the damage states for each specimen at different displacement ductilities are summarized in Table 4.2. Important damage parameters characterizing the performance of specimens at certain displacement ductility levels were found to be the location, width, and orientation of the cracks, yielding of longitudinal reinforcing bars, spalling of cover concrete, buckling of longitudinal bars together with opening of tie hooks, and crushing of core concrete.

Typically, during the initial low-displacement cycles ($\Delta_r/4$ and $\Delta_r/2$), other than inclined hairline cracks near the top and bottom of the columns, no significant damage was

observed in the specimens. During the cycles to nominal yield displacement, the first yielding in longitudinal bars was observed in all columns except Specimen-2, which was subjected to very high axial load. In addition, at this displacement level, inclined cracks developed near the ends of the columns in each specimen. At $2\Delta_y$ displacement level, Specimen-2 lost its axial and lateral-load-carrying capacity suddenly. At this level, spalling of the cover concrete and extensive shear cracks were observed in other columns. During the subsequent displacement cycles, large vertical cracks over the longitudinal reinforcement, and continued development and extension of inclined cracks resulted in a substantial loss in lateral strength of the columns. In general, the final failure, that is, the loss of axial-load-carrying capacity, was associated with opening of column tie hooks, buckling of longitudinal bars, and the core concrete crushing.

Based on the observed damage and performance of twelve columns tested in this study and by Lynn et al. (1996), four engineering limit states and corresponding damage descriptions are suggested in Table 4.3. The bond cracks are defined as large vertical cracks following the longitudinal reinforcing bars along the column height (e.g., Figure 4.8.a). The lateral load-displacement relations with limit-state envelopes for the specimens tested in this research are shown in Figure 4.29. In Table 4.3, in addition to transient drift ratio, the proposed four damage levels are described by the engineering limit states, which include the first yielding in the longitudinal reinforcing bars, maximum lateral strength, and loss of lateral and axial capacity. The damage level III, with a limit state defined as the loss of lateral capacity, can be assumed to be reached when the lateral strength becomes smaller than 80 percent of the maximum lateral

strength, after the maximum strength/damage level II is reached. It should be noted that the table does not include structural performance levels for frames, as that requires in-depth knowledge of component interaction and additional information. The limit states and damage levels described in Table 4.3 are developed for individual nonductile columns based on limited available experimental data.

FEMA 273 (1997) describes typical performance levels and damage states for vertical components of existing reinforced concrete frames. Comparison of damage states and performance levels suggested in Table 4.3 with those described in FEMA 273 (Table 4.4) shows that the performance of columns with transient drifts larger than 2% should be evaluated more cautiously. The test results of this study suggest that the transient drift ratio, or the displacement ductility, largely depends on the column axial load, especially for drift values larger than 2 percent. The effect of column axial load is not mentioned explicitly as an engineering parameter in Table 4.3 and Table 4.4.

4.7 DAMAGE DISTRIBUTION AND CRACK PATTERN

Before the tests, shrinkage cracks were marked in black color on each test specimen. As described in the previous sections, the displacement histories were applied with three cycles to a maximum displacement that was a fraction or multiple of the nominal yield displacement. During the first cycle of each displacement level, the lateral loading was stopped after the peak displacement was reached. Then, new cracks were marked on the specimen in different colors, i.e. in blue and red, in the positive and negative directions, respectively. During each stop at the peak displacement, in addition to marking new

cracks on the specimen, the maximum crack width was also measured. The residual crack widths at zero lateral displacement were not measured systematically. The crack widths measured at each displacement level are listed in Table 4.5. Typical hairline cracks are assumed to range between 0.002 and 0.004 inches.

The crack widths reported in the Table 4.5 suggest that the crack opening is directly related to the magnitude of the axial load applied on the column. For example, a comparison of the measured crack widths at the nominal yield displacement level shows that the crack width is the smallest in Specimen-2 with an axial stress of $0.60f'_c$ and it is the largest in Specimen-3 with an axial stress approaching zero. The typical crack width measured in the other two specimens with an axial stress of approximately $0.15f'_c$ is between those of the second and third specimens. Similarly, at the $2\Delta_y$ displacement level, the crack width in Specimen-3 is much larger than that measured in Specimen-1. At this displacement level, Specimen-3 was subjected to tensile axial load. It should be noted that after the yield displacement is exceeded, the crack distribution and crack widths are related to local failure mechanisms including concrete spalling and development of shear, flexural or bond cracks.

Figure 4.30 shows the crack distribution on each face of every specimen at 0.55-in. ($\Delta_y/2$) lateral displacement. The crack distribution shown in the figure includes shrinkage cracks as well as cracks developed during the displacement cycles to one fourth and one half of the yield displacement. As described in the previous sections, in general, well-distributed horizontal cracks were dominant in these early stages of the tests.

Figure 4.31 shows the crack pattern in the columns after the completion of the yield displacement cycles. In general, more inclined cracks appeared within the top and bottom third of each column. The inclined cracks developed in Specimen-2 were steeper and fewer as compared with the other specimens. The typical inclinations of inclined cracks, which were developed during the yield displacement level and beyond, were measured to be approximately sixty degrees. In all columns, as an indication of joint distress, at this stage some new cracks developed inside joint regions in the beams. The inclined cracks in the beams in Specimen-3 occurred consistent with the inclined concrete strut direction carrying compression forces while the specimen was subjected to higher axial load.

The crack distribution at 3.30-in. displacement level in Specimen-1 and Specimen-3 are shown in Figure 4.32. As an indication of lateral load failure, in addition to formation of new inclined cracks near the top and bottom, large inclined cracks were distributed in the middle third of both columns. Both specimens had significant number of hairline cracks as a result of distress inside the joint regions in the beams.

Figure 4.33 shows the damage pattern at the end of each test. Except for the last specimen, the failure mechanism involved combination of large vertical cracks and inclined cracks near the either end of the column. The first two specimens failed near the top of the column, while the last two specimens failed near the column base. No additional cracks were noticed in the joint regions near the end of the tests.

4.8 AXIAL RESPONSE

As illustrated in Figure 3.2, Specimen-1 and Specimen-4 were subjected to 150 kip constant compressive axial load. Specimen-2 was tested under a 600-kip constant axial load. The axial load applied on Specimen-3 varied as a function of lateral load as defined in Equation 3.2 and demonstrated in Figures 3.20 and 3.21. Along with the relation between the lateral load and lateral displacements, Figure 4.34 shows the variation of axial load as a function of lateral displacement for each specimen. As shown in the figure, the axial load on Specimen-2 and Specimen-4 had to be reduced near the end of the tests as the specimens became unstable and were not able to support the applied axial load.

During the tests, vertical displacements were recorded by DCDTs installed on both sides of the test columns between the top and bottom beams (Figure 3.27). The average of vertical displacements measured by the two DCDTs on each specimen is plotted in Figure 4.35. The calculated theoretical elastic vertical displacements, $\Delta_{vertical}$, are also shown in the figure ($\Delta_{vertical} = PL/A_gE_c$, P = applied axial load, L = column length, A_g = gross cross-sectional area, and E_c = modulus of elasticity for concrete). After the application of constant gravity load in Specimen-1 and Specimen-4, an initial vertical displacement of approximately 0.017 in. was calculated using elastic material properties. Note that the calculated initial vertical displacements were lower than the initial average measured displacements in Specimen-2 and Specimen-4. The vertical displacement distribution along the height of the columns indicated that vertical displacements measured by the local displacement potentiometers near the interface of the column and

beams were larger than those measured along the rest of the column. This was possibly because open cracks at the interface closed under high compressive axial loading.

Figure 4.36 shows the relation between the measured vertical displacement and lateral displacement, and the relation between the vertical displacement and lateral load for each specimen. As expected, the vertical displacements vary with variations in vertical and lateral loading history. Under small lateral loads, the columns tend to elongate with increasing lateral deformations, presumably as a result of crack opening along the column height. As the columns experience further damage, the tendency to elongate reverses into shortening which continues until the final failure occurs. Figure 4.37 compares the vertical displacements with lateral load and lateral displacement for the two columns with nominally identical properties, i.e., Specimen-1 and Specimen-4. The envelope of vertical displacements for Specimen-1, which was subjected to cyclic lateral loads, and for Specimen-4, which was subjected to low level cyclic loads and then monotonic lateral load, show surprising similarity. Under monotonic loading, Specimen-4 elongates until the peak lateral strength is reached. When the lateral strength degradation starts, the axial response is also reversed and the column starts to shorten. Shortening continues until the column loses its axial-load-carrying capacity.

4.9 DATA REDUCTION AND CORRECTION OF FORCES

Figure 4.38 shows the test specimen and loading apparatus under the applied loads, and free-body diagrams of the test setup. The dimensions are also shown in the figure. The length of the vertical actuators connecting the loading beam to the rigid base was 102

inches. Since the axial loads applied by actuators P_1 and P_2 (shown in Figure 4.38) were relatively large, the horizontal components of the actuator loads (P_{1x} and P_{2x}) were significant at large lateral displacements. These horizontal forces were acting in the direction opposite to the direction of the lateral load, V . Therefore; the shear force on the column (or the corrected lateral load, V_{cor}) was calculated by subtracting the horizontal component of vertical actuator loads from the applied lateral load. When the axial load is in tension, P_{1x} and P_{2x} act in the same direction as the applied lateral load. Thus, the corrected lateral load increases when the effect of tensile vertical actuator loads is considered. The shear force on each specimen and all lateral loads reported in this study are corrected to include the effect of the horizontal component of the “vertical” actuator loads.

The equilibrium equations for the calculation of vertical and horizontal components of the “vertical” actuator loads and moments at the top and bottom of the columns (M_T and M_B , respectively) are provided in Figure 4.38. The effect of vertical displacements was assumed to be relatively small and was ignored in the calculations. No correction was needed in the calculation of total axial load, P , since the difference between the vertical load components (P_{1y} and P_{2y}) and the actuator loads (P_1 and P_2) was negligible. A summary of the applied and corrected peak lateral loads, corresponding displacements, and moments at the top and bottom of each test column is presented in Table 4.6.

P - Δ effects were not considered in the data reduction process to correct the lateral loads. P - Δ effects tend to reduce the lateral shear force resistance. Measured data were not adjusted to remove this effect. However, the effect of P - Δ on the moment calculated

about the base of the column is included implicitly in the equilibrium equations given in Figure 4.38. The moment equation shows that the base moment, M_B , increases by $P*\Delta_h$ ($= (P_{1y}+P_{2y})\Delta_h$) when the lateral displacements are included in the calculations. The lateral loads applied by the horizontal actuator can be corrected using Equation 4.1 to include the effect of P - Δ . The effect of P - Δ on the lateral load-displacement relations is shown in Figure 4.39. The P - Δ effect reduces the lateral load with increasing axial load and lateral displacement. On the other hand, the lateral load increases if the axial load is in tension, e.g., as in the response of Specimen-3 near negative lateral displacement peaks.

$$V_{with P-\Delta} = V_{cor} - \frac{P \Delta_h}{L} \quad (4.1)$$

4.10 DATA PROCESSING AND STRAIN DISTRIBUTION

The measurements from the strain gages installed on the transverse and longitudinal reinforcing bars are presented in Appendix E. The recorded data were not processed to eliminate the high frequency content in strain measurements, e.g., in Strain Gage-E3, -F3 and -F4 in Specimen-2, and Strain Gage -D7 in Specimen-1. However, the strains were corrected for drifting and other possible errors caused during waiting periods between the different loading cycles during the tests. The waiting periods, up to three hours long, included the time needed for marking the cracks on the test specimens and the time needed to fix unexpected mechanical or electronic problems. Typically, drifting or other errors were observed in the form of a sudden jump between the last strain measurement in a recorded data file and the first strain value in the successive data file with a waiting

period between the two data recordings. In a few such cases, the first measured strain value in the second data file was matched by the strain measured at the end of preceding cycles by subtracting the drift observed during the waiting period. In the same way, the local deformations measured by the displacement potentiometers installed on the test columns were corrected for similar errors. It was not uncommon for people occasionally to accidentally touch or hit the displacement potentiometers, DCDTs or connection wires while marking cracks on the columns during waiting periods. The displacements presented in Appendix E were corrected to eliminate similar errors by assuming that the displacements remained unchanged during the waiting period.

The strain distributions over the height of test columns in the transverse and longitudinal directions are shown in Figure 4.40 and Figure 4.41, respectively. The strains were plotted in each loading direction (pull and push), at first yielding in the longitudinal bars, peak lateral load, and loss of lateral capacity (ultimate) level, which is assumed to occur when the lateral load drops to 80 percent of peak lateral strength after the peak strength is reached. The yield strain for the transverse and longitudinal steel is also indicated in the figures. In general, when the longitudinal reinforcement yielded the first time, the strains in the transverse reinforcement were much smaller than the yield strain. Because of the larger flexural demand near the column ends, the strains measured on the longitudinal reinforcement are much larger near the top and bottom of columns. On the other hand, at peak and ultimate levels, the transverse reinforcement strains tended to be the largest some short distance away from column ends, where most of the damage and extensive cracking were observed due to combined shear and flexural demand. As the damage progresses and more cracks intersect the transverse reinforcement, the strains in

the transverse direction increase. For instance, in Specimen-2 with less damage and less number of cracks at failure, the strains in the transverse direction were much smaller prior to failure.

4.11 AVERAGE CURVATURES

The average curvatures, ϕ_ϵ , were obtained by calculating the difference between the measured strains, $\Delta\epsilon$, measured on the longitudinal reinforcing bars at the same height and dividing this strain difference by the distance between the two bars, d_ϵ ($\phi_\epsilon = \Delta\epsilon/d_\epsilon$ where d_ϵ = section height-two times the concrete cover). Along with the calculated yield curvature, the average curvature profiles from strain measurements are shown in Figure 4.42 for each specimen at 0.5, 1.0 and 2.0 times the nominal yield displacement levels. If the data from a strain gage were missing or unreliable, instead of using the average of two strain measurements, the reading from only one strain gage at that location was used (e.g., Strain Gage-A or -B, or Strain Gage-C or-D in Figure 3.22). In general, except for beam-column interfaces at the top and bottom, the average curvatures were smaller than the calculated yield curvature and the average curvature distribution was almost linear over the height and similar in shape in all specimens. The failure of Specimen-2 near the top at two times the nominal yield displacement gives rise to relatively large curvatures measured around that location.

The average curvatures, ϕ_Δ , were also calculated using the displacements, δ_i , measured by the displacement potentiometers installed on the test specimens. The average strain, ϵ_Δ , at the average height of two potentiometers was calculated as the difference between

the two displacements recorded by the potentiometers divided by the distance, h_{ij} , between them. Then, the average curvature was calculated as the strain difference, $\Delta\epsilon$, on both sides of the column divided by the horizontal distance between the potentiometers, b_p ($\phi_\Delta = \Delta\epsilon / b_p$, where $\Delta\epsilon = (\delta_j - \delta_i) / h_{ij}$). The calculated average curvature profiles for each specimen are shown in Figure 4.43. Apparently, because of additional deformations due to longitudinal bar slip, the average curvatures were much larger near the beam-column interfaces at the top and bottom of columns. Other than the large average curvatures near the top and bottom, in general, the average curvatures were smaller than the calculated yield curvature and varied almost linearly over the height. The calculation of effective curvatures and deformations due to bar slip or flexure alone will be demonstrated in the next chapter.

Table 4.1 Maximum and minimum applied axial loads (Specimen-3)

Cycle	Maximum axial load (kips)	Minimum axial load (kips) *
$\Delta_y/4$	-390	-120
$\Delta_y/2$	-500	-62
$1\Delta_y$	-600	0
$2\Delta_y$	-600	+51
$3\Delta_y$	-530	+56

* (+): in tension, (-): in compression

Table 4.2 Qualitative damage description

Cycle	Specimen-1	Specimen-2	Specimen-3	Specimen-4
$\Delta_y/2$	inclined cracks near bottom and top of column	inclined cracks near bottom and top of column	inclined cracks near bottom and top of column	inclined cracks near bottom and top of column
$1\Delta_y$	yielding of long. bars, inclined cracks at the top and bottom 1/3 of the column	inclined cracks within 25" from the ends of the column	yielding of long. bars, inclined cracks at the top and bottom 1/3 of the column	yielding of long. bars, inclined cracks at the top and bottom 1/3 of the column
$2\Delta_y$	large inclined cracks at midheight, concrete spalling	combined large inclined and vertical cracks in upper half of column	cover concrete spalled off, large inclined cracks over the height	-
$3\Delta_y$	large inclined and vertical cracks	-	core concrete crushed, large inclined and vertical cracks, longit. bars buckled, tie hooks opened	-
$4\Delta_y$	cover concrete spalled off, long. bars exposed, extensive vertical cracks	-	-	-
$5\Delta_y$	longitudinal bars buckled, tie hooks opened, core concrete crushed	-	-	longitudinal bars buckled, tie hooks opened, core concrete crushed

Table 4.3 Damage description and limit states for nonductile columns

Damage level	Engineering limit state	Damage state	Damage description	Transient drift (%) (or displ. ductility)
I	Yielding	minimal damage	longitudinal bar yielding, minor inclined cracks near ends	1
II	Peak lateral strength	limited damage	cover concrete spalling, bond cracks and shear cracks at midheight	1-2
III	Loss of lateral capacity (~80% peak strength)	major damage	large shear and bond cracks	≥ 2
IV	Loss of axial capacity	repair not possible	crushing of core concrete, opening of tie hooks, buckling of longitudinal bars	≥ 2

Table 4.4 Performance levels and damage description for vertical elements (FEMA 273)

Table 2-4 Structural Performance Levels and Damage¹—Vertical Elements

Elements	Type	Structural Performance Levels		
		Collapse Prevention S-5	Life Safety S-3	Immediate Occupancy S-1
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking ($< 1/8"$ width) for ductile columns. Minor spalling in nonductile columns. Joint cracks $< 1/8"$ wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints $< 1/16"$ width.
	Drift ²	4% transient or permanent	2% transient; 1% permanent	1% transient; negligible permanent

Table 4.5 Measured crack widths (in inches)

Cycle	Specimen-1	Specimen-2	Specimen-3	Specimen-4
$\Delta_y/4$ and $\Delta_y/2$	hairline	hairline	hairline	hairline
$1\Delta_y$	0.016	0.008	0.024	0.020
$2\Delta_y$	0.028	-	0.098	-
$3\Delta_y$	> 0.40	-	0.120	-

Table 4.6 Summary of applied and corrected loads and moments at peak lateral load

Specimen	P	Δ_{peak}	V	$P_{1x}+P_{2x}$	V_{cor}	M_T	M_B
No.	kips	in.	kips	kips	kips	k-in.	k-in.
1	-150	2.2	74.0	3.2	70.8	4202	4331
2	-600	1.1	86.6	6.0	80.6	5102	4863
3	-600	1.1	74.1	6.4	67.7	3763	4743
-	+56	3.4	53.6	-1.9	55.5	2902	3342
4	-150	2.4	69.8	3.5	66.2	3932	4111

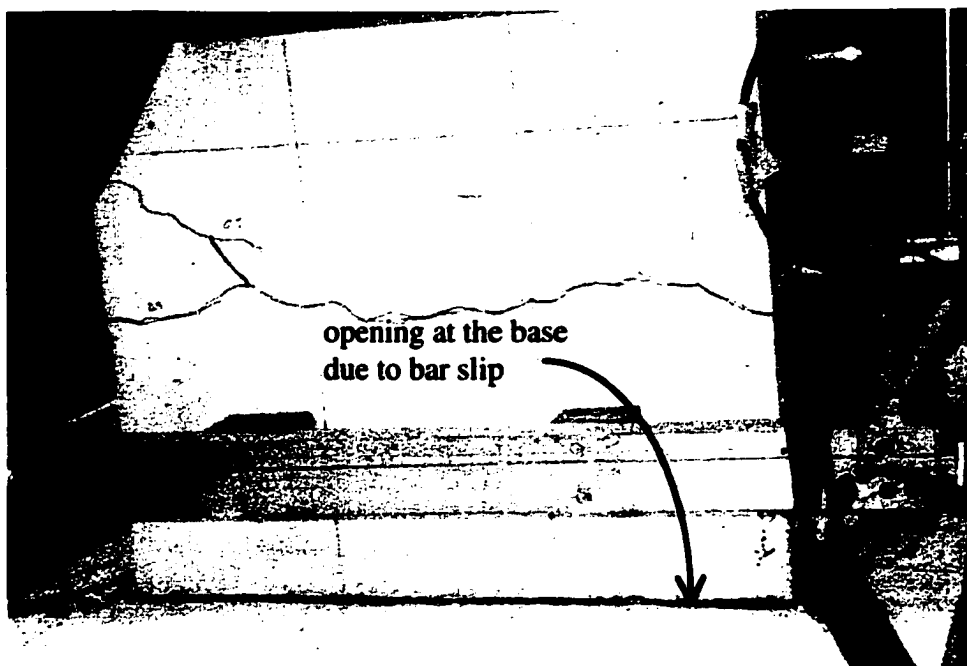


Figure 4.1 Slip in the tension zone at the column base (Specimen-1)



(a) Beginning of $2\Delta_y$



(b) $3\Delta_y$

Figure 4.2 Damage progress at $2\Delta_y$ and at $3\Delta_y$ (Specimen-1)



(a) Overall view

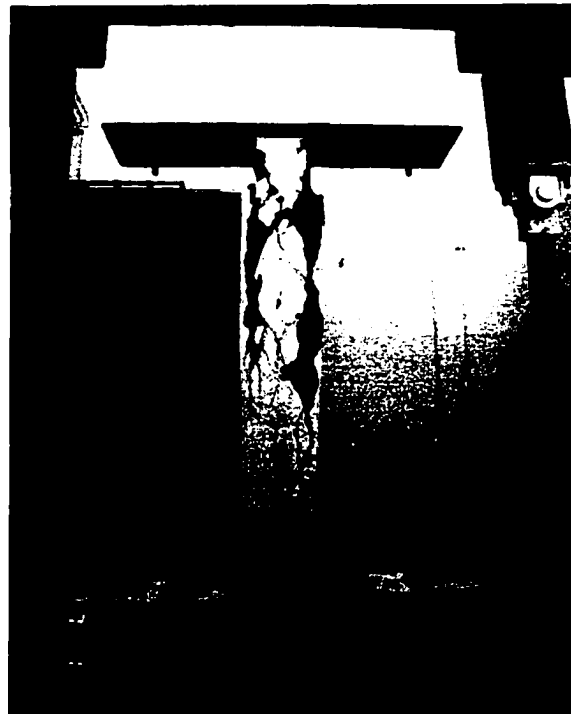


(b) close-up view

Figure 4.3 Concrete spalling over the longitudinal reinforcement at $3\Delta_y$ (Specimen-1)



(a) with instrumentation in place



(b) instrumentation frame removed

Figure 4.4 Damage at a) first cycle to $4\Delta_y$, and b) end of $4\Delta_y$ cycles (Specimen-1)



(a) Overall view



(b) Close-up view

Figure 4.5 Column damage at the end of the test (at $5\Delta_y$ displacement, Specimen-1)

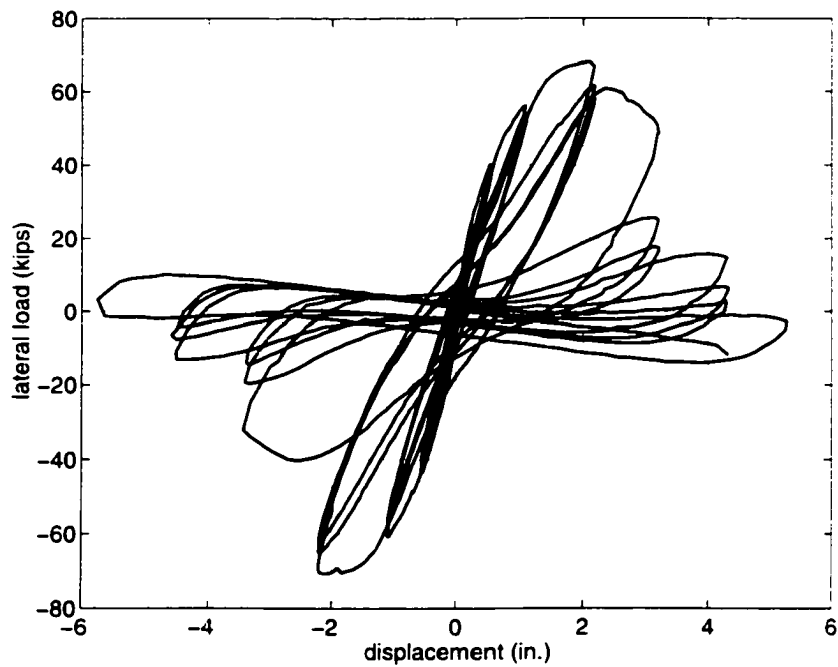


Figure 4.6 Lateral load-displacement relation for Specimen-1

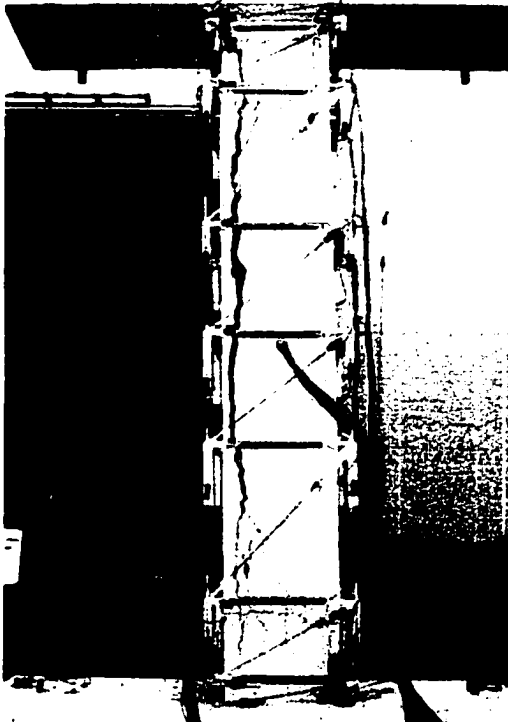


(a) top of column

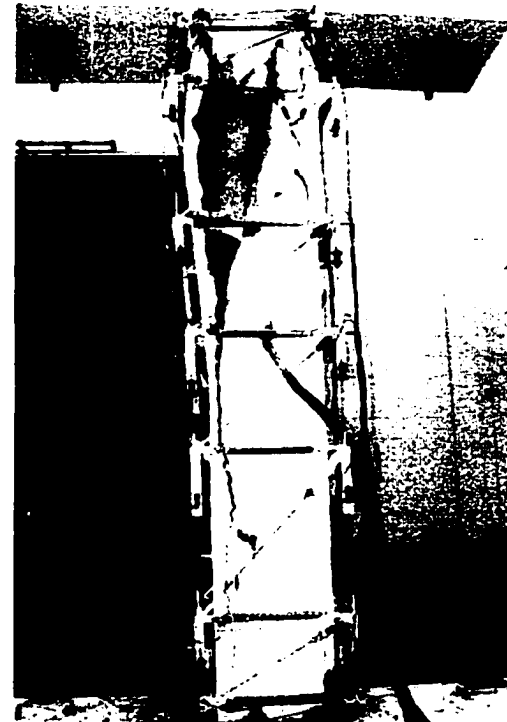


(b) bottom of column

Figure 4.7 Concrete spalling in the compression zones (Specimen-2)



(a) large vertical cracks just before failure



(b) inclined and vertical cracks at failure

Figure 4.8 Damage progress in Specimen-2 at $2\Delta_y$

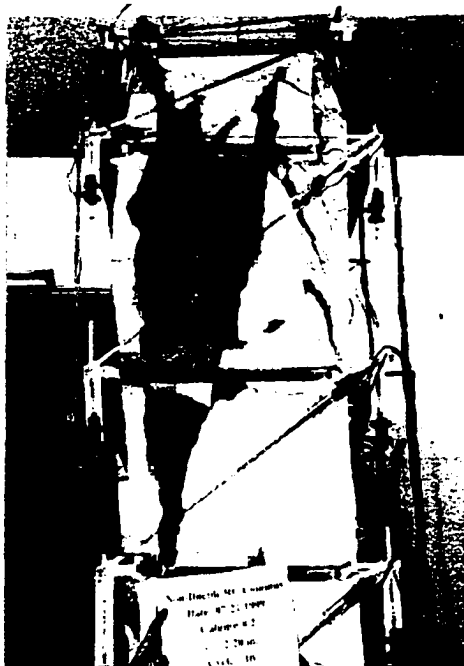


(a) overall view

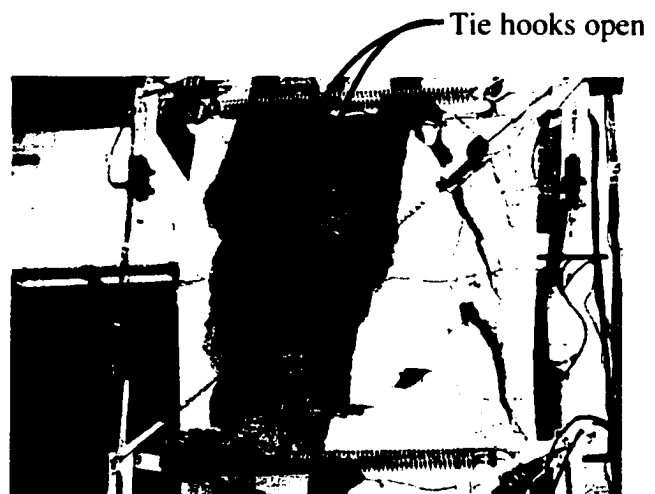


(b) close-up view and buckled bar

Figure 4.9 Specimen-2 after failure (south face)



(a) upper half of Specimen-2



(b) close-up view

Figure 4.10 Crack plane, buckled bars, and open column ties in Specimen-2 (north face)



(a) north face



(b) north and east faces

Figure 4.11 Specimen-2 after instrumentation frame removed



(a) south-east



(b) north-east



(c) south-west



(d) north

Figure 4.12 Different views of failure plane (Specimen-2)

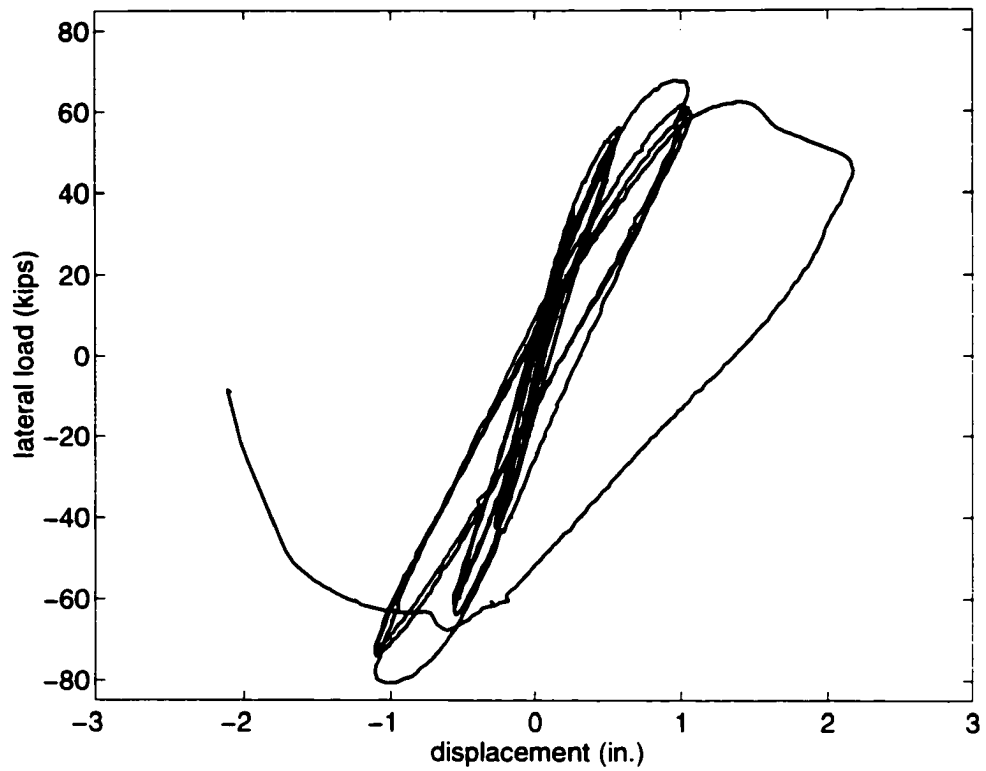


Figure 4.13 Lateral load-displacement relation for Specimen-2

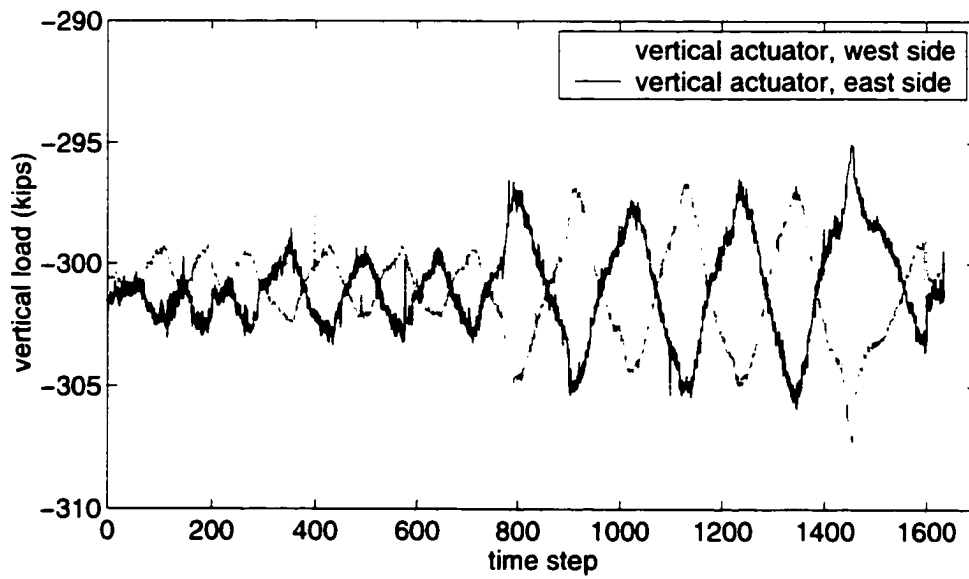
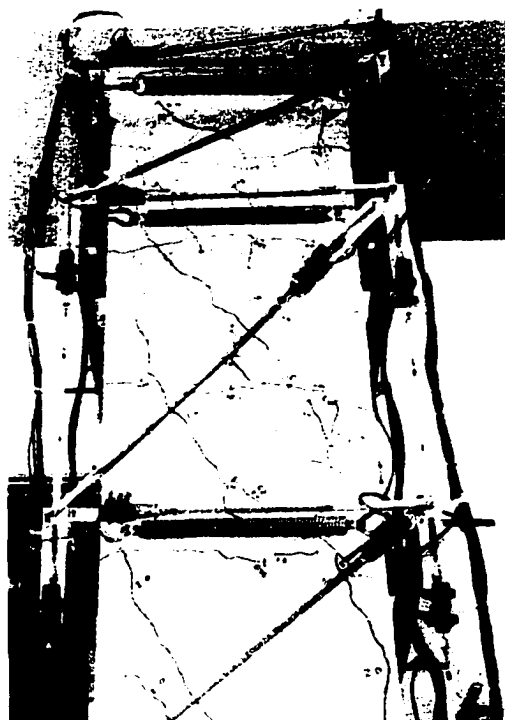


Figure 4.14 Axial loads applied by the two vertical actuators (Specimen-2)



(a) overall view

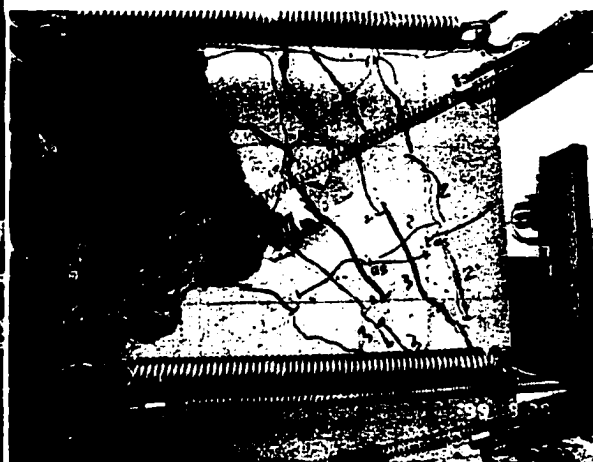


(b) upper half of column

Figure 4.15 Specimen-3; crack pattern at $2\Delta_y$

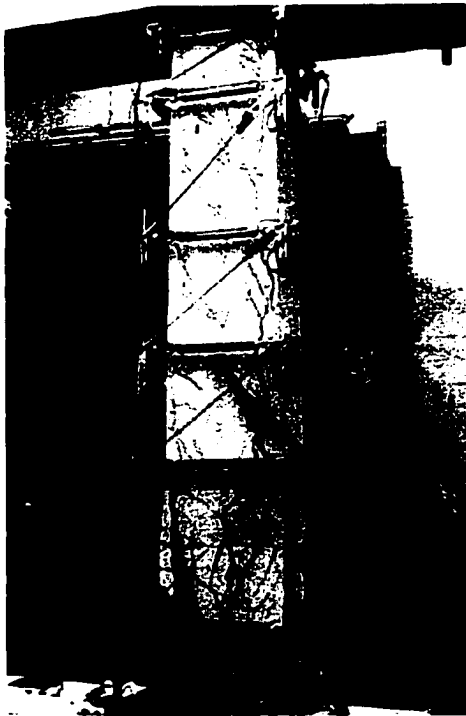


(a) concrete crushing



(b) longitudinal bar buckling

Figure 4.16 Specimen-3 at $3\Delta_y$; flexural compression zone



(a) overall view



(b) close-up view; open column ties

Figure 4.17 Specimen-3 at 34%;

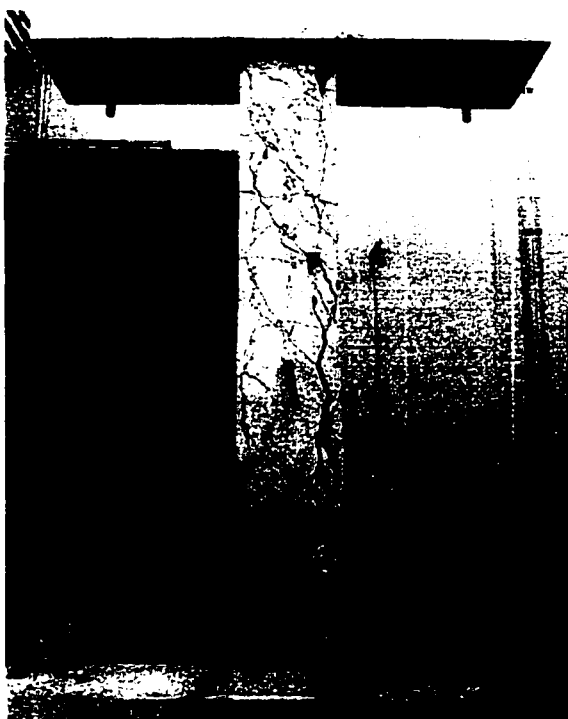


(a) north face



(b) south face

Figure 4.18 Damage pattern at failure (Specimen-3)



(a) overall view (north face)



(b) upper portion of column(north-west)

Figure 4.19 Specimen-3 after loss of lateral load carrying capacity



(a) south-west view



(b) south-east view

Figure 4.20 Specimen-3 after loss of axial load carrying capacity

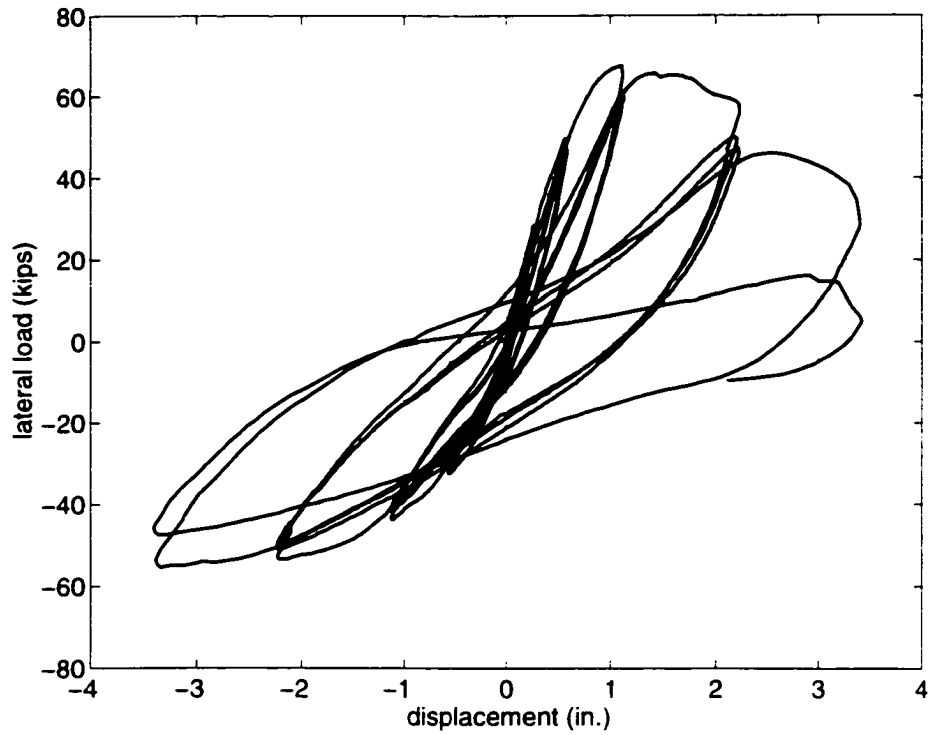


Figure 4.21 Lateral load-displacement relation for Specimen-3

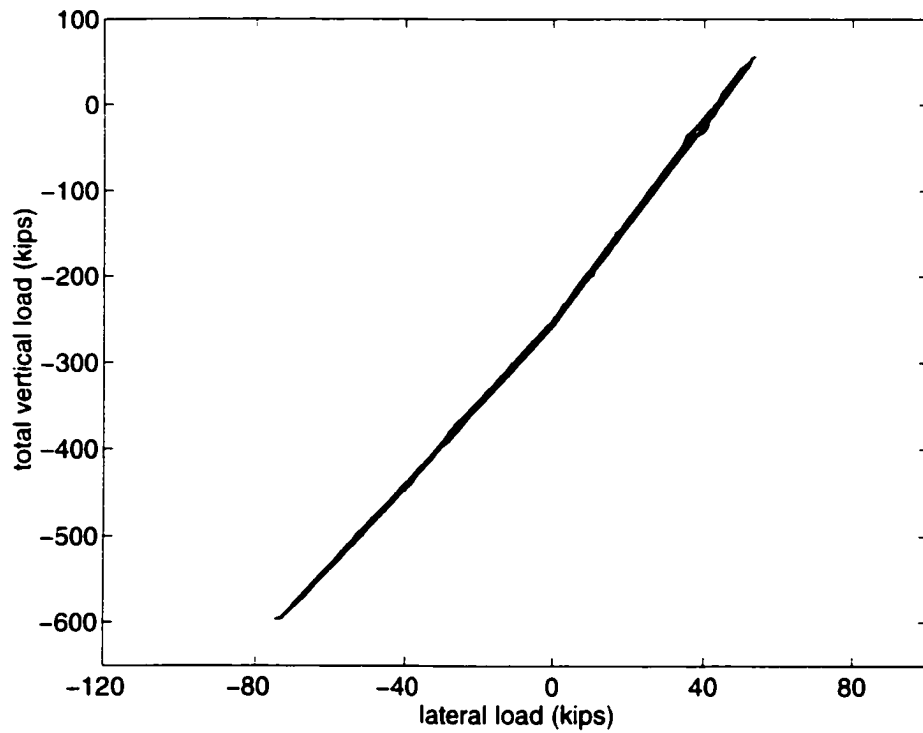
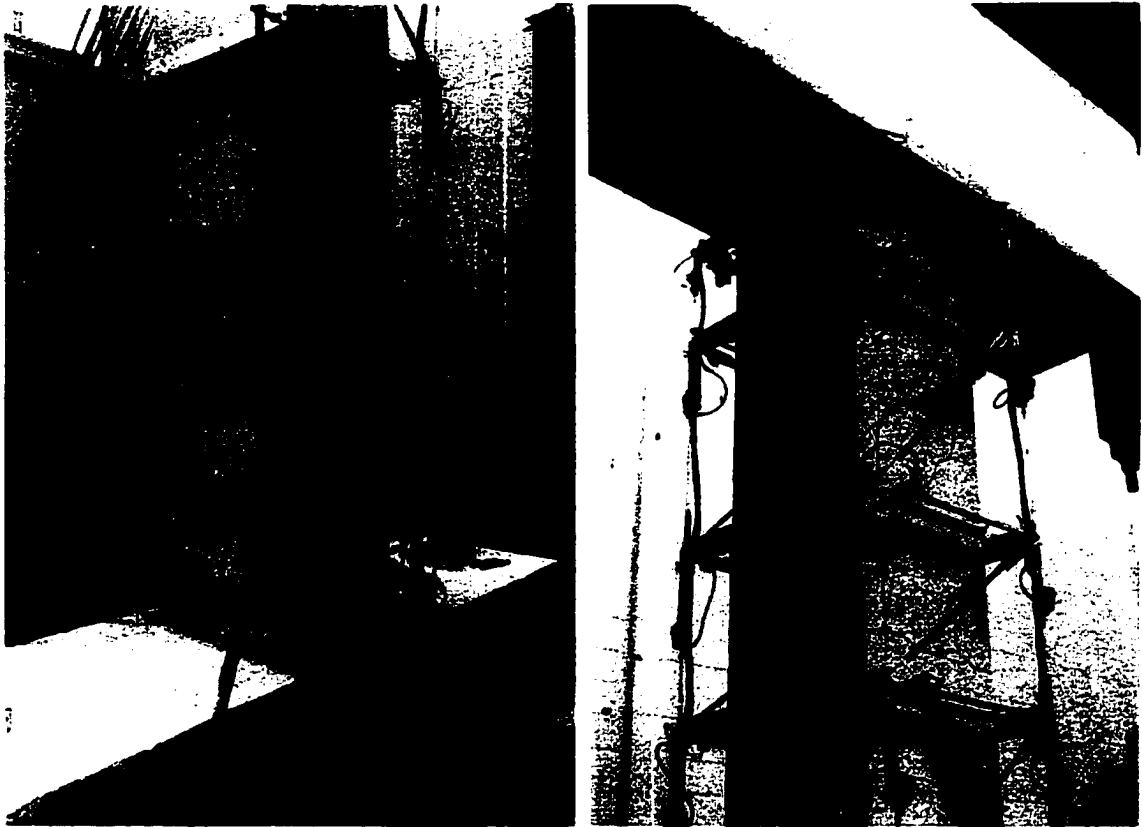


Figure 4.22 Recorded axial-lateral load relations for Specimen-3



(a) bottom half

(b) top half

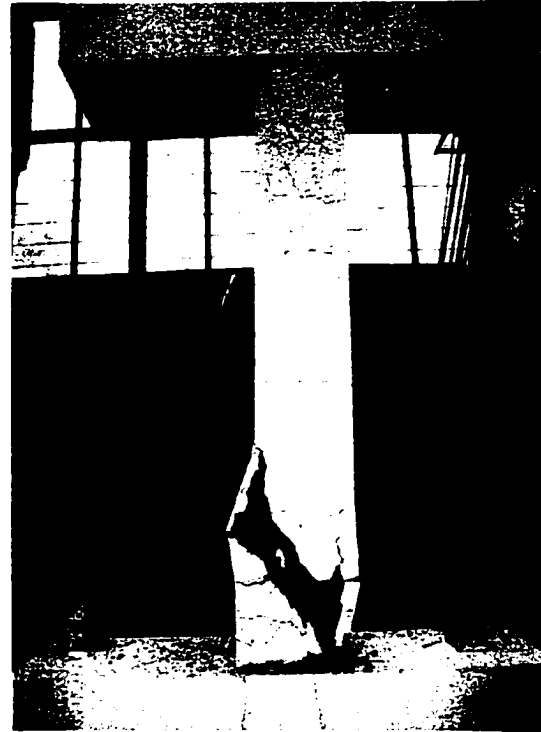
Figure 4.23 Crack pattern of Specimen-4 during the first cycle to yield displacement



Figure 4.24 Damage progress under monotonic loading (Specimen-4)



(a) north face



(b) south face

Figure 4.25 Specimen-4 after loss of lateral-load-carrying capacity



(a) before



(b) after

Figure 4.26 Loss of axial-load-carrying capacity in Specimen-4

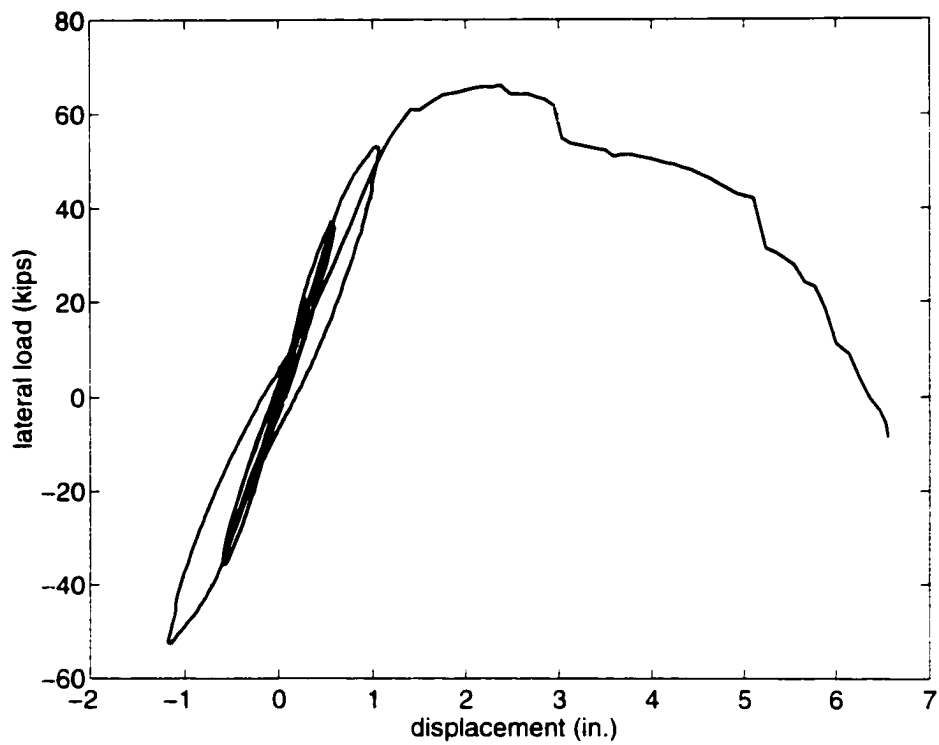


Figure 4.27 Lateral load-displacement relation for Specimen-4

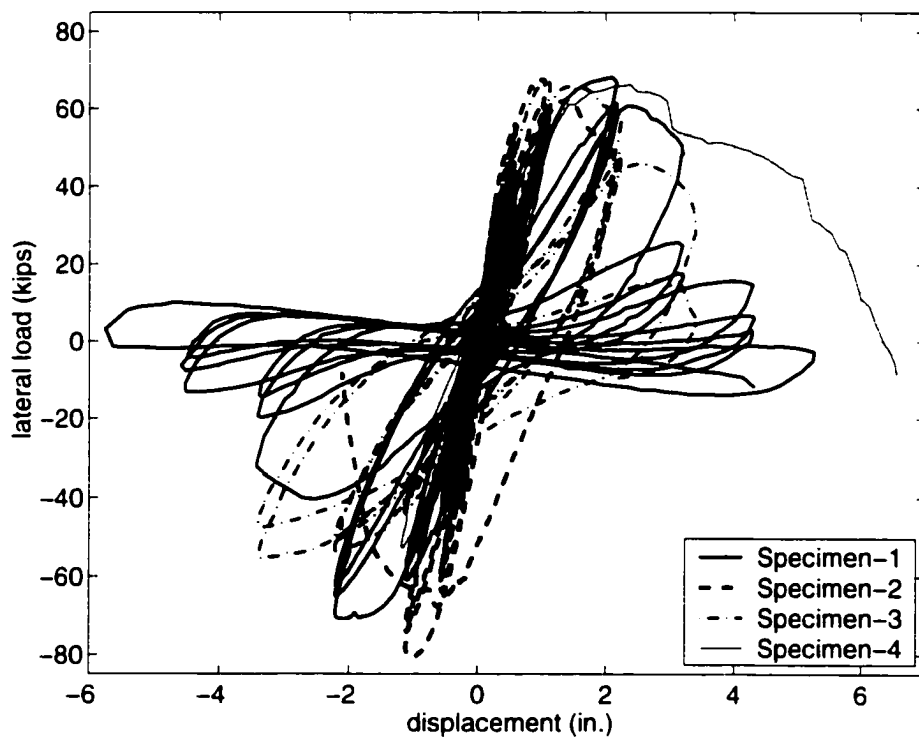


Figure 4.28 Lateral load-displacement relations for all specimens

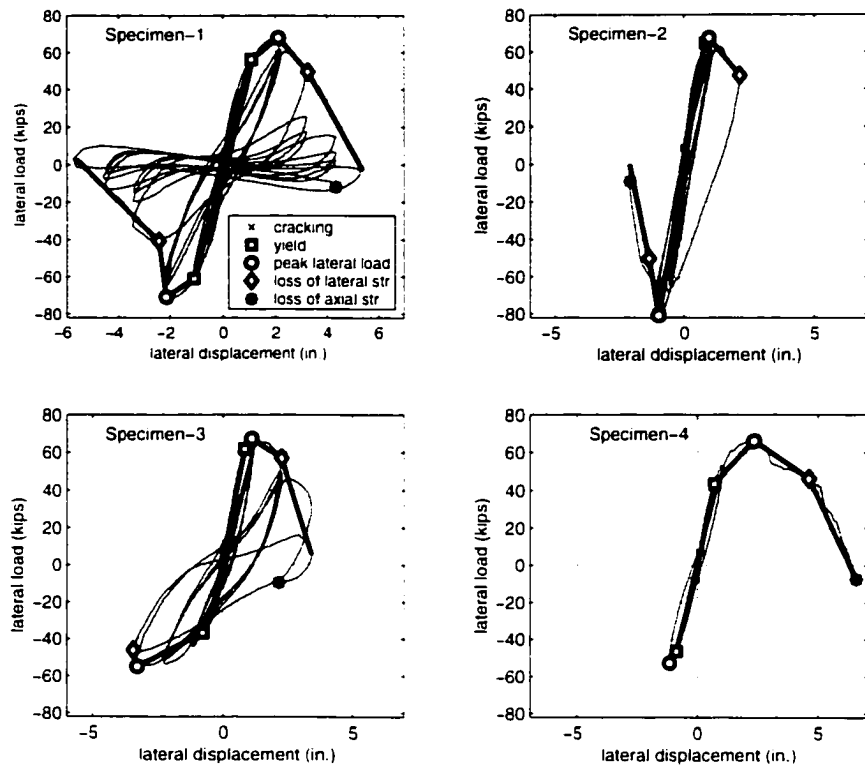


Figure 4.29 Lateral load-displacement relations with limit state envelopes

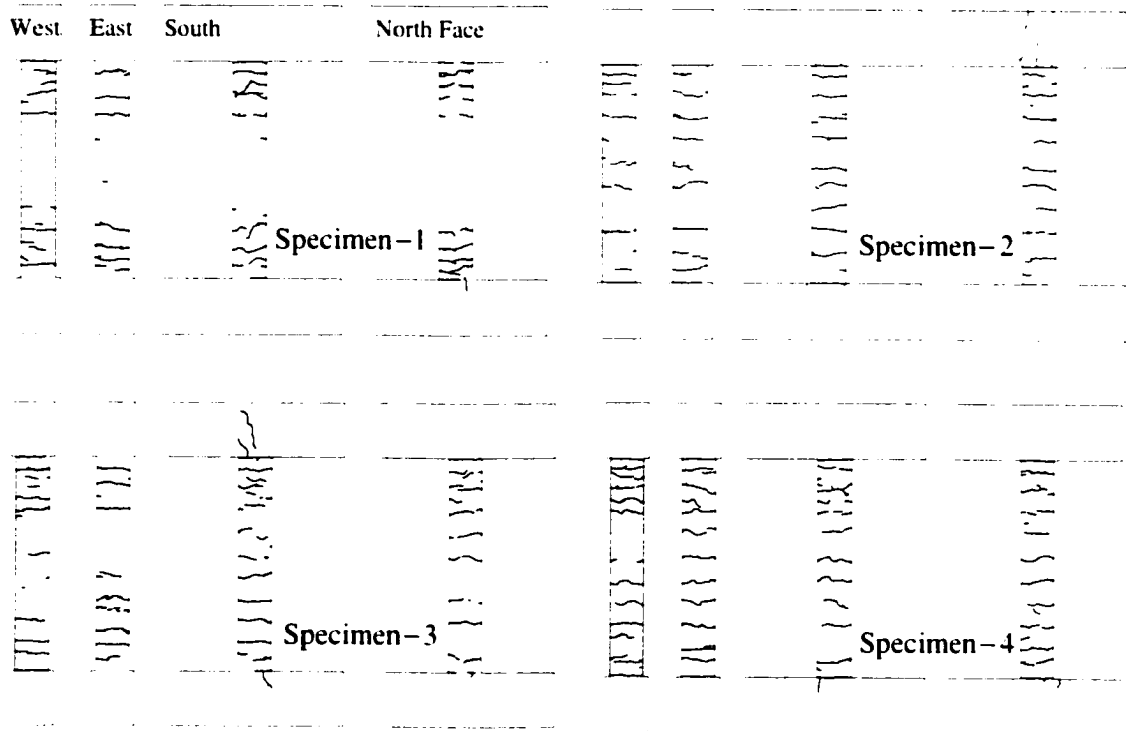


Figure 4.30 Crack pattern at 0.55 in. ($0.5\Delta_v$) lateral displacement

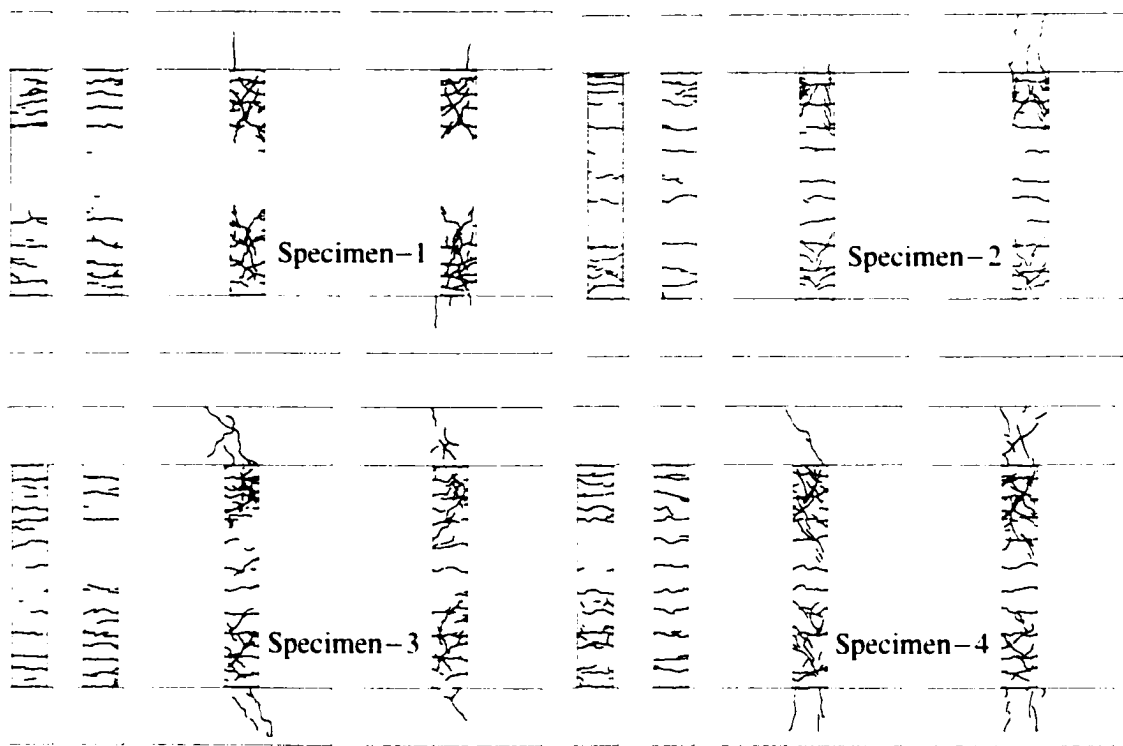


Figure 4.31 Crack pattern at 1.10 in. ($1\Delta_y$) lateral displacement

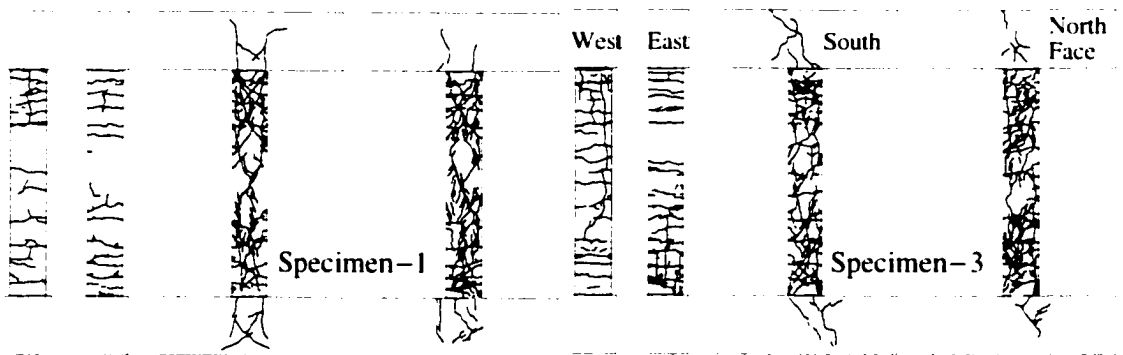


Figure 4.32 Crack pattern at 3.30 in. ($3\Delta_y$) lateral displacement

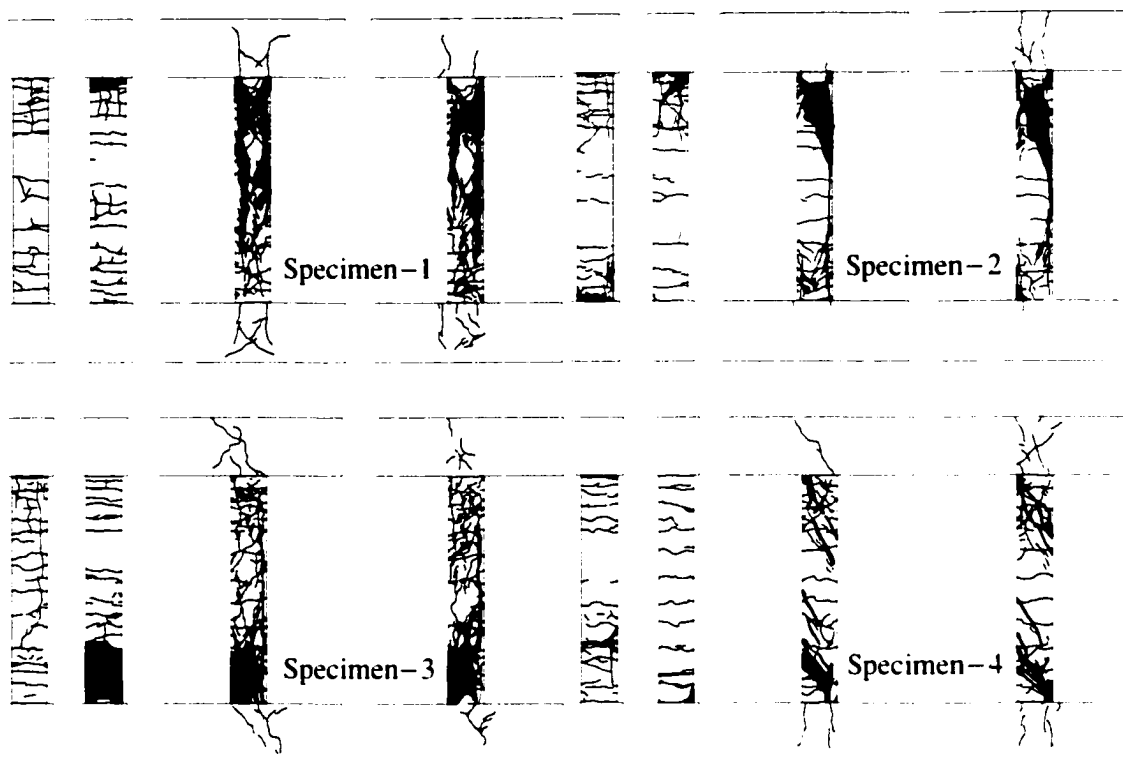


Figure 4.33 Crack pattern at the end of tests

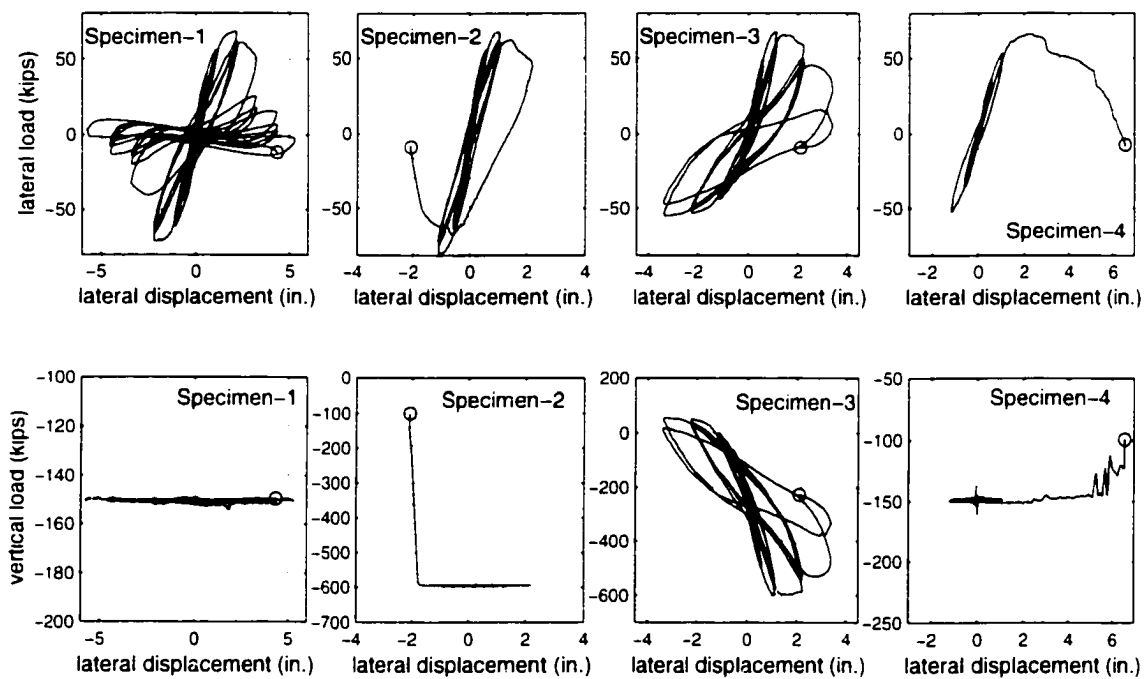


Figure 4.34 Relations among lateral load, vertical load, and lateral displacement

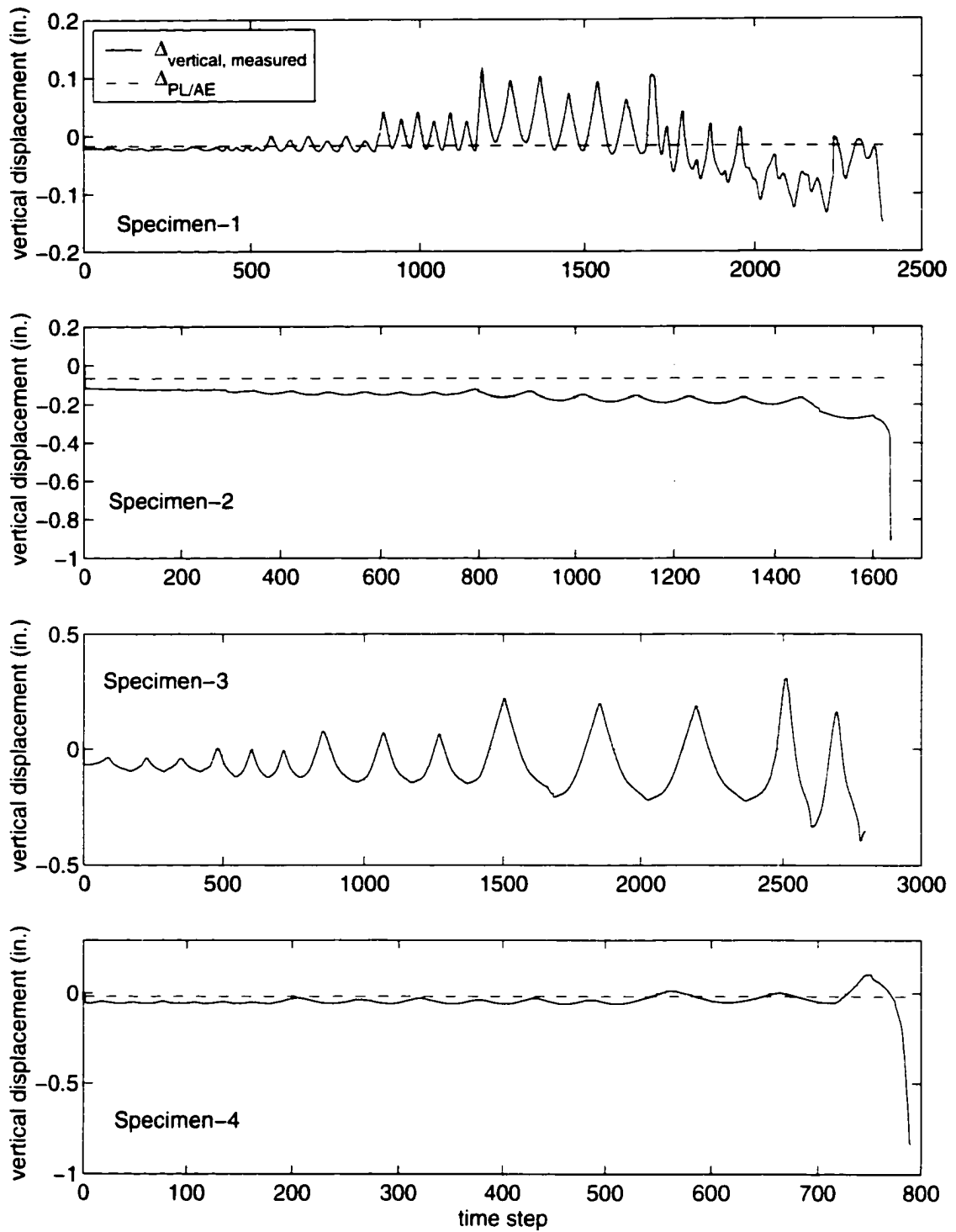


Figure 4.35 Average measured vertical displacements

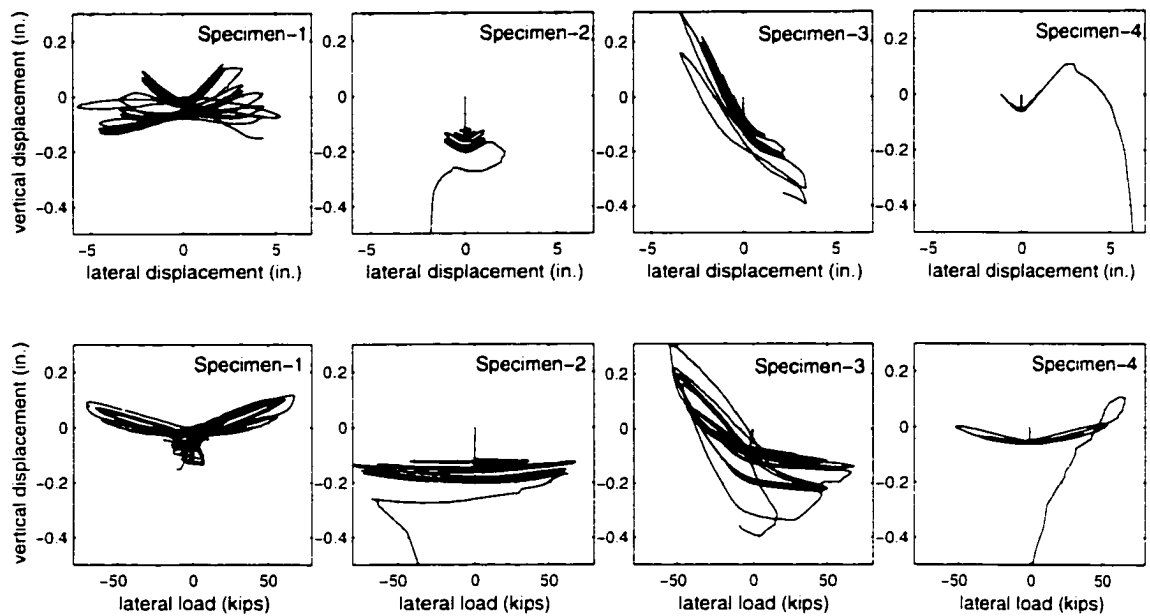


Figure 4.36 Relations among vertical displacement, lateral displacement and lateral load

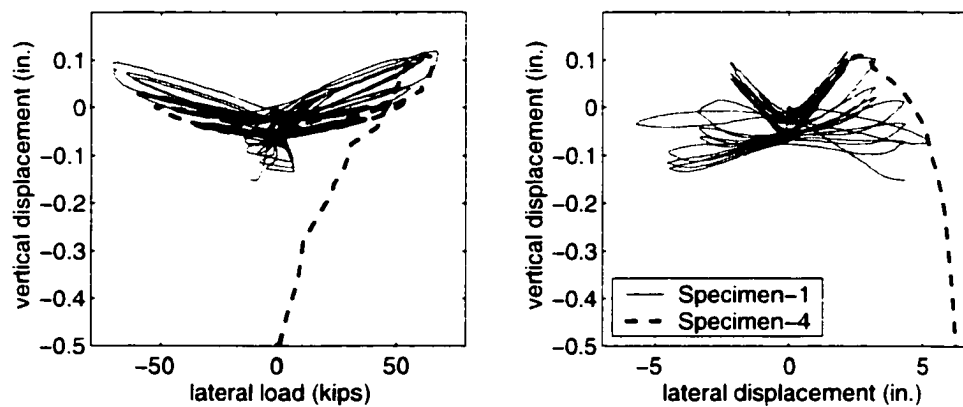
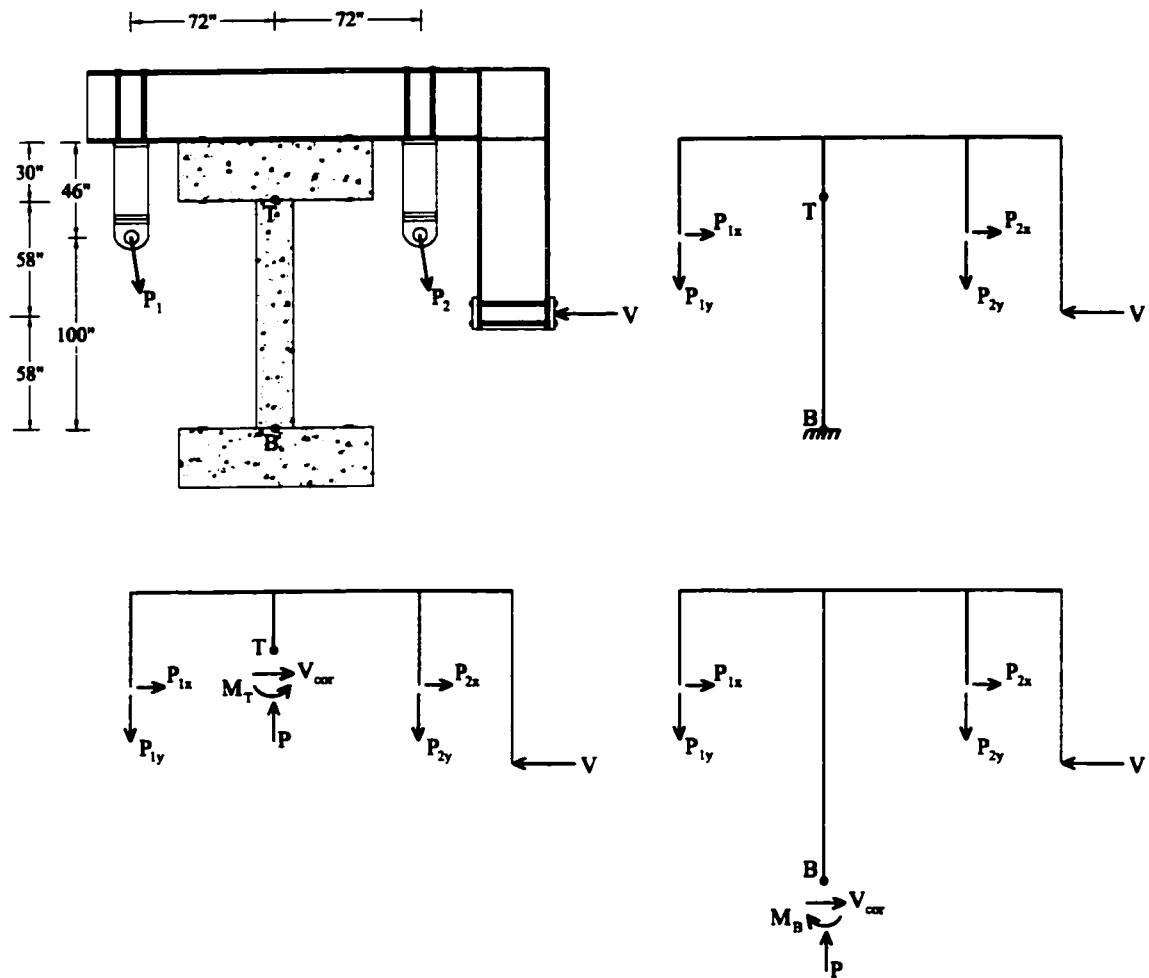


Figure 4.37 Comparison of vertical displacement, lateral load and lateral displacement in Specimen-1 and Specimen-4.



$$P_{1x} = P_1 * \frac{J_h}{\sqrt{L_u^2 + J_h^2}} \quad P_{1y} = P_1 * \frac{L_u}{\sqrt{L_u^2 + J_h^2}}$$

$$P_{2x} = P_2 * \frac{J_h}{\sqrt{L_u^2 + J_h^2}} \quad P_{2y} = P_2 * \frac{L_u}{\sqrt{L_u^2 + J_h^2}} \quad L_u = 102"$$

$$M_B = V * 58 + P_{1x} * (72 + J_h) - P_{2x} * (72 - J_h) - (P_{1y} + P_{2y}) * 100$$

$$M_T = V * 58 - (P_{1x} - P_{2x}) * 72 - (P_{1y} + P_{2y}) * 16$$

$$P = P_{1y} + P_{2y} \cong P_1 + P_2$$

$$V_{corr} = V - (P_{1x} + P_{2x})$$

Figure 4.38 Free-body diagrams and calculation of corrected forces

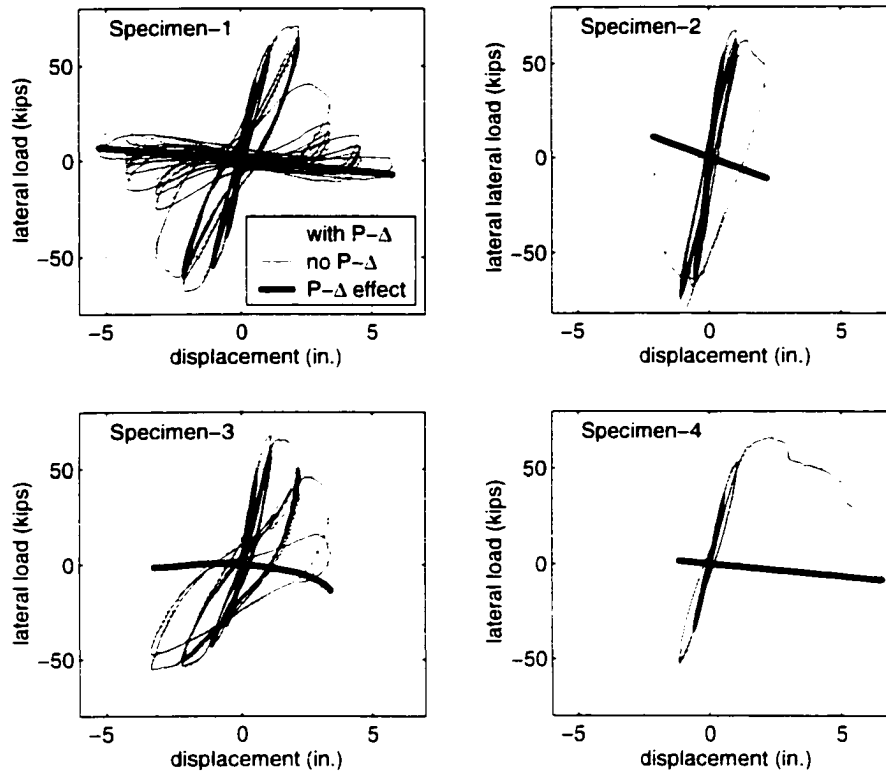


Figure 4.39 Effect of P- Δ on the lateral load-lateral displacement relations

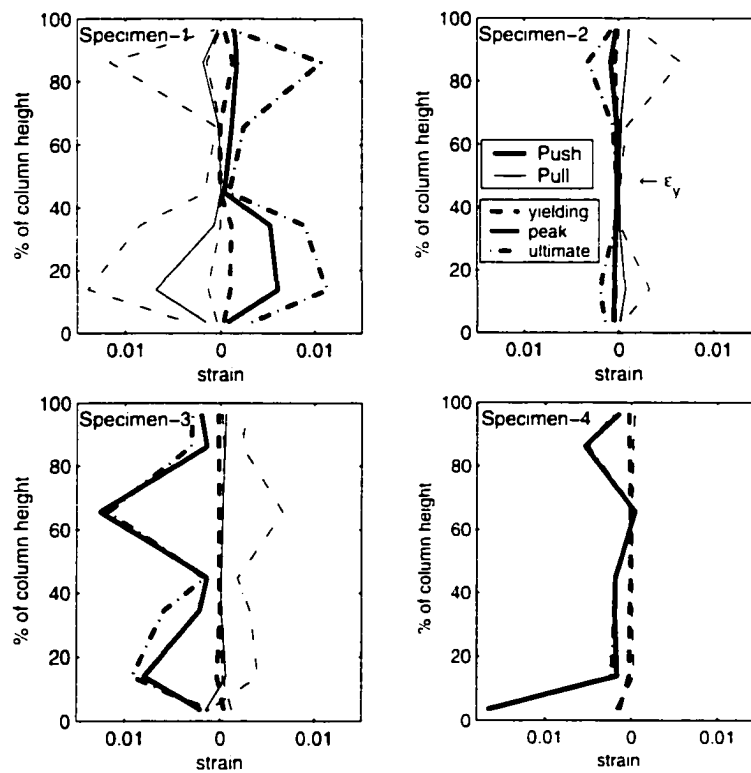


Figure 4.40 Transverse steel strain distribution over the height of specimens

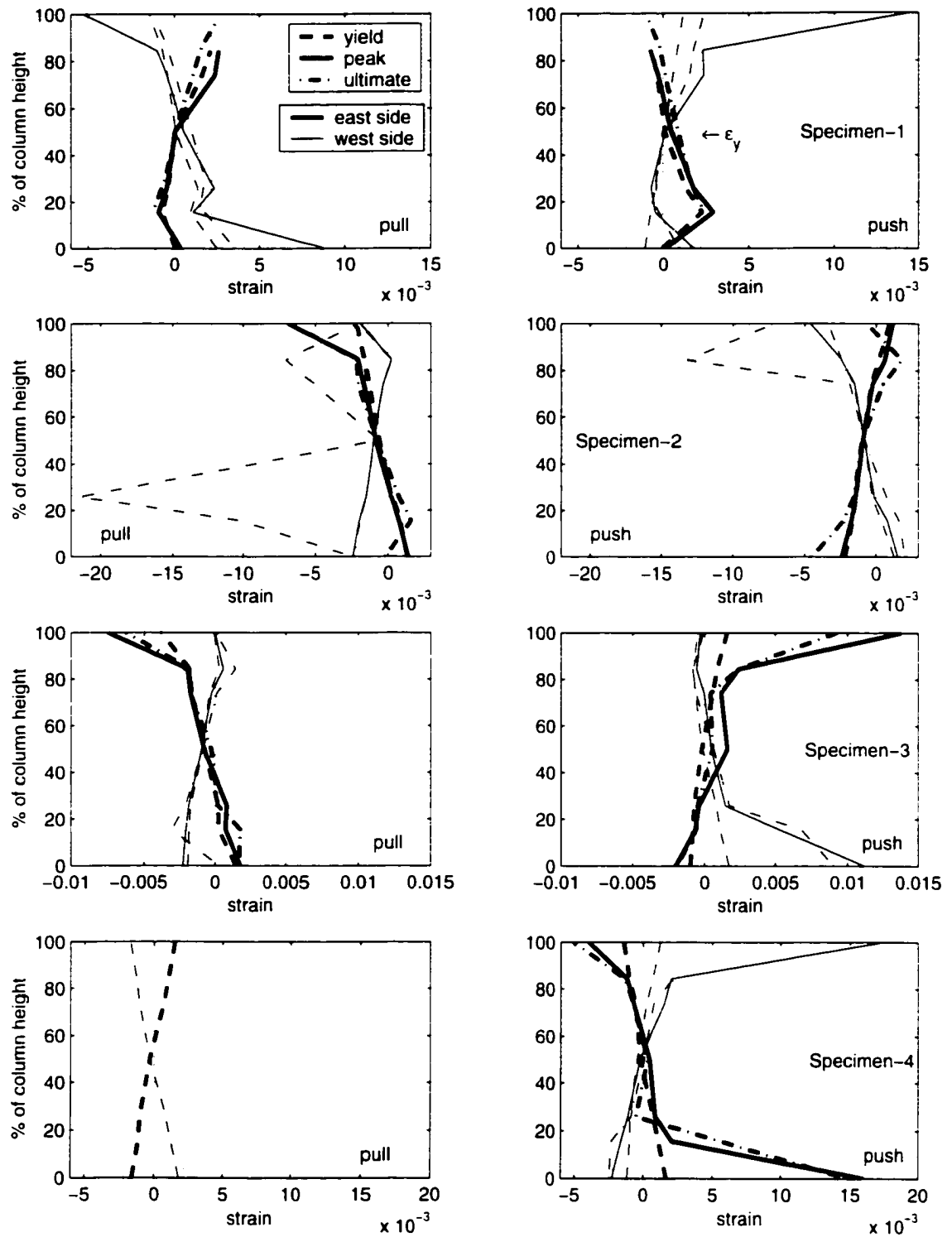


Figure 4.41 Longitudinal steel strain distribution over the height of specimens

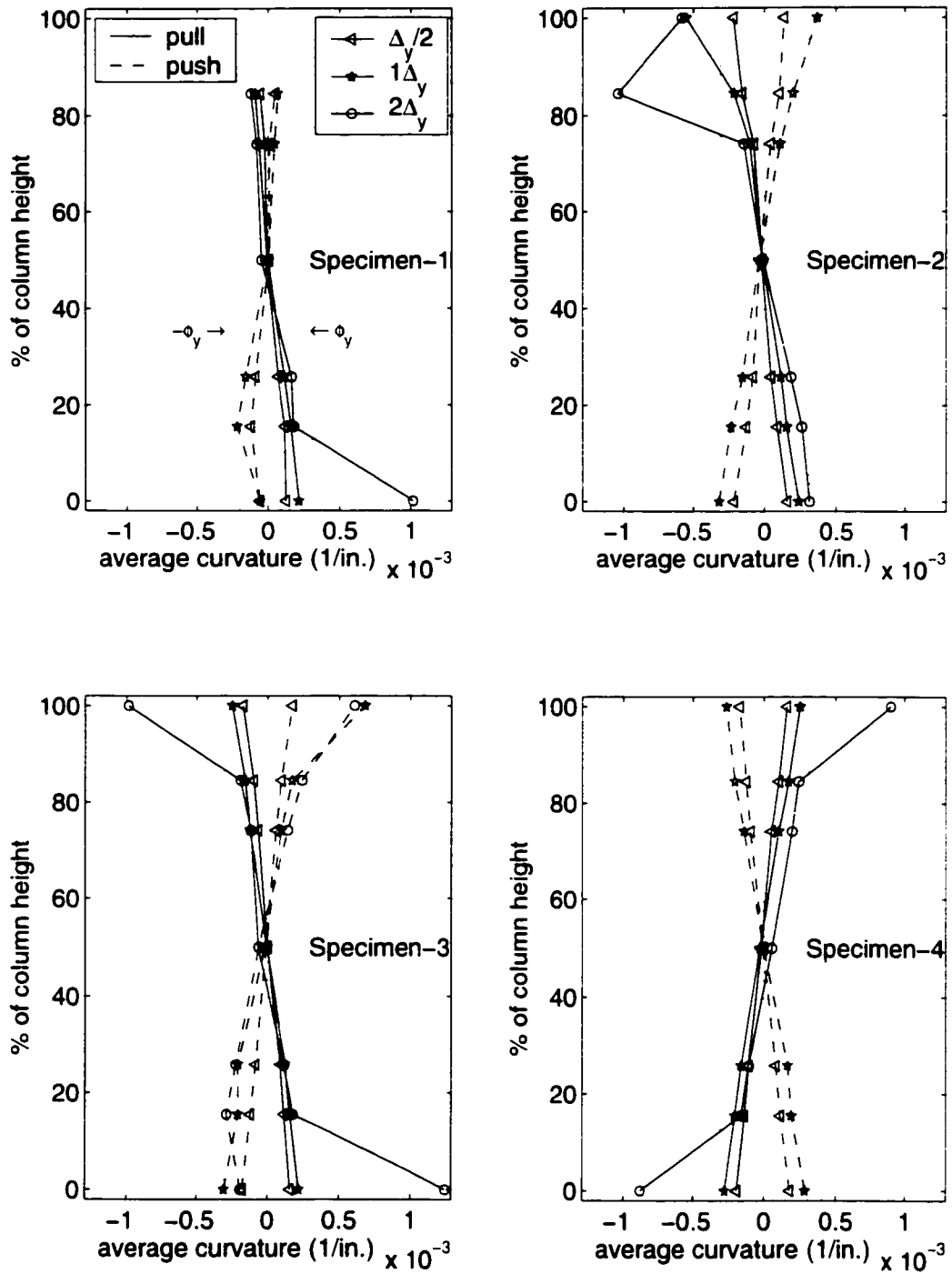


Figure 4.42 Average curvature profiles from longitudinal reinforcement strain measurements

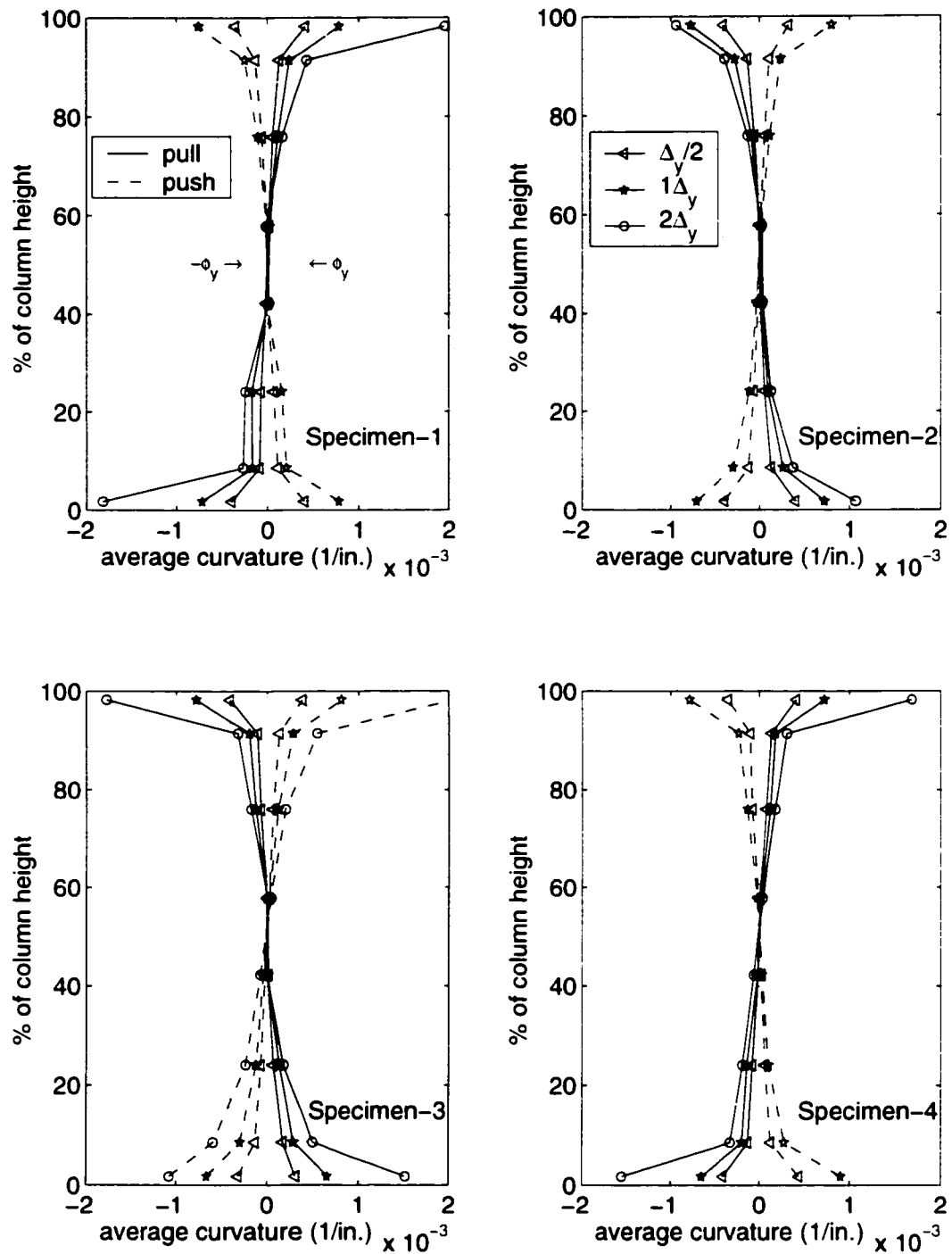


Figure 4.43 Average curvature profiles from displacement potentiometer measurements

5 EVALUATION OF TEST RESULTS AND ANALYTICAL STUDIES

5.1 INTRODUCTION

In this chapter, the displacement components due to flexure and longitudinal bar slip are computed using the local displacements measured on each test specimen. Then, the behavior of each specimen is evaluated based on observed damage and measured displacement components. Analytical models are developed to represent the cross-section moment-curvature response and moment-slip relations for the longitudinal reinforcing bars at the top and bottom beam-column interfaces. The analytical moment-curvature and moment-bar slip displacement relations are compared with the

corresponding measured deformation components. Each component behavior under monotonic lateral load is subsequently idealized by a simple spring for computer modeling. The calculated monotonic lateral load-total displacement relations are compared with the measured response.

5.2 MATERIAL MODELS FOR MOMENT-CURVATURE ANALYSIS

Moment-curvature analysis is carried out using a fiber model of the test columns (UCFyber, 1999). This section presents the uniaxial steel and concrete material models used in the moment-curvature analysis. The longitudinal steel and concrete were modeled using the data from steel coupon tests and concrete cylinder tests shown in Appendix B.

Figure 5.1 compares the stress-strain model for the longitudinal reinforcing bars with the measured stress-strain relations from the steel coupon tests. The elastic modulus of elasticity, E , was 29000 ksi. The flat yield plateau was modeled by a straight line with one percent strain hardening. The reasons behind not using a horizontal line for the yield plateau will be explained in the section describing the bar slip model in this chapter. In the work-hardening range, the steel stress, f_s , was calculated from

$$f_s = f_u - (f_u - f_{sh}) \left[\frac{\epsilon - \epsilon_{sh}}{\epsilon_{su} - \epsilon_{sh}} \right]^6 \quad (5.1)$$

where ϵ_{sh} = strain at the onset of strain hardening, f_{sh} = stress at the onset of strain hardening (that is, $f_{sh} = f_y + (\epsilon_{sh} - \epsilon_y)(0.01E)$), ϵ_{su} = strain at maximum stress, and f_u = maximum stress.

A combination of confined and unconfined concrete models was used to represent the concrete behavior in the analysis. Previous research (e.g., Mander et al. 1988) suggests that the concrete compressive strength increases if sufficient transverse steel is provided. Following the procedure developed by Mander et al. (1988), the confined concrete stress-strain relations are calculated and presented in Figure 5.2. The concrete stress, f'_c , is given by

$$\begin{aligned} f'_c &= \frac{f'_{cc} r (\epsilon / \epsilon_{cc})}{r - 1 + (\epsilon / \epsilon_{cc})^r} \\ \epsilon_{cc} &= \epsilon_{co} \left[1 + 5 \left(\frac{f'_{cc}}{f'_c} - 1 \right) \right] \\ r &= \frac{E_c}{E_c - E_{sec}} \end{aligned} \quad (5.2)$$

where f'_{cc} = peak confined concrete stress calculated according to Mander et al. (1988), ϵ_{cc} = strain at peak stress for confined concrete, ϵ_{co} = strain at peak stress in unconfined concrete, f'_c = modulus of elasticity of concrete ($= 57000 \sqrt{f'_c}$, in psi), and E_{sec} = a secant modulus of concrete ($= f'_c / \epsilon_{co}$). Figure 5.2 shows that, for strains smaller than ϵ_{co} ($= 0.002$), the confined concrete model compares very well with the measured stress-strain relations from unconfined concrete cylinder tests.

In this research, after the peak confined stress, the concrete is assumed to unload more rapidly than suggested by the Mander et al. model because the transverse reinforcement spacing is relatively large and unable to restrain the crushed concrete core. For the post-peak behavior, the unconfined concrete model with a descending straight-line stress-strain relationship developed by Roy and Sozen (1964) was used (Figure 5.2). According to Roy and Sozen, the strain, ϵ_{50u} at fifty percent of the peak concrete stress, f'_c , is calculated from

$$\epsilon_{50u} = \frac{3 + 0.002f'_c}{f'_c - 1000} \quad (\text{in psi units}) \quad (5.3)$$

According to Roy and Sozen, the residual concrete stress capacity is represented by twenty percent of the peak concrete stress and the corresponding strain is calculated from linear extrapolation of the straight line between the peak stress and the fifty percent of the peak stress. As illustrated in Figure 5.2, in the proposed model, the unloading slope is the same as the one calculated from the unconfined concrete model proposed by Roy and Sozen.

5.3 MOMENT-CURVATURE ANALYSIS

The moment-curvature response under monotonic loading was calculated by using the Bernoulli assumption that plane sections remain plane under imposed axial and moment actions. The cross section was discretized into multiple fibers. A linear strain distribution was imposed and the stress on each fiber was based on the uniaxial stress-strain relations for the material of that fiber, with the strain defined at the centroid of that fiber. The

strain distribution was iterated until equilibrium was achieved under imposed axial and moment actions. The solution was implemented using the computer program UCFyber (UCFyber, 1999).

Figure 5.3 shows the calculated moment-curvature relations under a series of constant axial loads ranging from 600 kips in compression to 50 kips in tension. In the analyses, under compressive axial loads of 500 kips and higher, the longitudinal steel did not attain its yield strength in tension when the peak moment strength was reached. Except for under these very high axial loads, as expected, the peak moment strength increased with increasing axial load. Figure 5.3 shows that with increasing axial load the initial stiffness also increases, and the maximum curvature capacity and curvature ductility decrease.

Using the prescribed axial load-lateral load relationship (Equation 3.2) and the equilibrium equations shown in Figure 4.38, the axial load-moment relations were calculated for Specimen-3 with varying axial load. For a series of moments under constant axial loads varying from 600 kips in compression to 50 kips in tension, the corresponding curvatures were obtained from Figure 5.3. The calculated moment-curvature relations under varying axial load are shown in Figure 5.4.

5.4 FLEXURAL BEHAVIOR OF TEST SPECIMENS

Curvature calculations from strain gage and displacement potentiometer measurements and curvature profiles were presented in Section 4.11 for each specimen. Total curvature calculations from local vertical displacements measured by displacement potentiometers

are demonstrated in Figure 5.5. As illustrated in the figure, total curvatures measured over the height of the column include both flexural curvatures and the deformations resulted from longitudinal bar slip near the ends of the column. In order to eliminate the effect of bar slip deformations on the total curvatures, the flexural curvatures are assumed to vary linearly near the column supports. As illustrated in Figure 5.5, the flexural curvature near the bottom of the column, $\phi_{1,flexure}$, was calculated from linear extrapolation of the two nearest total measured curvatures, ϕ_2 and ϕ_3 . Figure 5.6 shows the extrapolated flexure and total measured curvature profiles for each specimen at first yielding in the longitudinal reinforcing bars. The extrapolated curvatures at the top and bottom are comparable with the theoretical yield curvatures, which are shown as vertical dotted lines in the plots.

Figure 5.7 shows the measured relations between the measured moment and curvature, the latter being obtained from extrapolation of total curvatures at the top and bottom of each column as described previously. The measured moment-curvature relations compare reasonably well with the calculated monotonic moment-curvature relations from fiber section analysis (Figure 5.3 and Figure 5.4). As an example of showing the flexural deformation distribution over the column height, the measured cyclic moment-curvature relations over the height of Specimen-1 are shown in Figure 5.8. The figure also compares the calculated monotonic moment-curvature analysis results with the cyclic response measured at eight sections along the column height. It is apparent that inelastic flexural curvature spreads away from the critical section at the column end even though moments are less than moment capacity at those sections.

The lateral displacement of a column due to flexure can be calculated by integrating the flexural curvatures along the height of the member, as in

$$\Delta_{flexure} = \int_0^L \phi \, x \, dx \quad (a)$$

$$\Delta_{flexure} = \sum_1^i \theta_i \, d_i = \sum_1^i \frac{\delta_{li} - \delta_{ri}}{b} \, d_i \quad (b) \quad (5.4)$$

where θ_i = average rotation angle for the i^{th} segment, d_i = vertical distance from the center of the segment to top of the column (see illustration in Figure 5.5), and δ_{li} and δ_{ri} are the relative vertical displacements measured by the displacement potentiometers on the left and right side of the segment, respectively. Equation 5.4b is the simplified form of Equation 5.4.a based on the curvature definitions illustrated in Figure 5.5.

Using Equation 5.4, the flexural curvatures were integrated over the height of each specimen. Similarly, curvatures from fiber section analysis were integrated to calculate total monotonic flexural displacements. The cyclic measured and monotonic calculated lateral load-flexural displacement relations are shown in Figure 5.9 through Figure 5.12 for each test specimen. Agreement between the calculated and measured responses is relatively good. Especially, the elastic lateral stiffness and the peak lateral strength estimated from the analysis compare very well with the test results.

The measured moment-strain relations at the top and bottom of Specimen-1, by Strain Gage-C7 and -C1 (see Figure 3.22 and Appendix E) are shown in Figure 5.13 and Figure 5.14, respectively. The relation between the calculated section moment and strain in the tensile longitudinal bar can be monitored during the moment-curvature analysis.

Comparison of calculated and measured longitudinal bar strains in the figures shows that, in the elastic range and during the first yield cycle, strains can be estimated reasonably well based on moment-curvature analysis.

5.5 BOND-SLIP MODELS

5.5.1 Background

The experiments show that relatively wide cracks develop at the interface between the end beams and column under lateral loads. Due to penetration of axial strains along the tensile reinforcement inside the joint, accentuated by bond deterioration between the steel and concrete, the extension and slip of reinforcing bar at the interface can be significant. The elongation and slip of the tensile reinforcement at the interface result in additional fixed-end rotations that are not included in the flexural analysis. These additional rotations can increase the total member lateral displacements considerably. This section presents the member end rotations and resulting member displacements due to bar slip for the four column specimens tested in this investigation. A bond stress-slip model is proposed to characterize the bond-slip behavior and corresponding member deformations under monotonic lateral loading. The proposed model is compared with other analytical models as well as the measured cyclic test results.

Numerous researchers have investigated the anchorage behavior of reinforcing bars experimentally, and a number of analytical bond-slip models have been developed over the years. The widely used bond stress-slip relationships by Eligehausen et al. (1983) and Ciampi et al. (1982) were based on an experimental program at the University of

California, Berkeley. Some aspects of the model proposed by Eligehausen et al. (1983) are shown in Figure 5.15. Within last twenty years, various refined computer models were proposed to implement this model, and a few similar models were developed to represent bond-slip deformations in reinforced concrete members under monotonic and cyclic loads (Morita and Kaku 1984, Filippou et al. 1986, Hawkins et al. 1987, Pochanart and Harmon 1989, and Soroushian and Choi 1989). A detailed analysis of bar slip under monotonic loading and a description of available analytical procedures for force-slip deformation relationships are summarized by Alsiwat and Saatcioglu (1992).

By assuming an average uniform bond stress, u_b , along the development length, l_d , of a reinforcing bar, the force in the bar, F_{bar} , can be defined by equilibrium as

$$F_{bar} = f_s A_s = u_b p_b l_d \quad (5.5)$$

Substituting the bar perimeter, $p_b = \pi d_b$ and $A_s = \pi d_b^2 / 4$ into Equation 5.5 yields

$$l_d = \frac{f_s d_b}{4 u_b} \quad (5.6)$$

Using a uniform bond stress along the development length, Otani and Sozen (1972) modeled deformations at the ends of a reinforced concrete member due to bar slip. The assumed uniform bond stress along the embedded tensile reinforcing bar was $6.5 \sqrt{f'_c}$ (in psi units). Otani and Sozen assumed that the stress in the bar decreases linearly with the distance and becomes zero at the distance of the development length. Then, the slip or the elongation of the reinforcing bar over the development length is given by

$$slip = \frac{\epsilon_s l_d}{2} = \frac{f_s l_d}{2 E_s} \quad (5.7)$$

If the embedded length of the bar at the end of a member is longer than the development length, by substituting Equation 5.6 for l_d in Equation 5.7, the slip can be rewritten as

$$slip = \frac{f_s^2 d_b}{8 E_s u_h} = \frac{f_s^2 d_b}{52 E_s \sqrt{f'_c}} \quad (5.8)$$

Otani and Sozen estimated the section rotation due to bar slip, θ_{slip} as

$$\theta_{slip} = \frac{slip}{d - d'} \quad (5.9)$$

where $(d - d')$ is the distance between the two longitudinal end bars in compression and tension. Otani and Sozen related the slip rotation to the bending moment by assuming a linear relationship between a moment and a stress in the tensile reinforcement ($f_s/f_y = M/M_y$, where M = bending moment, and M_y = yield moment at the end of the member).

Thus, substituting Equation 5.8 into 5.9 gives

$$\theta_{slip} = \frac{M^2 f_s^2 d_b}{8 E_s u_h M_y^2 (d-d')} = \frac{M^2 f_s^2 d_b}{52 E_s \sqrt{f'_c} M_y^2 (d-d')} \quad (5.10)$$

This model is expected to be more representative of the elastic range as it uses the elastic modulus of elasticity for steel, and approximates the stress in the bar from the section bending moment using a linear relationship.

Experimental test results from Ismail and Jirsa (1972), Viwathanatepa et al. (1979), Saatcioglu et al. (1992), and Lehman and Moehle (2000) demonstrate that at the beam-column interface, the strains in the reinforcing bar can be much larger than the yield strain causing columns to experience significant fixed-end rotations due to bar slip. Based on experimental results from well-confined concrete column tests, Lehman and Moehle (2000) proposed the bi-uniform bond stress-slip model shown in Figure 5.16. In this model, for slip values less than the slip corresponding to the yield strain in the bar, the uniform bond stress is approximated as $12\sqrt{f'_c}$. For slip values exceeding the slip at yield, the bond stress capacity is $6\sqrt{f'_c}$ (in units of psi).

Alsiwat and Saatcioglu (1992) proposed an analytical procedure for the force-deformation relationship of a reinforcing bar embedded in concrete. The numerical procedure included calculation of the displacement due to slippage of the bar with nonlinear strain distribution (Figure 5.17). According to this model, four regions are developed along a reinforcing bar in tension, namely, elastic region with length L_e ; yield plateau region with length L_{yp} ; strain-hardening region with length L_{sh} ; and pullout-cone region with length L_{pc} . In the model, an elastic uniform bond stress, u_e , is assumed along the length of the bar except for the pullout-cone region. An additional frictional uniform bond stress, u_f , is assumed in the yield plateau and strain-hardening regions.

In the elastic region with strains smaller than the yield strain, the elastic development length, L_e , is calculated from Equation 5.11, in which the uniform elastic bond stress, u_e , is calculated from the expressions proposed by ACI Committee 408 (1979).

$$L_e = \frac{f_y d_b}{4 u_e} \quad \text{where} \quad u_e = \frac{f_y d_b}{4 l_{d,ACI}} \quad (5.11)$$

The basic development length, $l_{d,ACI}$, proposed by ACI Committee 408 (1979) is

$$l_{d,ACI} = \frac{5500 A_s}{K \sqrt{f'_c}} \frac{f_y}{60000} \quad (\text{psi}) \quad (5.12)$$

where K is a function of concrete cover and transverse reinforcement. K is the smaller of $3d_b$ and two other parameters defined in the ACI Committee 408 (1979) report. As in many practical applications, Alsiwat and Saatcioglu assumed a value of K equal to $3d_b$. Thus, with $A_s = \pi d_b^2/4$, Equation 5.12 simplifies to $l_{d,ACI} = f_y d_b / (41.7 \sqrt{f'_c})$. Consequently, from Equation 5.11, the elastic uniform bond stress u_e would be equal to approximately $10.4 \sqrt{f'_c}$, which is comparable with the elastic uniform bond stress of $12 \sqrt{f'_c}$ proposed by Lehman and Moehle (2000).

In the model proposed by Alsiwat and Saatcioglu (1992), the development length in the yield plateau region is given by

$$L_{vp} = \frac{(f_y - f_t) d_b}{4 u_f} \quad (5.13)$$

If a steel material model with a flat yield plateau is used, this expression yields zero length and a sudden jump in the longitudinal bar strain. The uniform frictional bond stress, u_f , is based on the results of an experimental investigation carried out by Pochanart and Harmon (1989).

$$u_l = 800 - 10 \frac{l_s}{l_h} \quad (\text{psi}) \quad (5.14)$$

where l_s and l_h are the clear spacing between lugs and height of lugs on the bar, respectively.

The length of the strain-hardening region, L_{sh} , can be calculated from Equation 5.13 by replacing the yield stress by the steel stress at the beginning of the strain-hardening range.

Given the axial strain distribution in the reinforcing bar, the slip can be calculated by integrating the strains over the length of the bar, which is the area under the strain diagram shown in Figure 5.17.

$$slip = \epsilon_s L_{pc} + 0.5(\epsilon_s + \epsilon_{sh}) L_{sh} + 0.5(\epsilon_{sh} + \epsilon_s) L_{sp} + 0.5(\epsilon_s) L_e \quad (5.15)$$

By substituting the lengths from Equation 5.11 and 5.13 into Equation 5.15, the slip can be rewritten as

$$\begin{aligned} slip &= \frac{\epsilon_s f_s d_h}{83.3 \sqrt{f'_c}} & \epsilon_s \leq \epsilon_y \\ slip &= \frac{\epsilon_s f_s d_h}{83.3 \sqrt{f'_c}} + \frac{(\epsilon_{sh} + \epsilon_s)(f_{sh} - f_s) d_h}{8 u_l} + \frac{(\epsilon_s + \epsilon_{sh})(f_s - f_{sh}) d_h}{8 u_l} & \epsilon_s > \epsilon_y \end{aligned} \quad (5.16)$$

5.5.2 Proposed Model

An analytical model was developed for modeling the moment-rotation relationship of a reinforced concrete section due to slip of the longitudinal reinforcing bars from adjacent anchorage zones. The model is described in the following paragraphs.

The slip resulting from accumulated axial strains in the bar inside the footing or beam-column joint can be calculated by integrating the strains over the distance between the interface and the point in the bar with no strain.

$$\begin{aligned} slip &= \int_0^{l_d} \epsilon \, dx & \epsilon \leq \epsilon_y \\ slip &= \int_0^{l_{dy}} \epsilon \, dx + \int_{l_{dy}}^{l_{dy}+l_d} \epsilon \, dx & \epsilon > \epsilon_y \end{aligned} \quad (5.17)$$

where l_d and l'_d are the development lengths over the elastic and inelastic portions of the bar, respectively (Figure 5.18). The length, l_{dy} is the portion of the bar with stress varying from zero to yield stress, and it can be calculated from Equation 5.6 using the yield stress for steel. The integration can be carried out by calculating the area under the strain diagram. Using a bilinear strain distribution shown in Figure 5.18, the slip is determined from

$$\begin{aligned} slip &= \frac{\epsilon_y l_d}{2} & \epsilon_s \leq \epsilon_y \\ slip &= \frac{\epsilon_y l_{dy}}{2} + \frac{l'_d}{2} (\epsilon_s + \epsilon_y) & \epsilon_s > \epsilon_y \end{aligned} \quad (5.18)$$

l_d in Equation 5.18 is calculated from Equation 5.6 with $f_s \leq f_y$. From equilibrium of forces in the bar (Equation 5.5), the inelastic development length, l'_d is calculated from the following equation

$$l'_d = \frac{(f_s - f_y) d_b}{4 u'_b} \quad (5.19)$$

where u'_b is the average uniform bond stress in the inelastic portion of the bar.

Figure 5.19 compares the steel stress-strain relations used in the section moment-curvature analysis and the proposed bond-slip model. As illustrated in the figure, for strains exceeding ϵ_{sh} , which is the strain at the onset of strain-hardening region ($= 0.014$ in this study), in Equation 5.19 the inelastic stress increment is approximated as the difference between the current stress in the bar and the yield stress (i.e., $f_s - f_y$).

It should be noted that the uniform bond stress model proposed here results in zero inelastic slip as the steel strain increases from ϵ_y to ϵ_{sh} if there is no strain increase along this yield plateau. This behavior seemed unrealistic. Therefore, in this study, the steel stress-strain relation was given a modest strain-hardening even along the yield plateau (see Figure 5.1).

Using equilibrium at yield level and assuming a linear strain distribution, by inserting Equation 5.6 into Equation 5.7 for l_d , an average uniform bond stress at yield, u_{by} , can be calculated in terms of slip.

$$u_{bv} = \frac{f_y^2 d_b}{8 E_s slip} \quad (5.20)$$

The slip was measured at the ends of twelve columns tested as part of this investigation and by Lynn et al. (1996). Using the measured slip values at yield displacement, uniform bond stresses were calculated from Equation 5.20. The calculated bond stresses are normalized by $\sqrt{f'_c}$ (psi) and presented in Figure 5.20. In the figure, following the convention used by Lynn et al. (1996), Specimen-1, Specimen-2, Specimen-3 and Specimen-4 are denoted as CLD12, CHD12, CVD12, and CLD12M, respectively. For the twelve columns considered, the average bond stress was $11.4\sqrt{f'_c}$ (psi) and the standard deviation was $2.5\sqrt{f'_c}$ (psi). Thus, in this study, a uniform bond stress of $12\sqrt{f'_c}$ ($= u_b$ in Figure 5.18) is used in the elastic range. In the portion of the reinforcing bar over which the yield strain is exceeded (l_d'), a uniform bond stress of $6\sqrt{f'_c}$ ($= u'_b$ in Figure 5.18) is adapted from Lehman and Moehle (2000). If Equations 5.6 and 5.19 are substituted into Equation 5.18 for the elastic and inelastic development lengths, and using the assumed average bi-uniform bond stresses, the slip is given by

$$\begin{aligned} slip &= \frac{\epsilon_s f_y d_b}{96\sqrt{f'_c}} & \epsilon_s \leq \epsilon_y \\ slip &= \frac{\epsilon_s f_y d_b}{96\sqrt{f'_c}} + \frac{(\epsilon_s + \epsilon_y)(f_y - f_y)d_b}{48\sqrt{f'_c}} & \epsilon_s > \epsilon_y \end{aligned} \quad (5.21)$$

In the proposed model, it was assumed that the section would rotate about its neutral axis. Then, as illustrated in Figure 5.21, the section rotation due to bar slip can be

calculated by dividing the bar slip by the width of the open crack, which is the difference between the depth of the section, d , and the neutral axis depth, c .

$$\theta_{slip} = \frac{slip}{d - c} \quad (5.22)$$

Substitution of Equation 5.21 into 5.22 yields

$$\begin{aligned} \theta_{slip} &= \frac{\epsilon_s f_s d_b}{96\sqrt{f'_c}(d - c)} & \epsilon_s \leq \epsilon_y \\ \theta_{slip} &= \frac{d_b}{96\sqrt{f'_c}(d - c)} \left[\epsilon_y f_y + 2(\epsilon_s - \epsilon_y)(f_s - f_y) \right] & \epsilon_s > \epsilon_y \end{aligned} \quad (5.23)$$

For the specimens tested in this study, the development length, l_d , was calculated from Equation 5.6 using an average elastic bond stress u_b of $12\sqrt{f'_c}$. The calculated development length was few inches smaller than the actual embedment length in test columns indicating that Equation 5.23 could be evaluated using the test results of this study.

5.6 COMPARISON OF TEST DATA AND BOND-SLIP MODELS

The bar stress-slip relations from the proposed model, Otani and Sozen (1972) model, and Alsiwat and Saatcioglu (1992) model are compared in Figure 5.22. The bar stress-slip relations were obtained from Equations 5.8, 5.16 and 5.21 using the uniaxial steel material model shown in Figure 5.1. For steel strains smaller than the yield strain, all models used the same elastic slip equation given in Equation 5.18 where l_d was calculated from Equation 5.6. The only difference between the three models was the

magnitude of the assumed average elastic bond stress, u_b , which was $12\sqrt{f'_c}$, $6.5\sqrt{f'_c}$ and $10.4\sqrt{f'_c}$ in the proposed, Otani and Sozen, and Alsiwat and Saatcioglu bond-slip models, respectively. After the first yielding in the bar, the difference between the bar stress-slip behavior from the proposed and Alsiwat and Saatcioglu models was a result of different assumptions for the development lengths and the average bond stress in the inelastic portion of the bar.

In the fiber section moment-curvature analysis, the stress in the tension reinforcement can be monitored and recorded as a function of moment. Then, using the analysis results and following the analytical procedures presented above, the section moment-bar slip relations can be obtained under different axial loads. Figure 5.23 shows the moment-slip relations under zero and 150-kip axial loads. Note that the slip in the Otani and Sozen model is a function of moment (Equation 5.10) and can be calculated without moment-curvature analysis. The moment-slip relations from all three models are most similar in the elastic range under zero axial load. Because the model developed by Otani and Sozen is largely based on the assumptions of elastic material behavior, the slip is less in the inelastic range. The difference between the other two models is a result of assumed inelastic development length and average bond stress. Under higher axial loads, the axial strain in the tension reinforcement decreases resulting in smaller slip as shown in Figure 5.23.

In the same way, the monotonic moment-slip rotation relations were calculated under constant 600-kip axial load, which was applied on Specimen-2, and under varying axial load as applied on Specimen-3. Figure 5.24 shows a comparison of moment-slip rotation

relations from the proposed model using moment curvature analysis results, and from the measurements at the peak of each lateral displacement cycle. The figure indicates that under monotonic lateral loading the slip rotation can be estimated reasonably well from the proposed analytical model. As indicated by the calculated moment-slip rotation relations for Specimen-2 and Specimen-3 in compression, under very high axial loads, the analytical model predicts slip rotations smaller than the measured response. This is because the whole section stays under compression resulting in a neutral axis depth, c , larger than the depth of the section. Consequently, Equation 5.22 yields a negative slip rotation under small moments and high axial loads. In this study, a zero slip rotation was assumed when the neutral axis depth was larger than the section depth.

The measured slip rotations were obtained from the displacements recorded by the displacement potentiometers near the column ends following the procedure illustrated in Figure 5.5. Specifically, the slip deformation is the difference between the total deformations and extrapolated flexural deformations measured at the member ends. In Figure 5.25, the measured slip rotations are plotted against the strains measured by selected strain gages at the base beam-column interface and top beam-column interface of each specimen. Especially in Specimen-1, top of Specimen-2, bottom of Specimen-3 and top of Specimen-4, after the first yielding in the bar at the interface, increase in the section rotation due to bar slip was very small with a large increase in strains. In other locations where a jump in the strain was not observed, either the first yielding was in compression as in Specimen-2 and Specimen-3, or the loading was reversed immediately after the first yielding as in the column base in Specimen-4.

Figure 5.26 shows the measured hysteretic moment-slip rotation relations at the base beam-column and top beam-column interface of each specimen. Monotonic moment-slip rotation relations computed with the proposed model are also shown in the figure. The computed and measured responses appear to agree better under lower axial loads. Under higher axial loads, in Specimen-2 and Specimen-3 with compressive axial loads larger than 250 kips, the slip rotation is underestimated because the calculated neutral axis is larger than the section depth under small moments.

If the slip rotation at the top and bottom of a double-curvature column ($\theta_{slip,top}$ and $\theta_{slip,bottom}$) are known, the total lateral displacement due to bar slip can be estimated from

$$\Delta_{slip} = (\theta_{slip,top} + \theta_{slip,bottom}) L \quad (5.24)$$

Figure 5.27 through Figure 5.30 show the resulting experimental lateral load-slip displacement relations for the four specimens tested. The monotonic lateral load-slip displacement relations calculated using the proposed model are also shown in the same figures. The comparisons in the figures indicate a consistent and good agreement between the predicted monotonic response and experimental cyclic response for each specimen.

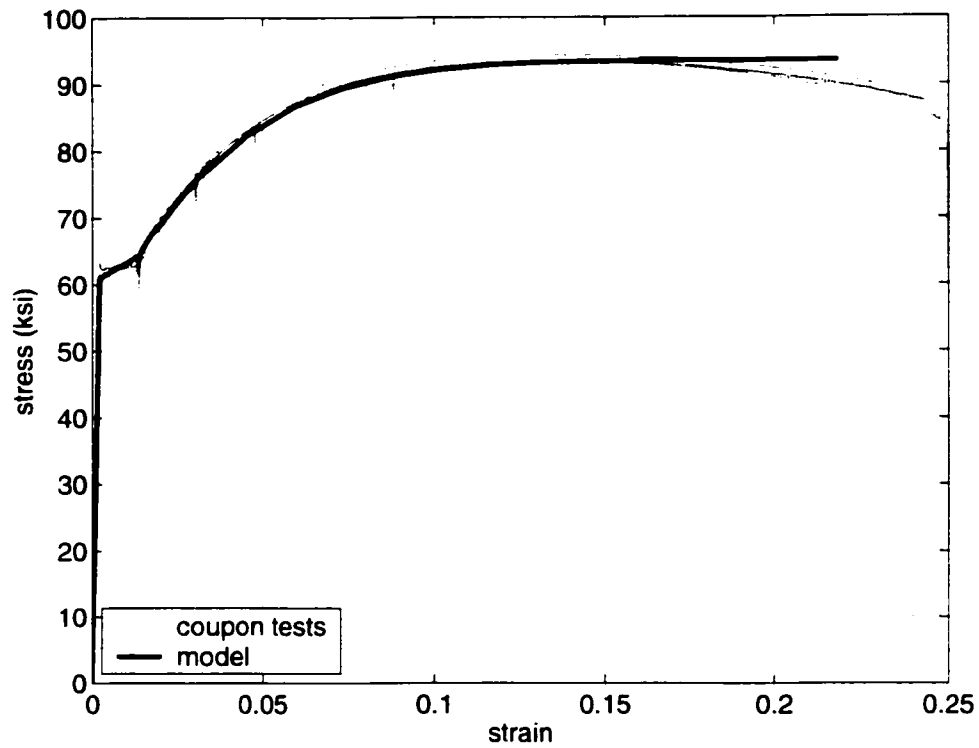


Figure 5.1 Measured longitudinal steel stress-strain relations and steel material model

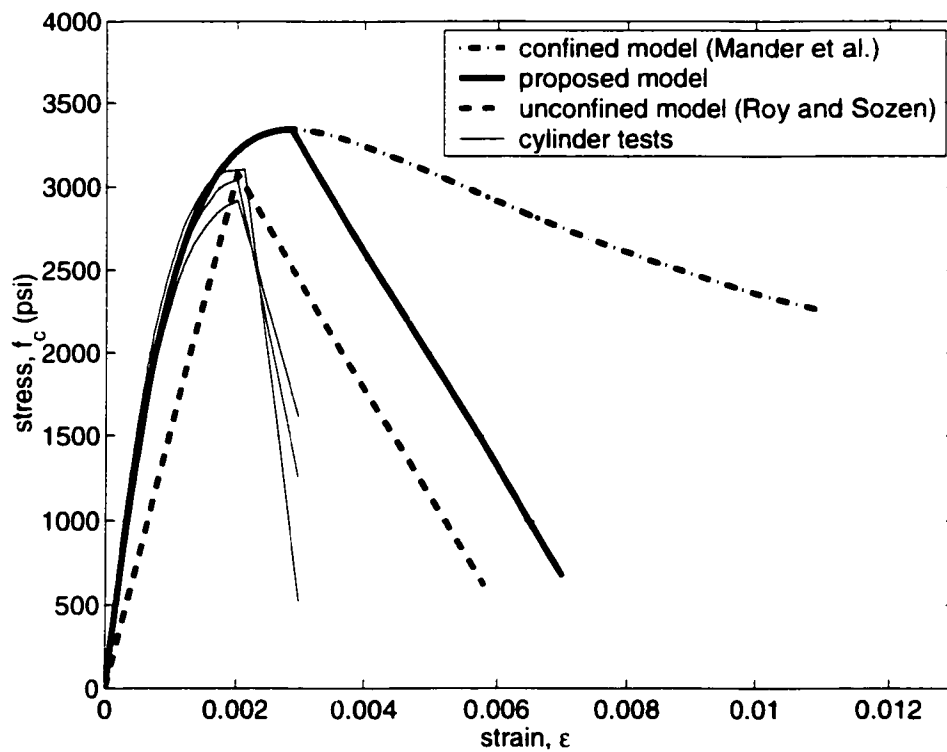


Figure 5.2 Compressive stress-strain relations for concrete

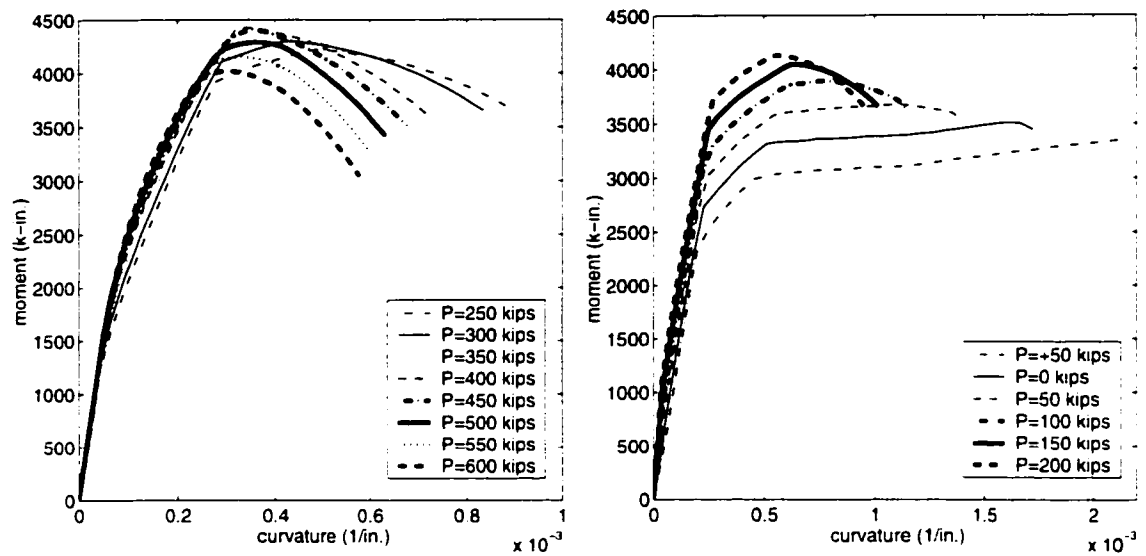


Figure 5.3 Moment-curvature relations under various constant axial loads

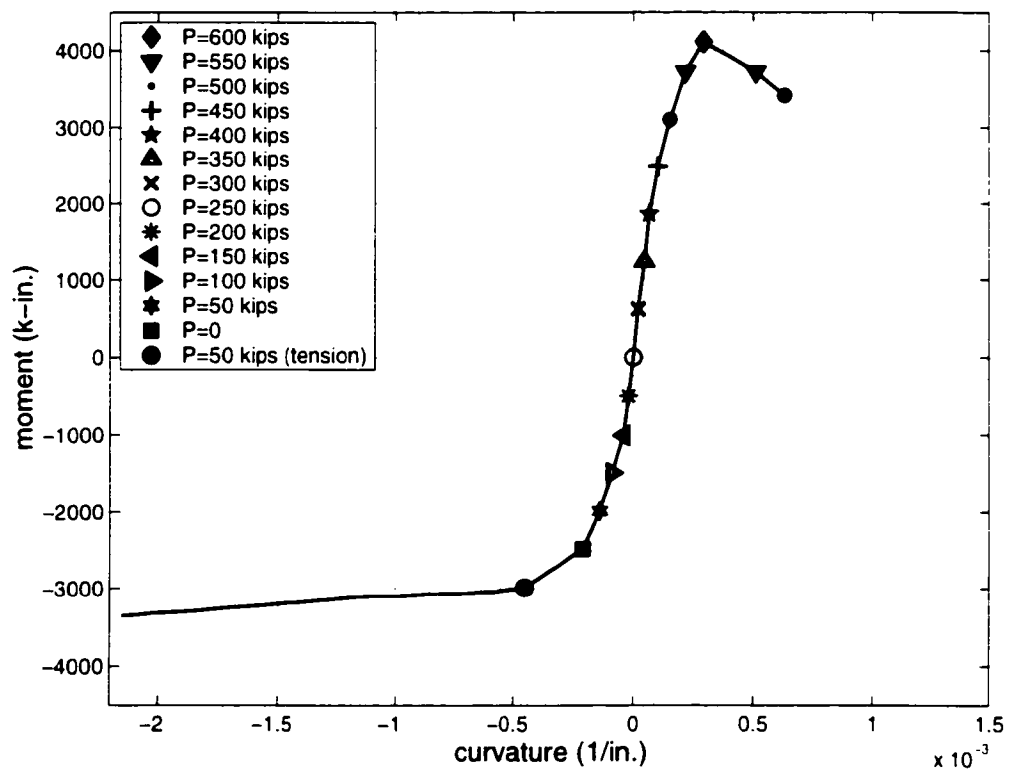


Figure 5.4 Calculated moment-curvature relations under varying axial load

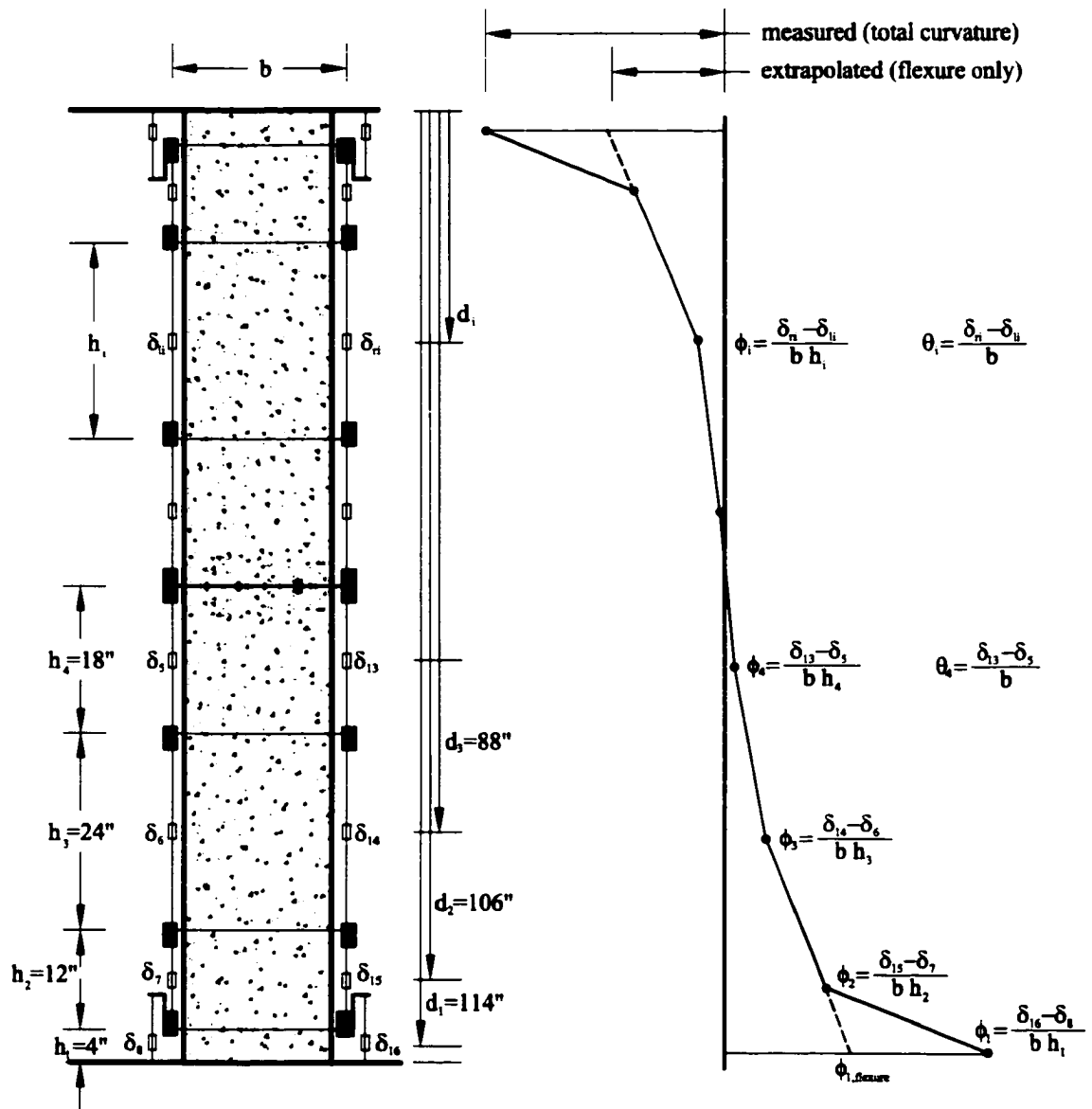


Figure 5.5 Curvature calculations from measured displacements

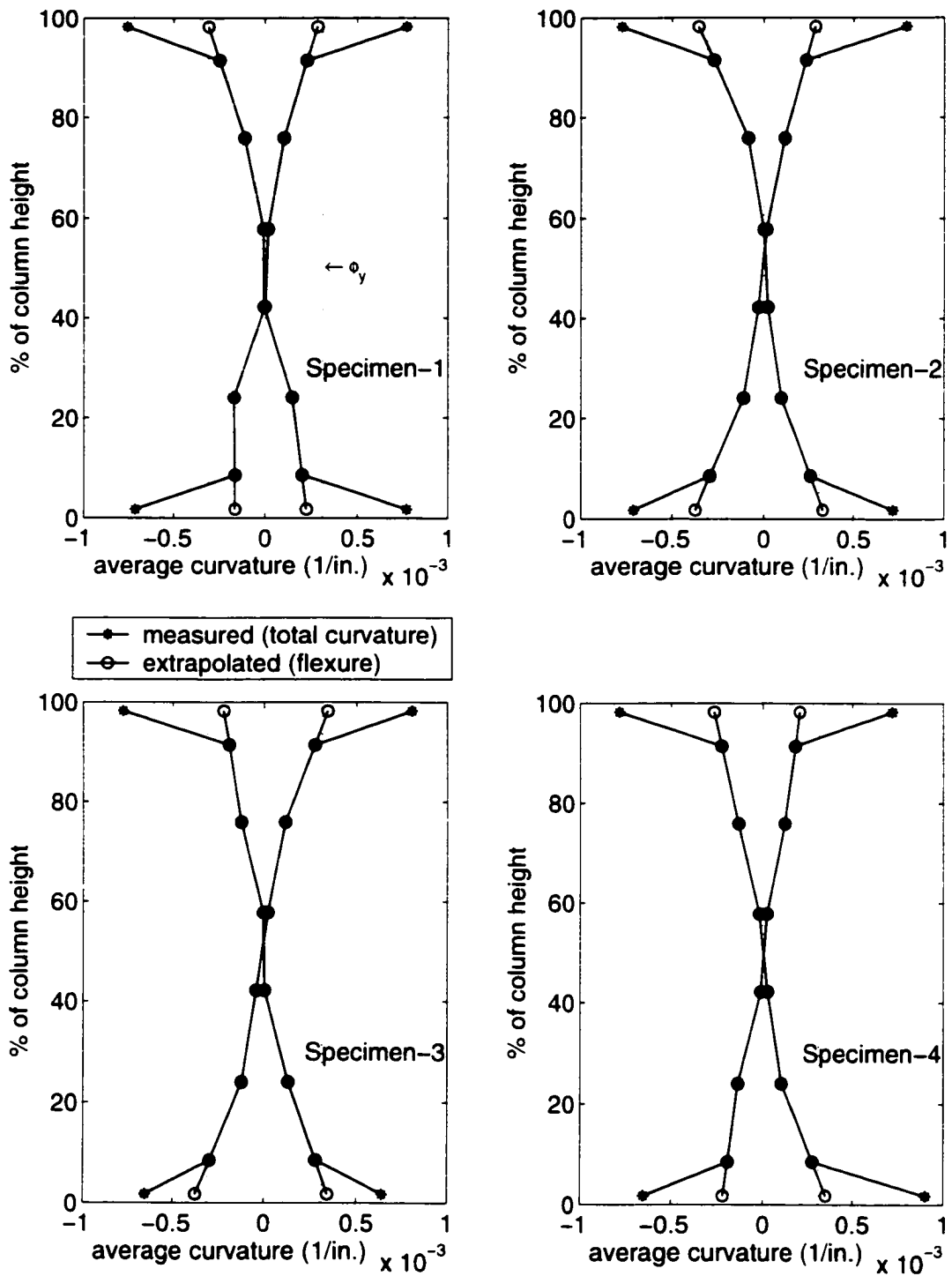


Figure 5.6 Extrapolated (flexure) and total measured curvature profiles at yield level

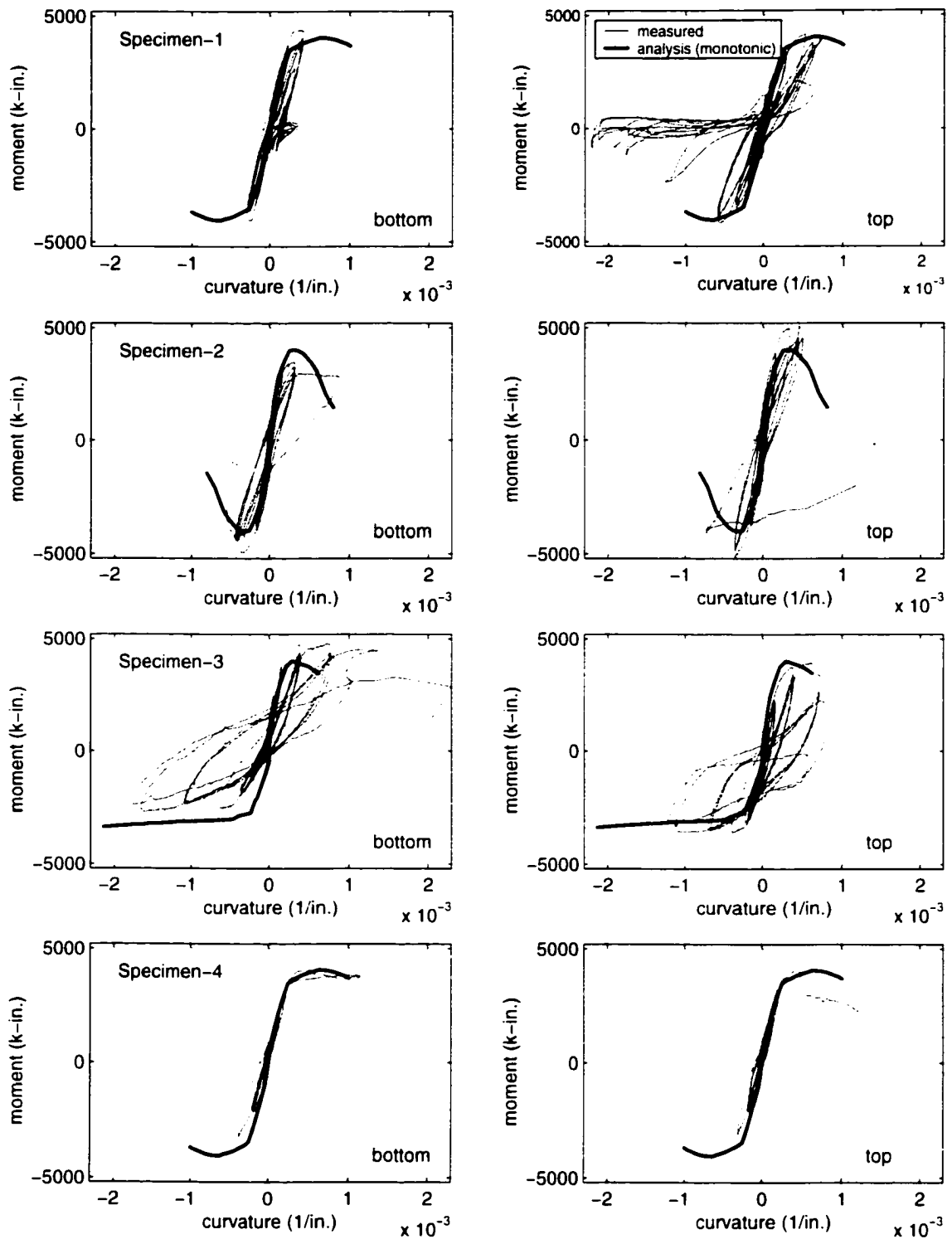


Figure 5.7 Comparison of calculated monotonic moment-curvature relations and measured cyclic moment-curvature relations at the top and bottom of each specimen

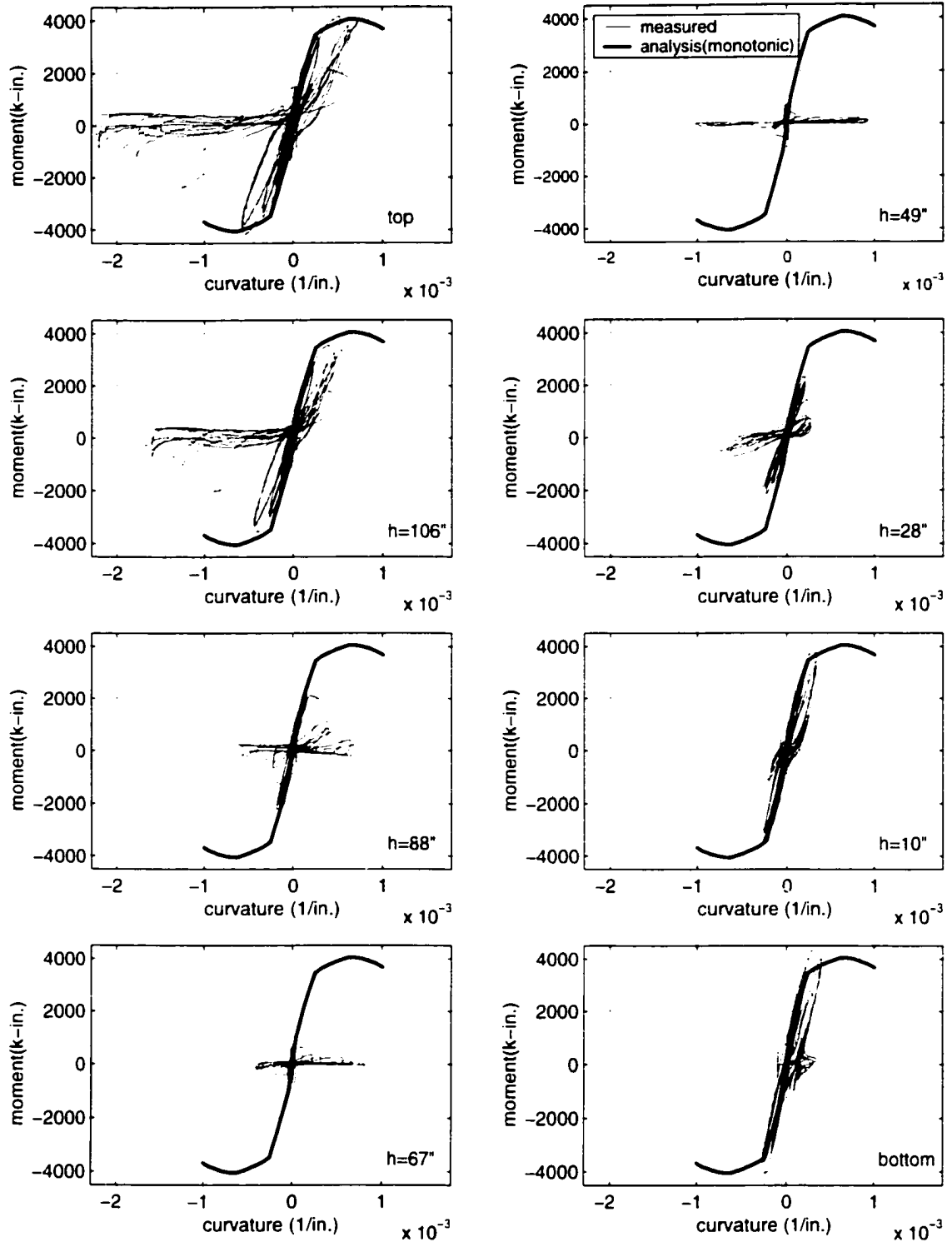


Figure 5.8 Comparison of calculated monotonic moment-curvature relations and measured cyclic moment-curvature relations over the height of Specimen-I

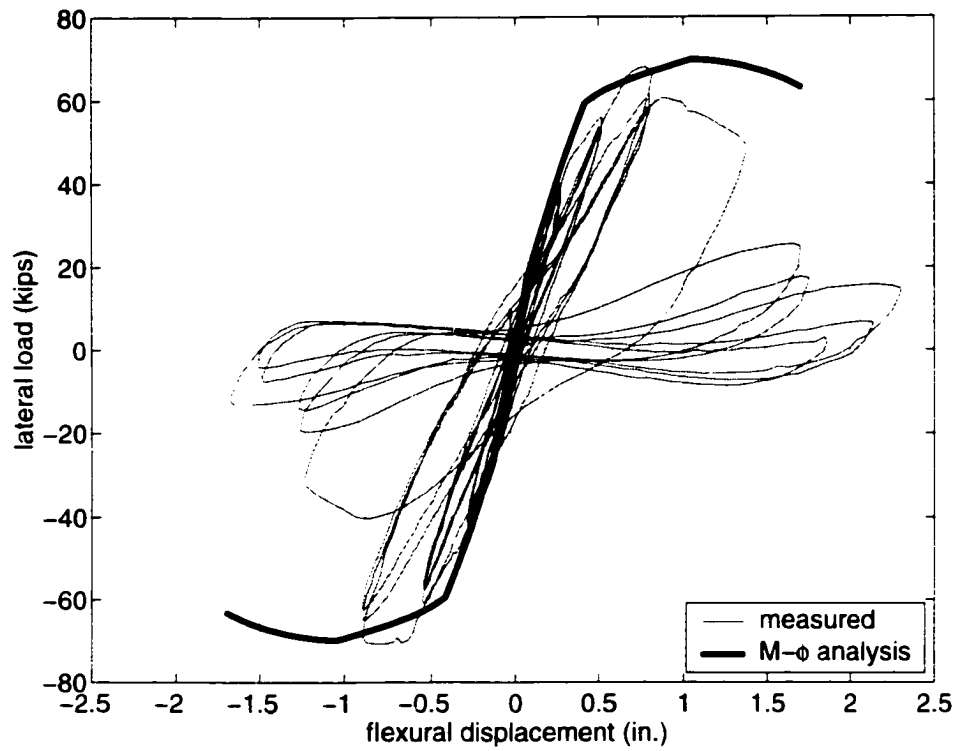


Figure 5.9 Lateral load-flexural displacement relations (Specimen-1)

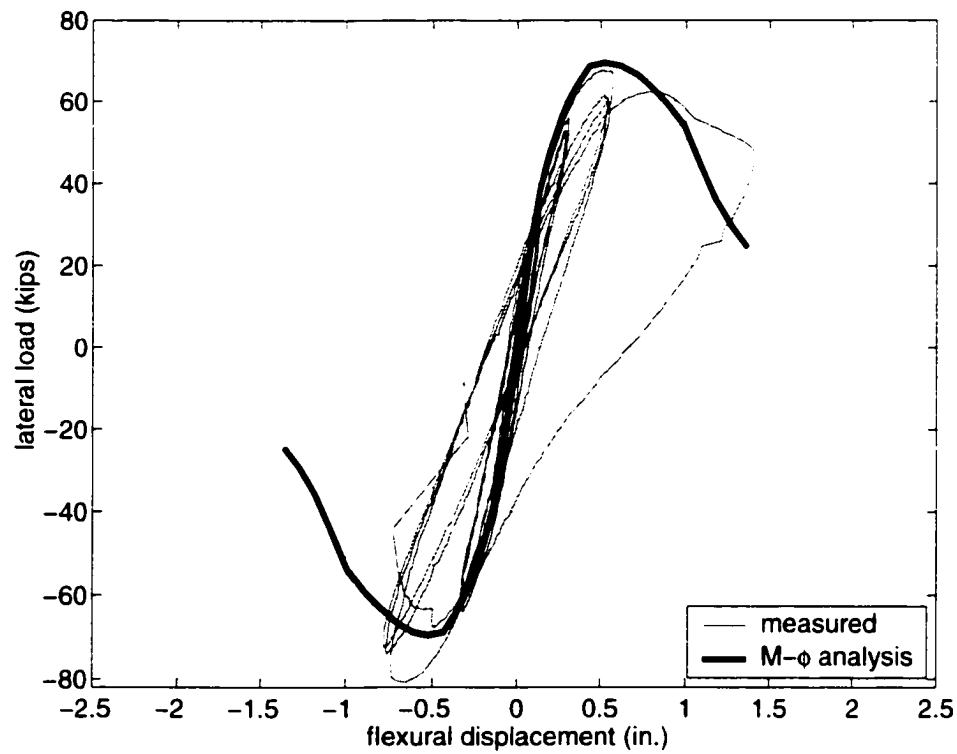


Figure 5.10 Lateral load-flexural displacement relations (Specimen-2)

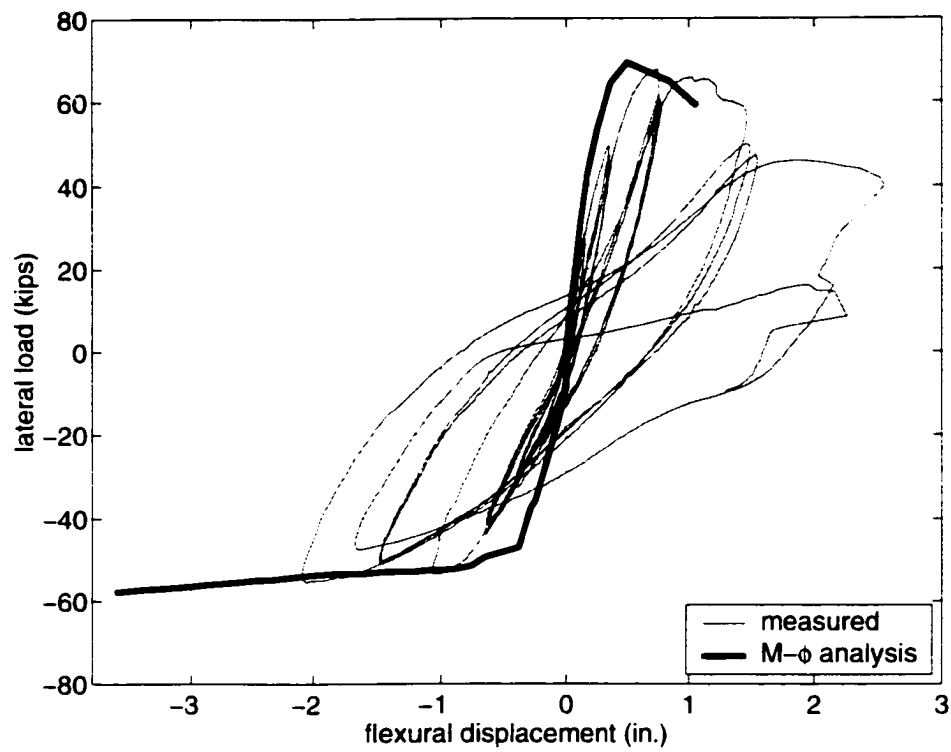


Figure 5.11 Lateral load-flexural displacement relations (Specimen-3)

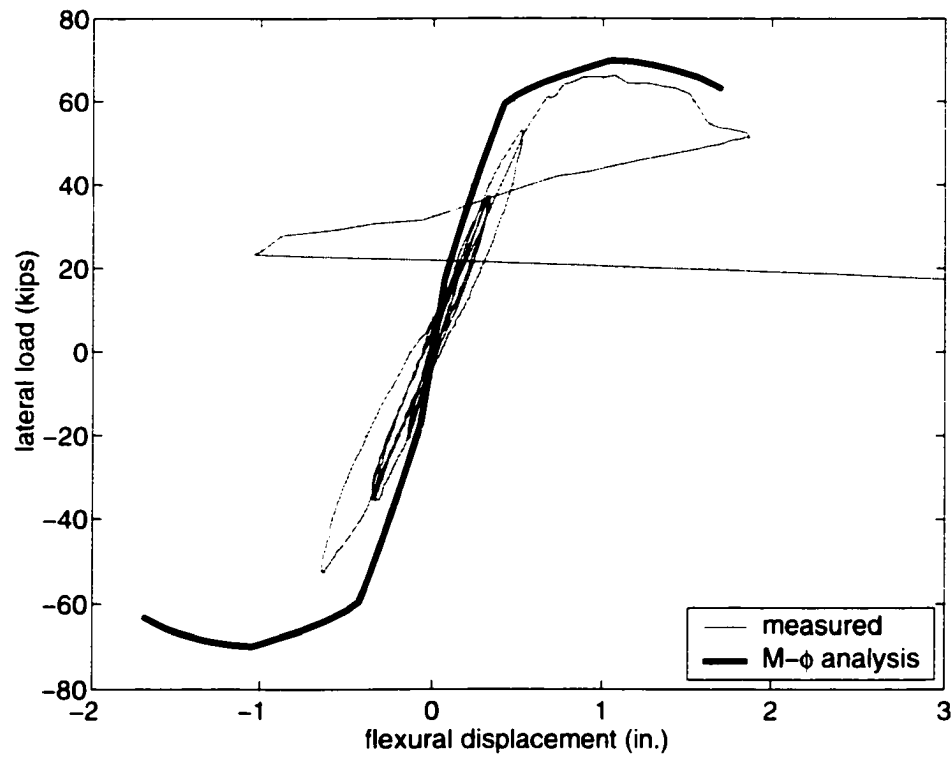


Figure 5.12 Lateral load-flexural displacement relations (Specimen-4)

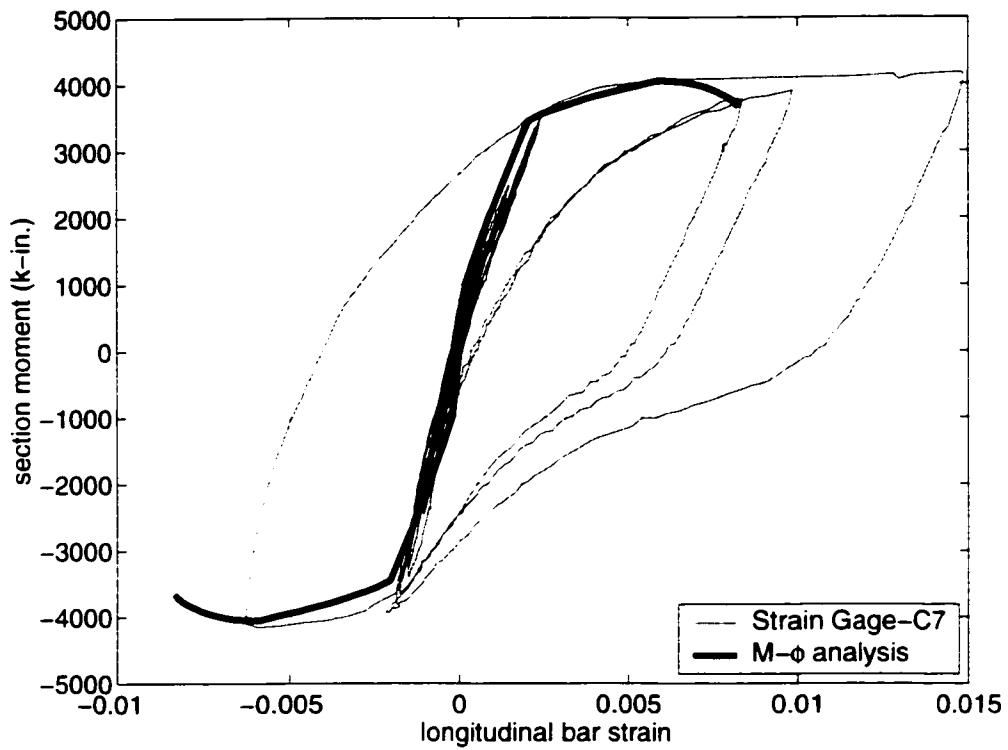


Figure 5.13 Comparison of calculated and measured strains (top of Specimen-1)

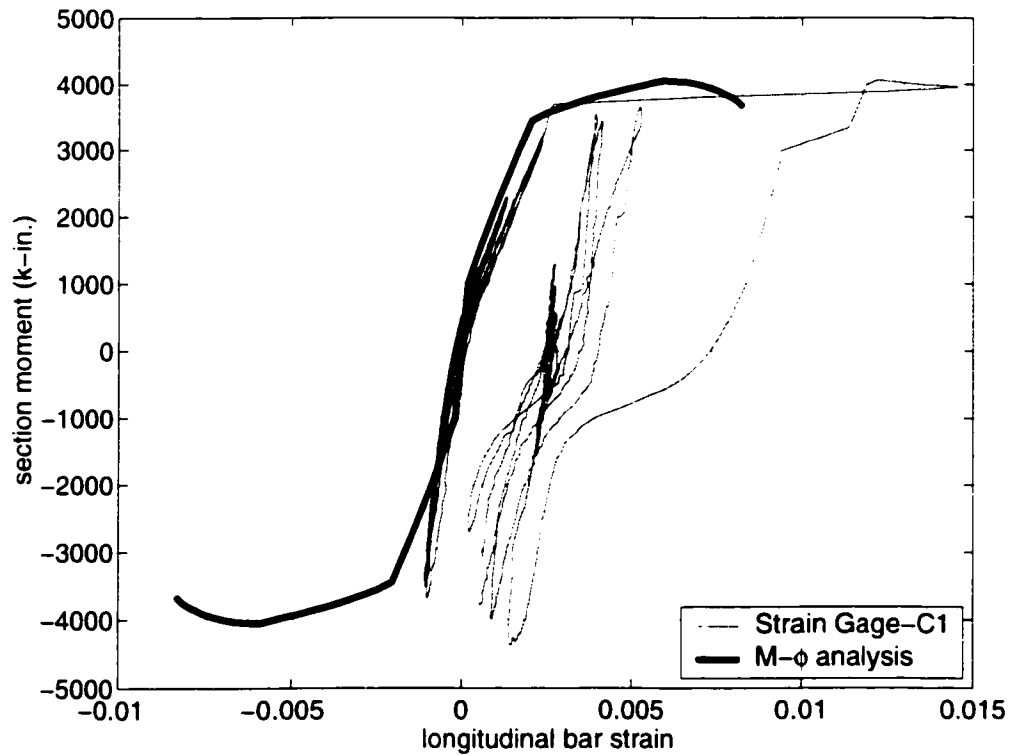


Figure 5.14 Comparison of calculated and measured strains (bottom of Specimen-1)

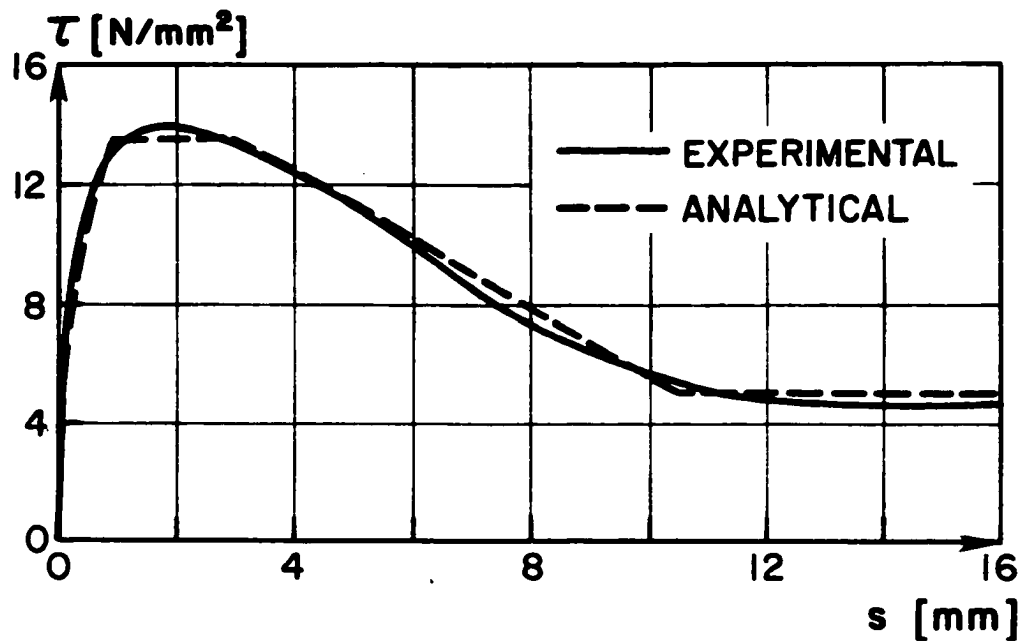


Figure 5.15 Bond stress-slip model proposed by Eligehausen et al. (1983)

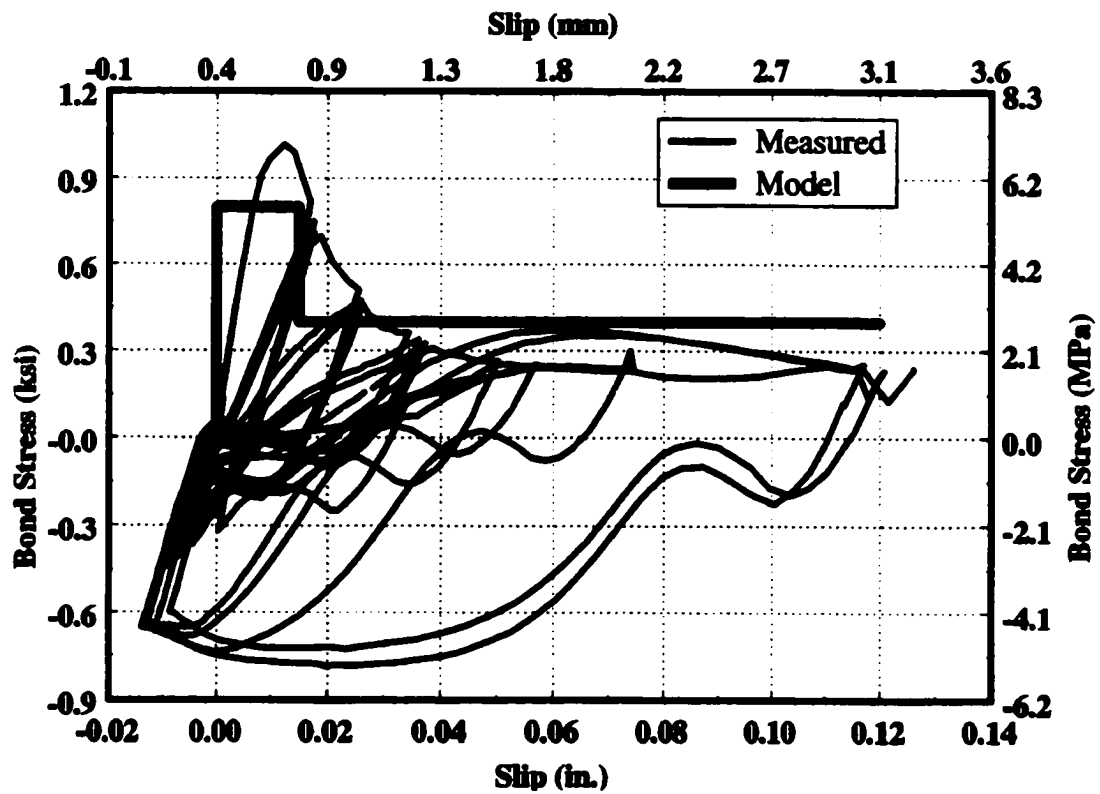


Figure 5.16 Bond stress-slip model proposed by Lehman and Moehle (2000)

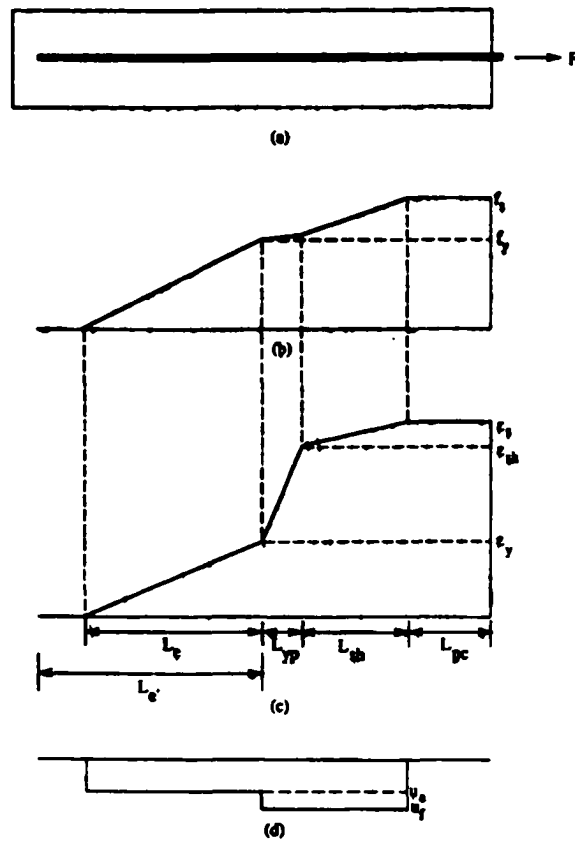


Figure 5.17 Stress, strain, and bond stress distribution (Alsiwat and Saatcioglu, 1992)

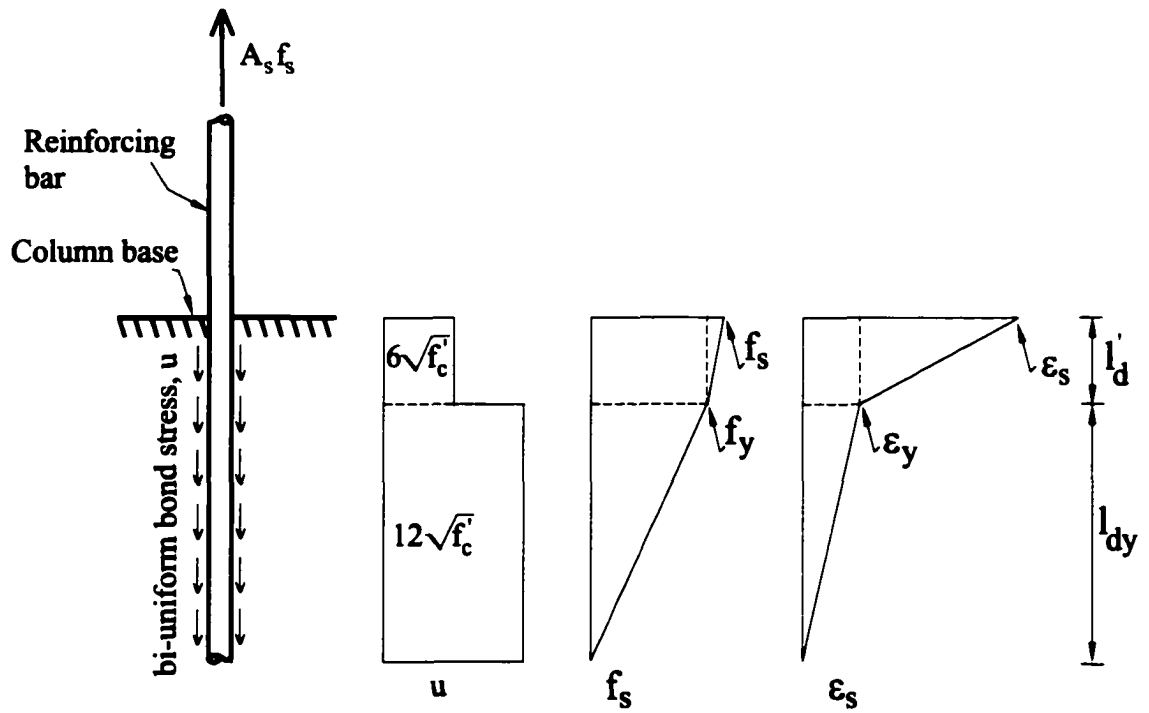


Figure 5.18 Assumed bar strain and stress distributions for the proposed bond-slip model

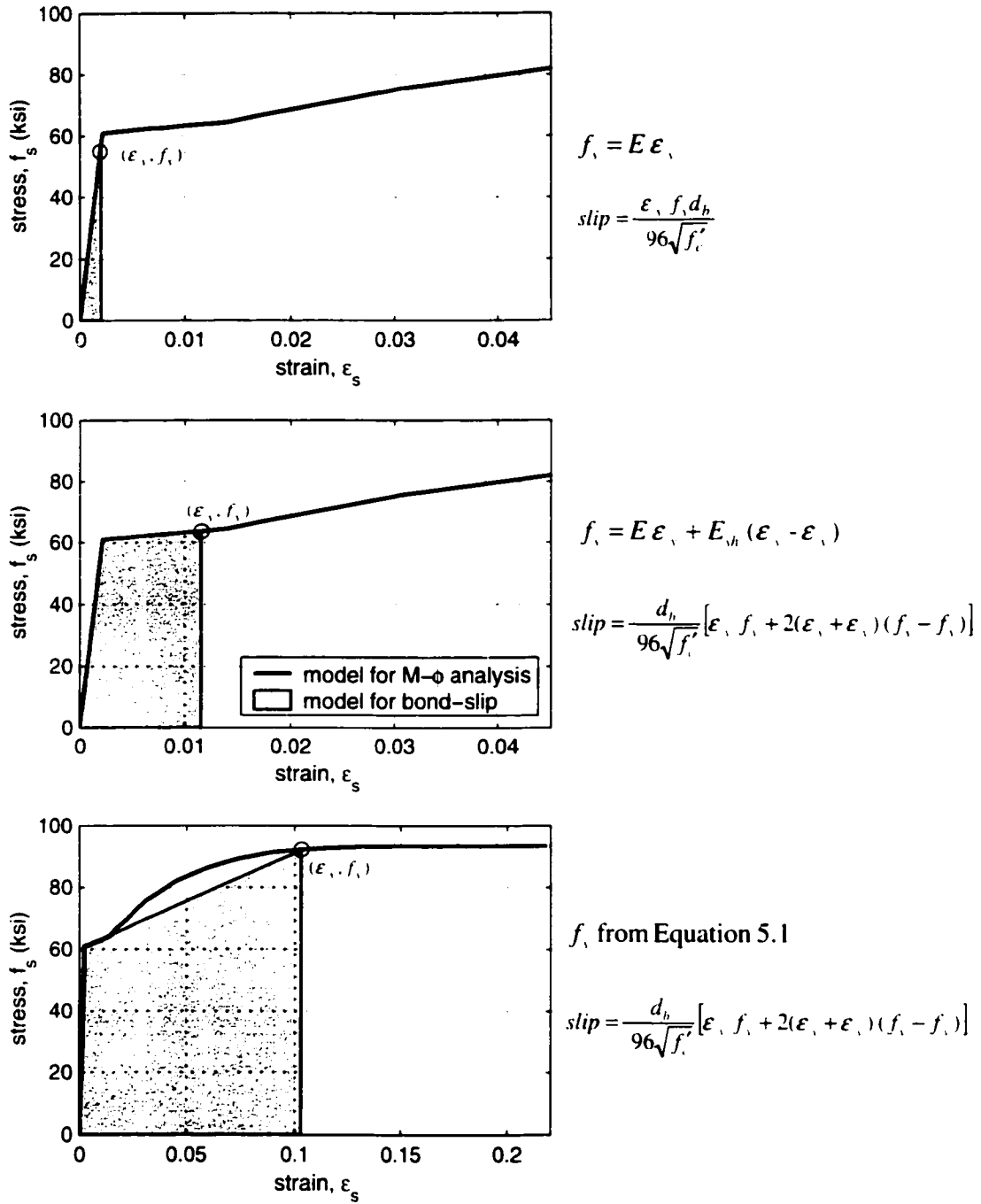


Figure 5.19 Comparison of longitudinal reinforcing bar stress-strain relations for the proposed bond-slip model and moment-curvature analysis

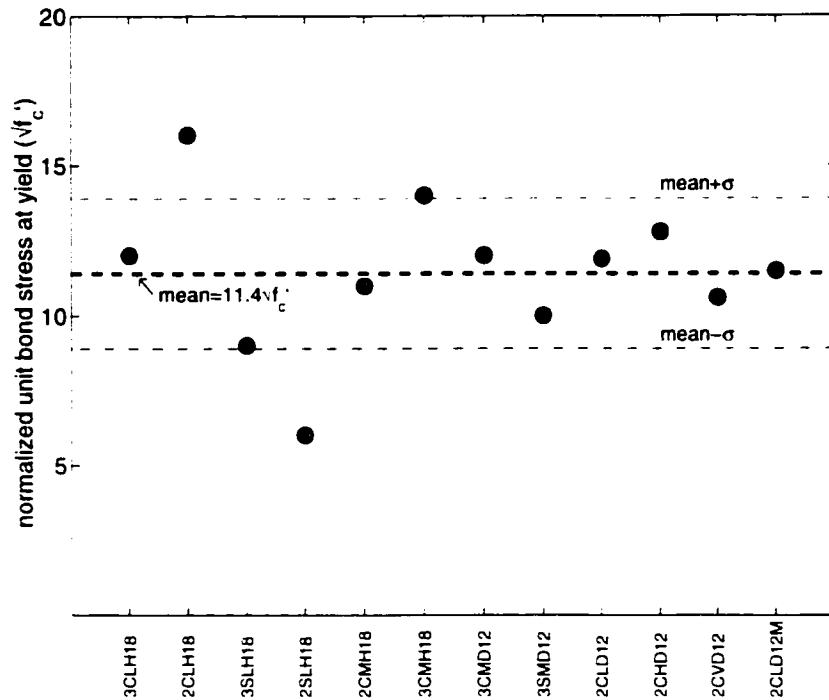


Figure 5.20 Calculated bond stresses at yield level

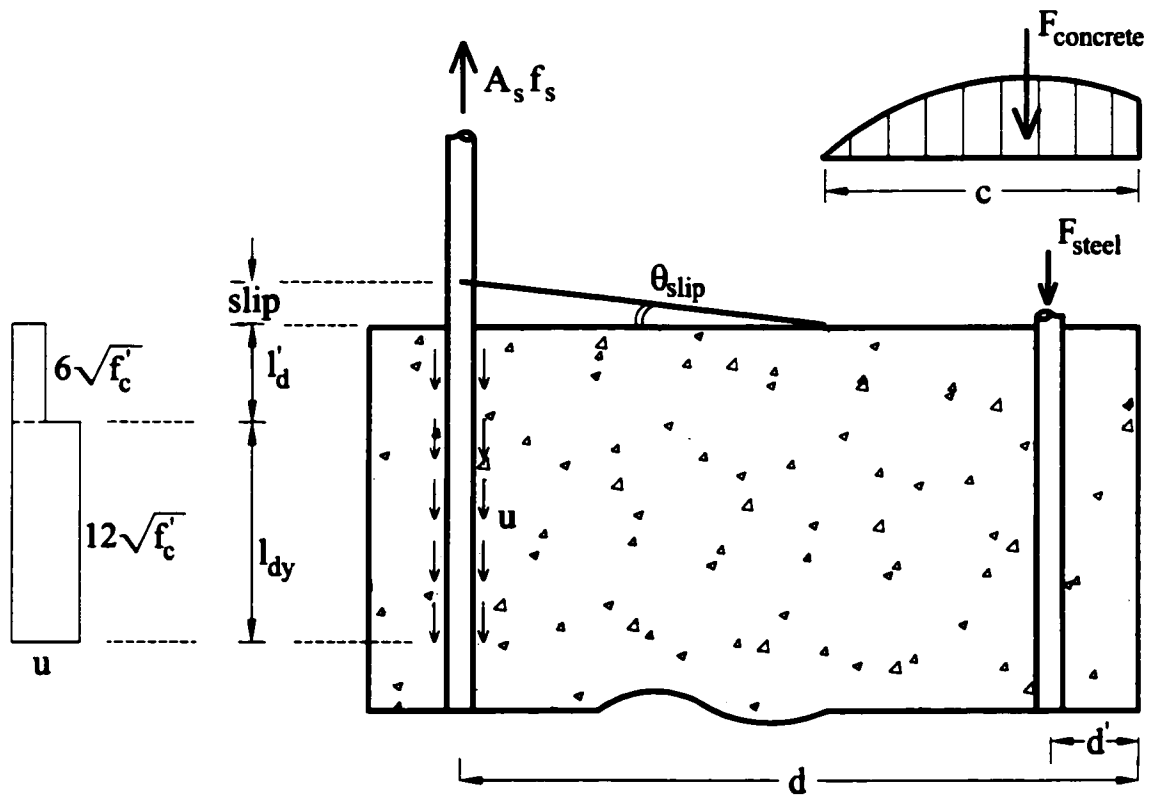


Figure 5.21 Slip rotation and forces in the proposed bond-slip model

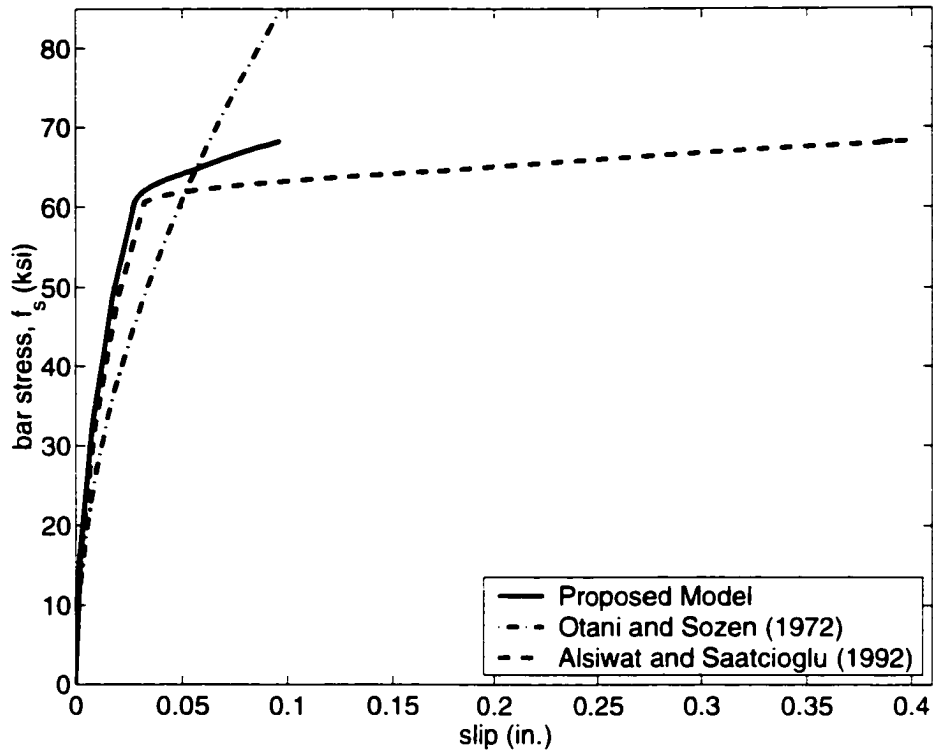


Figure 5.22 Reinforcing bar stress-slip relations from analytical models

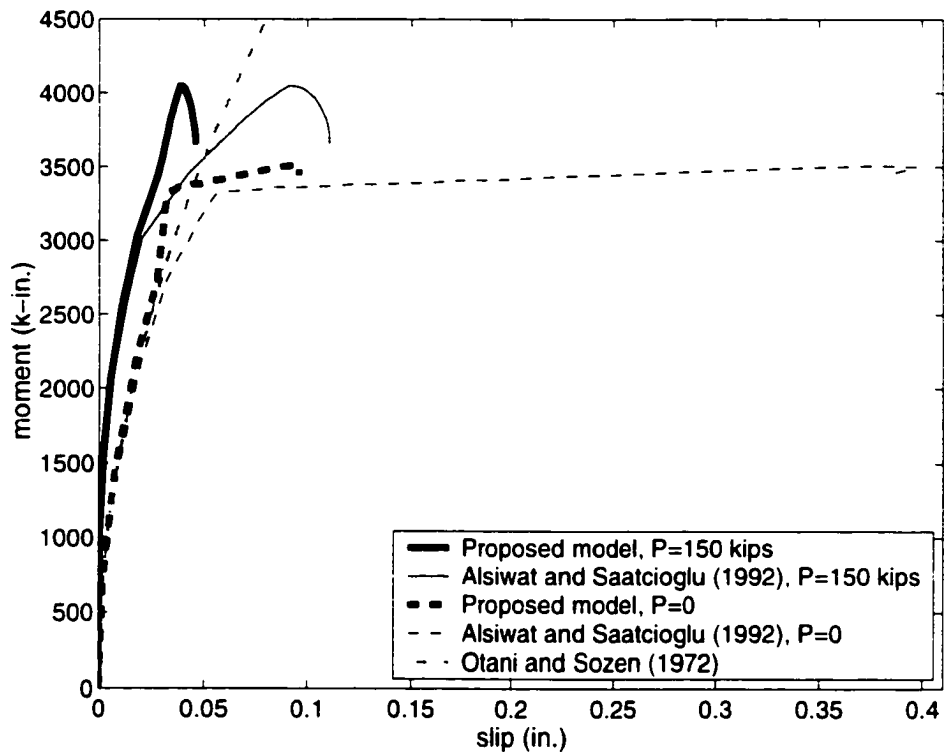


Figure 5.23 Comparison of moment-slip relations

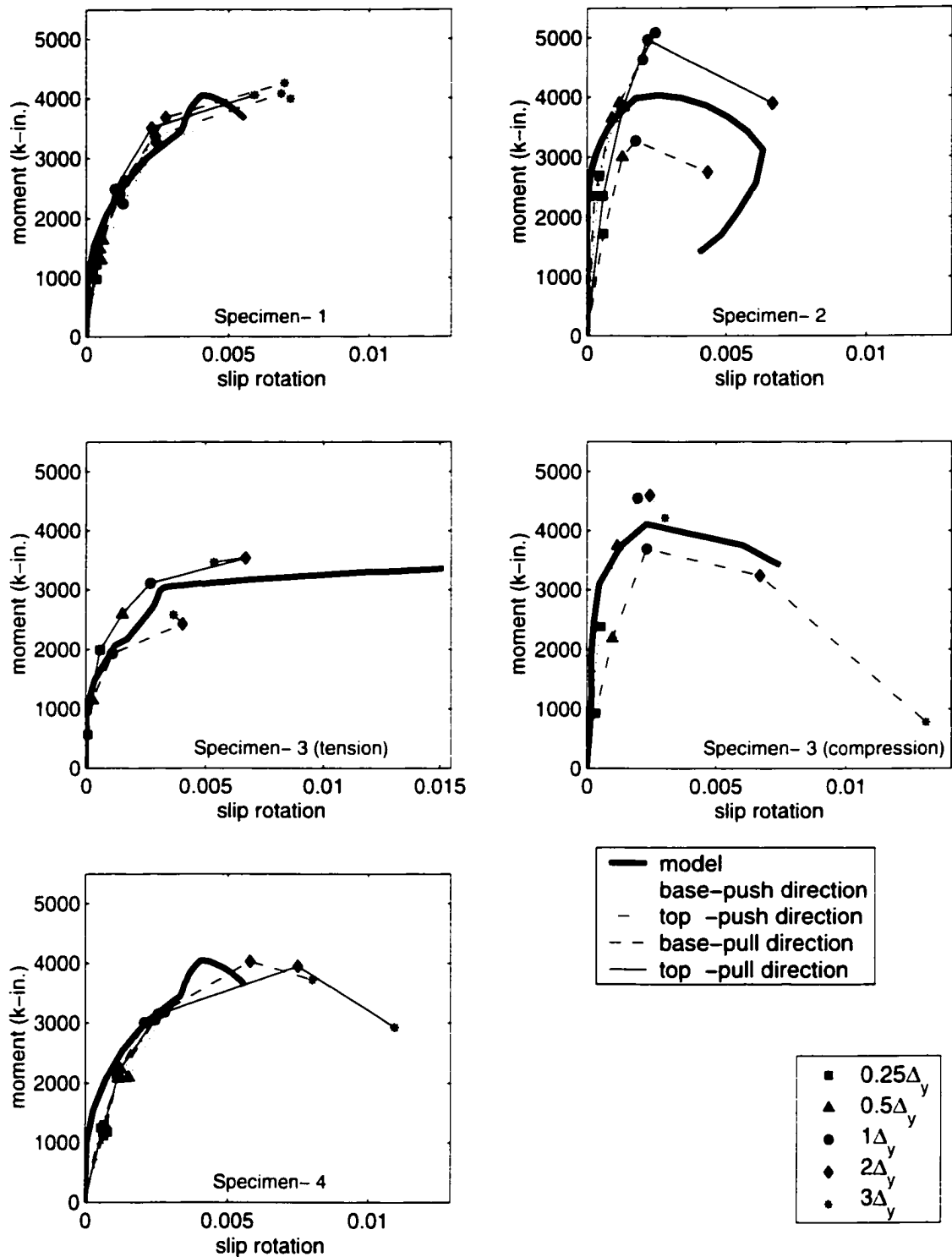


Figure 5.24 Comparison of calculated monotonic moment-slip rotation relations and measured peak slip rotations at each displacement level

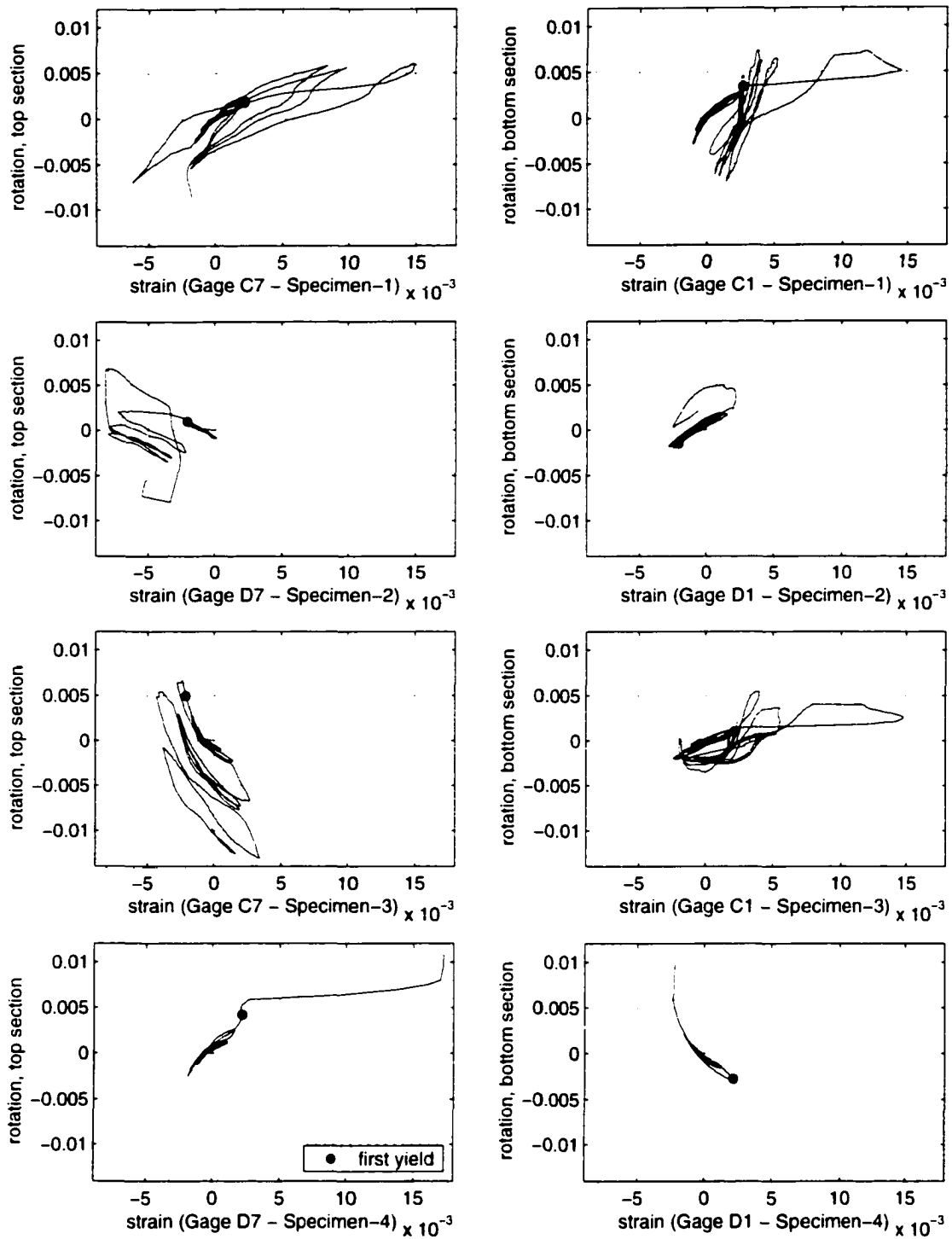


Figure 5.25 Measured slip rotation-strain relations at the top and bottom interface

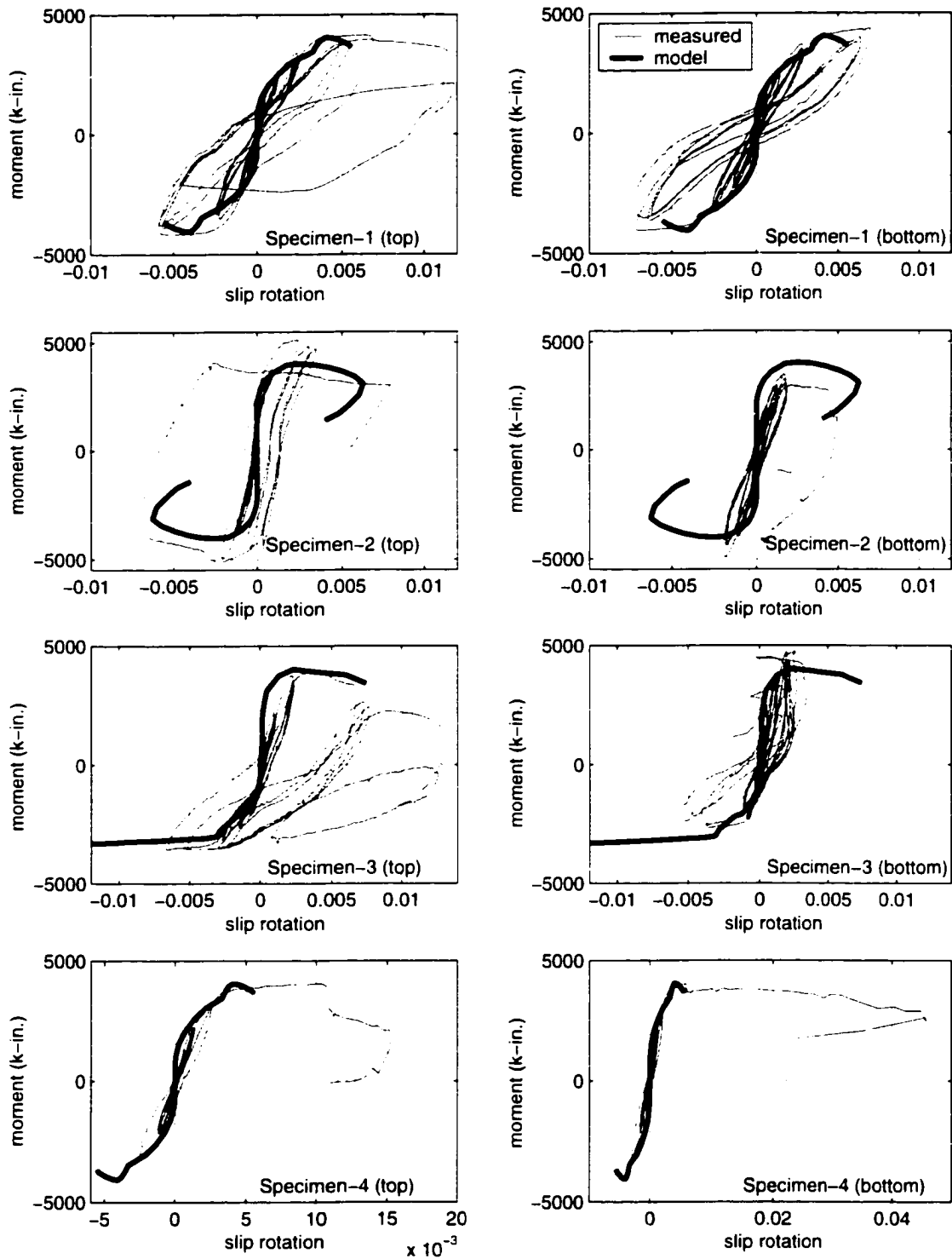


Figure 5.26 Comparison of calculated monotonic moment-slip rotation relations and measured cyclic moment-slip rotation relations at the top and bottom of each specimen

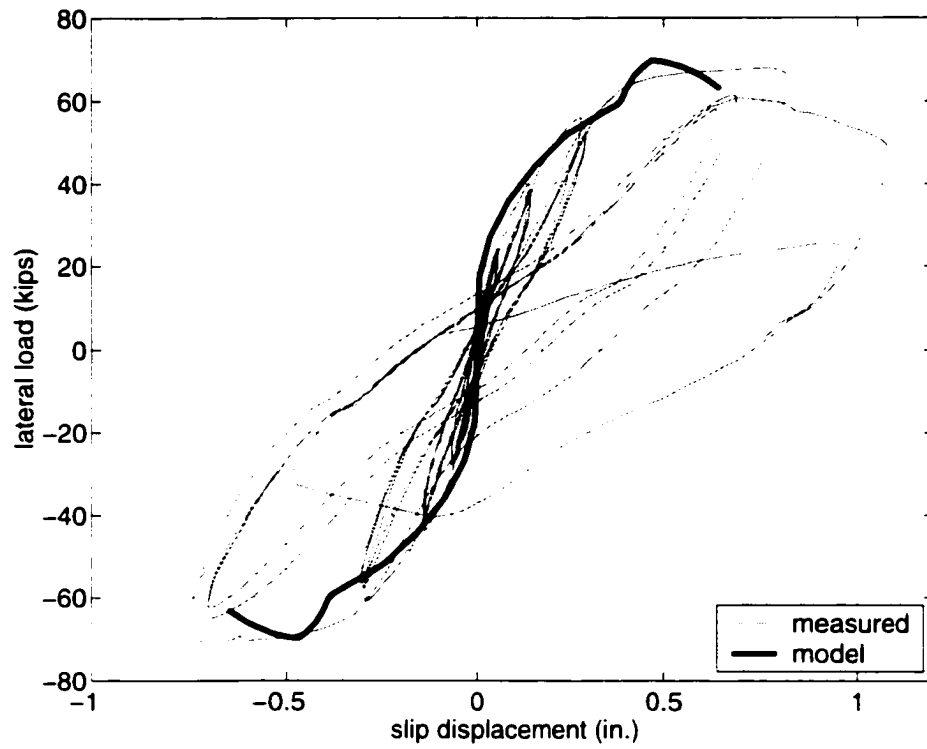


Figure 5.27 Lateral load-slip displacement relations (Specimen-1)

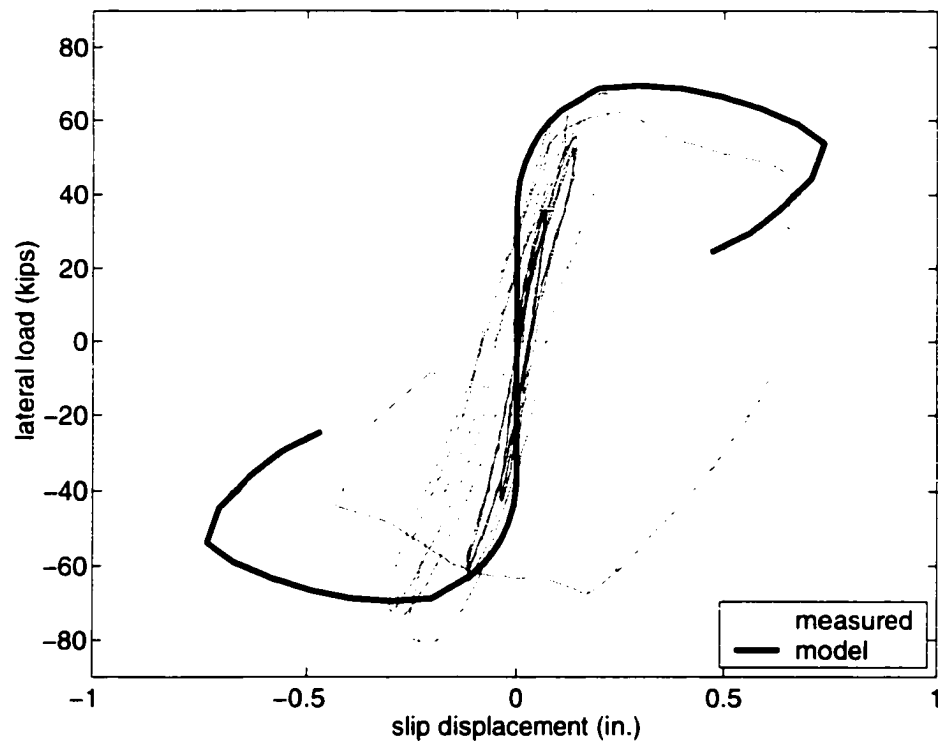


Figure 5.28 Lateral load-slip displacement relations (Specimen-2)

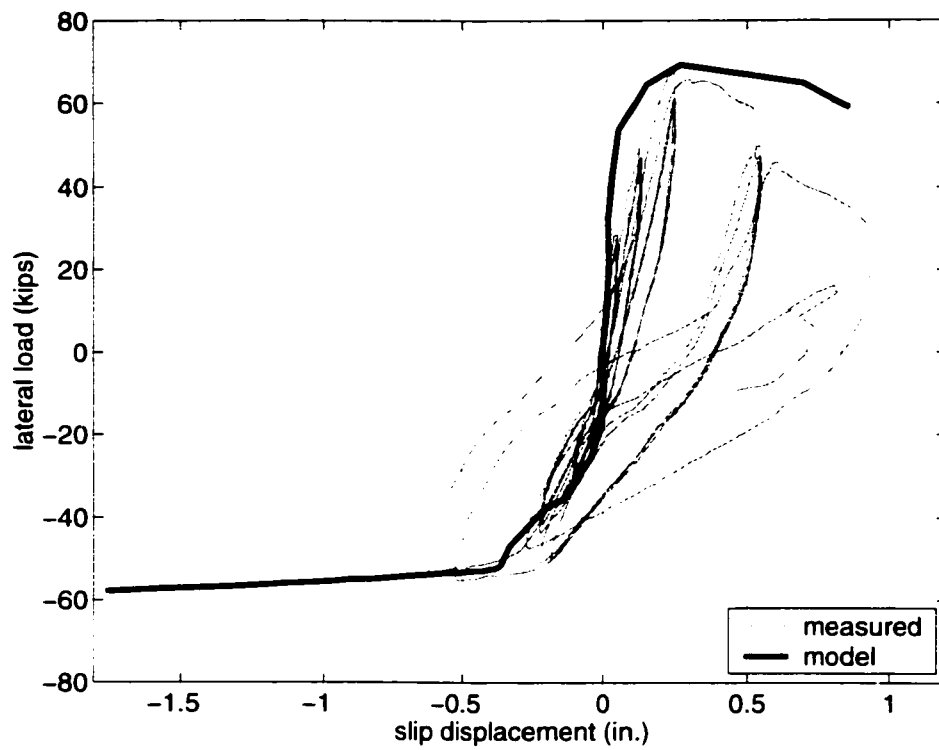


Figure 5.29 Lateral load-slip displacement relations (Specimen-3)

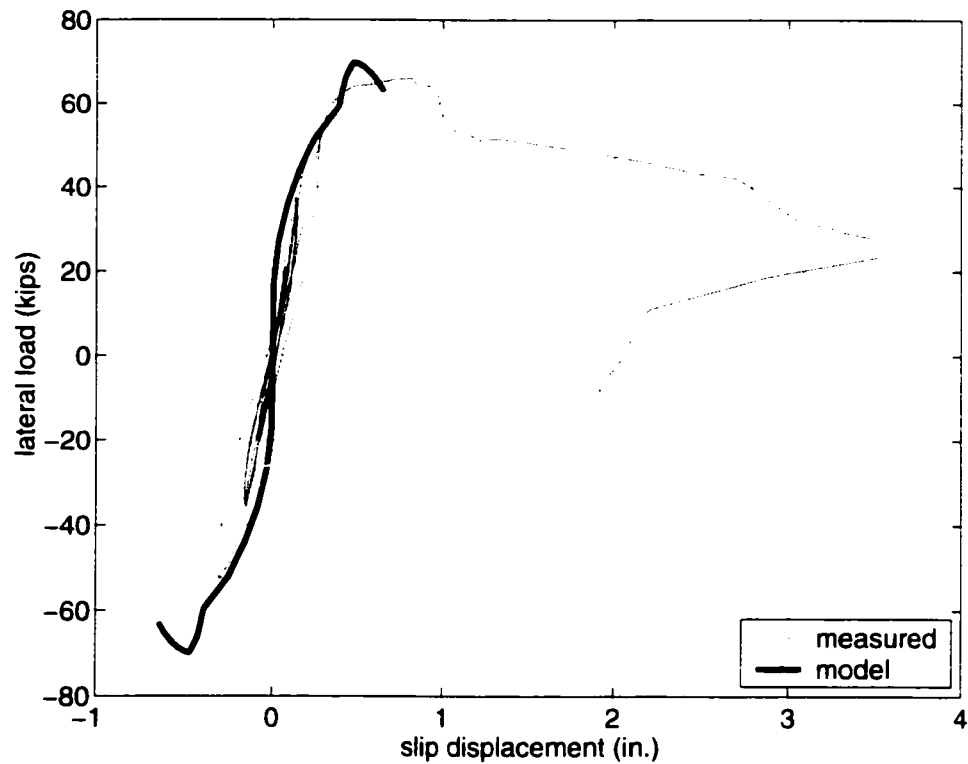


Figure 5.30 Lateral load-slip displacement relations (Specimen-4)

6 SHEAR STRENGTH EVALUATION

6.1 INTRODUCTION

For performance evaluation of existing buildings, it is important to identify and investigate the effectiveness of available models for shear strength of existing columns. Chapter 2 presented several shear strength models that were proposed and used for the design and evaluation of reinforced concrete columns. Most of these models calculate the column shear strength as the summation of the strength contributions from concrete and transverse reinforcement. Effects of various parameters such as axial load, displacement ductility, and column aspect ratio are represented differently or not included in some models.

The ACI 318-02 Building Code provides requirements for the design of new building components. The code requirements are seldom used by practicing engineers to evaluate shear strength of older existing buildings. However, it is of interest to understand and assess the ability of code provisions to predict the shear strength of existing columns. In addition to evaluation of ACI 318-02 column shear strength model (Section 2.3.1), the shear strength equations proposed by Priestley et al. (1994) (Section 2.4.7) are evaluated in this chapter. A database of 51 shear-critical column specimens with aspect ratios varying between 2.0 and 4.0 are used as a basis for assessing the shear strength equations. The columns included in the database are selected to meet the criteria including certain material, behavior and geometric properties.

An alternative shear strength model, which is a modification of the model reported by Moehle et al. (1999), is proposed and evaluated using the experimental data included in the database. The proposed model is also compared with other strength models. Both concrete and transverse steel contributions to the shear strength are related to displacement ductility in the proposed model.

6.2 TEST COLUMN DATABASE

A total of 51 test columns were selected for calibration of shear strength models. The test columns were selected because they met the following criteria: column aspect ratio or shear span to depth ratio, $2.0 \leq a/d \leq 4.0$; concrete strength, $2500 \leq f'_c \leq 6500$ psi; reinforcement nominal yield stress, $40 \leq f_y \leq 80$ ksi; longitudinal reinforcement ratio, $0.01 \leq \rho_l \leq 0.08$; transverse reinforcement, $0.01 f'_c \leq \rho_w f_{yw} \leq 0.12 f'_c$; cyclic lateral load

reversals; and apparent shear distress at failure. The behavior description and experimental lateral load-displacement relations for the selected columns are presented in Appendix A. The behavior columns tested by Lynn (2001) were described in Chapter 2. Most of the columns included in the database were tested using one of the test configurations shown in Figure 2.1. Although double-curvature column specimens are desirable for the evaluation of shear critical building columns, those specimens tested using other configurations were also included in the database but presented separately. Table 6.1 and Table 6.2 list relevant information for the test specimens included in the database.

The double-curvature, or Type-A, specimens had end conditions in which member end rotations were prevented. Zero end-rotation was provided through almost-rigid beam at the top and footing at the bottom of the test column. In addition, in Type-A specimens, the development or extension of shear cracks was not constrained around the midheight of the test columns. Unlike double-curvature specimens, in Type-B or cantilever specimens, the shear cracks developing near the tip of the column were constrained through a rigid-end-cap that is usually used to apply lateral loads. In Type-C and Type-D specimens, usually both ends of the specimen and the stubs in the middle were allowed to rotate freely during the tests (Figure 2.1). Although, it is not unrealistic to have some rotation in the slabs above and below the columns in a typical frame building, in most experiments, the rotation of the stubs was not reported and the effects of those rotations on the lateral deformations are unknown. Using a similar test setup, Wight and Sozen (1975) minimized the rotation of the stub by clamping the middle joint by a pair of hydraulic actuators as illustrated in Figure A.16 in Appendix A.

Table 6.2 and Table 6.3 show the details and material properties of test specimens included in the database. The moments and deformation characteristics of the specimens are included in Table 6.4 and Table 6.5. In these tables, b = width of column cross-section, d = distance from the extreme compression fiber to centroid of tension reinforcement (section depth), a = shear span (= total length of a cantilever column or half of the double-curvature column length), s = transverse reinforcement spacing, ρ_l = longitudinal reinforcement ratio, ρ_w = transverse reinforcement ratio, f_{yl} = longitudinal reinforcement yield strength, f_{yw} = transverse reinforcement yield strength, f_c = compressive strength of concrete, P = axial load, δ_y = yield displacement, and μ_δ = displacement ductility.

For the specimens with no reported yield displacement ($\delta_{y,test}$), the yield displacement was estimated by scaling the measured lateral load-displacement relations (Figure 6.1). A secant was drawn to intersect the lateral load-displacement relation at 70% of the maximum lateral load. This line was extended to the intersection with a horizontal line corresponding to the maximum lateral load, and then projected onto the horizontal axis to obtain the yield displacement ($\delta_{y,scal}$). For the columns tested in this study and the ones tested by Lynn et al. (1996), the ratio of the mean experimental yield displacement to yield displacement estimated from this method was 1.02 with a standard deviation of 0.22. As the method provides a reasonably good estimate of yield displacement, in this research, for the columns with no reported experimental yield displacement, the yield displacement is estimated using this method. The ultimate displacement, δ_u , reported in

the tables was defined as the maximum measured displacement at which the lateral load dropped to 80% of the maximum applied lateral load.

In Table 6.4 and Table 6.5, the measured maximum moments (M_{test}) are compared with the moments calculated from the ACI 318-02 procedure (M_{ACI}). The moments calculated using the ACI 318-02 procedure are based on the measured material properties, not specified f_y and f'_c as the ACI 318-02 code requires. The ACI 318-02 procedure underestimates the moments by more than ten percent on average. Comparison of calculated and measured moments indicates that almost all of the test columns reach their flexural strength before failure. The measured flexural overstrength may partly be attributable to strain hardening.

Figure 6.2 shows the relation between the shear strength and flexural strength of the specimens included in the database. The measured shear and moment strengths were normalized by the shear strengths and moments calculated using the ACI 318-02. It appears from the figure that while the vast majority of columns reach their flexural strength, about one half of them failed before reaching the calculated shear strength.

6.3 PROPOSED SHEAR STRENGTH MODEL

Evaluation of experimental data from the column database indicated that the column shear strength is influenced by several factors including the concrete strength, effective concrete area, column aspect ratio, axial load, and amount of transverse reinforcement. Recognizing the effect of these parameters, an alternative column shear strength model is proposed in this section. The proposed shear strength model is modified from the

model postulated in Moehle et al. (1999). Details of a similar version of this model are included in Lynn (2001).

The ASCE-ACI Committee 426 (1973) describes five basic shear transfer mechanisms in reinforced concrete members: shear transfer by uncracked concrete (V_{cz} in Figure 6.3), interface shear transfer in the cracked concrete, or aggregate interlock (V_a in Figure 6.3), dowel shear carried by the longitudinal and transverse reinforcement (V_d and V_s in Figure 6.3, respectively), and the arch action. As it is difficult to measure and model contributions of V_{cz} , V_{ay} and V_d separately, in column design calculations and strength predictions, these three components are commonly identified as the shear transfer by concrete and denoted by V_c . As summarized in Section 2.4, the majority of available column shear strength models estimate the shear strength as the summation of shear carried by concrete, V_c , and shear carried by transverse reinforcement, V_s . Similarly, the proposed shear strength equation includes contributions from concrete, V_c , and transverse reinforcement, V_s .

6.3.1 Concrete Contribution

Shear failure involves inclined cracking and crushing of concrete when subjected to biaxial state of stress due to combined shear and normal stresses resulting from applied lateral and axial loads (Figure 6.4). The shear strength of members is associated with the load required to develop inclined cracking. The inclined cracking is assumed to occur when the principal tensile stress of concrete (σ_1 in Figure 6.4) reaches its tensile strength. Consequently, the inclined cracking load or the shear force resulting in shear failure is a

function of the tensile strength of the concrete. Frequently, concrete tensile stress capacity, $f_t (= \sigma_t)$, is related to split cylinder tension tests and found to be approximately proportional to $\sqrt{f'_c}$ (Park and Paulay 1975, and MacGregor 1997). Thus, concrete tensile stress capacity is expressed as

$$f_t = C\sqrt{f'_c} \quad (6.1)$$

where f'_c = compressive strength of concrete, and C = a constant. For example, for strength calculations, the ACI 318-02 code suggests a C value of 6.0.

Statistical investigation of reinforced concrete beam/column experiments also indicates that one of the most important parameters affecting the shear strength at shear failure is the concrete tensile strength, f_t or $(f'_c)^{1/n}$. As reported in the ACI-ASCE Committee 426 report (1973), based on statistically derived equations, some researchers (e.g., Zsutty, 1968) suggest that the member shear strength is a function of $(f'_c)^{1/3}$.

In order to determine the concrete contribution to the shear strength of a reinforced concrete column with inclined cracks, the limiting shear stress in concrete has to be determined. The principal stresses of an uncracked section subjected to normal and shear stresses can be calculated from the following stress transformation equations.

$$\sigma_{1,2} = \frac{\sigma_x + \sigma_y}{2} \mp \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2} \quad (6.2)$$

where, σ_y = normal stress on plane perpendicular to the longitudinal axis of the member, σ_x = normal stress on plane parallel to the longitudinal axis of the member, τ_{xy} = shear stress on planes perpendicular to the transverse and longitudinal axes of the member, and σ_1 and σ_2 = principle stresses (Figure 6.4).

The shear stress, τ_{xy} , corresponding to the positive principle stress, σ_1 , can be expressed as

$$\tau_{xy} = \sqrt{\left(\sigma_1 - \frac{\sigma_x + \sigma_y}{2}\right)^2 - \left(\frac{\sigma_x - \sigma_y}{2}\right)^2} \quad (6.3)$$

Considering no normal stress applied in the direction perpendicular to the axial load direction ($\sigma_x = 0$), and assuming that the principle tension stress, σ_1 , is equal to tensile concrete stress, f_t , the shear stress (given as τ for simplicity of notations) can be calculated as

$$\begin{aligned} \tau &= \sqrt{\left(\sigma_1 - \frac{\sigma_y}{2}\right)^2 - \left(\frac{-\sigma_y}{2}\right)^2} = \sqrt{\sigma_1^2 - \sigma_1\sigma_y} = \sigma_1 \sqrt{1 - \frac{\sigma_y}{\sigma_1}} \\ \tau &= f_t \sqrt{1 - \frac{\sigma_y}{f_t}} \end{aligned} \quad (6.4)$$

Based on the discussion above, the tensile concrete stress capacity, f_t , is set equal to $6\sqrt{f'_c}$ (in psi units). If the axial stress is defined by $\sigma_y = -P/A_g$ (P is positive and σ_y is negative under compressive axial load), the shear stress, τ , can be rewritten as

$$\tau = 6\sqrt{f'_c} \sqrt{1 + \frac{P}{6\sqrt{f'_c}A_g}} \quad (6.5)$$

A similar shear stress equation was derived and used by Konwinski (1996), Moehle et al. (1999), and Lynn (2001) to predict the column shear strength.

The following subsections describe the parameters that could be affecting the shear strength of columns. The effect of each parameter is investigated using the column test data presented in the previous section.

6.3.1.1 Effect of Cross Section

The shear force carried by concrete, V_c , is related to the shear stress carried by concrete, τ , by the effective concrete section area, A_{eff} .

$$V_c = \tau A_{eff} \quad (6.6)$$

On the topic of the effective concrete area resisting the shear force, the ACI-ASCE Committee 426 report (1973) examines the effect of cross-sectional size, shape and depth of section, bond quality between longitudinal bars and concrete, and aggregate size on the shear strength. The ACI-ASCE Committee 426 (1973), ACI 318-02 (2002), and Aschheim and Moehle (1992) assume that the effective concrete area, A_{eff} , is equal to bd , where b and d are section width and depth of centroid of tension reinforcement, respectively. According to Aschheim and Moehle (1992), d may be taken equal to 0.8 times the section height, h , in the direction of the shear. Based on the average shear stress distribution shown in Figure 6.5, MacGregor (1997) suggests that A_{eff} is equal to bjd , where $j = 0.875$. In a similar way, the effective concrete area is assumed to be 80

percent and 85 percent of the gross cross-sectional area, A_g , by Priestley et al. (1994) and Konwinski (1996), respectively.

Based on the experimental data, the ACI-ASCE Committee 426 (1973) reports a larger strength reduction with increasing longitudinal bar diameter due to poorer bond quality between the concrete and longitudinal bars. In this investigation, prior to shear failure a large number of wide vertical cracks over the longitudinal bars were observed (see Figures 4.2, 4.7, 4.8, and 4.19). Most likely, these cracks resulted from weak bond action between the concrete and longitudinal bars, and reduced the effective concrete area resisting the shear force. During the tests, it appeared that under cyclic lateral loads, cover concrete was not very effective in resisting the lateral load after the development of bond cracks. Therefore, assuming that the cover concrete cannot resist shear force when the maximum lateral strength is reached, A_{eff} can be calculated as $b(h-d_c)$, where d_c = thickness of concrete cover over the longitudinal reinforcement. This is a reasonable assumption because the average shear stress within the concrete cover will be equal to zero as illustrated by the average shear stress distribution shown in Figure 6.5. Alternatively, under uni-directional lateral load, as the average shear stress within the concrete cover in the compression zone is relatively small (Figure 6.5), A_{eff} can be assumed to equal $b(h-2d_c)$.

The columns tested in this study had a cross section of 18 in. by 18 in., and 2 in. clear concrete cover over the longitudinal reinforcement. Thus, for the columns tested in this investigation, $A_{eff} = b(h-d_c) = 288 \text{ in.}^2 = \mathbf{0.89A_g}$, or $A_{eff} = b(h-2d_c) = \mathbf{0.77A_g}$. The ACI 318-02 design procedure approximates the effective concrete area of the test columns as

86% of the gross cross-sectional area ($A_{eff} = bd = \mathbf{0.86A_g}$). According to MacGregor (1997), $A_{eff} = bjd = 0.875bd = \mathbf{0.75A_g}$. According to Aschheim and Moehle (1992), $A_{eff} = bd = 0.80bh = \mathbf{0.80A_g}$. In this study, A_{eff} is set equal to 80% of the gross cross-sectional area, A_g . Thus, from Equations 6.5 and 6.6, V_c can be expressed as

$$V_c = 6\sqrt{f'_c} \sqrt{1 + \frac{P}{6\sqrt{f'_c}A_g}} 0.80A_g \quad (6.7)$$

6.3.1.2 Effect of Column Aspect Ratio

Figure 6.6 shows the relation between the normalized shear strength and column aspect ratio, a/d , for the test columns included in the database. The aspect ratio of the columns varied between 2.0 and 4.0. The figure shows a decrease in the shear strength with increasing aspect ratio within the range of data considered. A similar trend is reported by the ASCE-ACI Committee 426 (1973) for the shear-compression capacity of columns with shear-tension or shear-compression failures (Figure 6.7). In this study, the concrete contribution to the shear strength, V_c , is reduced by a/d as suggested by the trend in Figure 6.6.

$$V_c = \frac{6\sqrt{f'_c}}{a/d} \sqrt{1 + \frac{P}{6\sqrt{f'_c}A_g}} 0.80A_g \quad (6.8)$$

The reason behind this strength reduction is the interaction with flexural stresses and resulting flexural cracks leading to further strength reduction in columns with relatively large aspect ratios. Because the proposed equation is based on column test data with a/d

between 2.0 and 4.0, a/d is limited to between 2.0 and 4.0 in Equation 6.8. For the same range of column aspect ratio, a similar strength reduction was suggested by the ASCE-ACI Committee 426.

6.3.1.3 Effect of Axial Load

Figure 6.8 shows the relation between the normalized shear strength and the applied axial load, P , normalized with respect to the product of gross cross-sectional area and concrete strength. The trend of the data suggests that the shear strength increases with increasing axial load. Consistent with the test data, Equation 6.8 results in an increase in shear strength with increasing axial load.

6.3.1.4 Effect of Longitudinal Reinforcement

Figure 6.9 shows that the shear strength may be independent of yield strength of longitudinal reinforcement, f_{yt} , for the test specimens selected for comparison. Regarding the longitudinal reinforcement yield strength, a similar conclusion is drawn in the ASCE-ACI Committee 426 report (1973). The report proposes that the nominal shear stress carried by concrete, τ (Equation 6.6), is related to longitudinal reinforcement ratio as $\tau = (0.8+100\rho_l)\sqrt{f'_c} \leq 2\sqrt{f'_c}$ (Equation 2.6), where $\rho_l = A_{sl}/A_g$, and A_{sl} = total longitudinal reinforcement area. Figure 6.10 shows the relation between the normalized shear strength and the longitudinal reinforcement ratio. The figure indicates that the ASCE-ACI Committee 426 (1973) shear stress equation provides a lower bound for the test data included in this study. The flat line fitted to test data (Figure 6.10) suggests that the shear strength is independent of longitudinal reinforcement ratio.

6.3.2 Transverse Reinforcement Contribution

Several of shear strength procedures presented in Section 2.4 estimate the column shear strength, V_n , as the summation of contributions from concrete, V_c , and transverse reinforcement, V_s . For the test data considered in this study, the relation between the normalized shear strength and the transverse reinforcement parameter, $\rho_w f_{yw} / f'_c$, is plotted in Figure 6.11, where f_{yw} = yield strength of transverse reinforcement, and ρ_w = transverse reinforcement ratio ($\rho_w = A_s / bs$, A_s = transverse reinforcement area within a spacing, s , in the loading direction). Figure 6.11 indicates that the shear strength increases as the amount of transverse reinforcement increases. Based on the assumption that $V_n = V_c + V_s$, and that V_c is nearly constant and equal to the average strength of a column with no transverse reinforcement ($V_c \approx 3.6 \sqrt{f'_c} bd$), from Figure 6.11 the transverse reinforcement contribution, V_s , can be calculated as

$$V_s = \alpha \frac{A_s f_{yw} d}{s} \quad (6.9)$$

where α is the slope of the line fitted to the test data (dashed line in Figure 6.11). IT should be noted that the variation in V_c may be large, and V_c may increase as the transverse reinforcement ratio increases resulting in slightly better confinement of core concrete.

The transverse reinforcement contribution, V_s , can be derived using a simple truss model assuming that shear is carried jointly by concrete in compression and transverse

reinforcement in tension in a single mechanism. Accordingly, Equation 6.10 is derived for the shear strength (see Aschheim and Moehle 1992, and MacGregor 1997)

$$V_s = \frac{A_v f_{yw} d}{s \tan \theta} \quad (6.10)$$

where θ is the average shear crack angle. For the 51 test specimens used in this study, the average crack angle is calculated to be 55 degrees ($\theta = \tan^{-1}(1/\alpha)$ from Equations 6.9 and 6.10, and $\alpha = 0.69$ from Figure 6.11). Most shear strength models (see Section 2.4) use Equation 6.10 with $\theta = 45$ degrees, which corresponds to $\alpha = 1.0$ in Figure 6.11 and is based on the assumption of approximate initial shear crack angle of 45 degrees. Thus, Equation 6.10 becomes

$$V_s = \frac{A_v f_{yw} d}{s} \quad (6.11)$$

6.3.2.1 Effect of Lateral Drift Capacity

The influence of the transverse reinforcement parameter, $\rho_w f_{yw}/f'_c$, and axial load on the lateral displacement drift at ultimate displacement is illustrated in Figure 6.12. There is a tendency of increase in the lateral drift capacity with decreasing axial load. A similar conclusion was drawn based on the response of Specimen-3 tested in this research (Figure 4.21). Higher axial load leads to larger shear strength (Section 6.3.1.3 and Figure 6.8), and smaller transverse reinforcement ratio tends to result in smaller shear strength (Section 6.3.2 and Figure 6.11). In addition to magnitude of axial load and amount of transverse reinforcement, other factors such as yielding and strain hardening of the

longitudinal reinforcement affect the column shear failure and resulting lateral drift. Therefore, it is not easy to predict a relation between the shear strength and lateral drift capacity at shear failure.

It should also be noted that the measured drift is directly related to the lateral loading history. A larger drift capacity is expected in columns subjected to monotonic loading or less number of loading cycles. As the number of loading cycles increases, the strength degradation and final failure are expected to occur earlier (compare the response of Specimen-1 and Specimen-4; Figure 4.28). The effect of loading history is not investigated in this chapter.

6.4 EFFECT OF DISPLACEMENT DUCTILITY DEMAND ON SHEAR STRENGTH

Some researchers (e.g., by Aschheim and Moehle 1992, and Priestley et al. 1994) reduced concrete contribution to shear strength as a function of displacement ductility using strength reduction parameters based on experimental data from ductile columns (see Section 2.4). Using the experimental data from ductile and nonductile columns, Konwinski et al. (1996) concluded that the column shear strength was independent of displacement ductility. In this section, the relation between the displacement ductility demand and column shear strength is investigated. Most specimens considered in this study had limited displacement ductility with an average displacement ductility of 3.35.

Provided that the transverse reinforcement is of sufficient size, well anchored with sufficiently long 135-degree end hooks, and spaced closely, it carries shear in tension (V_s

in Figure 6.3), restricts the growth of inclined cracks and thus helps prevent degradation of interface shear (V_{ay} in Figure 6.3), holds longitudinal bars and thereby prevents bar buckling and increases V_d (Figure 6.3), and tends to increase V_{cz} (Figure 6.3) by confining the concrete with resulting increase in depth and strength of the compression zone. As observed in this research, if the transverse reinforcement is not well anchored or closely spaced, the longitudinal bars tend to buckle and end hooks tend to open causing significant damage (Figures 4.5, 4.9, 4.10, 4.12, 4.16, and 4.17). The consequent column failure may also include loss of interface shear resistance, and dowel splitting associated with weak bond action between the longitudinal bars and concrete (e.g., Figure 4.10).

As illustrated in Figure 6.13, prior to flexural cracking, all shear in the column is carried by the uncracked concrete. The transverse reinforcement starts resisting the shear after the development of inclined cracks. Prior to formation of inclined cracks, the strain in the transverse reinforcement is small, and equal to the strain of the concrete. The transverse reinforcement strain measurements from the specimens tested in this research confirm that the strains (hence tension) in the transverse reinforcement were relatively small prior to inclined cracking (see Figures E.4, E.8, E.12, and E.16 in Appendix E). For example, in Specimen-1 and Specimen-4, the inclined cracks started to form during the yield displacement cycles ($\Delta_y = 1.1$ in., after the time step 860 in Figure E.4). Note that the inclined thin/hairline cracks observed during the early stages of the tests are defined as flexural cracking by the ACI-ASCE Committee 426 (Figure 6.13). Prior to inclined cracking, i.e., during the $\Delta_y/4$ and $\Delta_y/2$ displacement cycles, in most cases the

strains were negligibly small as compared with the strains during the rest of the tests. Figure 6.13 implies that the transverse reinforcement contribution, V_s , increases with increasing shear and stays constant after yielding of the well anchored and closely spaced transverse reinforcement. Similarly, experimental evidence from this research suggests that V_s , which may be assumed to be a function of strain and stress in the transverse reinforcement (Park and Paulay 1975, and Priestley et al. 1996), increases with increasing shear until the transverse reinforcement yields. However, after yielding, it seems that V_s decreases with increasing number of inclined cracks, extension of cracks, bending of column ties along the inclined cracks, and other damage such as bond cracking in the column. Such damage also tends to reduce the effective section depth, d , resulting in a smaller V_s (Equation 6.11). Note that, unlike well anchored and closely spaced transverse reinforcement considered in Figure 6.13, in this research, the transverse reinforcement was widely spaced and had 90-degree end-hooks. In addition, as in this research, columns with large diameter longitudinal reinforcing bars and relatively low strength concrete are susceptible to bond cracking. In this research, 1½ in.-diameter longitudinal reinforcing bars were used with concrete with an average compressive strength of 3077 psi.

When Specimen-1 reached its peak lateral strength, the transverse reinforcement yielded near the top and bottom of the column where a large number of flexural and inclined cracks were observed (Figure 4.2 and Figure 4.40, Strain Gage-F2, F3, F6 in Figure E.4). Figure 6.13 implies that, after yielding, V_s should be constant while dowel splitting, loss of interface shear, and other damage to concrete are observed. However, when

Specimen-1 failed in shear (Figure 4.3 through Figure 4.5), the transverse reinforcement did not seem to be very effective in resisting the shear and confining the core concrete.

Similarly, before Specimen-2 lost its lateral-load-carrying capacity in a brittle manner, the transverse reinforcement yielded (Figure 4.40 and Figure E.8). Unlike the pre-failure behavior presented in Figure 6.13 where V_{ay} decreases after yielding, when the inclined cracks widened the transverse reinforcement was not able to resist shear or prevent longitudinal bar buckling, because the 90-degree tie hooks opened (Figure 4.8 through Figure 4.12). It appears that the decrease in V_{ay} (and hence V_c) was associated with a decrease in V_s in this specimen as part of an interaction mechanism between concrete and transverse reinforcement.

A similar behavior including transverse reinforcement yielding followed by longitudinal bar buckling and column tie opening was observed in Specimen-3 (Figure 4.17). In this specimen, after the bar buckling and tie opening, the lateral strength reduction was about 15 percent under low axial load (i.e., strength reduction from 55 kips to 44 kips in Figure 4.21). It seems that this strength reduction should be reflected in both V_s and V_c . Because the open column tie shown in Figure 4.21 was not effective in resisting shear, which leads to a reduction in V_s . Similarly, when the strength reduction was observed during this cycle, portion of the concrete including side cover spalled off near that column tie was not effectively resisting shear causing a reduction in V_c .

Figure 4.24 shows the damage progress in Specimen-4 shortly after the peak lateral load was reached. Figures 4.25 and 4.26 show that, when the lateral-load-carrying capacity

was lost, the column ties crossing the inclined crack opened, and the longitudinal bars buckled and bent along the crack. The growth of the inclined crack width (Figure 4.24) and the corresponding strength degradation under monotonic loading (Figure 4.27) appear to increase in parallel with the damage to the longitudinal and transverse reinforcement crossing the crack and deterioration in concrete along the crack.

These observations and experimental results suggest that, after the transverse reinforcement yielding, more and wider inclined cracks cross the transverse reinforcement. Unlike the behavior of well detailed and closely spaced transverse reinforcement (Figure 6.13), as concrete deteriorates, V_s decreases with increasing lateral displacement if the transverse reinforcement with 90-degree end hooks is widely spaced. Such transverse reinforcement becomes less effective in carrying the shear (and eventually opens) following spalling of cover concrete near the 90-degree end hooks. Thus, in this research, it is proposed that the steel contribution to the shear strength, V_s , decreases with increasing lateral displacement.

Similarly, in addition to reasons presented in this section such as concrete spalling near column tie hooks, since the aggregate interlock or interface shear transfer along the inclined cracks (V_a in Figure 6.3) is reduced with increasing displacement as suggested in the ASCE-ACI Committee 426 report (1973), it is also proposed that the concrete contribution to the shear strength, V_c , decreases with increasing lateral displacement. Note that, contrary to what is illustrated in Figure 6.13, in shear critical columns the contribution of V_a (hence V_{ay} along the shear crack) to V_c will be considerably larger than that of V_{cz} indicating a significant reduction in V_c near shear failure. This is because

under cyclic loading, shear cracks usually extend between the longitudinal bars on each end of the cross-section including the confinement region where V_{cz} acts (Chapter 4).

Shear strengths of the 51 test columns included in the database (Table 6.2 and Table 6.3) are calculated as the summation of contributions of concrete (Equation 6.8) and transverse reinforcement (Equation 6.11). The ratios of measured-to-calculated shear strengths are plotted against the measured displacement ductility in Figure 6.14. The trend of the plotted data (dashed line) suggests that the ratio of measured to calculated shear strength decreases with increasing displacement ductility. As shown in the figure, the trend can be represented by three solid lines: two horizontal lines at ratios of 1.0 and 0.7 for displacement ductilities less than 2.0 and larger than 6.0, respectively; and a third linear line connecting these two horizontal lines.

Based on the discussion in this section and the trend identified in Figure 6.14, the proposed shear strength equation is related to displacement ductility through a parameter, k , shown in Figure 6.15. Then, from Equations 6.8 and 6.11, the proposed shear strength equation can be expressed as

$$V_n = k (V_c + V_s) = k \left(\frac{6\sqrt{f'_c}}{a/d} \sqrt{1 + \frac{P}{6\sqrt{f'_c} A_g}} \right) 0.80 A_g + k \frac{A_s f_{yw} d}{s} \quad (6.12)$$

The parameter, k , applied to V_s term, which is based on empirical data, could be related to physical behavior through the parameter, α , shown in Figure 6.11. Note that in Section 6.3.2, α was calculated as 0.7 and 1.0 based on average shear crack angle for the test columns in the database (55-degrees) and 45-degree crack angle, respectively.

Based on the experimental data, a parameter similar to k was proposed and applied to concrete contribution to the shear strength, V_c , by Aschheim and Moehle (1992) (Section 2.3.4), Priestley et al. (1994) (Section 2.3.7), and Moehle et al. (1999). Konwinski (1996) (Section 2.4.9) and Moehle et al. (1999) modified the ACI 318 equation for transverse steel contribution (Equation 2.3) using the experimental data from ductile and nonductile columns, respectively. Konwinski et al. (1996) reduced the steel contribution calculated from the ACI 318-02 by fifteen percent.

Figure 6.16 plots the ratio of measured shear strengths to shear strengths calculated from Equation 6.12. Comparison of Figure 6.14 and Figure 6.16 indicates an improvement in calculated shear strengths especially for larger displacement ductilities. For columns with larger displacement ductilities, the shear strength is consistently overestimated when the displacement ductility factor, k , is not included (Figure 6.14).

6.5 SHEAR STRENGTH EVALUATION

6.5.1 Statistical Variations and Implications

For design and assessment of columns, it is important to be able to estimate the shear strength with some certainty. Even though the shear strengths calculated from the proposed model (Equation 6.12) were slightly conservative with some scatter, for design and assessment purposes, a lower bound is sought in estimating the shear strength based on a reliable measure. This objective is pursued using a statistical approach here. Normal distribution and a histogram of the ratio of the measured strengths to the strengths calculated from the proposed model are plotted in Figure 6.17. If the data corresponds to

normal distribution, 68 percent of the data will lie within one standard deviation above or below the mean ($0.89 \leq V_{test}/V_{proposed} \leq 1.21$ in Figure 6.17). For a normal distribution, 10 percent of the data, or 1 test column in 10, will have measured to calculated strength ratios less than λ_{90} . $\lambda_{90} = \mu(1-1.282 \cdot cov)$, where cov is the coefficient of variation ($cov = \sigma/\mu$), μ is the mean value, and σ is the standard deviation. For the 51 test columns considered, the mean ratio of measured-to-calculated shear strengths is 1.05 with a standard deviation of 0.16 and a coefficient of variation of 0.15. Thus, no more than 1 test column in 10 will have a ratio of measured to calculated shear strength less than 0.85 ($\lambda_{90} = 0.85$). In other words, 90 percent of the test columns will have a measured to calculated strength ratio more than 0.85, and will be on the right hand side of the vertical dashed line ("90% limit line") shown in Figure 6.17. Therefore, given the statistical variations identified by the mean ratio and standard deviation, based on a normal distribution and "90% limit" exceedence criteria, for design and assessment of columns, a strength reduction factor, ϕ_{90} , of 0.85 is proposed (i.e., $\phi_{90} \approx \lambda_{90}$). This is based on the assumption that the mean ratio of measured to calculated shear strengths, μ , is close to 1.0.

Implications of statistical variations and uncertainty involved in shear strength calculations are illustrated in Figure 6.18. Similar to the ATC (1983) demand-capacity relation depicted in Figure 3.1, when the demand curve crosses the shear capacity curve, the shear strength is assumed to start degrading with increasing displacement ductility. The shear capacity curve, solid line shown in the middle in Figure 6.18, is calculated from the proposed model (Equation 6.12) as a function of displacement ductility

demand. Probability of shear strength of a column will be smaller or larger than the shear strength calculated from the proposed model (i.e., below or above the solid line) is approximately 50 percent. As demonstrated in Figure 6.17, this is based on the assumption that the measured shear strengths can be estimated reasonably well (mean strength ratio, $\mu \approx 1.0$), and the data correspond to normal distribution. If the column shear demand can be calculated as shown in Figure 6.18, there is 50 percent probability ($p_2=0.50$) that the maximum column shear strength will be V_{n2} , and the displacement ductility will be $\mu_{\delta 2}$ when the strength degradation starts. The dashed area under the normal distribution curve, p_2 , will be equal to 0.50 if the total area under the curve is 1.00.

If one wants to calculate the shear strength with more certainty, a strength reduction factor, ϕ_n , could be applied to the proposed model. The shear capacity curve corresponding to the “90% limit” in Figure 6.18 is obtained using a strength reduction factor, ϕ_{90} , of 0.85 (i.e., Equation 6.12 is multiplied by ϕ_{90}). The shear capacity curve corresponding to the “90% limit” implies that there is 90 percent chance (dashed area, $p_1= 0.10$) that the column shear strength will be equal to or larger than V_{n1} , when strength degradation starts due to shear failure at a displacement ductility of $\mu_{\delta 1}$. Similarly, “10% limit” capacity curve indicates that there is 10 percent chance ($p_3 = 0.90$) that the column shear strength will be equal to or larger than V_{n3} , and the displacement ductility will be $\mu_{\delta 3}$ at shear failure. The “10% limit” capacity curve is based on normal distribution with a strength reduction factor, or rather strength amplification factor, ϕ_{10} , of 1.15.

The model illustrated in Figure 6.18 is intended mainly for column shear strength calculations. If the demand curve cannot be calculated reliably or is not available, for a given displacement ductility demand, column shear strength at shear failure can be estimated from the proposed shear capacity curve reasonably well. This is because the increase in shear strength is usually limited after flexural yielding. Figure 6.18 demonstrates that for the range of displacement ductilities between $\mu_{\delta 1}$ and $\mu_{\delta 3}$, the variation in the shear strength (i.e., between V_{n1} and V_{n3}) is not large. Therefore, even if the measured displacement ductility is significantly smaller or larger than the predicted ductility demand, the shear strength can be estimated reasonably accurately. However, the proposed model should not be used to estimate the displacement ductility demand for a given shear strength, because a small variation in shear strength corresponds to a large difference in displacement ductility.

Figure 6.19 compares the response of Specimen-1 tested in this research with the shear strength capacity curves calculated from the proposed model. The column shear demand (thickest solid line in the figure) is obtained by connecting the measured peak lateral loads at each displacement level. The plot indicates that the shear strength can be calculated reasonably well from the proposed model (Equation 6.12) without using a strength reduction factor.

Figure 6.20 compares the lateral load-displacement ductility relations calculated from the proposed equation and the measured relations from the four specimens tested in this research. The lateral strengths, V_p ($= 2M_p/L$), required to develop plastic hinges at column ends are also shown in the figure. It should be noted that the calculated shear

strength of each test column is a single point on the shear strength envelope corresponding to the measured displacement ductility at shear failure. As an example, the calculated shear strength of Specimen-1 (64 kips) is indicated by a circle in Figure 6.20. The plots imply that a lower bound shear capacity curve such as the “90% limit” curve, which is based on a statistical investigation suggesting a strength reduction factor Φ_{90} , provides a more conservative estimate of shear strength that could be used for design and assessment purposes.

6.5.2 Comparison of Models with Column Database

Along with the shear strengths calculated from the proposed model (Equation 6.12) and shear strengths corresponding to the flexural capacity (V_p), Table 6.6 and Table 6.7 show the measured shear strengths and the strengths calculated from the ACI 318-02 equation (Section 2.4.1), and the equation proposed by Priestley et al. 1994 (Section 2.4.7). In Equation 2.21 (which was proposed by Priestley et al. (1994) to calculate the axial load component, V_p), the neutral axis depth, c , was calculated using the ACI 318-02 flexural design procedure under the given axial load using the measured material properties.

The mean measured-to-calculated strength ratio and the standard deviation are 1.05 and 0.16, respectively, for the proposed strength equation. The mean ratio is 1.01 with a standard deviation of 0.21 for the ACI strength equation. Both models do reasonably well in modeling shear strength, however, it is noted that the model by Priestley et al. tends to overestimate the shear strength with a mean ratio of 0.76 and standard deviation of 0.15. As demonstrated in Figure 6.21, it appears that this is mainly because the

transverse steel contribution to shear strength, V_s , is overestimated by Priestley et al. For approximately 20 percent of the test columns included in the database, the measured shear strength is smaller than the steel contribution, V_s , calculated from the model by Priestley et al. suggesting that the concrete contribution, V_c , should be negligible. In fact, V_s from the Priestley et al. model is approximately 70 percent larger than the V_s from the ACI 318-02 method. This is mainly because Priestley et al. (1994) assumes a truss mechanism with a 30-degree crack angle (Equation 2.21).

The ratio of measured to calculated shear strengths from the proposed equation, $V_{proposed}$, the ACI 318-02 method, V_{ACI} , and the method proposed by Priestley et al. (1994), $V_{Priestley}$, are plotted against the measured displacement ductility in Figure 6.22. Also shown in the figure are the strength reduction factors, ϕ_{90} , proposed for design and assessment of columns. Based on a normal distribution with the probability of 90 percent of the measured column strengths exceeding the calculated strengths (i.e., “90% limit” exceedence criterion), the proposed strength reduction factors are calculated as 0.85, 0.74 and 0.56, for the proposed, ACI 318-02, and Priestley et al. (1994) models, respectively. Since strengths predicted by ACI 318-02 and Priestley et al. have to be multiplied by a smaller factor to obtain 90 percent reliability, these models tend to overestimate the shear strength more.

Figure 6.23 shows the ratio of measured to calculated shear strengths versus the column aspect ratio according to three methods considered. It appears that for columns with aspect ratios between 2.0 and 4.0, there is no systematic variation in calculated shear strengths for different aspect ratios. Note that the concrete contribution to the shear

strength was reduced by the aspect ratio in the proposed model, and the aspect ratio was included indirectly in the V_p term (Equation 2.20) in the method proposed by Priestley et al. (1994).

Figure 6.24 and Figure 6.25 show the ratio of measured to calculated shear strengths versus the axial load ratio, $P/(A_g f'_c)$, and the transverse reinforcement ratio, $\rho_w f_{yw}/f'_c$, respectively. The correlation between the measured and calculated data indicates that the effect of axial load and transverse reinforcement are represented by these methods relatively well. The models are relatively consistent across the range of these two parameters.

Table 6.1 Test setup and boundary conditions for specimens in the database

Test setup	Type-A	Type-B	Type-C	Type-D
constrained shear crack	no	yes	yes	yes
clean end conditions	yes	no	no	no
specimens tested by	Sezen, Moehle Lynn, Moehle Ohue et al. Esaki	Li et al.	Ikeda Umemuro, Endo Kokusho Wight, Sozen	Umemuro, Endo (#220, 231,232,233, 234)

Table 6.2 Details and material properties for Type-A specimens

Specimen	b	d	a	s	ρ_l	ρ_w	f_{vt}	f_{vw}	f'_c
	in.	in.	in.	in.			ksi	ksi	ksi
Sezen and Moehle (2002)									
2CLD12	18.0	15.5	58.0	12.0	0.025	0.0017	64	68	3.06
2CHD12	18.0	15.5	58.0	12.0	0.025	0.0017	64	68	3.06
2CVD12	18.0	15.5	58.0	12.0	0.025	0.0017	64	68	3.03
2CLD12M	18.0	15.5	58.0	12.0	0.025	0.0017	64	68	3.16
Lynn and Moehle (1996)									
3CLH18	18.0	15.0	58.0	18.0	0.030	0.0010	48	58	3.71
3SLH18	18.0	15.0	58.0	18.0	0.030	0.0010	48	58	3.71
2CLH18	18.0	15.0	58.0	18.0	0.020	0.0010	48	58	4.80
2SLH18	18.0	15.0	58.0	18.0	0.020	0.0010	48	58	4.80
2CMH18	18.0	15.0	58.0	18.0	0.020	0.0010	48	58	3.73
3CMH18	18.0	15.0	58.0	18.0	0.030	0.0010	48	58	4.01
3CMD12	18.0	15.0	58.0	12.0	0.030	0.0017	48	58	4.01
3SMD12	18.0	15.0	58.0	12.0	0.030	0.0017	48	58	3.73
Ohue, Morimoto, Fujii, and Morita (1985)									
2D16RS	7.87	6.89	15.7	1.97	0.020	0.0057	54	46	4.65
4D13RS	7.87	6.89	15.7	1.97	0.027	0.0057	54	46	4.34
Esaki (1996)									
H-2-1/5	7.87	6.89	15.7	1.97	0.025	0.0052	52	53	3.34
HT-2-1/5	7.87	6.89	15.7	2.95	0.025	0.0052	52	53	2.93
H-2-1/3	7.87	6.89	15.7	1.57	0.025	0.0065	52	53	3.34
HT-2-1/3	7.87	6.89	15.7	2.36	0.025	0.0065	52	53	2.93

Table 6.3 Details and material properties for other specimens

Specimen	b	d	a	s	ρ_l	ρ_w	f_{vt}	f_{vw}	f'_c
	in.	in.	in.	in.			ksi	ksi	ksi
Li, Park, and Tanaka (1995)									
U-7	15.8	14.8	39.4	4.7	0.024	0.0047	64.7	55.4	4.21
U-8	15.8	14.8	39.4	4.7	0.024	0.0052	64.7	55.4	4.86
U-9	15.8	14.8	39.4	4.7	0.024	0.0057	64.7	55.4	4.95
Saatcioglu and Ozcebe (1989)									
U1	13.8	12.0	39.4	5.9	0.033	0.0030	62.4	68.2	6.32
U2	13.8	12.0	39.4	5.9	0.033	0.0030	65.7	68.2	4.38
U3	13.8	12.0	39.4	3.0	0.033	0.0060	62.4	68.2	5.05
Yalcin (1997)									
BR-S1	21.7	19.0	58.5	11.8	0.020	0.0010	64.5	61.6	6.50
Ikeda (1968)									
43	7.87	6.81	19.7	3.9	0.02	0.0028	63	81	2.84
44	7.87	6.81	19.7	3.9	0.02	0.0028	63	81	2.84
45	7.87	6.81	19.7	3.9	0.02	0.0028	63	81	2.84
46	7.87	6.81	19.7	3.9	0.02	0.0028	63	81	2.84
62	7.87	6.81	19.7	3.9	0.02	0.0028	50	69	2.84
63	7.87	6.81	19.7	3.9	0.02	0.0028	50	69	2.84
64	7.87	6.81	19.7	3.9	0.02	0.0028	50	69	2.84
Umemuro and Endo (1970)									
205	7.87	7.09	23.6	3.9	0.02	0.0028	67	47	2.55
207	7.87	7.09	15.8	3.9	0.02	0.0028	67	47	2.55
208	7.87	7.09	15.8	3.9	0.02	0.0028	67	47	2.55
214	7.87	7.09	23.6	7.9	0.02	0.0014	67	47	2.55
220	7.87	7.09	15.8	4.7	0.01	0.0011	55	94	4.77
231	7.87	7.09	15.8	3.9	0.01	0.0013	47	76	2.14
232	7.87	7.09	15.8	3.9	0.01	0.0013	47	76	1.90
233	7.87	7.09	15.8	3.9	0.01	0.0013	54	76	2.02
234	7.87	7.09	15.8	3.9	0.01	0.0013	54	76	1.90
Kokusho (1964)									
372	7.87	6.69	19.7	3.9	0.01	0.0031	76	51	2.88
373	7.87	6.69	19.7	3.9	0.02	0.0031	76	51	2.96
Kokusho and Fukuhara (1965)									
452	7.87	6.69	19.7	3.9	0.03	0.0031	52	88	3.18
454	7.87	6.69	19.7	3.9	0.04	0.0031	52	88	3.18
Wight and Sozen (1973)									
40.033a	6.00	10.5	34.5	5.0	0.024	0.0033	72	50	5.03
40.033	6.00	10.5	34.5	5.0	0.024	0.0033	72	50	4.87
25.033	6.00	10.5	34.5	5.0	0.024	0.0033	72	50	4.88
00.033	6.00	10.5	34.5	5.0	0.024	0.0033	72	50	4.64
40.048	6.00	10.5	34.5	3.5	0.024	0.0048	72	50	3.78
00.048	6.00	10.5	34.5	3.5	0.024	0.0048	72	50	3.75

Table 6.4 Moments and deformation characteristics of specimens tested in double bending

Specimen	P	$\delta_{y,test}$	$\delta_{y, scale}$	$\delta_{y,test}/\delta_{y, scale}$	δ_u	μ_δ	M_{test}	M_{ACI}	M_{test}/M_{ACI}
	kips	in.	in.		in.		k-in.	k-in.	
Sezen and Moehle (2002)									
2CLD12	150	1.03	1.04	0.99	2.97	2.88	4320	3870	1.12
2CHD12	600	0.79	0.57	1.39	1.02	1.29	5100	3720	1.37
2CVD12	500*	0.82	0.76	1.08	2.23	2.72	4740	3670	1.29
	-56	1.13	1.22	0.93	3.41	3.01	3340	3060	1.09
2CLD12M	150	1.06	1.11	0.96	3.33	3.14	4110	3900	1.05
Lynn and Moehle (1996)									
3CLH18	113	0.75	0.78	0.96	1.20	1.58	3970	3780	1.05
3SLH18	113	0.62	0.61	1.02	1.15	1.69	3670	3780	0.97
2CLH18	113	0.59	0.72	0.82	3.00	4.17	3280	2820	1.17
2SLH18	113	0.51	0.63	0.81	2.40	2.65	4020	2820	1.43
2CMH18	340	0.65	0.61	1.07	1.20	1.94	4500	3490	1.29
3CMH18	340	0.89	0.61	1.46	1.20	2.14	4300	4460	0.96
3CMD12	340	0.77	0.74	1.04	1.80	2.50	4980	4460	1.12
3SMD12	340	0.89	0.86	1.04	1.80	2.73	5230	4340	1.20
Ohue, Morimoto, Fujii, and Morita (1985)									
2D16RS	41.1	-	0.30	-	1.08	1.74	360	308	1.17
4D13RS	41.1	-	0.26	-	0.58	2.42	390	366	1.07
Esaki (1996)									
H-2-1/5	36.2	-	0.16	-	0.79	4.94	364	303	1.20
HT-2-1/5	31.8	-	0.19	-	0.82	4.32	360	287	1.25
H-2-1/3	60.4	-	0.14	-	0.63	4.50	425	326	1.30
HT-2-1/3	53.0	-	0.19	-	0.79	4.16	394	306	1.29

Table 6.5 Moments and deformation characteristics of other specimens

Specimen	P	$\delta_{y, \text{test}}$	δ_u	μ_δ	M_{test}	M_{ACI}	$M_{\text{test}}/M_{\text{ACI}}$
	kips	in.	in.		k-in.	k-in.	
Li, Park, and Tanaka (1995)							
U-7	104	0.35	1.40	4.00	2900	2430	1.19
U-8	241	0.33	0.83	2.50	3480	2980	1.16
U-9	368	0.30	1.20	4.00	3800	3200	1.19
Saatcioglu and Ozcebe (1989)							
U1	0	0.67	2.09	3.12	2430	2060	1.18
U2	135	0.59	1.69	2.87	2390	2540	0.94
U3	135	0.63	1.77	2.81	2370	2470	0.96
Yalcin (1997)							
BR-S1	469	0.32	0.91	2.88	7610	8280	0.92
Ikeda (1968)							
43	18	0.13	0.52	4.13	327	272	1.20
44	18	0.13	0.32	2.56	338	272	1.24
45	35	0.19	0.32	1.74	364	312	1.17
46	35	0.19	0.24	1.26	356	312	1.14
62	18	0.12	0.73	5.96	256	259	0.99
63	35	0.12	0.55	4.00	303	299	1.01
64	35	0.14	0.66	4.82	303	299	1.01
Umemura and Endo (1970)							
205	35	0.19	0.49	2.51	378	358	1.06
207	35	0.16	0.25	1.60	375	358	1.05
208	88	0.16	0.47	2.99	478	339	1.41
214	88	0.24	0.41	1.73	439	339	1.30
220	35	0.06	0.47	7.83	277	226	1.23
231	35	0.04	0.32	8.42	180	183	0.98
232	35	0.05	0.32	6.40	206	177	1.16
233	35	0.06	0.27	4.50	245	217	1.13
234	35	0.06	0.32	5.33	237	214	1.11
Kokusho (1964)							
372	35	0.10	0.42	4.12	329	263	1.25
373	35	0.14	0.39	2.78	390	350	1.11
Kokusho and Fukuhara (1965)							
452	88	0.12	0.30	2.53	487	400	1.22
454	88	0.09	0.20	2.32	487	486	1.00
Wight and Sozen (1973)							
40.033aE	42.5	0.30	1.25	4.19	742	711	1.04
40.033E	40.0	0.48	1.73	3.62	756	703	1.08
25.033E	25.0	0.47	1.24	2.65	680	650	1.05
00.033E	0	0.30	1.10	3.67	628	556	1.13
40.048W	40.0	0.57	1.91	3.38	814	685	1.19
00.048W	0	0.53	1.30	2.45	742	543	1.37

Estimated (scaled) values in italics

Table 6.6 Calculated and measured shear strengths

Specimen	V_{test}	$V_{proposed}$	$V_{test} / V_{proposed}$	V_{ACI}	V_{test} / V_{ACI}	$V_{Priestley}$	$V_{test} / V_{Priestley}$	V_p	V_{test} / V_p
	kips	kips		kips		kips		kips	
2CLD12	70.8	64.0	1.11	71.0	1.00	95.5	0.74	66.7	1.06
2CHD12	80.7	92.0	0.88	92.4	0.87	118.	0.68	64.1	1.26
2CVD12C	67.6	82.8	0.82	87.4	0.77	101.	0.67	63.3	1.07
2CVD12T	55.5	52.6	1.07	63.7	0.87	81.3	0.68	52.8	1.05
2CLD12M	66.2	63.1	1.05	71.6	0.92	91.7	0.72	67.2	0.99
3CLH18	61.0	49.9	1.22	54.3	1.12	86.1	0.71	65.2	0.94
3SLH18	60.0	49.9	1.20	54.3	1.11	86.1	0.70	65.2	0.92
2CLH18	54.0	44.7	1.21	59.6	0.91	54.4	0.99	48.5	1.11
2SLH18	52.0	50.8	1.02	59.6	0.87	82.2	0.63	48.5	1.07
2CMH18	71.0	63.9	1.11	65.9	1.08	97.9	0.73	60.1	1.18
3CMH18	76.0	64.4	1.18	67.8	1.12	98.8	0.77	76.8	0.99
3CMD12	80.0	73.8	1.08	79.4	1.01	107.	0.75	76.8	1.04
3SMD12	85.0	71.4	1.19	77.5	1.10	101.	0.84	74.8	1.14
2D16RS	22.9	28.6	0.80	24.1	0.95	39.2	0.58	19.6	1.17
4D13RS	24.9	27.4	0.91	23.7	1.05	37.1	0.67	23.3	1.07
H-2-1/5	23.2	21.3	1.09	23.0	1.01	30.6	0.76	19.3	1.20
HT-2-1/5	22.9	21.7	1.05	22.3	1.03	29.6	0.77	18.3	1.25
H-2-1/3	27.1	27.1	1.00	28.0	0.97	37.9	0.72	20.8	1.31
HT-2-1/3	25.1	26.9	0.93	27.1	0.93	36.7	0.68	19.5	1.29
U-7	73.7	87.1	0.85	97.3	0.76	128.	0.58	61.7	1.20
U-8	88.3	119.	0.74	115.	0.76	177.	0.50	75.6	1.17
U-9	96.6	119.	0.81	131.	0.74	172.	0.56	81.2	1.19
U1	61.8	51.3	1.20	60.2	1.03	76.7	0.81	52.3	1.18
U2	60.7	60.4	1.00	63.6	0.95	83.8	0.72	64.5	0.94
U3	60.3	94.0	0.64	99.6	0.61	136.	0.44	62.7	0.96
BR-S1	130.	120.	1.08	124.	1.04	161.	0.81	141.	0.92
43	16.6	16.6	1.00	18.7	0.89	23.1	0.72	13.8	1.20
44	17.2	18.9	0.91	18.7	0.92	27.5	0.60	13.8	1.24
45	18.5	21.3	0.87	19.5	0.95	30.8	0.59	15.8	1.17
46	18.1	21.3	0.85	19.5	0.93	30.8	0.59	15.8	1.14

Table 6.7 Calculated and measured shear strengths (continued)

Specimen	V_{test}	V_{proposd}	V_{test} /	V_{ACI}	V_{test} /	V_{Priestley}	V_{test} /	V_p	V_{test} /
	kips	kips	V _{proposd}	kips	V _{ACI}	kips	V _{Priestley}	kips	V _p
62	13.0	12.6	1.03	16.9	0.77	20.6	0.63	13.1	0.99
63	15.4	16.5	0.93	17.7	0.87	22.1	0.70	15.2	1.01
64	15.4	15.4	1.00	17.7	0.87	22.1	0.70	15.2	1.01
205	16.0	14.4	1.11	14.6	1.10	21.9	0.73	15.2	1.05
207	23.8	18.7	1.26	14.6	1.63	24.9	0.95	22.7	1.05
208	30.4	21.7	1.40	17.0	1.79	23.3	1.30	21.5	1.41
214	18.6	14.4	1.29	13.3	1.40	18.5	1.00	14.4	1.29
220	17.6	14.0	1.26	15.6	1.12	19.2	0.92	14.3	1.23
231	11.4	11.4	1.00	12.1	0.94	15.9	0.72	11.6	0.98
232	13.1	11.1	1.18	11.8	1.11	15.4	0.85	11.2	1.17
233	15.5	13.1	1.19	11.9	1.30	15.6	0.99	13.7	1.13
234	15.1	11.9	1.27	11.8	1.28	15.4	0.98	13.5	1.11
372	16.7	14.6	1.15	15.6	1.07	18.5	0.90	13.4	1.25
373	19.8	16.4	1.21	15.7	1.26	22.3	0.89	17.8	1.11
452	24.8	26.3	0.94	24.5	1.01	33.7	0.74	20.3	1.22
454	24.8	26.7	0.93	24.5	1.01	34.4	0.72	24.7	1.01
40.033aE	22.3	18.3	1.17	22.0	0.98	21.4	1.01	20.6	1.04
40.033aW	22.8	19.0	1.15	21.6	1.01	22.9	0.96	20.4	1.07
40.048E	23.5	19.3	1.02	20.7	0.95	26.4	0.74	18.8	1.05
40.048W	22.1	15.4	1.18	19.0	0.96	19.9	0.91	16.1	1.13
40.033E	21.2	22.7	0.94	25.0	0.86	29.3	0.73	19.9	1.08
40.033W	23.6	20.8	0.93	22.8	0.85	30.5	0.63	15.7	1.23
mean			1.05		1.01		0.76		1.12
std. dev.			0.16		0.21		0.15		0.11

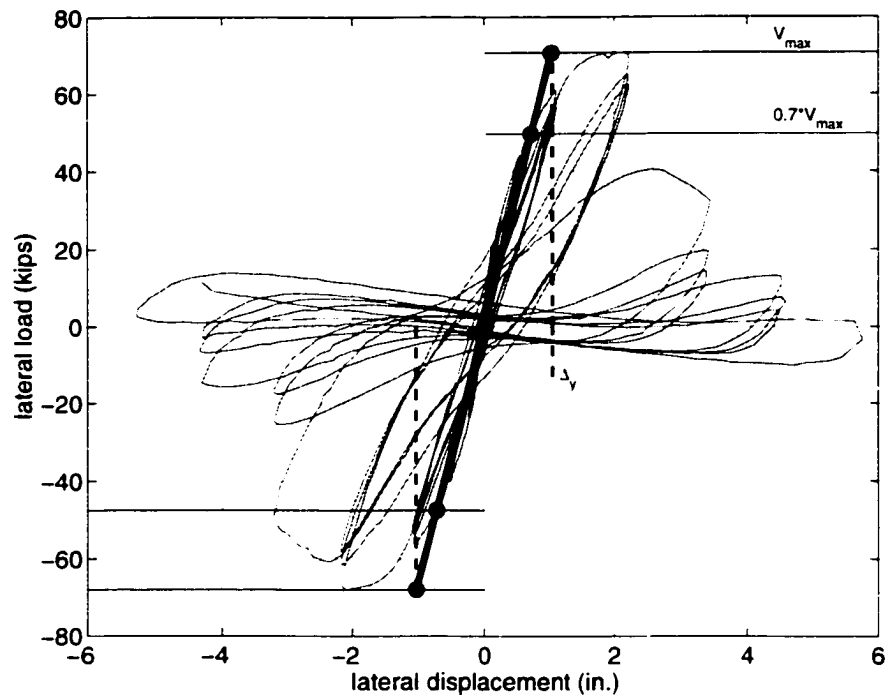


Figure 6.1 Example of yield displacement estimation (Specimen-1)

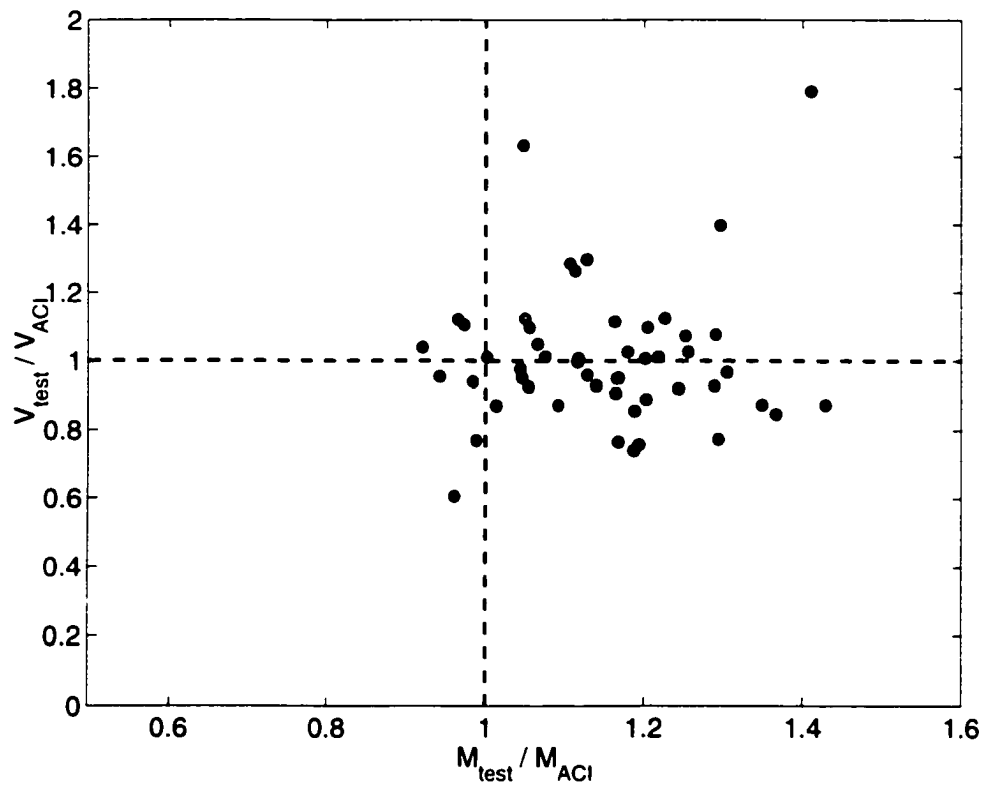


Figure 6.2 Ratio of measured to calculated shear strength versus moment

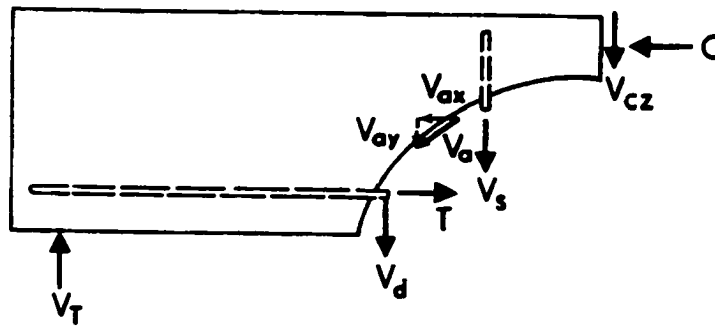


Figure 6.3 Internal forces at inclined crack in a cracked member (ACI-ASCE 426 1973)

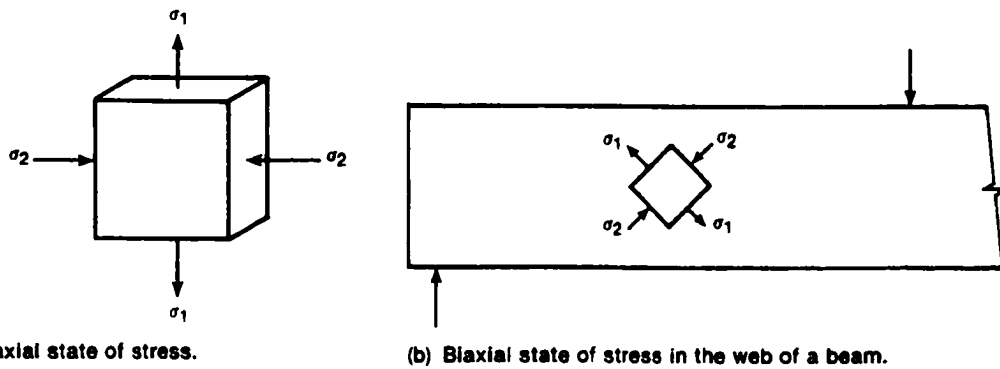


Figure 6.4 Biaxial state of stress (MacGregor 1997)

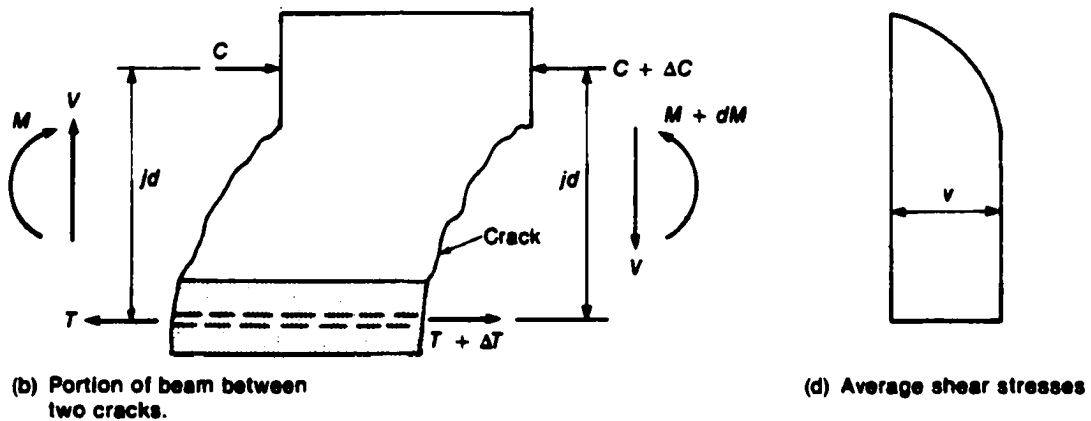


Figure 6.5 Distribution of average shear stress (MacGregor 1997)

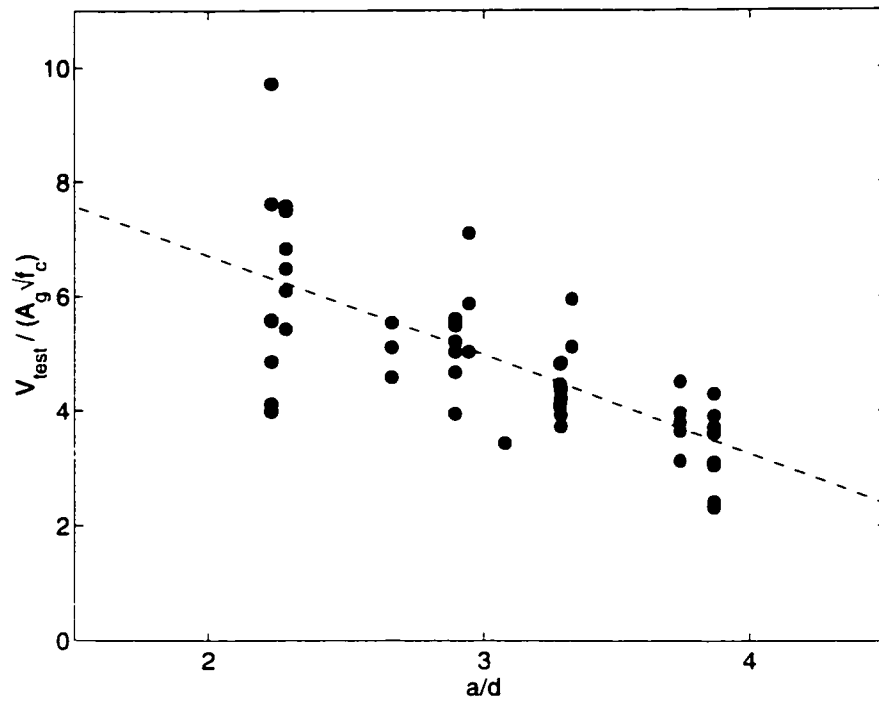


Figure 6.6 Normalized shear strength versus column aspect ratio

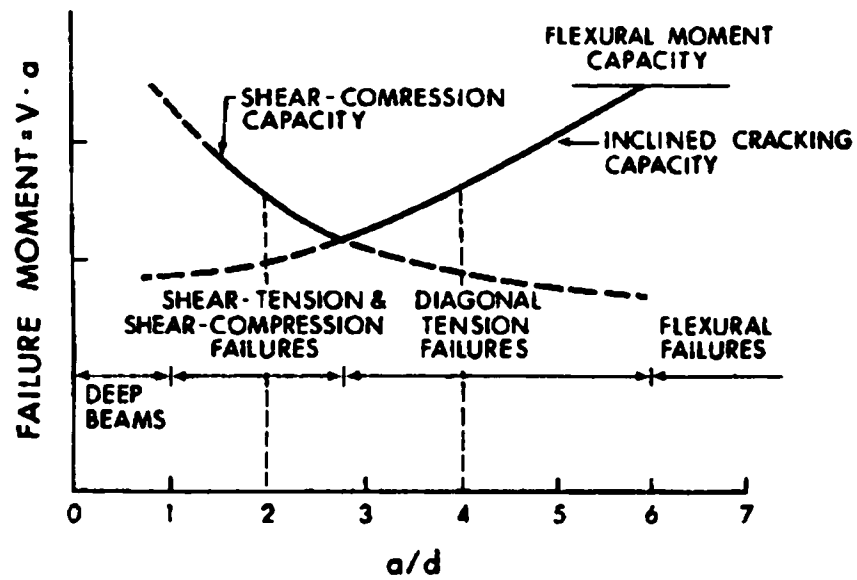


Figure 6.7 Moment-aspect ratio relationship (ASCE-ACI Committee 426 1973)

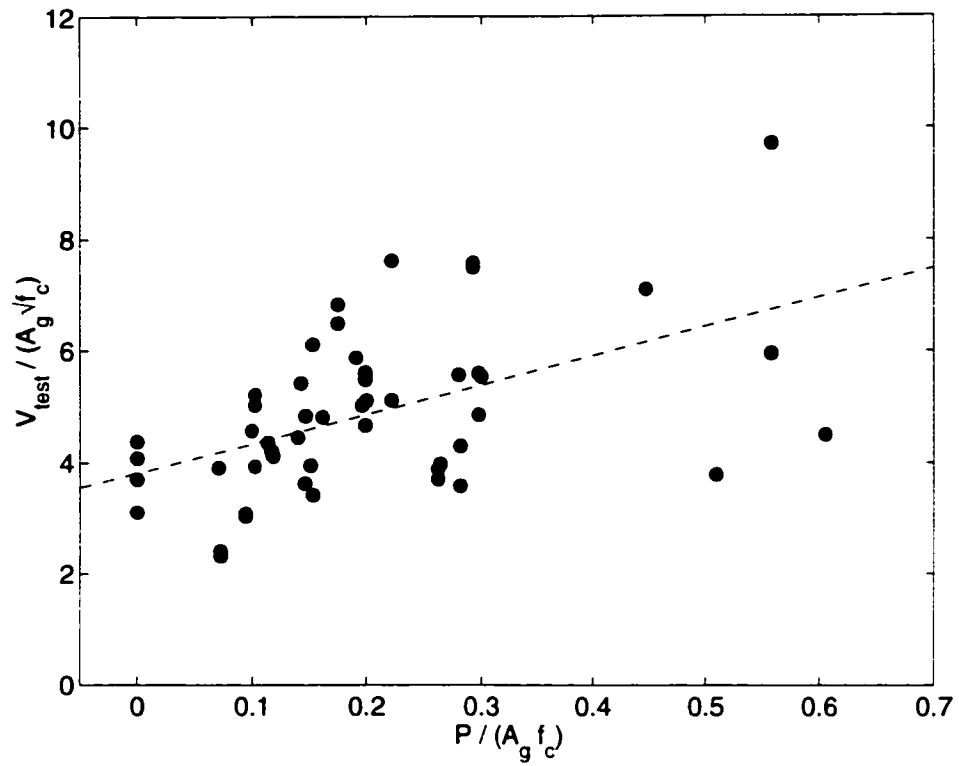


Figure 6.8 Normalized shear strength versus axial load ratio

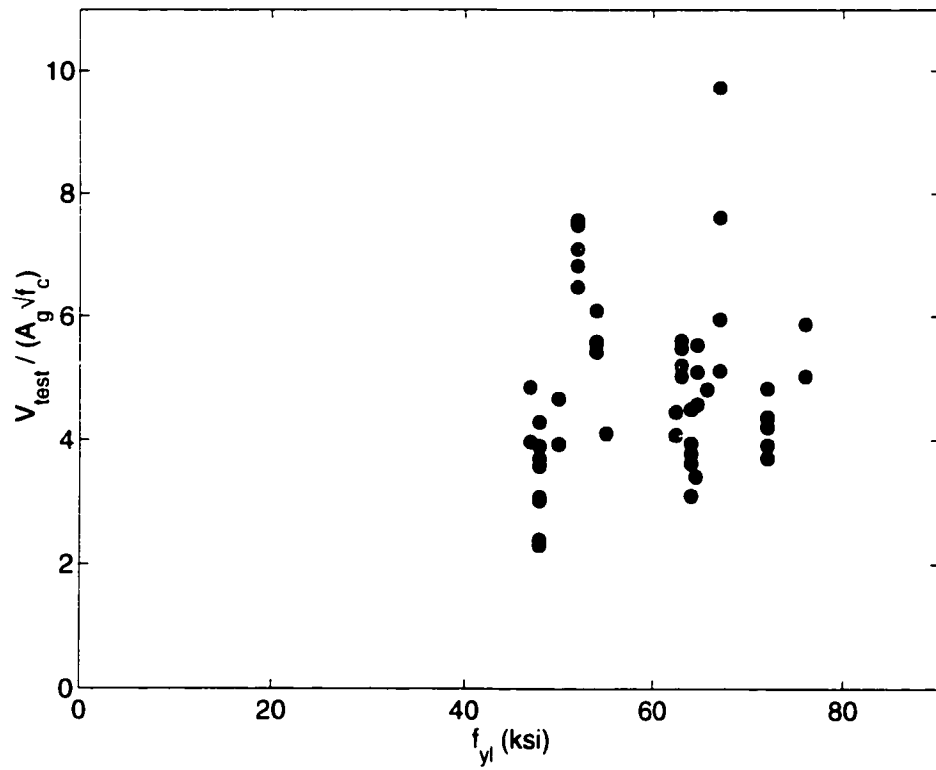


Figure 6.9 Normalized shear strength versus longitudinal reinforcement strength, f_y

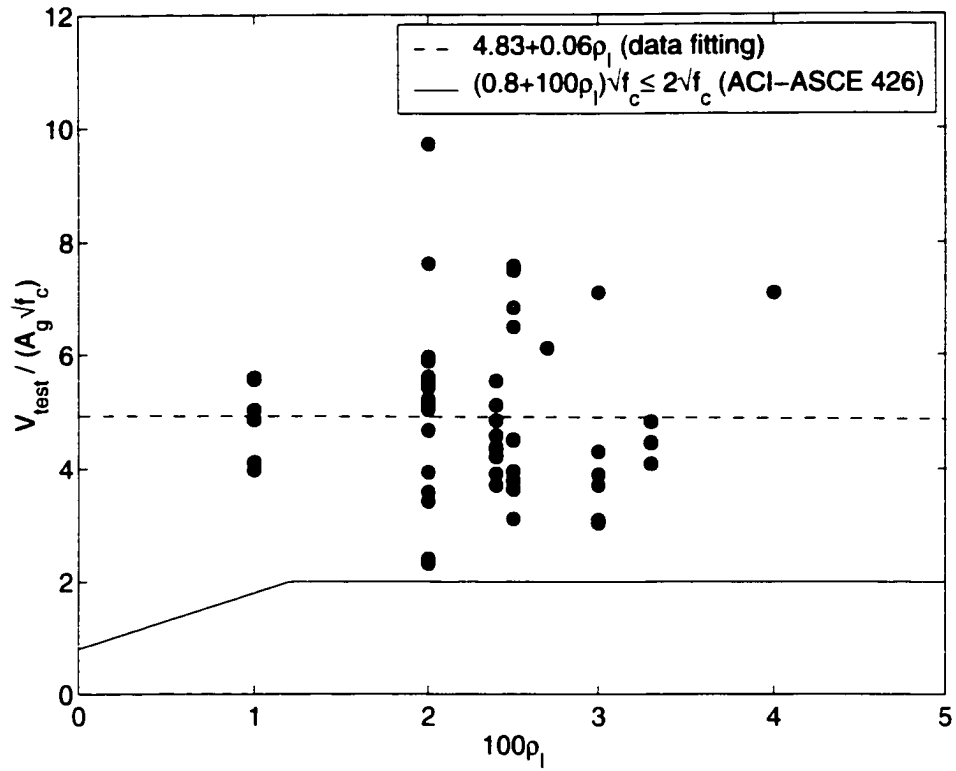


Figure 6.10 Normalized shear strength versus longitudinal reinforcement ratio

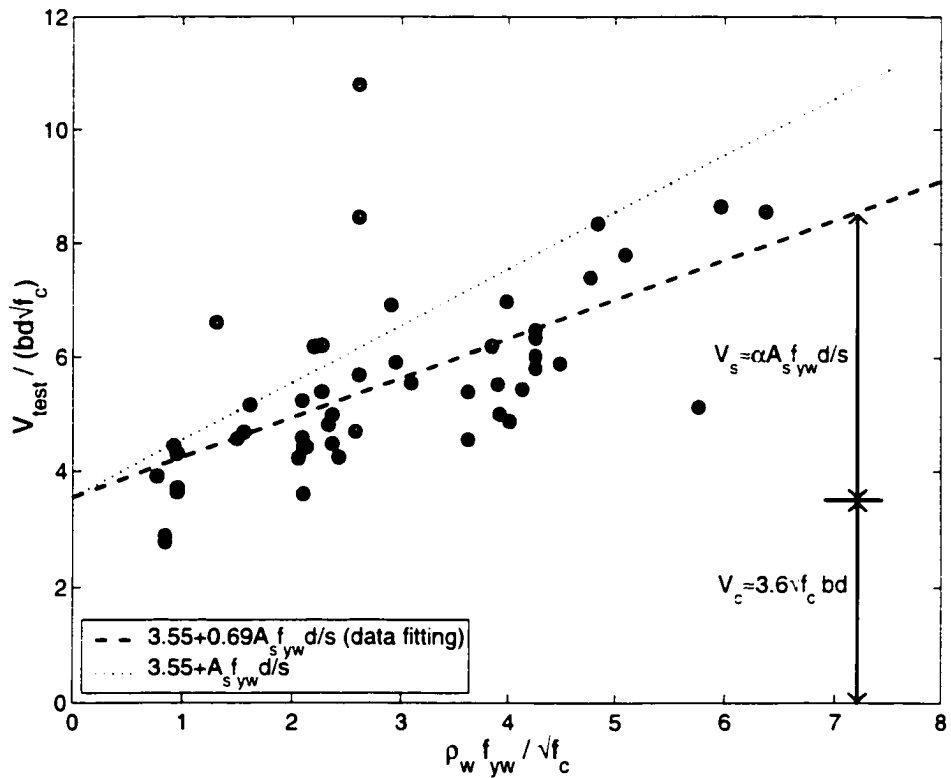


Figure 6.11 Normalized shear strength versus transverse reinforcement parameter

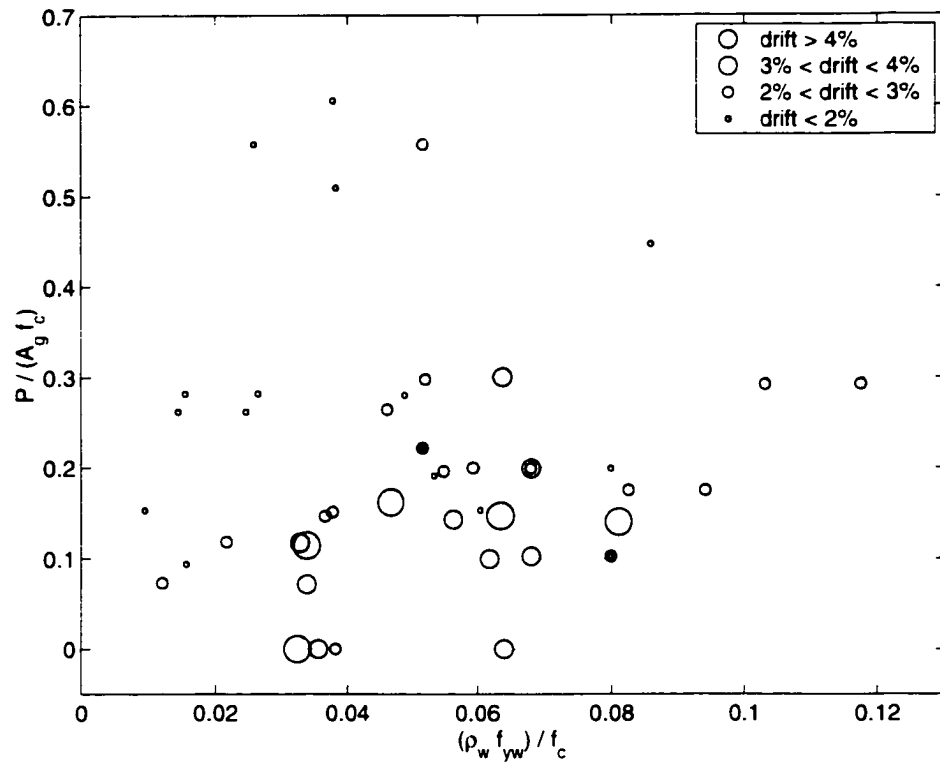


Figure 6.12 Relationship between axial load, transverse reinforcement ratio, and drift

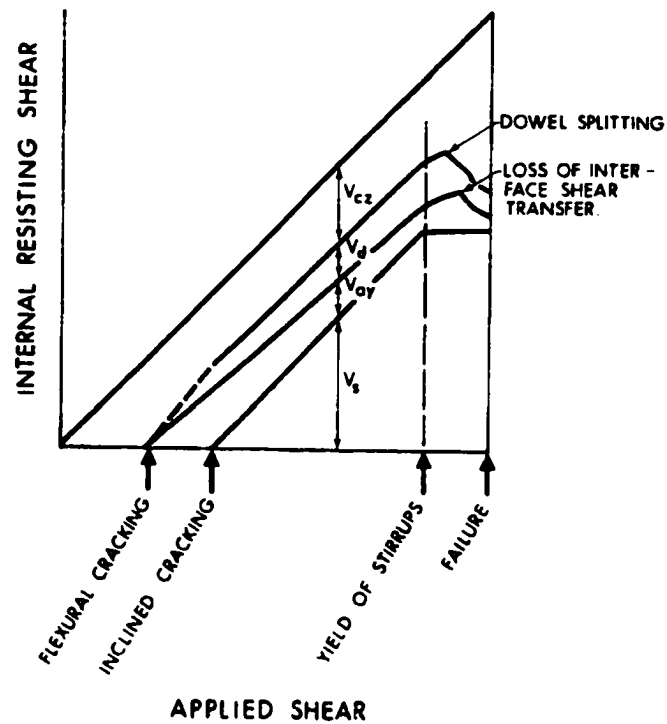


Figure 6.13 Distribution of internal shears in a beam/column (ACI-ASCE 426 1973)

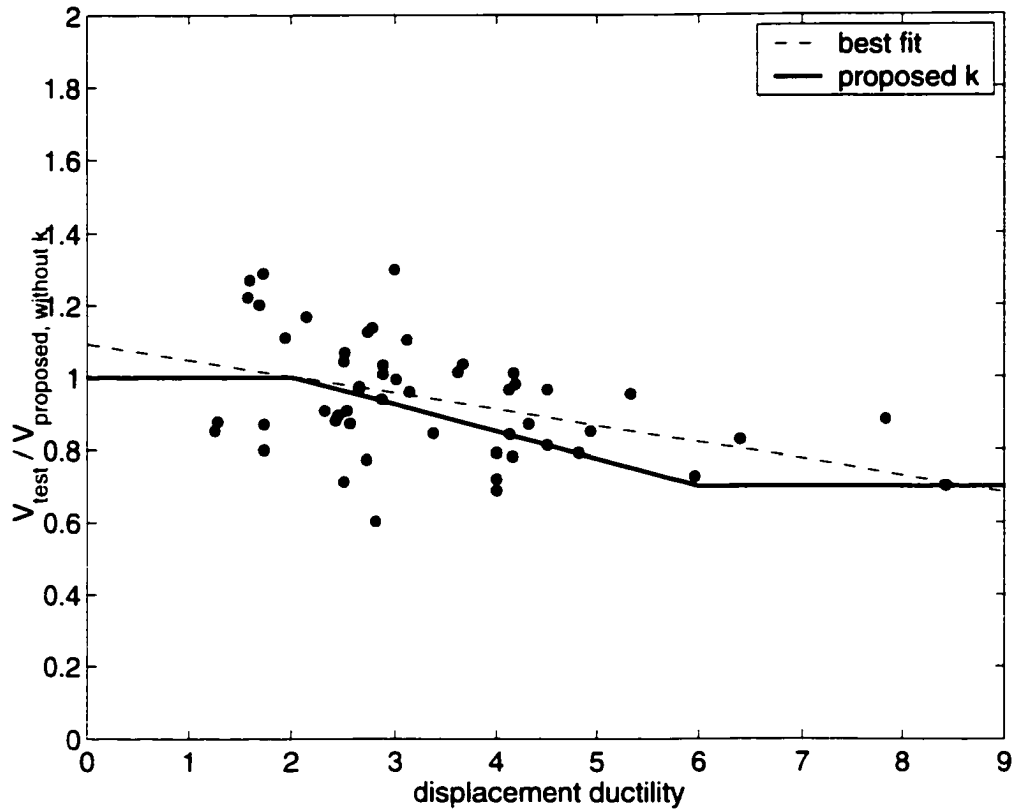


Figure 6.14 Ratio of measured to calculated shear strength versus displacement ductility

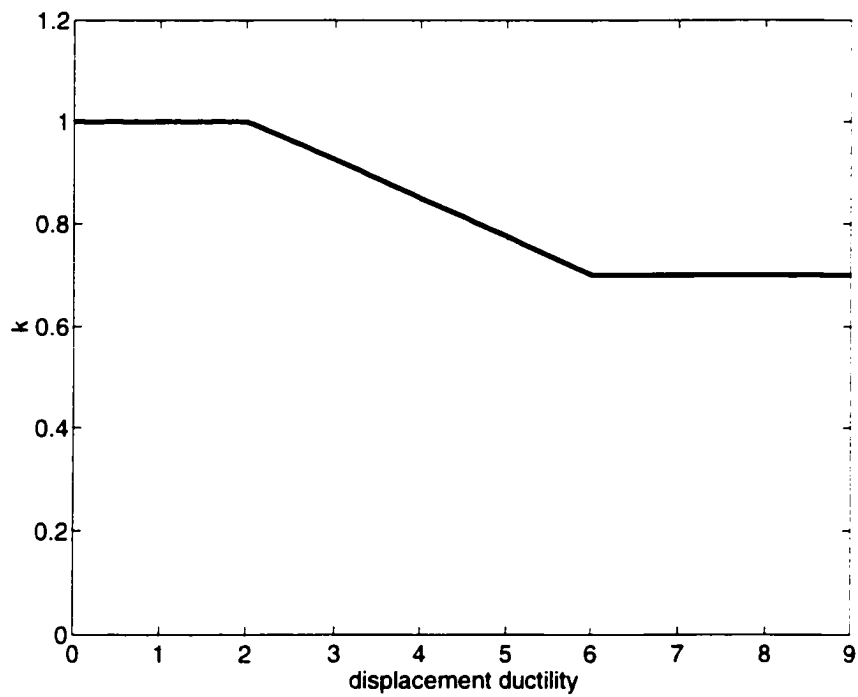


Figure 6.15 Shear strength degradation with displacement ductility

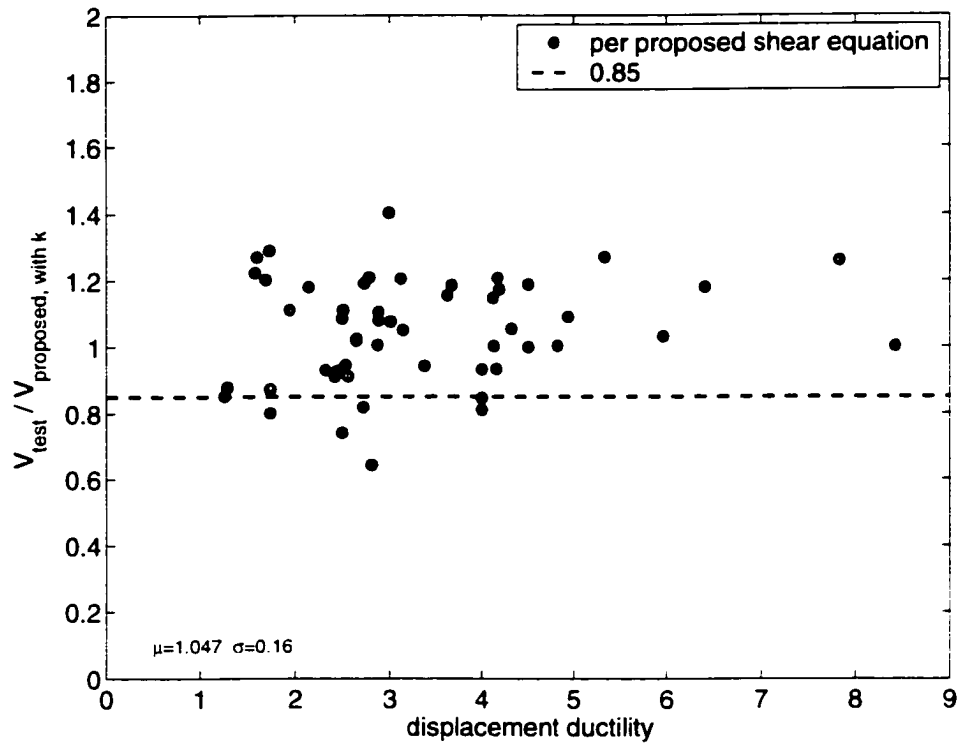


Figure 6.16 Ratio of measured to calculated shear strength including displacement ductility factor

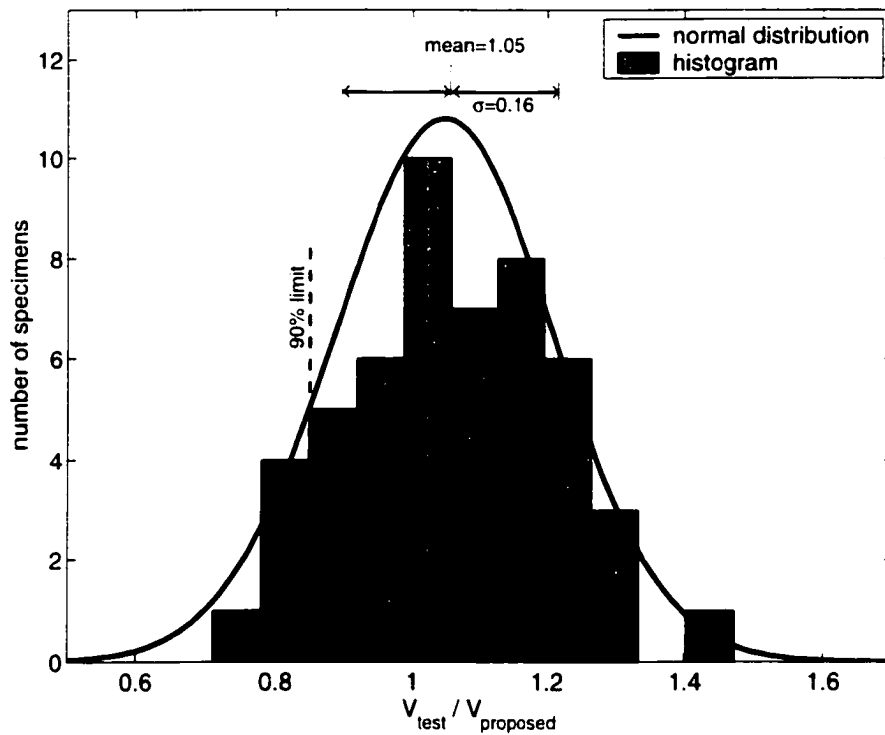


Figure 6.17 Distribution of ratio of measured to calculated shear strengths

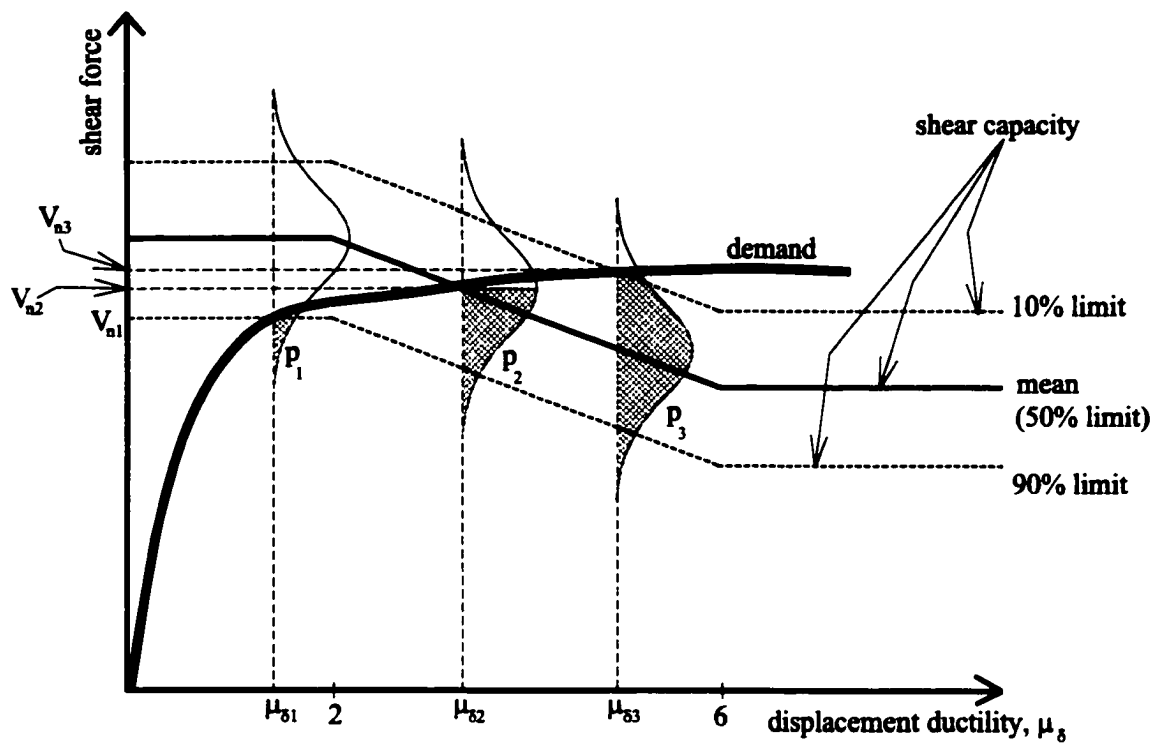


Figure 6.18 Shear strength model

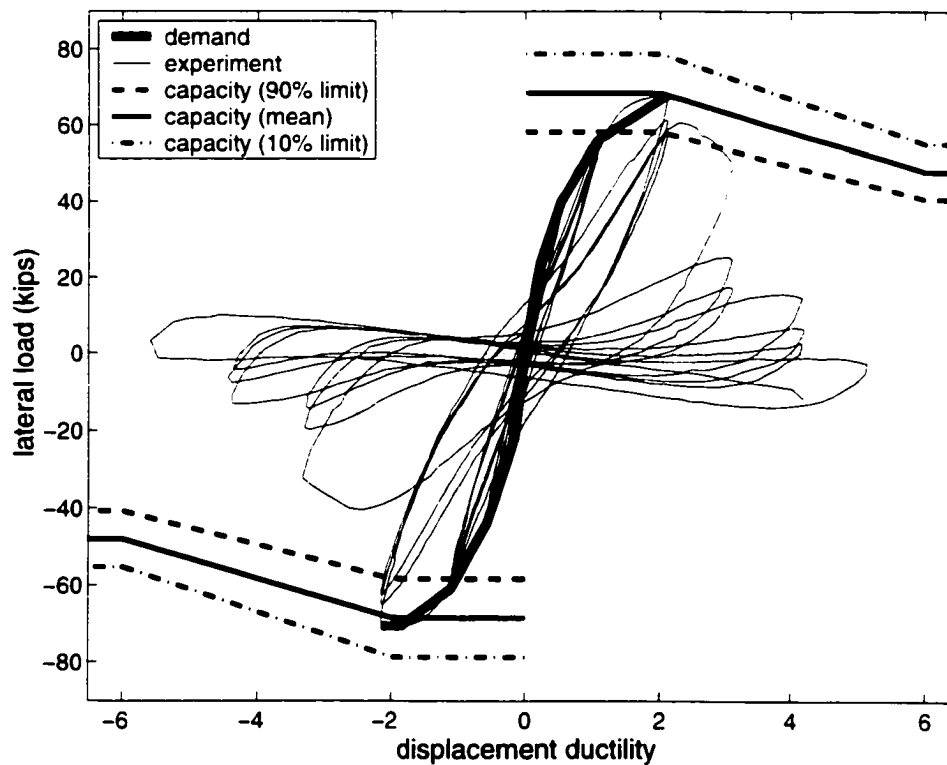


Figure 6.19 Shear strength-demand relations (Specimen-1)

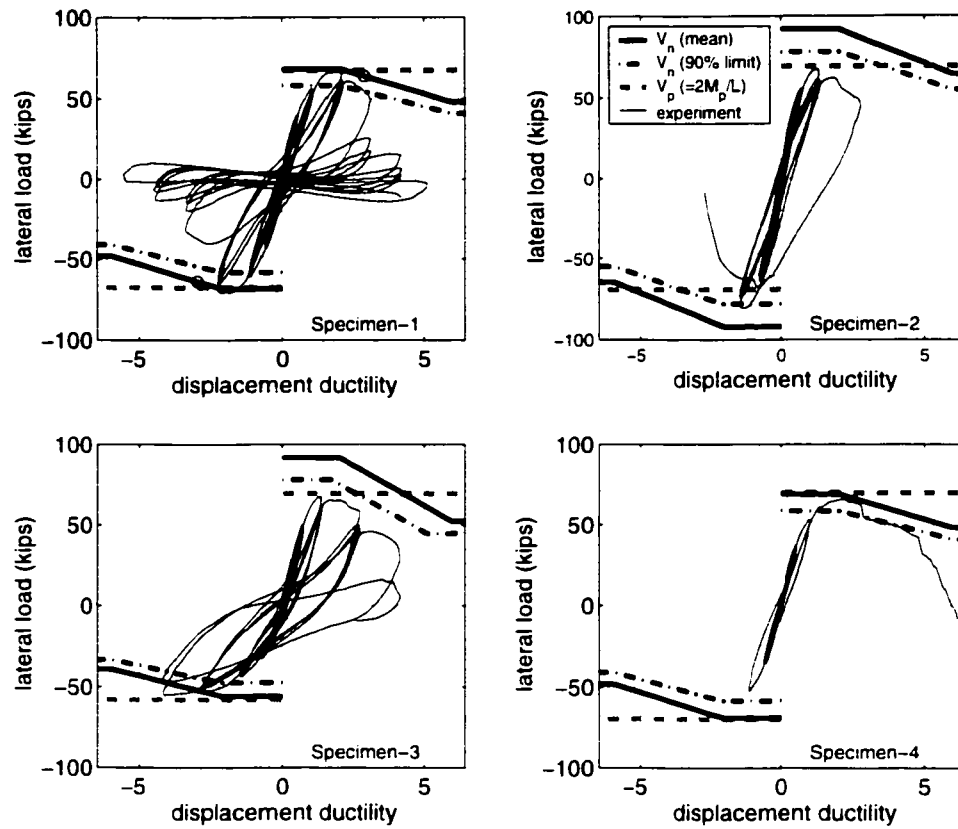


Figure 6.20 Measured lateral load-displacement ductility relations and predicted lateral flexural and shear strength

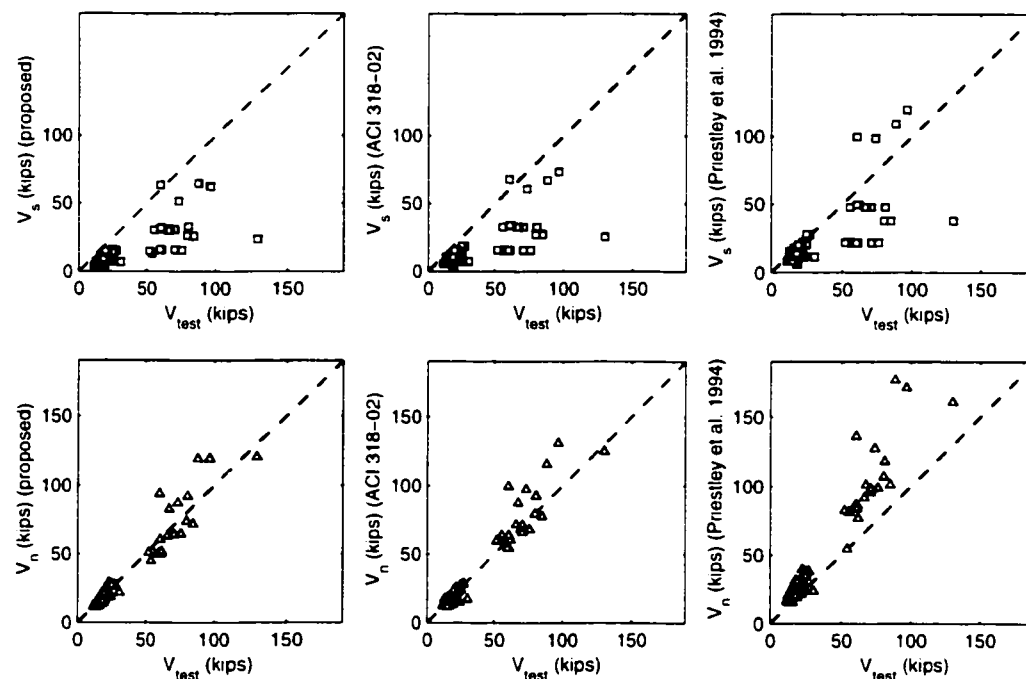


Figure 6.21 Measured shear strength, V_{test} , versus calculated V_n and V_s

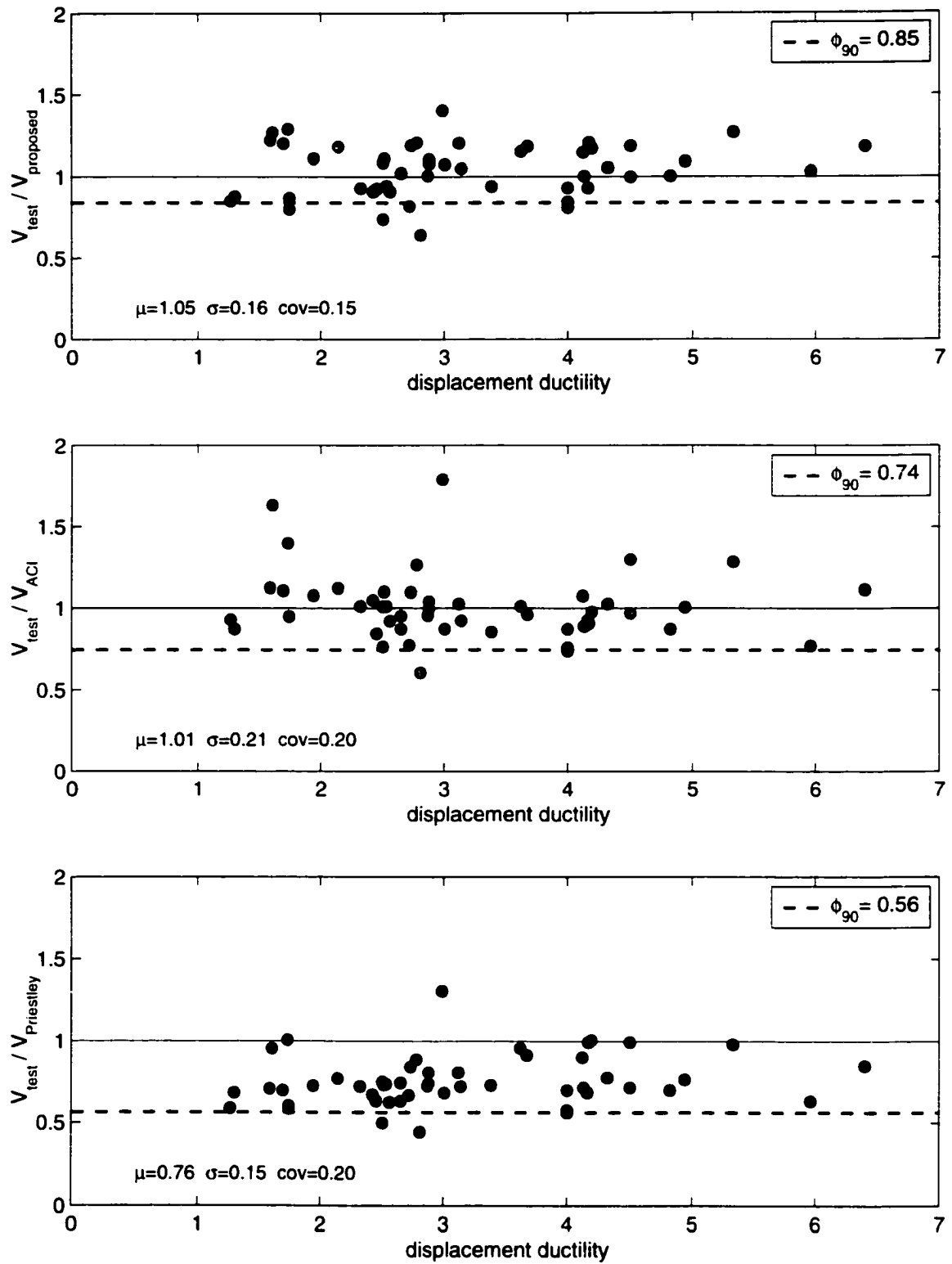


Figure 6.22 Ratio of measured to calculated shear strengths versus displacement ductility

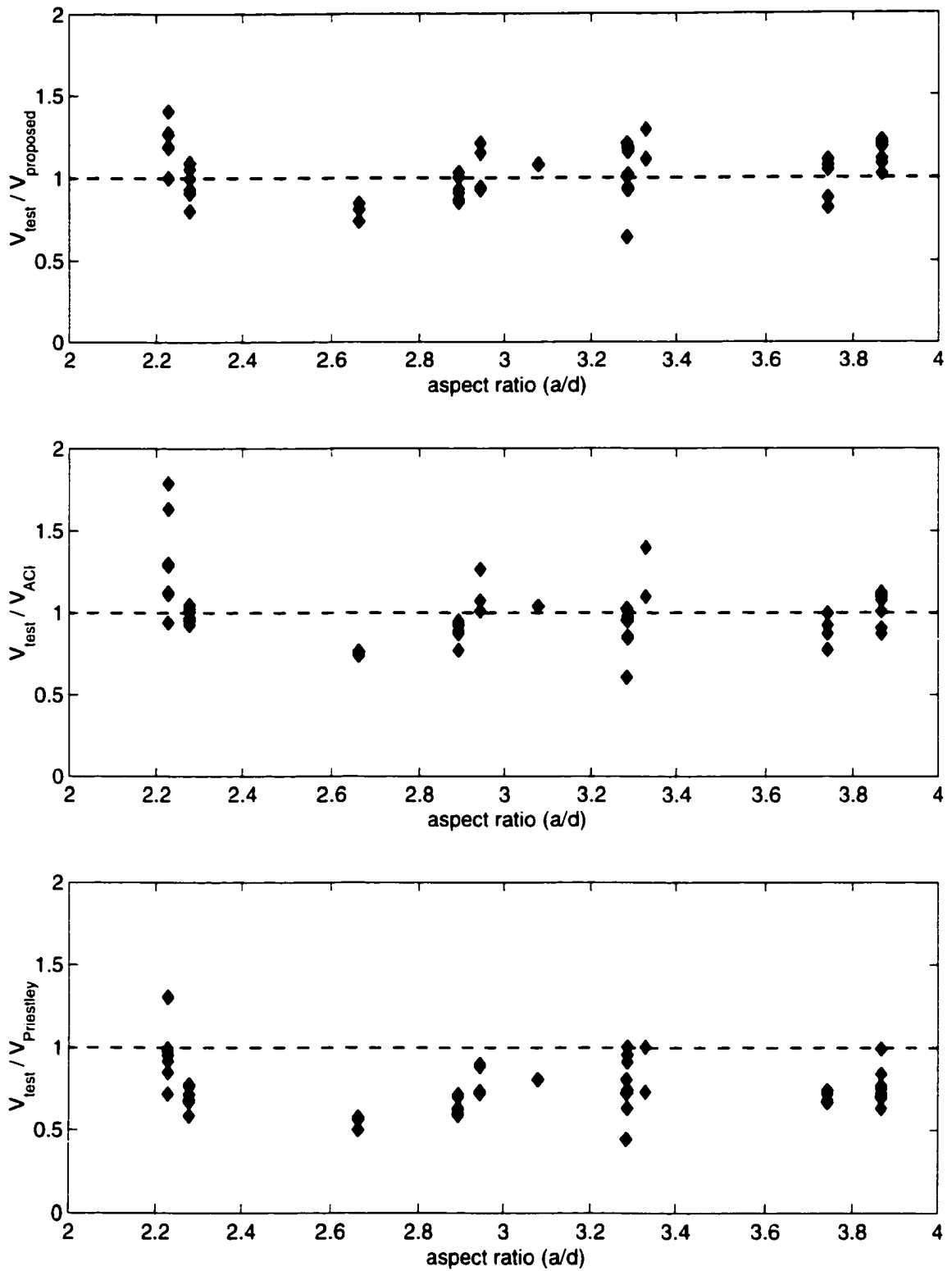


Figure 6.23 Ratio of measured to calculated shear strengths versus column aspect ratio, a/d

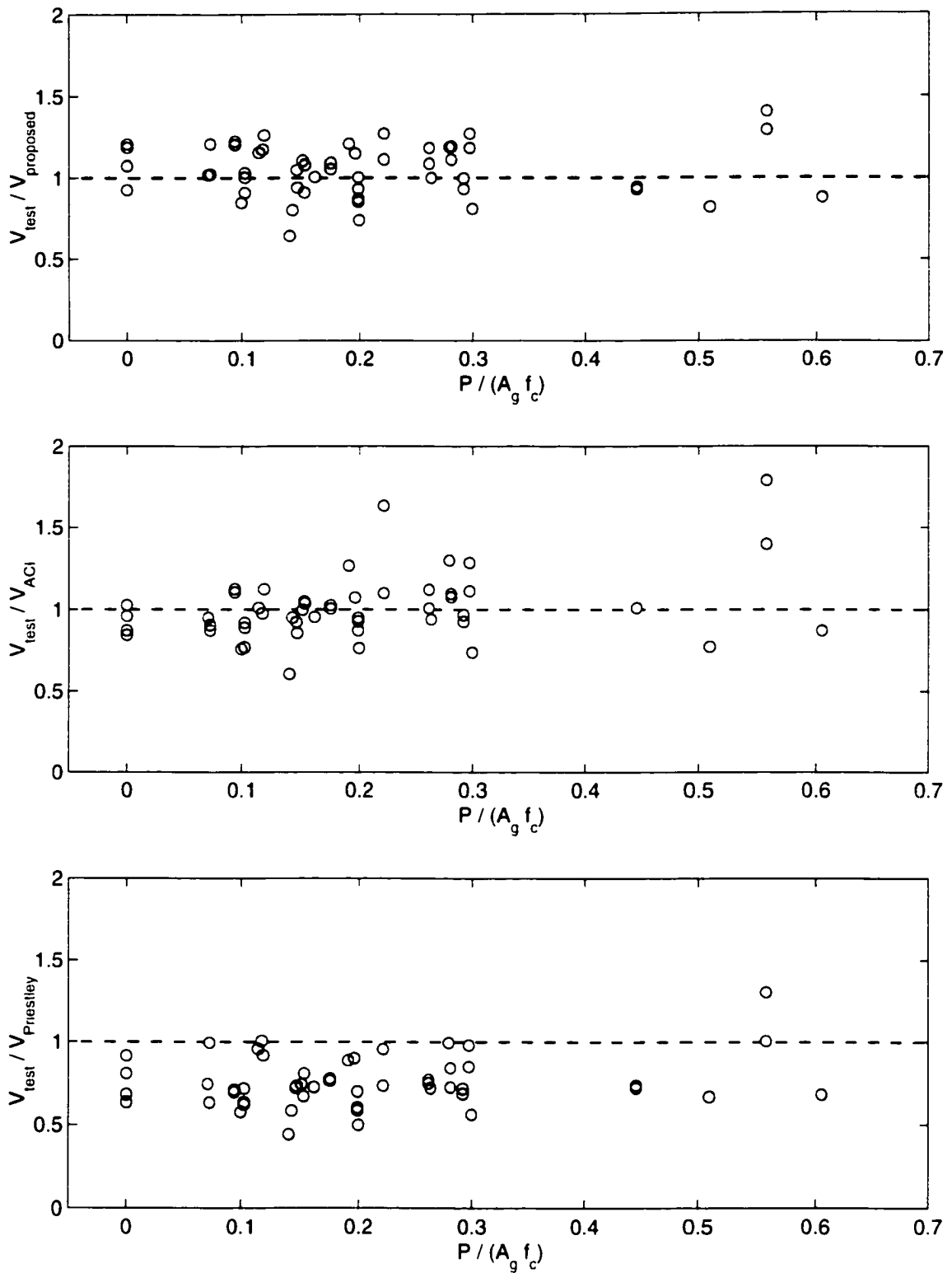


Figure 6.24 Ratio of measured to calculated shear strengths versus axial load ratio

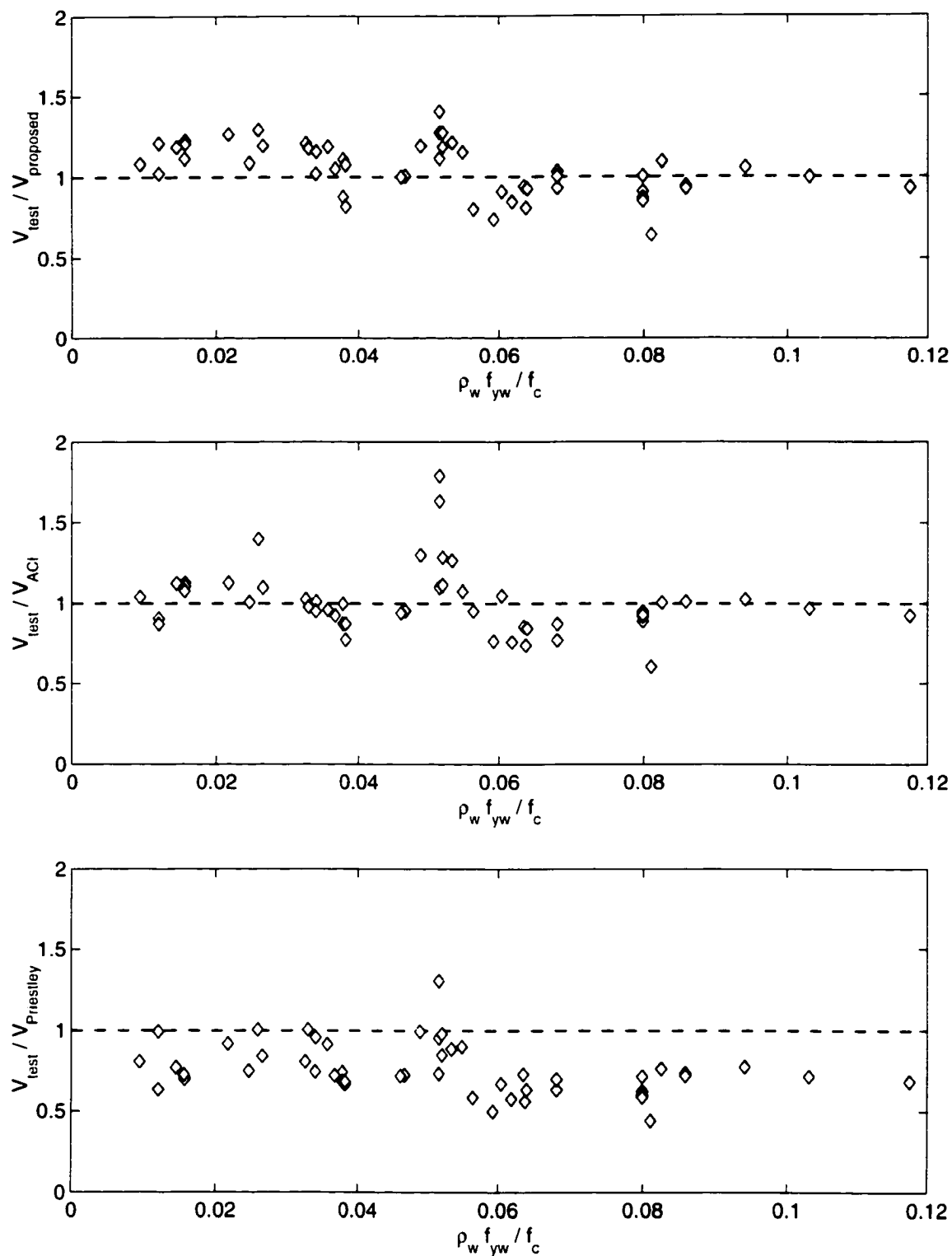


Figure 6.25 Ratio of measured to calculated shear strengths versus transverse reinforcement ratio

7 ASPECTS OF LOAD- DEFORMATION MODELING AND DEFORMATION COMPONENTS

7.1 INTRODUCTION

This chapter provides an overview of contribution of shear deformations to total column deformations. An analytical model is developed to model the column shear behavior. The piecewise linear monotonic model included four shear displacement and force pairs at 1) flexural cracking, 2) first yielding in the longitudinal reinforcement, 3) peak lateral

strength, and 4) loss of axial capacity. An equation is proposed to calculate shear displacement at first yielding in the longitudinal reinforcement. The analytical lateral load-shear displacement relations are compared with the measured shear response of test columns. Each deformation model representing flexure, bar slip, and shear behavior is idealized by a simple spring for computer modeling. A spring model with three springs in series is proposed to combine these representative deformation components. The calculated monotonic lateral load-total displacement relations are compared with the total measured response. The contribution of each deformation component to the total lateral response is also investigated.

7.2 MEASURED DEFORMATION COMPONENTS

As described in Appendix F, total displacements and shear displacements can be calculated along the height of the column specimens by using the principle of virtual work. Total member displacements measured by the DCDTs and wire potentiometers at the midheight and top of each specimen (Figure 3.27) and the displacements calculated using the principle of virtual work are compared in Figure 7.1. The comparison indicates a reasonably good agreement between the calculated and measured displacements, which may include some data errors especially at larger displacement amplitudes.

Using the calculated displacements over the height of a column, the total and shear drift ratio distribution over the height can also be calculated. Drift ratio of a segment of the column is defined as the lateral displacement difference between the top and bottom of the segment (drift) divided by the segment height (see Figure F.5 in Appendix F). Figure

7.2 shows the total drift ratio distribution over the height of four specimens tested in this research. The drift ratio profiles are calculated at the first positive and negative peaks of the displacement cycles to $\Delta_y/2$, $1\Delta_y$, $2\Delta_y$, and $3\Delta_y$, where Δ_y is the calculated lateral yield displacement (= 1.10 inches). Similarly, using the calculated shear displacements over the column height, the drift distribution due to shear is obtained and shown for each specimen in Figure 7.3. The flexural displacements over the column height and the corresponding drift ratios are calculated by integrating the measured flexural curvatures along the height (Equation 5.4). Figure 7.4 shows the flexure drift distribution over the height of each test specimen.

Total drift distributions shown in Figure 7.2 indicate that, especially during the low-amplitude displacement cycles, column deformations tend to be larger around the midheight of the columns. Similarly, flexure drift distributions in Figure 7.4 show a trend of larger drifts around the midheight of the columns consistently at all displacement levels. These trends are consistent with behavior expected on the basis of classical linear-elastic mechanics (see Figure F.5). In general, larger shear drifts are observed within the lower and upper one third of the columns (Figure 7.3). Especially after the development of initial cracks in member end regions, the larger shear drifts measured in these regions under larger force/moment demand can be attributed to increasing shear deformations resulting from opening and closing of cracks. Note that in most cases, following the initial flexural cracking, inclined shear cracks developed during or after the yield displacement cycles.

The reported shear drift values at the top and bottom of the columns (Figure 7.3) tend to be much larger as compared to rest of the column. This might be due to large longitudinal bar slip displacements at the column ends, and the fact that shear and slip displacements in the end of the column cannot be mathematically separated using the available instrumentation. This can be explained mathematically using Equation F.7 by assuming a very small or zero displacement in the right, top, and bottom chords of a typical instrumentation segment near the column ends. Because of assumed relatively large displacements in the left and diagonal chords due to slip in the tension reinforcement, the resulting calculated shear displacement in the segment would be considerably large.

7.3 SHEAR DEFORMATIONS AND MODELING

The shear displacement histories over the height of each specimen can be calculated by using the principle of virtual work (PVW) in conjunction with the local relative displacements measured by the displacement potentiometers installed on the test specimens (Appendix E and Appendix F). As a typical example, the calculated total and shear displacement histories over the height of Specimen-1 are shown in Figure 7.5. The displacements measured by the DCDTs at the midheight and top of the column are also shown in the same figure. The plots indicate that, in Specimen-1, the contribution of shear displacements to total displacements was relatively small for displacements smaller than the yield displacement. The sudden increase in shear displacements in the first and second cycles of 34 displacement level is consistent with the damage (i.e.,

wide inclined cracks) observed in this test column at this time. The shear failure in Specimen-1 was observed in those cycles (Figure 4.2 and Figure 4.3).

7.3.1 Shear Models

Models for response of columns with details satisfying current code requirements may reasonably ignore shear deformations, or model them using simplified procedures, because shear deformations are relatively small. The amount of shear deformations in older (shear-critical) columns, especially after the development of shear cracks, can be significant and needs to be modeled in order to represent total deformations in these members. Several different models have been developed to represent the shear displacement at the first longitudinal bar yielding using basic principles of mechanics (Park and Paulay 1975, Konwinski et al. 1996, and Priestley et al. 1996). However, very few researchers and design standards (e.g., Park and Paulay 1975, and CEB 1985) have attempted to calculate the shear displacements in a damaged column after the development of flexural and shear cracks. Park and Paulay (1975) and CEB-1985 provided equations to predict the shear displacement of cracked concrete members.

Based on the theory of linear elastic mechanics with the assumption of uniform shear strain along the member length, the shear displacement of a reinforced concrete column with an uncracked rectangular cross section can be calculated from

$$\delta_{shear} = \frac{6}{5} \frac{V L}{G A_g} = 3 \frac{V L}{E_t A_g} \quad (7.1)$$

where the shear modulus, G , is approximated from Equation 7.2. The factor (6/5) accounts for the nonuniform shear stress distribution for rectangular sections.

$$G = \frac{E_c}{2(1+\nu)} \quad (7.2)$$

Poisson's ratio, ν , for normal weight concrete is approximated as 0.25, and E_c = modulus of elasticity of concrete.

The measured shear displacement histories from the four specimens tested in this research indicate that shear displacements are related to the magnitude of the axial load. The measured axial load-shear displacement relations for Specimen-3, which was subjected to varying axial load, show an increase in shear displacement with decreasing axial load (see Figure 7.6, where axial load in compression is shown as positive. In addition, relations between the axial load and measured shear displacements (Figure 7.7) indicate a similar trend at first yielding in the longitudinal reinforcing bars. Based on the measured shear displacements of the four columns tested in this research, the following equation is proposed to calculate shear displacement at yield

$$\delta_{v, shear} = \left(\frac{3}{0.2 + 0.4P_r} \right) \frac{V_y L}{E_c A_v} \quad (7.3)$$

where $V_y = 2M_y/L$ for double-curvature specimens (M_y = moment capacity at yield), and P_r = axial load ratio, which is the ratio of the applied axial load, P , to the axial load capacity of the column, P_n .

$$P_o = 0.85 f'_c A_g (1 - \rho_l) + f_y A_{sl} \quad (7.4)$$

where ρ_l = longitudinal reinforcement ratio ($= A_{sl}/A_g$), and A_{sl} = total area of longitudinal reinforcement.

Currently, there is no consensus among researchers on how to model the shear behavior of cracked concrete. While some models are based on relatively simple rules (e.g., Park and Paulay 1975), others consider effects of many parameters including the aggregate size, stress in the steel bars crossing the crack, crack width, and crack spacing (Zhu et al. 2001).

Park and Paulay (1975) used well-known truss-analogy and developed the following equation for the prediction of shear deformations in fully cracked beam-columns.

$$\delta_{shear} = \frac{V_s L}{d b} \left[\frac{1}{\rho_w E_s} + \frac{4}{E_c} \right] \quad (7.5)$$

where ρ_w = transverse reinforcement ratio, and V_s = shear force carried by the transverse reinforcement (Equation 6.9). Based on the truss analogy, the CEB (1985) recommends an equation very similar to Equation 7.5 except that the CEB model uses total member shear force, V , instead of transverse steel force, V_s .

7.3.2 Proposed Model

A simple analytical method is proposed to model the shear behavior of lightly reinforced concrete columns under monotonic lateral loading (Figure 7.8). The piecewise linear

model includes stiffness changes at cracking, yielding, and at peak lateral strength before shear failure.

In this research, it was not easy to identify a single force-displacement pair that represented the change in the lateral stiffness and the increase in shear displacement due to development of first flexural cracks. However, after the development of flexural cracks, there was a gradual decrease in the stiffness and an increase in shear deformations, especially within the upper and lower one third of the column as indicated by shear drift distributions in Figure 7.3. Most likely, this increase in shear deformations was resulted from opening and closing of cracks. Therefore, a limit point corresponding to the first flexural cracking is defined in the proposed model. The shear displacement at the cracking limit, δ_{cr} , is calculated from Equation 7.1, and the corresponding lateral load, V_{cr} , for a double-curvature column is calculated from the following expression

$$V_{cr} = \frac{2 M_{cr}}{L} \quad (7.6)$$

where $M_{cr} = (7.5 \sqrt{f'_c} I) / c$, I = uncracked cross-sectional moment of inertia and c = neutral axis depth.

The shear displacement at first yielding in the longitudinal reinforcement, δ_y , is obtained from Equation 7.3 for cracked concrete. The lateral load at yield, V_y , for a double-curvature column is calculated by dividing the yield moment (from section moment-curvature analysis) by half the total column length. Assuming that large inclined cracks develop when the peak lateral strength is reached, the shear displacement, δ_n ,

corresponding to the peak strength is calculated from the relationship developed by Park and Paulay (1975) (Equation 7.5). The peak lateral strength, V_n , and the shear force carried by transverse reinforcement, V_s , are calculated from Equations 6.15 and 6.10, respectively. Equation 6.10 assumes that the transverse reinforcement yields and reaches its yield strength, f_{yw} . Experimental data from the four columns tested in this research indicated that the transverse reinforcement yielded at peak lateral strength prior to shear failure (Figure 4.40). In the proposed model (Figure 7.8), the peak lateral strength, V_n , must be smaller than the lateral load, V_p , required to reach the maximum flexural capacity, M_p , at column ends ($V_p = 2M_p/L$ for double-curvature specimens).

The shear displacement at the end of monotonic loading, δ_{end} , is calculated from

$$\delta_{end} = \delta_{total,end} - \delta_{flex,n} - \delta_{slip,n} \geq \delta_n \quad (7.7)$$

where, $\delta_{total,end}$ = total displacement capacity of column at loss of axial capacity. If the final failure is dominated by shear, then the column does not develop its maximum flexural and slip deformation potential. Therefore, flexure and bar slip displacements at axial failure are the same as flexure and bar slip displacements corresponding to peak lateral strength, $\delta_{flex,n}$ and $\delta_{slip,n}$, respectively (Chapter 5). δ_n is calculated from Equation 7.5.

Total displacement capacity at axial load failure ($\delta_{total,end}$) is calculated from the relation shown in Figure 7.9. The relation among column axial load, transverse reinforcement, and drift ratio was derived by Moehle et al. (2000) using a shear friction model (Elwood 2002) and column test data from Lynn (2001) and this study. For a given axial load ratio,

P/P_o (P_o calculated from Equation 7.4) and transverse reinforcement parameter, $A_{sv}f_y h/(sP_o)$, the drift ratio at axial load failure could be obtained from Figure 7.9 ($\delta_{total,end} = \text{drift ratio} * L$).

Figure 7.10 and Figure 7.11 compare the measured response with the calculated secant stiffnesses using the shear displacement and shear force pairs based on uncracked cross-section (Equations 7.1 and 7.6), first yielding in the longitudinal reinforcement (Equation 7.3 and $V_y = 2M_y/L$), and peak lateral strength (Equations 7.5 and 6.15). Overall, the calculated stiffness compares well with the average experimental stiffness. The comparison for cracked stiffness for Specimen-2 is somewhat poor probably because it sustains compressive failure. Consistent with the experimental stiffness, the shear stiffness of Specimen-3 from the proposed equation is different under high and low axial loads as the calculated shear displacement at yield is a function of axial load (Equation 7.3).

The measured lateral load-shear displacement relations and the calculated monotonic response envelopes for each test specimen are shown in Figure 7.12 through Figure 7.15. In developing the rules for the construction of monotonic response envelope, the main concern was to be able to justify, experimentally and theoretically, each shear displacement and shear force pair used in the proposed multi-linear model. Consequently, the success of the model depends on how accurate the shear displacement and force estimates are. Even though the available relations were used for the shear displacement and strength predictions, some predictions are not as good as other points. Nevertheless, overall, the predicted monotonic envelopes compare reasonably well with

the measured response. Note that the predicted shear response envelopes shown in Figure 7.12 through Figure 7.15 will probably be modified when the interaction between flexure, bar slip and shear response components are considered to obtain the lateral load-total lateral displacement relations as described below.

7.4 LOAD-DEFORMATION MODELS AND COMBINED RESPONSE

The flexure, bar slip, and shear deformation models presented in Chapter 5 and Section 7.3 represent the monotonic response reasonably. While these component responses can be modeled separately, very few researchers (e.g., Kim and Mander 1999, and Petrangeli 1999) attempted to investigate the interaction between flexure and shear components and the contribution of each response component to the total member response. Kim and Mander (1999) and Petrangeli (1999) implemented computer models to combine the cyclic lateral load-deformation relationships for flexure and shear components using a truss model and a fiber element model, respectively. Lee and Elnashai (2001) implemented hysteretic flexure and shear models into a computer analysis program considering the interaction between these two components using a simple spring model. In the model by Lee and Elnashai (2001), the monotonic response envelopes for both components were based on a piecewise linear model obtained from the Modified Compression Field Theory (Vecchio and Collins, 1986). Researchers who investigated the flexure-shear component interaction used springs lumped at member ends as a typical case, and usually did not consider the effect of bar slip deformations.

Using the individual component models developed in the previous section and Chapter 5 to characterize the flexure, bar slip, and shear behavior, an analytical model with three springs in series is proposed in this study. The combined spring model gives the envelope of the lateral load-displacement relation of a lightly reinforced concrete member.

As illustrated in Figure 7.16, total flexural deformations are calculated from integration of moment-curvature relations. The moment-curvature relations themselves are obtained from the fiber section analysis (Section 5.4). In the flexural displacement calculations, consistent with the experimental local displacement measurement locations (Figure 3.26), eight sections were used along the height of the test column (Equation 5.4). The behavior of a single lumped flexural spring is assumed to be represented by the calculated lateral load-flexural displacement relations shown in Section 5.4 for each specimen.

The rotation due to bar slip is assumed to be concentrated at column ends in the form of rigid body rotation (Figure 7.16). Consequently, the displacement due to bar slip can be calculated from Equation 5.23. The calculated lateral load-slip displacement relations for each bar slip spring at specimen ends were shown in Section 5.6.

Figure 7.16 also illustrates the idealized single-spring model for the monotonic shear behavior. Assuming a uniform shear strain, γ , distribution along the column height, the total shear displacement can be obtained as the product of the uniform shear strain and

column height. The measured and calculated lateral load-shear displacement relations for each test specimen were compared in Figure 7.12 through Figure 7.15.

The lateral load-displacement response due to flexure, bar slip, and shear can be modeled by three springs connected in series resulting in the same force in each spring. The summation of individual spring displacements corresponding to the same lateral load in each spring gives the total member displacement for that lateral load.

In the combined spring model, the maximum column capacity is presumed to be the smaller of the calculated lateral load, V_p , required for the formation of plastic hinges at column ends ($V_p = 2M_p/L$) and the shear strength, V_n , calculated from Equation 6.12.

The post-peak response and final failure could be dominated by either shear or flexure. If the post-peak response is controlled by shear, then as load decreases due to shear failure, the deformation due to flexure and slip are assumed to remain constant at values equal to the values at maximum load. In other words, the flexure and bar slip springs are locked at peak value with resulting constant post-peak displacements corresponding to smaller of V_p and V_n . It is assumed that the post-peak response will be controlled by shear if V_p is larger than the shear strength reduced by the strength reduction factor, ϕ_{90} , defined in Section 6.5 (i.e., shear dominated post-peak behavior if $V_p \geq \phi_{90}V_n$). Based on the test data and statistical evaluation of shear strength equation in Chapter 6 (Equation 6.12), the strength reduction factor, ϕ_{90} , was calculated as 0.85 for the proposed shear strength model.

If V_p is smaller than ϕ_{90} times V_n , the post-peak behavior is controlled by flexure. In such a case, the shear spring is locked at V_p , and the displacement in the shear spring stays constant in the post-peak range. Then, the total post-peak member displacement is the summation of constant shear spring displacement and flexural and bar slip spring displacements.

As indicated in the lateral load-shear displacement plot in Figure 7.17 (bottom left plot), the calculated shear strength, V_n , for Specimen-1 is slightly larger than V_p . The plot suggests that, theoretically, the column will reach its maximum moment capacity before it reaches its shear strength, V_n . In fact, this column specimen was designed to have nearly equal V_n and V_p . As a result, the consequent failure of Specimen-1 can be described as combined flexure-shear failure. As discussed in Chapter 4, in this column widespread flexural cracks and yielding and some subsequent strain hardening in the longitudinal reinforcement as well as extensive shear (inclined) cracks were observed before failure. Considering the significant contribution of flexure and shear components to the final failure, it is not easy to conclude whether the flexural or shear capacity of this relatively slender lightly reinforced column was reached first.

According to combined spring model described above, the maximum lateral load that can be applied on Specimen-1 is equal to V_p ($V_p < V_n$). As indicated in the bottom left plot in Figure 7.17, V_p is larger than $\phi_{90}V_n (= 0.85V_n)$ for this specimen. Thus, post-peak behavior prior to loss of axial-load-carrying capacity is controlled by the shear spring while the displacements in flexure and bar slip springs stay constant. From the three individual spring responses (thick solid lines in three plots on the left in Figure 7.17), the

total displacement is calculated using the combined spring model and shown as the thickest solid line in the right top plot in Figure 7.17.

Specimen-2 failed in a brittle manner with no sign of yielding in the longitudinal tensile reinforcement. Concrete crushing prior to yielding of the longitudinal reinforcement is expected since the applied axial load is larger than the balanced axial load, as demonstrated on the axial load-moment interaction diagram (Figure 3.2). Therefore, the inclined crushing failure can be described as a compressive shear failure occurring as the flexural capacity is reached.

The monotonic flexure, bar slip, and shear spring models for Specimen-2 are presented in Figure 7.18. Since the calculated V_p is smaller than the lower bound for shear strength, ($V_p < \phi_{90} V_n$), according to the proposed three-spring model, the maximum lateral strength of Specimen-2 is equal to V_p , and the post-peak behavior is controlled by the flexural and bar slip springs (Figure 7.18). The displacement in the shear spring stays constant in the post-peak range.

Figure 7.19 shows the individual and combined spring models and the measured response of Specimen-3 when subjected to high compressive axial loads. Upper two plots on the left in Figure 7.19 show the calculated flexure and bar slip response under the axial load varying between 250 kips and 600 kips in compression. Since the lateral load corresponding to the maximum flexural capacity, V_p , is smaller than $\phi_{90} V_n$, the response of Specimen-3 under high axial loads is similar to that of Specimen-2. The

maximum lateral load capacity is equal to V_p , and the shear displacement remains constant in the post-peak range.

As shown in Figure 7.20, under low axial loads varying between 250 kips in compression and 56 kips in tension, the calculated shear strength, V_n , of Specimen-3 is smaller than the maximum lateral load corresponding to the flexural capacity, V_p . According to the proposed spring model, the maximum lateral strength is equal to V_n , and the post-peak behavior is controlled by shear. The middle right plot indicates that the lateral load-total displacement relation (the thickest line) hits the shear strength envelope where the displacement ductility is larger than 2.0. Therefore, shear failure is expected to occur earlier as indicated in the plot. The final response envelope is modified accordingly, and the initiation of strength degradation due to shear failure is indicated by "o" in the figure. The maximum lateral capacity of the column, V_n , at shear failure is smaller than the initially calculated V_n based on a displacement ductility smaller than 2.0.

The measured cyclic lateral load-total displacement relations and the monotonic lateral response calculated from the combined three-spring model are compared in Figure 7.21 through Figure 7.24 for the four specimens tested in this research. The calculated monotonic response of Specimen-4 is the same as that of Specimen-1. Note that, except for a slightly different concrete strength and lateral loading history, Specimen-1 and Specimen-4 were nominally identical. Considering the assumptions made in each spring model and simplicity of the component interaction, the agreement between the monotonic responses from the combined spring model and the measured responses is reasonable. The good agreements between each calculated and measured component

responses and the agreement between the total measured response and combined spring model confirm the reasonable accuracy of each individual and combined spring model developed in this study.

7.4.1 Summary of Load-Deformation Models

7.4.1.1 Flexure Model

A brief summary of the flexure model used in Chapter 5 is provided here.

- Define uniaxial material models: Define stress-strain relations for concrete and longitudinal steel using concrete cylinder and steel coupon test results. (Section 5.2)
- Carry out moment-curvature analysis using a fiber cross-section and uniaxial material properties. (Section 5.3)
- Integrate the calculated section curvatures over the height of column to get total flexural displacement. (Section 5.4)

7.4.1.2 Bar Slip Model

The proposed model to calculate lateral displacement due to longitudinal bar slip was provided in Section 5.5 and Section 5.6. The following is a brief summary of the model.

- Assume a bi-uniform stress distribution along the length of the longitudinal bar inside the joint.
- Using equilibrium in the bar, calculate bar slip at beam-column interface.
- Section rotation at the interface is slip of the reinforcing bar divided by the distance where concrete is in tension (Equation 5.23).

- Lateral displacement due to bar slip is the product of section rotation at member ends and member length (Equation 5.24).

7.4.1.3 Shear Model

A piecewise linear monotonic response envelope was developed in the previous section to model column shear behavior. The model included four shear displacement and shear force pairs.

- Cracking point: shear displacement (Equation 7.1) and the corresponding lateral load (Equation 7.6) are calculated based on linear elastic mechanics with an uncracked cross section.
- Yield point: the lateral load is calculated from section moment capacity at yield, and an equation is proposed to calculate shear displacement (Equation 7.3).
- Peak point: The shear strength is the smaller of the proposed shear strength, V_n (Equation 6.15), and the lateral strength corresponding to the maximum moment capacity of the section, V_p . The shear displacement at peak is based on the equation developed by Park and Paulay (1975) (Equation 7.5).
- Loss of axial capacity: Shear strength is equal to zero. Shear displacement is the difference between the total displacement suggested by Moehle et al. (2001) and the sum of maximum flexure and bar slip displacements (Equation 7.7).

7.4.1.4 Combined Three-Spring Model

The combined three-spring model includes

- The response before the peak column strength is reached: Total lateral displacement is the summation of flexure, bar slip, and shear displacements from the three individual springs, in which spring forces are the same.
- The peak column strength is the smaller of calculated shear strength, V_n (Equation 6.12), and the lateral load corresponding to the maximum flexural capacity, $V_p (= 2M_p/L)$.
- If $V_p \geq \phi_{90} V_n = 0.85 V_n$, the post-peak behavior is influenced/dominated by shear (e.g., Specimen-1, Figure 7.17, and Specimen-3 under low axial load, Figure 7.20). Total lateral displacement at axial load failure is the summation of flexural and bar slip displacements at peak lateral load and the shear displacement calculated from Equation 7.7. Note that there still is some chance of flexural failure rather than shear failure in reality.
- If $V_p < \phi_{90} V_n = 0.85 V_n$, the post-peak behavior is dominated by flexure (e.g., Specimen-2, Figure 7.18, and Specimen-3 under high axial load, Figure 7.21). Total lateral displacement at axial load failure is the summation of calculated flexural and bar slip displacements at failure, and the shear displacement where V_p is reached. Note that there still is some possibility of shear failure in reality.

7.5 COMPONENT RESPONSE CONTRIBUTIONS

Total displacements measured at the top of each column and the calculated flexure, bar slip, and shear displacement histories are shown in Figure 7.25. From the plots, contribution of each displacement component to the total response can be obtained at any time during the tests.

The shear displacement and total displacement distributions over the height of each specimen are shown in Figure 7.26. The figure highlights the contribution of shear displacements to total response at the performance limit states described in Section 4.6. The displacement corresponding to the limit state “loss of lateral capacity” (“loss of V” in Figure 7.26) was defined as the measured displacement at which the lateral load dropped to 80 percent of the maximum lateral strength. As illustrated in the figure, the contribution of shear component increases substantially at this limit state where the lateral strength is lost significantly (more than 20 percent) due to shear failure.

The measured cyclic and calculated monotonic lateral load-flexure, bar slip, shear, and total displacement relations are shown in Figure 7.27 through Figure 7.30 for the four test specimens. The peaks of the first cycles in each displacement level are marked in the figures. Note that the individual monotonic spring models presented in the figures were modified according to the combined spring model in the previous section, and they are different from the individual flexure, bar slip and shear components plotted in Chapter 5 and Section 6.3.

The bottom two plots in Figure 7.27 through Figure 7.30 also show the contribution of each displacement component to total displacement at the peak displacement during the first cycle of each displacement level. The maximum absolute error was calculated by subtracting the summation of calculated individual component displacements from the total measured displacement at the peak of the first cycle of each displacement level. The history of the maximum absolute error can be obtained from Figure 7.25.

In all specimens, the percentage contribution of bar slip deformations, which is about twenty to forty percent of the total deformations, did not change significantly during the tests. In Specimen-1, which was subjected to relatively low axial load, the contribution of shear deformations increased gradually to approximately twenty percent of the total deformations at a displacement ductility of approximately two, at which time shear strength degradation was observed and shear deformations increased dramatically (Figure 7.27). In contrast, Specimen-2 under very high axial load had nearly negligible shear deformations throughout testing until just before the failure (Figure 7.28). It appears that the sudden failure of this specimen was a result of combined brittle flexure and compressive shear failures. In Specimen-3 (Figure 7.29), which was subjected to varying axial load, the apparent behavior was a combination of behavior of Specimen-1 under smaller axial loads and the behavior of Specimen-2 under larger axial loads. Figure 7.30 shows that the behavior of Specimen-4 under monotonic lateral load was similar to that of Specimen-1, except that the shear deformations did not increase dramatically near the end of the test as in the cyclic loading case.

7.6 SUMMARY OF KEY RESULTS

This section provides a summary of measured and calculated displacements, lateral load and moments at different performance limit states. Typical limit states and the corresponding damage and performance descriptions for nonductile columns were provided in Section 4.6. Damage descriptions and the related displacement ductility or transient drift ratios for the four damage levels and engineering limit states were summarized in Table 4.3. The four engineering limit states included: 1) the first yielding

in the longitudinal reinforcement; 2) peak lateral strength; 3) loss of lateral-load-carrying capacity, which is assumed to be reached when the lateral strength becomes smaller than 80 percent of the maximum lateral strength; and 4) loss of axial-load-carrying capacity.

Table 7.1 shows the measured lateral displacements at three of these critical limit states including the first yielding, Δ_y , loss of lateral-load-carrying capacity, Δ_u , and loss of axial-load-carrying capacity, Δ_{ug} . The measured displacement ductilities are also reported in the table. The displacement ductilities, $\mu_{\delta u}$ and $\mu_{\delta ug}$ corresponding to the loss of lateral and axial-load-carrying capacities are defined as the respective ultimate displacements, Δ_u and Δ_{ug} , divided by the measured lateral displacement at first yielding in the longitudinal reinforcement, Δ_y . The measured displacement ductilities, varied between 1.3 and 3.1 at loss of lateral-load-carrying capacity, and varied between 2.8 and 6.0 when the axial capacity was lost.

The measured and calculated flexure, bar slip, and shear displacements at yield level are compared in Table 7.2. Note that the longitudinal bars in Specimen-2 did not yield in tension (see Appendix E). The reported values for Specimen-2 correspond to the first yielding of longitudinal bars in compression. As demonstrated in Figure 4.29, in this specimen the peak strength was reached immediately after the first yielding in compression was observed. Therefore, the predicted flexure and bar slip displacement components at yield level were calculated at the peak lateral load.

The flexural displacements, which are calculated from the section moment-curvature analysis, and the measured flexural displacements shown in Table 7.2 are relatively

close. On the other hand, bar slip displacements are consistently overestimated at yield level. The measured shear displacements are underestimated by Equation 7.3 with relatively large variation. The summation of flexure, bar slip and shear displacement components, Δ_{sum} , compares reasonably well with the measured total yield displacement. The mean measured-to-calculated total yield displacement ratio is 1.08 with a standard deviation of 0.09.

The calculated and measured lateral loads and moments at yield level are summarized in Table 7.3. The lateral strength at yield level, $V_{y,calc.}$, is calculated by dividing the yield moment capacity, $M_{y,calc.}$, from section moment-curvature analysis by half of the column height. The mean ratio between measured and calculated yield strengths ($V_{y,test}/V_{y,calc.}$) is 0.92. The measured yield moments reported in the table are the larger of the measured moments at the top and bottom of the test columns (i.e., larger of M_B and M_T in Figure 4.38).

The peak lateral strengths calculated from the moment-curvature analysis are compared with the maximum measured lateral strengths in Table 7.4. The agreement between the measured and calculated values is due to the fact that all specimens reached their flexural capacity before they failed in shear. Even though the measured peak strength in Specimen-2 (80.7 kips) seems to be sixteen percent larger than the calculated strength corresponding to the maximum flexural capacity (69.3 kips), the measured peak strength in the other loading direction, which was 67.7 kips, was smaller than the calculated flexural strength of 69.3 kips (see Figure 7.22). Overall, the maximum measured moments appear to be larger than the moment capacities calculated from both ACI-318-

02 procedure (M_{ACI}) and the section moment-curvature analysis ($M_{p,calc.}$). The moment capacity tends to be underestimated for cases of higher axial loads. The measured moments reported in the table are the larger of the moments at the top and bottom of the test columns reported in Table 4.6.

The displacement, lateral strength, and moments at the performance level corresponding to the loss of lateral-load-carrying capacity are summarized in Table 7.5. Loss of lateral-load-carrying capacity is defined as the level where lateral load drops to 80 percent of the maximum lateral strength. The lateral strength and moments calculated from the section moment-curvature analysis are also compared in the table.

The data presented in this section indicate that flexure, bar slip, and total displacements, lateral load and moment values calculated from the moment curvature analysis compared reasonably well with the experimental data. This is an expected result as the specimens reached their maximum flexural capacity before shear failure. However, at peak lateral strength, the moments were underestimated by both ACI 318-02 and moment-curvature analysis.

Table 7.1 Summary of key displacement results

Specimen	P	Δ_y	Δ_u	Δ_{ug}	$\mu_{\delta u}$	$\mu_{\delta_{ug}}$
	kips	in.	in.	in.		
1	-150	1.03	2.97	5.76	2.88	5.59
2	-600	0.79	1.02	2.18	1.29	2.76
3	-600	0.82	2.23	3.42	2.72	4.17
	0	1.13	3.41	3.40	3.02	3.01
4	-150	1.06	3.33	6.35	3.14	5.99

Table 7.2 Summary of measured and calculated displacements at yield level (C*: under 600 kips compressive axial load; T*: under 56 kips tensile axial load)

No.	$\Delta_{flexure}$ (in.)		$\Delta_{test}/$ $\Delta_{calc.}$	Δ_{slip} (in.)		$\Delta_{test}/$ $\Delta_{calc.}$	Δ_{shear} (in.)		$\Delta_{test}/$ $\Delta_{calc.}$	Δ_{sum} (in.)	$\Delta_{test}/$ Δ_{sum}
	test	calc.		test	calc.		test	calc.			
1	0.50	0.46	1.09	0.27	0.39	0.69	0.011	0.008	1.42	0.93	1.11
2	0.39	0.50	0.78	0.18	0.27	0.67	0.003	0.006	0.52	0.83	0.95
3C*	0.50	0.49	1.02	0.20	0.27	0.74	0.007	0.006	1.17	0.82	1.00
3T*	0.65	0.38	1.71	0.22	0.33	0.67	0.009	0.006	1.44	0.96	1.18
4	0.48	0.46	1.04	0.27	0.39	0.69	0.007	0.007	0.95	0.92	1.15
mean			1.13			0.69			1.10		1.08

Table 7.3 Summary of calculated and experimental shear and moments at yield level

Spec.#	$V_{y,test}$	$V_{y,calc.}$	$V_{y,test}/$ $V_{y,calc.}$	$M_{y,test}$	$M_{y,calc.}$	$M_{y,test}/$ $M_{y,calc.}$
	kips	kips		k-in.	k-in.	
1	55.7	59.5	0.94	3370	3450	0.98
2	64.1	69.3	0.93	4570	4020	1.14
3 (P=600k)	61.8	69.3	0.89	4400	4020	1.09
3 (P=0k)	43.6	47.1	0.93	3150	2730	1.15
4	53.0	59.5	0.89	3210	3450	0.93
mean			0.92			1.06

Table 7.4 Summary of measured and calculated lateral load and moments at peak level

Spec. No.	P (kips)	V (kips)		$V_{test}/V_{calc.}$	M (k-in.)		M_{test}/M_{ACI}	$M_{p,calc.}$ (k-in.)	$M_{test}/M_{p,calc.}$
		test	calc.		test	ACI			
1	-150	70.8	67.6	1.05	4330	3870	1.12	4050	1.07
2	-600	80.7	69.5	1.16	5100	3720	1.37	4020	1.27
3	-600	67.7	69.3	0.98	4740	3720	1.28	4020	1.18
	+56	55.5	57.9	0.96	3340	3060	1.09	3360	1.00
4	-150	66.2	67.6	0.98	4110	3900	1.05	4050	1.02
mean				1.02			1.18		1.11

Table 7.5 Summary of key results at loss of lateral-load-carrying capacity (at 80% of maximum strength)

Specimen	P (kips)	Δ_u (in.)	$\mu_{\delta u}$	V_u (kips)	$V_{u,calc.}$ (kips)	M_u (k-in.)	$M_{u,calc.}$ (k-in.)
1	-150	2.97	2.88	54.4	63.0	3780	3650
2	-600	1.02	1.29	64.5	52.4	4940	3040
3	-600	2.23	2.72	54.2	58.9	4600	3420
	+56	3.41	4.49	44.4	57.9	3020	3360
4	-150	3.33	3.14	53.0	63.0	3720	3650

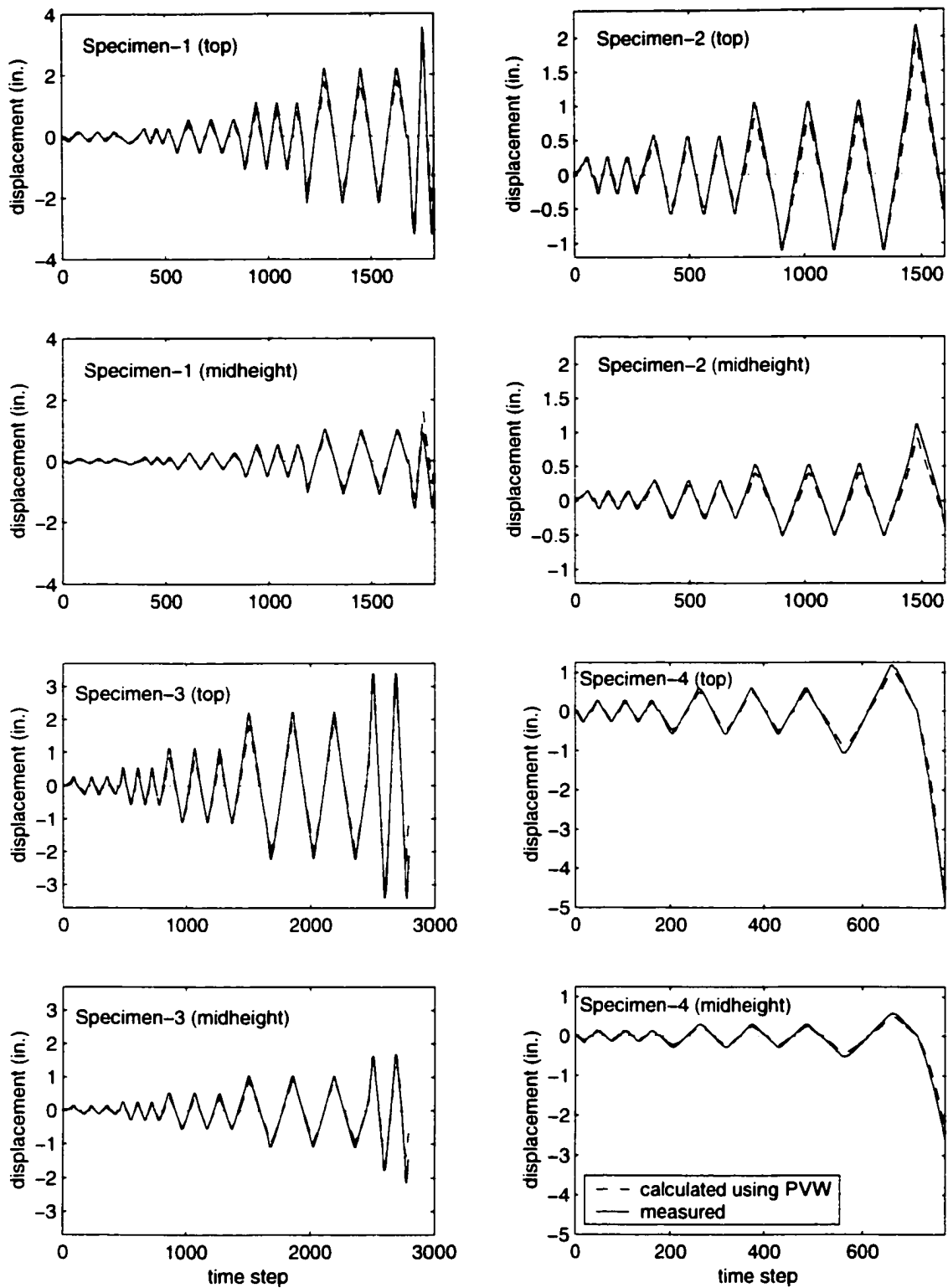


Figure 7.1 Comparison of calculated and measured lateral displacements at the top and midheight of each specimen

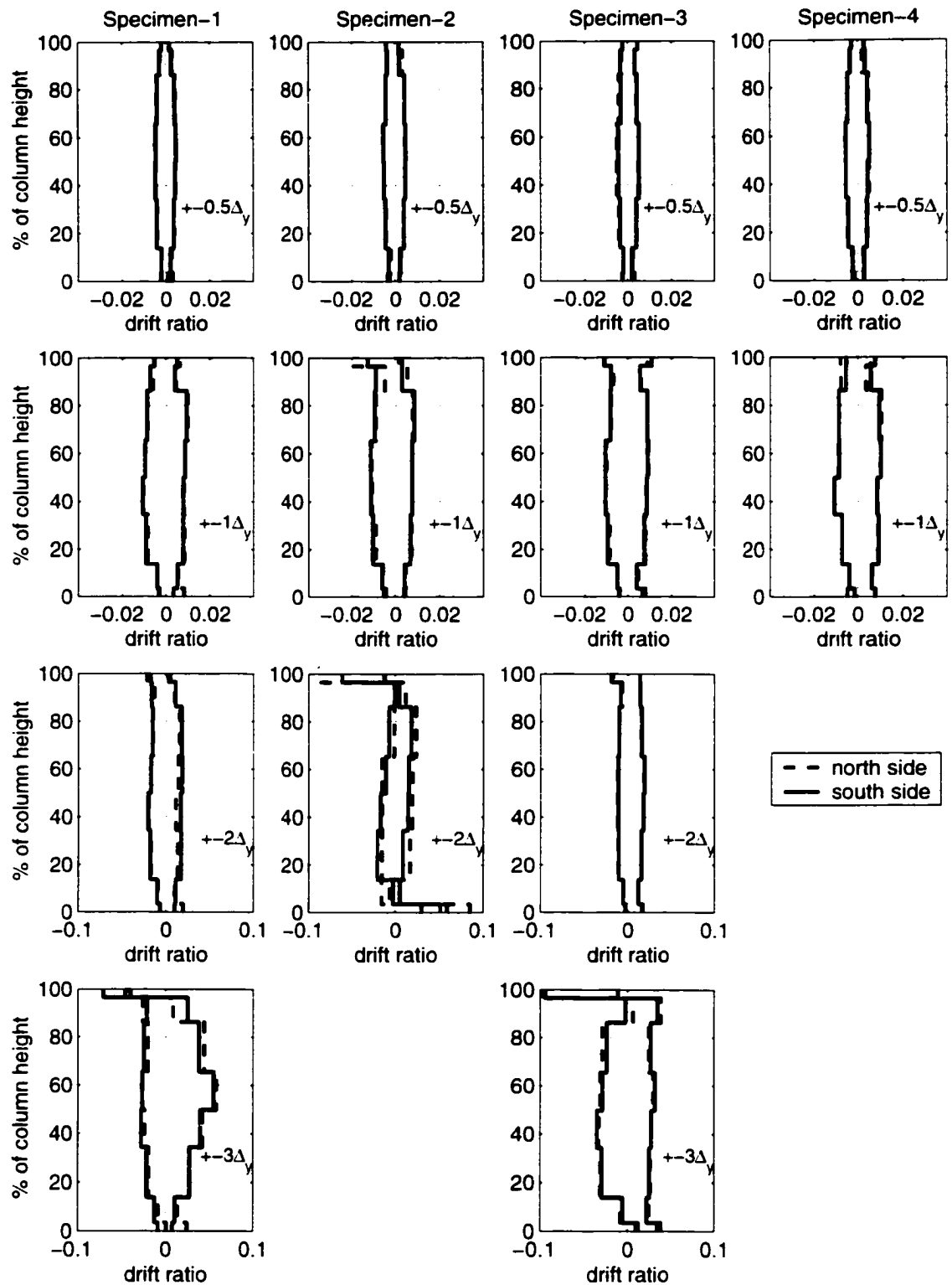


Figure 7.2 Total drift ratio distribution over the height of each column at different lateral displacement levels

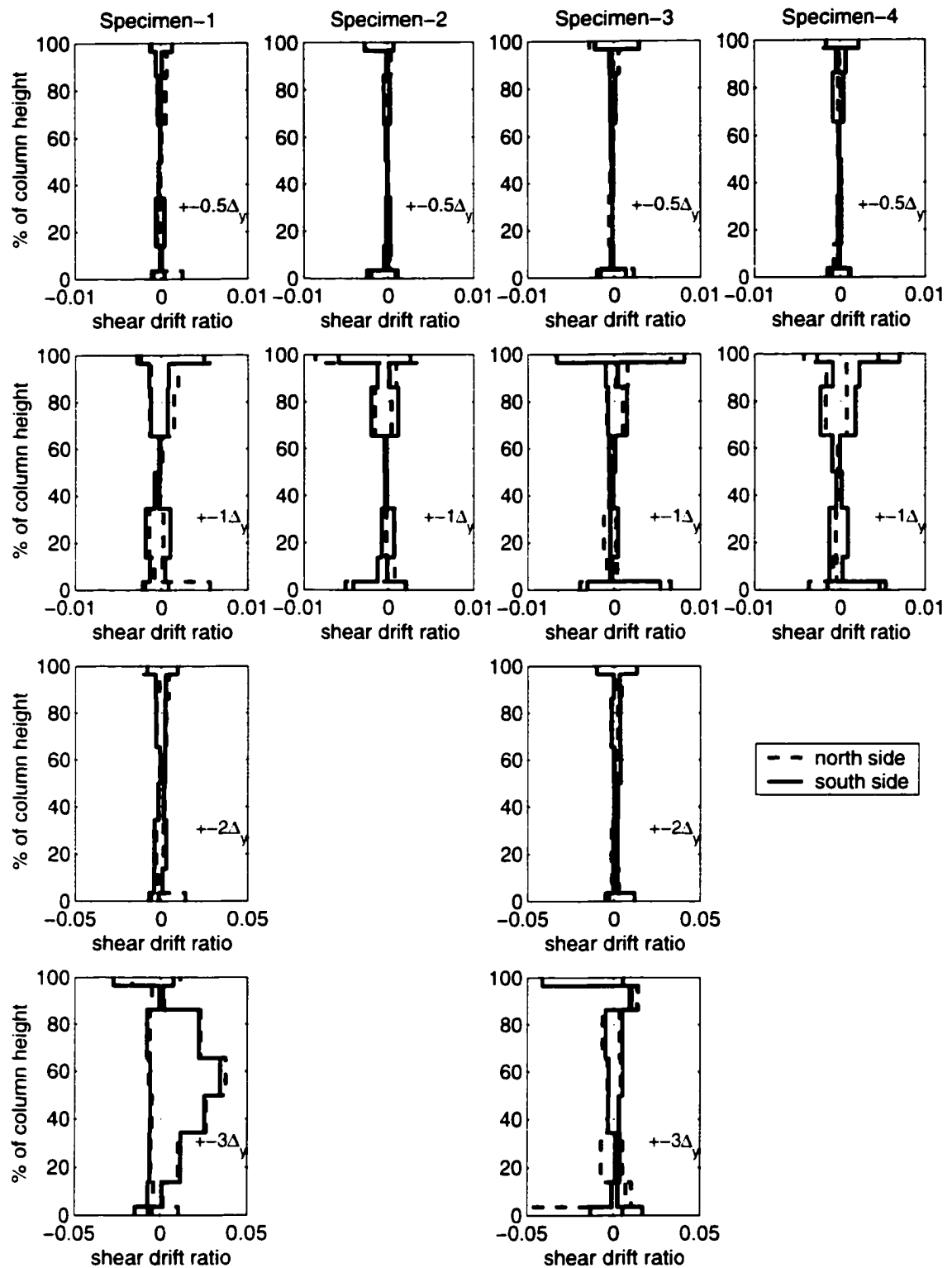


Figure 7.3 Shear drift distribution over the height of each column at different lateral displacement levels

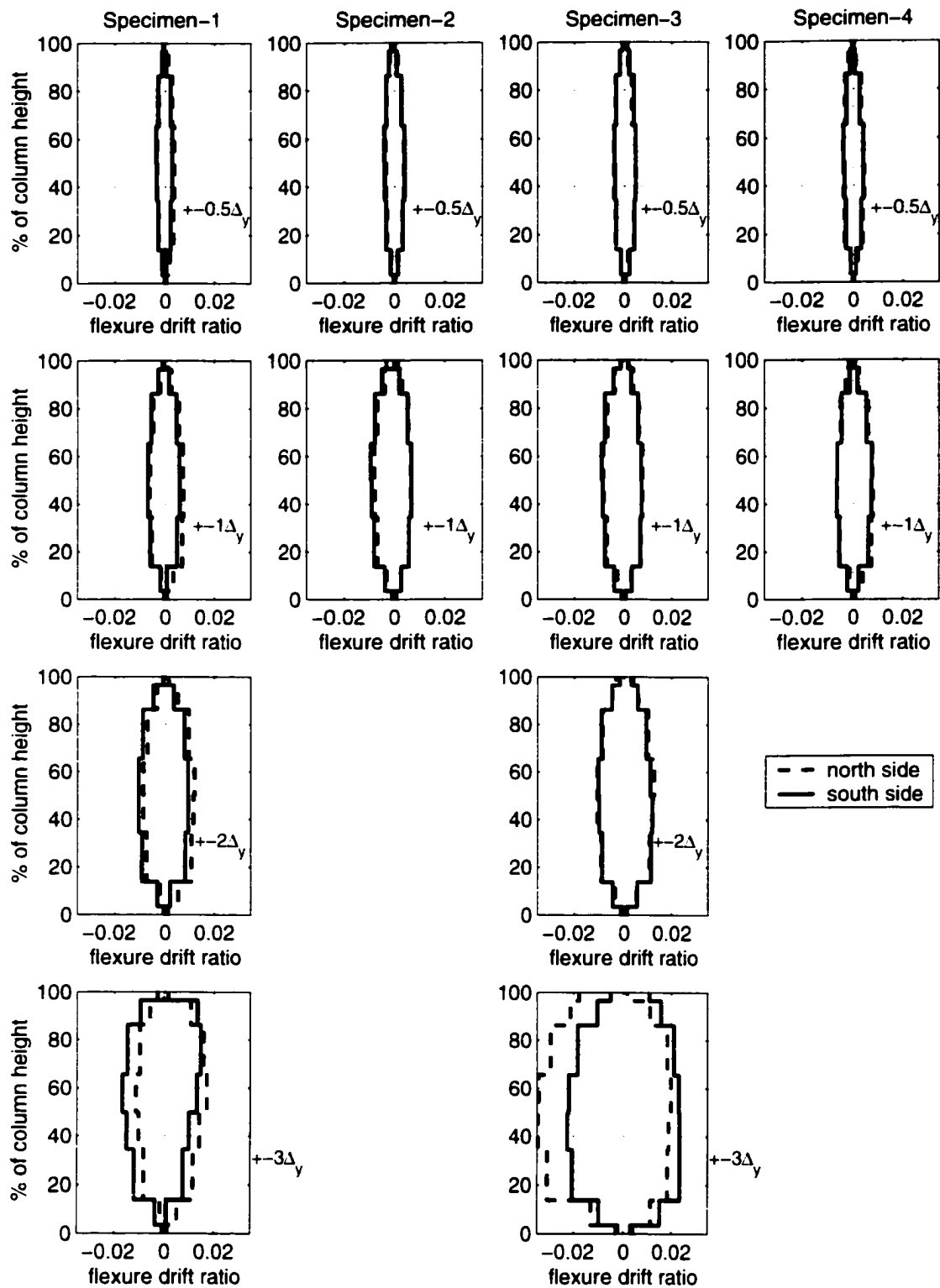


Figure 7.4 Flexure drift distribution over the height of each column at different lateral displacement levels

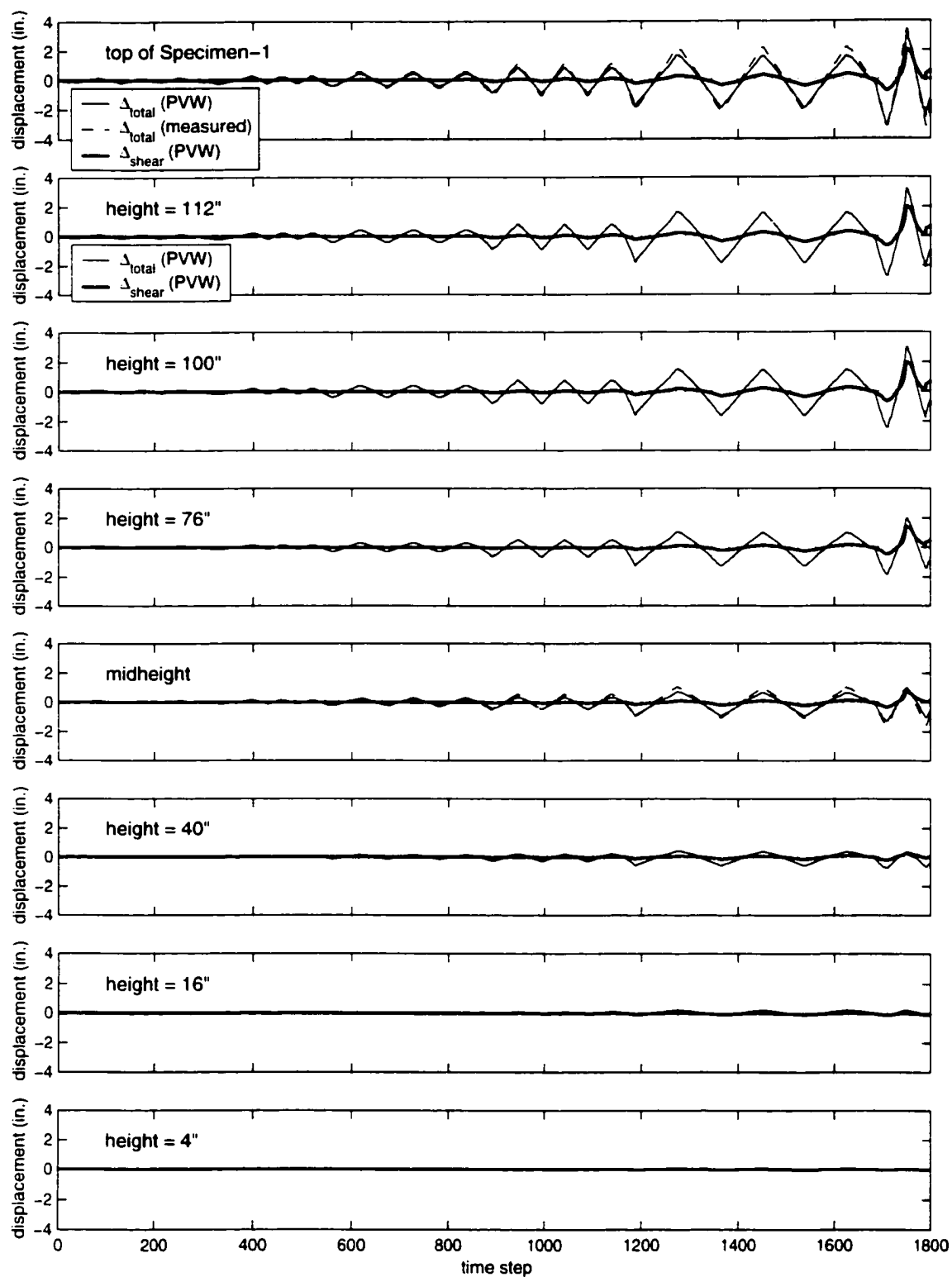


Figure 7.5 Total and shear displacement distribution over the height of Specimen-1

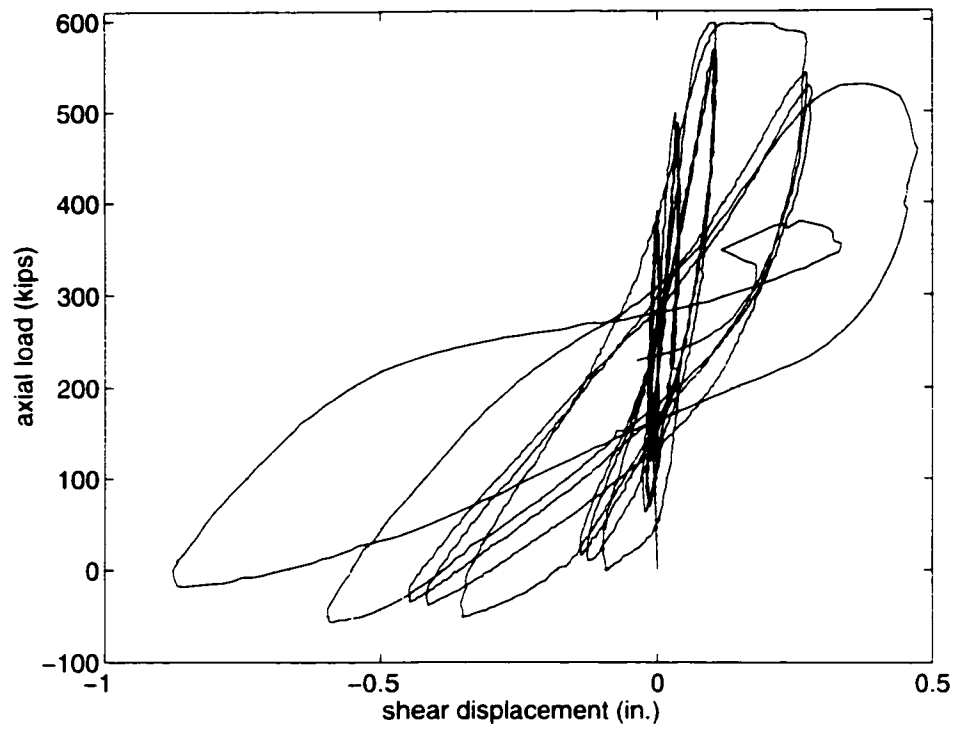


Figure 7.6 Measured axial load-shear displacement relations for Specimen-3

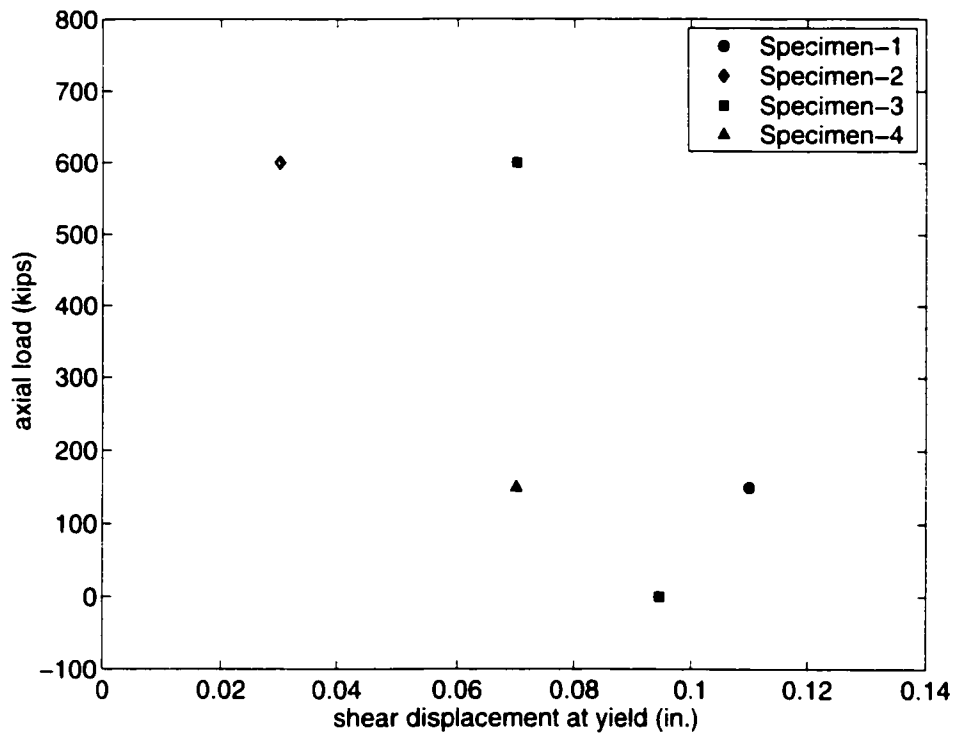


Figure 7.7 Axial load versus measured shear displacement at yield

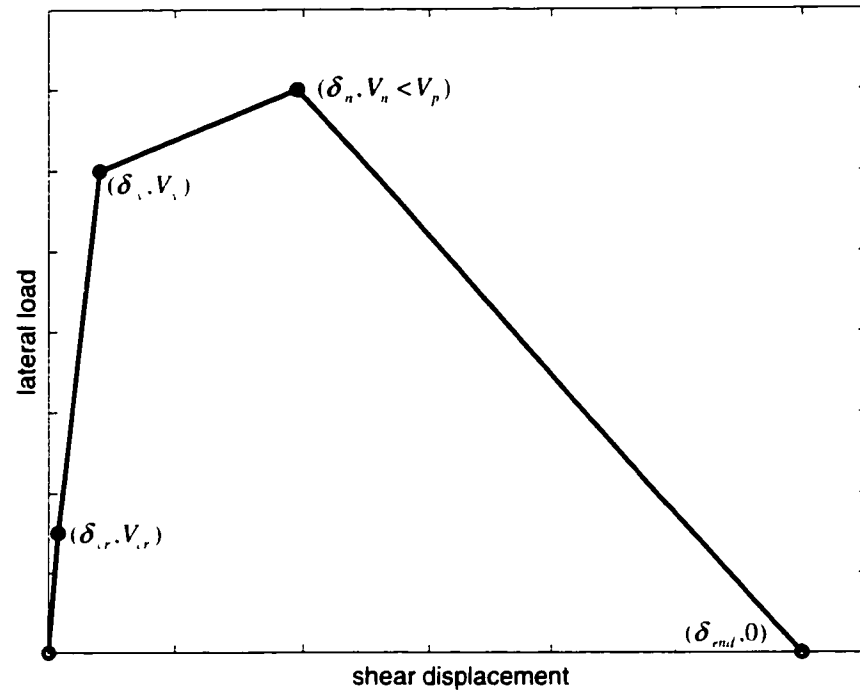


Figure 7.8 Proposed monotonic lateral load-shear displacement model

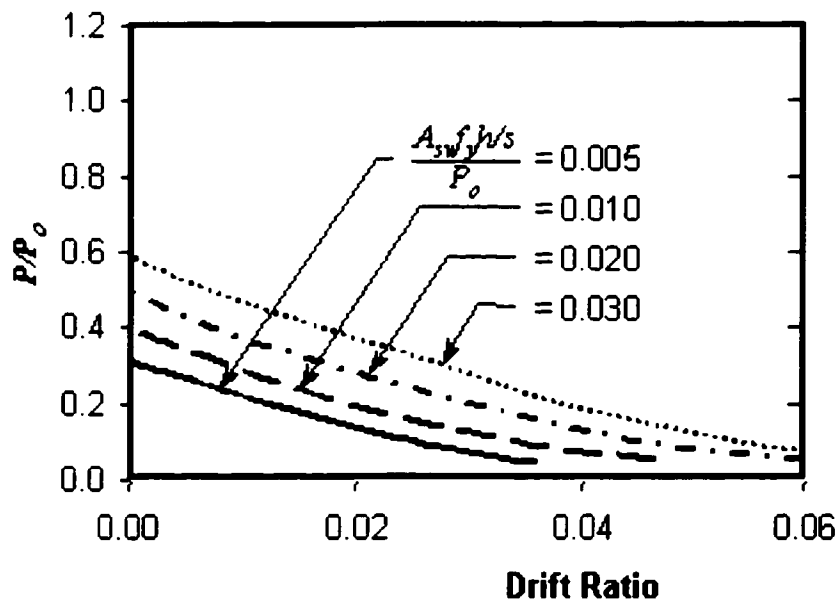


Figure 7.9 Relations among axial load, transverse reinforcement, and drift capacity at loss of axial load capacity (Moehle et al. 2000)

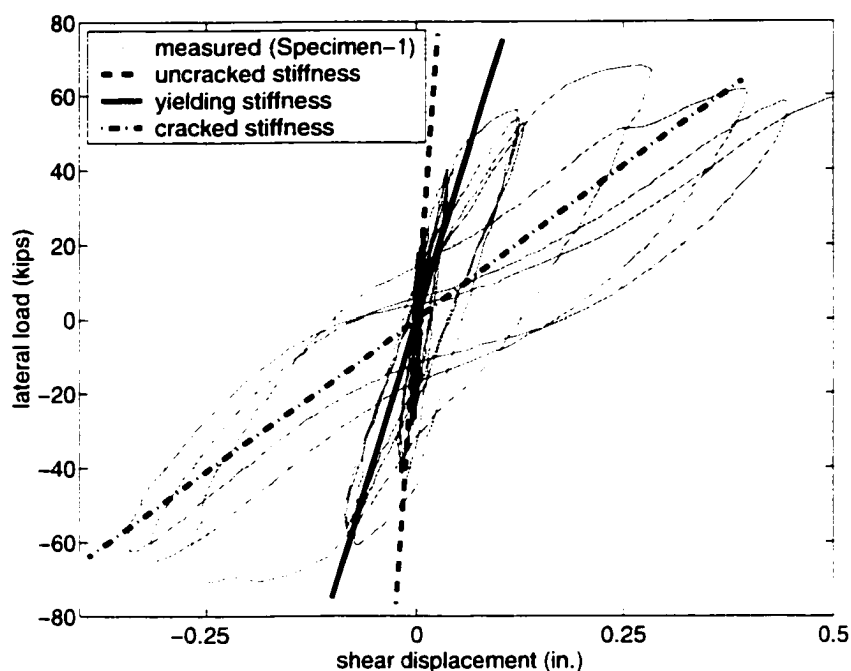


Figure 7.10 Shear stiffness and test data (Specimen-1 with $\Delta_y/4$, $\Delta_y/2$, Δ_y and $2\Delta_y$ cycles)

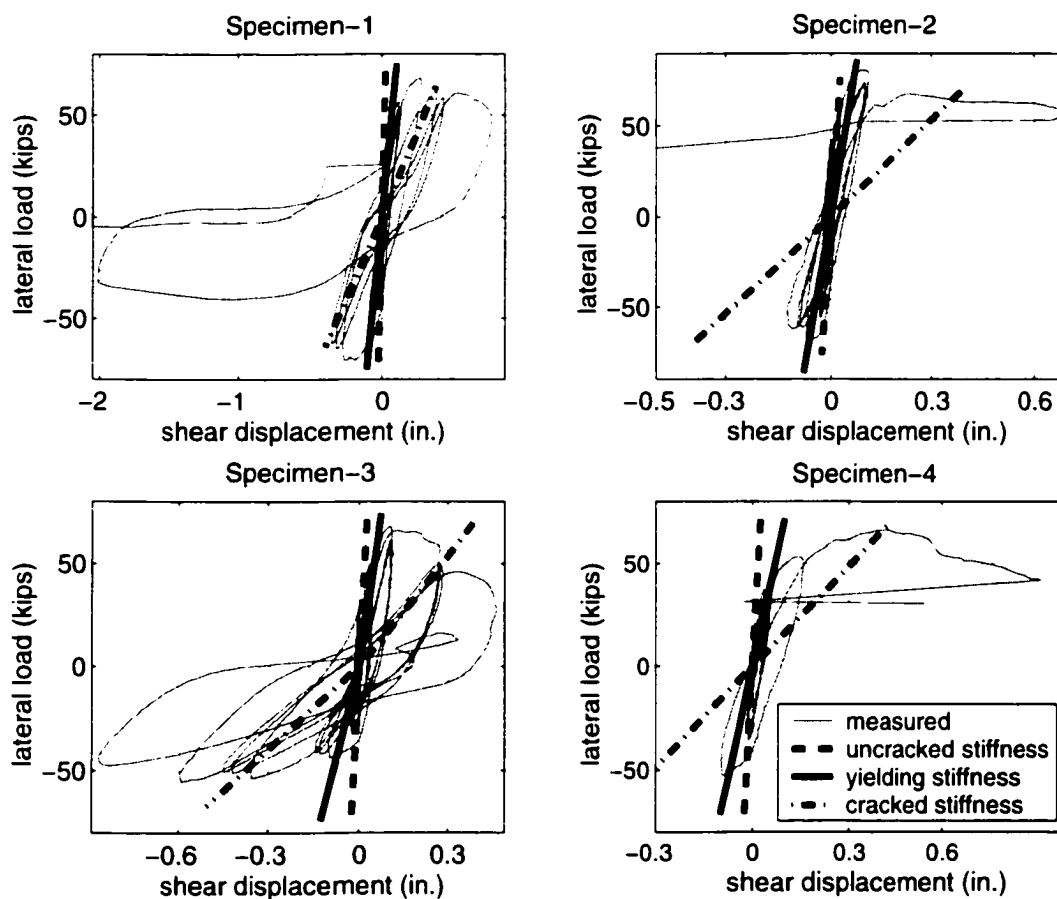


Figure 7.11 Comparison of estimated shear stiffness with test data

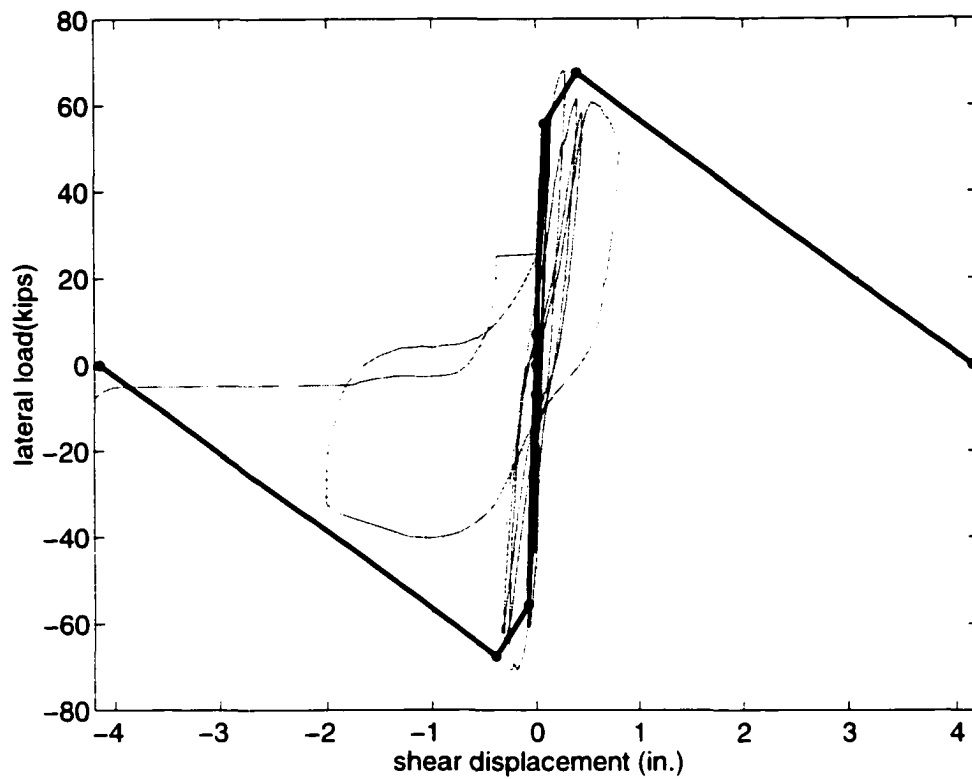


Figure 7.12 Lateral load-shear displacement relations (Specimen-1)

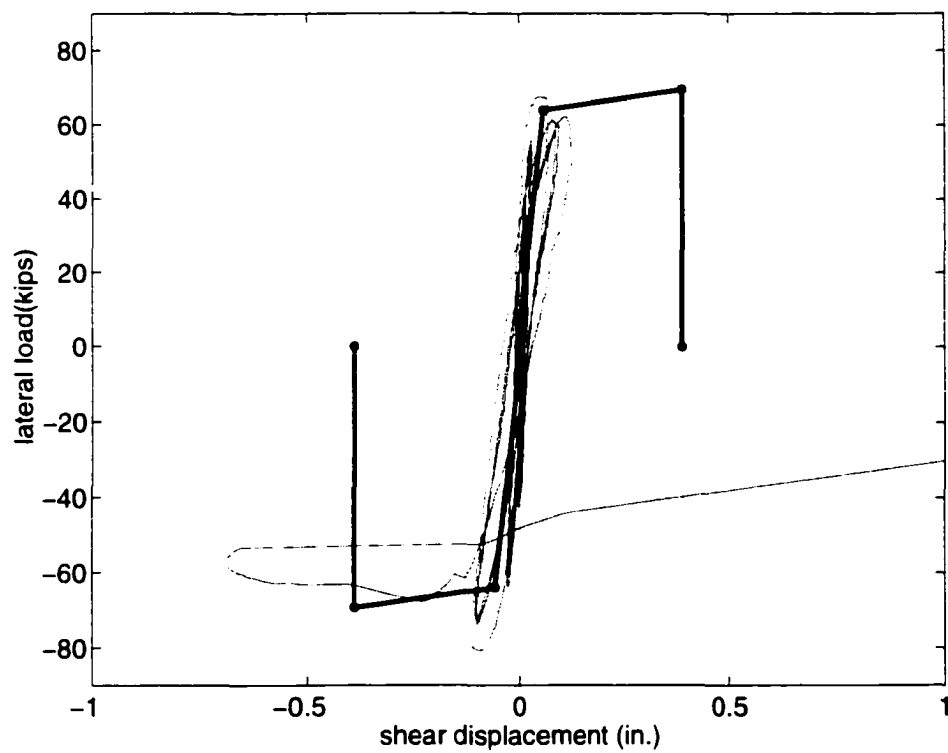


Figure 7.13 Lateral load-shear displacement relations (Specimen-2)

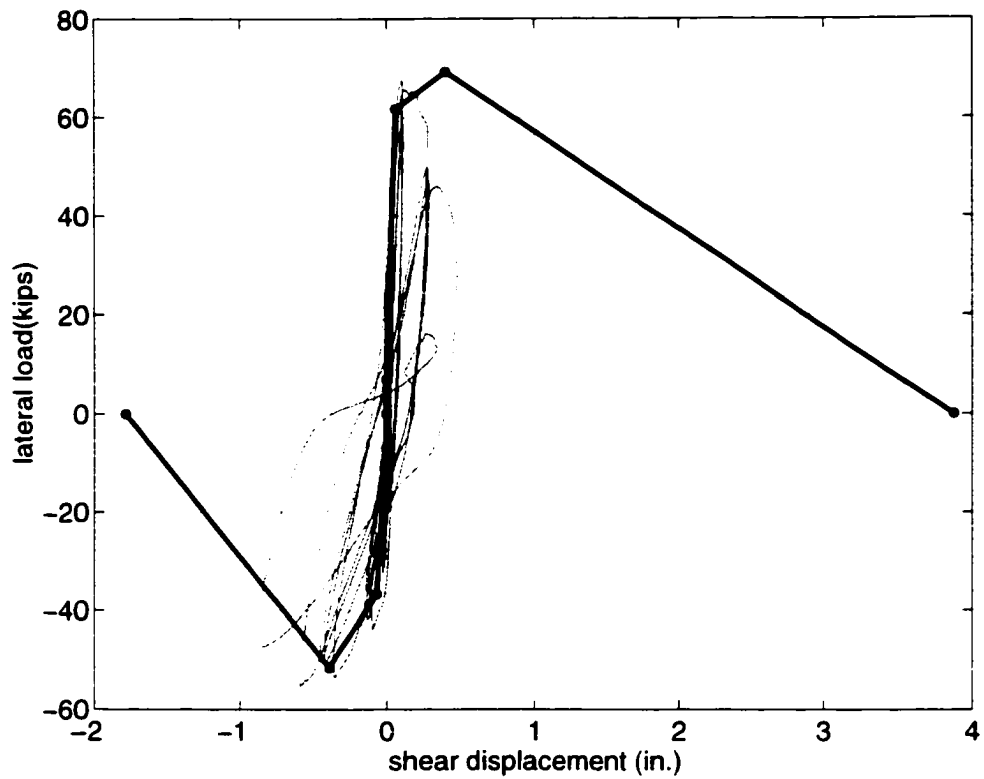


Figure 7.14 Lateral load-shear displacement relations (Specimen-3)

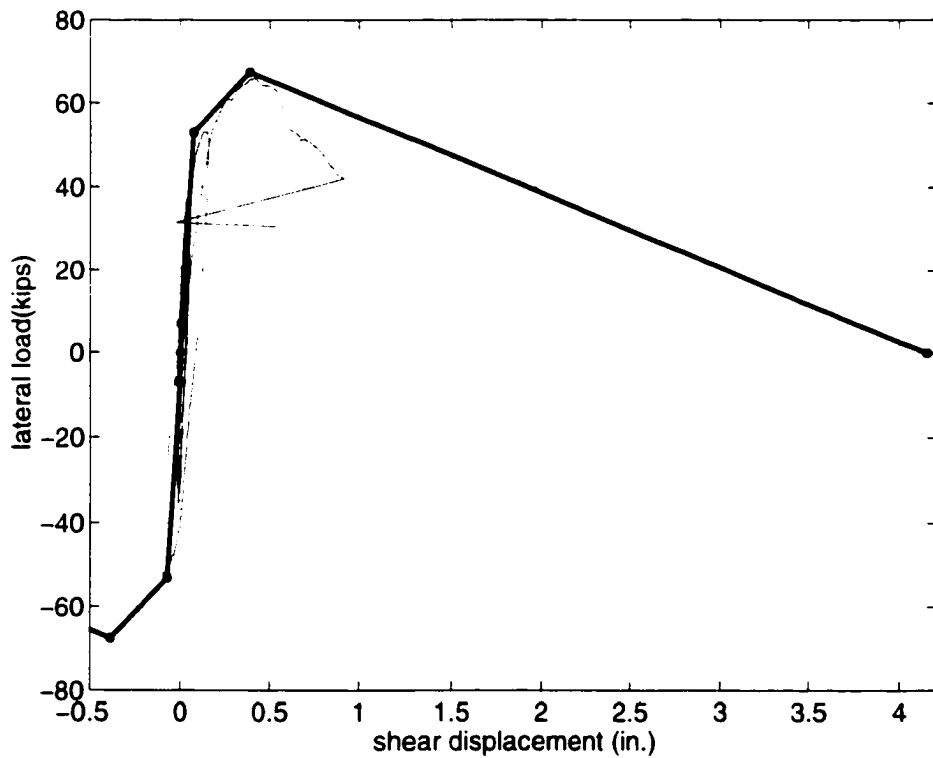


Figure 7.15 Lateral load-shear displacement relations (Specimen-4)

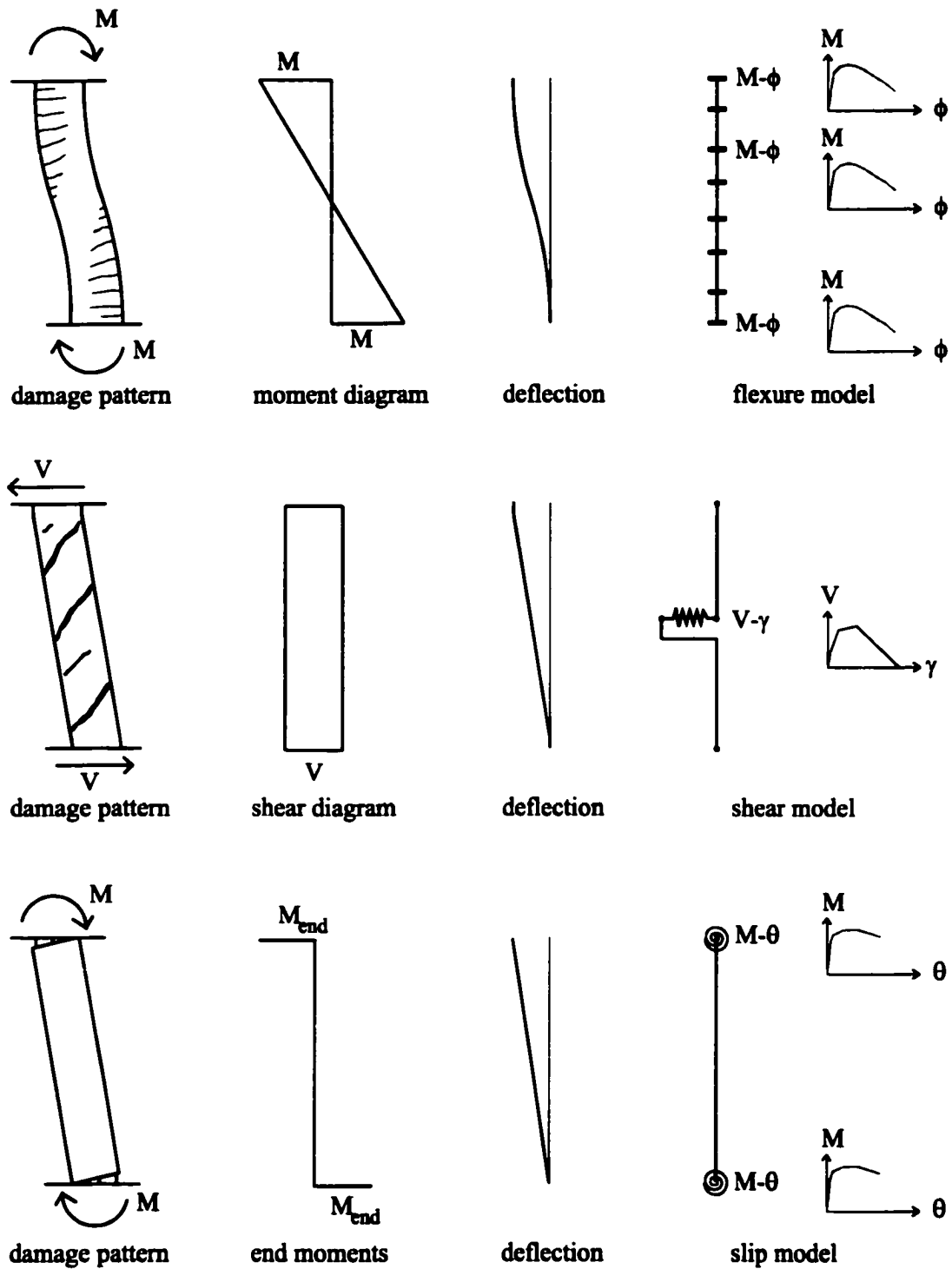


Figure 7.16 Idealized monotonic flexure, shear and slip element models for reinforced concrete columns

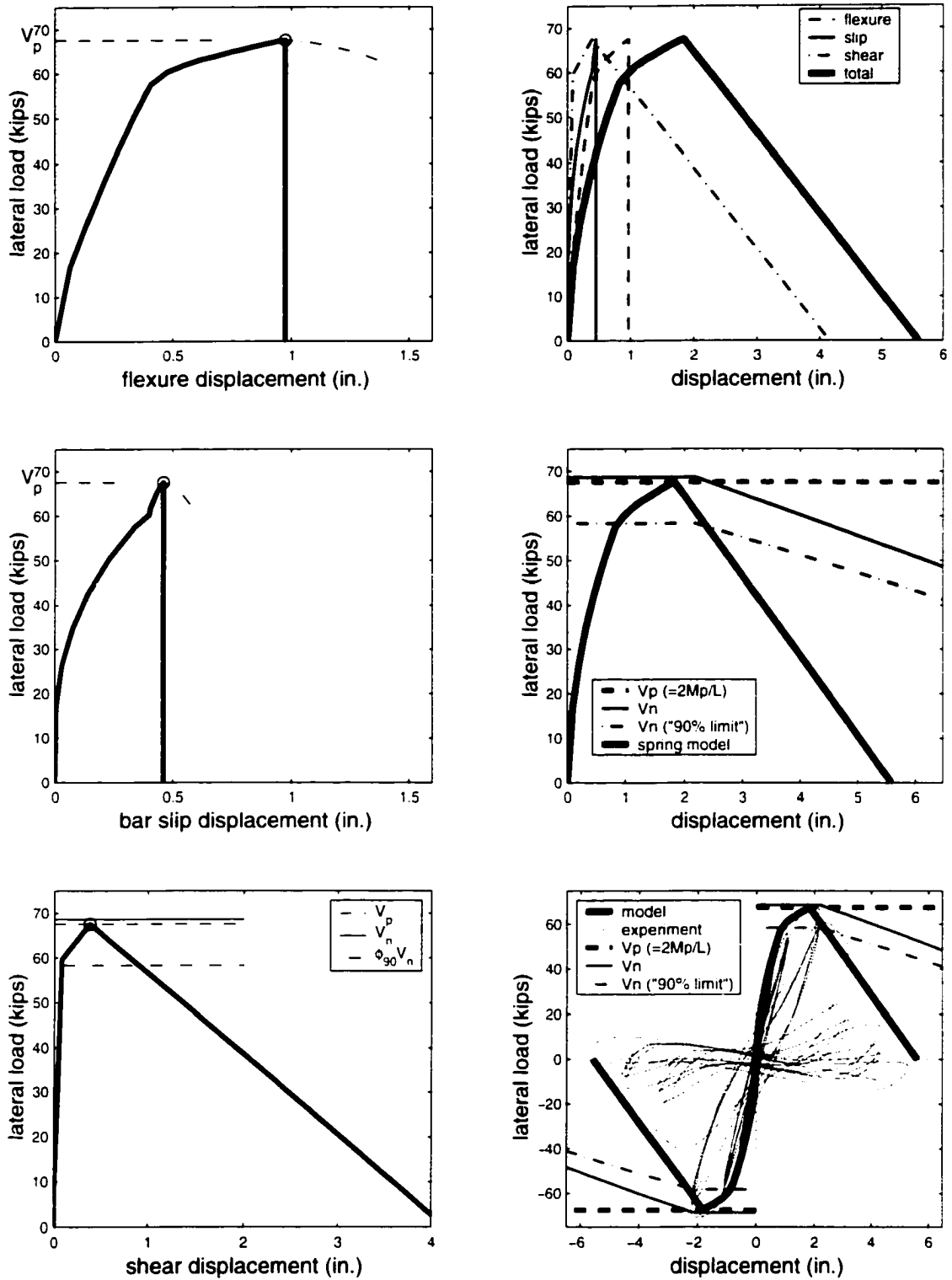


Figure 7.17 Monotonic spring models for flexure, bar slip and shear, and comparison of combined spring model with test data (Specimen-I with $P=150$ kips)

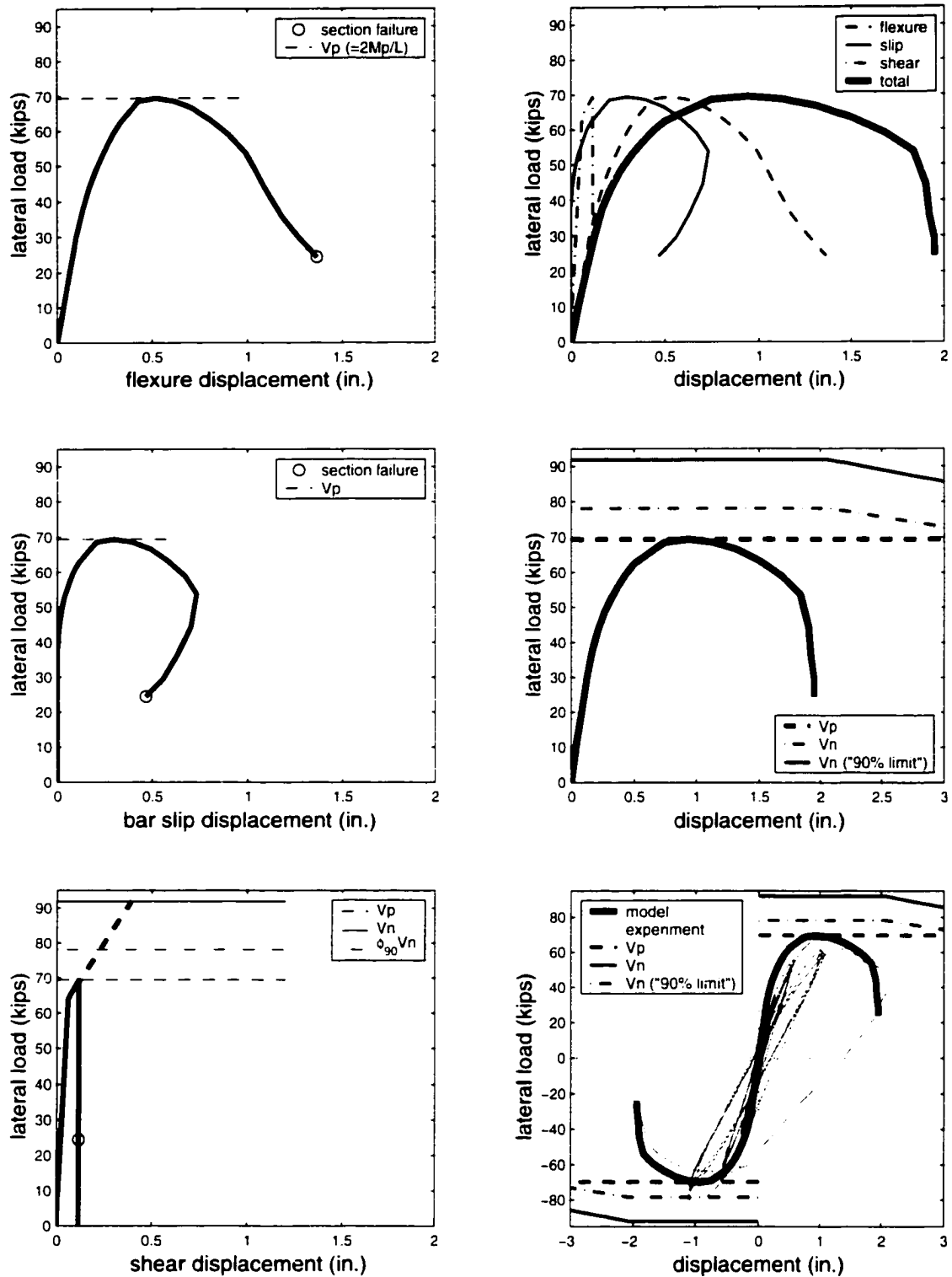


Figure 7.18 Monotonic spring models for flexure, bar slip and shear, and comparison of combined spring model with test data (Specimen-2 with $P=600$ kips)

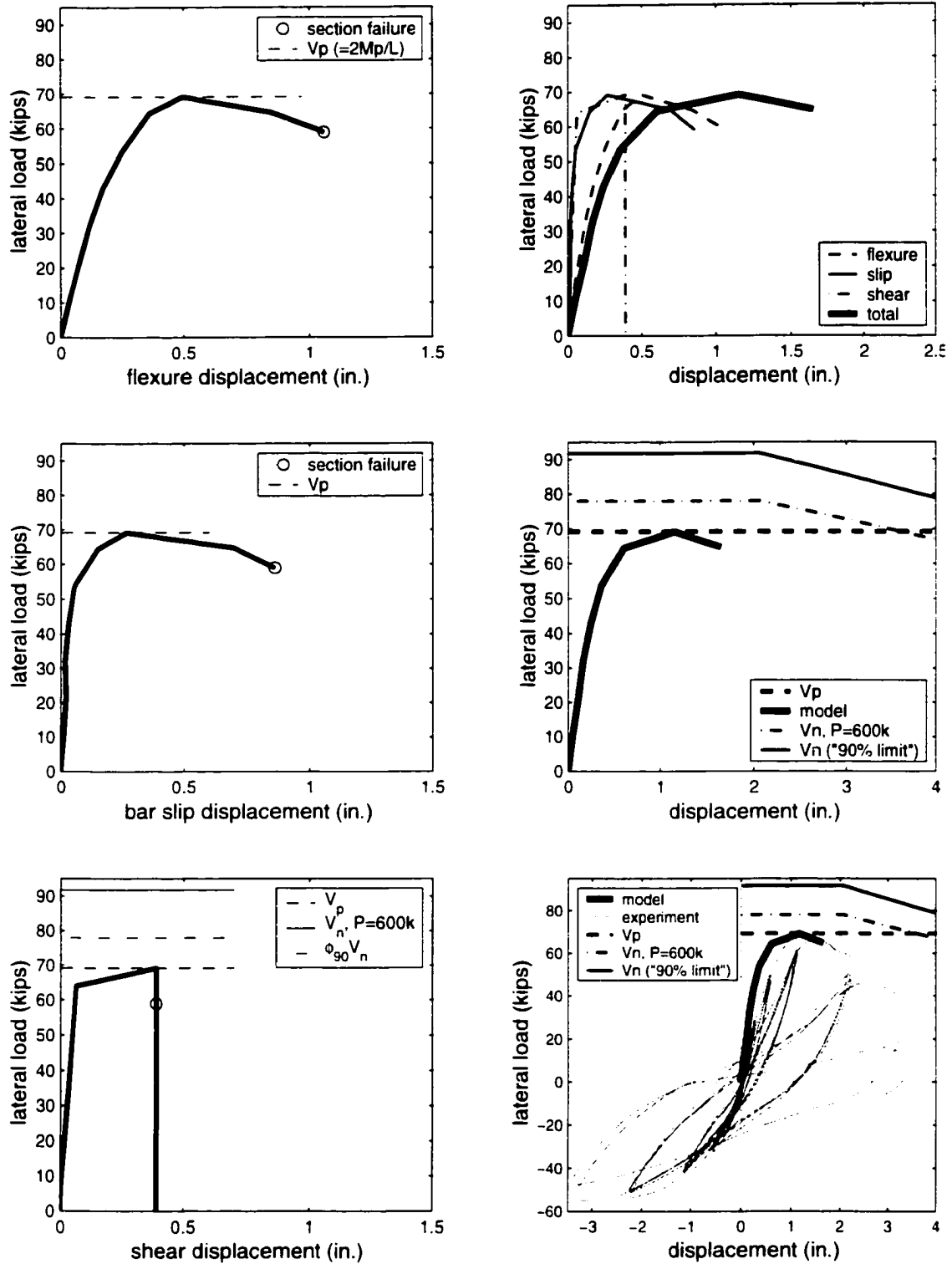


Figure 7.19 Monotonic spring models for flexure, bar slip and shear, and comparison of combined spring model with test data (Specimen-3 under high axial load2)

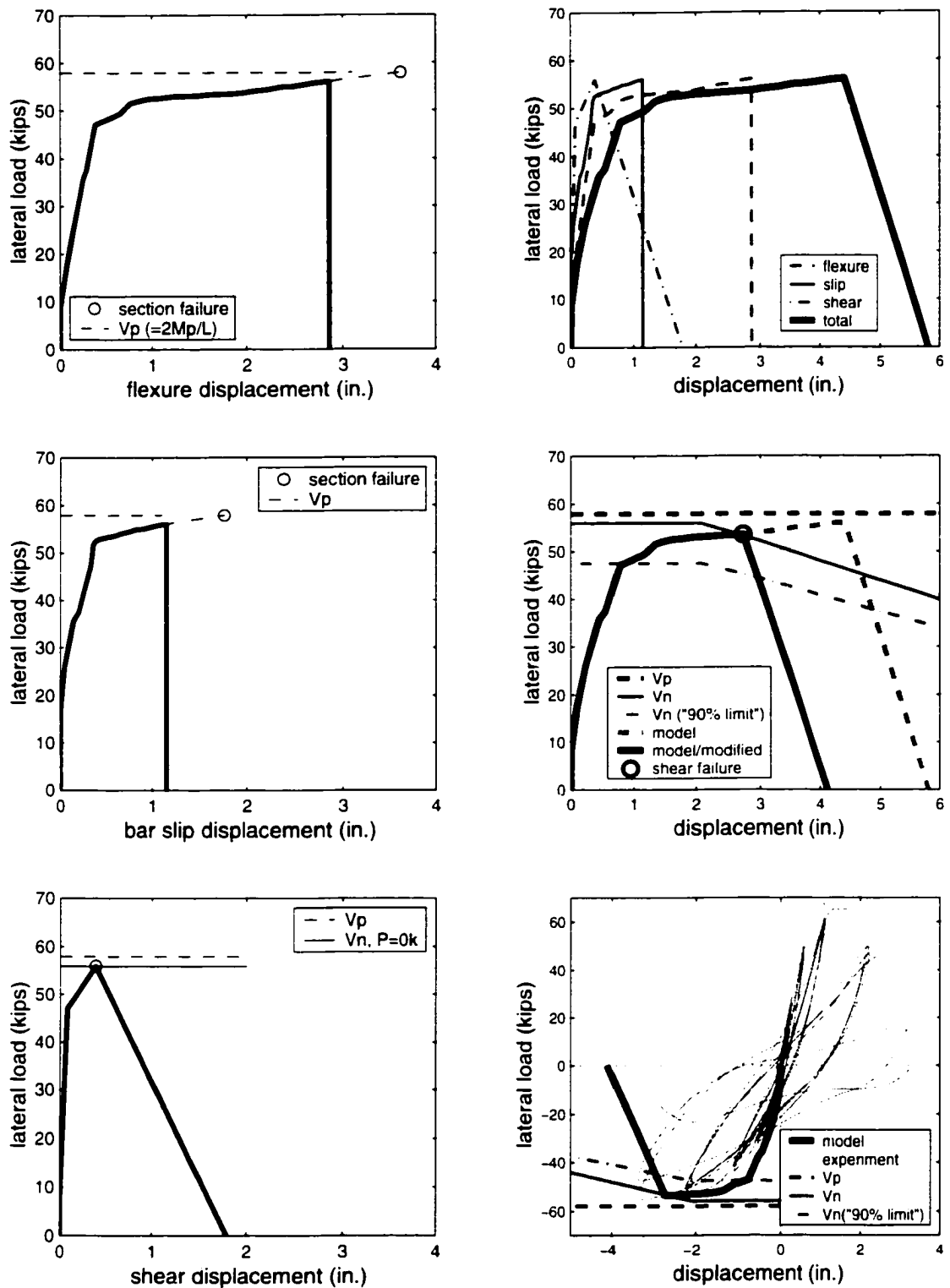


Figure 7.20 Monotonic spring models for flexure, bar slip and shear, and comparison of combined spring model with test data (Specimen-3 under low axial load)

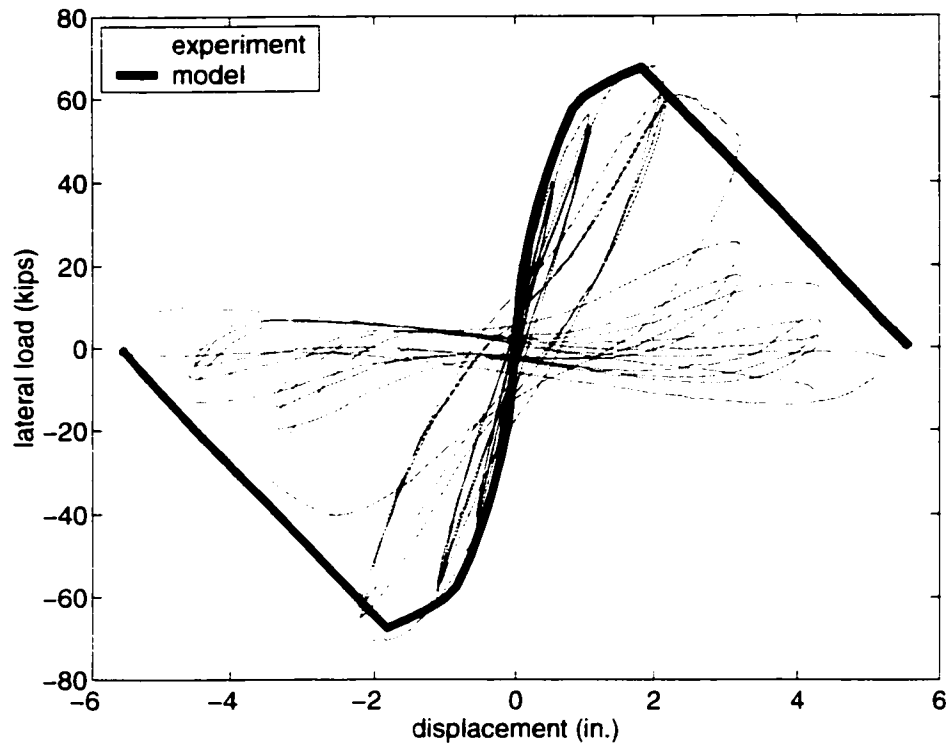


Figure 7.21 Lateral load-displacement relations with monotonic model (Specimen-1)

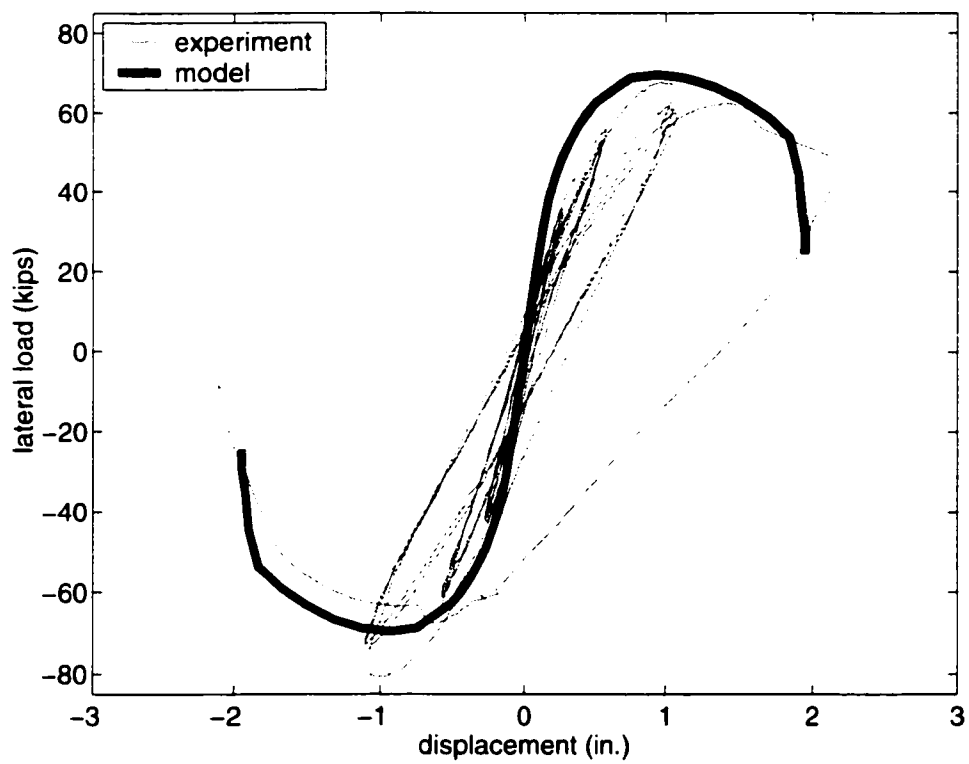


Figure 7.22 Lateral load-displacement relations with monotonic model (Specimen-2)

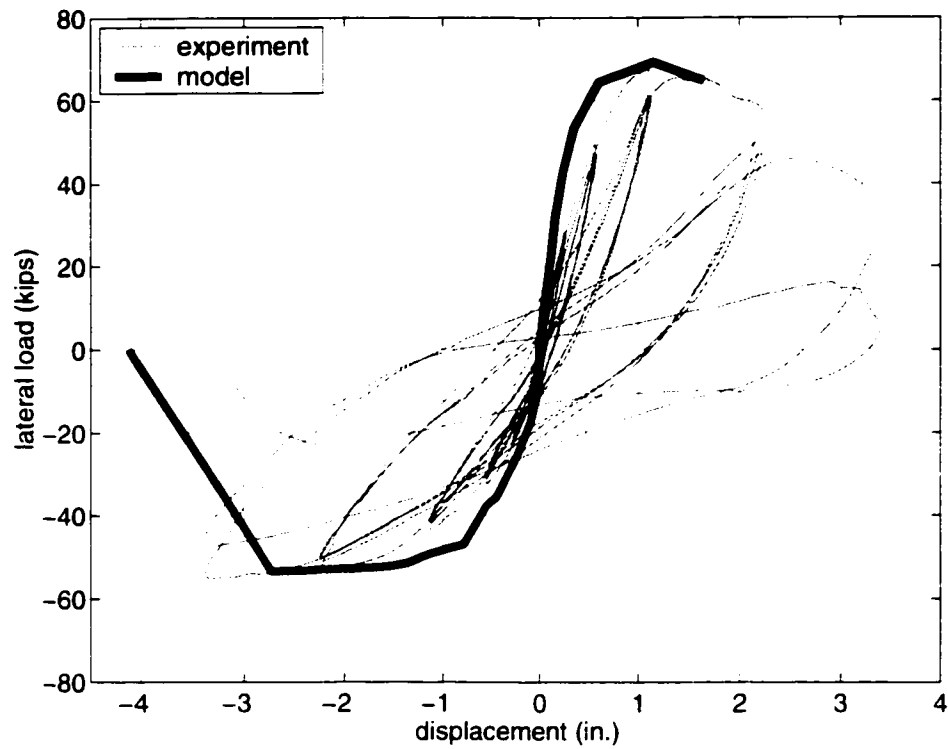


Figure 7.23 Lateral load-displacement relations with monotonic model (Specimen-3)

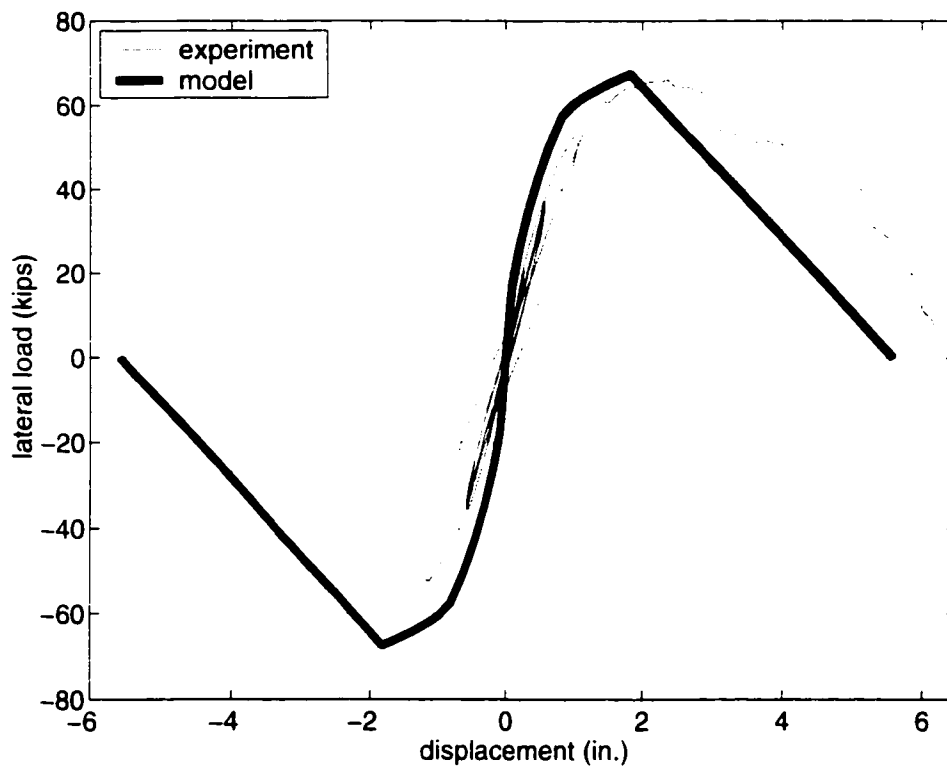


Figure 7.24 Lateral load-displacement relations with monotonic model (Specimen-4)

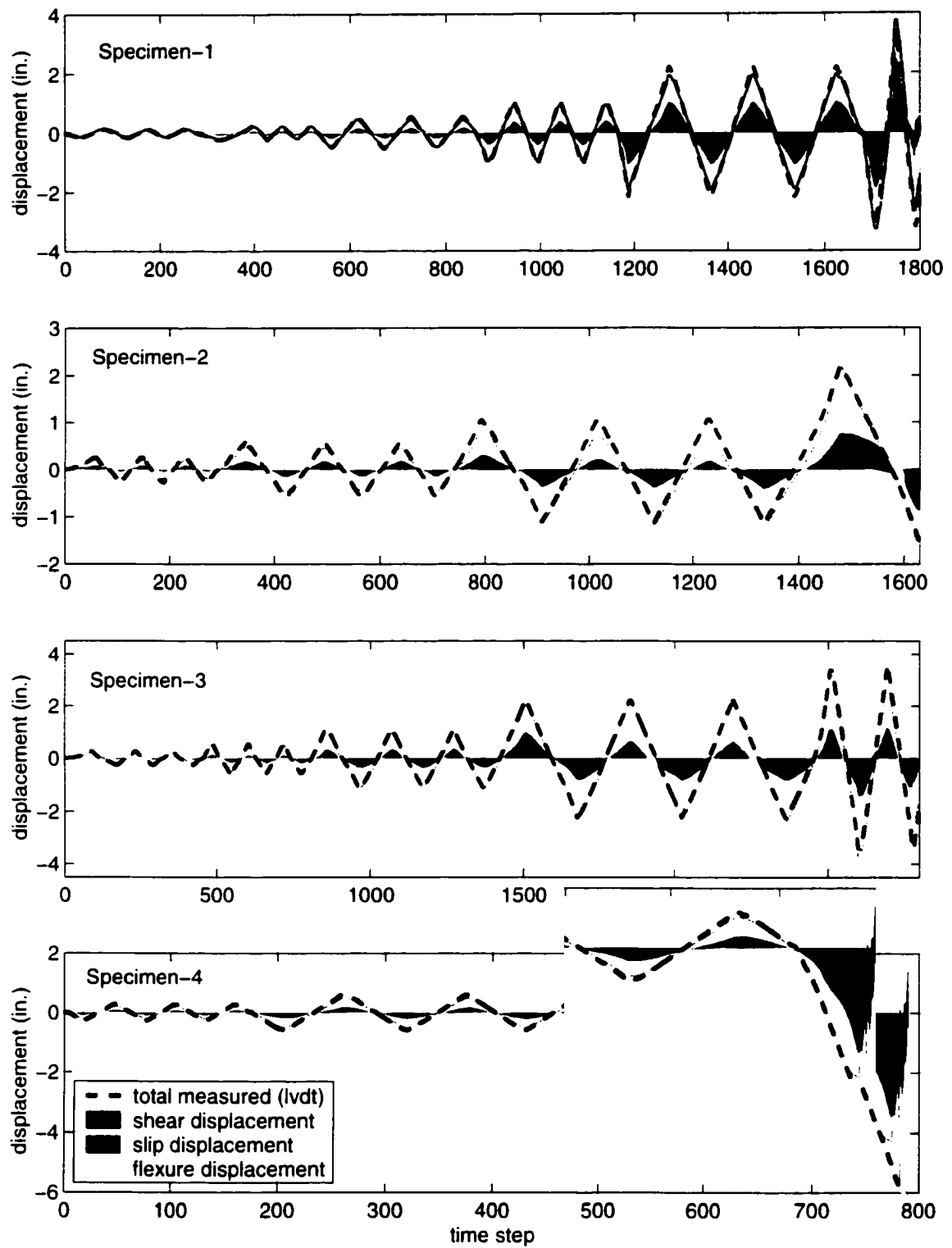


Figure 7.25 Calculated shear, slip and flexure, and measured displacement time histories

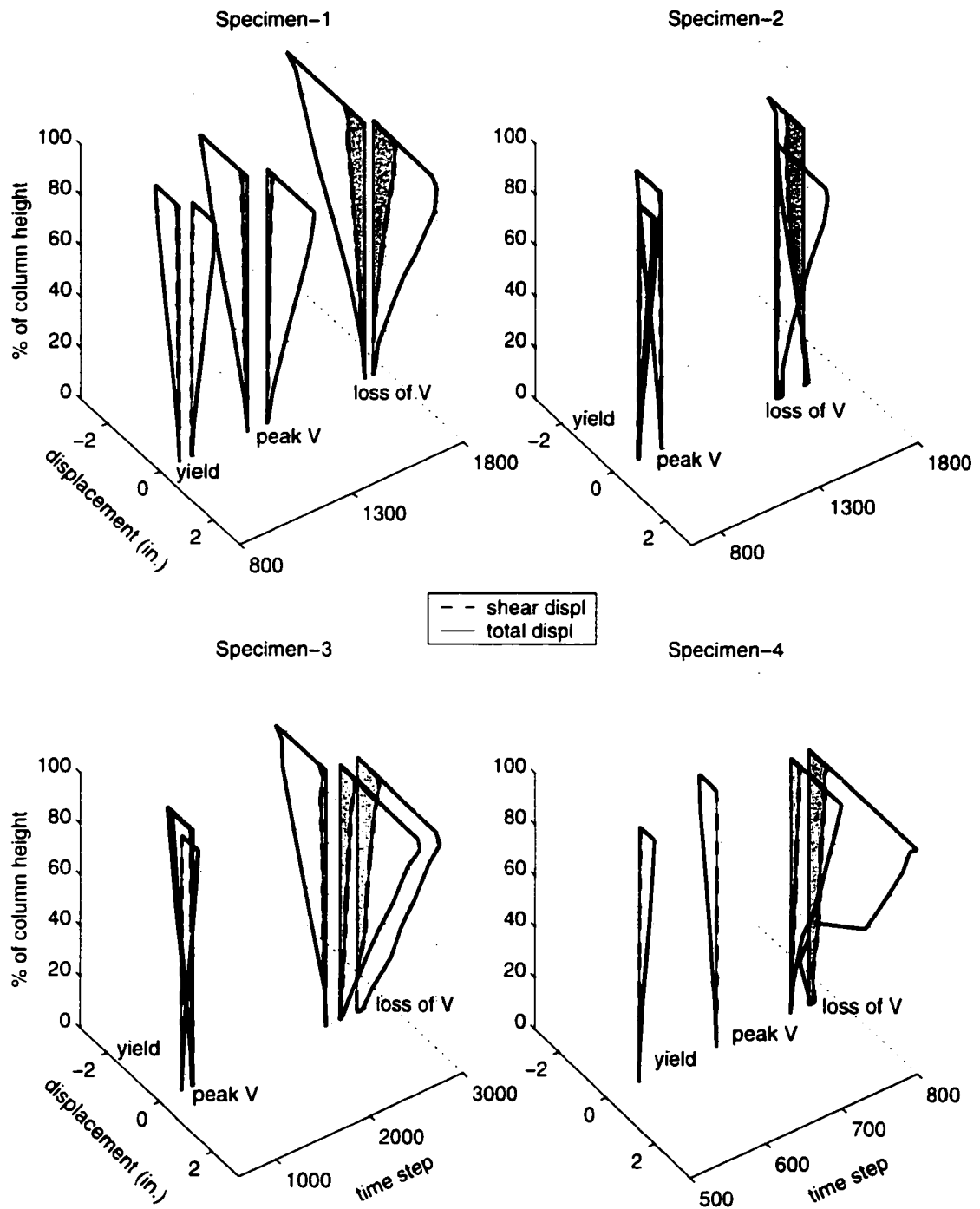


Figure 7.26 Variation of total and shear displacement distribution over the height of specimens

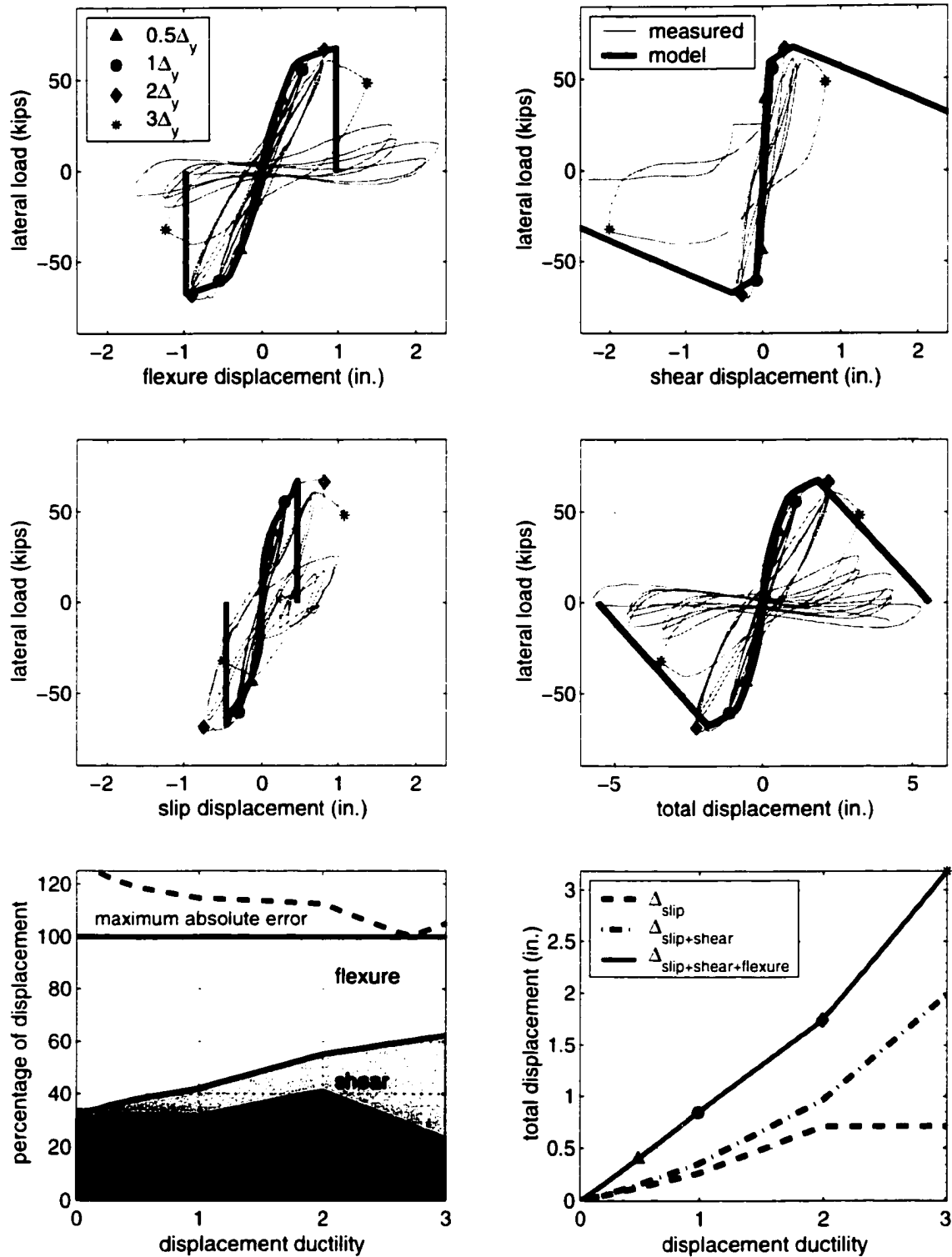


Figure 7.27 Proposed monotonic spring models, recorded flexure, shear and slip displacement components and their contribution to total displacement (Specimen-1)

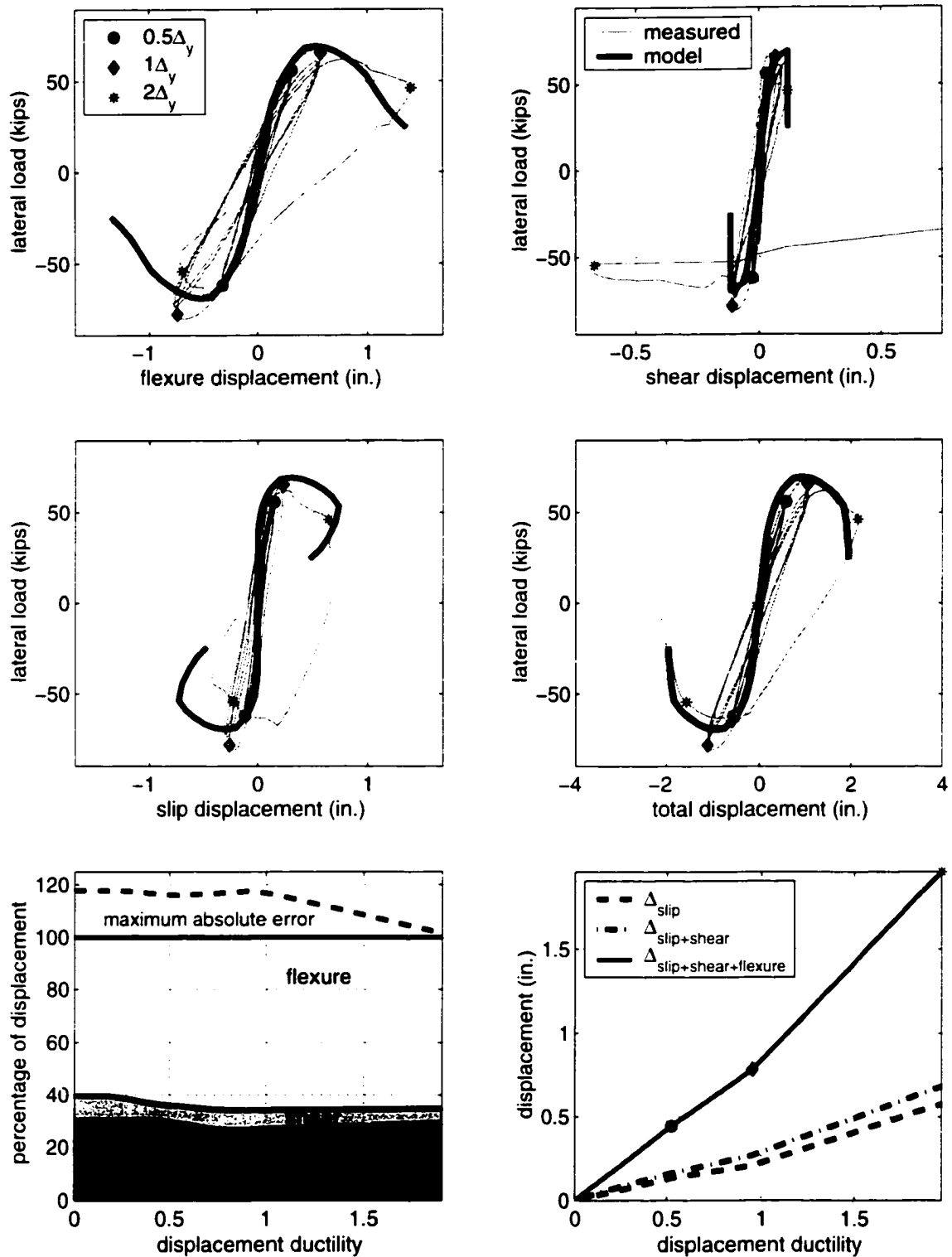


Figure 7.28 Proposed monotonic spring models, recorded flexure, shear and slip displacement components and their contribution to total displacement (Specimen-2)

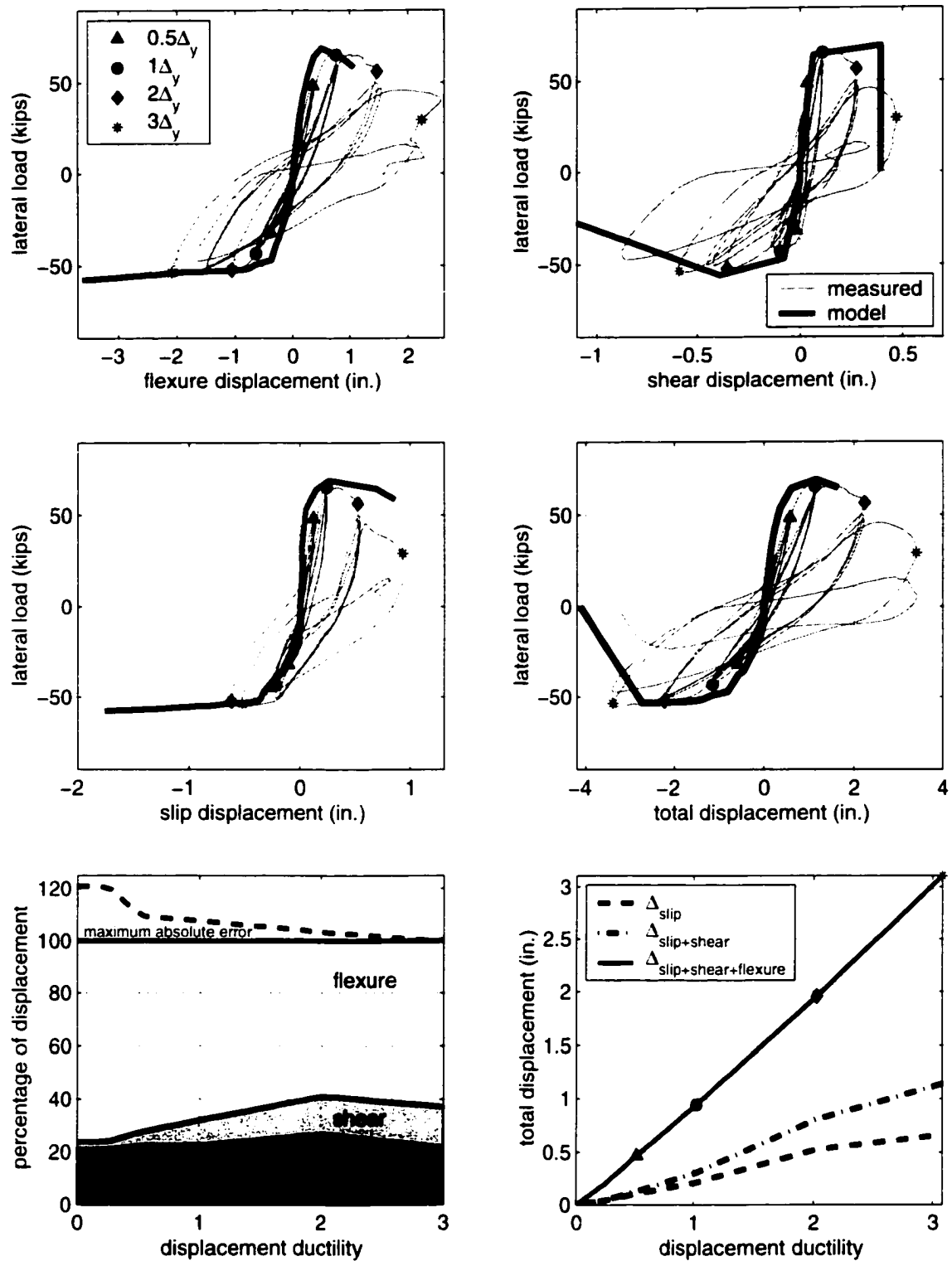


Figure 7.29 Proposed monotonic spring models, recorded flexure, shear and slip displacement components and their contribution to total displacement (Specimen-3)

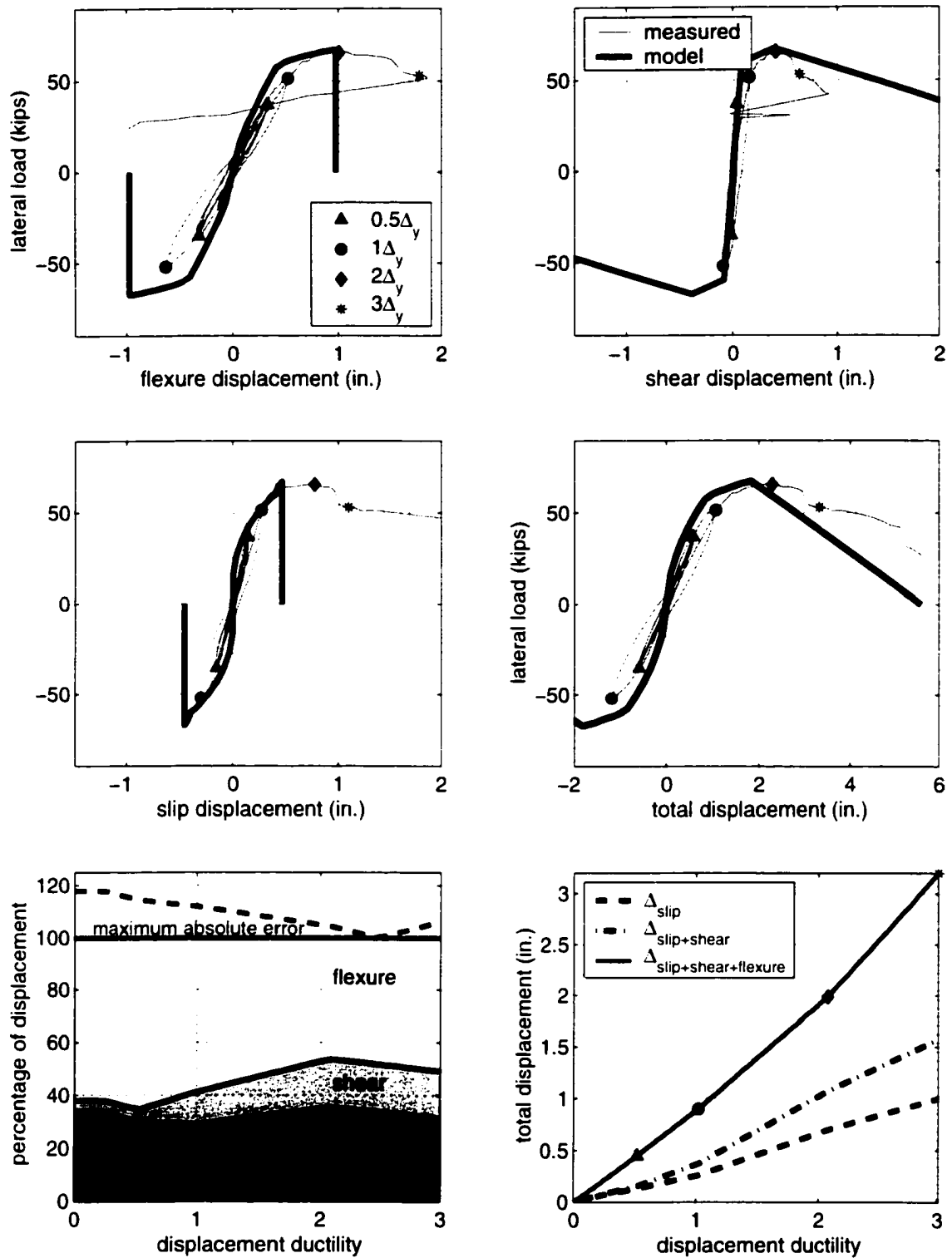


Figure 7.30 Proposed monotonic spring models, recorded flexure, shear and slip displacement components and their contribution to total displacement (Specimen-4)

8 SUMMARY, CONCLUSIONS AND FUTURE WORK

8.1 SUMMARY

As part of the experimental investigation, four full-scale reinforced concrete building columns with nominally identical properties and details were tested statically under axial load and uni-directional lateral load. The principal variables of the testing program were history and magnitude of the axial load, and lateral displacement history. The behavior of specimens were observed visually and monitored through various load, displacement, and strain measurements. Based on these observations and measurements, a detailed description of damage progression and the implications on the behavior of test columns are presented.

The lateral displacement components due to flexure, longitudinal bar slip, and shear were obtained using the local displacements measured by the displacement potentiometers installed on the columns. In the analytical part of the investigation, each component behavior was calculated under monotonic lateral load and compared with the test results. The moment-flexural displacement relations were calculated from the moment-curvature analysis using a fiber cross-section and uniaxial material models. An analytical model was proposed to predict the moment-rotation relations at the top and bottom beam-column interfaces due to longitudinal bar slip under monotonic lateral load. Similarly, a piecewise linear shear model is proposed to represent the monotonic lateral load-shear displacement response. A spring model with three springs in series is proposed to combine the flexure, bar slip, and shear response components. The model was intended to predict the monotonic behavior of lightly reinforced columns with significant stiffness and strength degradation after the flexural strength is attained.

A brief summary of experimental investigations, and selected experimental data from lightly reinforced rectangular columns were presented. The shear strength of these columns is investigated using available strength models. An alternative shear strength model relating the column shear strength to displacement ductility demand and column aspect ratio is proposed. The alternative model is evaluated and compared with other strength models using the experimental data from selected test columns.

8.1.1 Observed Behavior of Test Specimens

- At low amplitude displacement cycles, the damage was concentrated mostly near the column supports in the form of light/thin inclined (flexural) cracks, and the damage pattern was very similar in all specimens.
- At yield displacement level, typically, the longitudinal reinforcing bars yielded, and the inclined (shear) cracks started to appear mostly within the upper and lower one third of the column height.
- During the displacement cycles to two times yield displacement, the specimen under very high axial load failed in a brittle manner. The final failure of this specimen (Specimen-2) was related to formation of large shear and bond cracks within the upper half of the column. At this displacement level, large shear cracks and spalling of cover concrete were observed in other specimens.
- During the displacement cycles to three times yield displacement, as a result of widespread flexure and shear cracks, all test specimens lost their lateral-load-carrying capacity significantly. The specimen subjected to varying axial load failed because of extensive flexural, shear, and bond cracks, opening of column tie hooks, longitudinal bar buckling, and core concrete crushing under high compressive axial load.
- Even though the specimens with low axial load suffered substantial damage and lost their lateral-load-carrying capacity significantly, they were able to sustain their axial load at larger displacements.

8.1.2 Evaluation of Experimental Data

The flexural curvatures were calculated along the height of the columns using both the strains measured on the longitudinal reinforcement and the local vertical displacements recorded by the displacement potentiometers. Displacement measurements were found to be more reliable and, therefore, used in the flexural displacement calculations.

The column end rotations due to longitudinal bar slip were obtained by assuming a linear flexural curvature distribution near the column ends and by subtracting the rotations due to flexure from the total measured rotations. The curvatures calculated from the measured longitudinal bar strains indicated that this assumption could introduce some error at very large inelastic deformations. This error would decrease the end rotation due to flexure. However, when the flexure and bar slip displacement components are added, the error would be eliminated in the total member deformations.

The shear displacements are calculated using the principle of virtual work with the local member displacements recorded by the displacement potentiometers installed on the column. The recorded total lateral displacements compared reasonably well with the summation of individual displacement components from flexure, bar slip, and shear.

8.1.3 Analytical Models

The flexural displacements were obtained by integrating the curvatures calculated from a fiber section computer model with uniaxial steel and concrete material models. A simple spring model is used to idealize the monotonic flexural response.

The moment-rotation relations due to bar slip at column ends are modeled using a bi-uniform bond stress-slip relationship for the longitudinal bars anchored inside the top and bottom beams. The monotonic moment-slip rotation relations for the proposed bar slip model are calculated using the neutral axis depth and longitudinal bar strains obtained from the section moment-curvature analysis.

A piecewise linear model is proposed to capture the lateral load-shear displacement relations. Shear displacements and the corresponding lateral loads at cracking, yielding, peak lateral strength, and at loss of axial-load-carrying capacity are calculated based on experimental observations and theoretical formulations.

A combined model with three springs in series is proposed to calculate the total monotonic lateral response. Assuming that the flexure, bar slip, and shear springs have the same force, the total lateral displacement is calculated by summing up the individual spring displacements at that force level. Once strength degradation starts after the peak lateral load is reached, the behavior is predominantly affected by either flexure or shear component. If the final column failure is mainly influenced by shear, then the flexure and bar slip springs are locked at the peak lateral load and post-peak deformations include primarily shear deformations. If flexure is the main failure mode, which was more likely to be the case under very high axial loads in this research, then the post-peak behavior is dominated by flexure and bar slip deformations while the shear spring remains locked in the post-peak range.

An alternative shear strength model is proposed and evaluated using the experimental data from 52 test columns. In the proposed model, both concrete and transverse steel contributions to the shear strength are related to displacement ductility demand. Based on a statistical evaluation, a strength reduction factor is proposed for design and assessment of columns. The proposed model is also compared with other shear strength models.

8.2 CONCLUSIONS

The full-scale lightly reinforced column specimens tested in this investigation lost their lateral strength significantly at low displacement ductility. Under low axial load, the specimens were able to sustain the axial load after the lateral strength was lost at large displacements. Under the same flexural demand and very high axial load, the lateral stiffness and strength increased at low displacements, however the specimen had a sudden shear and axial load failure. It was concluded that such a failure could lead to sudden changes in member deformations and forces in neighboring frame components with possible building collapse without warning.

The measured cyclic lateral load-flexural displacement relations for each specimen compared very well with the monotonic flexural response calculated from the moment-curvature analysis using a fiber section model. The proposed spring model for longitudinal bar slip deformations predicted the monotonic lateral load-bar slip displacement relations reasonably well.

The total lateral displacements measured at the top of the test columns compared well with the total displacements calculated using the principal of virtual work and local displacement measurements. Drifts and shear displacements calculated using the principal of virtual work provided valuable information in understanding the shear behavior of test columns. The proposed piecewise linear shear spring model provided a reasonably good response envelope for the measured cyclic lateral load-shear displacement relations.

The agreement between the total monotonic lateral response from the proposed model with three springs in series and the experimental lateral load-displacement relations were reasonable.

An alternative shear strength model is developed and evaluated using experimental data from 52 test columns with shear failures, and it is compared with two other strength models. Theoretical formulations involving stress transformations, experimental results and statistical evaluation of test data indicated that the concrete contribution to column shear strength is a function of concrete strength, axial load, cross-sectional area, and column aspect ratio. It was also concluded that the concrete contribution decreases with increasing displacement ductility demand.

Similarly, the experimental investigation indicated that the contribution of widely spaced and poorly detailed transverse reinforcement to the column shear strength decreases significantly at large displacements. This is mainly because the effective section depth is reduced after the development of bond cracks between the longitudinal bars and

concrete, and 90-degree hooks at the end of column ties tend to loosen or open up especially after the cover concrete spalls off and shear cracks develop.

The proposed model is found to be reasonably accurate in predicting the shear strength of columns with aspect ratios varying between two and four. The proposed model provided an improved strength estimate as compared with other models.

8.3 FUTURE WORK

As the number of experimental investigations on the behavior of full-scale lightly reinforced columns is very limited, the effect of various parameters on the column behavior is not understood very well. In addition to variations in the magnitude and axial load, and lateral displacement histories, the effects of bi-directional lateral loading on such columns remain to be studied. Even though the behavior of older beam-column joints and column members have been studied separately, it would be interesting to experimentally investigate the behavior of assemblages/frames including both poorly detailed beam-column joints and double curvature columns with lap splices.

Currently, available computer programs cannot directly incorporate hysteretic behavior of column members with significant stiffness and strength degradation due to shear failure after the flexural strength is attained. Development of such computer models involves several complicated issues including the coupling and interaction between the flexural, shear, and axial components. These analytical models need to be verified using the experimental data and be sufficiently accurate, yet simple and easy to modify to implement in an open computer analysis platform. Currently, one such platform,

OpenSees is being developed at the University of California, Berkeley. Among other modeling problems, the following major analytical issues remain to be investigated.

8.3.1 Axial Load

Experimental research indicates that the magnitude and nature of the axial load affect the behavior of column members significantly. Very few researchers attempted to investigate the effect of very high compressive axial loads and tensile loads. Before considering these extreme-loading conditions, the effect of typical constant gravity loads and varying axial loads on the behavior of column members needs to be incorporated into the computer models. In this investigation, the effect of axial load on the flexure and bar slip response is included through the fiber section moment-curvature analysis. The effect of axial load on the shear behavior was also considered by including the axial load term in the calculation of shear displacement at yielding and maximum shear strength. However, the interaction between the axial, flexure, and shear components, especially after shear failure, has not been studied comprehensively. One such investigation is being carried out by Elwood (2002).

8.3.2 Hysteretic Behavior Modeling and Component Interaction

Analytical part of this investigation did not involve modeling of hysteretic flexure, bar slip, and shear behavior. Thus, the modeling issues such as stiffness and strength degradation, pinching of the loops, and energy dissipating characteristics of the hysteretic behavior were not investigated analytically. As discussed in Chapter 5, even though a number of researchers investigated and proposed hysteretic models for flexure,

bar slip, shear, and axial behavior, very few researchers incorporated the interaction and coupling of some of these models. Typically, these studies involved an ad hoc approach to include the interaction between two components and the other components were usually ignored. Each hysteretic component behavior needs to be studied analytically and modeled considering the interaction between them.

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APPENDIX A: STRENGTH AND DEFORMABILITY OF SELECTED TEST COLUMNS

During the last few decades, a number of researchers carried out experiments to investigate the seismic behavior of building columns with limited displacement ductility. The test parameters included column height, cross-sectional dimensions, material properties, details and amount of longitudinal and transverse reinforcement, axial and lateral load magnitude and history, test setup and boundary conditions for the test columns. This appendix provides a summary and brief description of the experiments conducted using test specimens representative of older rectangular building columns. The recorded lateral load-displacement responses of a number of column specimens tested previously are provided in this appendix. The test data presented here, together with the data generated in this study, enabled broader understanding of column shear behavior and validation of the shear strength model proposed in Chapter 6.

In the following discussions, the transverse reinforcement ratio, ρ_w is defined as the ratio of horizontal shear reinforcement area in the direction parallel to the shear force to gross concrete area of vertical section between the two column ties. Transverse reinforcement yield stress is denoted by f_{yw} . The shear span, a , is the distance between the externally applied lateral load point and the face of the support for members loaded in single curvature. For columns tested in double curvature, the shear span is equal to one half the clear height of the column.

In the selection of test columns presented in the next section, the following criteria were used:

Shear span to depth ratio: $2 \leq a/d \leq 4$

Concrete strength: $2500 \leq f'_c \leq 6500$ psi

Reinforcement nominal yield stress: $40 \leq f_y \leq 80$ ksi

Longitudinal reinforcement ratio: $0.01 \leq \rho_l \leq 0.08$

Transverse reinforcement: $0.01 f'_c \leq \rho_w f_{yw} \leq 0.12 f'_c$

In addition, only tests involving cyclic lateral load reversals and displaying considerable shear distress at failure were included.

Selected Column Tests

Two of the eleven double-curvature columns tested by *Ohue et al. (1985)* had details similar to those considered in this study. These specimens were subjected to constant axial load and static cyclic lateral load (Figure A.1). The final failure mode and the lateral strength degradation (Figure A.2) were influenced significantly by splitting bond cracks developed along the height of specimens.

Esaki (1996) studied the effect of transverse reinforcement configuration on the ductility of double-curvature columns with square cross-section. Figure A.3 shows details of the test setup and specimen configuration. Rotation of the top beam was prevented by a mechanical system (a pantograph) and the axial and cyclic lateral loads were applied using a vertical and a horizontal actuator as shown in Figure A.3. The specimens were designed to fail in shear after flexural yielding at the critical section. Figure A.4 shows

the measured lateral response of the four test columns subjected to two different, constant axial loads. It was concluded that the rate of strength deterioration was influenced significantly by the magnitude of the applied axial load and the formation of bond splitting cracks.

Li et al. (1995) tested nine large-scale cantilever columns under proportionally varying axial load using the test setup shown in Figure A.5. In order to achieve shear-dominated failure, in three of the test columns the amount of transverse reinforcement was reduced considerably. The measured lateral load-displacement relations for these lightly reinforced columns are shown in Figure A.6. The axial load varied between $0.30 f'_c A_g$ in compression to $0.15 f'_c A_g$ in tension. As shown in Figure A.6, the lateral strength was reduced significantly under tensile or low compressive axial loads. Figure A.7 through Figure A.10 show the test setup and lateral response of large-scale cantilever columns tested by Saatcioglu and Ozcebe (1989), and Yalcin (1997).

The test setup and the recorded lateral load-displacement relations for the six columns tested by *Ikeda (1968)* at the National Yokohama University are shown in Figure A.11 and Figure A.12, respectively. A similar test setup (Figure A.13) was used by *Kokusho (1964)* at the Building Research Institute in Japan, and *Kokusho and Fukuhara (1965)* at the Tokyo Industrial University. The recorded lateral load-displacement plots from these tests are shown in Figure A.13. A large number of column specimens tested by *Umemura and Endo (1970)* at the Tokyo University fall into specified parameters defined in the previous section. Two different specimen configurations were used in these tests (Figure A.14). The recorded lateral load-displacement relations for the test specimens considered

in this study are shown in Figure A.15. Results from these nineteen similar tests carried out in Japan provided valuable information about the behavior of columns with shear failure.

Of the twelve specimens tested by *Wight and Sozen (1975)*, six specimens had properties falling in the range of interest for the present study. As shown in Figure A.16, unlike other test setups presented in Figure 2.1, the central joint was clamped by a pair of hydraulic actuators. Two cantilever columns on each side of the joint were simultaneously loaded in opposite directions. For the six columns considered, the recorded lateral load-column tip displacements are shown in Figure A.17. Based on test results, *Wight and Sozen* suggested that the shear capacity of the column be based on the shear capacity of the core concrete only, and that the stirrup spacing within hinging zone be small enough to ensure confinement of the core.

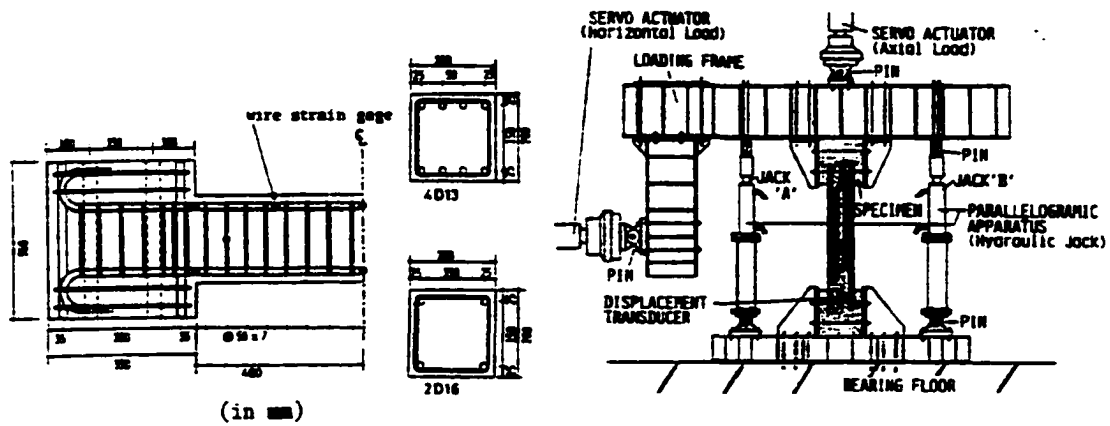


Figure A.1 Specimen details and test setup (Ohue et al. 1985)

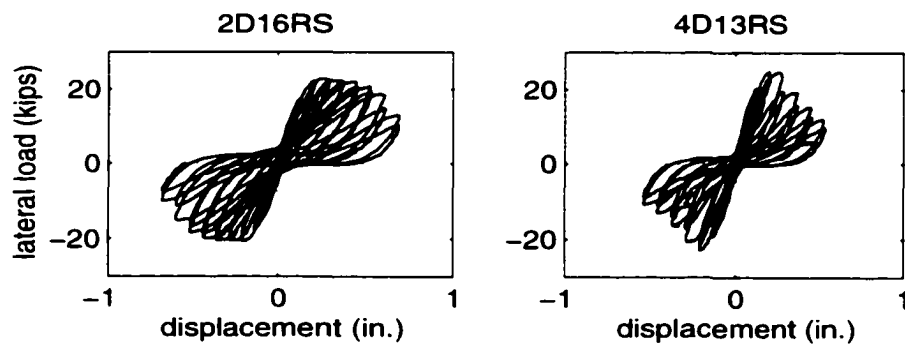


Figure A.2 Lateral load-displacement relations (Ohue et al. 1985)

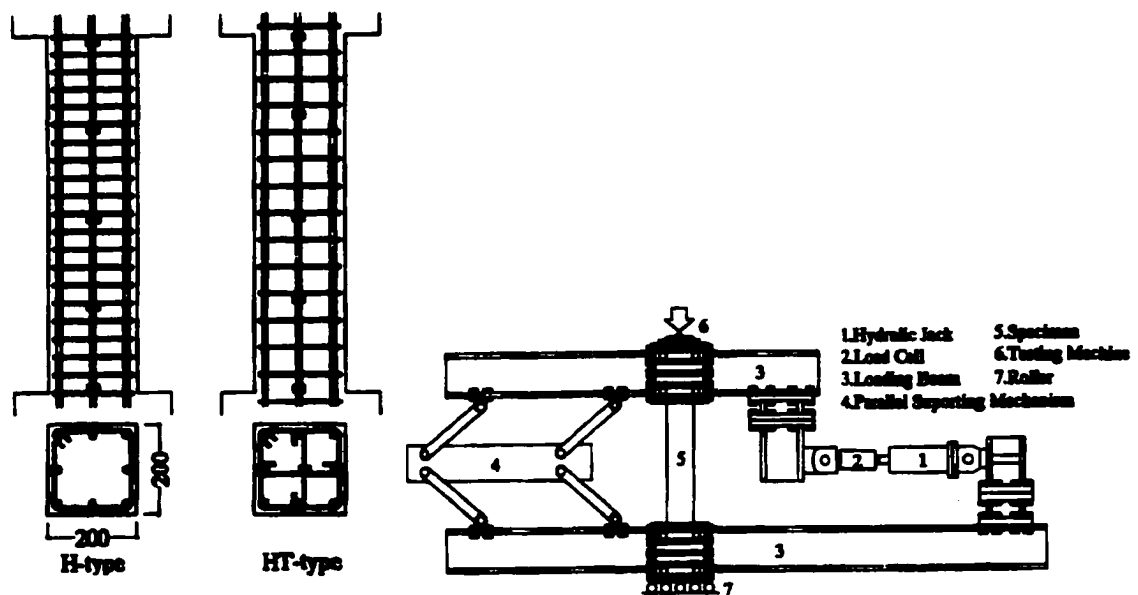


Figure A.3 Specimen details and test setup (Esaki 1996)

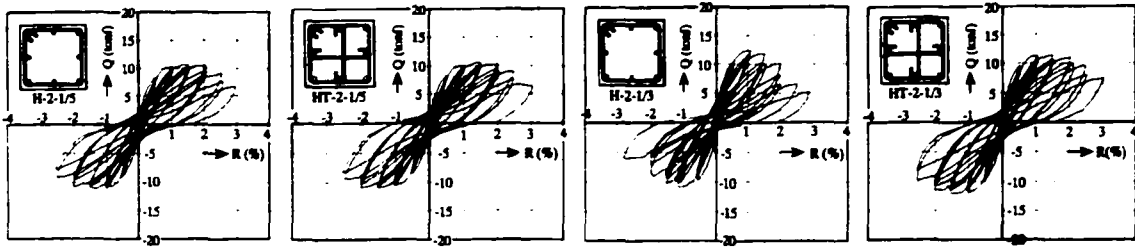


Figure A.4 Lateral load-displacement relations (Esaki 1996)

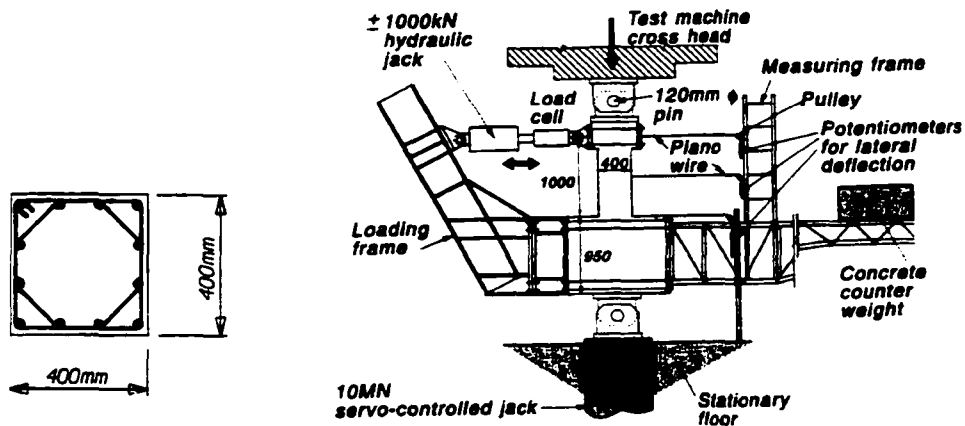


Figure A.5 Column detail and test setup (Li et al. 1995)

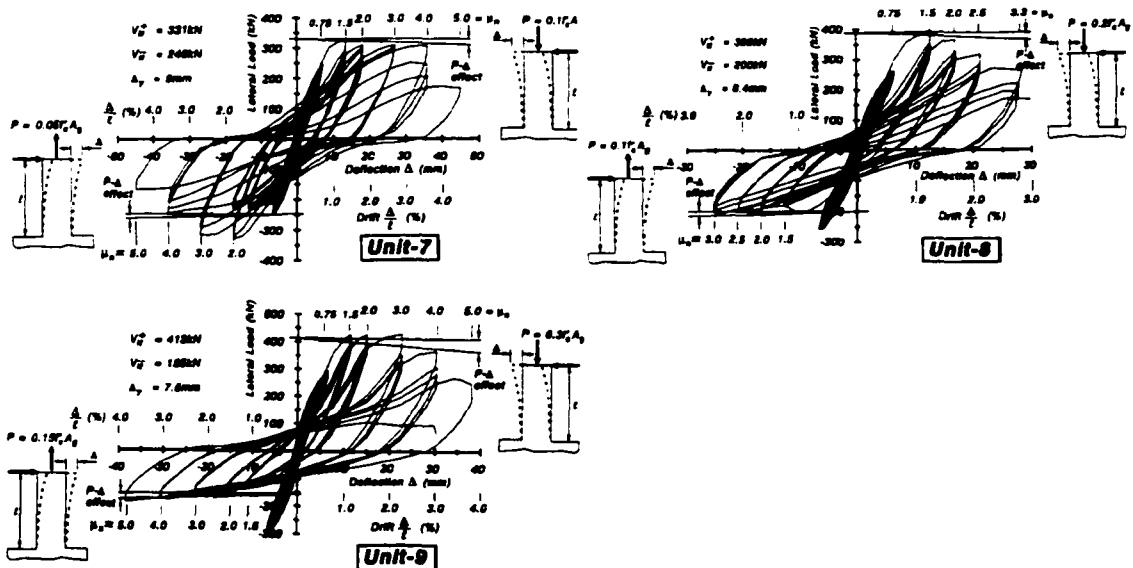


Figure A.6 Lateral load-displacement relations (Li et al. 1995)

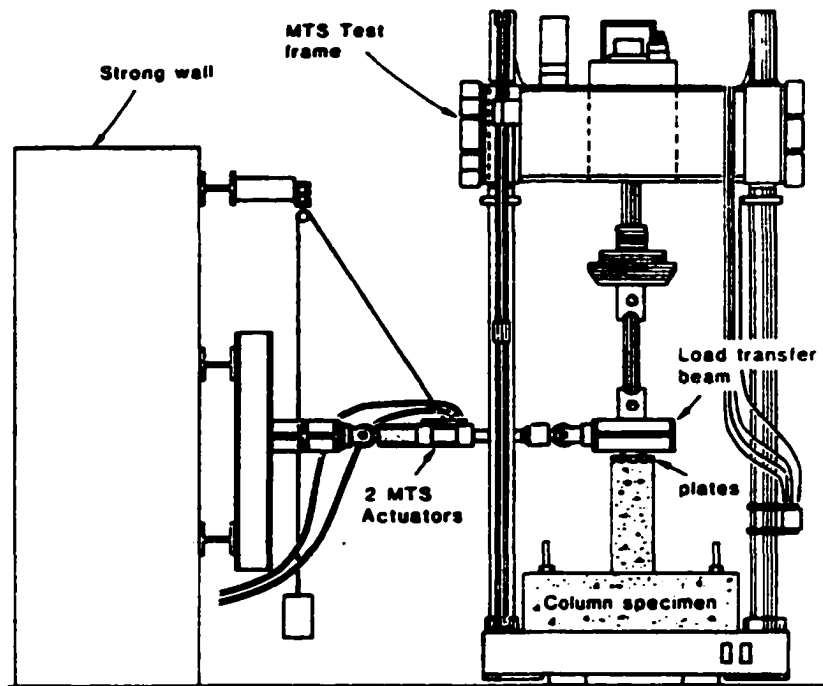


Figure A.7 Test setup (Saatcioglu and Ozcebe 1989)

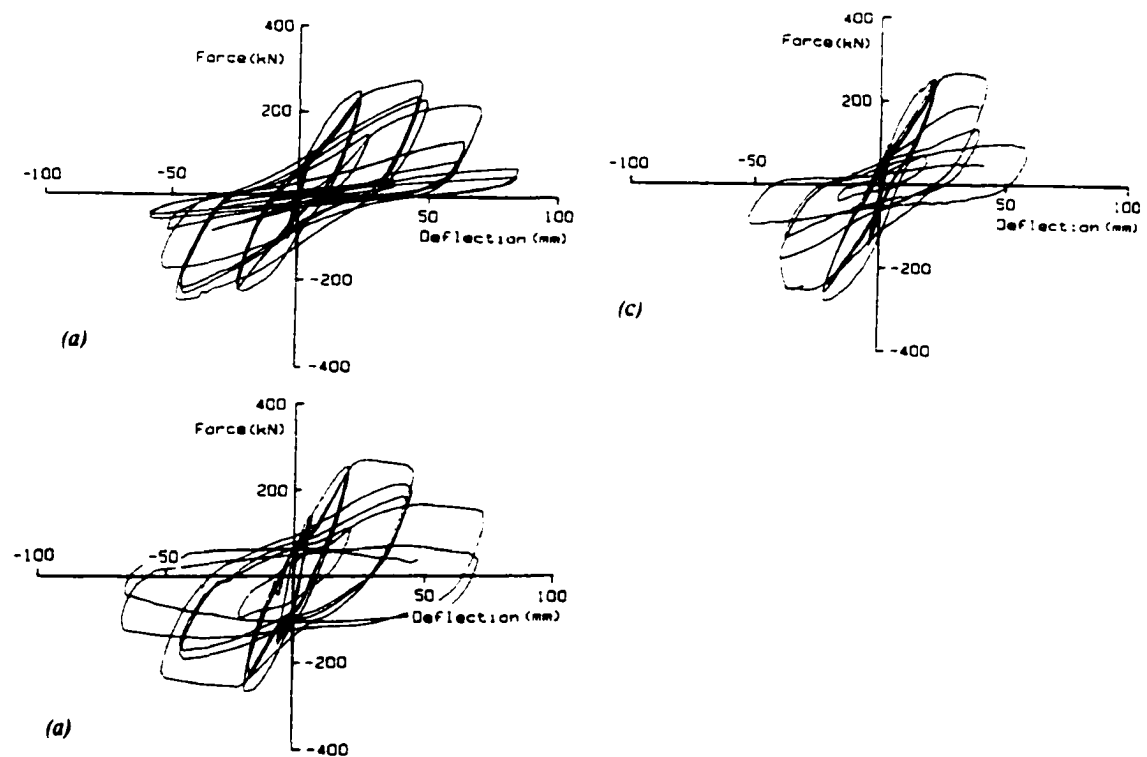


Figure A.8 Lateral load-displacement relations (Saatcioglu and Ozcebe 1989)

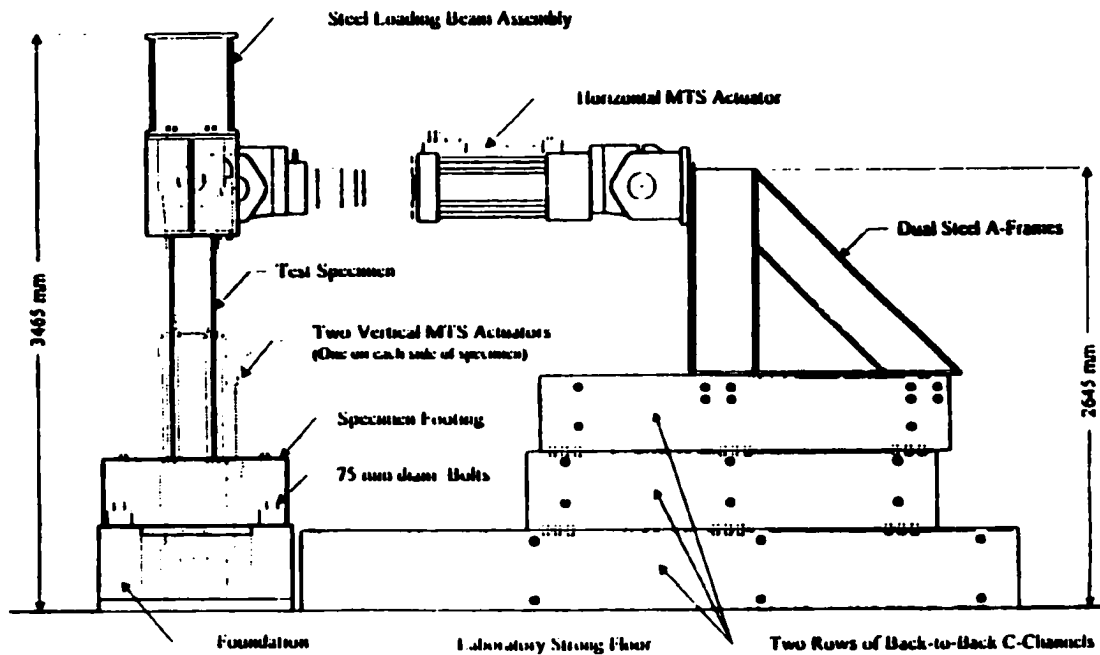


Figure A.9 Test setup (Yalcin 1997)

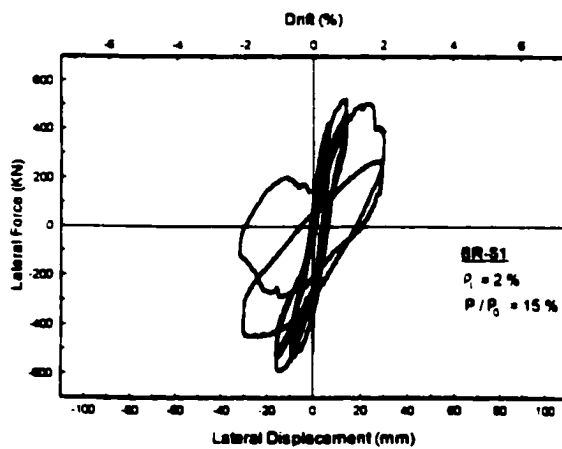


Figure A.10 Lateral load-displacement relations (Yalcin 1997)

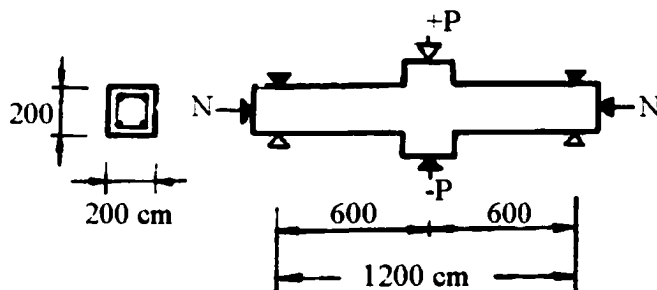


Figure A.11 Column detail and test setup (Ikeda 1968, Kokusho 1964)

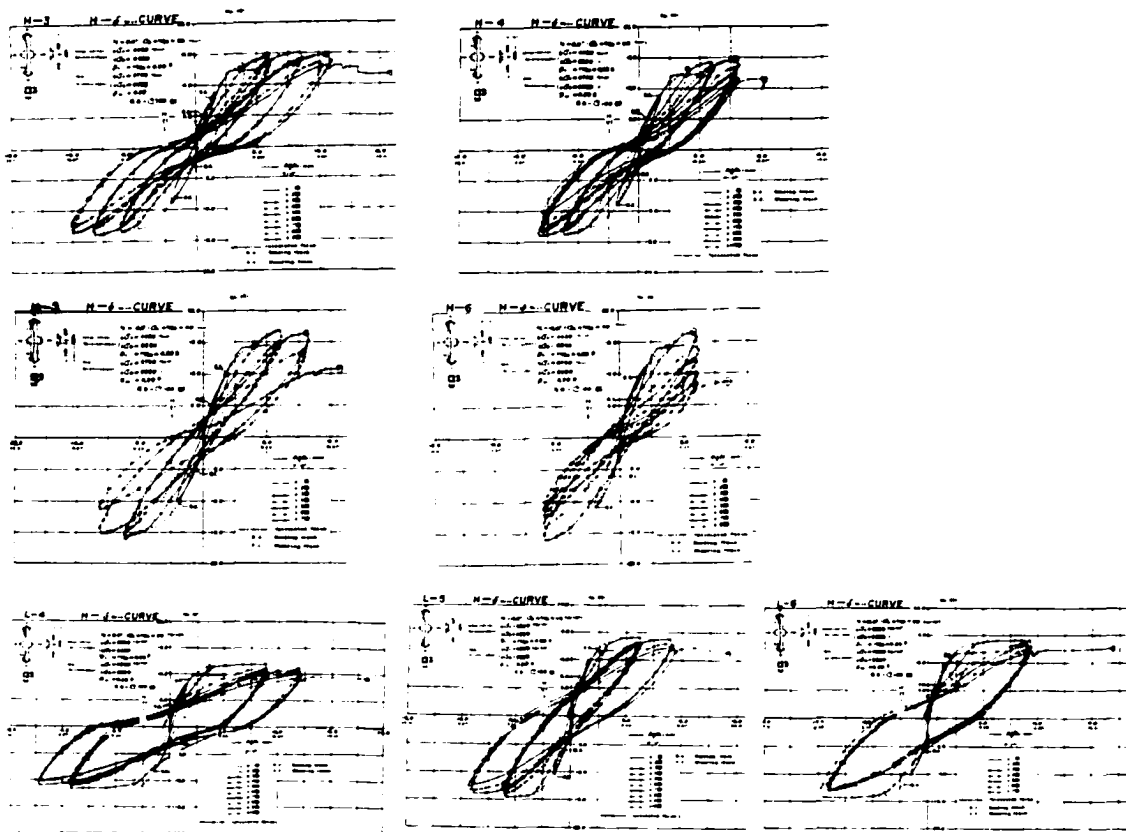


Figure A.12 Lateral load-displacement relations (Ikeda 1968)

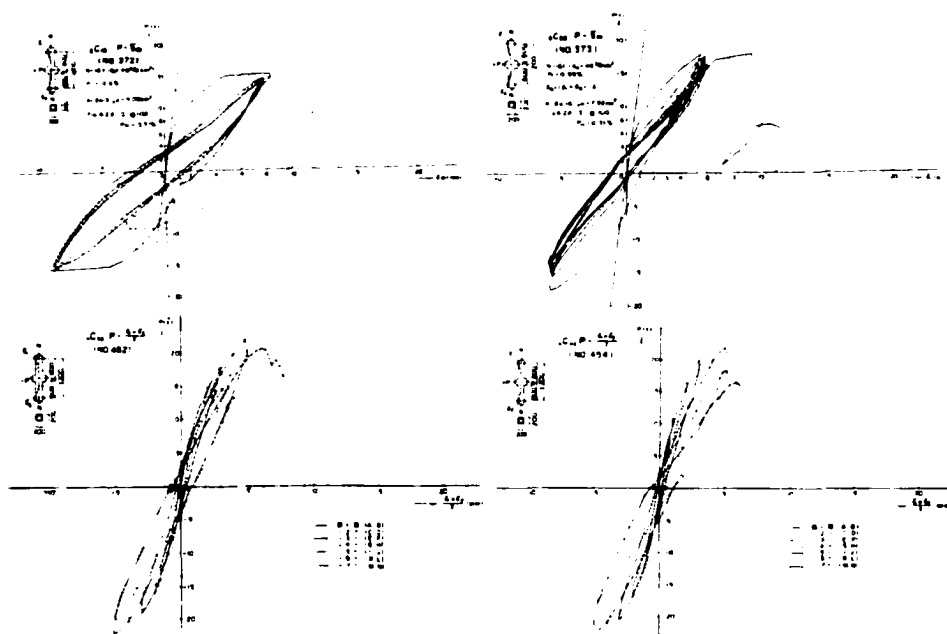


Figure A.13 Lateral load-displacement relations (Kokusho 1964, 1965)

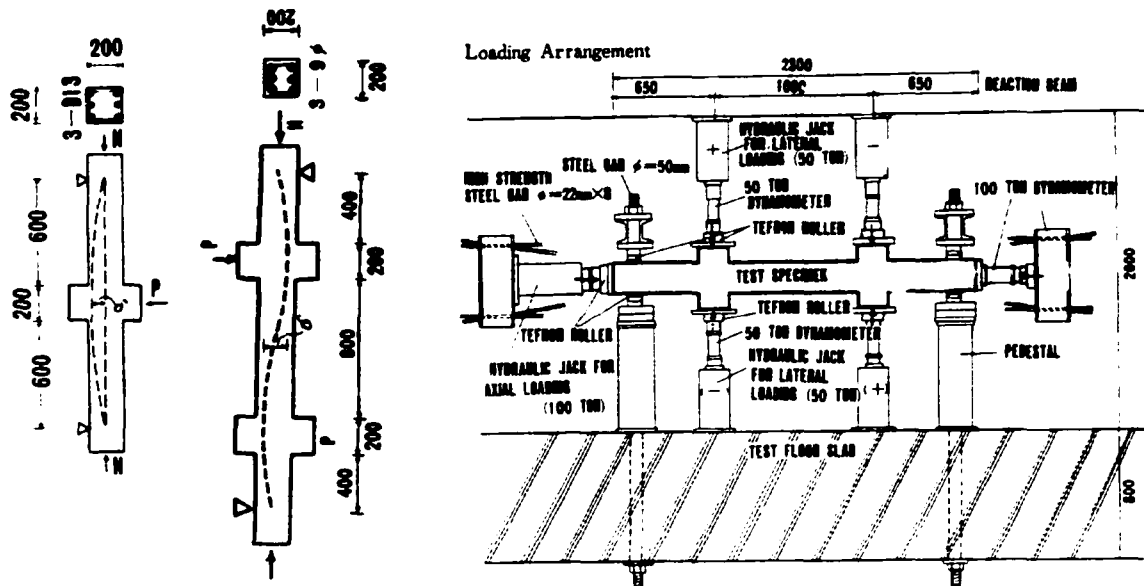


Figure A.14 Details of test columns and test setup (Umemura and Endo 1970)

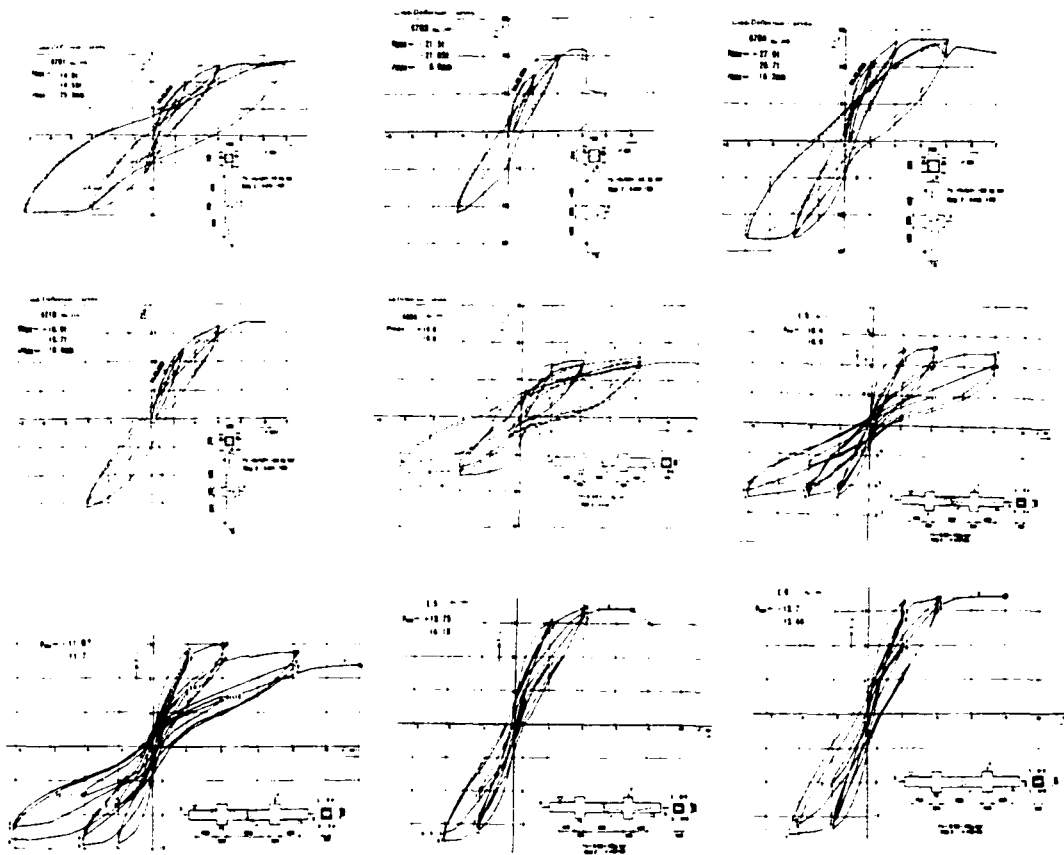


Figure A.15 Lateral load-displacement relations (Umemura and Endo 1970)

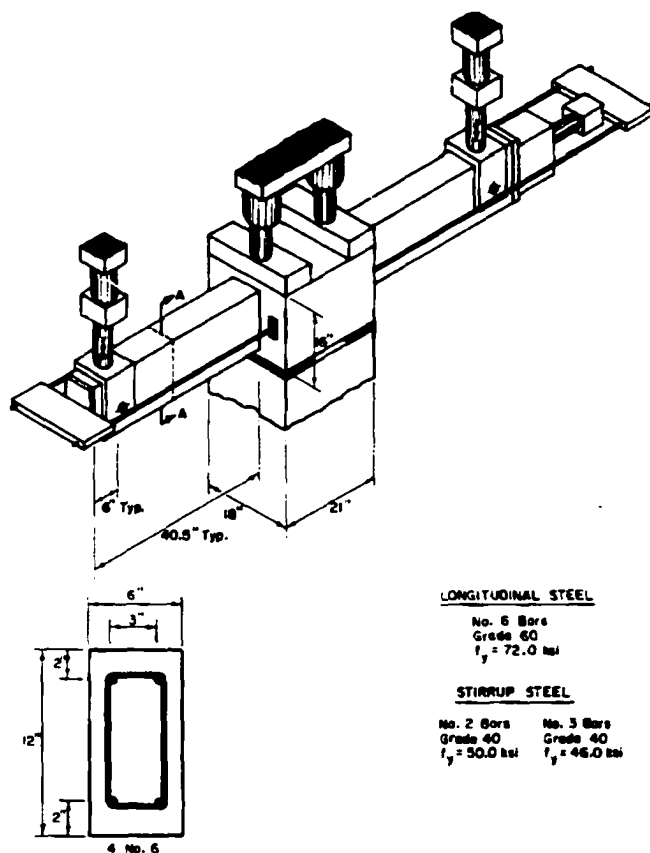


Figure A.16 Typical section detail and test setup (Wight and Sozen 1975)

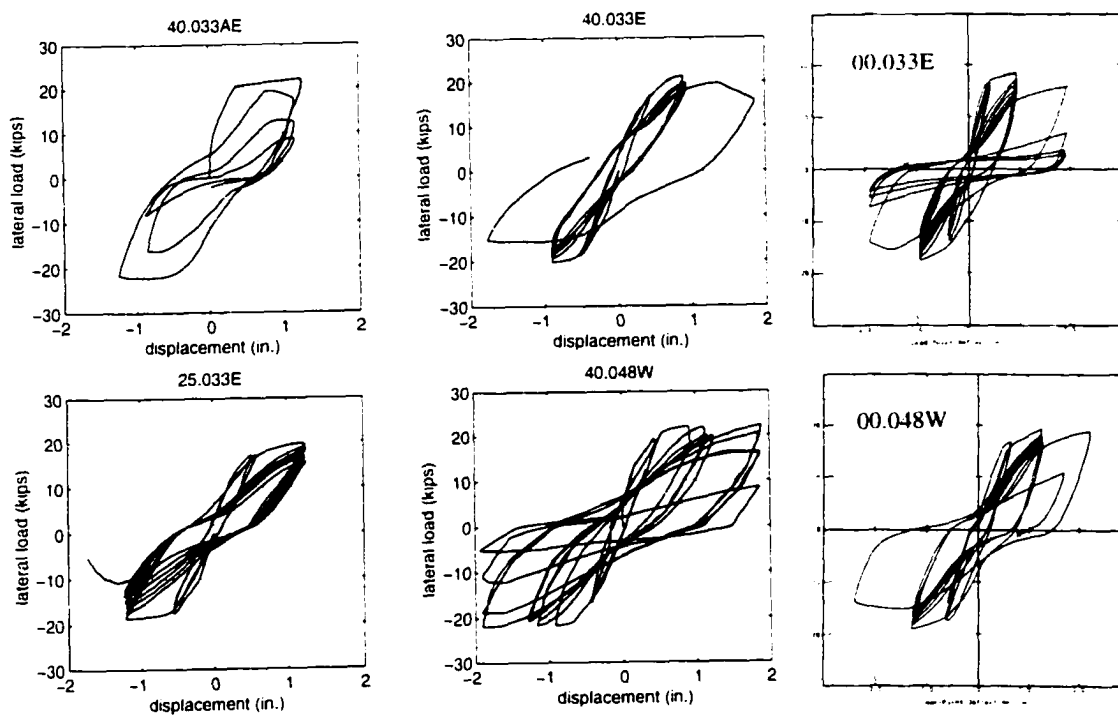


Figure A.17 Lateral load-displacement relations (Wight and Sozen 1975)

APPENDIX B: MATERIAL PROPERTIES

Concrete Properties

During concrete casting, 6-in. diameter by 12-in. high standard cylinders were cast in three equal layers. Following the ASTM C31- 88 requirements, each layer was rodded 25 times with a 5/8-inch diameter steel rod. Concrete cylinders were stored next to test specimens and subjected to nominally the same curing conditions as the actual test specimens. Cylinders were capped after casting and remained capped until the day of each cylinder test. To check the difference between capped and exposed concrete cylinder strengths, a few cylinders were stripped of their plastic covers seven days after casting when the forms were removed from the test specimens. As shown in Tables B.3 and B.5, the strength difference between the capped (not stripped) and exposed (stripped) cylinders was less than 15 percent.

Before the compression tests, concrete cylinders were capped using high-strength sulfur mortar and the mortar was allowed to harden before the testing. Compression testing of cylinders was done in accordance with the ASTM C39-93, *Standard Test Method for Compressive Strength of Cylinder Concrete Specimens*, where the specified rate of loading is 20 to 50 psi per second for 6-in. diameter cylinders. For the calculation of modulus of elasticity, in accordance with the ASTM C-469, the rate of loading applied was 35 psi/second.

Concrete strength and failure types (Figure B.1) observed during the cylinder tests are tabulated in Tables B.3 through B.5. Compressive stress versus strain relationship was

recorded during the cylinder tests. As an example, column concrete compressive stress-strain relationships for three cylinders tested on the day of third model test (222 days after casting) are shown in Figure B.2. A plot of column and beam concrete strength increases with age is shown in Figure B.3. To determine the tensile strength of concrete, splitting tension tests were performed only on the cylinders cast using column concrete on the days of the third and last tests. Results are shown in Table B.6.

Reinforcing Steel

Three or four steel coupons were tested in tension for each rebar size used. Coupons were prepared using approximately 23-inch long reinforcing bars. Approximately 6 in. at both ends and 3 in. in the middle were machined smooth. The diameter of the middle segment was reduced more in order to ensure steel yielding at this location. Coupons were tested in tension following the ASTM A370 Standard. Applied tension load rate was 20 ksi per minute. Measured tensile stress-strain relationships are plotted in Figure B.3. Measured yield stress, f_y , yield strain, ϵ_y , and ultimate stress, f_u , for each bar size are listed in Table B.7. Yield stress was defined as the average of lowest points on the yield plateau for each bar size. Yield strain is defined as the strain corresponding to first yielding, i.e., the first peak before yield plateau. Ultimate stress is defined as the maximum stress carried by the steel coupon.

Table B.1 Mix Specifications

Cement	ASTM C-150v Type II
Concrete sand and gravel	ASTM C-33
Water reducer	Pozzolith 322N, ASTM C-494 Type A
28-day design strength, f'_c	3000 psi
Cementitious material	4.29 sacks
Maximum aggregate size	1 inch
Slump	5 in. (+/- 1 in.)
Water/cement ratio	0.70

Table B.2 Mix Design and Quantities

Material	Specific gravity	Solid volume (cubic ft)	SSD weight
Cement Type II	3.15	2.08	409 lbs
Water	1.00	4.54	283 lbs
Pozzolith 322N	-	0.41	20.5 fl. oz
1" x #4 gravel	2.69	10.13	1700 lbs
Regular top sand	2.67	6.90	1150 lbs
SR blend sand	2.62	2.97	482 lbs
Total		27.00	4018 lbs

Table B.3 Column concrete tests

Age (days)	Strength (psi)	Mean (psi)	Std. dev.	Failure mode
7	1319, 1284, 1302, 1418	1331	60	C / C / C / C
14	1620, 1631, 1659	1636	20	C / C / C
21	1857, 1821, 1889	1856	34	C / C / C
28 (unstripped)	2030, 1967, 2066	2021	50	C / C / C
28 (stripped)	2055, 2137, 2115	2102	42	C / C-C / C
93	2823, 2978	2901	110	C-S / C
146 (unstripped)	3074, 2996, 3095	3055	52	C-C / C-C / C
146 (stripped)	3809, 3664, 3152	3542	346	C / C-C / C
166	2982, 3119, 3092	3064	73	C / C-C / S
222	3119, 3045, 2936	3033	93	C / C / C
244	3369, 3141, 2968	3159	201	C / C / C

C: Cone, S: Shear, C-C: Cone-Columnar, and C-S: Cone-Shear

Table B.4 Base beam concrete tests

Age (days)	Strength (psi)	Mean (psi)	Std. dev.	Failure mode
3	1139,1128,1196,1093,1061	1123	51	C / C / C / C / C
7	1694,1653,1719,1793,1861	1744	82	C-S / C / C / C /
15	2388,2324,2508,2430	2413	77	C-S / C / C / C
21	2667,2879,2692,2894	2783	120	C / C / C-C / C
28	3414,3196,3383,2833,3206	3206	231	C / C / C / C / C
59	3728,3541,3898	3722	179	C-S / C / S
252	3969,3902,3834	3902	67	C / S / C
275	3941,3845,4153	3979	158	C / C-S / C

Table B.5 Top beam concrete tests

Age (days)	Strength (psi)	Mean (psi)	Std. dev.	Failure mode
7	1574,1733,1610,1581,1638	1627	64	C / C / C / C / C
15	1768,1690,2023	1827	174	C / C / C
21	2104,2239,2214	2186	72	C / C / C
28(unstripped)	2359,2388,2448	2398	45	C / C / C
28(stripped)	2512,2328,2572	2471	127	C / C / C-S
48	2699,2692	2696	5	C / C-S
222	3328,3360,3530	3406	108	C / C / S
222(stripped)	3187,3223,3155	3188	34	C / C / C
244	3601,3219,3410	3410	191	C / C / C

C: Cone, S: Shear, C-C: Cone-Columnar, and C-S: Cone-Shear

Table B.6 Column concrete: splitting tension tests

Age (days)	Load (kips)	Mean (kips)	Std. deviation
222	32.7, 37.1, 32.7	34.2	2.54
244	39.4, 32.8	36.1	4.67

Table B.7 Reinforcement yield and ultimate strength

Steel bar	Yield stress, f_y (ksi)	Ultimate stress, f_u (ksi)	Yield strain, ϵ_y
#3	69	105	0.0026
#4	66	102	0.0029
#9	63	93.5	0.0023

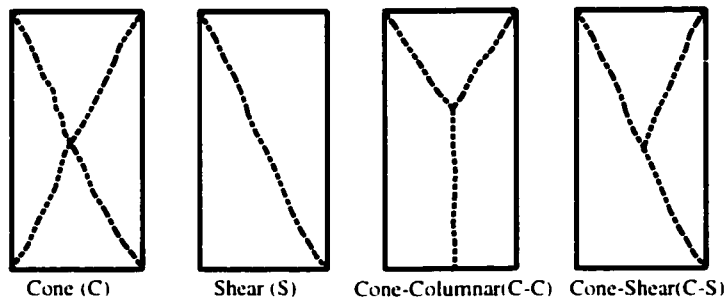


Figure B.1 Compression test failure modes for 6-in. diameter by 12-in. high concrete cylinders

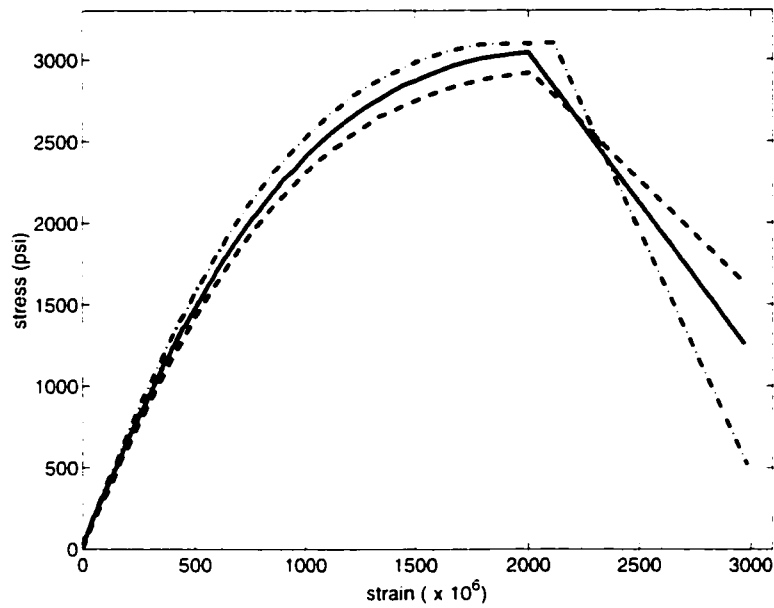


Figure B.2 Column concrete stress-strain relationships on the day of third test

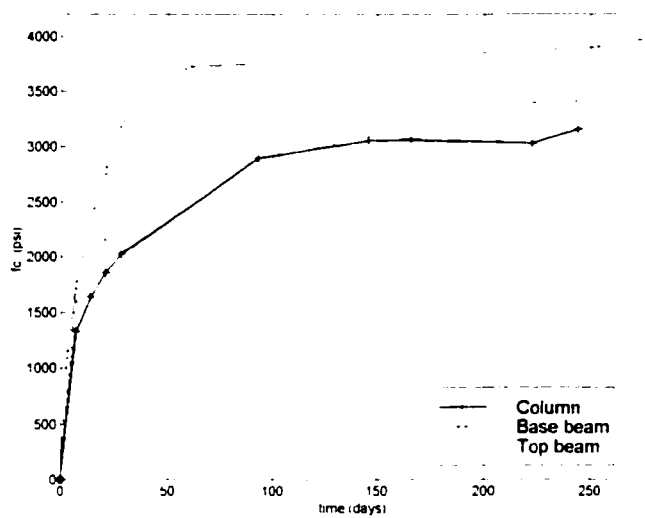


Figure B.3 Concrete compressive strength increase with time

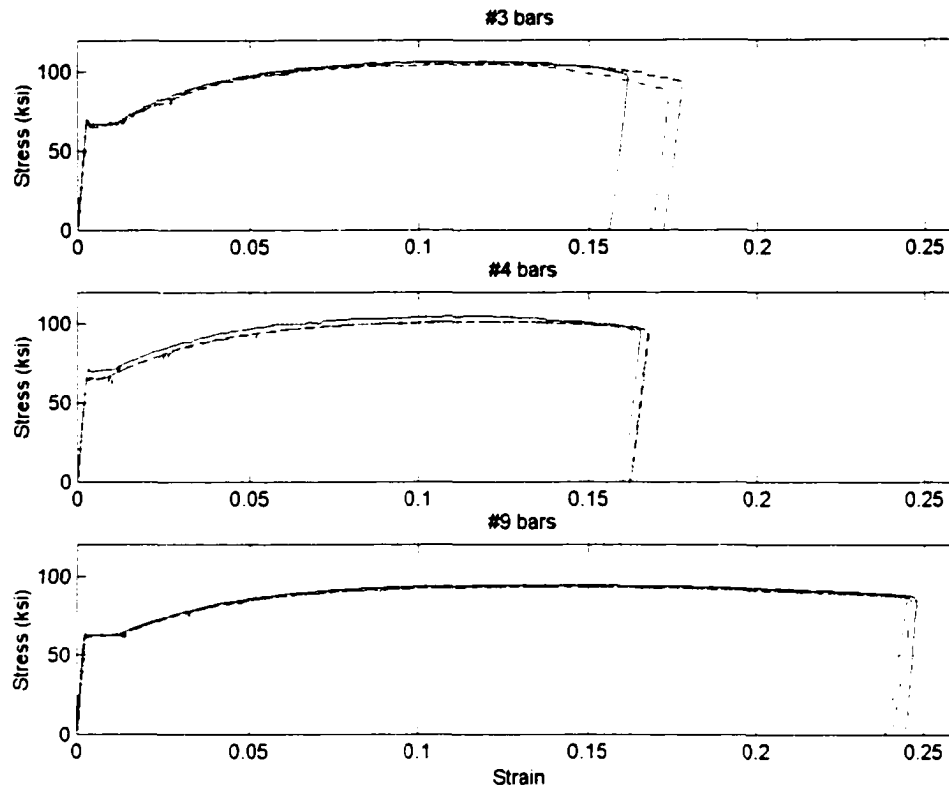


Figure B.4 Reinforcing steel stress-strain relationships

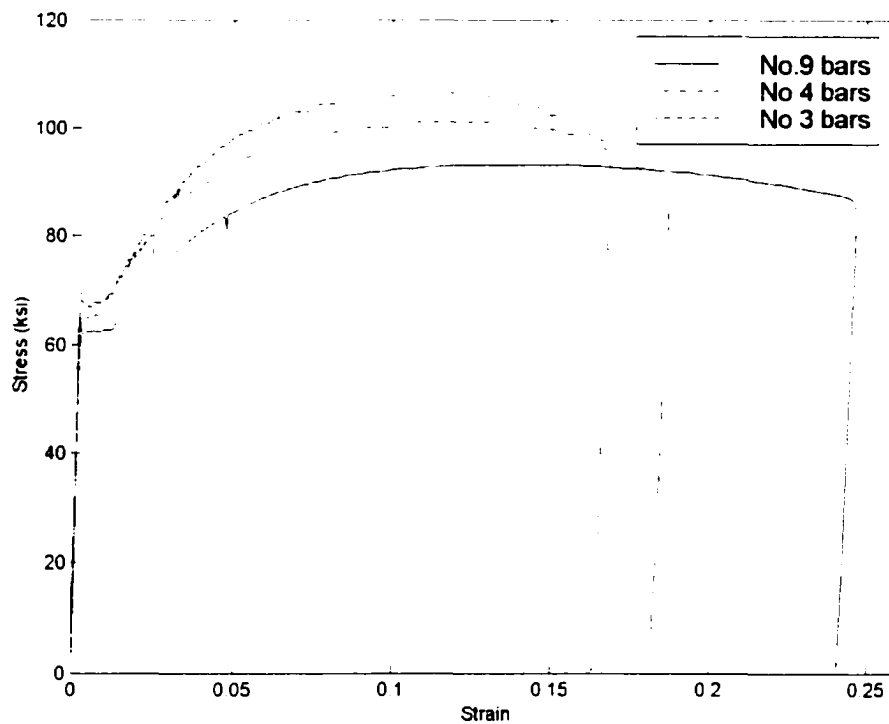


Figure B.5 Typical reinforcing steel stress-strain relationships

APPENDIX C: TEST SETUP

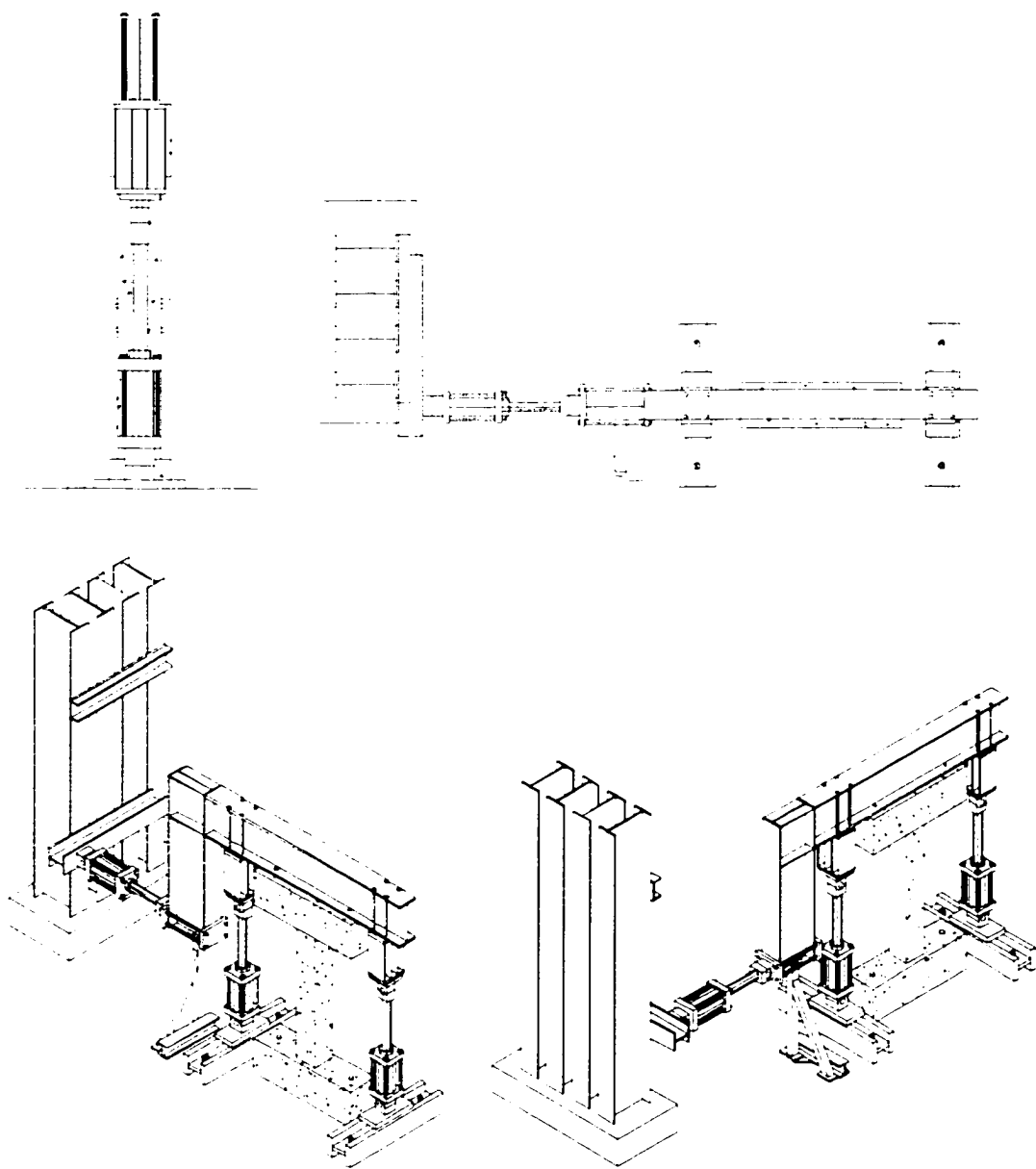


Figure C.1 Test setup; top: front (left) and top (right) view, bottom: perspectives

APPENDIX D: TEST CONTROL SYSTEM

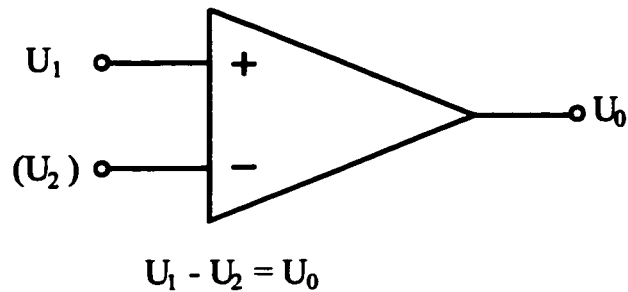


Figure D.1 Controller unit with input, U_1 and feedback, (U_2)

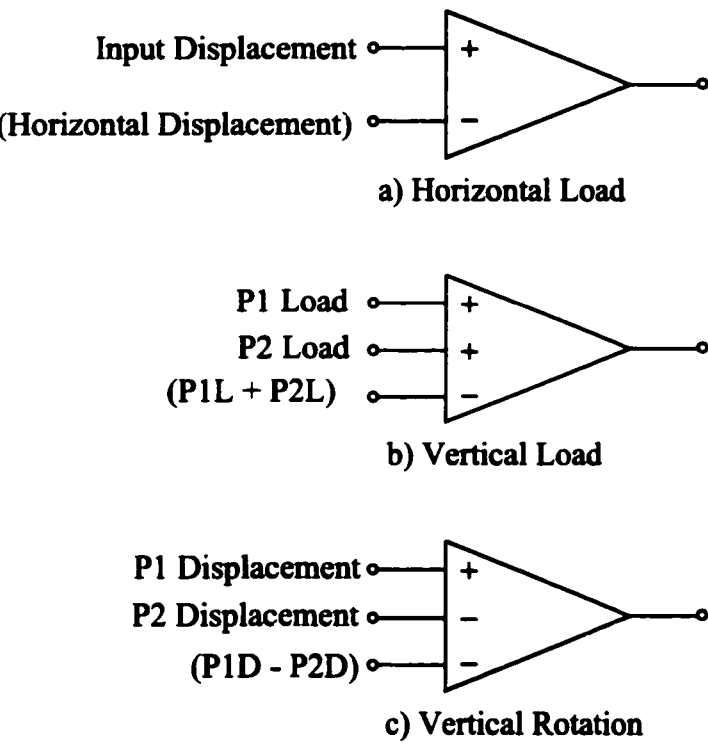


Figure D.2 Typical load and displacement controllers used in the tests

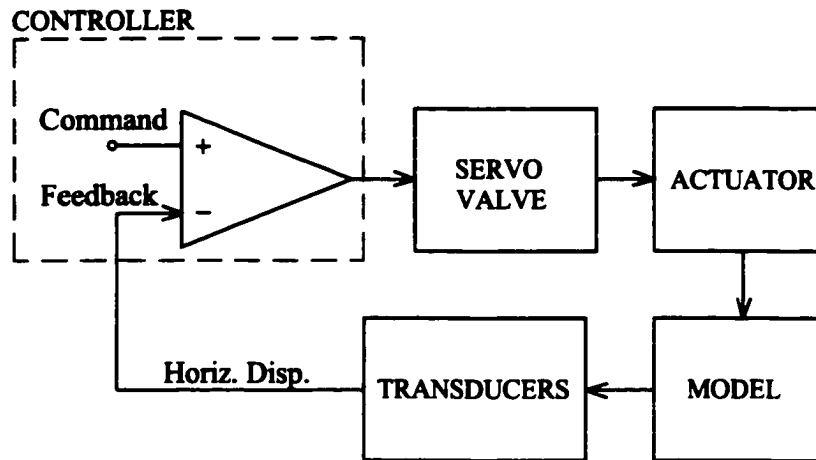


Figure D.3 Operation of horizontal actuator under displacement control

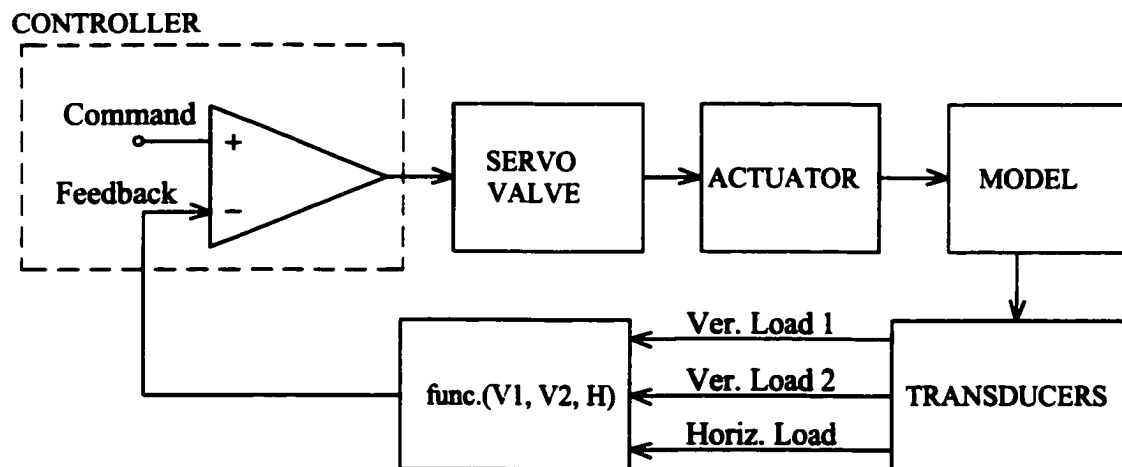


Figure D.4 Operation of vertical actuator under load control

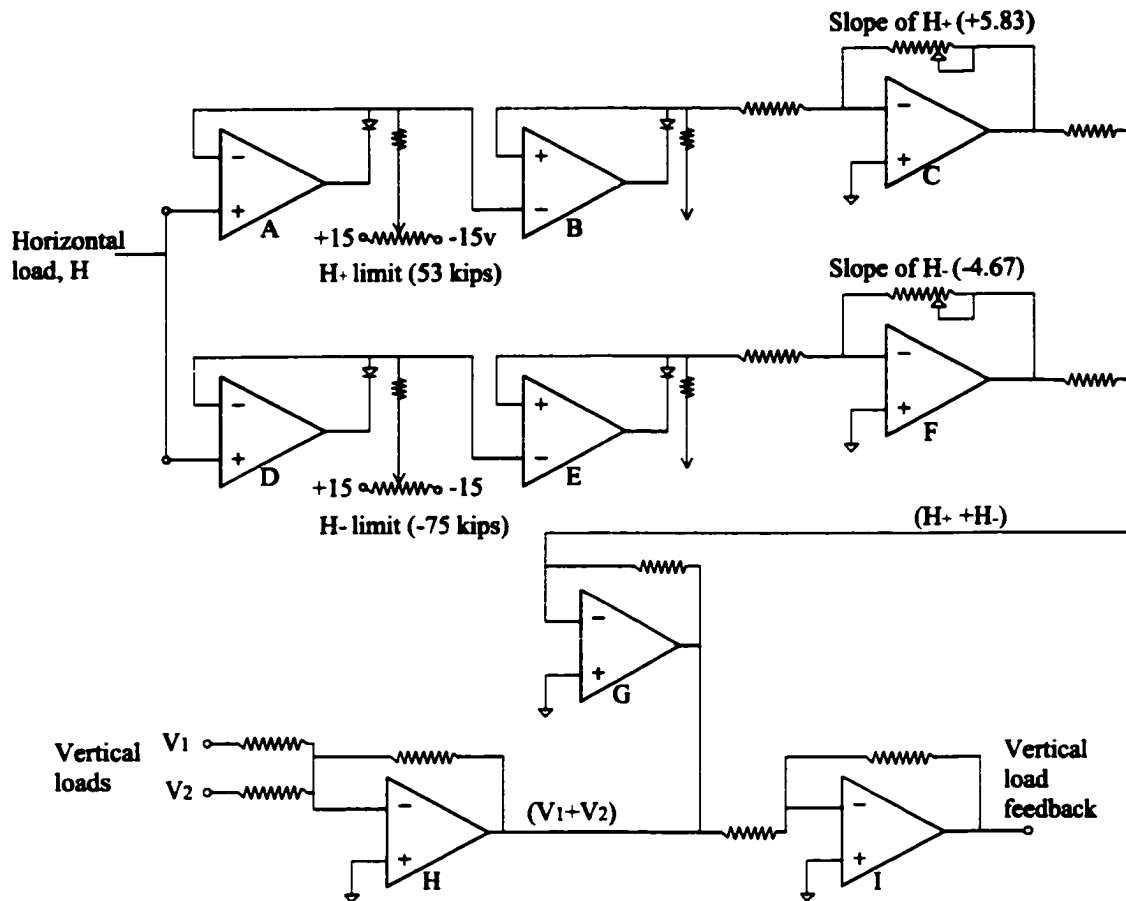


Figure D.5 Control box for the application of varying axial load

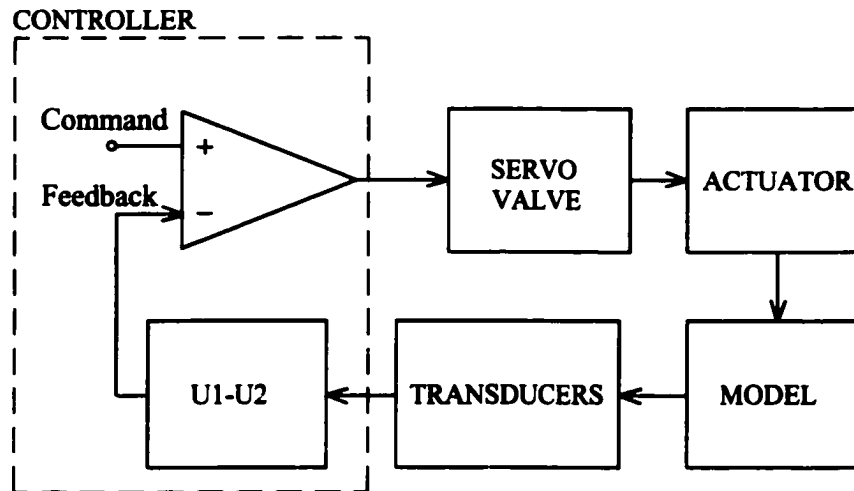


Figure D.6 Operation of vertical actuator under displacement (rotation) control

APPENDIX E: DISPLACEMENT AND STRAIN MEASUREMENTS

This appendix presents the measurements from the local displacement potentiometers (L.P.) installed on the specimen, and from strain gages (S.G.) attached on the transverse and longitudinal reinforcing bars.

Numbering and arrangement of strain gages are described in Section 3.8 (Figure 3.22). S.G. -A, -B, -C, and -D measure the strains on the longitudinal reinforcement, and S.G. -E and -F measure the strains on the transverse reinforcement. Numbering and arrangement of local potentiometers are described in Section 3.8 (Figure 3.26). For example, "L.P. A7" denotes the displacement potentiometer measures the vertical displacement between 12 in. and 16 in. above the column base on the north face (side "A") of the specimen. The displacement potentiometer, L.P. B29, measures the diagonal displacement along the column height between 16 in. and 40 in. above the column base on the south face (side "B").

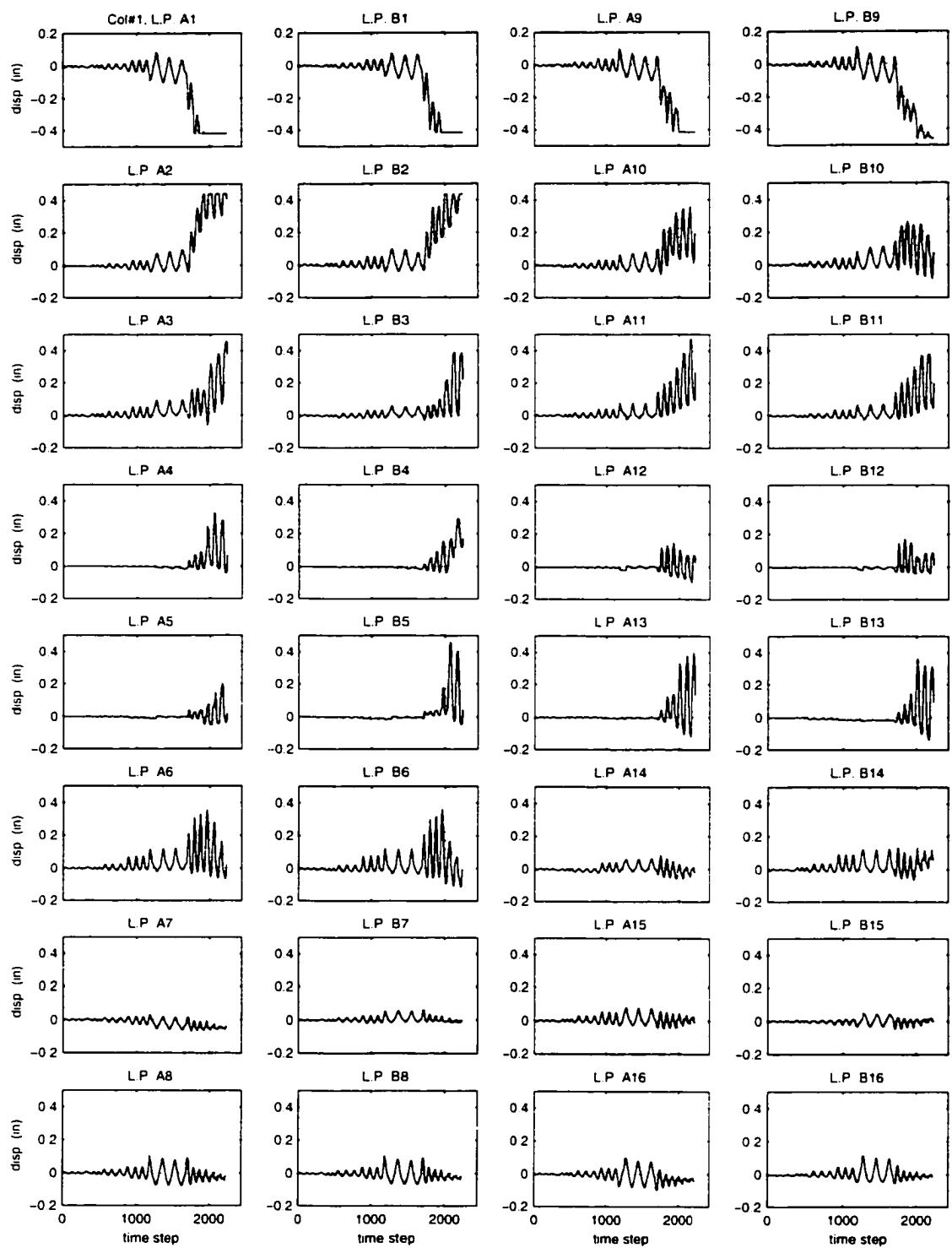


Figure E.1 Displacements recorded by vertical potentiometers (Specimen-1)

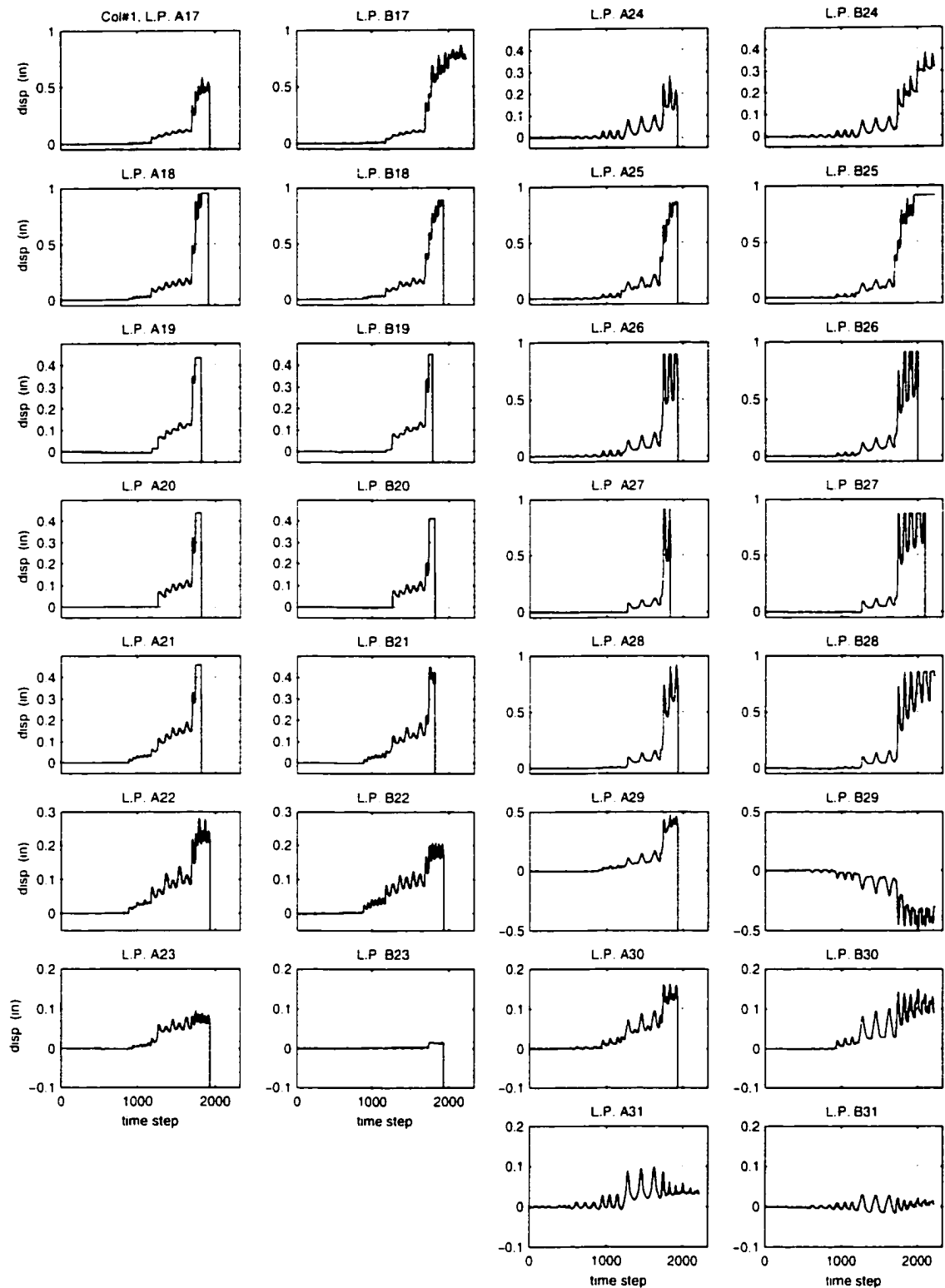


Figure E.2 Displacements recorded by horizontal and diagonal pots (Specimen-1)

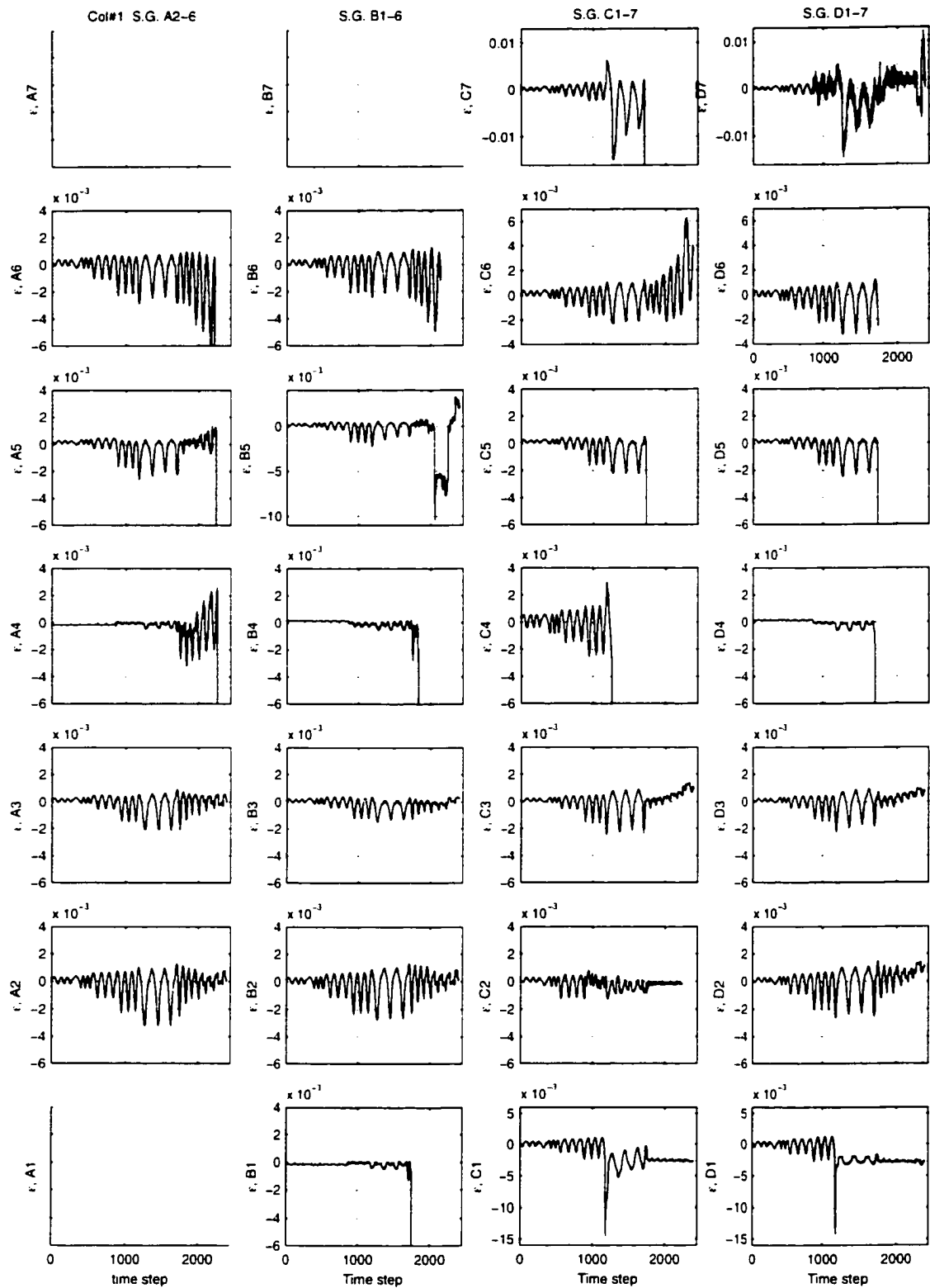


Figure E.3 Longitudinal reinforcement strain measurements (Specimen-1)

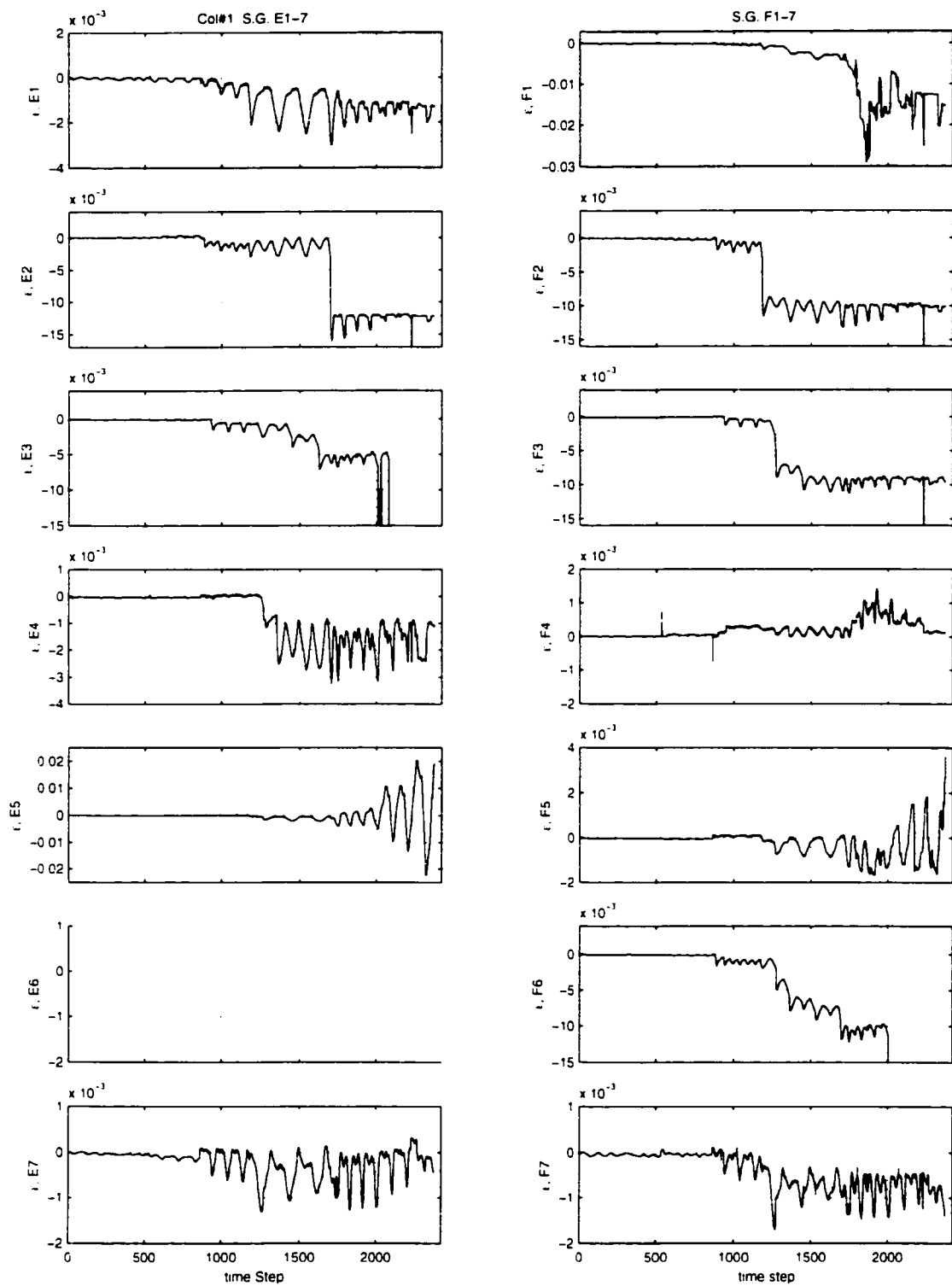


Figure E.4 Transverse reinforcement strain measurements (Specimen-1)

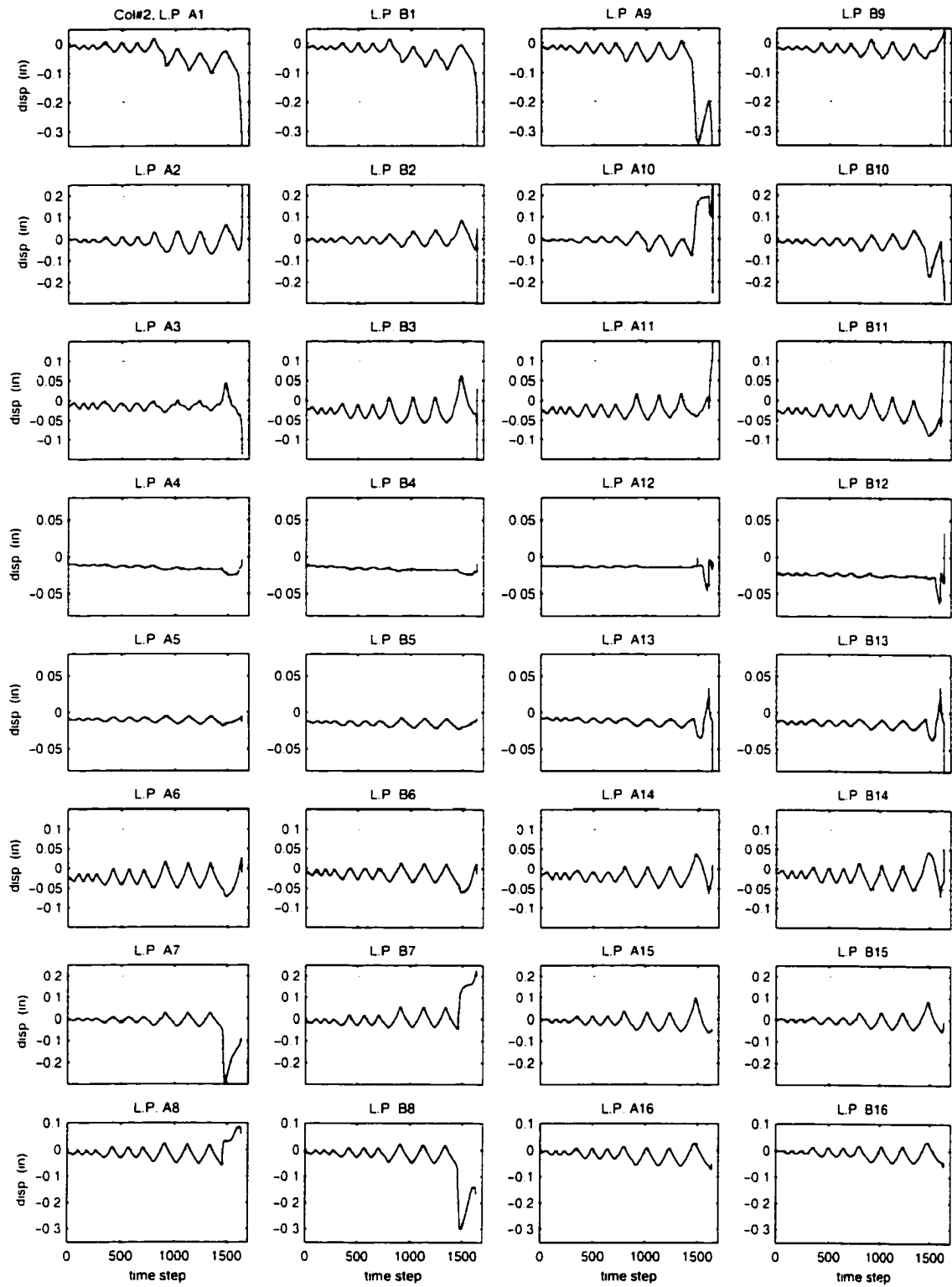


Figure E.5 Displacements recorded by vertical potentiometers (Specimen-2)

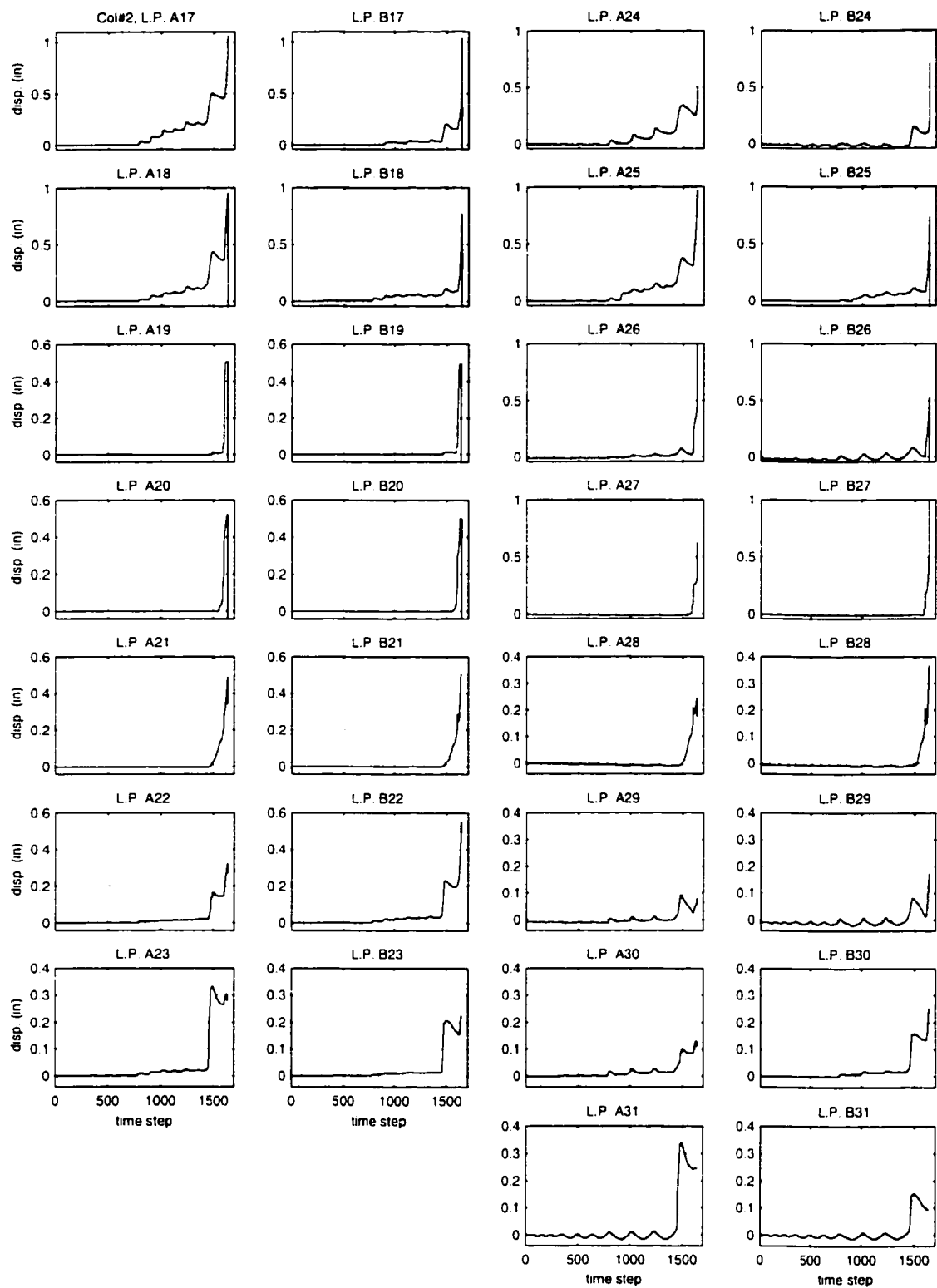


Figure E.6 Displacements recorded by horizontal and diagonal potentiometers (Specimen-2)

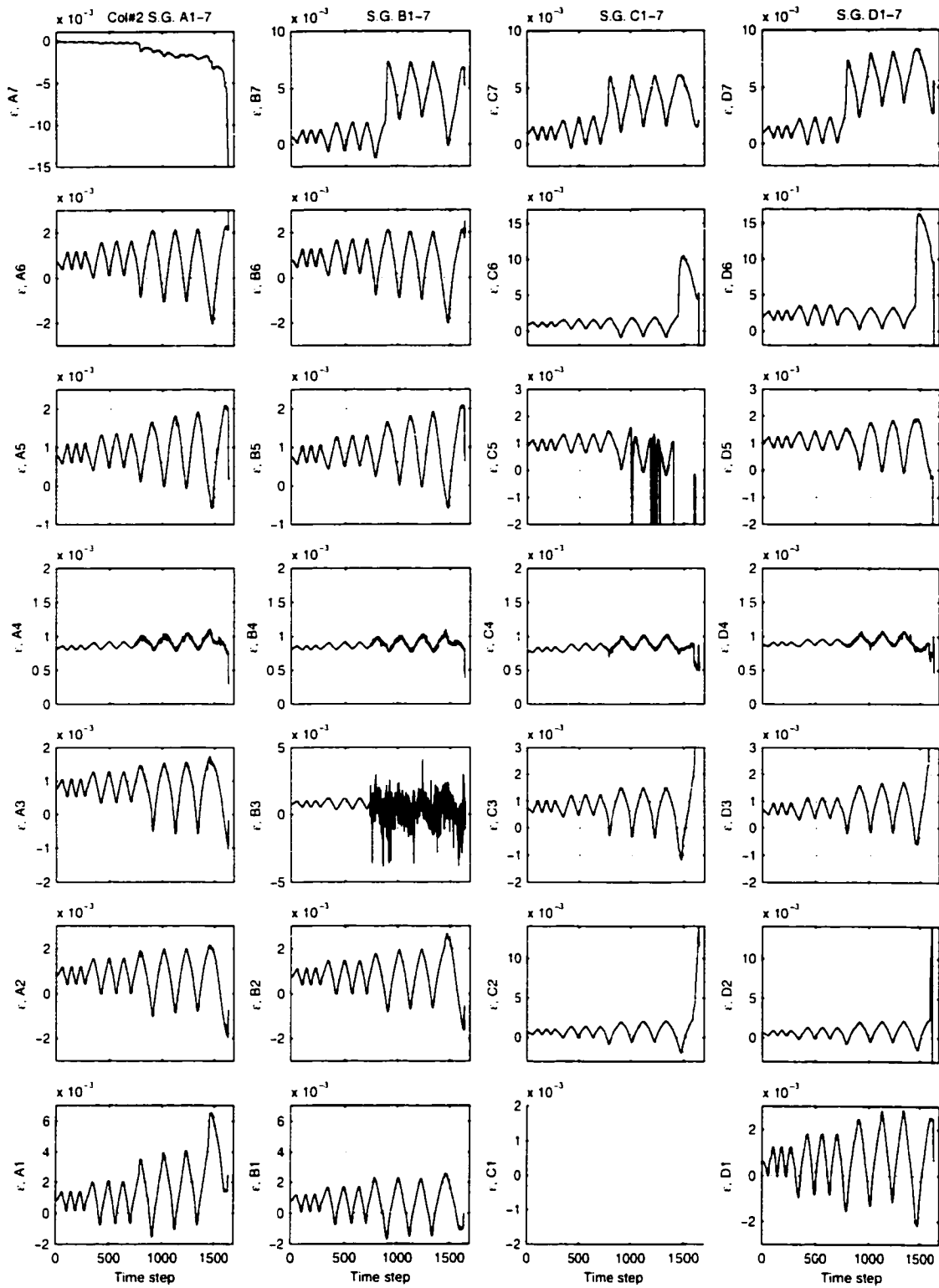


Figure E.7 Longitudinal reinforcement strain measurements (Specimen-2)

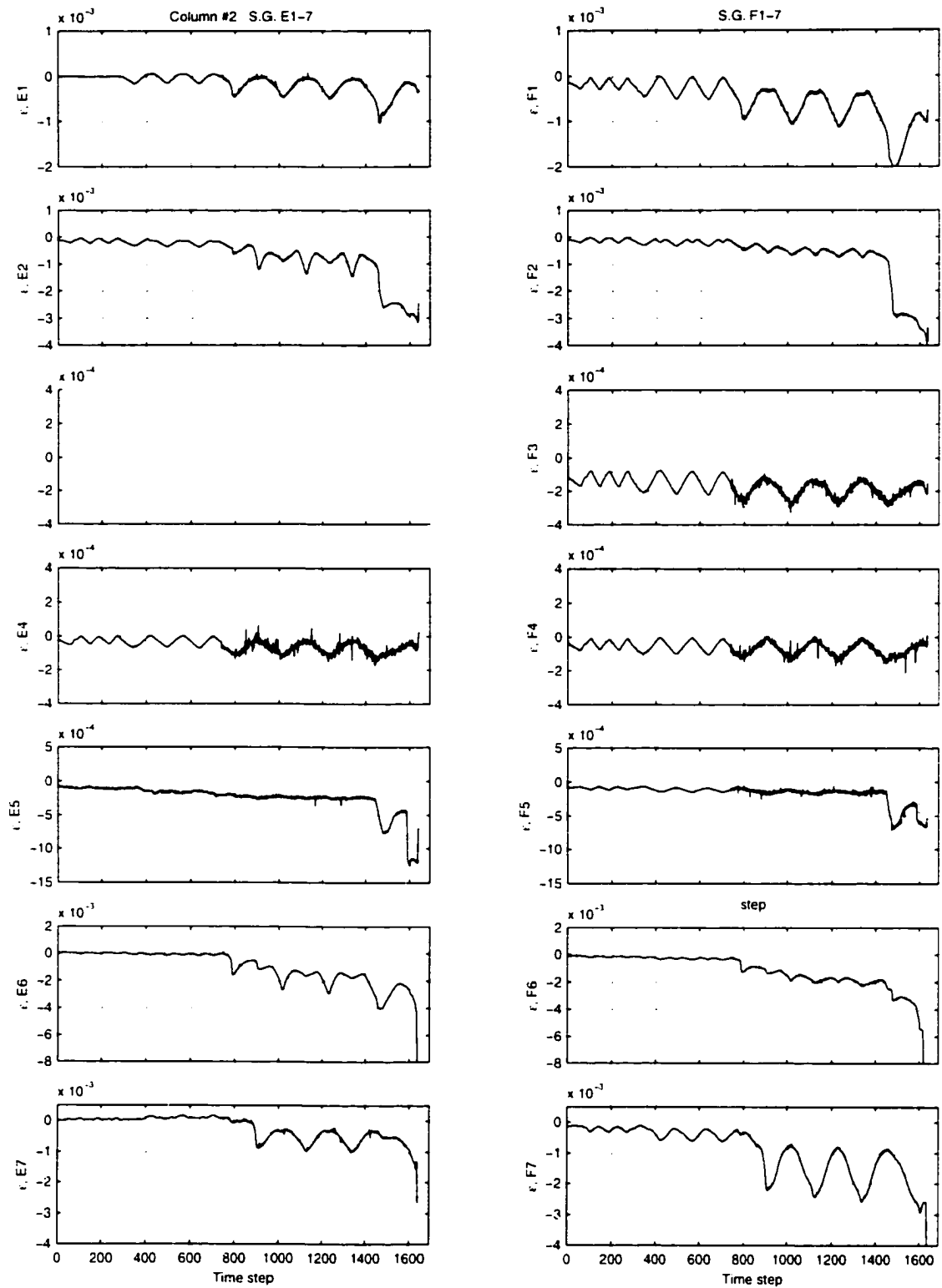


Figure E.8 Transverse reinforcement strain measurements (Specimen-2)

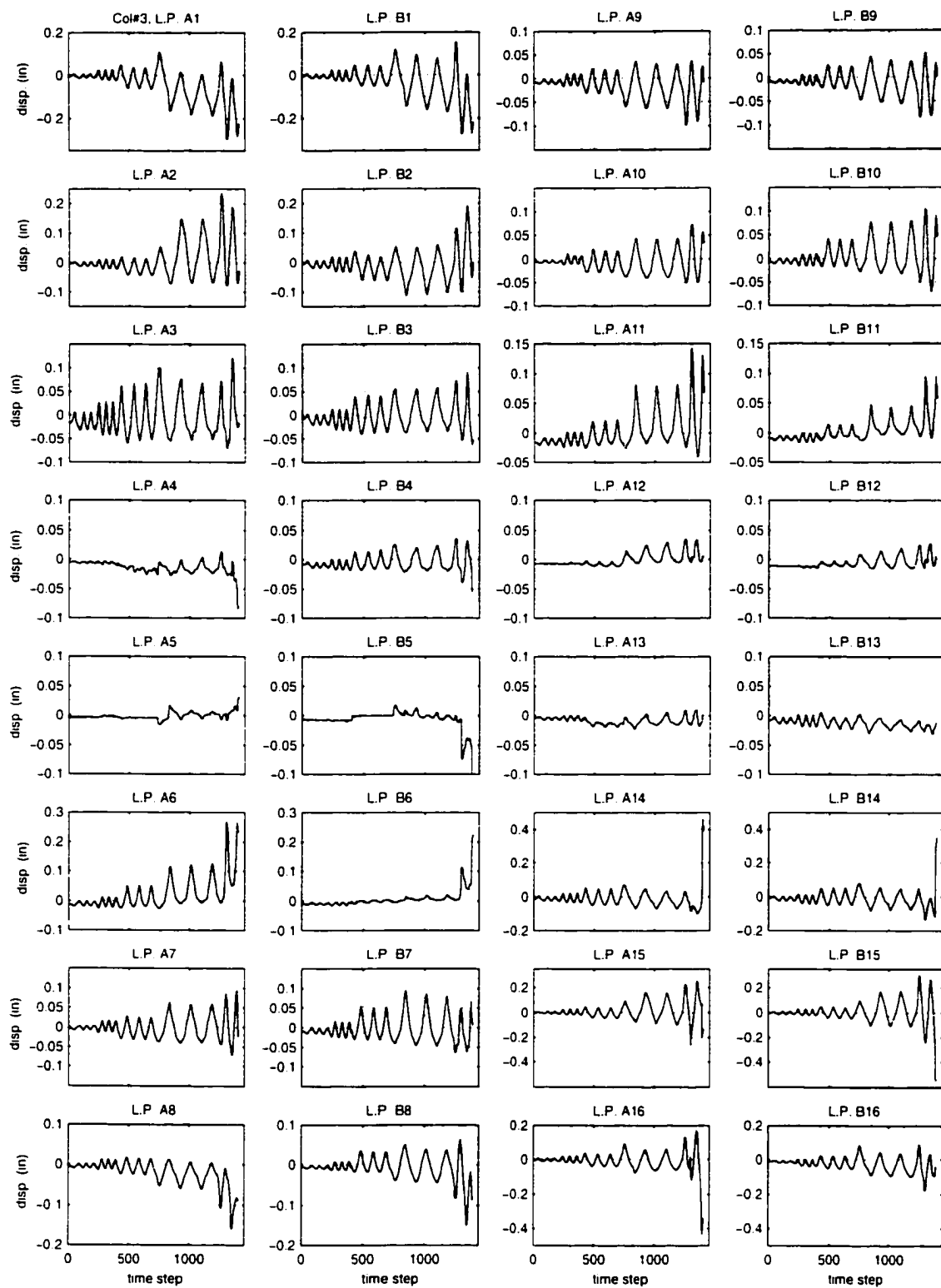


Figure E.9 Displacements recorded by vertical displacement potentiometers
(Specimen-3)

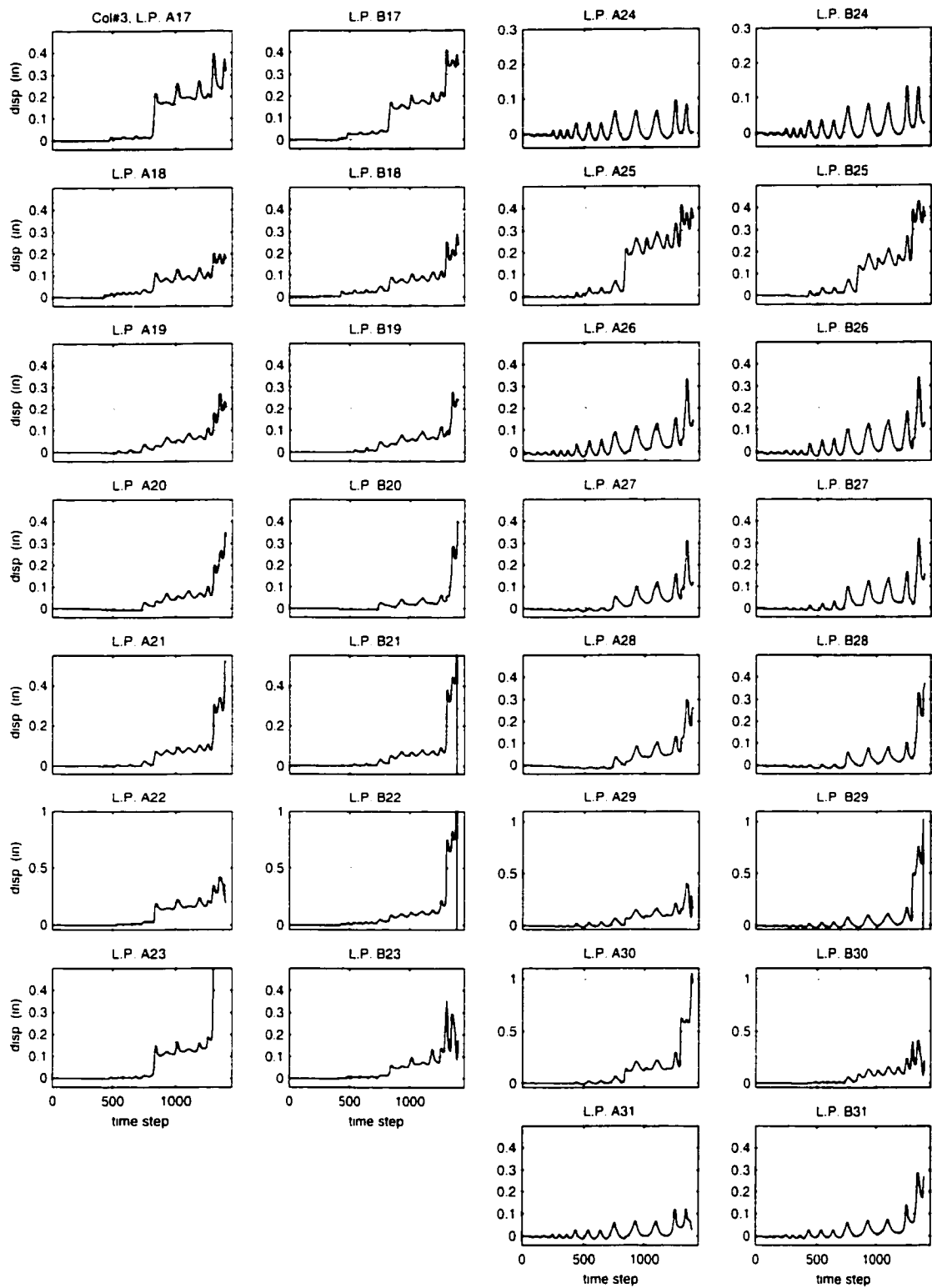


Figure E.10 Displacements recorded by horizontal and diagonal potentiometers (Specimen-3)

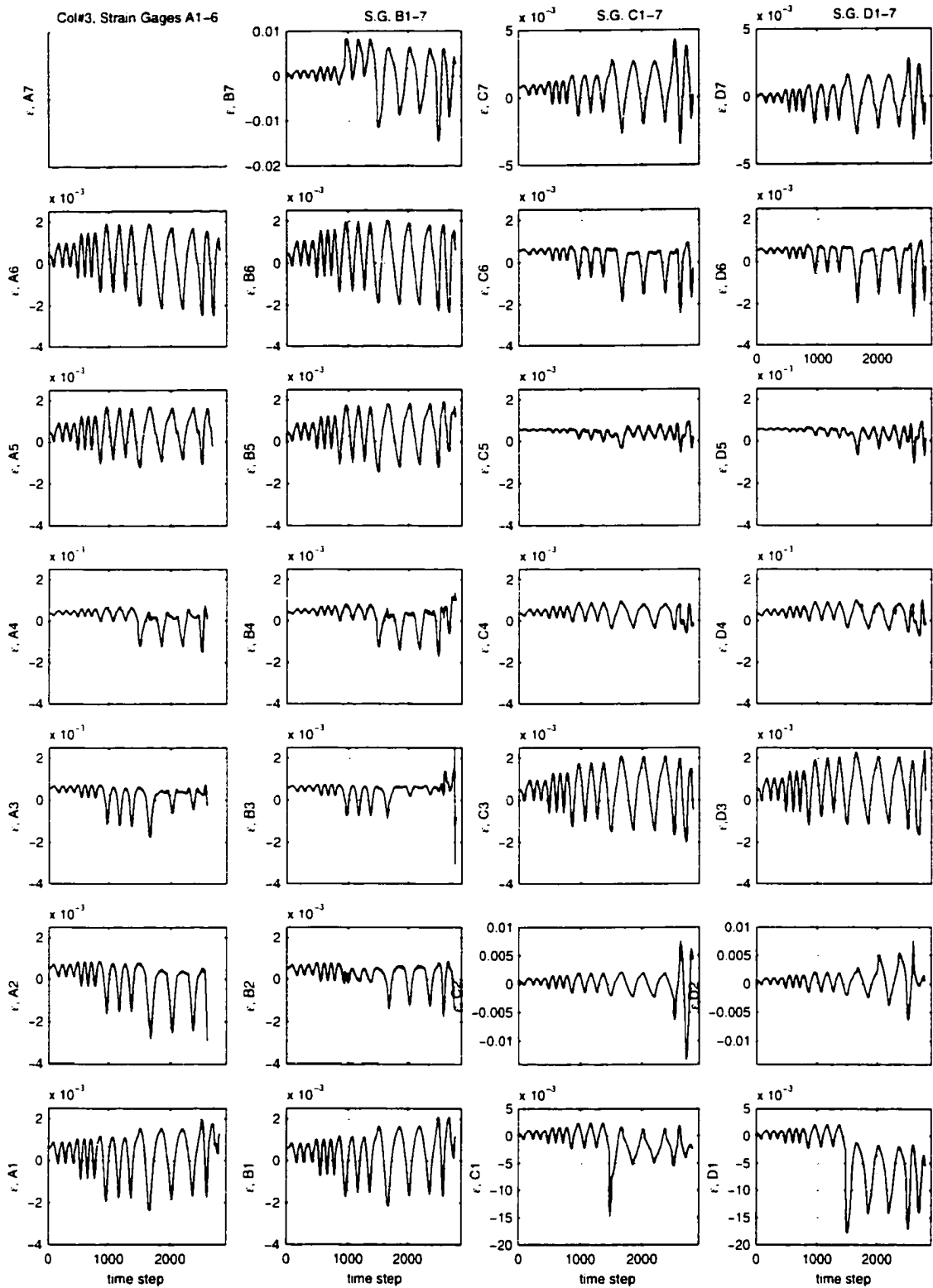


Figure E.11 Longitudinal reinforcement strain measurements (Specimen-3)

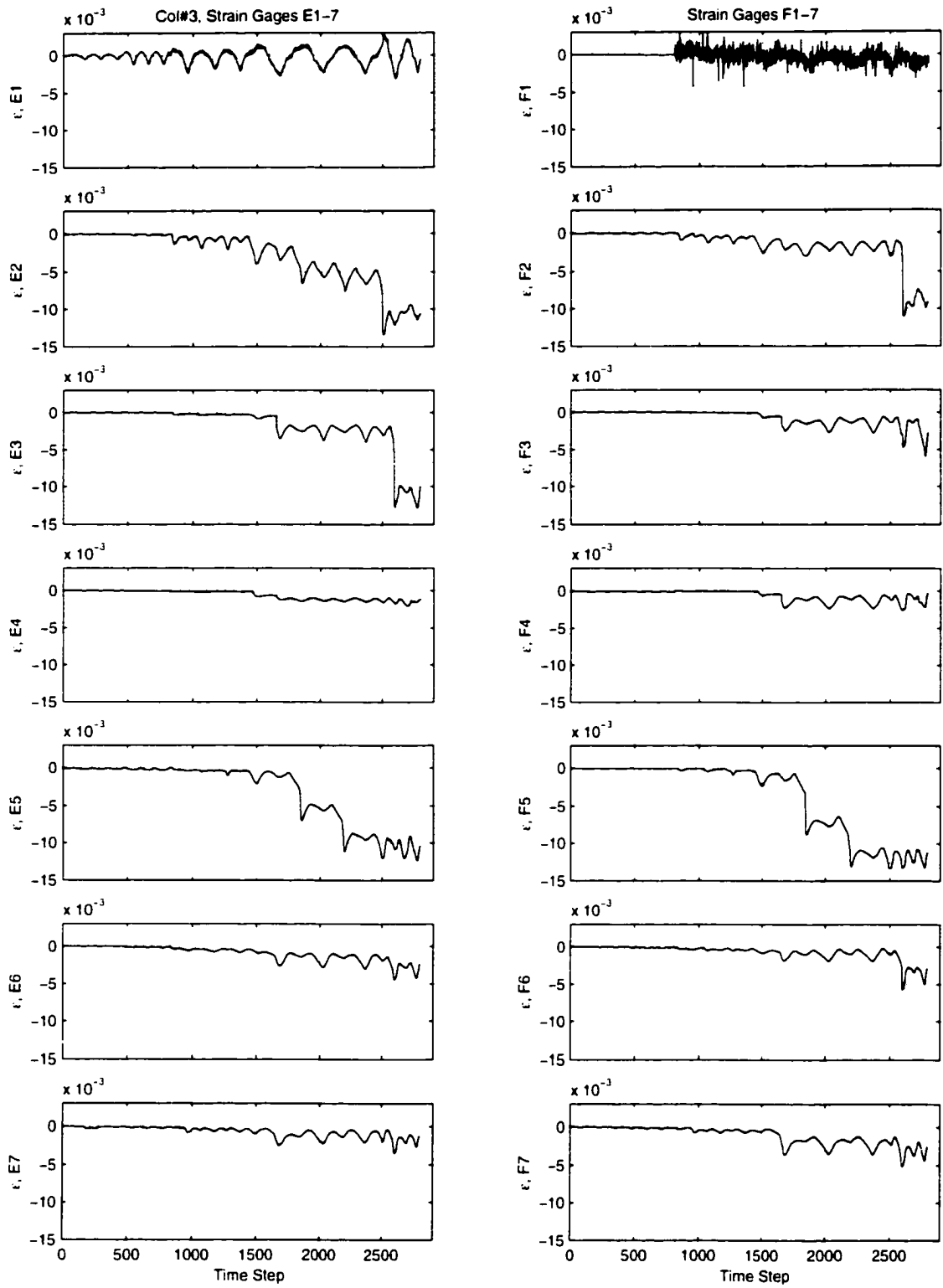


Figure E.12 Transverse reinforcement strain measurements (Specimen-3)

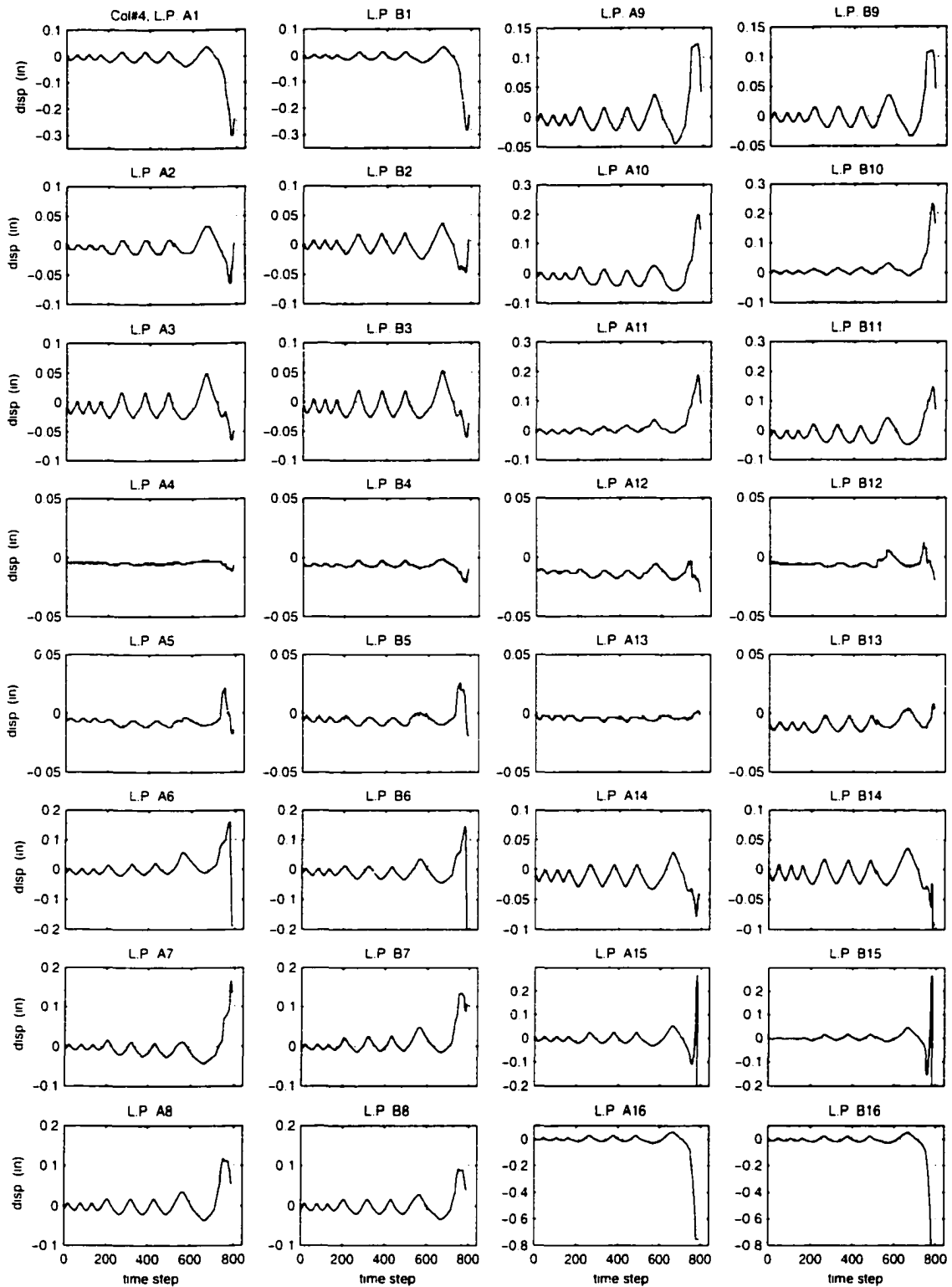


Figure E.13 Displacements recorded by vertical potentiometers (Specimen-4)

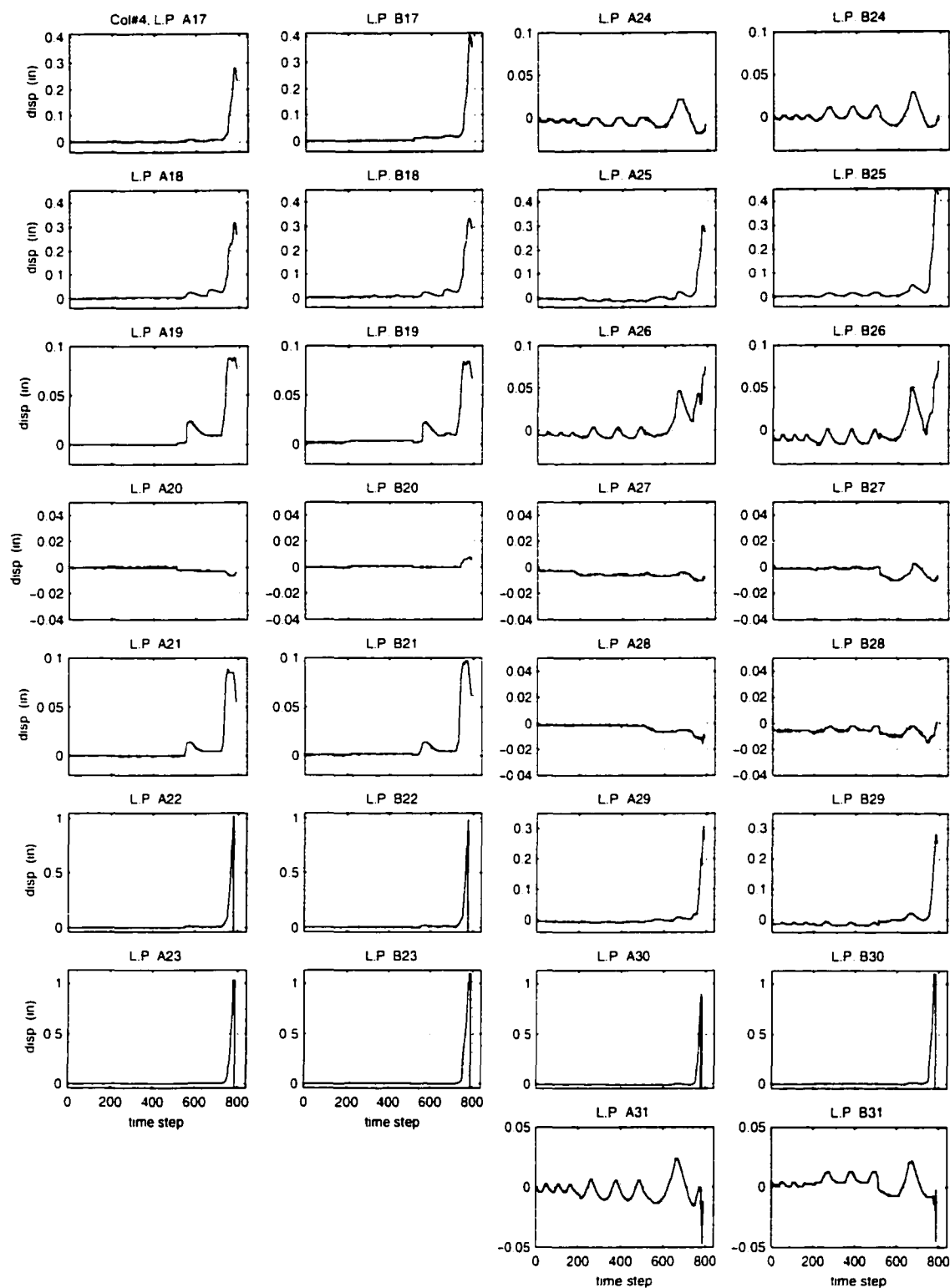


Figure E.14 Displacements recorded by horizontal and diagonal potentiometers (Specimen-4)

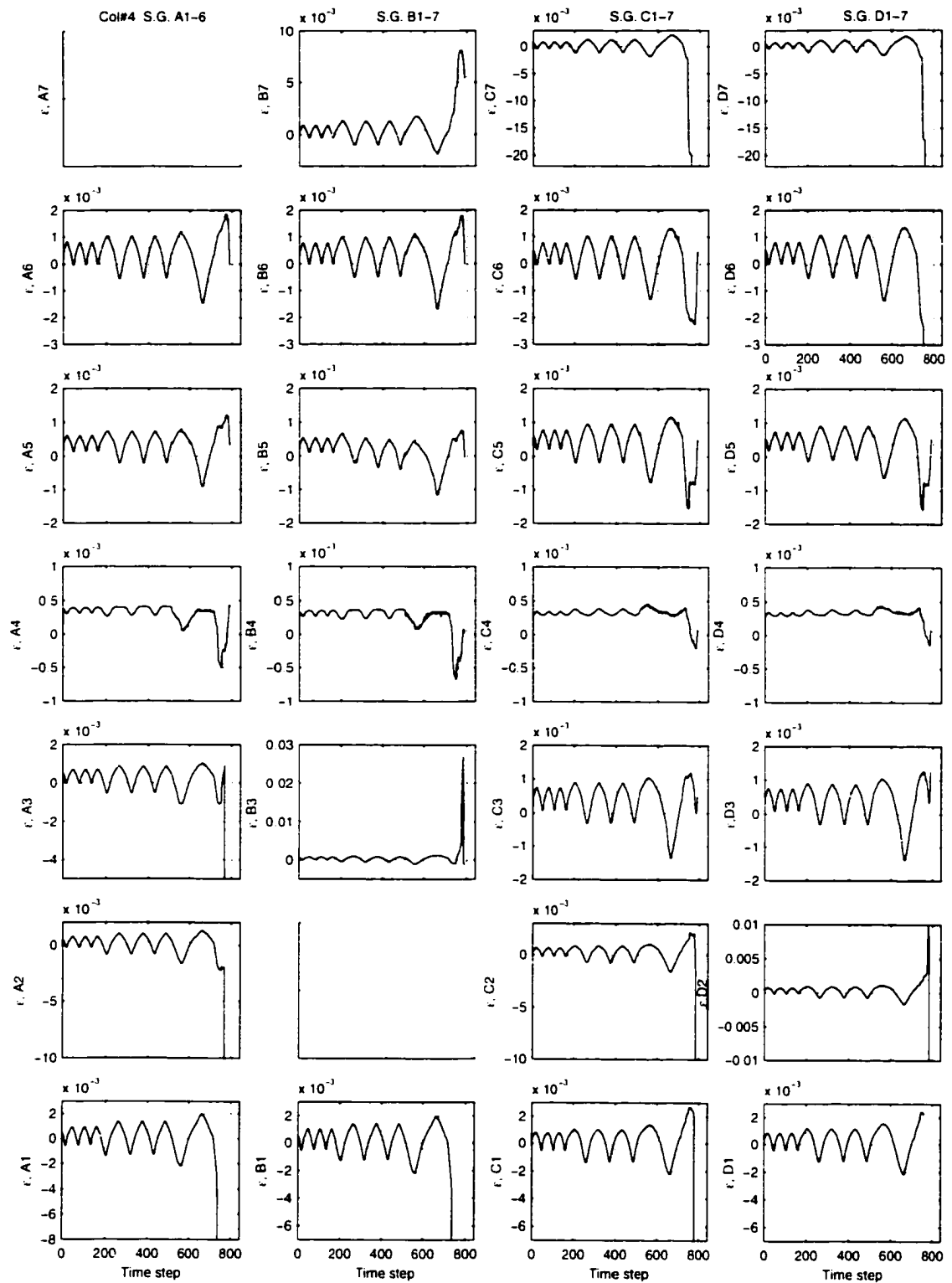


Figure E.15 Longitudinal reinforcement strain measurements (Specimen-4)

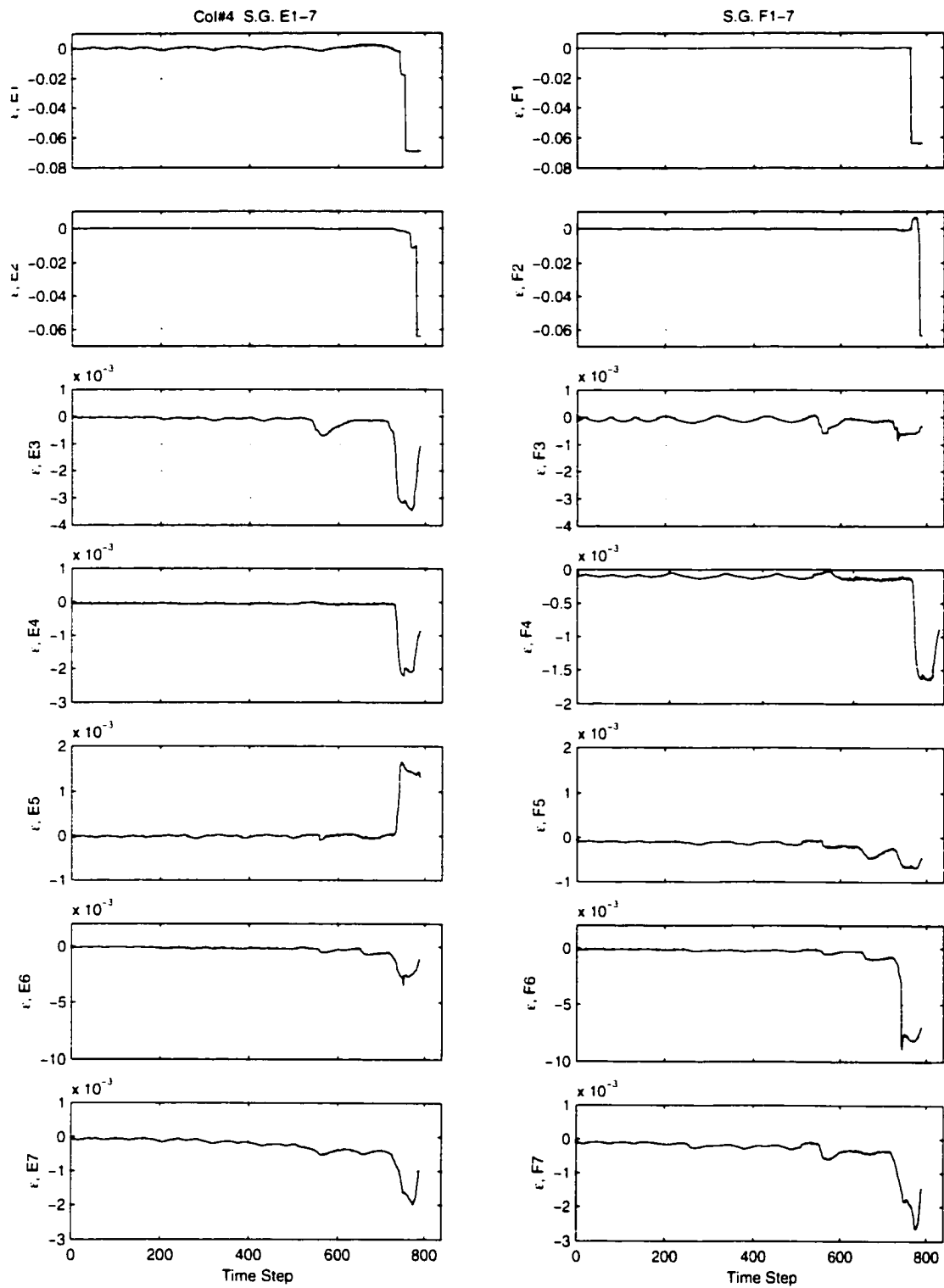


Figure E.16 Transverse reinforcement strain measurements (Specimen-4)

APPENDIX F: PRINCIPLE OF VIRTUAL WORK AND DISPLACEMENT CALCULATIONS

Background Information: Principle of Virtual Work

The principle of virtual work is based on the law of conservation of energy, according to which the work done by a set of external loads applied on a structure equals to the internal energy stored in the structure assuming that the system is elastic. If a structure subjected to a set of real displacements is acted on by a set of equilibrating virtual forces, then the virtual work done by the external forces, δW_{ext} , is equal to the internal virtual work of deformation done by the internal stresses, δW_{int} ; namely,

$$\delta W_{int} = \delta W_{ext} \quad (F.1)$$

One important application of the principle of virtual work, a very powerful tool in structural analysis, is found in the computation of deflections of structures. If a single external virtual force with a unit magnitude is placed at a point in a structural frame, the displacement of this point in the direction of the unit virtual force is equal to the sum of the products of the real displacements and the respective internal virtual member forces that are in equilibrium with the unit virtual force. For example, if one wishes to compute the horizontal component of the deflection, Δ of point a in the truss shown in Figure F.1.a, a virtual force $P = 1$ is applied to that point as shown in the figure. According to the principle of virtual work

$$1 \times \Delta = \sum p_i \times \delta_i \quad (F.2)$$

where p_i and δ_i are internal member forces due to unit virtual force and the member displacements caused by the applied actual forces, respectively.

Similarly, the horizontal displacement at point b and the vertical displacement at point a can be calculated by applying unit virtual forces at those points as shown in Figures F.1.b and F.1.c, respectively. It should be noted that the virtual truss system does not need to be compatible with the real truss system, and does not need to be stable. The virtual truss system shown in Figure F.1.c is unstable in the horizontal direction. However, it is in equilibrium in the vertical direction under the vertical unit load. Likewise, as shown in Figure F.1.d, because the segment $cdfe$ is in equilibrium, the segment $cdfe$ of the truss can be used to calculate the relative displacement between the points c and e along the line joining these two points.

The principle of virtual work can be used to calculate deflections of the structural systems with material nonlinearity. However, this method is valid for small deformations only and does not give accurate results when the geometric nonlinearities are significant. Consider, for example, a virtual unit force applied to a truss as shown in Figure F.2. The true vertical displacement, δ , and the vertical displacement calculated using the principle of virtual work are given as

$$\begin{array}{ll} \text{Virtual Work :} & \delta_v = L - L' \\ \text{True :} & \delta_v = L - L' \cos \beta \end{array} \quad (\text{F.3})$$

where L and L' are the initial and final length of the vertical truss member, respectively.

With increasing angle of rotation, β , the difference between the two quantities, i.e. the error, increases.

Displacement Calculations

The virtual work principle is a powerful tool for displacement calculations since it can replace complicated geometric relations. Under a given loading condition, the total lateral displacement at point c in the frame shown in Figure F.3 can be calculated by applying a unit load at that point. Similarly, the virtual work principle can be used to calculate the shear displacement only at the same location. As shown in Figure F.3.c, the shear displacement at point c can be obtained by calculating virtual member forces due to a horizontal unit load at point c while preventing the rotation of members dc and eb . In other words, in the frame analysis, the points c and d , and points e and b are constrained to have the same vertical displacement.

As demonstrated in Figure F.4 and presented by many researchers, such as Kowalsky et al. (1995), and Ohtaki et al. (1996), the shear deformation, Δ_s , of a typical frame segment, e.g. $ABCD$ in the figure, can be calculated using geometrical relationships.

$$\Delta_s = \frac{\Delta_{ds}}{\cos \theta} \quad (F.4)$$

The change in the diagonal length, Δ_{ds} , can be calculated as

$$\Delta_{ds} = \Delta_d - 2\Delta_{dv} - 2\Delta_{dh} \quad (F.5)$$

where Δ_d is the measured diagonal displacement (Figure F.4a), and Δ_{dv} and Δ_{dh} are half of the diagonal displacements due to vertical extension and horizontal expansion, and they can be calculated as

$$\begin{aligned}\Delta_{dv} &= (\Delta_v / 2) \sin \theta \\ \Delta_{dh} &= (\Delta_h / 2) \cos \theta\end{aligned}\tag{F.6}$$

where Δ_v is the vertical displacement that can be obtained as the average of the measured displacements on the left and right hand side of the segment (Δ_l and Δ_r in Figure F.4a), and Δ_h is the horizontal displacement that can be obtained as the average of the measured top and bottom displacements (Δ_t and Δ_b). Substituting Equation F.5 and F.6 into Equation F.4 gives

$$\Delta_v = \frac{\Delta_d}{\cos \theta} - \left(\frac{\Delta_t + \Delta_b}{2} \right) - \left(\frac{\Delta_l + \Delta_r}{2} \right) \tan \theta\tag{F.7}$$

Thus, as an alternative to the virtual work method, in the truss system shown in Figure F.3, the total shear displacement at point *c* can be calculated by summing up shear displacements of individual segments *abef* and *bcde* from Equation F.7. However, in more complicated frame systems, use of virtual work method would be more convenient for shear displacement calculations.

Application of Principle of Virtual Work: Displacements

The instrumentation frame installed on the specimens tested in this investigation (Figure 3.23) can be idealized as a truss system as shown in Figure F.1. In this idealized

determinate truss system, material properties for the truss members are not required for displacement calculations. Equation F.1 can be rewritten to calculate total and shear displacements at any level over the height of the columns

$$\begin{aligned}\Delta_{total} &= \sum p_i \delta_i \\ \Delta_{shear} &= \sum s_i \delta_i\end{aligned}\tag{F.8}$$

where δ_i is the experimental displacements measured by each potentiometer representing the members in the truss system, p_i and s_i are the member forces under unit load without and with rotational constraints on the horizontal members in the truss analysis, respectively. In other words, for shear displacement calculations, rotations of the horizontal members are prevented in the truss analysis. Member forces in the virtual truss system due to unit loads at midheight and top of the truss, i.e. at points *a* and *b* in Figure F.1, are listed in Table F.1. The numbering of the truss members is shown in Figure 3.26. Substituting measured displacement histories δ_i and virtual truss member forces shown in the table into Equation F.8, the shear and total displacement histories at midheight and top of each test column can be calculated.