

**PERFORMANCE OF CIRCULAR
HIGH-STRENGTH CONCRETE COLUMNS
UNDER LATERAL LOAD REVERSALS**

by

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Abstract

In recent years, high-strength concrete (HSC) has gained a more widespread acceptance in the construction industry. This is evident from the increasing number of building construction projects where HSC has been successfully utilized. HSC offers both structural efficiency and economy over normal strength concretes, especially when used in building columns. In spite of the advantages inherent to HSC, concerns still exist about its relative brittleness, particularly in regions of high seismic risk. Clearly, the lack of experimental data is the fundamental cause of such concerns.

An experimental and analytical investigation was conducted to study the behaviour of circular HSC columns subjected to simulated seismic loading. The experimental program included the testing of nine full-scale circular columns under combined axial compression and lateral load reversals. The emphasis was on the strength and deformation characteristics in relation to various confinement parameters. These parameters included the compressive strength of concrete, the yield strength, volumetric ratio and spacing of transverse reinforcement, the level of axial load, the type of circular reinforcement, and the presence of concrete cover. The effects of test parameters were analysed by comparing the force-displacement relationships among pairs of companion specimens. The results show that ductile behaviour can be obtained from HSC columns subjected to lateral deformation reversals, provided they are suitably confined.

The analytical component of the research program involved the computation of theoretical force-displacement relationships through the application of well established analysis techniques, commonly used for normal-strength concrete elements, and a recently proposed confinement model. Member response was calculated by modelling the plastic hinge region and following an algorithm for the progression of hinging. The analysis provided the theoretical inelastic column displacements caused

by flexure and anchorage slip. The applicability of these analytical methods to HSC columns was verified by comparing the force-displacement relationships obtained from the theoretical computations with those recorded experimentally. The results indicate that the analytical methods utilized can be applied to high-strength concrete members, given that proper material and inelastic behaviour models are used.

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Notations

A_g	=	gross cross-sectional area of column
A_s	=	total area of longitudinal reinforcing steel
d	=	diameter of circular column section
d_b	=	bar diameter of confinement reinforcement
F	=	lateral force acting on the column
f_c'	=	concrete compressive strength based on standard cylinder test
f_{cc}'	=	strength of confined concrete in column core
f_{co}'	=	strength of unconfined concrete in column core
f_{yt}	=	yield strength of longitudinal reinforcing steel
f_{ys}	=	yield strength of transverse reinforcing steel
h	=	theoretical plastic hinge length
M	=	base moment in column caused by lateral force
M_n	=	nominal moment capacity of column
P	=	axial force acting on the column
P_o	=	concentric axial load capacity as expressed by <i>Eq.</i>
s	=	center-to-center spacing of spiral reinforcement along column height
u	=	frictional bond stress between concrete and reinforcing steel
ρ_f	=	longitudinal reinforcement ratio
ρ_t	=	volumetric ratio of transverse reinforcement determined as total volume of transverse steel divided by volume of concrete
Δ_y	=	yield displacement
Δ_{max}	=	maximum displacement
μ	=	displacement ductility factor

ϵ_{01} = strain corresponding to peak stress of unconfined concrete
 ϵ_{85} = concrete strain at 85 percent of peak stress on descending branch

CHAPTER 1

Introduction

1.1 General

High-strength concrete (HSC) has recently gained acceptance in the construction industry, and has been utilized successfully in a number of building construction projects. HSC offers structural efficiency and economy over normal strength concretes, especially when used in building columns. The advantages of HSC include higher strength and stiffness which permit smaller column sections and lower anchorage lengths. In addition, the performance of concrete improves significantly with strength in terms of durability, corrosion resistance, freeze-thaw resistance, abrasion resistance and permeability. The use of higher strength concrete in lower storey columns of multistorey buildings also permits uniform column sections, resulting in savings associated with formwork. In spite of apparent superiority of HSC over conventional concretes, the use of HSC as a column material has been hindered, especially in areas of high seismic risk, because of concerns over the brittleness of this material.

In moment-resisting frame buildings, columns are the critical lateral load resisting elements. This is because of the potentially catastrophic consequences of column failure on the performance of an entire structure. Designing lower storey columns to remain elastic during a strong earthquake is an uneconomical proposition in most cases. The common strategy in the design of structures for earthquake resistance is to ensure ductile post-yield response, recognizing that the critical structural elements may develop significant inelasticity during response to strong ground excitations.

Ductility may be defined as the ability of a structure or a structural component to deform beyond

elastic limits, without excessive strength or stiffness degradation. Distinct elements of the primary lateral force resisting system must be suitably designed and detailed for energy dissipation. Furthermore, to ensure that stability and vertical load carrying capacity are maintained while a structure undergoes large lateral displacements, a "strong column - weak beam" concept is suggested in most design codes. The critical regions, where inelastic actions take place, are commonly referred to as "plastic hinges". The intent is to ensure that the plastic hinges form in beams rather than in columns. Nevertheless, the structural damage that occurs during many strong earthquakes indicates that the formation of plastic hinges at the base of first storey columns can not be entirely avoided. The ideal sway mechanism is one where hinges form at the beam ends and at the base of the ground level columns. The ability of a structure to withstand a severe ground motion depends mainly on the capacities of these plastic hinges to absorb and dissipate energy without significant loss of strength.

The ductile behaviour of columns and that of the structure can be attained by properly confining the plastic hinge regions by transverse reinforcement. The improvement in ductility, either through confinement of core concrete or by promoting tension dominant flexural response is the preferred way of avoiding catastrophic collapse during a strong earthquake, rather than trying to estimate the required force levels to be attained during an earthquake. This is true since the precise nature of the ground motion is impossible to predict, and may be considerably greater than what is allowed for in design. The ideal structure is therefore one which resists the extreme earthquake, not by brute force, but by allowing plastic deformations to absorb the kinetic energy induced by the ground shaking.

Improved seismic performance can also be expected from properly designed HSC columns. With adequate confinement of the core concrete ductile behaviour of columns can be achieved even if otherwise brittle material like HSC is used in the member. However, experimental data on the ductility of HSC columns is very limited. To provide the much needed information, it is necessary to carry out experimental research on full-scale or large-scale HSC columns. The aim of this research project is to study the behaviour of confined circular HSC columns under simulated seismic loading, in relation to various confinement parameters.

1.2 Previous Research on HSC Columns

The majority of previous research on high-strength concrete columns involved experimental studies. A number of projects also included analytical work which consisted primarily of developing models to predict the stress-strain behaviour of confined HSC in columns. The primary objective of previous work was to establish the relationship between ductility and confinement of concrete columns. This involved parametric studies on the effects of confinement variables on the behaviour of HSC columns. A summary of research reported in the literature on HSC columns under simulated seismic loading is presented below.

H. Muguruma, F. Watanabe, T. Iwashimizu, and R. Mitsueta, 1983

The authors conducted tests of 18 reinforced concrete column specimens under concentric axial compression. The specimens consisted of scaled columns with a height of *400 mm* and a square cross section of *147 mm x 147 mm*. All columns were transversely reinforced with high strength steel hoops, with the exception of 4 plain concrete control specimens. No longitudinal reinforcement was provided. The hoop spacing was uniform at *50 mm* and the yield strength of steel was *1360 MPa*. The concrete strength and the volumetric ratio of transverse steel were considered as test variables. Of the 10 reinforced specimens, 8 had a volumetric ratio of 2.13% and the remaining had 4.26%. The unconfined concrete strength ranged between *34 MPa* and *88 MPa*.

The primary objective of this investigation was to establish the effect of lateral confinement on axial ductility of square HSC columns. The authors also wanted to verify an idealized bi-linear stress-strain model, developed earlier for confined normal-strength concrete, by introducing the data obtained in their experimental program. It was concluded that the brittle behaviour of HSC can be significantly improved by an adequate amount of lateral reinforcement with a high yield strength. The authors implied that a high volumetric ratio of transverse steel was most beneficial to ductile behaviour. Their proposed stress strain model for was verified to be applicable to HSC columns.

This report summarizes the results of an experimental investigation conducted to study the response of HSC columns confined with steel spirals under compressive loading. A total of 94 short columns, in the form of cylinders, were tested. Three diameters were used, including 4, 5, and 6 inches (102, 127, and 152 mm) and most of the specimens were cast without cover. Both normal and light-weight concrete mixes were used. No longitudinal reinforcement was used. The reinforcement consisted of spirals of various diameters and pitch. The main variables studied were the strength of concrete, volume of transverse steel, and the specimen size. Concrete strength ranged between 21 MPa and 69 MPa for normal-weight concrete.

The main objective of the study was to establish the differences between behaviour of HSC and normal-strength concrete (NSC). These differences can be summarized as follows:

1. The column compressive strength, strain at peak stress, and plasticity ratio of the core all increase as the confinement stress increases, regardless of concrete strength. The strain in concrete at peak stress is substantially higher for HSC than for NSC.
2. The modulus of elasticity of spirally confined cylinders is essentially the same as for unconfined concrete.
3. Spirals designed in accordance with the ACI 318-83 code recommendations provide sufficient strength gain in columns to compensate for spalling of cover up to 83 MPa. However, the decrease in capacity is much higher in HSC columns than in NSC columns.
4. ACI 318-83 design recommendations for spiral reinforcement are not applicable to low-weight concretes regardless of strength.
5. The use of spiral steel with yield strength above 414 MPa may result in unconservative designs if it is assumed that the steel is at yield at the computed column failure load.
6. The design method for spiral steel in the ACI 318-83 code should be re-examined because they lead to low confinement stresses for large columns. Also shown was the importance of the size effect, since small-scale columns had higher strengths than larger specimens.

R. Basset, and S. M. Uzumeri, 1986

An experimental investigation was conducted by the authors on the behaviour of high-strength light-weight aggregate concrete (HSLWAC) columns. A total of 15 reinforced and 3 unreinforced specimens were tested under monotonically increasing axial compression. The core dimensions, measured to the centre of perimeter ties, were all $267\text{mm} \times 267\text{mm}$. Concrete strength was 40 MPa at a density of only 1845 kg/m^3 . The variables of interest were, a) the longitudinal steel distribution and tie configuration, b) tie spacing, c) amount of tie reinforcement, and d) amount of longitudinal reinforcement.

The objective of this investigation was to determine the behaviour of confined HSLWAC columns in order to better judge its suitability for structural use. Also, the effects of several variables which affect the effectiveness of confinement were considered. The following conclusions were drawn as a result of the test program.

1. Unconfined HSLWAC is a brittle material.
2. The addition of lateral confinement steel significantly improves the behaviour of this material. With a large amount of lateral steel, the behaviour is very ductile.
3. The tie configuration and the resulting distribution of longitudinal steel greatly contributes to confinement.
4. The ratio of specimen-to-cylinder strength was observed to be 0.98 and not 0.85 commonly assumed.
5. Existing concrete confinement models give unconservative results for HSLWAC and overestimate the ductility that can be achieved.

A. Fafitis, and S. P. Shah, 1987

The scope of this research project consisted of three phases. The first part consisted of proposing a stress-strain relationship for unconfined and confined normal strength concrete. This simple model was based on the results of small-diameter confined concrete cylinders. The second part involved the verification of the proposed model against the results of experimental research conducted on full-size square and round columns at the University of Canterbury, University of Toronto and at the Portland Cement Association. In the final phase, the authors conducted an

analytical study to assess the influence of concrete strength, degree of confinement, axial load level, and column shape, on the capacity of columns subjected to large deformations. A computer program was employed using the theoretical procedure which successfully predicted the other researchers' data. Both square and circular columns were analyzed and moment-rotation envelopes were computed. The circular and square columns were adjusted to keep the gross and core areas the same to facilitate comparison. The total longitudinal steel was also kept uniform in all columns. The strength of concrete varied from 27.6 MPa and 62 MPa, and three axial load levels were considered, consisting of 25%, 50%, and 80% of concentric capacity.

The objectives of this investigation were to:

- 1) Compare the analytical prediction of ultimate loads and moments, as well as load-moment-curvature relationships with available experimental data.
- 2) Conduct a parametric study to evaluate the effects of some major variables on the ultimate behaviour of columns subjected to eccentric loading and large deformations.

It was concluded that:

- 1) At high axial load levels, ductility of columns is substantially reduced.
- 2) The results attained from axial load tests (moment-rotation) can be used to predict the cyclic behaviour of columns.
- 3) Under constant axial load, the moment resistance of columns may exhibit a peak followed by a drop.
- 4) The extent of drop in moment depends on compressive strength of concrete, axial load level, section shape, and confinement.
- 5) The square sections examined exhibit higher moment capacity than those of the circular section, especially at large deformations.
- 6) The contribution of the cover concrete becomes negligible beyond an axial strain of 0.01.
- 7) The contribution of confined concrete to moment variation is substantial (about 30%).

F. Watanabe, H. Muguruma, T. Matsutami, and T. Sanda, 1987

A thirty-storey building was designed to find stresses and deformations in each member, induced by earthquake excitation. Based on the results of structural analysis, 1/3.8 scale model specimens of beams, columns, and beam-column connections were made to confirm the structural behaviour of members under seismic loading. Cyclic tests were carried out on these specimens. Each specimen was cast of HSC having an unconfined strength of 58 MPa . The test results obtained were discussed mainly in terms of strength and deformation capacity of each member.

The focus of this study was to investigate the benefits of using HSC in members of high-rise framed buildings. The following conclusions were reached:

- 1) For high-rise reinforced concrete buildings in seismic areas the use of HSC provides an effective alternative in obtaining the required strength and ductility of members, especially in lower stories.
- 2) The use of high-strength lateral steel as both shear and confinement reinforcement is desirable to avoid possible reduction in confinement efficiency that may result from yielding.
- 3) The efficiency of lateral reinforcement in beam-column joints is very small. Consequently, the use of HSC seems to be a desirable approach to meet the joint shear strength requirement.

R. L. Chen, and J. Lund, 1988

The authors presented an analytical study on the design of HSC bridge piers and columns. The mechanical properties of HSC were also briefly investigated with particular attention given to slenderness effects. Comparison between the use of HSC and NSC in bridge column design was presented based on detailed static and seismic analyses. A three-dimensional response spectrum analysis was carried out using the computer program SEISAB (Seismic Analysis of Bridges). Several other effects were also investigated, such as the impacts of HSC use on column vibration characteristics, tip displacements, shears and moments, and the economic considerations.

The main objective of this research was to study potential benefits of using HSC in the design of

bridge piers and columns. The results show that the application of HSC will reduce column size and stiffness. For columns on rigid foundations this translates to lower shears and moments. Cost estimates show that columns made of HSC and reinforced with minimum steel offer the most economical solution. Economy is also achieved because of the smaller foundations possible with a reduction in size of the columns. Savings as high as 33% in material cost are possible when HSC is used instead of NSC.

J. G. Smith, and F. N. Rad, 1989

The objective of this parametric study was to investigate the economic advantages of using HSC in columns of low-rise and medium-rise buildings. The parameters considered consisted of loading, structure geometry, concrete strength, and costs related to formwork, reinforcing steel, and concrete.

The results are summarized below.

- 1) An increase in concrete strength leads to a decrease in column size which in turn means more rentable floor space. Smaller HSC columns carry the same loads as larger NSC columns.
- 2) Since the construction formwork can be stripped sooner due to sufficiently high early strength of HSC, the cost of the project is reduced.
- 3) The benefits of using HSC in columns outweigh the added expense and effort needed to manufacture this material.
- 4) A significant reduction in reinforcing steel is achieved when using HSC, regardless of the gravity and lateral loadings on the structure.
- 5) The reduction in cost of column construction is in the order of 26% for 55 MPa concrete, and 42% for 83 MPa concrete.

H. Muguruma, and F. Watanabe, 1990

Eight columns, confined with Grade 328 MPa and 792 MPa lateral steel were tested under reversed cyclic lateral loads and constant axial compression. The axial compression corresponded to 25.4%, 40.0%, 42.3% and 62.3% of column concentric capacity. The compressive strength of the concrete used was 85.7 MPa and 115.8 MPa, respectively. The volumetric ratio for

transverse steel was 1.61% based on core area, and the reinforcement ratio for longitudinal steel was 3.61% based on gross concrete area, in all specimens. Each column comprised of a 200 mm x 200 mm cross-section and a 1500 mm length. A stub with dimensions of 200 mm x 300 mm x 500 mm was located at the column mid-height. The principal variables considered were the compressive strength of concrete, the axial load level, and the transverse steel yield strength. The test results obtained were discussed in terms of the effects of these variables upon the flexural ductility of columns. In addition, modifications to previously proposed stress-strain models were made based on the test results obtained, to extend their applicability to high strength concretes of up to 120 MPa.

The following conclusions were reported by the authors:

- 1) Very large ductility can be achieved by using high strength lateral confining reinforcement, even for HSC columns. For example, a column made with 115.8MPa concrete developed a deflection ductility factor of more than 8 when high grade steel was used as confinement reinforcement, even under a high axial compression of 42.3% of column concentric capacity.
- 2) The confinement efficiency of lateral steel is reduced if concrete strength is increased. Therefore, when utilizing concrete with $f'_c > 100 \text{ MPa}$ into reinforced concrete high rise buildings in seismic areas, the use of high yield strength confinement reinforcement becomes indispensable for providing the necessary flexural ductility.
- 3) Based on their test results, the researchers modified the Muguruma et al. model and the modified Kent and Park model for confined concrete. The modified models were then applied to HSC column curvatures calculations. The calculated moment-curvature relationships showed good agreement with those measured experimentally.

S. Sugano, T. Nagashima, T. Kimura, H. Tamura, and A. Ichikawa, 1990

The authors reported the results of tests on eight full scale columns subjected to earthquake type lateral loading. All specimens had a square 250 mm x 250 mm cross section, a 12-bar arrangement and confinement comprising of various hoop and cross-tie configurations. Three

concrete strengths were used, consisting of 39 MPa, 59 MPa and 78 MPa. Longitudinal reinforcement was kept constant at 2.5% of gross area and with yield strength of 588 MPa, in all specimens. Confinement steel had yield strengths of 834 MPa and 1376 MPa and was spaced at 35mm and 55mm, respectively. Two axial load levels; 30% and 55% of concentric capacity were used.

The main objectives of this program were to investigate the seismic behaviour of reinforced concrete members made of HSC, and to obtain empirical guidelines in their design for high-rise buildings. The results indicate that the combination of HSC and high strength steel confinement can be quite effective in improving the strength and ductility of columns. It was concluded that:

- 1) High strength steel is effective in confining columns with concrete strengths of up to 78 MPa.
- 2) Ductile behaviour of HSC columns can be ensured by increasing the steel strength in proportion to concrete strength. The strength of lateral reinforcement normalized by concrete strength, $(\rho_w f_{yh} / f_c')$ must be kept at 0.10 or greater, where ρ_w is the area ratio of lateral reinforcement.
- 3) To achieve 2% lateral drift in a column subjected to axial compression of 60% of concrete strength, the strength of lateral reinforcement normalized by concrete strength must be greater than 0.10.
- 4) Confined columns with concrete strengths of 59 MPa and 78 MPa, tested in this research program, exhibited lateral drift of in excess of 5% under an axial load level of 30% of column concentric capacity. However, companion columns with the same concrete strengths showed reduced ductility when tested under an increased axial load level of 55% of concentric capacity.
- 5) Displacement ductility increases proportionately with strength of lateral reinforcement. This trend was observed in all specimens.
- 6) Ultimate drift increased proportionately to the amount of lateral reinforcement, in all specimens.

A. Azizinamini, S. Baum Kuska, P. Brungardt, and E. Hatfield, 1994

Nine 2/3-scale column specimens were tested under a constant axial load and cyclic lateral loads

to study flexural ductility of square HSC columns. Each specimen had a cross section of $305\text{ mm} \times 305\text{ mm}$, a height of 2.44 m , and a $305\text{ mm} \times 305\text{ mm} \times 555\text{ mm}$ stub dividing the specimen into an upper and a lower partial column. The upper portion which served as the test section was longer than the lower portion to force hinging to occur in the test area. All specimens were designed according to the seismic provisions of the ACI 318-89 code. Reinforcement consisted of eight longitudinal bars and two configurations of transverse steel, consisting of either single peripheral hoops or hoops with cross ties. Compressive strength of concrete ranged from 26 MPa to 102 MPa . Nominal yield strength of transverse steel was 414 MPa in seven columns and 828 MPa in two columns. Spacing of ties in the critical region was 41 mm in some specimens, while the limit of 67 mm ($2/3$ of 4 in) prescribed by the code was used in others. Finally, the axial load was kept constant at 20% of P_o , except in two specimens where 30% and 40% of P_o were used. The parameters of interest were; concrete compressive strength, transverse reinforcement spacing, yield strength of transverse reinforcement, and the axial load level. The analytical part of the study included the development of a stress-strain model for confined HSC and a new expression for the equivalent rectangular compression block for use in sectional analysis. The confinement model was verified against the experimental data.

The ductility of HSC columns was investigated by evaluating the parameters mentioned above. The study also addressed the adequacy of the present ACI code provisions for confinement reinforcement. The following was reported as part of the conclusions.

- 1) HSC columns designed according to the seismic provisions of ACI 318-89, possess adequate curvature and displacement ductilities when subjected to axial load levels below 20% of the column concentric capacity.
- 2) Until further research is conducted, the stress intensity of the equivalent rectangular stress block can be reduced linearly to $0.6 f_c'$ for HSC from $0.85 f_c'$ used for 30 MPa .
- 3) The use of high strength transverse reinforcement in HSC columns results in larger spacing than recommended by ACI 318-89. This leads to early buckling of longitudinal bars. Therefore, for axial load levels below 20% of concentric capacity the yield strength of transverse reinforcement should be limited to 414 MPa .
- 4) The ductility of HSC columns decreases as the level of axial load increases.

- 5) The ductility of HSC columns increases as the volume of transverse reinforcement increases.
- 6) The confinement model proposed in the research program results in good agreement with experimentally obtained moment-curvature relationships.

S. Razyi, and M. Saatcioglu, 1994

The objective of this project was to evaluate the existing test data on confined HSC columns in terms of strength and deformability. About 250 column tests, recently conducted worldwide, were considered and analyzed in two groups. The first group included tests under concentric compression and the second group contained columns tested under combined axial force and lateral load reversals. Square, rectangular, and circular columns with different types and arrangements of confinement reinforcement were taken into account. It was noted that only a few of these columns were full size specimens reinforced both longitudinally and laterally and with compressive concrete strengths of 100 MPa or higher. For the columns under axial compression, the strength enhancement due to confinement was expressed as a ratio of confined to unconfined concrete strengths in the member. Deformability for these columns was evaluated in terms of strain ductility ratio. For the columns subjected to lateral deformation cycles, deformability was evaluated in terms of displacement ductility and drift ratios. The parameters considered included the effects of axial load level, shear stress, rate and direction of loading, loading history, the volumetric ratio, spacing, arrangement, and strength of confinement steel, section geometry and size, the amount and strength of longitudinal reinforcement, concrete strength, and the aggregate density. The following conclusions were drawn from the evaluation of test data:

- 1) Unconfined HSC shows extremely brittle behaviour.
- 2) Lateral confinement pressure required for HSC is much higher than for NSC. This can be satisfied by either increasing the volume of confinement steel or by using higher grades steel. There is evidence that columns with the same reinforcement arrangement exhibit similar deformabilities, regardless of concrete strength, if the ratio $\rho_s f_y / f_c'$ is maintained.
- 3) High strength steel of up to 1000 MPa is effective in confining HSC columns, especially under high axial load levels.
- 4) Deformability of HSC columns decreases with increasing level of axial load,

however that can be remedied in the same manner as mentioned in point 2.

- 5) Reduction in tie and laterally supported longitudinal reinforcement spacing improves the strength and deformability of HSC columns.
- 6) Although circular spirals are more effective than rectilinear ties, the confinement efficiency of rectilinear reinforcement can be increased to the same level as that for spirals with proper spacing and arrangement of tie reinforcement.
- 7) Unconfined concrete strength in column core varies between approximately 85% and 100% of cylinder strength.
- 8) The use of $0.85 f'_c$ in determining concentric capacity of HSC columns may result in overestimation of capacity because of the behaviour of cover concrete. Until these characteristics are well established, the concentric capacity calculations should be based on the core area instead of the gross cross-sectional area.

J. H. Thomsen IV, and J. W. Wallace, 1994

The authors reported the results of twelve 1/4 scale square HSC columns. The specimens were subjected to simulated seismic loadings. All specimens had a height of 19 inches (483 mm) and a cross-section of 4.5 x 4.5 inches (114 mm x 114 mm). The longitudinal bar arrangement was kept constant at 8 bars of 60 ksi (414 MPa) yield strength. The arrangement of transverse steel consisted of either perimeter hoops with two cross ties, or of perimeter hoops with diagonally placed interior hoops, effectively restraining every longitudinal bar. Two grades of tie steel, 115 ksi (794 MPa) and 185 ksi (1270 MPa), and four different tie spacings were used. Concrete strength ranged from 9,800 psi (68 MPa) to 14,900 psi (103 MPa). Three axial load levels were considered, consisting of 10%, 20%, and 30% of concentric capacity (P_o). The parameters of interest were the spacing, configuration, and yield strength of transverse steel, and the level of axial compression.

The authors reported stable hysteretic behaviour and ductility in all specimens up to drift levels of 2%. Also, the transverse steel arrangement as well as its yield strength did not affect the confinement effectiveness within this range of drift and the axial load levels considered. Buckling of longitudinal steel did not occur until drift levels of 4% (displacement ductility of 8) and higher were attained. Of importance was the conclusion that high strength transverse reinforcement

provided a more effective confinement after it yielded.

O. Bayrak, and S. A. Sheikh, 1995

Experimental research was carried out which included tests of seven large-size column specimens. The specimens were tested under axial load and cyclic lateral displacements simulating earthquake forces. Each specimen consisted of a $305\text{ mm} \times 305\text{ mm} \times 1473\text{ mm}$ column and a large stub at its base which represented a discontinuity such as a footing. Unconfined concrete strength varied between 30 MPa and 102 MPa , and the axial load level ranged from 36% to 63% of column concentric capacity. Other major variables included steel configuration and the amount of confinement steel.

The focus of this research was to evaluate the behaviour of HSC columns in relation to that of NSC columns, and to examine the results in relation to the confinement provisions of current design codes. A summary of results in terms of the parameters considered follows.

- 1) As in NSC columns, proper distribution of steel in the core, providing lateral support to longitudinal bars improves deformability and energy dissipation of columns significantly.
- 2) An increase in axial load reduces column ductility and accelerates stiffness degradation within every additional load cycle. Larger amount of confinement steel is needed for such columns.
- 3) Overall behaviour of HSC columns was observed to be only slightly less ductile than of NSC columns. However, beyond the peak, HSC columns showed much lower deformation and dissipation capacities which improved during later stages of load excursions. For columns to be comparable, amount of lateral steel should be proportional to the concrete strength and P/P_o should be equal.
- 4) Axial load level and steel distribution must be considered for effective confinement, and hence the code provisions must be modified to deal with these issues.

The test results of six large scale HSC columns are presented. The columns were subjected to constant axial loads and reversed cyclic flexural loading. The specimens had a square cross section of $305\text{ mm} \times 305\text{ mm}$ and a height of 2.15 m up to the assumed point of inflection. Each specimen was representative of a ground floor column in a typical building having a storey height of about 4.0 m . Axial load level and tie spacing were the two parameters studied. The specified strength of concrete was 100 MPa . Three axial load levels, consisting of 15%, 25%, and 40%, of concentric capacity were considered, representing a point below, at about, and above the balanced point, respectively. To obtain ductile behaviour, 60 mm spacing was used in three of the columns. The remaining columns had a spacing of 130 mm , chosen on the basis of minimum spacing for shear design. Finally, different stress-strain models, developed by others for confined concrete, were used to obtain moment-curvature relationship which were compared with test results.

It was concluded that the level of axial load influenced the ductility of columns considerably. The researchers emphasized the importance of considering the effects of axial load in building codes. Also it was conclude that the ACI and CSA codes were too conservative for HSC under the load levels considered in the test program.

1.3 Research Needs

It is evident from the review of previous research that experimental data on seismic behaviour of HSC columns is very limited. The brittle behaviour of HSC in relation to NSC is well established. However, lack of sufficient research addressing cyclic flexural behaviour of HSC columns explains the reluctance of structural engineers to use this material in seismic areas. Among the limited research on HSC columns reported in the literature, the majority is based on experiments involving columns tested under monotonically increasing concentric compression. In addition, many of these tests were conducted on scaled down specimens confined with only transverse steel, without any longitudinal reinforcement. Results of such tests were shown to be unreliable, due to scale effects [Martinez, *et al.*, 1984]. Many of the full scale tests were also limited to

monotonically increasing concentric compression. Very few full-size HSC columns were tested under simultaneous axial compression and lateral force reversals. In addition, data for columns tested under 20% to 60% of their concentric capacities is very scarce because of the high capacity testing system required. Furthermore, of particular importance to this project is the fact that there is virtually no information on the behaviour of spirally confined circular HSC columns subjected to combined axial compression and lateral force reversals.

Another important consideration is that the requirements for column confinement stated in current design codes, such as ACI 318-95 [1995] and CSA23.3-1994 [1994], were developed for concrete compressive strengths not exceeding 40 MPa. These requirements were shown by other researchers to be invalid for HSC and cannot be simply extrapolated to concretes of higher compressive strength. Also, the codes do not consider the level of axial load as a design variable. It was shown that axial load level has a very significant influence on column ductility [Saatcioglu, *et al.*, 1989, Azizinamini, *et al.*, 1992]. Currently there is no clear procedure formulated for design of HSC columns against seismic forces, leaving structural engineers in the dark.

Most of the previous research does indicate ductile behaviour in HSC columns under axial compression and flexure if they are confined with transverse steel. However, the amount, arrangement and spacing of confinement steel required for HSC columns have not been well established yet. Furthermore, many disagreements among researchers remain unaddressed. For instance, some researchers [Martinez, *et al.*, 1984] claim that the use of high-strength confinement steel serves few benefits since its strength may not be fully developed prior to the failure of concrete. Others [Muguruma, *et al.*, 1983, Watanabe, *et al.*, 1987, Muguruma, *et al.*, 1990, Sugano, *et al.*, 1990] claim that ductility of HSC columns confined by such steel is not only greatly enhanced but that high-strength steel is indispensable for providing the necessary flexural ductility. This lack of clear conclusions can only be ascertained by experimental investigation.

Clearly, further research is crucial in resolving issues such as those mentioned above. It is also imperative that studies pertaining to full scale HSC columns in seismic regions be conducted. In doing so, several main parameters that govern the confinement of HSC columns must be considered. These parameters, established by previous research, include the concrete strength, transverse steel strength, the spacing of transverse steel, axial load level, the amount of cover

concrete, and the configuration of transverse steel. The influence of these parameters on performance of HSC columns must be studied and quantified if design guidelines are to be developed. Furthermore, it is necessary to verify the applicability of existing confinement models, which were developed for HSC columns under concentric compression, in predicting column behaviour under combined axial and lateral load reversals.

1.4 Objective

The main objective of this research program is to evaluate significance of confinement parameters on performance of circular HSC columns subjected to lateral load reversals. The objective also includes generation of test data on full-scale HSC columns and design recommendations for confinement of HSC columns. The emphasis is placed on column strength and ductility as affected by the parameters of confinement.

1.5 Scope

The scope of the research program includes the steps needed to realize the objective. It consists of several stages of experimental and analytical work, the details of which are summarized below.

- Review of previous research and state-of-the-art on the behaviour of high strength concrete columns to determine the parameters considered relevant to the present study.
- Design of 9 full-scale circular HSC column specimens encompassing these parameters.
- Construction and instrumentation of the reinforcement cages of the test specimens.
- Construction of a new experimental set-up for testing of large-scale columns under simultaneous axial and lateral loads.
- Casting of test columns.
- Testing of all 9 columns using the test set-up in conjunction with three MTS hydraulic actuators and two data acquisition systems.

- Evaluation and interpretation of test data with respect to parameters studied.
- Theoretical analysis of each column, utilizing a confinement model for HSC, to obtain force-displacement characteristics.
- Comparison of analytical and experimental results.
- Formulation of design recommendations for confinement of circular HSC columns.
- Preparation of a thesis and presentation of results.

CHAPTER 2

Experimental Program

2.1 General

The experimental program involved construction and testing of nine circular columns under axial compression and lateral load reversals. The columns were designed to investigate the effects of selected confinement parameters on strength and ductility of HSC columns. The parameters considered included concrete strength as well as yield strength, volumetric ratio and spacing of transverse reinforcement, level of axial load, concrete cover, and the type of circular transverse reinforcement (hoop versus spiral). The details of test specimens, including material properties, instrumentation, test setup, and test procedure are described in this chapter.

2.2 Description of Test Specimens

Nine full scale columns were designed, built and tested as part of this investigation. The specimens were labelled RC-1 through RC-9, and represented part of a first storey column between the foundation and the inflection point. Each column had a *250 mm* diameter cross-section and *1365 mm* height. The shear span was *1645 mm* since the point of application of lateral force was located on a steel loading block *280 mm* above the column. This implies that each specimen represented a prototype column with an inter-storey height of *3.29 m*. The shear span and the cross-sectional dimension were chosen to represent the majority of columns used in practice with predominantly flexural mode of behaviour. Figure 2.1 illustrates the geometry of a typical test column.

High-strength concrete was used to cast all the columns with either a cylinder strength of *66 MPa*

or 90 MPa for the period in which the column tests were performed. These strength values represent a lower and a higher value within the range typically considered for high-strength concrete. All columns were confined with circular spirals, with the exception of column RC-8 which was confined with individual circular hoops. The longitudinal reinforcement in all specimens consisted of eight 16 mm diameter (*No. 15*) deformed bars, with a yield strength of 419 MPa. This resulted in a longitudinal reinforcement ratio of 3.26%. The bars were continued approximately 360 mm into the footing and were bent to have 90-degree hooks, with hook extensions of 180 mm, conforming to the requirements of CSA standard A23.3-M84. The volumetric ratio of confinement steel varied between 1.59% and 3.67%. The spacing of circular reinforcement was 50 mm in each specimen, except for column RC-6 which had a spacing of 100 mm. Grades 400 and 1000 MPa steel were used as transverse reinforcement. A clear concrete cover of 10 mm was provided in all columns except in RC-9, which had no cover. Figures 2.2 through to 2.4 illustrate the general details of reinforcement cages used in columns. A detailed summary of column properties is presented in Table 2.1.

Each column had a heavily reinforced concrete footing through which it was fixed to the laboratory strong floor for testing. The dimensions of the footing were 400 mm x 400 mm x 1100 mm. Four plastic (PVC) tubes with inside diameter of 38 mm were placed near the corners of the footing prior to concrete casting. These tubes provided the holes needed to connect the specimen to the setup foundation, details of which are discussed in subsequent sections. Furthermore, each column had four Grade 8, 19 mm diameter bolts cast in the concrete at the top of the column, protruding 50 mm to facilitate the attachment of the loading beam assembly.

The properties and the number of columns selected depended on the design parameters investigated. The columns were paired in such a way that only one of the test parameters was varied at a time, while the others remained constant. A matrix of the relationship between research parameters and column pairs is summarized in Table 2.2. Two pairs of columns were used to study the effect of concrete strength with two different grades of confinement reinforcement.

2.3 Description of Test Setup

The columns were tested under simultaneously applied axial compression and lateral displacement reversals. This type of simulated seismic loading necessitated the construction of a test-setup that allowed separate application of each type of load in both the vertical and horizontal directions. The setup consisted of a steel loading beam assembly, composite steel-concrete foundation, and three MTS hydraulic actuators. The following sub-sections describe the details of the test setup.

2.3.1 Hydraulic Actuators

Each MTS actuator had a load capacity of *1000 kN* in tension and compression, with a maximum stroke of *500 mm*. This allowed the application of lateral loading over a maximum displacement of $\pm 250 \text{ mm}$ relative to the neutral position. Each actuator was equipped with two multi-directional swivels, one at each end. The swivels facilitated rotational displacements at actuator ends which prevented potential damage to actuators that might be caused by the eccentricity of force. The hydraulic pressure for all actuators was supplied by a *33 GPM* gear driven pump. The pressure was controlled by an MTS servo-valve which allowed the application of desired load by each actuator. Figure 2.5 shows the overall geometry of a typical actuator.

2.3.2 Horizontal Loading Mechanism

Horizontal deformation reversals were applied with a single MTS actuator supported by a pair of steel A-frames. A *50 mm* thick steel plate, bolted to the frames, was used to attach the actuator. The other end of the actuator was bolted to the loading beam that had been attached to a specimen at the top. The details of the loading beam are described in the next section. The A-frames were bolted to three pairs of C-channels which were placed back-to-back and secured to the laboratory strong floor by means of *1.8 m* long and *64 mm* diameter bolts. The bolts had yield strength of *400 MPa*. The maximum force applied by this actuator was no more than *200 kN*, which was significantly below the capacity of the support mechanism. Both ends of the actuator were connected using Grade 8 high-strength bolts, *38 mm* in diameter and *400 mm* in length. The details of the horizontal load setup are shown in Figures 2.6 and 2.7.

2.3.3 Vertical Loading Mechanism

The vertical loading mechanism consisted of a steel loading beam assembly, a composite foundation, and two MTS actuators. The actuators were placed on either side of the specimen vertically for the purpose of applying constant axial compression throughout the tests. The top ends were connected to the loading beam assembly and at the bottom ends were connected to the setup foundation. The foundation was bolted to the laboratory strong floor. Figure 2.8 illustrates the front view of the vertical load mechanism.

The loading beam assembly was made up of two parts. The upper part was a built-up welded box section beam. Four holes were drilled on either side of the bottom flange of this beam to allow connection to the vertical actuators by means of 38 mm diameter 400 mm long Grade 8 bolts. The bottom part was a built-up I-section stiffened in the direction of the horizontal load by means of three steel plates. This part, referred to as a spacer block, served a dual purpose of accommodating the application of horizontal force as well as providing additional vertical room for full height of vertical actuators between the loading beam and the foundation. The two parts were connected together with eight 25 mm diameter 100 mm long Grade 8 bolts. The loading beam assembly was manufactured by welding Grade 300, 44W steel plates. Four holes were drilled in the bottom flange of the spacer block to accommodate the bolts that had been cast in the test column, thereby allowing connection to the column. Details of the loading beam assembly are illustrated in Figure 2.9.

The setup foundation was constructed to have an H-shape, and consisted of a steel-concrete composite structure. It included four C-channels welded to a large bottom steel plate and a cast-in-place reinforced concrete portion above. The reinforced concrete portion provided support for the attachment of the specimen footing while increasing the overall stiffness of the foundation. The bottom steel plate, with a thickness of 75 mm and a yield strength of 280 MPa, served as the base of the foundation and facilitated the connection of vertical actuators. A total of sixteen holes were drilled in the base plate to fulfil three tasks. Eight countersunk holes were used to connect the actuators, four for each. Four larger holes were also provided near the corners of the foundation to connect it to the laboratory strong floor. These holes, with a diameter of 75 mm, travelled through the plate and the cast-in-place reinforced concrete portion. The bolts used for

the connection of foundation had *64 mm* diameter and *1800 mm* length and were made of Grade 400 steel. Finally, four holes were drilled in the central area of the foundation, congruent to the holes of the specimen footings. Four threaded bars were inserted from underneath the bottom plate, passing through these holes and protruding out of the foundation. These bars, which had a diameter of *32 mm* and a yield strength of *800 MPa*, were used for connecting the specimen to the foundation. Figure 2.10 shows the details of the composite foundation. A drawing illustrating the assembly of the loading beam, a test specimen, and the foundation is presented in Figure 2.11.

2.3.4 Lateral Restraint Frames

To provide lateral bracing as well as additional safety against unexpected out-of-plane failure of columns at high inelastic deformations, a pair of steel frames was positioned on either side of the vertical loading setup. The frames were made out of hollow steel sections and were secured to the laboratory strong floor. They were also connected to each other at the top by two hollow section steel beams. For the purpose of clarity, these frames are not shown in the drawings presented in figures, however they are visible in the photographs of Figure 2.12.

2.4 Material Properties

2.4.1 Concrete

Two different mix designs were used for four batches of concrete. The main objective of the mix designs was to obtain the required concrete strength with good workability. It was necessary to use a low water-cement ratio and a high cement content to achieve the desired strength level. However, the use of a superplasticizer was essential to ensure the required workability. Normal Portland Cement with silica fume was used for all batches. Local aggregates were used, with the coarse aggregate being *10 mm* crushed limestone. The concrete compressive strengths attained were *66 MPa* (RC-1 and RC-2) and *90 MPa* (RC-3 to RC-9) at time of specimen testing. Table 2.3 provides the material quantities of the two mix designs used in casting the specimens.

More than 25 standard cylinders were cast to determine cylinder strength of concrete. Standard cylinder tests were performed at predefined time intervals to determine strength gain with time. Several cylinders were also tested at the time of column testing to obtain the stress-strain characteristics of concrete. The cylinders were ground smooth at both ends prior to testing to ensure uniform application of load. Table 2.4 illustrates the strength development of each of the column batches with respect to time (Batches #2 and #4).

The stress-strain relationships for both concrete strengths, obtained by standard cylinder tests, are shown in Figures 2.13 and 2.14. The average of at least three cylinders per batch were used to define these relationships. The strain data was collected by using standard strain gauges and a data acquisition system. The load was applied and monitored on a Forney testing machine with a 2200 kN capacity. This machine had no deformation control capability, and hence, it was not possible to obtain the descending branches of the curves. Figure 2.15 depicts the instrumentation and testing of a standard cylinder.

2.4.2 Steel

The longitudinal reinforcement in all specimens consisted of *16 mm* diameter (*No. 15*) deformed bars with a yield strength of *419 MPa*. Three different grades and sizes of reinforcing steel were used as lateral reinforcement. These were; *11.3 mm* diameter (*No. 10*) deformed bars, *8.0 mm* plain reinforcement and *7.5 mm* diameter plain reinforcement, with yield strengths of *420 MPa*, *580 MPa*, and *1000 MPa*, respectively. The deformed bars displayed distinct yield plateaus. However, the plain reinforcement did not. It was therefore necessary to use the 0.2% offset method to establish the yield strengths of plain reinforcement. Stress-strain relationships for all steel types, established by coupon tests, are shown in Figs. 2.16 to 2.19. Each relationship represents the average result of at least three tests.

2.5 Construction of Test Specimens

The construction of test specimens was done in four phases; I) the assembly of steel reinforcement cages, ii) installation of steel strain gauges, iii) construction of formwork, and iv)

casting of specimens. All four phases of construction were conducted at the structures laboratory of the University of Ottawa. The reinforcing steel, with the exception of high-strength reinforcement, was ordered from a local steel supplier. The plain high strength steel was already available at the University, however, it needed to be shaped into spirals. This difficult task was accomplished by a local precast concrete company.

Steel strain gauges for longitudinal reinforcement were installed at predetermined locations prior to cage assembly. The details of strain gauge locations are described in Section 2.6, under "Instrumentation". The assembly of a typical reinforcement cage was executed by first tying the longitudinal bars to the inside diameter of the spirals, ensuring proper and uniform spacing. This cage was suspended above the floor and positioned between two light steel frames. The footing reinforcement cage was then built around and through the column cage. The steel frames served as adjustable supports for the footing reinforcement while it was being tied together. The strain gauges were then applied to the spiral reinforcement and the necessary wiring was soldered to the gauge terminals. The construction process of a typical specimen cage can be seen in Figures 2.20 through to 2.22.

The footing formwork was prepared in the laboratory using plywood. Formwork for column portions consisted of 250 mm diameter sona tubes. Braced frames were also constructed from wood and connected to the footing formwork. The purpose of these frames was to provide a means of effective levelling and stabilizing of the sona tubes and column reinforcement during casting. The formwork was held together by a combination of screws at the joints, and threaded rods and nuts through the middle portions of the siding. This prevented warping and bulging of the formwork due to lateral concrete pressure. The steel cages were placed in the footing formwork with the aid of the laboratory crane before the bracing frames and sona tubes were built. Figures 2.23 and 2.24, display the formwork and placement of reinforcement cages.

A total of four batches of concrete were prepared for casting. This was necessary since two different strengths of concrete were needed and the footing and column portions of the specimens had to be cast separately. Batches No. 1 and No. 2, prepared at the laboratory, were used to cast the footings and the columns, respectively, in the 66 MPa specimens (RC-1 and RC-2). Because of the large amount of concrete needed to cast the remaining 90 MPa specimens

(RC-3 to RC-9), Batches No.3 and No. 4 had to be prepared and delivered to the laboratory by a local ready mix company. After providing the ready mix plant with a mix design, the materials were proportioned at the plant and delivered to the laboratory in a ready-mix truck. The moisture content of the fine and coarse aggregate was reported by the ready-mix company to account for the extra water already present. The moisture in the drum of the concrete truck was also estimated prior to adding water. Additional superplasticizer was added at the laboratory to ensure good workability of the concrete. The slump of all batches was approximately *180 mm*. A team of people were available to aid in casting before hardening of the concrete could occur. Concrete was cast in vertical layers and vibrated thoroughly. Figures 2.25 and 2.26 illustrate the casting of specimens. Moist curing of the finished specimens continued for 28 days from the time of casting. This was achieved by applying burlap cloth over the specimens and saturating it with water on a daily basis. Plastic sheets were used to cover the moist burlap and limit the amount of evaporation. Figure 2.27 shows the specimens during curing. The cured specimens were eventually painted white in a flat finish to promote visibility of cracks during testing. The location of longitudinal and transverse reinforcement was also drawn onto the white surface of each column. Completed and painted specimens are presented in Figure 2.28.

2.6 Instrumentation

Each specimen was instrumented to measure lateral column displacements, rotations of the hinging region, strains in both the longitudinal and transverse reinforcement, as well as the applied horizontal and vertical loads. Data was recorded by two data acquisition systems. These consisted of a Sciometric System 200 Data Acquisition System connected to a micro computer, and an MTS controller connected to a second micro computer. These data recording devices are illustrated in Figure 2.29.

Linear variable differential transducers (LVDT's) were used to measure lateral column displacements as well as rotations of the plastic hinge region at the base of the column. A total of eight LVDT's of three different types were used for this task. Horizontal column displacements at the point of application of horizontal force (inflection point) was measured with a single MTS tempo sonic LVDT. This LVDT was connected directly to the MTS controller data acquisition

system. It had a usable stroke of $\pm 250 \text{ mm}$ with the output given directly in a choice of units. Three additional LVDT's with a stroke of 200 mm were used for measurements of lateral displacements at different heights, two at the tip of the column (LVDT's 5 and 6) and one at the top of the plastic hinge region (LVDT 7). These LVDT's were mounted with brackets to a light steel mecano frame which was in turn mounted to the footing of the specimens so that all readings of displacements were relative to the column footing. These three LVDT's had to be calibrated before every test and gave readings in millivolts.

The rotations in the hinging region were measured by four LVDT's with a 50 mm stroke, mounted vertically, two on each side of the column face. These LVDT's were secured with brackets on threaded rods that were cast in column core concrete so that continued readings could be taken after the spalling of cover concrete. Two of the four LVDT's (1 and 4) were placed at the top of the theoretical plastic hinge region, one on each side, at a height equal to depth of the cross section, or 250 mm above the surface of the footing. They were each positioned at approximately 50 mm away from the face of the column, so that the horizontal distance between the vertical LVDT's was equal to 350 mm (column width + $2 \times 50 \text{ mm}$). The difference between the vertical readings divided by the horizontal distance between the two LVDT's corresponded to total rotation of the hinging region. Total rotation consisted of rotations due to both flexure and anchorage slip. The other two LVDT's (2 and 3) were placed in the same manner, however, at a distance of 25 mm above the surface of the footing and each at 100 mm away from the face of the column. Their purpose was to provide values of anchorage slip. It was noted however that since it was not possible to mount them at the actual interface of the column and footing, these readings also included some flexural rotations within the gauge length. Figures 2.30 and 2.31 show the configuration details of all three types of LVDT's on a typical specimen. These sensors can also be seen in the photographs of Figure 2.32.

Electrical resistance strain gauges were placed on lateral and longitudinal reinforcement to measure strains in steel. Each specimen was instrumented with four strain gauges on longitudinal reinforcement. Three of these strain gauges were attached to one of the extreme longitudinal bars, one at a location coinciding with the column-footing interface, one at 125 mm ($h/2$) above, and another at 125 mm ($h/2$) below the footing surface. The fourth strain gauge was affixed to the opposite extreme longitudinal bar at the column footing interface so that strains of the opposite

sign could be observed. The data obtained from these four strain gauges was used to obtain a profile of the strains in longitudinal reinforcement along its length and to observe yield penetration into the footing anchorage region. Figure 2.33 illustrates the locations of strain gauges on longitudinal reinforcement.

Columns with spiral reinforcement had five strain gauges attached to the spirals beginning at half the spacing of transverse steel ($s/2$) above the footing surface, and continued at these intervals up to a height of one half of the depth of the column cross-section ($h/2$), or 125 mm . Column RC-8, which was confined with separate circular hoops, had only three strain gauges beginning at the same distance above the footing and continued at the spacing intervals of the next two hoops (s). Together with the longitudinal steel strain gauges, this specimen had a total of seven gauges. All other specimens had a total of nine strain gauges. Arrangement of strain gauges on transverse reinforcement is shown in Figures 2.34 through 2.36.

2.7 Test Procedure and Loading Program

Each specimen was first positioned in the test setup over the middle portion of the setup foundation with the aid of the laboratory crane. The four holes running through the column footing were then lined up with the four high strength threaded rods protruding from the foundation. Next, the specimen was lowered onto the foundation and bolted into place. The loading beam and spacer block assembly was then affixed to the top of the specimen. The horizontal actuator was positioned and connected to the web of the spacer block. This was followed by the connection of the two vertical actuators to the bottom flange of the loading beam. Finally, the LVDT's and strain gauges were calibrated and initialized for the collection of data.

The actual test begun with the application of axial load up to the desired level. Three axial load levels were considered, namely, 0.22, 0.30, and 0.43 of the concentric column capacity, P_o . Specimens RC-1 and RC-2 were tested under an axial load of 1000 kN , RC-3 through to RC-6, RC-8, and RC-9 were tested under 1850 kN , and RC-7 under 925 kN . These axial loads corresponded to the axial load levels listed in Table 2.1. Lateral loading was applied in displacement control mode, i.e., the horizontal actuator applied a specified level of displacement

to the column and the resulting load was monitored. The lateral displacement history shown in Fig. 2.37 was followed throughout testing. Each level of displacement was expressed as an increment of yield displacement Δ/Δ_y . Three initial cycles were applied in the elastic range, followed by three complete cycles at each of the superseding displacement increments corresponding to Δ_y , $2\Delta_y$, $3\Delta_y$, etc. The value of yield displacement, Δ_y , for each column was determined visually during testing as the displacement at which a significant drop occurred in the slope of the force-displacement relationship, signifying yielding of the column as a whole. The rate of lateral loading was very low, with each cycle taking approximately 5 to 10 minutes. Total duration of a typical test was about three hours depending on the ductility of the given specimen. Testing was considered finished when the lateral load resistance of a column dropped by at least 30%.

TABLE 2.1 - Properties of Test Specimens

Column	Compressive Strength of Concrete f'_c (MPa)	Diameter of Transverse Steel d_b (mm)	Spacing of Transverse Steel s (mm)	Volumetric Ratio of Transverse Steel ρ_t (%)	Yield Strength of Transverse Steel f_{yt} (MPa)	$\rho_s f_{yt} / f'_c$	Level of Axial Load P/P_o
RC-1	65	7.5	50	1.59	1000	24.5	0.30
RC-2	65	11.3	50	3.67	420	23.7	0.30
RC-3	90	7.5	50	1.59	1000	17.7	0.43
RC-4	90	8.0	50	1.81	580	8.5	0.43
RC-5	90	11.3	50	3.67	420	17.1	0.43
RC-6	90	2(11.3)	100	3.67	420	17.1	0.43
RC-7	90	7.5	50	1.59	1000	17.7	0.22
RC-8	90	7.5	50	1.59	1000	17.7	0.43
RC-9	90	11.3	50	3.36	420	15.7	0.43

Note: - Longitudinal Reinforcement in all specimens consists of 8 No. 15 bars giving a reinforcement ratio of $\rho_f = 3.26\%$.

- Column RC-8 is confined with hoops

- Column RC-9 has no effective concrete cover

TABLE 2.2 - Summary of Research Parameters and Comparison Specimen Pairs

Parameter Studied	Comparison Specimens
Concrete Compressive Strength - f'_c	(RC-1 and RC-3) & (RC-2 and RC-5)
Yield Strength of Transverse Steel - f_{yt}	RC-3 and RC-4
Volumetric Ratio of Transverse Steel - ρ_t	RC-4 and RC-5
Spacing of Transverse Steel - s	RC-5 and RC-6
Level of Axial Load - P/P_o	RC-3 and RC-7
Type of Transverse Reinforcement	RC-3 and RC-8
Cover Concrete	RC-5 and RC-9

TABLE 2.3 - Concrete Mix Designs

Batch	Coarse Aggregate (kg)	Fine Aggregate (kg)	Cement (10% SF) (kg)	Water (kg)	Super-plasticizer (L)	Retarder (L)	Water - Cement Ratio
No.1 and No. 2	1130	726	550	160	12	0	0.291
No.3 and No. 4	1122	726	555	114	20	1.6	0.205

TABLE 2.4 - Development of Concrete Strength with Time

Time (Days)	Batch No. 2 (MPa)	Batch No. 4 (MPa)
1	46	40
3	53	54
7	58	60
14	62	72
21	64	----
28	65	84
Test Period	66	90

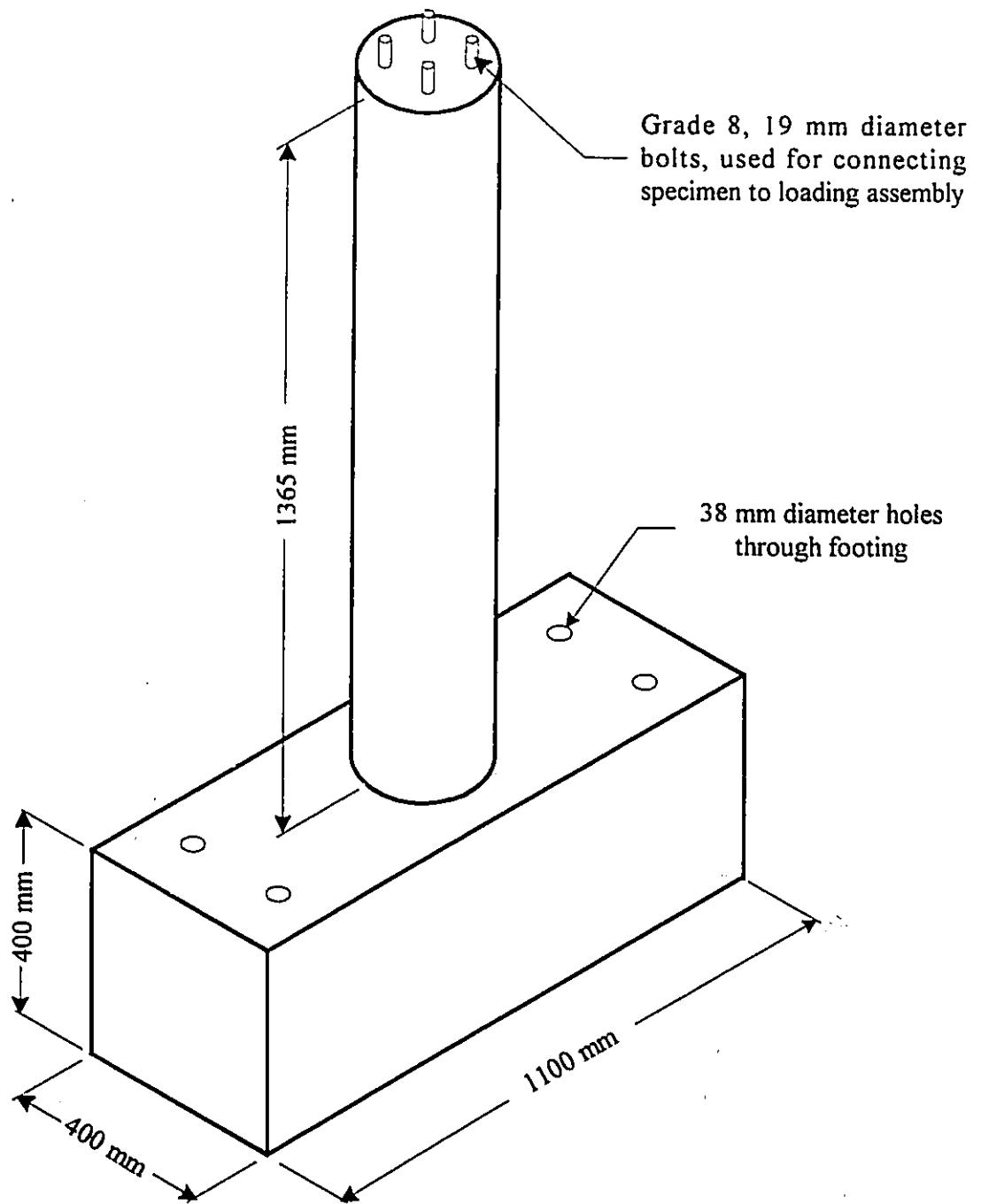


Figure 2.1: Geometry of a Typical Specimen

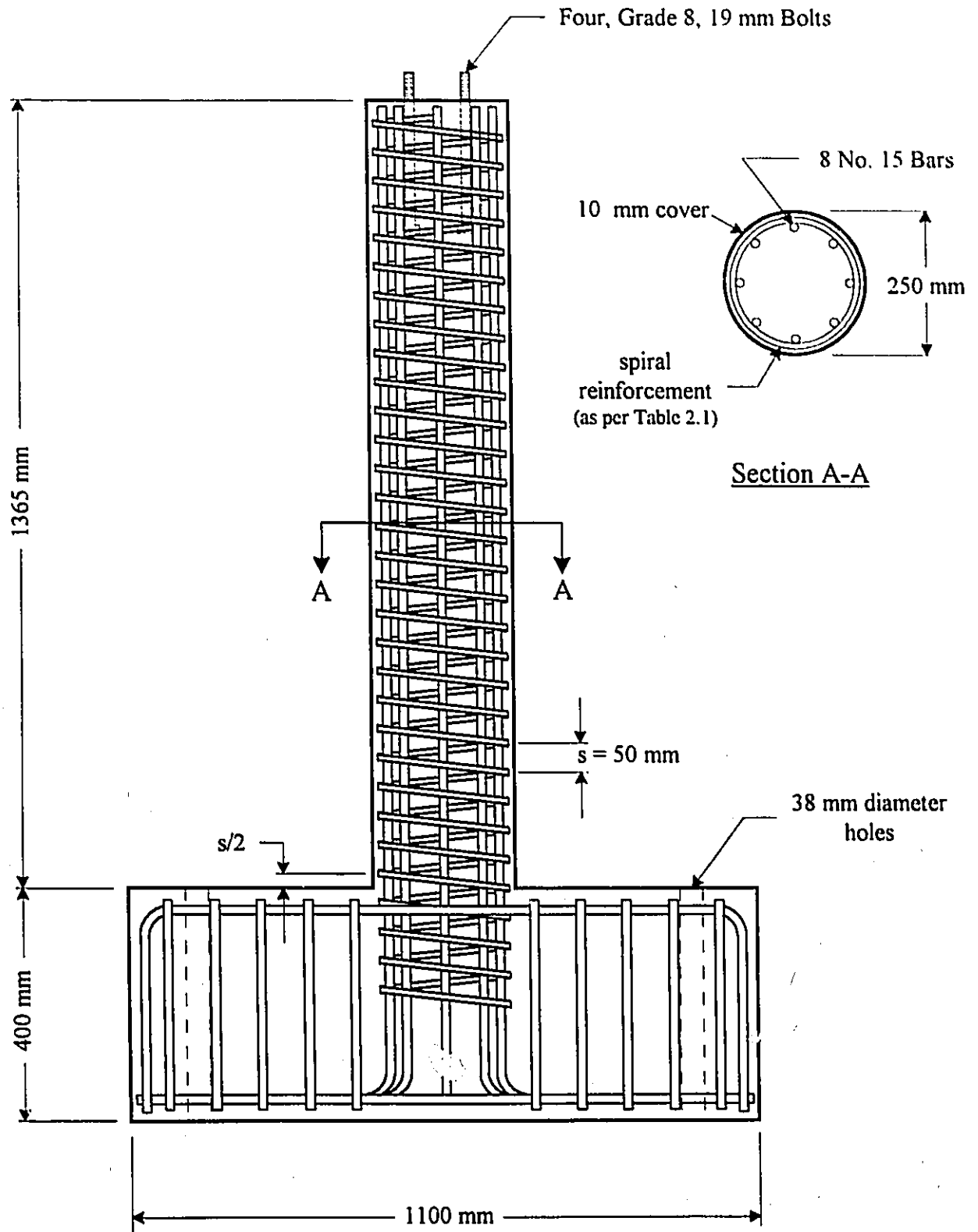


Figure 2.2: Reinforcement Details of Specimens RC-1 to RC-5, RC-7 and RC-9

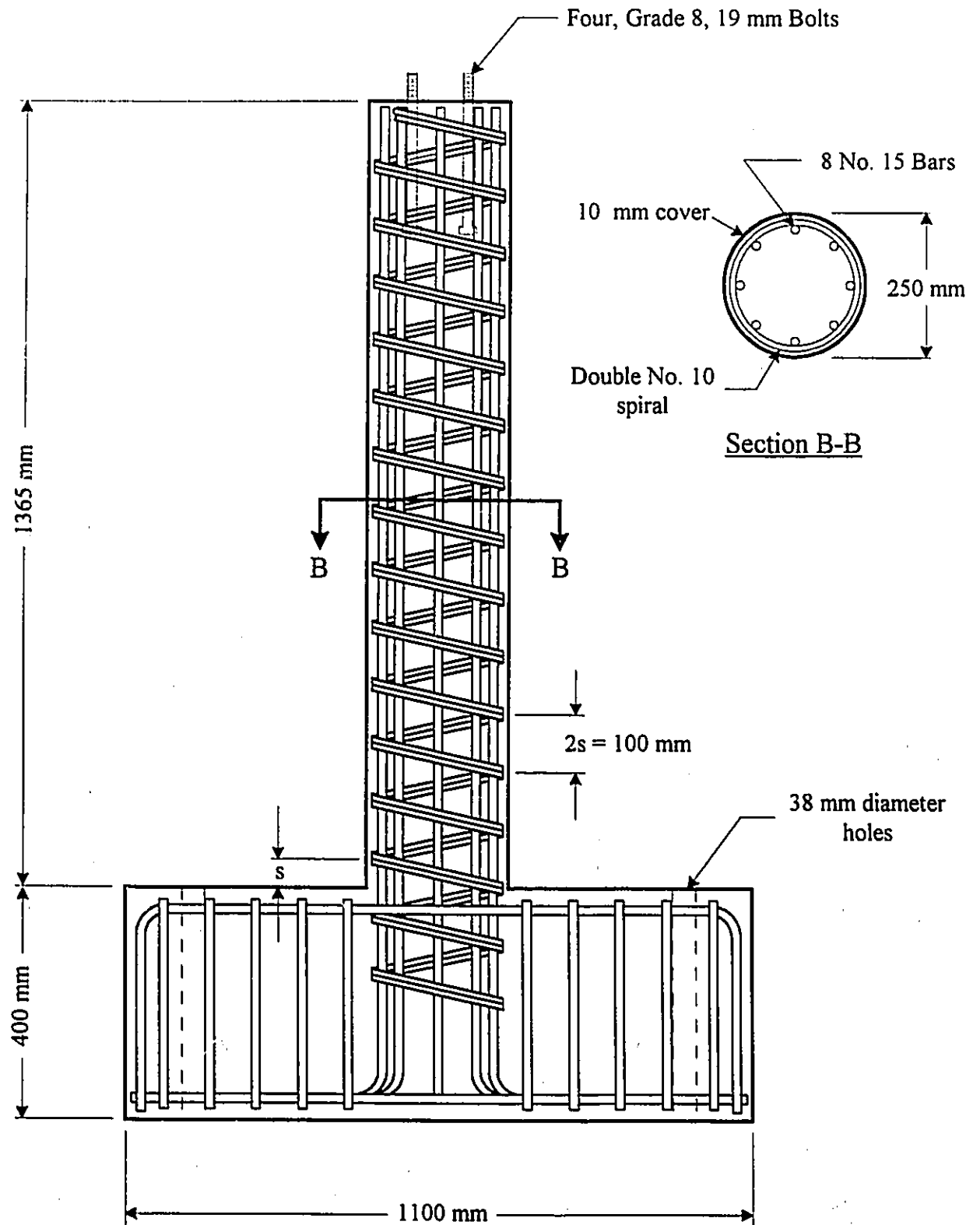


Figure 2.3: Reinforcement Details of Specimen RC-6

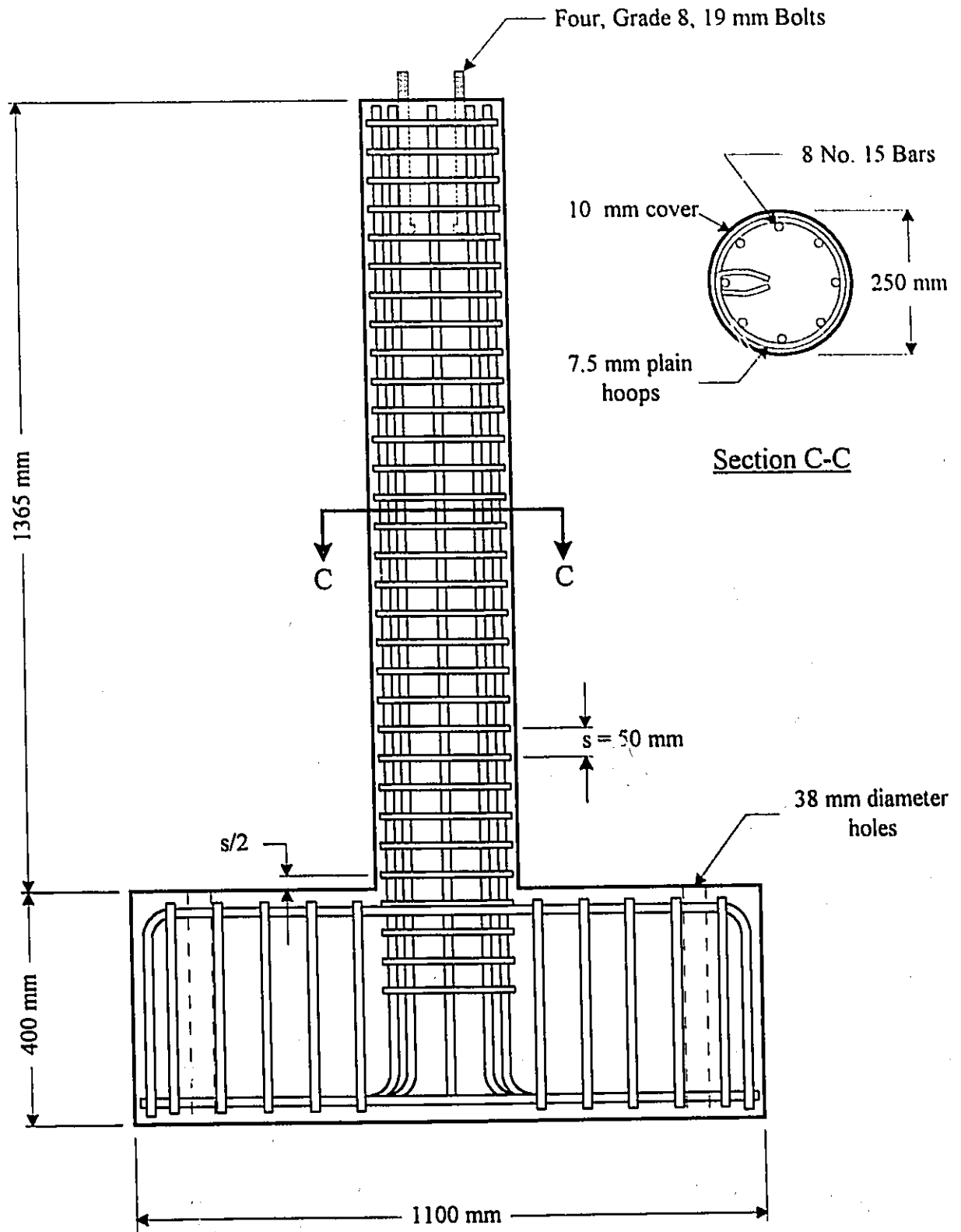


Figure 2.4: Reinforcement Details of Specimen RC-8

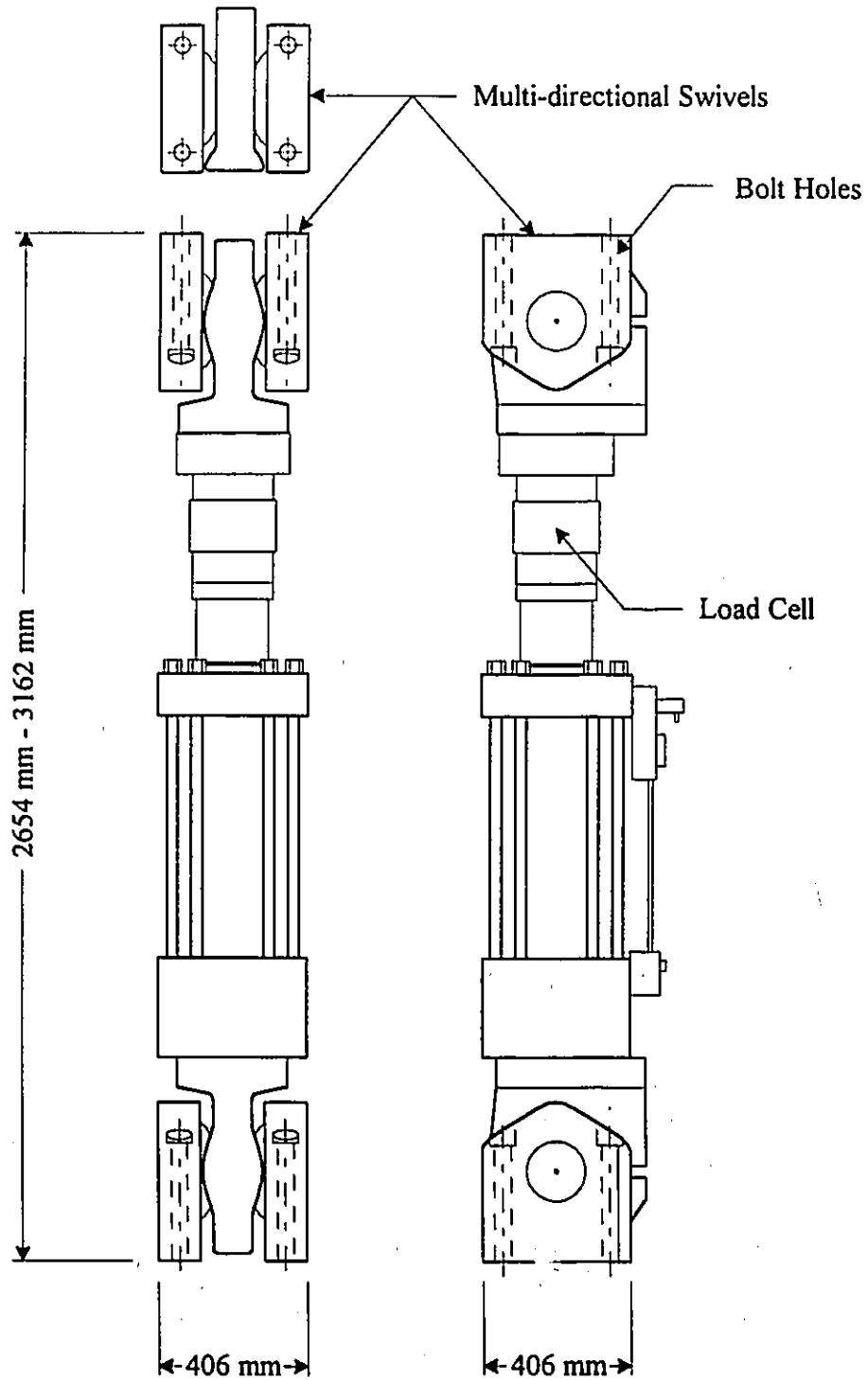


Figure 2.5: Details of the MTS Actuators..

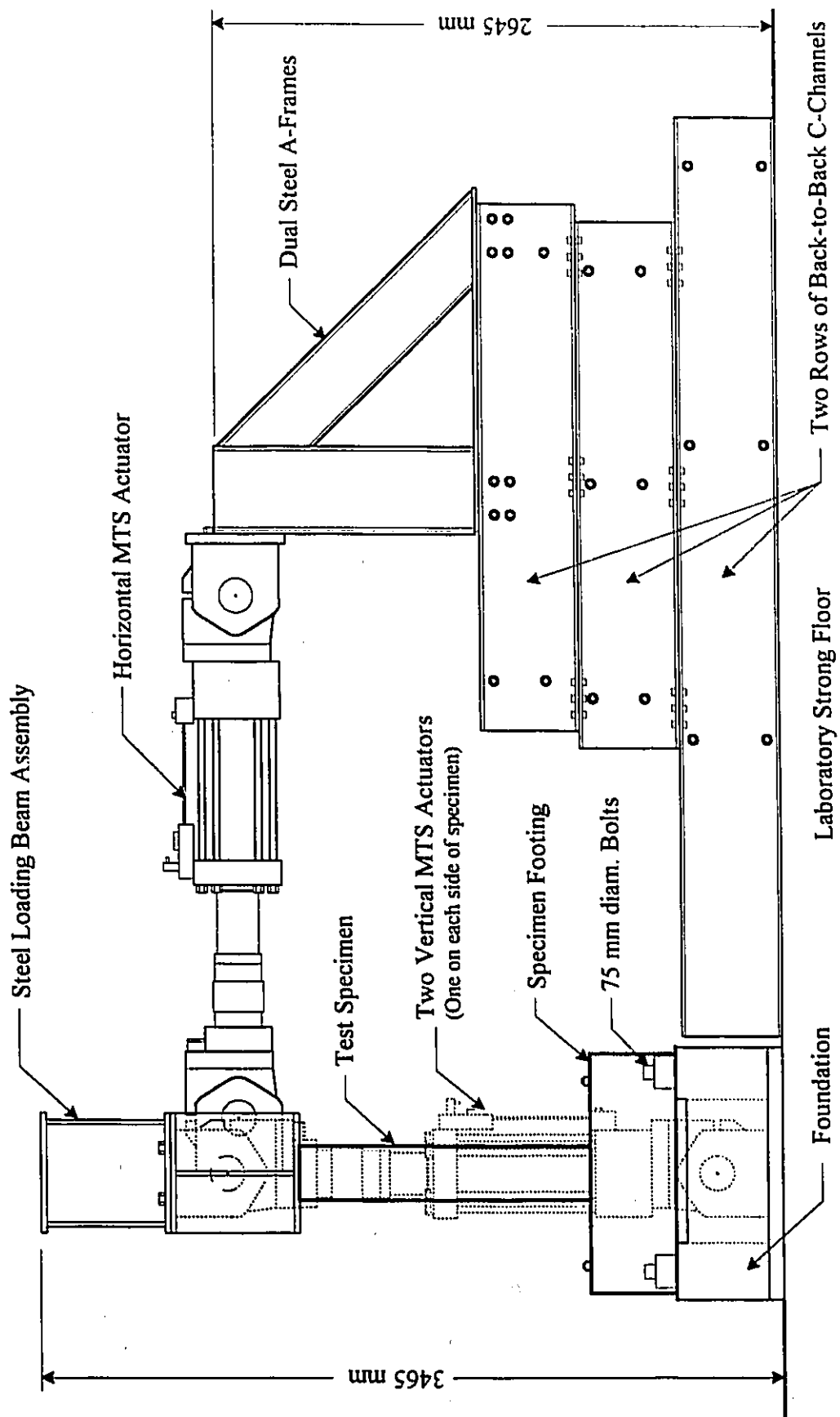


Figure 2.6: Side Elevation of the Test Setup

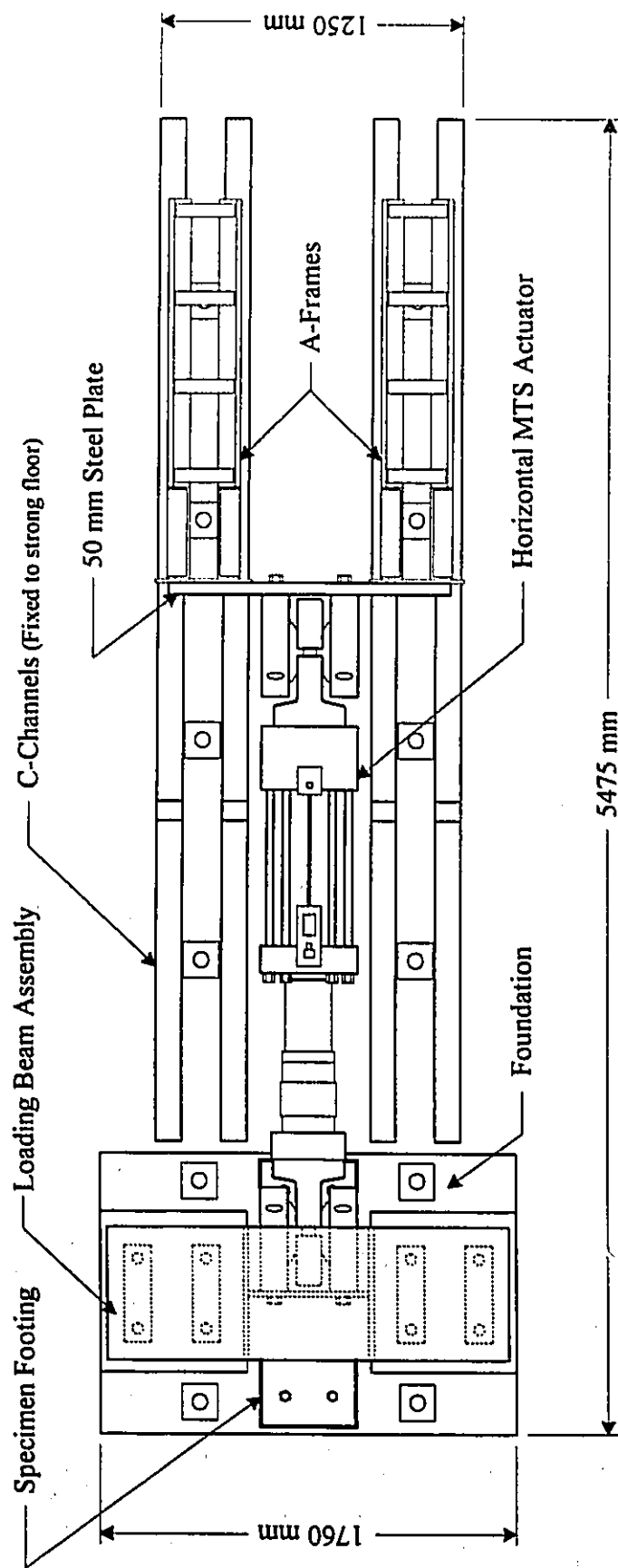


Figure 2.7: Plan View of Test Setup

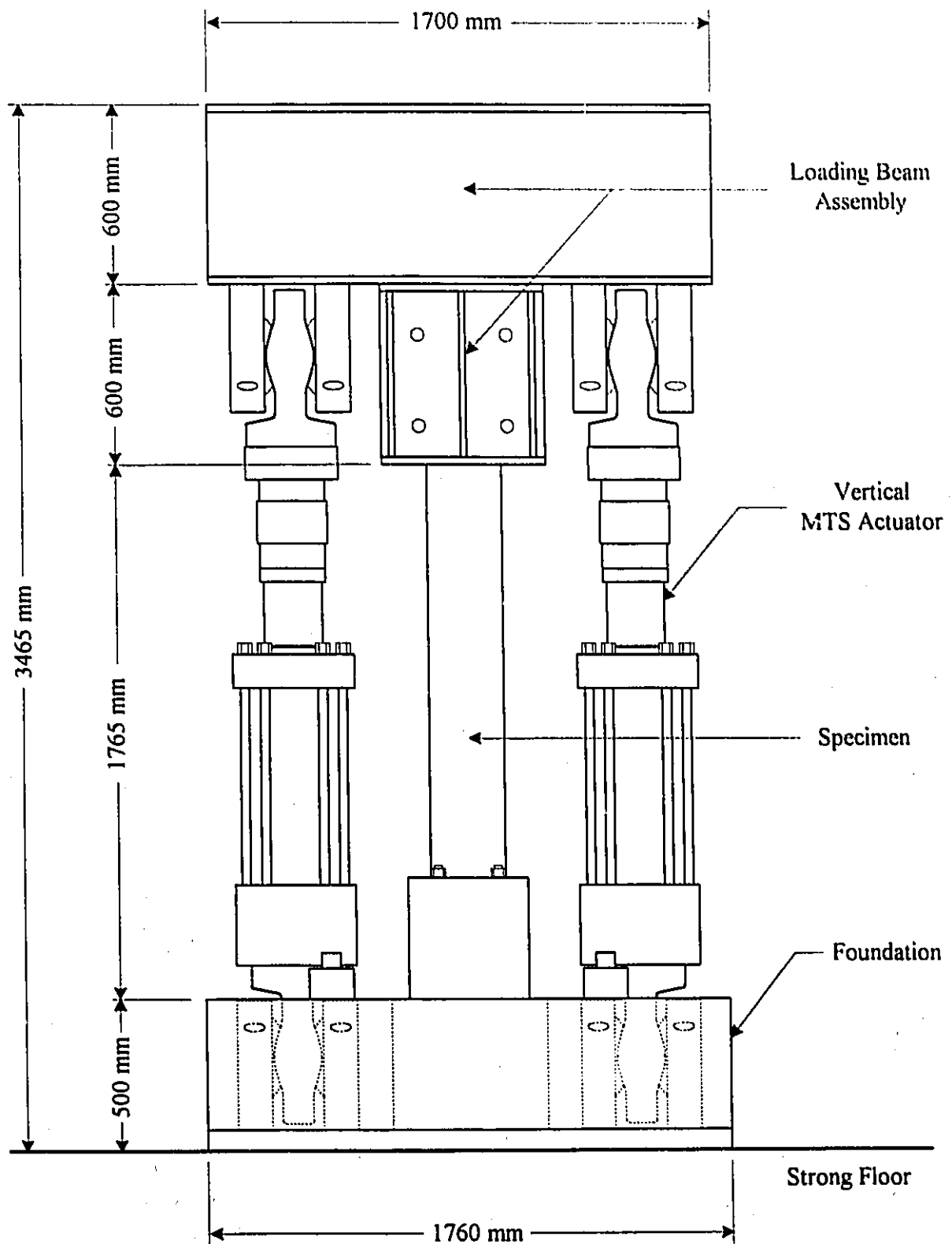


Figure 2.8: Front Elevation of the Test Setup

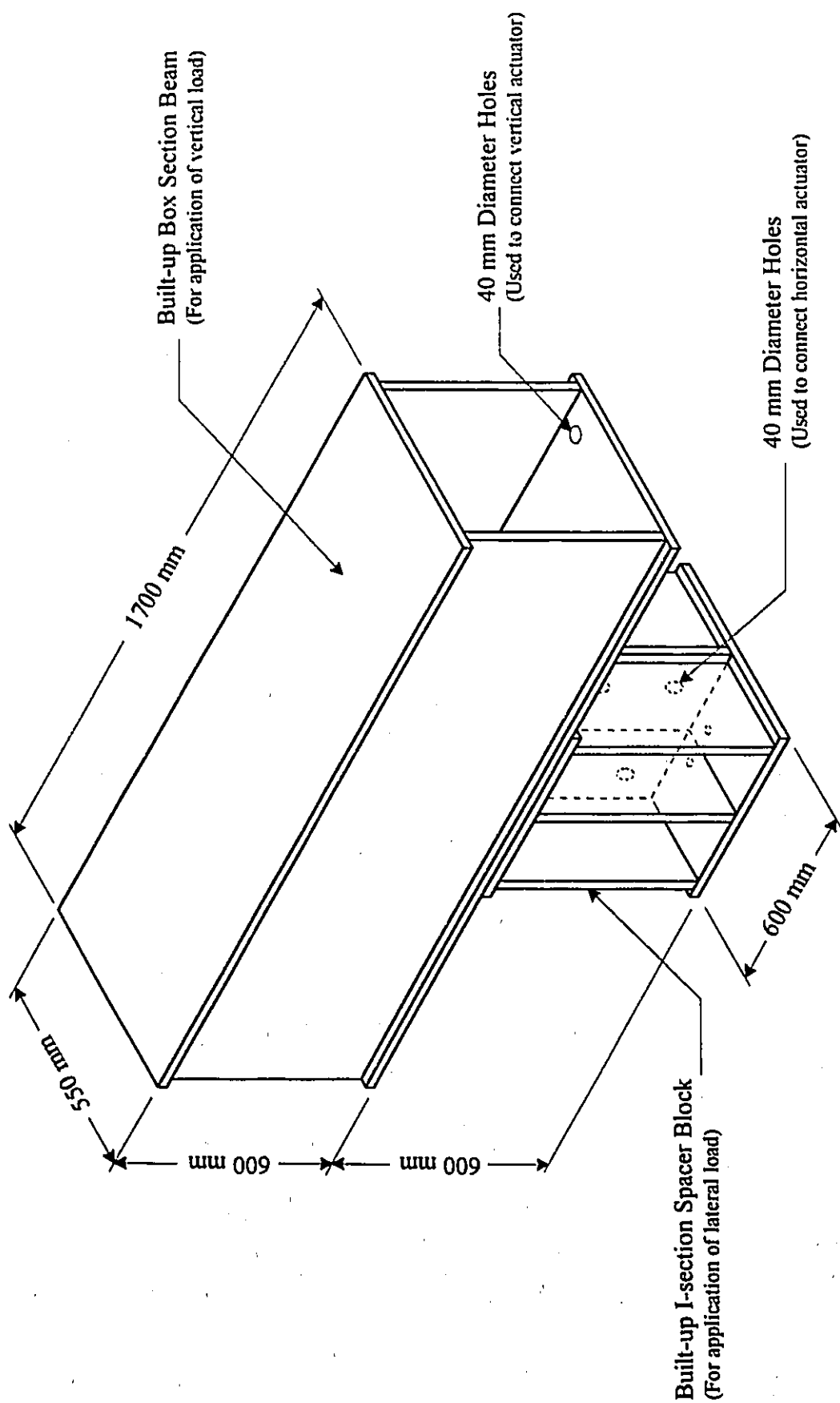


Figure 2.9: Details of Loading Beam Assembly

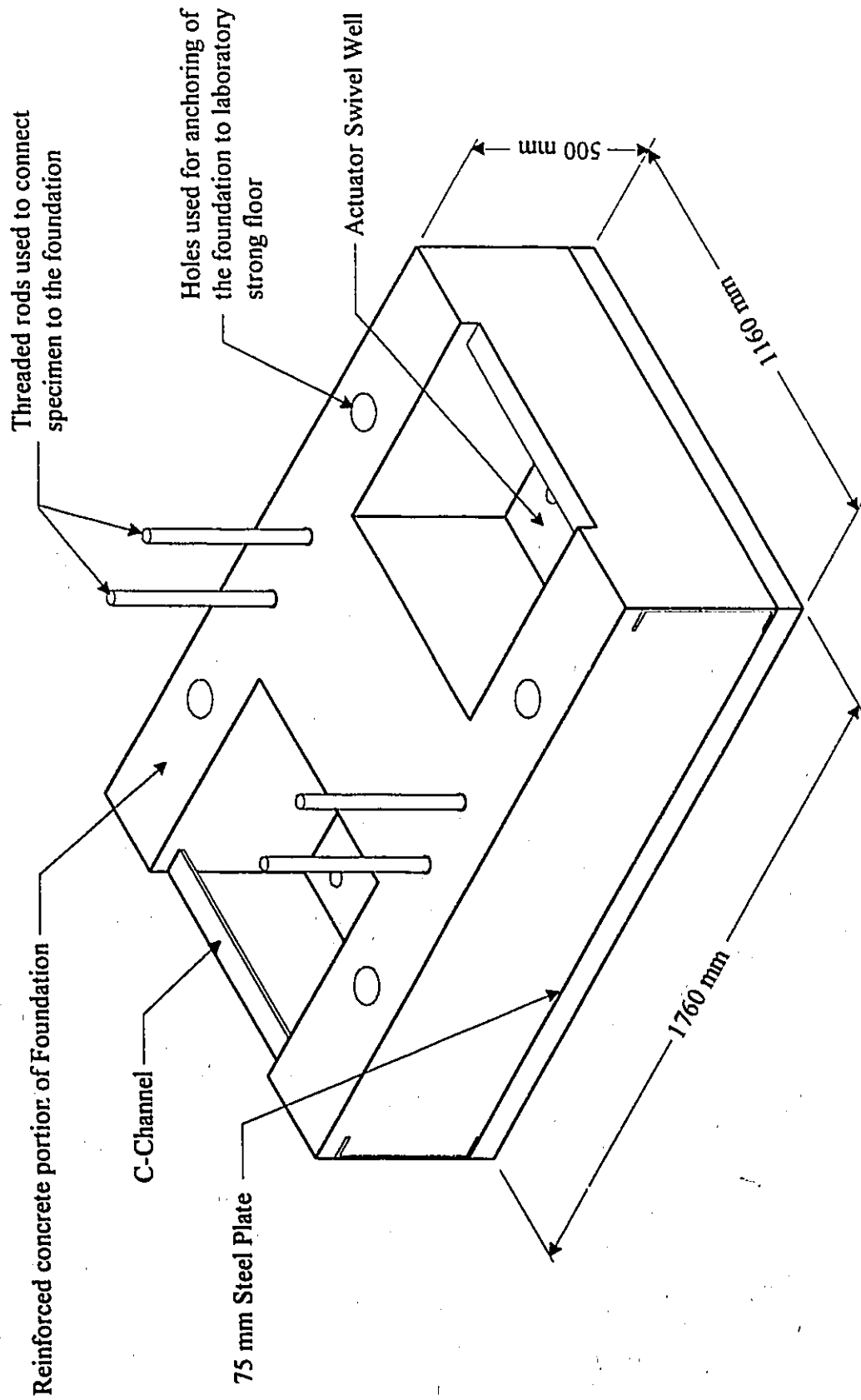


Figure 2.10: Details of Composite Foundation

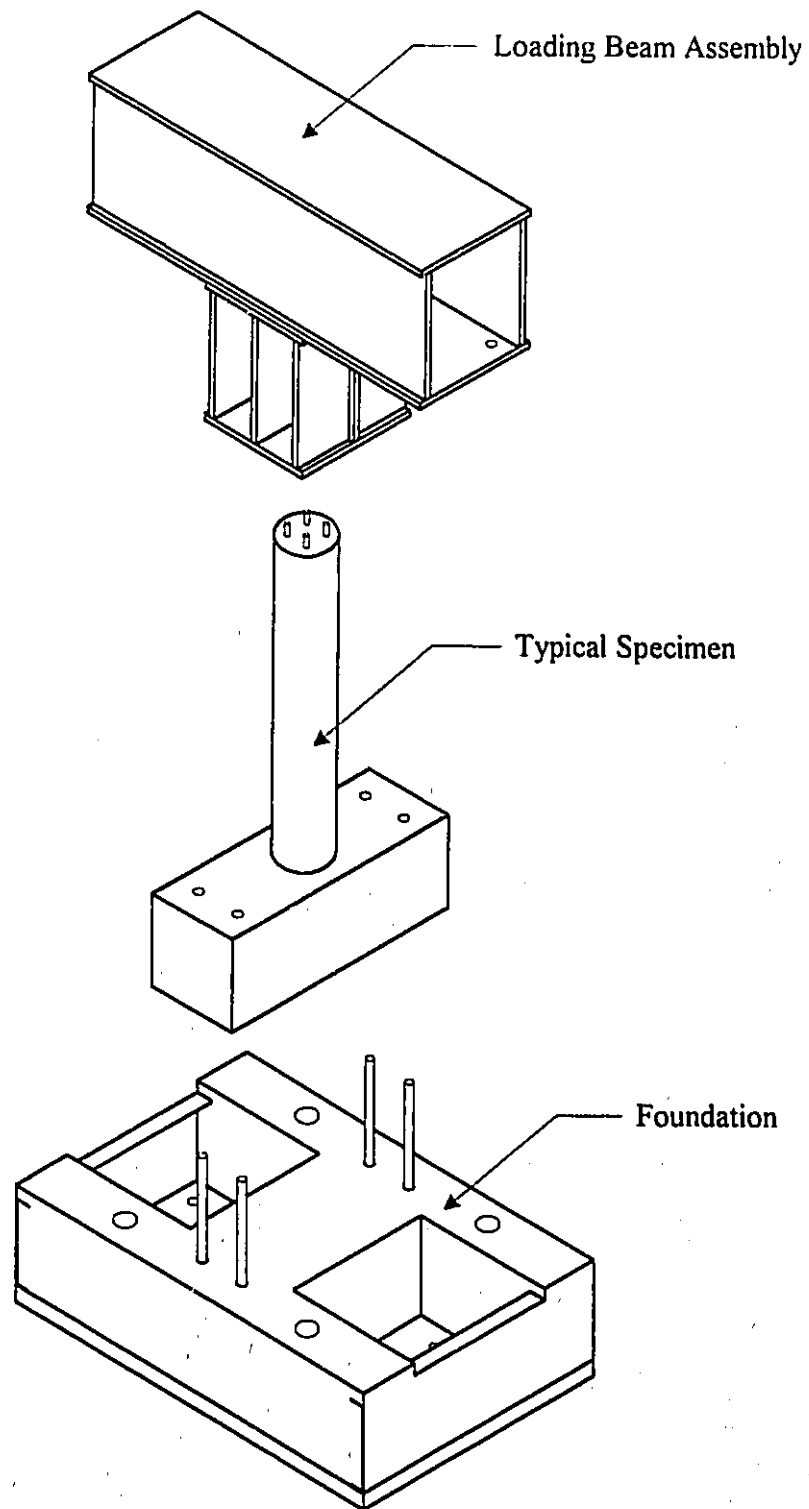
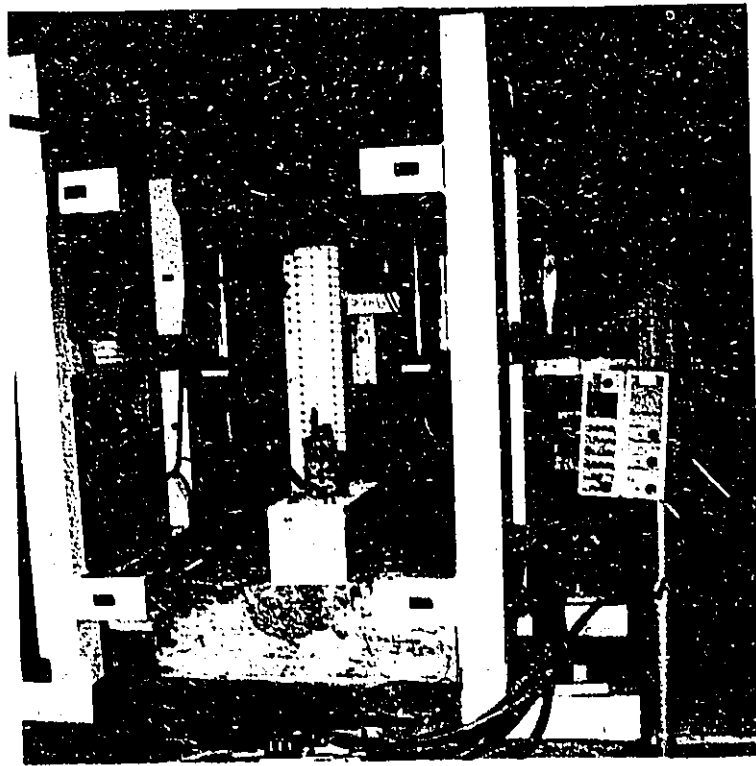
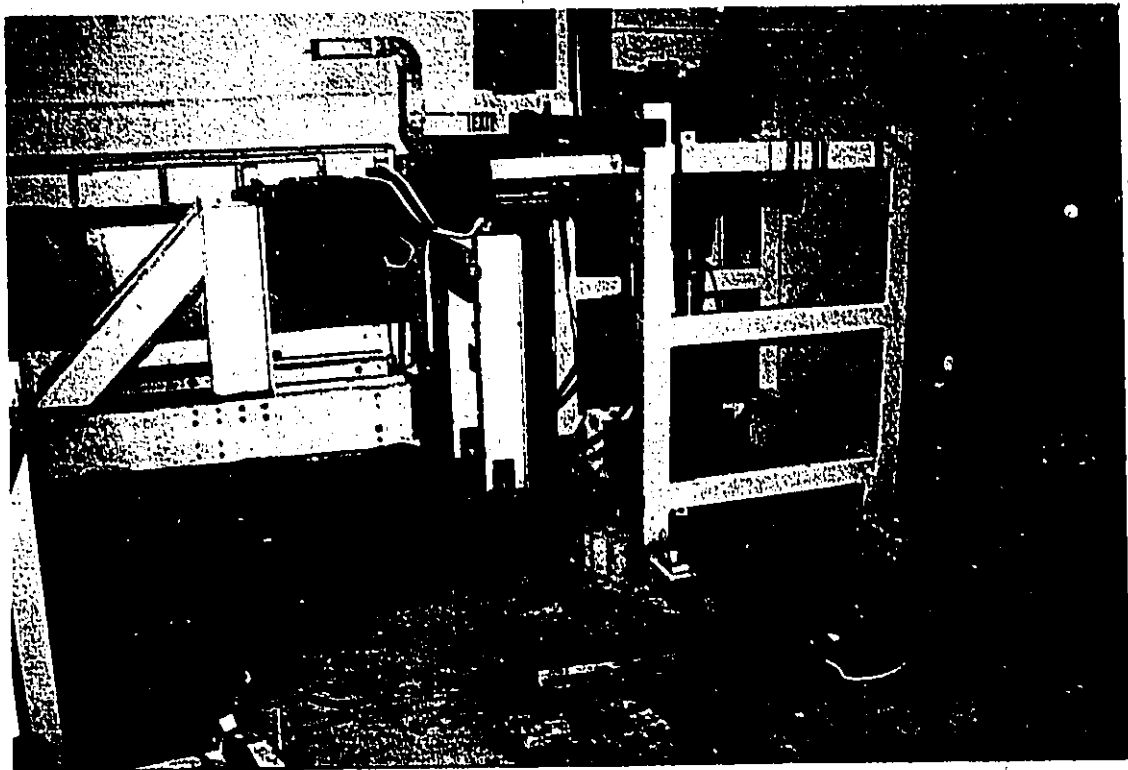


Figure 2.11: Assembly Drawing of Vertical Test Setup



(a) Front Elevation



(b) Side Elevation

Figure 2.12: General Views of The Test Setup

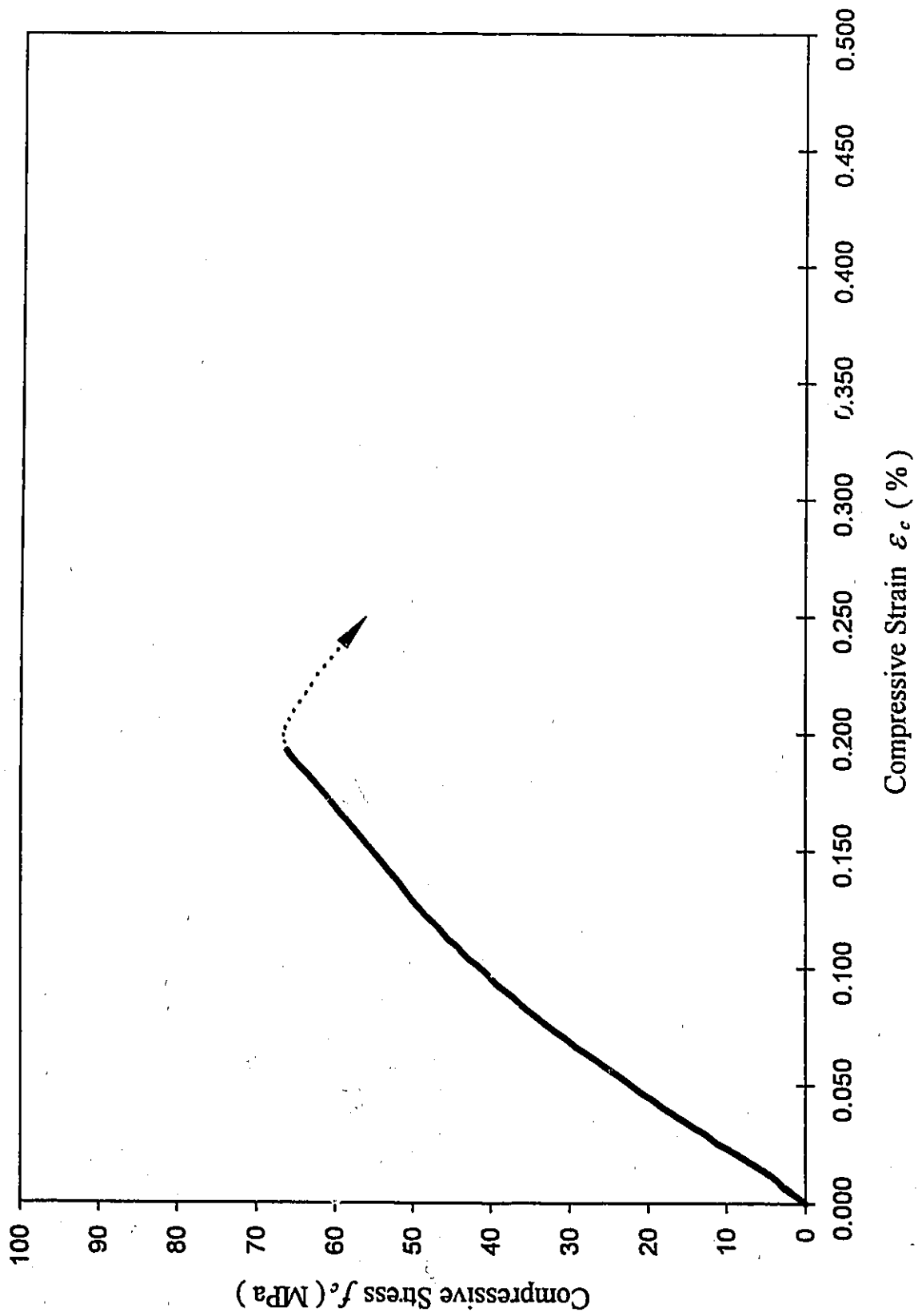


Figure 2.13: Stress - Strain Relationship of Concrete in Columns RC-1 and RC-2

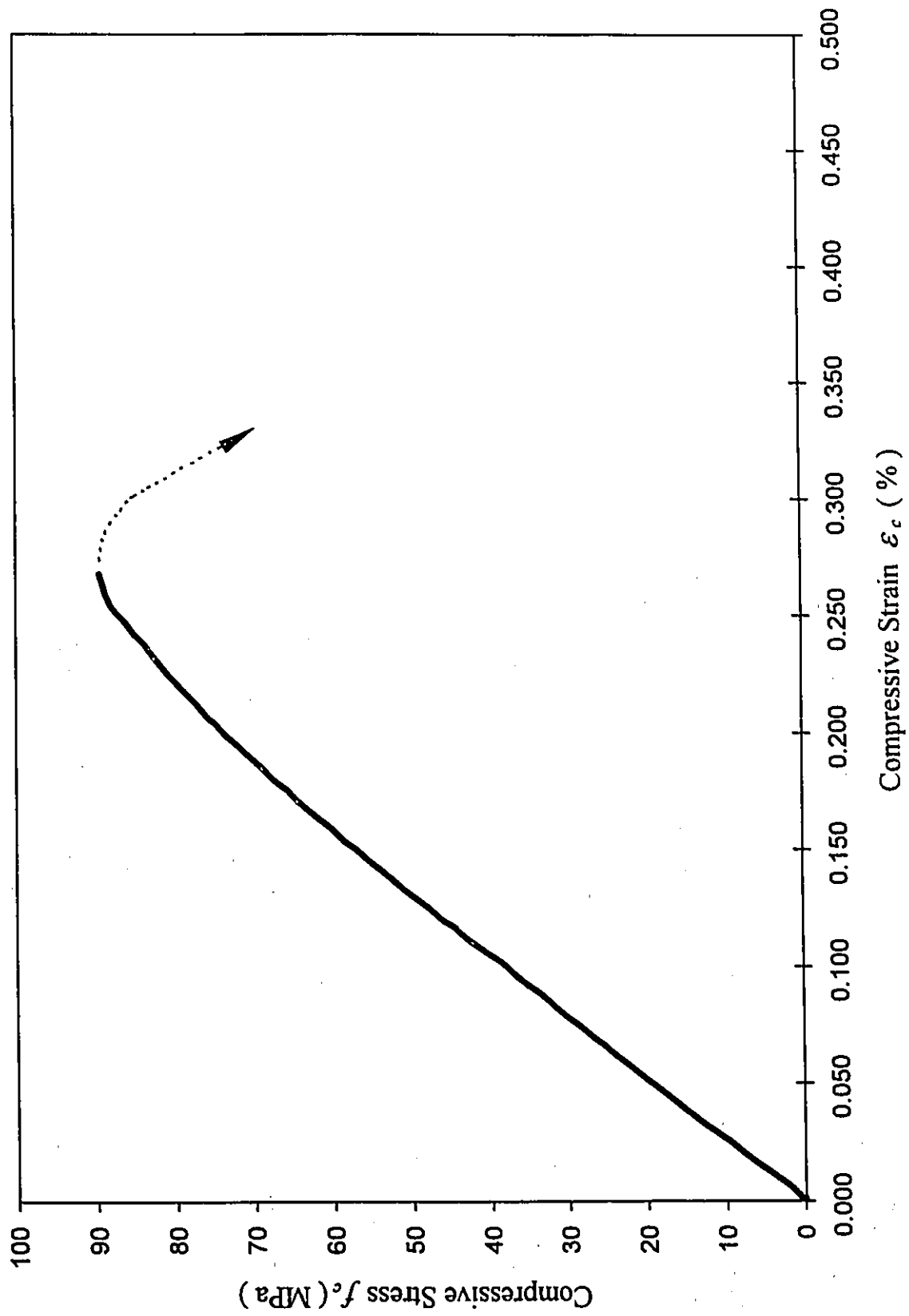
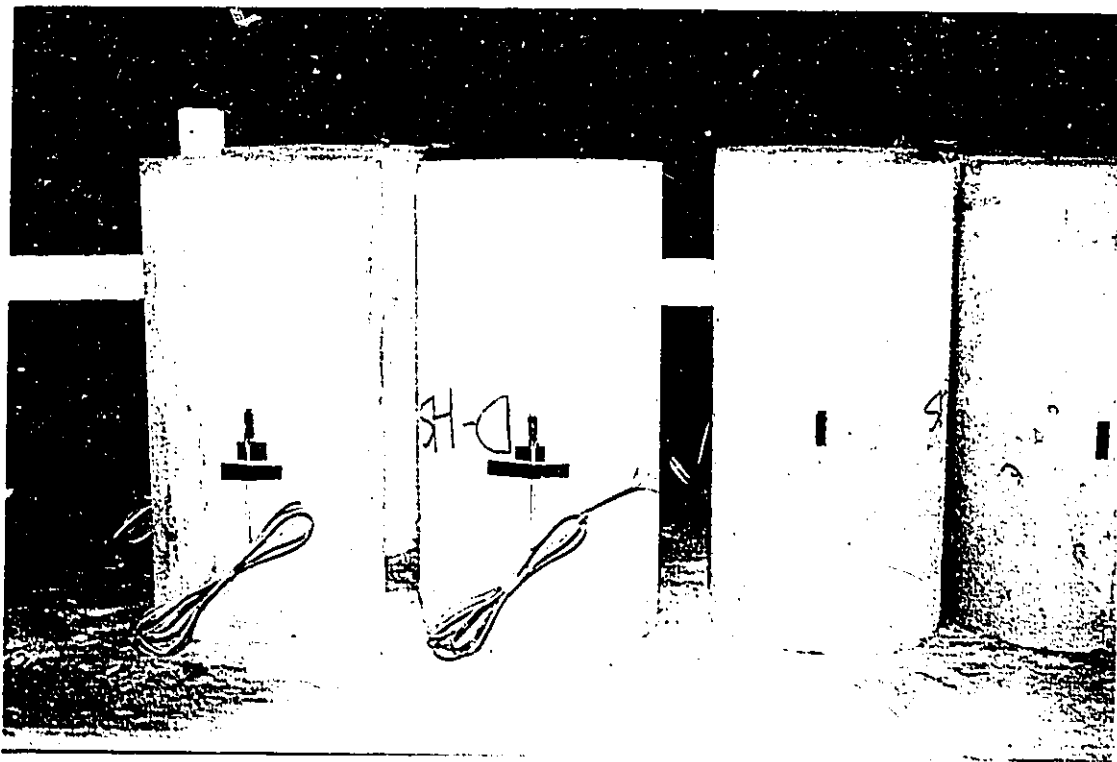
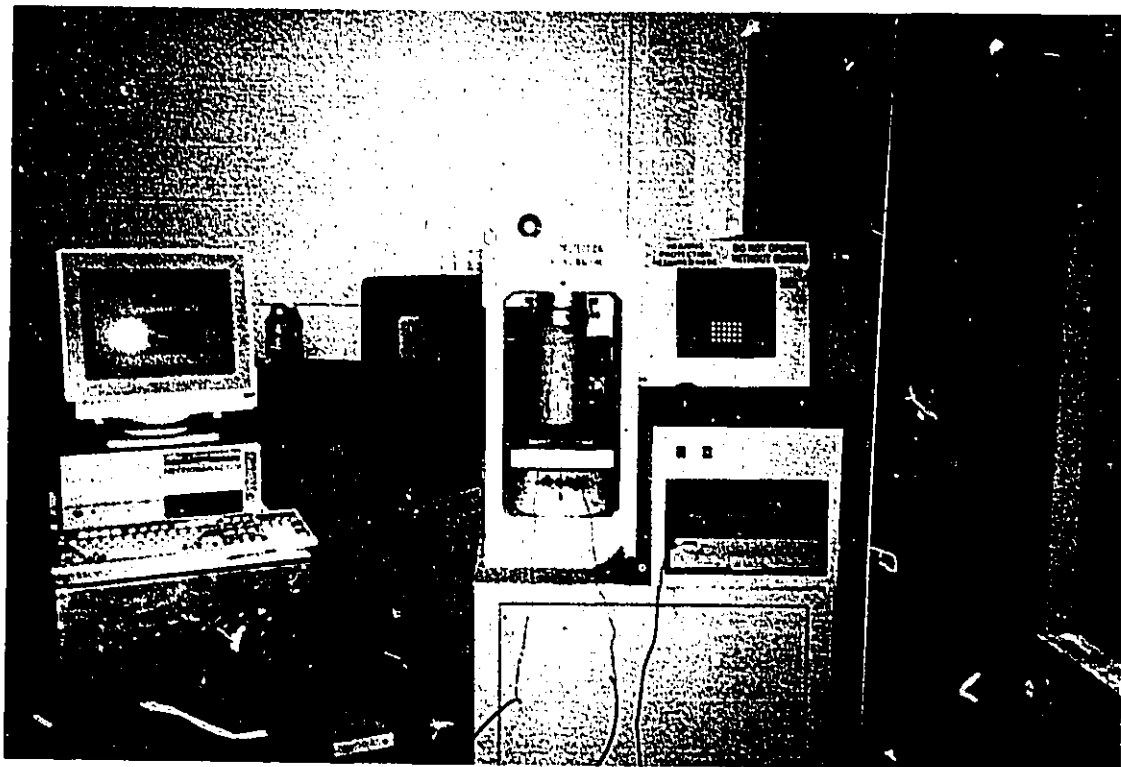


Figure 2.14: Stress - Strain Relationship of Concrete in Columns RC-3 through to RC-9



(a) Instrumentation of a Standard Concrete Cylinder with Strain Gauges



(b) Testing of Cylinder on a Forney Testing Machine

Figure 2.15: Instrumentation and Testing of Standard Concrete Cylinders

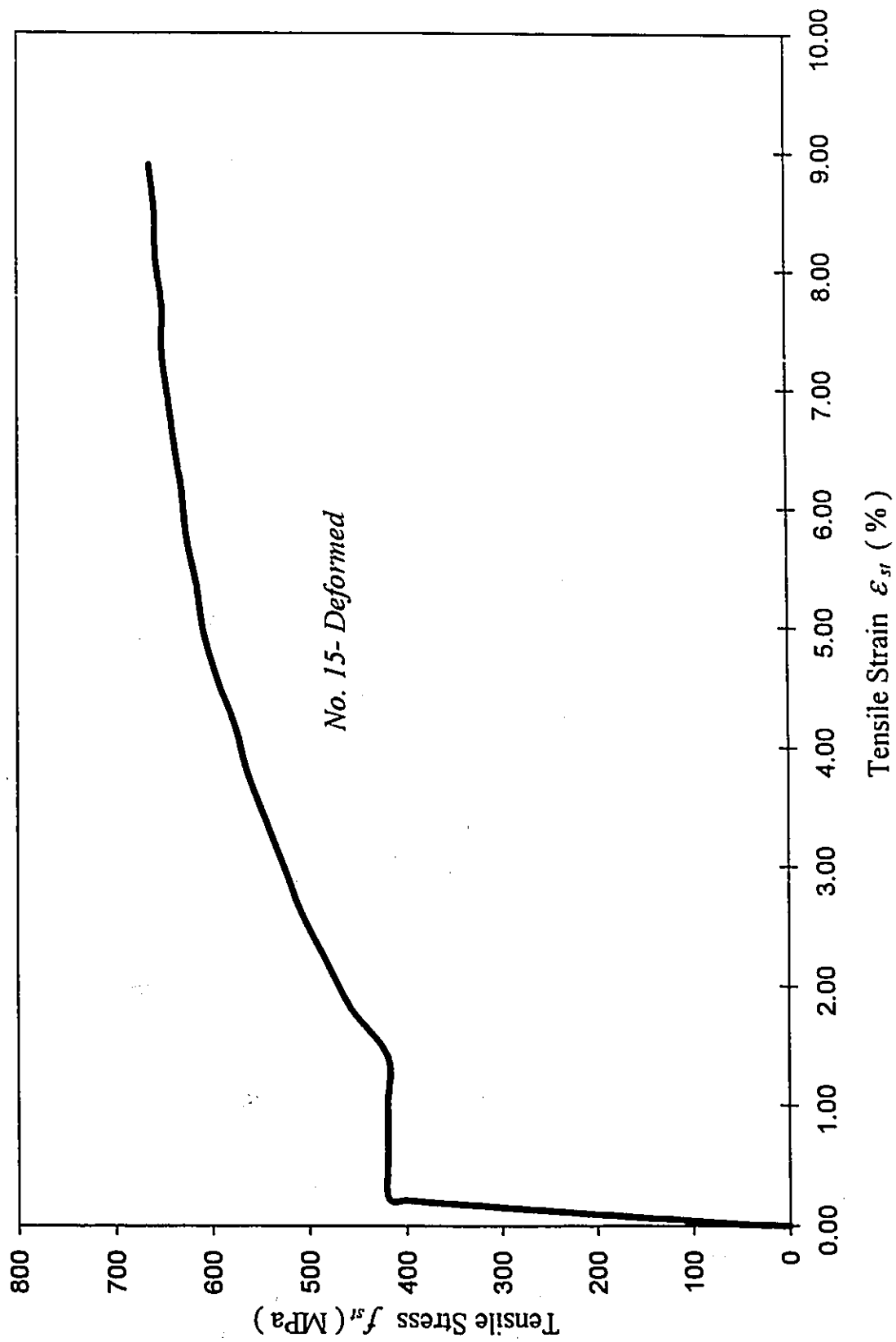


Figure 2.16: Stress - Strain Relationship of Longitudinal Steel (All Columns)

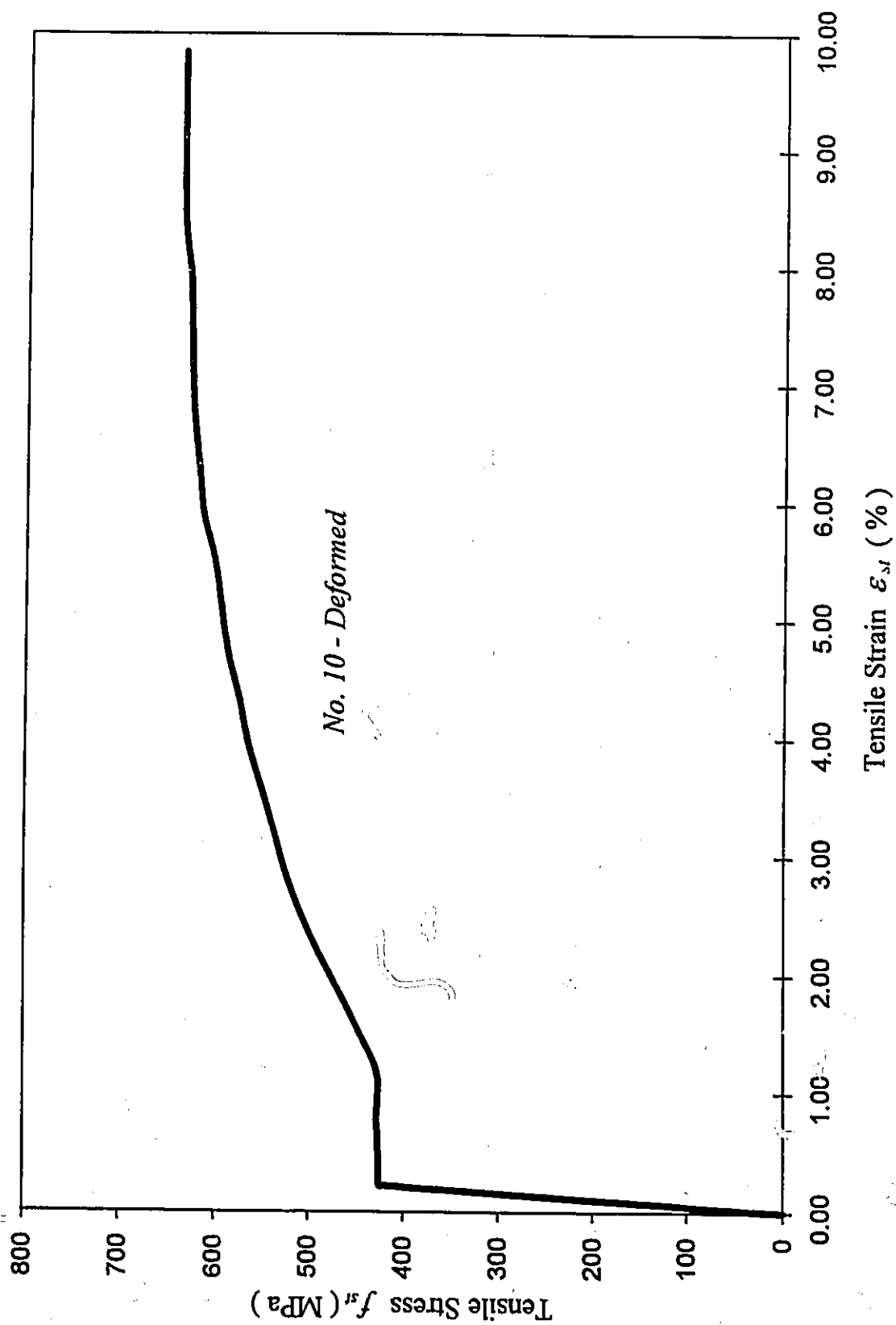


Figure 2.17: Stress - Strain Relationship of Transverse Steel (Columns RC-2, RC-5, RC-6, and RC-9)

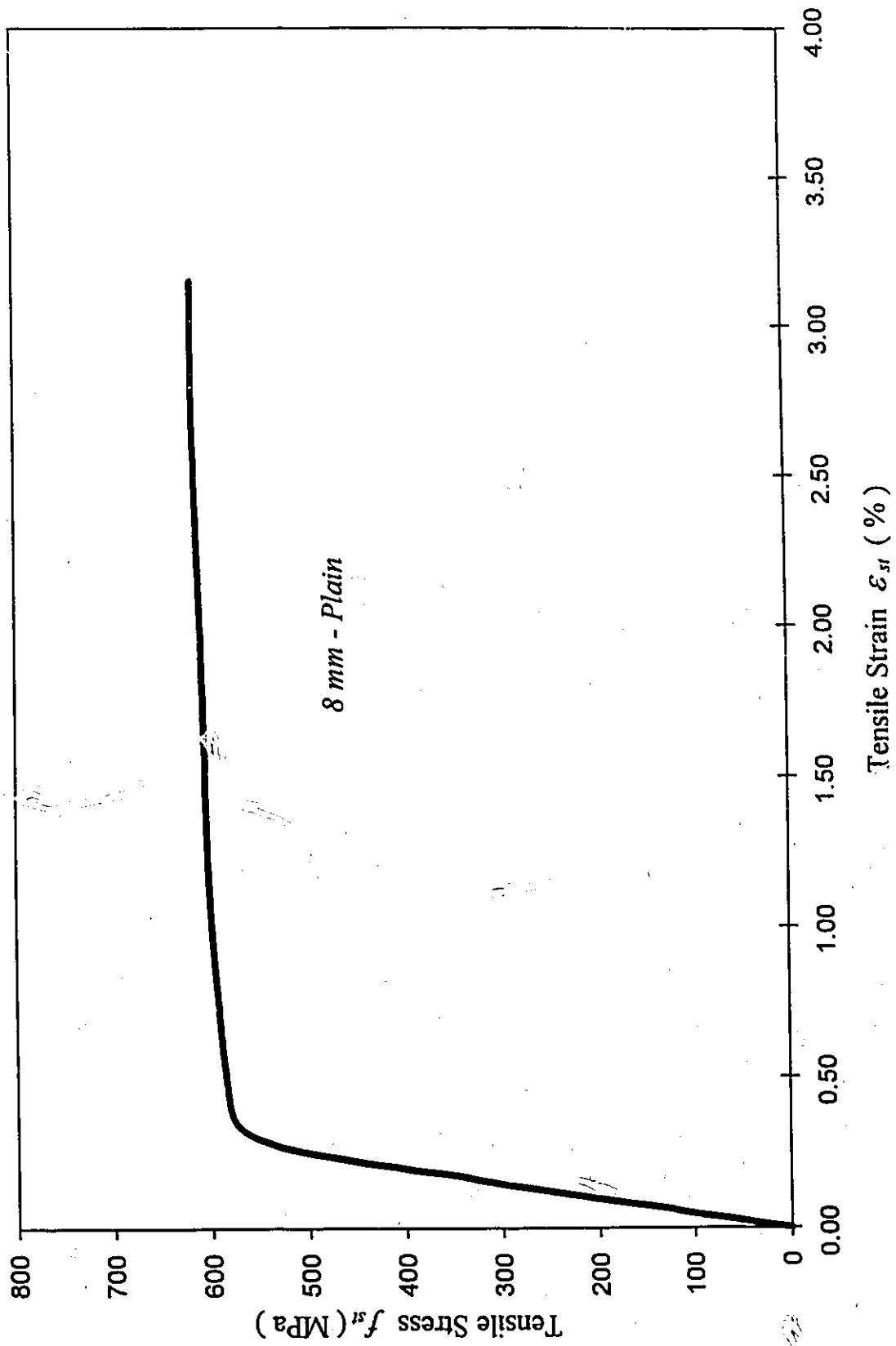


Figure 2.18: Stress - Strain Relationship of Transverse Steel (Column RC-4)

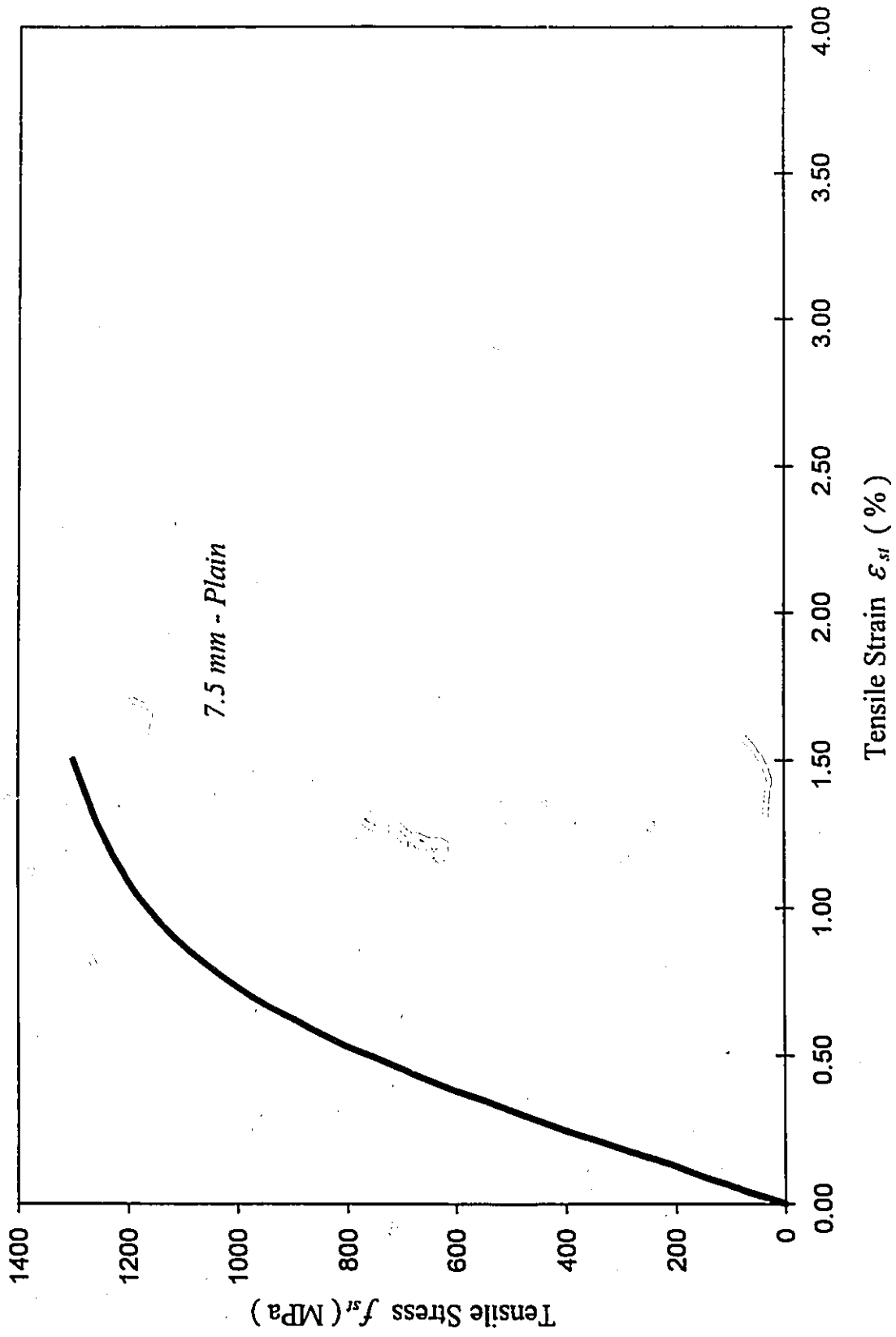


Figure 2.19: Stress - Strain Relationship of Transverse Steel (Columns RC-1, RC-3, RC-7, and RC-8)

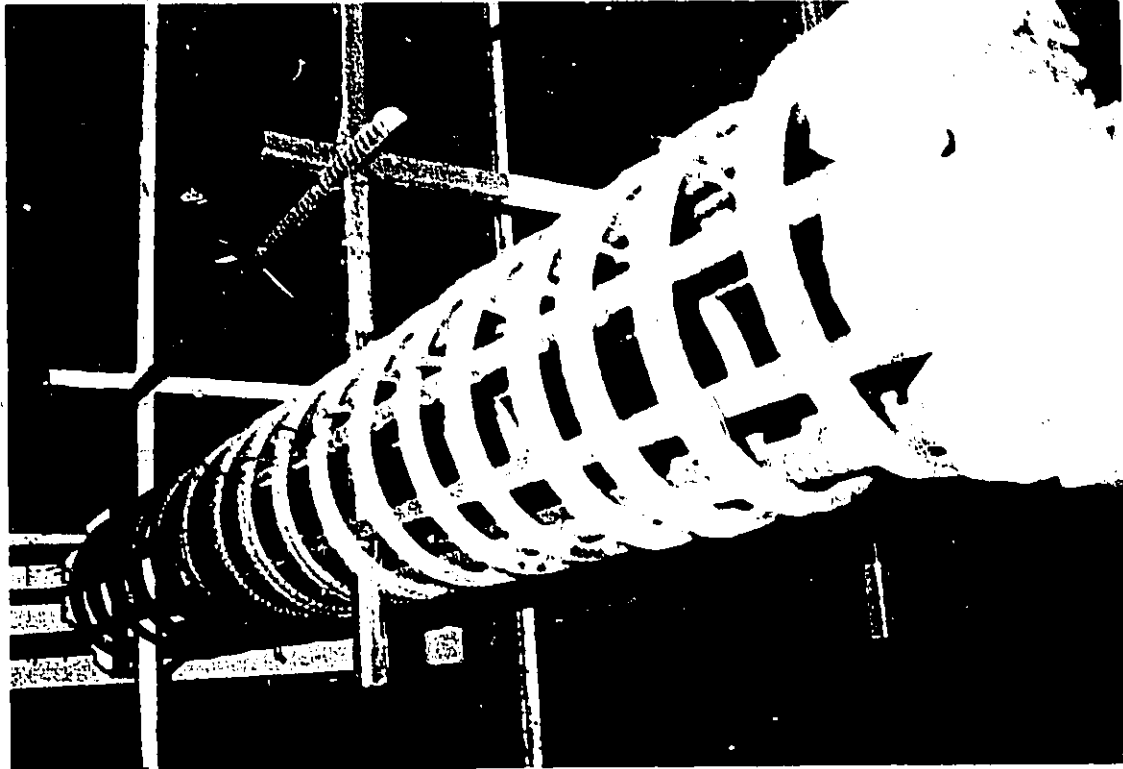
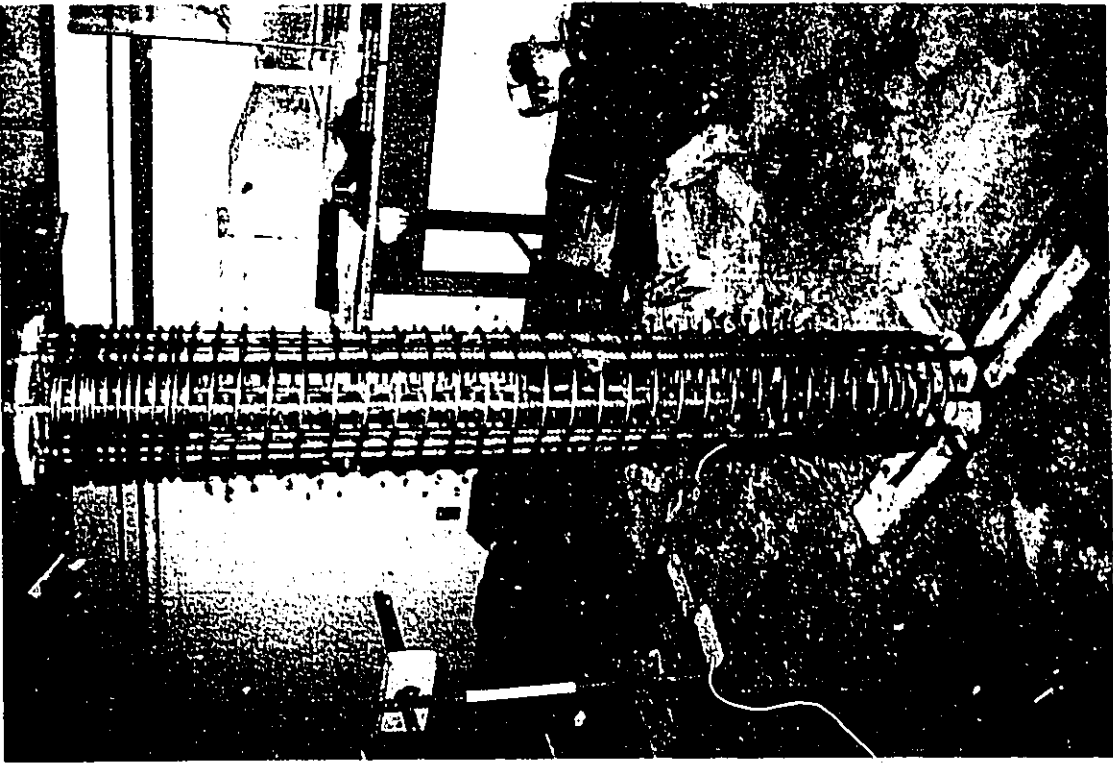


Figure 2.20: Assembly of a Column Reinforcement Cage

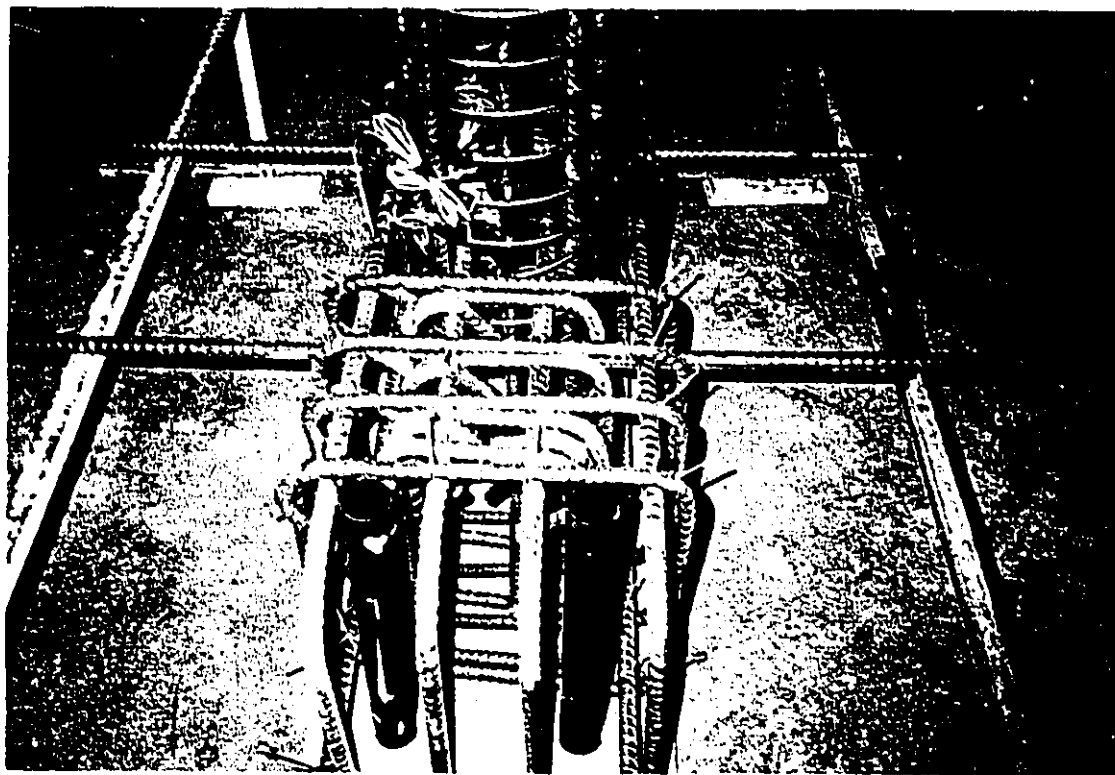
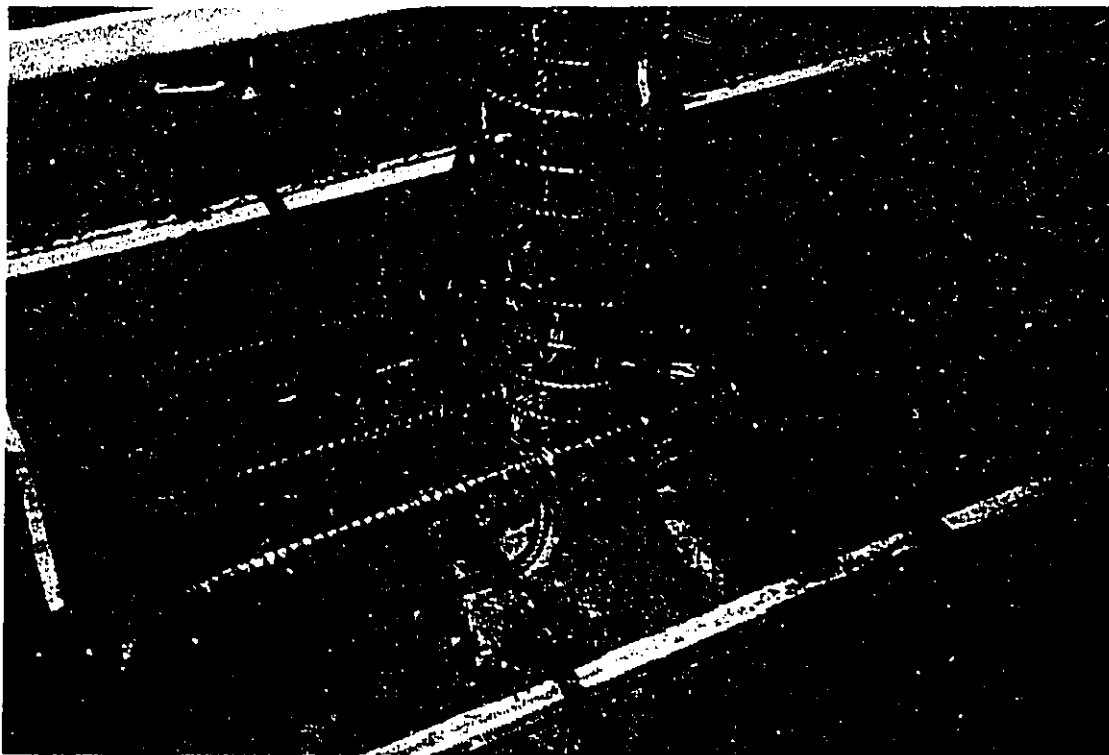
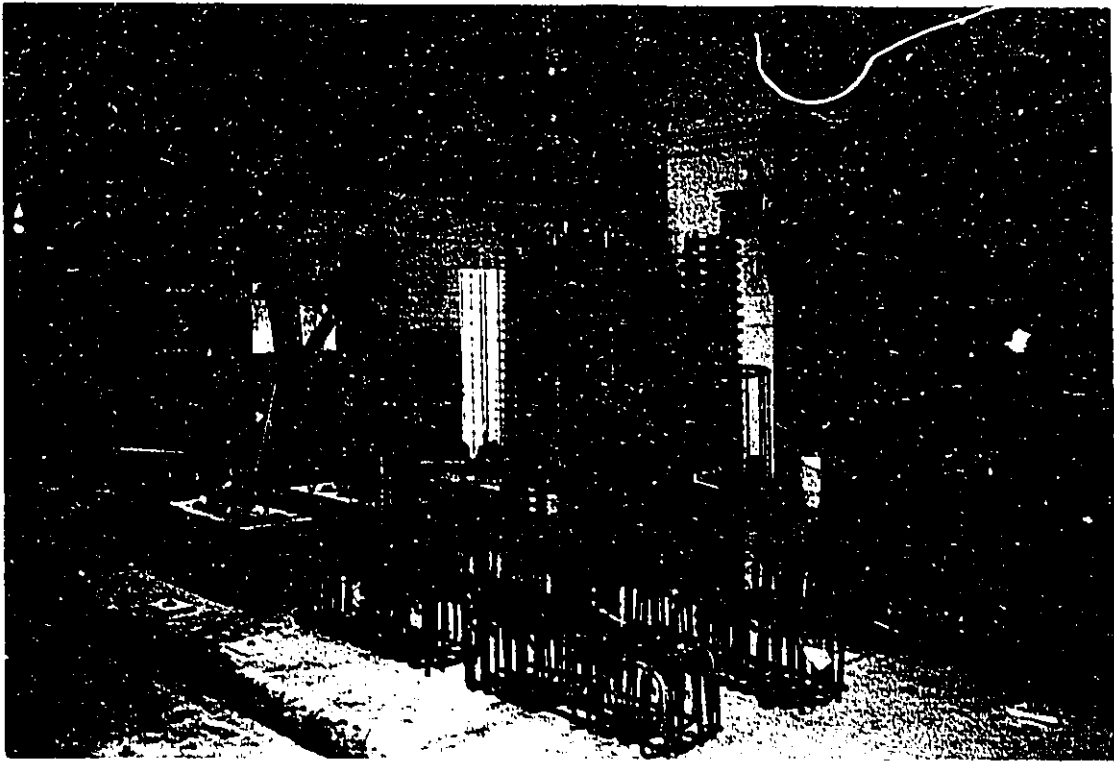


Figure 2.21: Assembly of a Typical Footing Reinforcement Cage



(a) Specimen Cages



(b) Strain Gauges and Wiring on Spiral Reinforcement of a Completed Cage

Figure 2.22: Completed Specimen Reinforcement Cages

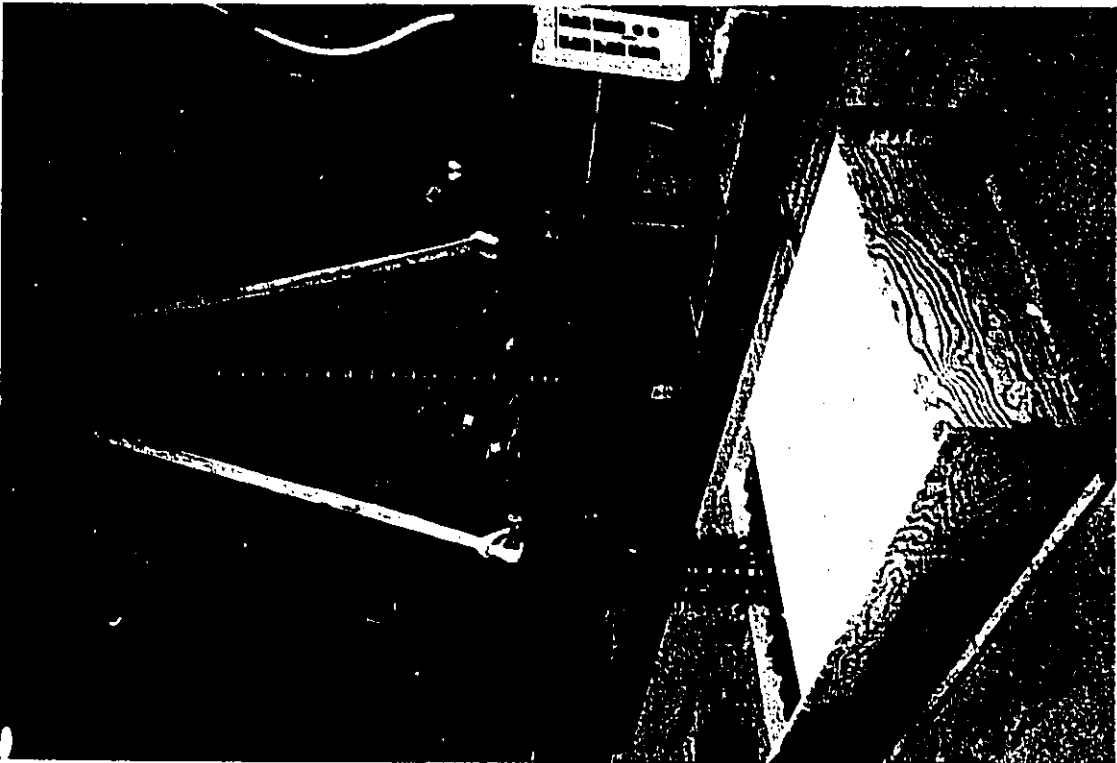
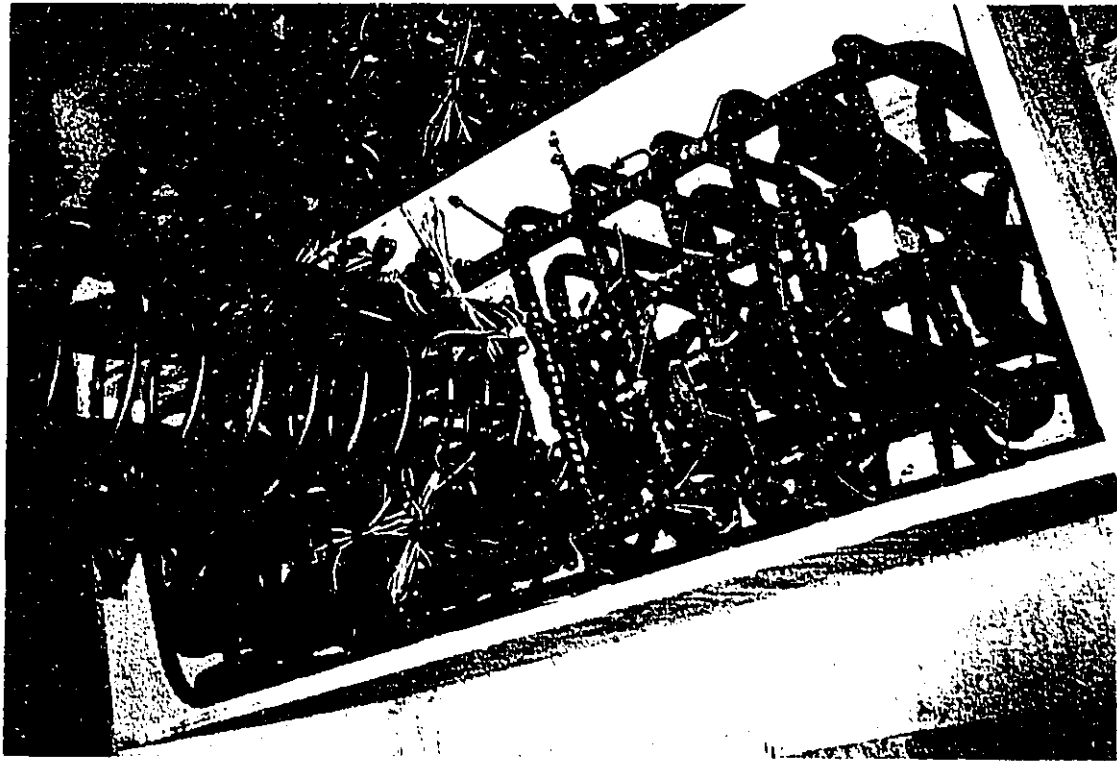
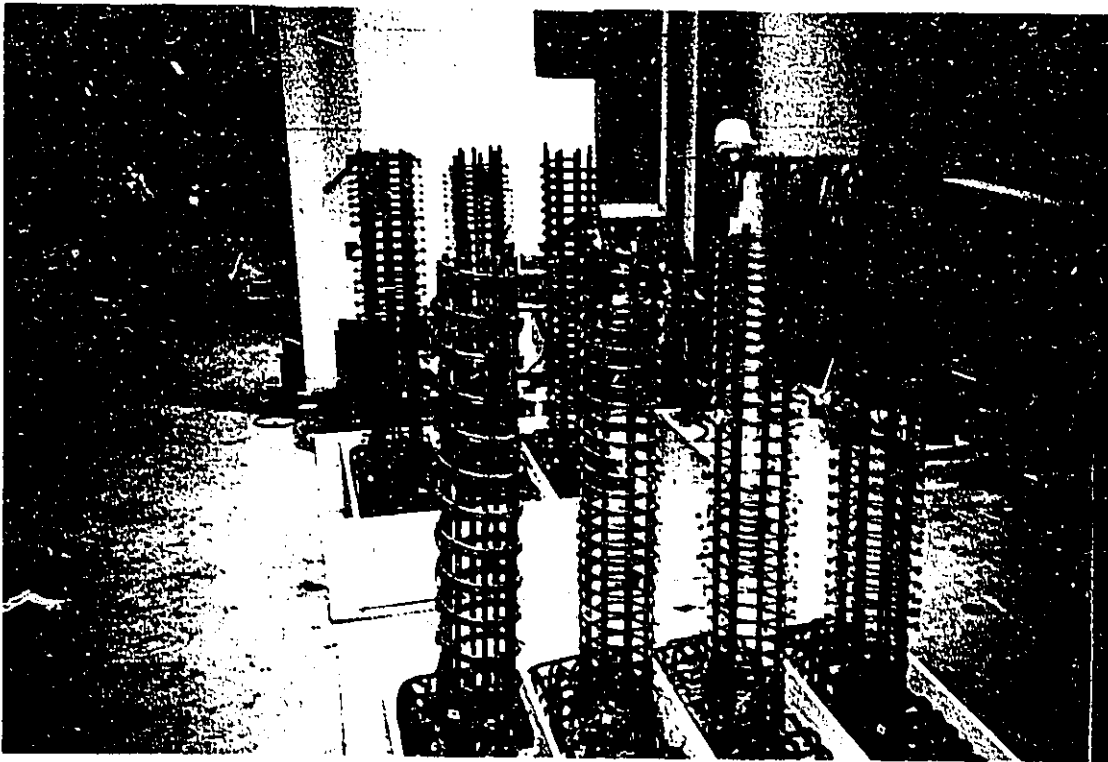
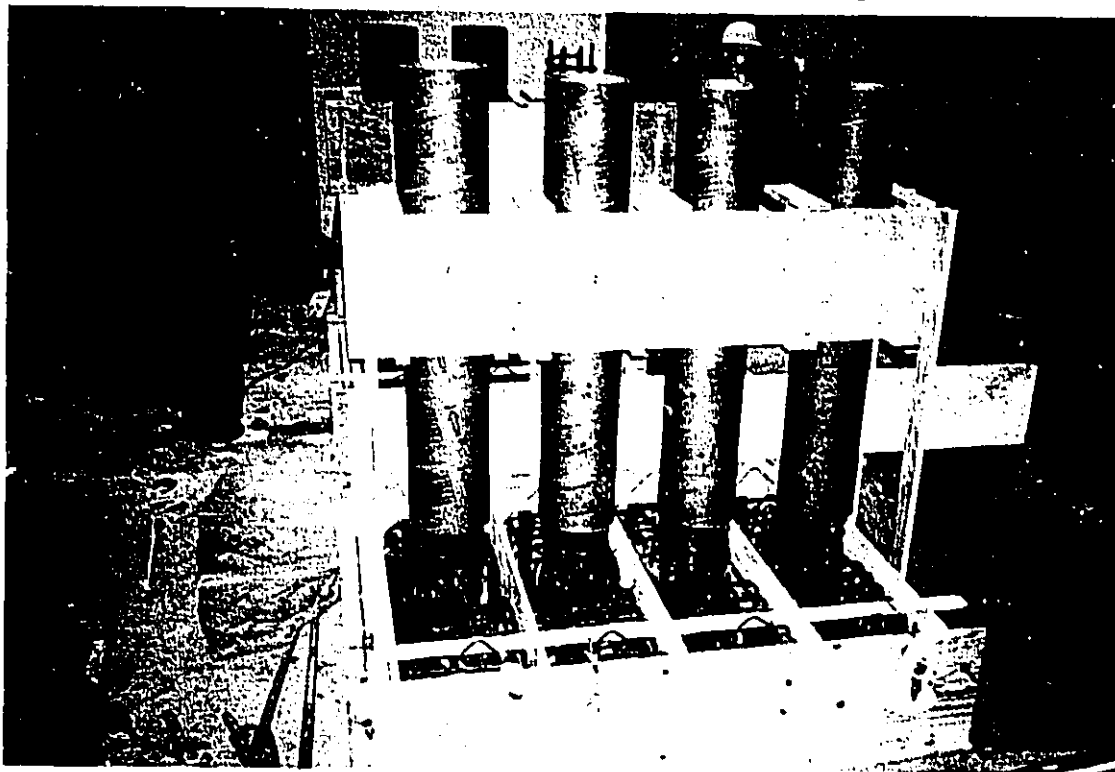


Figure-2.23: Placement of Reinforcement Cages in the Formwork

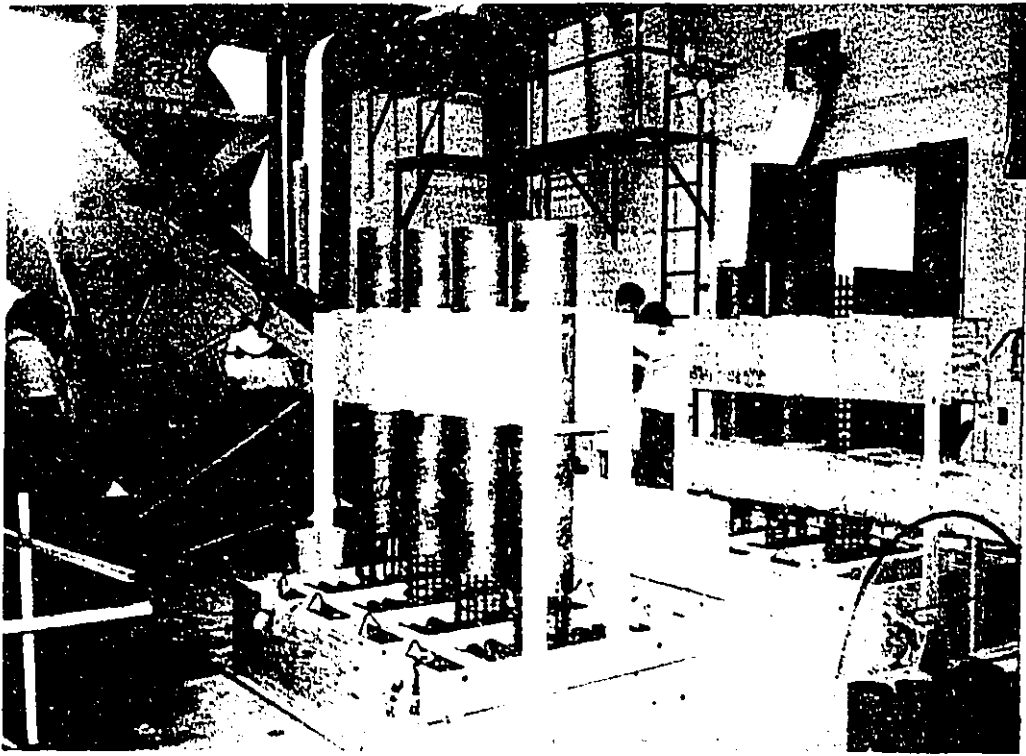


(a) Prior to Attachment of Sona Tubes and Bracing

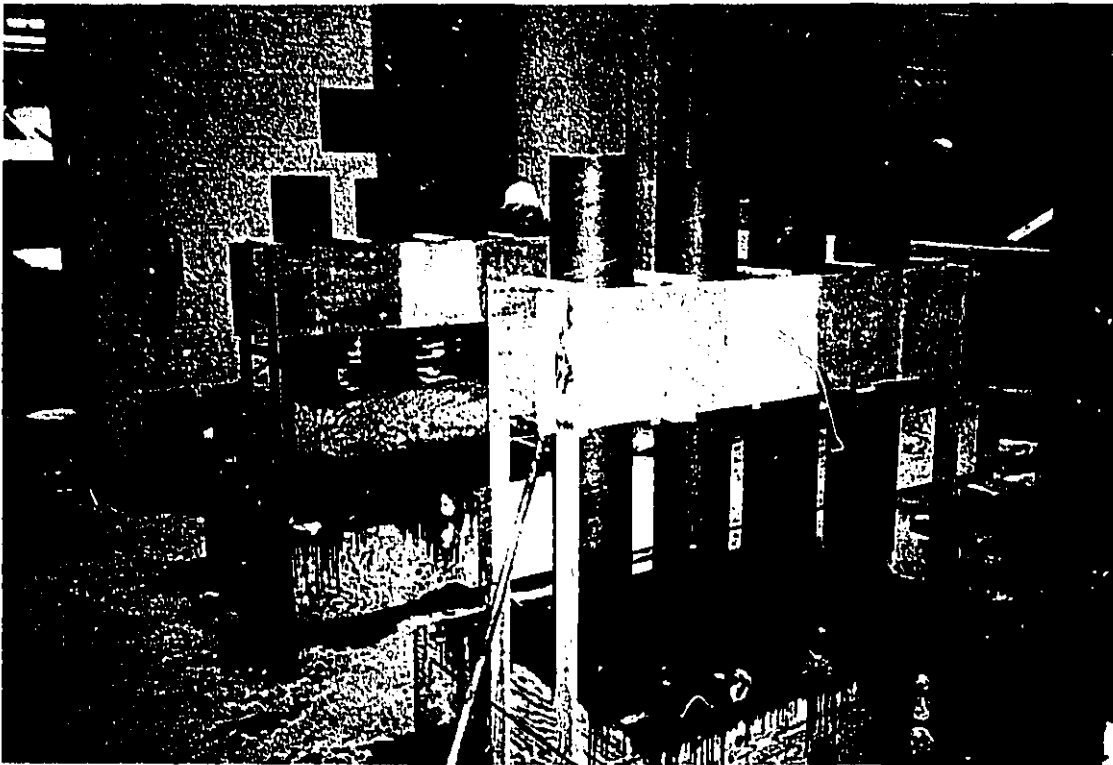


(b) With Sona Tubes and Bracing in Place

Figure 2.24: Reinforcement Cages in Formwork

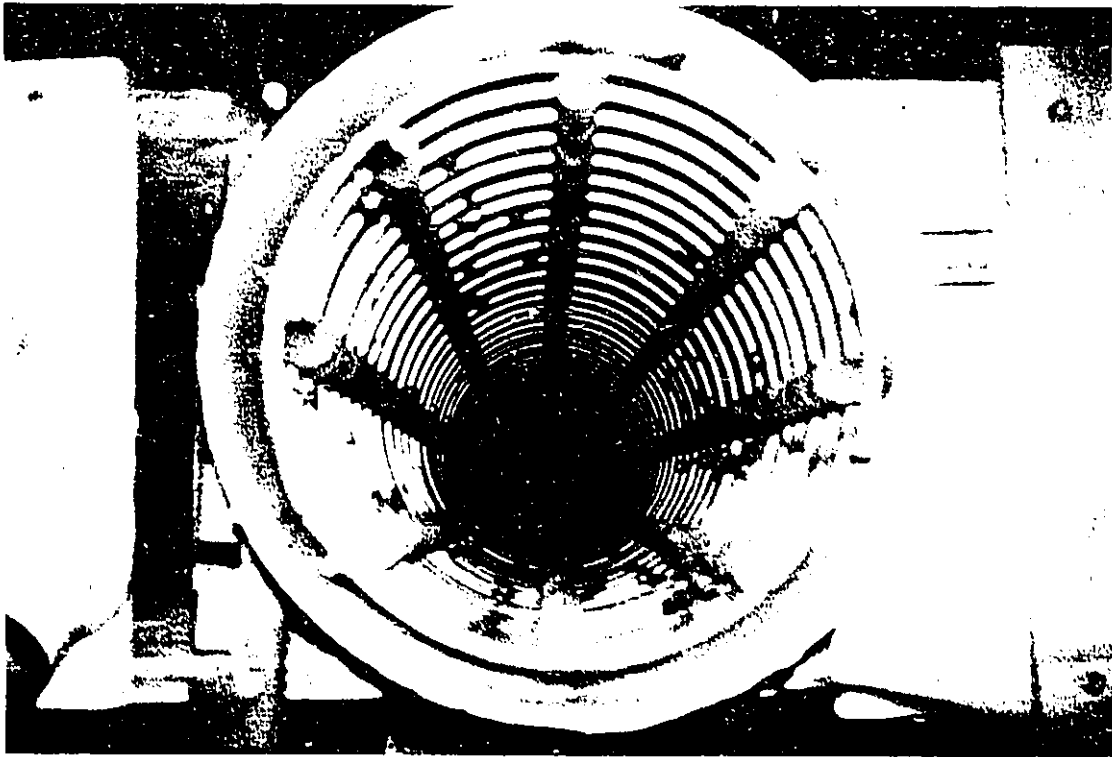


(a) Shortly Before Casting

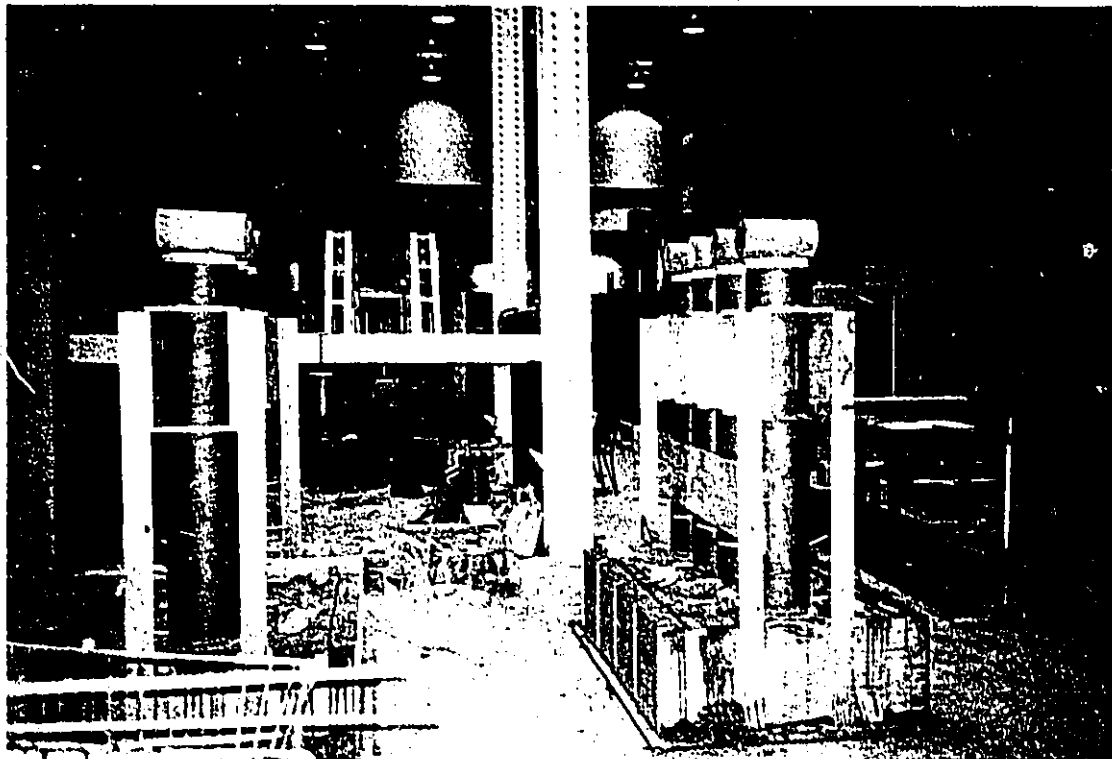


(b) At Completion

Figure 2.25: Casting of Specimen Footings



(a) Before Casting



(b) At Completion

Figure 2.26: Casting of Specimen Columns

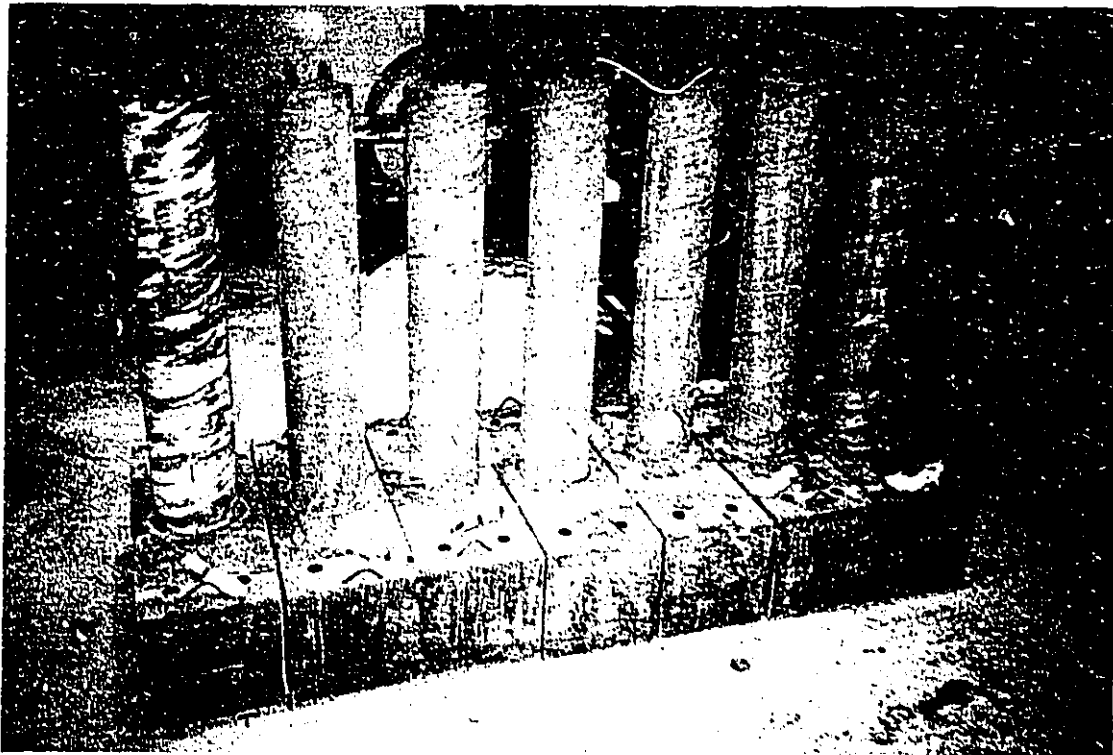


Figure 2.27: Curing of Specimens in the Laboratory

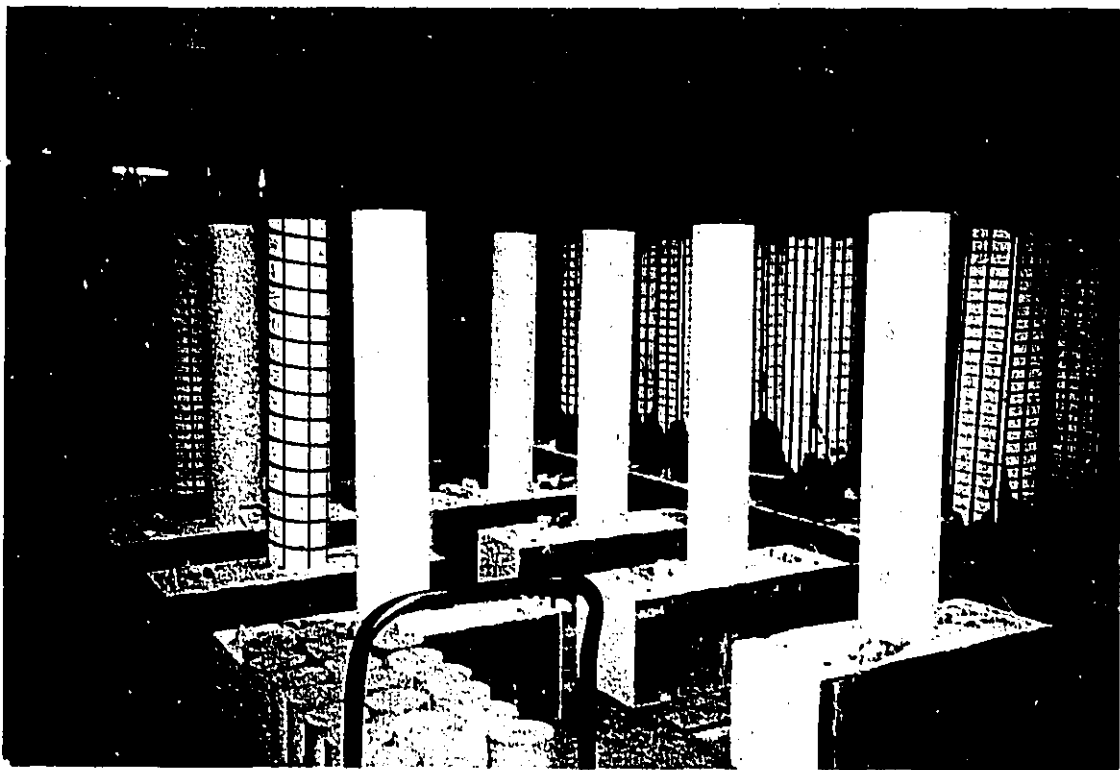
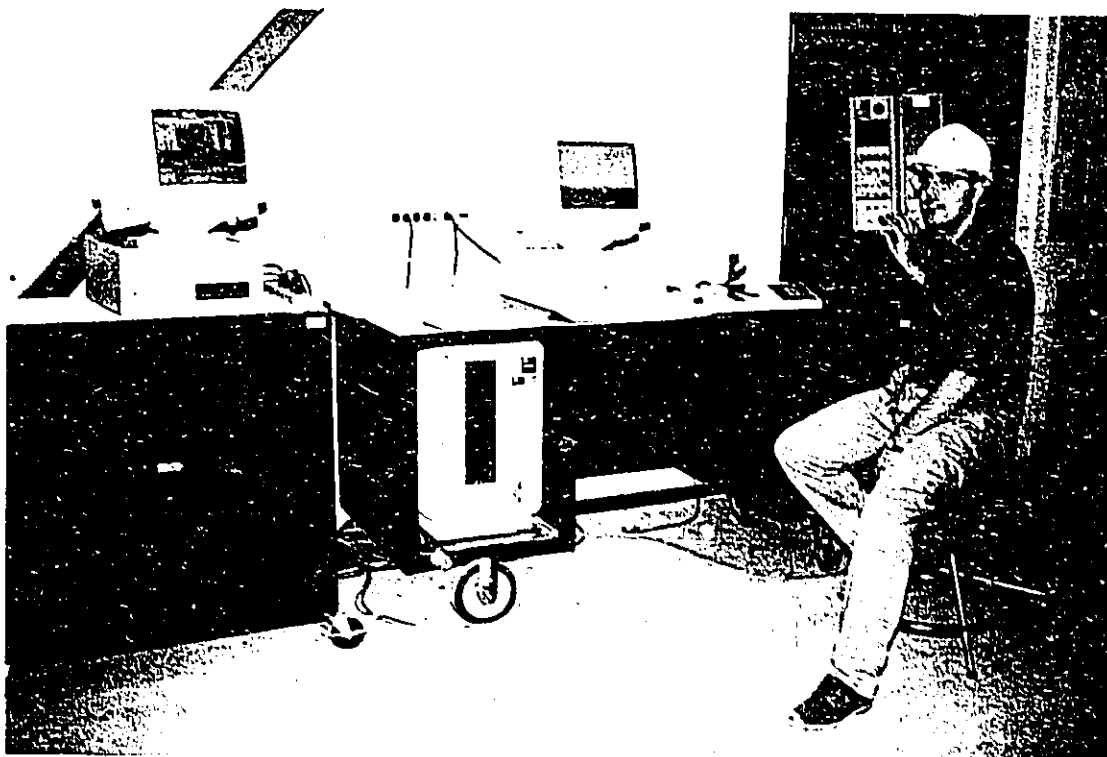
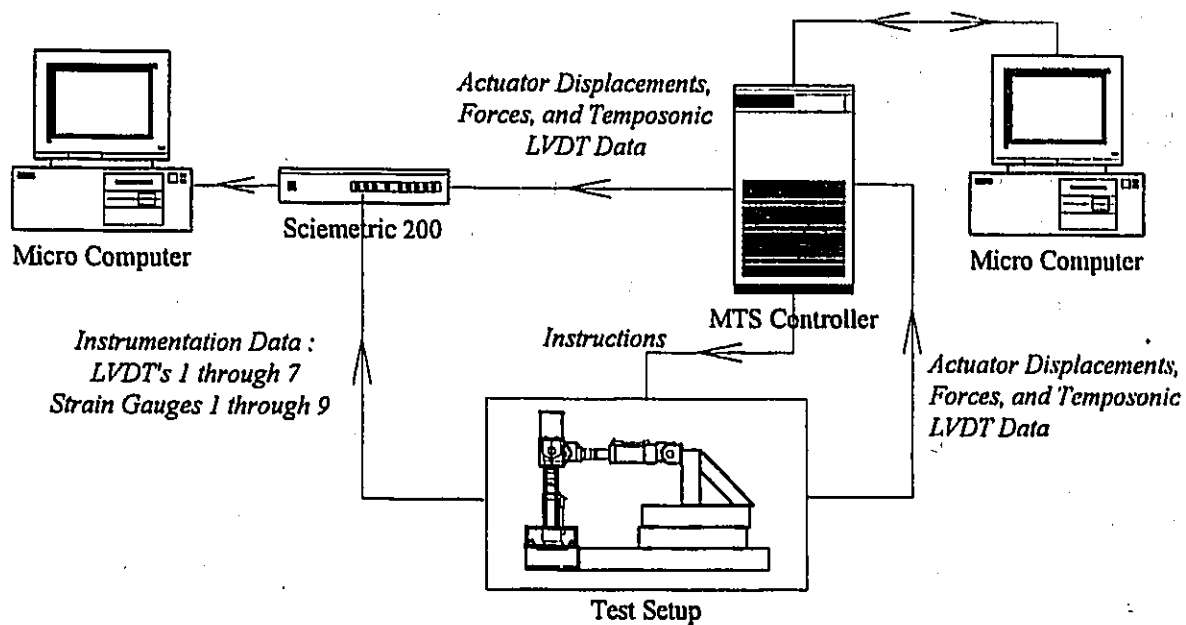


Figure 2.28: Completed Test Specimens



(a) Data Acquisition System and MTS Controller



(b) Schematic Network Diagram

Figure 2.29: Acquisition of Test Data

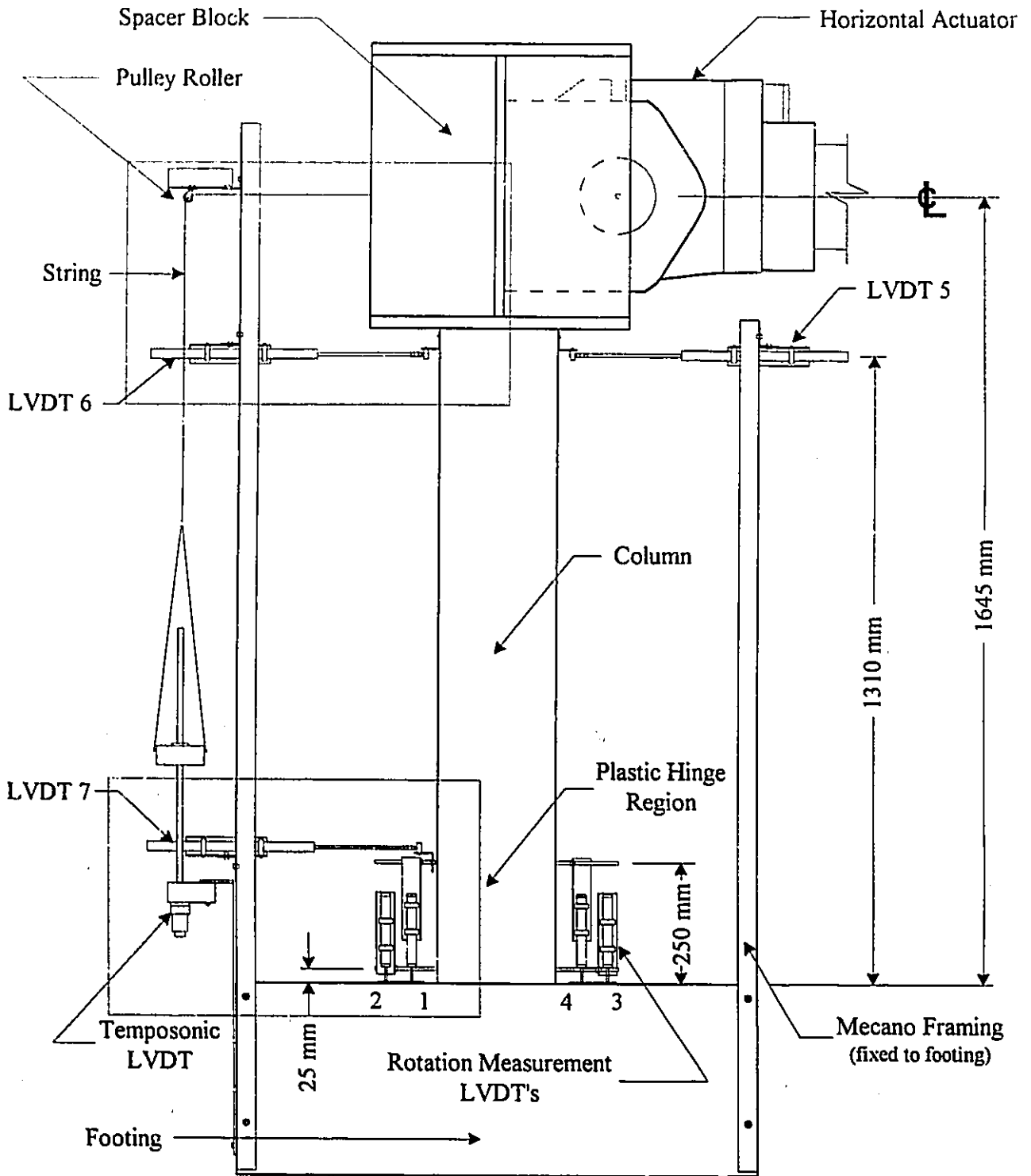
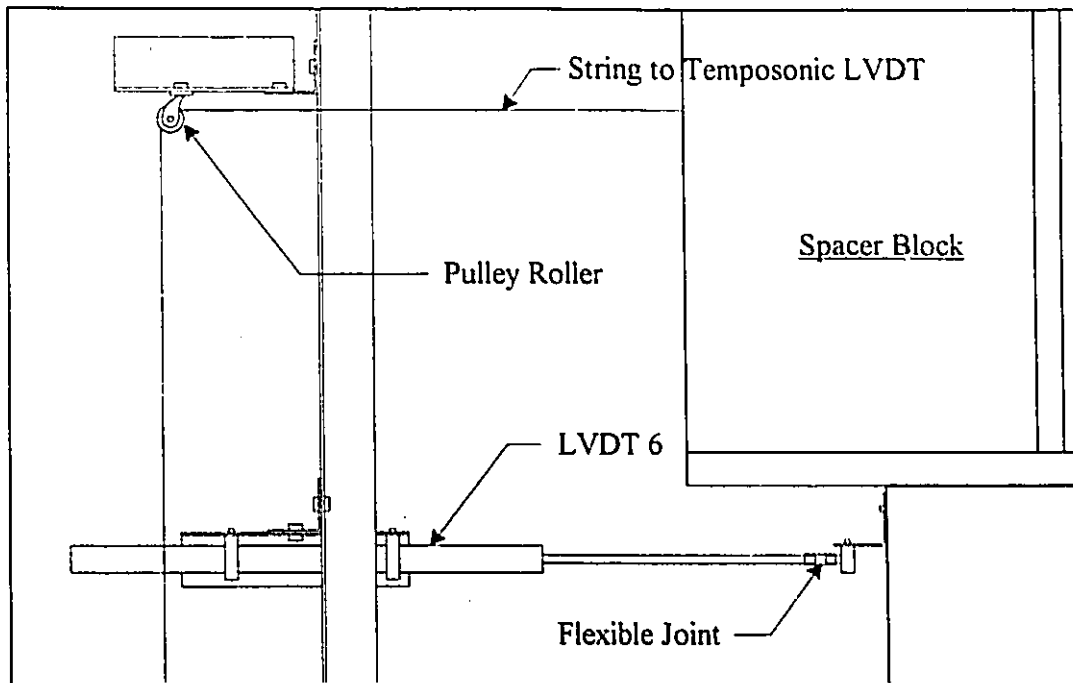
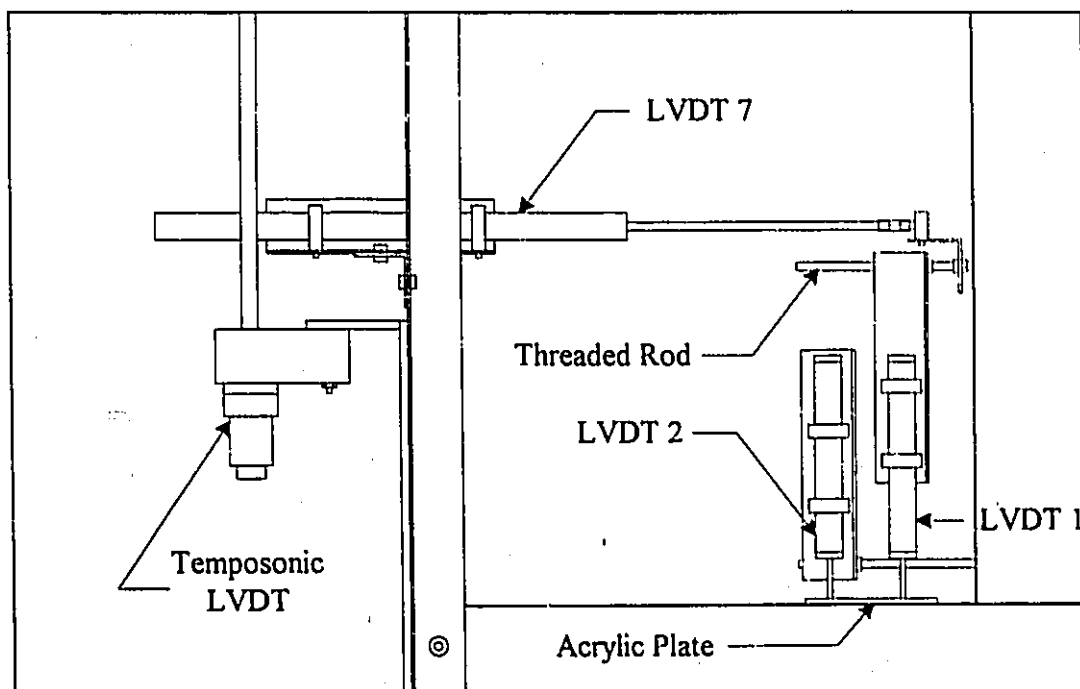


Figure 2.30: Configuration of LVDT's on a Typical Specimen



(a) Measurement of Column Tip Displacement (LVDT 6)



(b) Measurement of Hinge Displacement (LVDT 7) and Hinge Rotations (LVDT 1 and 2)

Figure 2.31: Close-up Views of LVDT's (from Fig. 2.30)

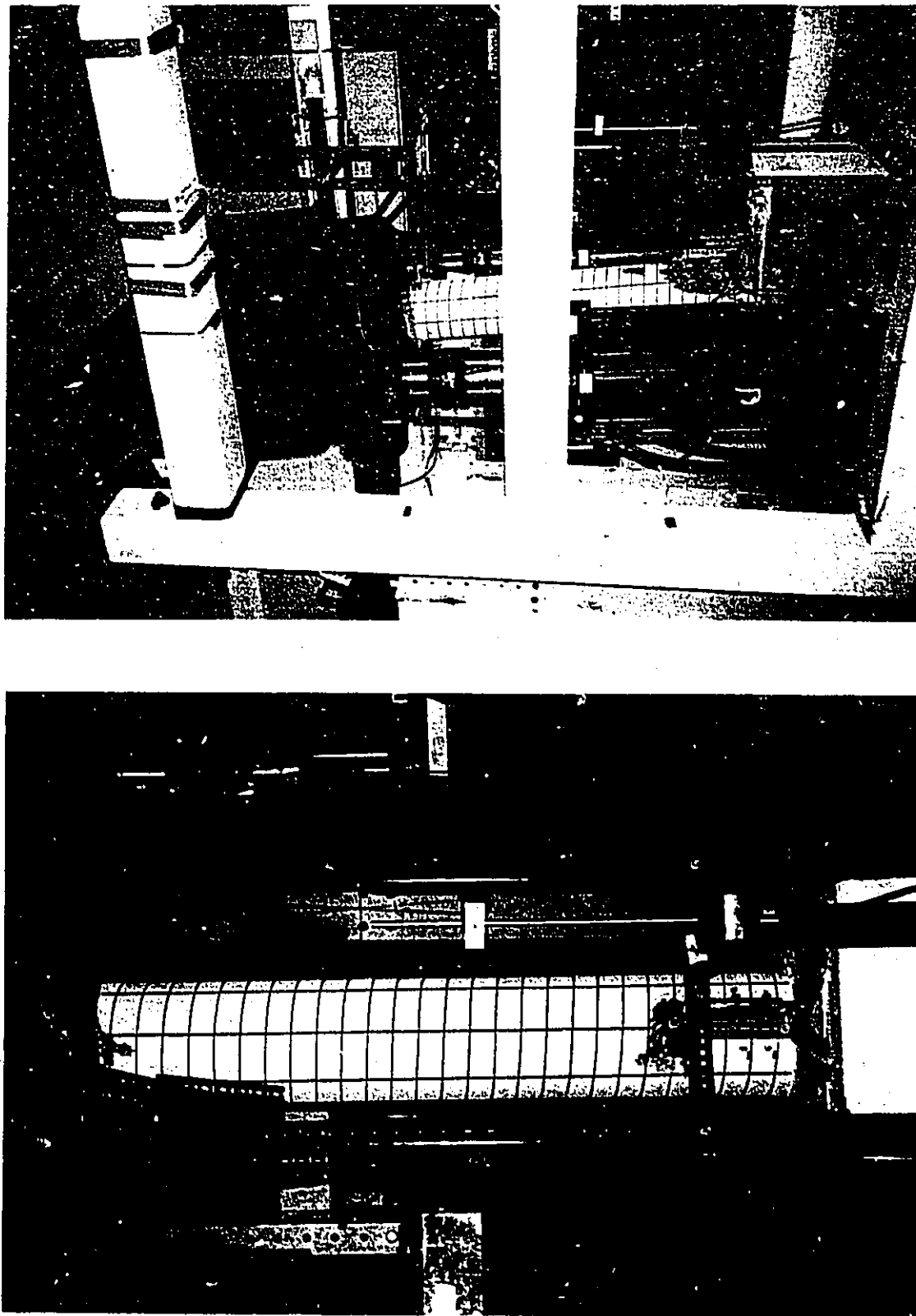
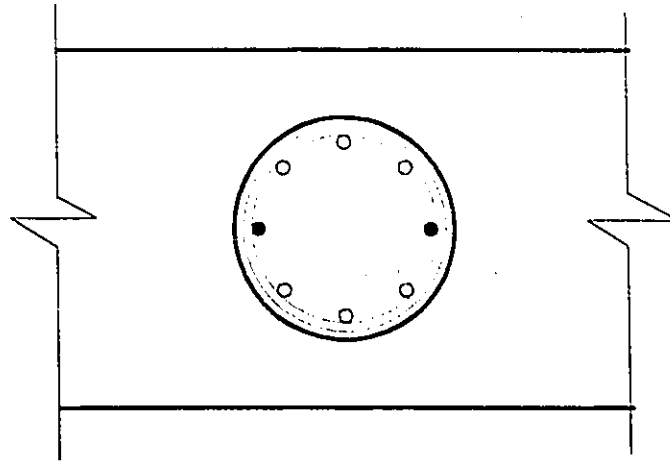
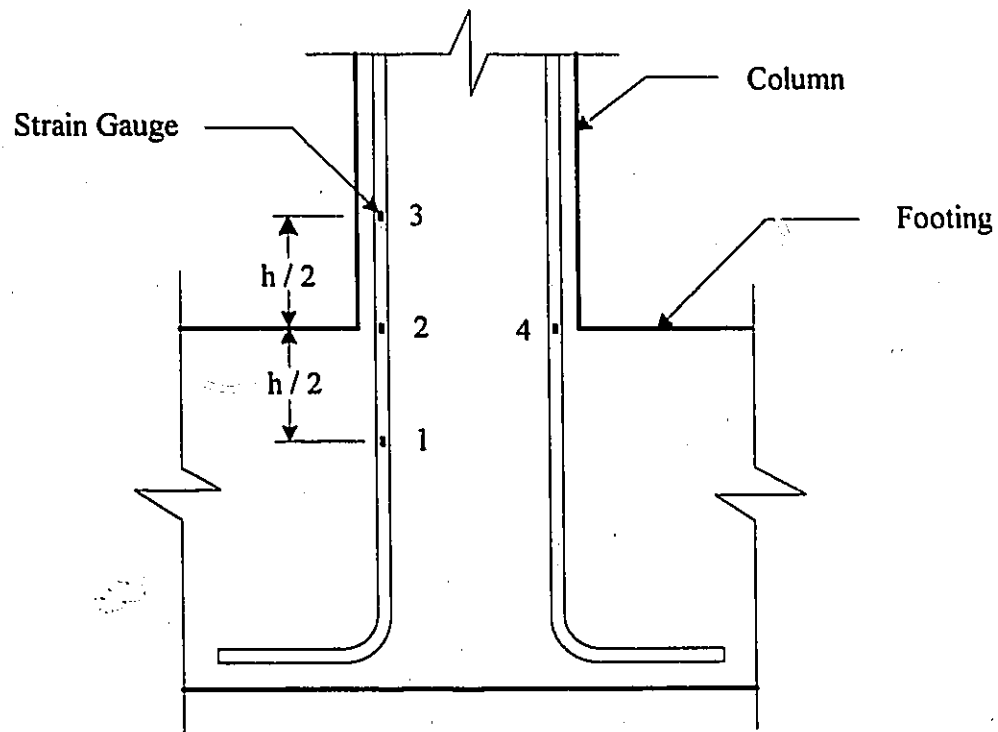


Figure 2.32: Views of LVDT Instrumentation of a Typical Specimen During Testing

Plan View



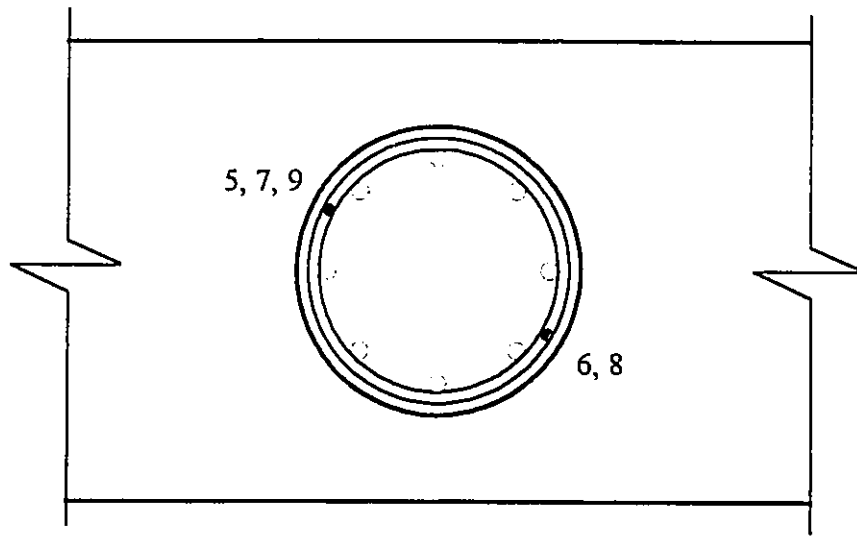
Elevation View



h = depth of column cross-section (250 mm)

Figure 2.33: Location of Strain Gauges on Longitudinal Bars in All Specimens

Plan View



Elevation View

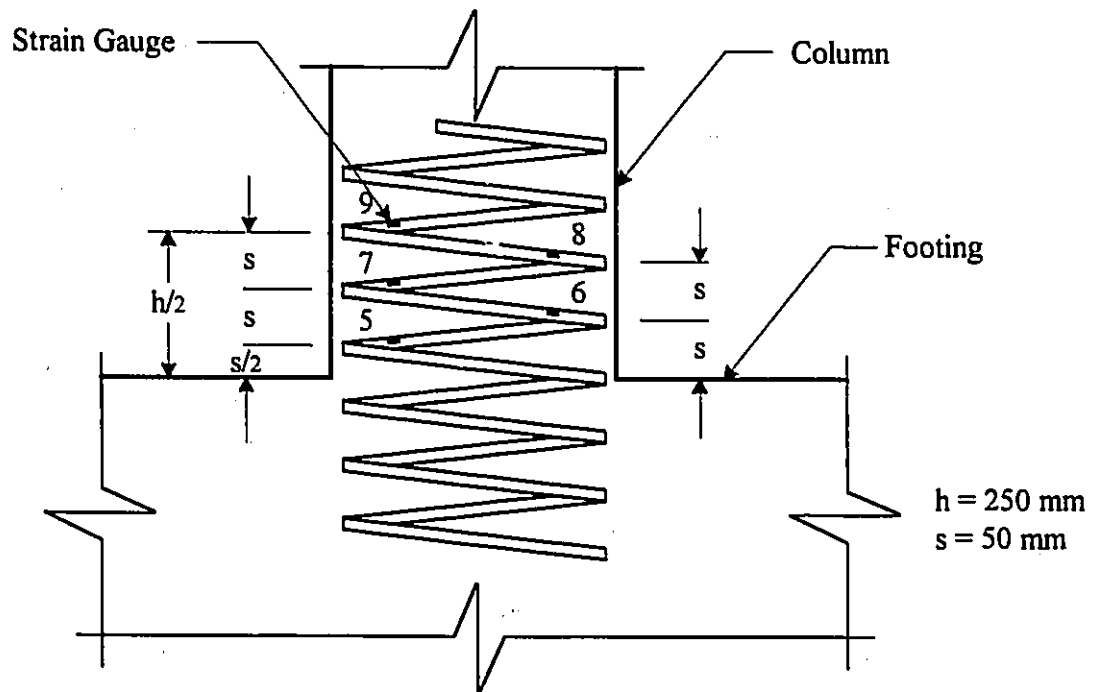
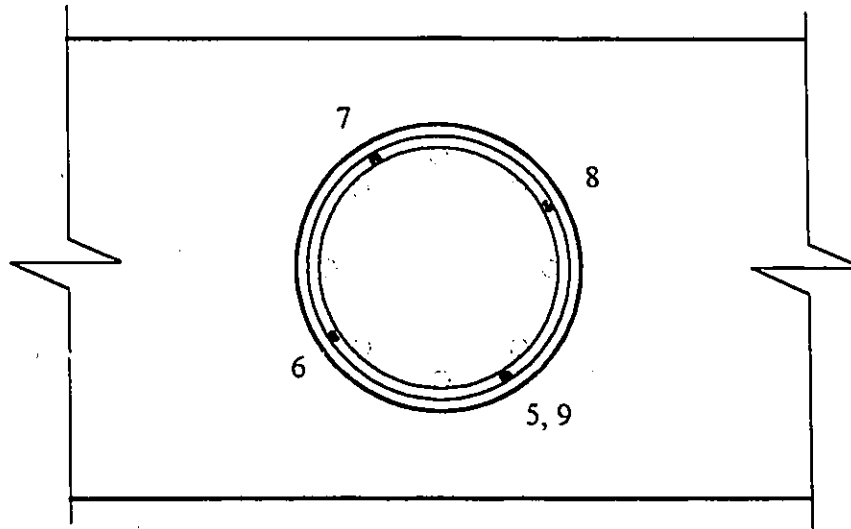


Figure 2.34: Location of Strain Gauges on Transverse Steel
(Specimens RC-1 to RC-5, RC-7, and RC-9)

Plan View



Elevation View

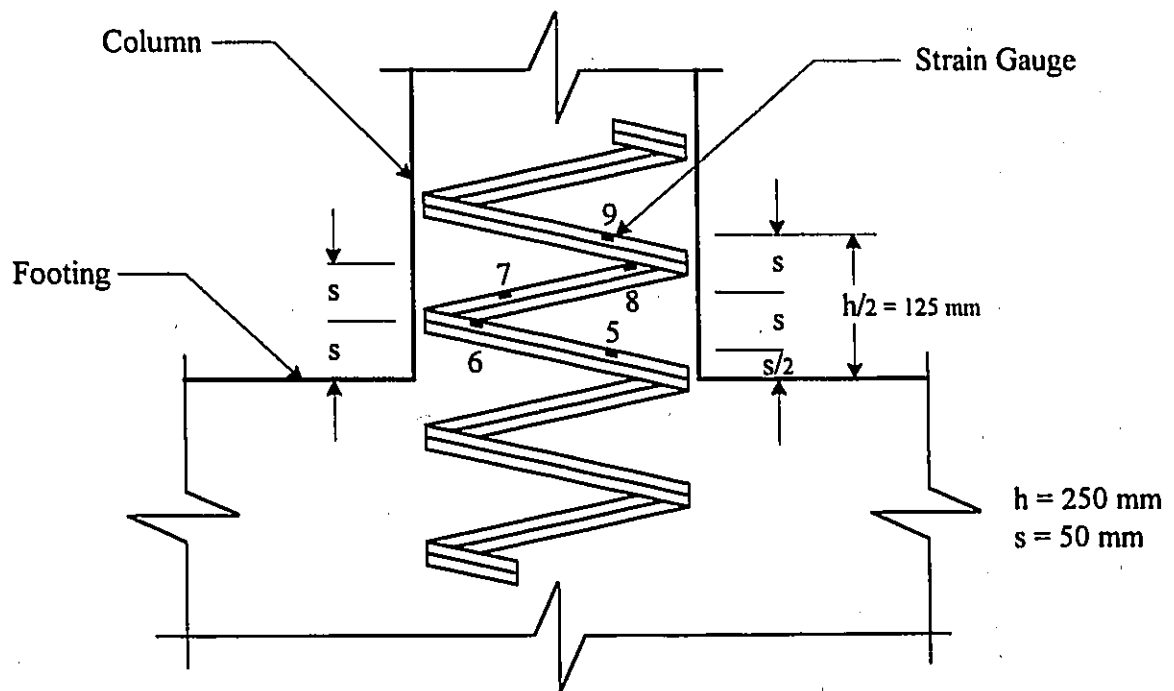
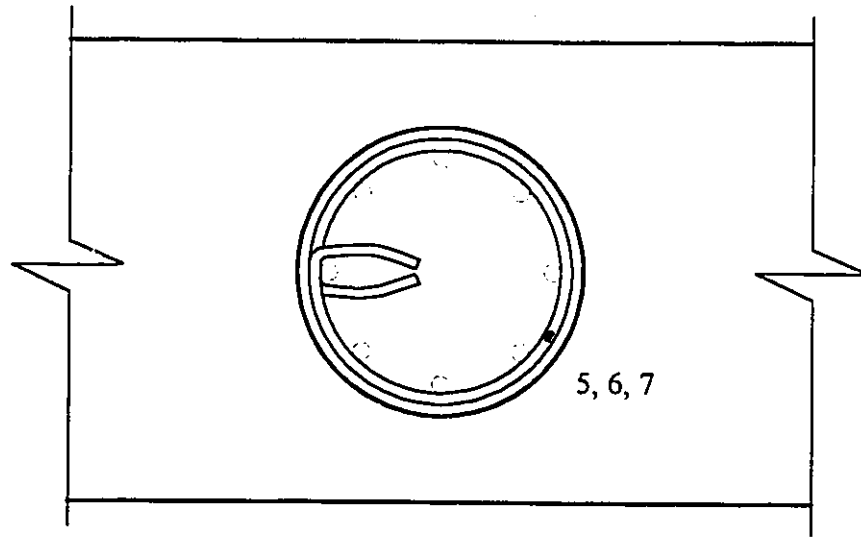


Figure 2.35: Location of Strain Gauges on Transverse Steel (Specimen RC-6)

Plan View



Elevation View

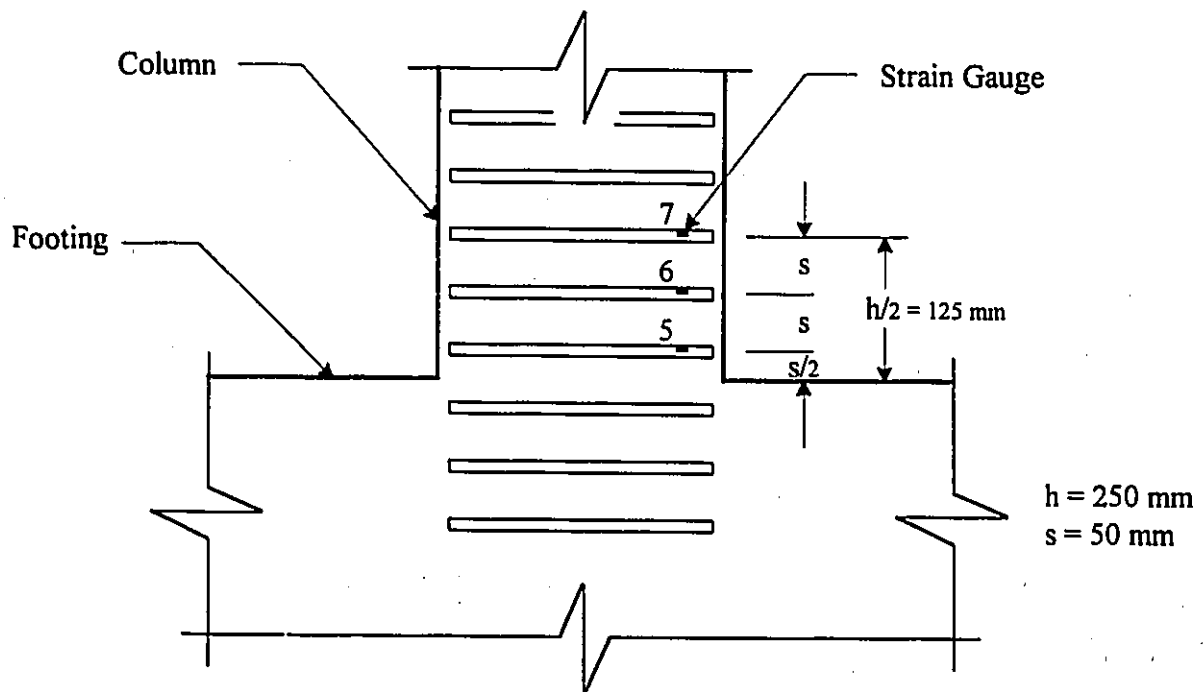


Figure 2.36: Location of Strain Gauges on Transverse Steel (Specimen RC-8)

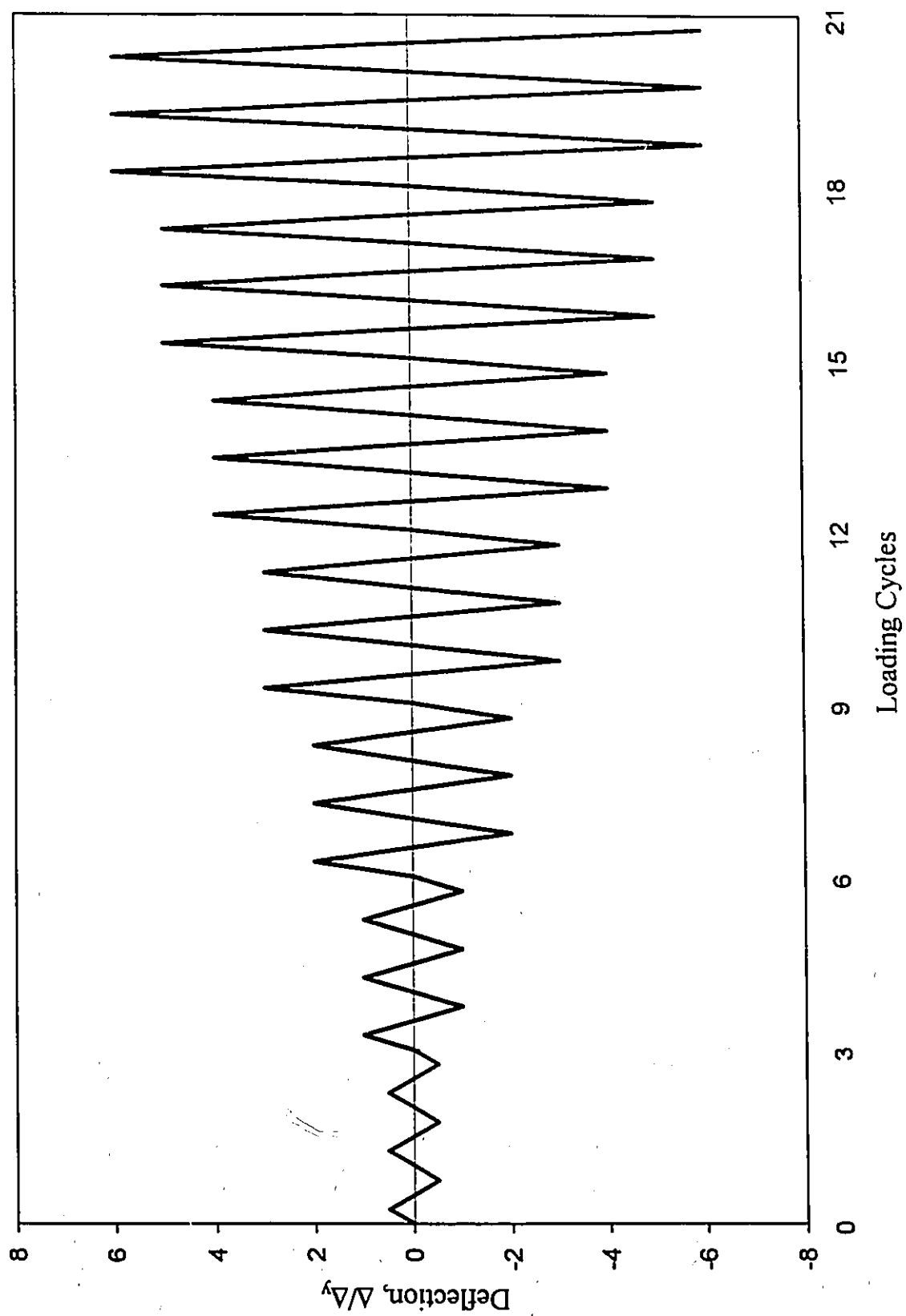


Figure 2.37: Horizontal Load Path

CHAPTER 3

Test Results and Evaluation of Data

3.1 General

Experimental observations made during column tests are presented and discussed in this chapter. Strength and deformation characteristics of columns are illustrated using hysteretic force-displacement and moment-rotation relationships. Strains in both the longitudinal and transverse reinforcement, recorded at different stages of testing, are also discussed. These observations and the data recorded are used in Chapter 4 to determine the effects of test parameters on the behaviour of circular high-strength concrete columns. Plots illustrating the distribution of strains in reinforcing steel are presented in Appendix A. The appendix also includes the hysteretic force-deformation relationships for each specimen recorded immediately below the loading beam as well as immediately above the potential hinging region.

3.2 Observed Behaviour

Observed behaviour of each specimen is discussed based on visual observations and data recorded during testing. The hysteretic force-displacement relationships are presented in two different plots, one with and the other without the $P-\Delta$ effect. The axial force was applied to be vertical initially, prior to the application of horizontal force. However, the axial force continuously changed its direction as the column deformed laterally, always pointing towards the bottom swivels of vertical actuators, which were located 590 mm below the column base (Fig. 3.1). The horizontal component of the axial force was sensed and recorded by the horizontal actuator while the vertical component produced $P-\Delta$ moments. Therefore, plotting the recorded horizontal forces resulted in force-displacement hysteretic relationships with the $P-\Delta$ effect. The vertical

component of axial force, multiplied by the accompanying horizontal displacement Δ produced the P- Δ moment. The P- Δ moment divided by the shear span resulted in a horizontal force component, which was added to the horizontal actuator force to obtain the hysteretic relationships without the P- Δ effect.

3.2.1 Specimen RC-1

Specimen RC-1 was one of the two specimens cast with *65 MPa* concrete. The transverse reinforcement consisted of a plain spiral steel with a yield strength of *1000 MPa* and a pitch of *50 mm*. The column was subjected to a constant axial compression of *1000 kN*, producing an axial load level of $0.30P_o$. The extent of damage at progressive stages of testing can be seen in the photographs of Figure 3.2. All pictures were taken after the first cycle of each load stage.

The yield displacement of the column was determined during testing to be $\Delta_y = 25 \text{ mm}$. This value corresponded to 1.5% lateral drift, and represented column displacement at the point of application of horizontal force. At yield displacement the column yielded as a whole, as indicated by a significant change in the slope of the force-displacement relationship, which was monitored throughout the test. No damage was observed during the three displacement cycles of this load stage. Several hairline flexural cracks were noticed on the opposite sides of column, perpendicular to the direction of loading. These cracks were widely spaced and continued up to approximately half of the column height. Strains in the extreme longitudinal bars reached values as high as 1.25% at a height of 125 mm above the footing surface. No yielding was observed at the footing level or 125 mm below its surface. The transverse steel did not begin yielding at this stage, however, the strains were higher in the gauges located further above the footing than in those near the footing. This implies a higher lateral expansion of concrete at locations above the base.

A significant change in behaviour was observed at the $2\Delta_y$ load level (3% drift). The existing flexural cracks became slightly wider and signs of cover spalling were noticed in the form of vertical cracking. These cracks extended up to approximately a quarter of the column height. They indicated further lateral expansion of the cover concrete, and hence an increase of strains in confinement steel. Yielding of transverse reinforcement was observed with the strains ranging from 0.10% to 0.35% in gauges 5 through to 8 respectively. Gauge 5 was located 25 mm above

the footing, while Gauge 8 was located at 100 mm. The extreme longitudinal bars started to yield at the level of the column base, however, yield penetration into the footing was not observed. Furthermore, spalling of cover concrete was noticed near the footing surface.

An increase in the longitudinal and flexural cracking, as well as the formation of diagonal cracks on the side faces of the column were observed during the cycles of $3\Delta_u$. Severe damage was also observed as the cover concrete in the hinge region spalled off, exposing confinement steel. The maximum transverse steel strain reached at this load level was 0.56%. No yielding of longitudinal reinforcement was recorded inside the footing.

Further damage was sustained at $4\Delta_u$, with the spalling of concrete extending beyond the plastic hinge region. With the exception of Strain Gauge 5, all gauges indicated yielding of the confinement steel. The lateral force degradation at this load stage was only 6% when P- Δ effects were ignored. However, the lateral load resistance was entirely lost just before the $4\Delta_u$ level, when P- Δ effects were considered. Yield penetration of longitudinal reinforcement into the column footing was recorded at $5\Delta_u$. Distribution of strains in longitudinal and transverse reinforcements is shown in Appendix A. Testing was ceased after the completion of one cycle at $6\Delta_u$, corresponding to a lateral drift of 9%.

The hysteretic force-displacement relationships are presented in Figures 3.3 and 3.4. These figures reflect the behaviour without and with P- Δ effects, respectively. The relationships indicate that the column never dropped to 80% of its lateral strength capacity when P- Δ effects were ignored. When these effects were accounted for, the drift capacity was observed to be 2.9% at 20% decay. Hysteretic moment-total rotation and moment-anchorage slip rotation relationships for column RC-1 are shown in Figures 3.5 and 3.6, respectively. The anchorage slip rotation was measured with a gauge length of 25 mm and therefore include flexural deformations within the bottom 25 mm segment. Furthermore, the yield penetration into the footing was not observed until after the $5\Delta_u$ displacement level and hence may be negligibly small. Therefore, almost all of the rotations measured can be attributed to flexure. Figures 3.5 and 3.6 indicate that at the end of the test, approximately 1/8 of the total rotation was caused by anchorage slip, resulting in 7/8 being attributed to flexure.

3.2.2 Specimen RC-2

Column RC-2 was the second of the two specimens cast with 65 MPa concrete. This column was confined with a No.10 spiral having a yield strength of 419 MPa and a spacing of 50 mm . It was also tested under a constant axial load level of $0.30P_o$. Figure 3.7 illustrates the extent of column damage at different stages of loading.

The yield displacement Δ_y , estimated during testing, was 20 mm . During this load stage no damage was observed except for the formation of several flexural cracks. The cracks were observed approximately within the bottom one third of the column. No yielding of transverse steel was observed and the maximum strain was measured to be 0.08% at 100 mm above the column footing. The longitudinal bars exceeded the yield strain at both column-footing interface and at 125 mm above the footing where 0.60% strain was recorded. It was not possible to obtain any strain readings within the footing due to damage to Gauge 1.

The existing flexural cracks became wider at $2\Delta_y$, and several new cracks formed further up the column height. Vertical cracks began forming in the plastic hinge region along the front and rear sides of the column indicating lateral expansion and spalling of cover concrete. Some spalling of cover concrete was also noticed near the footing. The strain in longitudinal reinforcement reached 1.10% at 125 mm above the base. Yielding of transverse reinforcement was reached, with strains ranging from 0.05% to 0.25% at and above the column-footing interface. No strength degradation was observed at this deformation level when the P- Δ effect was not considered and the lateral drift was 2.4% . The lateral load capacity was reduced by 15% when the P- Δ effect was considered. The formation of diagonal cracks was observed during the cycles of $3\Delta_y$ on column faces parallel to the direction of loading. The longitudinal and flexural cracks propagated further during this level of deformation and the cover concrete in the hinge region completely spalled off. Progression of cover spalling was observed during subsequent cycles. Longitudinal steel strain of 0.35% was reached at column-footing interface, implying that yield penetration into the footing was initiated. The maximum transverse steel strain reached at this load level was 0.39% . The column capacity dropped by 38% during this load stage when P- Δ effects were considered at 3.6% lateral drift.

Complete spalling of the cover concrete extended at $4\Delta_y$, up to about 500 mm above the column-footing interface. This distance is twice the theoretical plastic hinge length and approximately a third of the column height. At $5\Delta_y$, the tensile strain in confinement reinforcement 125 mm above the footing (Gauge 9) exceeded 0.60%. Figures in Appendix A illustrate the distribution of strains along the longitudinal and transverse reinforcement at different stages of deformation. The strength decay at $5\Delta_y$ was limited to 6% when P- Δ effects were not accounted for. The lateral strength decay however, exceeded the column capacity when P- Δ effects were considered. Testing continued until three cycles at $6\Delta_y$ were completed, corresponding to a lateral drift of 7.3%.

Figures 3.8 and 3.9 provide complete hysteretic force-displacement relationships obtained from test data without and with P- Δ effects, respectively. The relationships indicate that the column never dropped to 80% of its lateral capacity when P- Δ effects were ignored. When these effects were considered, the drift capacity was observed to be 2.6% at 20% strength decay. Hysteretic base moment-total hinge rotation and base moment-anchorage slip rotation relationships are shown in Figures 3.10 and 3.11, respectively. It can be concluded that approximately 1/4 of the total rotation was caused by anchorage slip, and 3/4 by flexure.

3.2.3 Specimen RC-3

Column RC-3 was the first of the seven specimens cast with 90 MPa concrete. With the exception of the concrete strength and the axial load level applied, this column was identical to RC-1. It was subjected to a constant axial force of 1850 kN, producing an axial load level of $0.43P_o$. The extent of column damage at different stages of loading is presented in Figure 3.12.

The yield displacement observed during testing was $\Delta_y = 20$ mm. After the first cycle of this load stage the formation of several flexural cracks was observed in the vicinity of the potential plastic hinge. Some vertical and diagonal cracks also appeared within the next two cycles. These cracks were mostly confined to the hinging region. Spalling of the cover occurred near the base on both sides of the column, approximately 100 mm above the footing. No yielding of longitudinal reinforcement was recorded at column-footing interface. However, at 125 mm above the base the strain in the extreme tension bar reached 1.0%. The maximum strain in the transverse steel at the

end of $1\Delta_y$ was 0.08% at 125 mm above the footing (Gauge 9).

Upon reaching the $2\Delta_y$ displacement (2.4% drift), the cover started to spall with small pieces popping off. A strain of 0.27% was recorded by Gauge 8 located at 100 mm above the base, signifying yielding of transverse reinforcement. The spiral steel below this level remained elastic during this deformation level. This is in agreement with the observed behaviour since spalling of cover occurred somewhat above the column-footing interface. Longitudinal steel began yielding at the base of the column, but no penetration into the footing was observed. By the end of this deformation stage, cover concrete crushed up to a third of the column height. Although there was little strength decay in moment capacity, the lateral load resistance with P- Δ effects exhibited a reduction of 30% .

During the $3\Delta_y$ cycles the spalling of cover extended towards the footing as well as further above the hinging region. At this point, only the Strain Gauge 1 on longitudinal steel inside the foundation provided data. The confinement steel near the base of the column never yielded, however, at 125 mm above the footing the strain exceeded 0.50% . Upon reaching the $4\Delta_y$ deformation level, complete spalling of the cover concrete was observed around the entire perimeter of the column, extending approximately half way up the column. This distance is equal to more than three times the cross-sectional dimension of column, which may be regarded as the theoretical plastic hinge length. The column showed a sudden strength decay of 21% at $5\Delta_y$ (6.1% drift), without considering secondary effects. Testing was concluded after one cycle at this stage since failure was imminent. Yielding of flexural reinforcement did not penetrate into the footing during testing.

The hysteretic force-displacement relationships are shown in Figures 3.13 and 3.14. The relationship without P- Δ effects indicates that the column strength dropped to 80% of its lateral capacity at a drift of 6% . The drift capacity of this column was observed to be only 2% , at a 20% decay in its lateral force resistance when P- Δ effects were accounted for. Figures 3.15 and 3.16 contain the moment-rotation relationships for this specimen. These results indicate that anchorage slip rotations make up for only a negligible portion of the total hinge rotation, and hence the total rotation was almost totally caused by flexure. This conclusion agrees with the fact that no significant yield penetration into the footing was observed.

3.2.4 Specimen RC-4

Column RC-4 was cast with 90 MPa concrete and confined with plain spiral steel, 8 mm in cross sectional diameter. The yield strength of this steel was 580 MPa and the pitch was 50 mm . This column was comparable to Specimen RC-5 with the exception of the volumetric ratio of confinement steel, which was 1.81% . The extent of damage at progressive stages of testing can be viewed in the photographs of Figure 3.17.

The yield displacement Δ_y , established during the test, was 18 mm . Spalling of the cover began at this load stage near the base at approximately 100 mm above the footing. Strains in extreme longitudinal reinforcement 125 mm above the column-footing interface reached a value of as high as 1.1% . The steel was also close to yielding at the footing level. At 125 mm below the footing the strains reached 0.1% . The transverse steel gauges indicated only negligible strains at this stage.

A significant change in behaviour was observed at $2\Delta_y$ (2.2% drift). Spalling of cover extended beyond the hinge region, and further into the sides of the column. Transverse steel strains remained very low during this stage especially near the footing (Gauges 5,6, and 7). This agrees with the fact that crushing of concrete had not occurred within this region, indicating very little lateral expansion. The strains in Gauge 8, which coincided with the bottom limit of the observed spalling region, was considerably higher (0.1%). Yielding of longitudinal reinforcement was observed at the level of the column base from the outset of this deformation stage. The lateral strength degradation at the end of this phase was equal to 25% of its capacity when P- Δ effects were accounted for.

Upon reaching the $3\Delta_y$ displacement level, the cover concrete had started to spall in a more violent manner. Small sharp pieces of concrete began projecting away from the column surface. At a depth of 125 mm inside the footing a strain of 0.18% was recorded for the longitudinal steel, indicating significant yield penetration. Although the transverse reinforcement strains continued to remain very low near the footing, Gauge 8 located at 100 mm above the column base, read a value of 0.30% during the first cycle. It was also observed that the column began leaning to one

side in the direction oblique to lateral loading. This action became increasingly more apparent as the test progressed. Nevertheless, no lateral strength decay was noticeable when secondary effects were ignored. Just before the completion of this load stage Strain Gauge 8 was damaged. However, the last value of strain recorded was approximately 3%, implying a very high radial expansion of the hinging region and very likely a loss in stability of longitudinal reinforcement.

The next deformation level ($4\Delta_y$) was never reached because of a sudden and explosive failure of the column hinging region. During the application of displacement towards $4\Delta_y$, the monitored lateral force began to decrease gradually. Upon reaching the previous $2\Delta_y$ displacement, the lateral strength had already dropped by at least 10% in only a few seconds. Then, at a displacement of 47 mm or a drift of 2.9% ($3\Delta_y$ occurred at 54 mm), the force suddenly fell to approximately 50% of the lateral capacity (without P- Δ effects). It then increased to 62% of its capacity at which point the failure occurred. The sudden drop and increase took place in about 6 seconds but failure was instant and typical of brittle behaviour. The dramatic column failure that occurred at a lateral drift of 3.5% was extremely loud and it caused a very perceptible vibration of the laboratory strong floor. Figure 3.18 illustrates the condition of the test specimen from different views after the failure. It was observed that the two levels of transverse reinforcement, directly above Gauge 8 level, completely ruptured allowing the longitudinal reinforcement to buckle outwards. The entire column began to collapse at that point, however, due to the displacement limits imposed on the two vertical actuators the testing was aborted automatically.

The hysteretic force-displacement relationships obtained from test data are presented in Figures 3.19 and 3.20. These figures reflect the behaviour of the column without and with P- Δ effects, respectively. The relationships indicate that the column strength dropped to 80% of its lateral capacity shortly before failure, at a drift of 2.9%, when P- Δ effects were ignored. The drift capacity was observed to be 2% at 20% strength decay when P- Δ effects were considered. Hysteretic moment-total hinge rotation and moment-anchorage slip rotation relationships are shown in Figures 3.21 and 3.22 respectively. They indicate that approximately 1/6 of the total rotation was caused by anchorage slip, resulting in 5/6 being attributed to flexure. This is in agreement with the longitudinal steel yield penetration into the footing observed during testing.

3.2.5 Specimen RC-5

Column RC-5 was reinforced in exactly the same manner as column RC-2. This specimen only differed in that it was made of 90 MPa concrete and that it was tested under a constant axial load level of $0.43P_0$. The extent of column damage at selected stages of lateral deformation is illustrated in Figure 3.23.

A yield displacement (Δ_y) of 17 mm was established during testing, which corresponded to a lateral drift of 1% . No visible flexural cracking or spalling of cover was observed at any point during the three cycles at this level. Strain data for longitudinal reinforcement above the footing was not available. The strain in the extreme longitudinal bars at column-footing interface however, has reached yield (0.2%). The maximum strain in transverse steel was recorded to be 0.06% at 75 mm above the footing (Gauge 6).

At $2\Delta_y$, corresponding to a drift of 2% , several diagonal cracks appeared on the side faces of column. Loud cracking sounds were heard and the cover began to spall in large pieces. The spalling extended from the base of the column up to a column height approximately equal to the cross-sectional dimension. A 20% reduction in lateral load capacity was observed at this point when $P-\Delta$ effects were accounted for. The confinement steel reached a strain of 0.16% as recorded by Gauge 8, located at 100 mm above the footing.

During $3\Delta_y$, the spalling of cover extended into the side regions of the column. The strains in the flexural reinforcement at the base and 125 mm inside the footing were 0.34% and 0.15% , respectively. Strain Gauge 8, located on transverse reinforcement at 100 mm above the footing indicated a sharp rise in strains up to 0.55% . No lateral force degradation was observed at the end of the three cycles when $P-\Delta$ effects were disregarded. More yielding of longitudinal and transverse reinforcement was observed during the cycles of $4\Delta_y$. Furthermore, the specimen started to lean to one side in a direction perpendicular to the applied lateral force due to uneven crushing of column concrete. Accounting for secondary effects, the column lost its entire lateral strength before the end of this load level. At $5\Delta_y$, the transverse steel developed a strain of 1.2% , indicating high lateral expansion of the concrete. Testing was halted after one complete cycle at $5\Delta_y$, because of the rapidly increasing out-of plane instability of column.

Complete hysteretic force-displacement relationships are shown in Figures 3.24 and 3.25. When P- Δ effects were ignored the moment resistance never dropped to 80% of its capacity. However, the drift capacity was observed to be 2% at 20% decay of lateral load resistance when the secondary moments due to the P- Δ effect were considered. Figures 3.26 and 3.27 show moment-total rotation and moment anchorage-slip rotation relationships. These figures indicate that less than 1/8 of the total rotation can be attributed to anchorage slip, resulting in more than 7/8 being caused by flexure. This is in line with the observation that some yield penetration into the footing did occur at later stages of loading.

3.2.6 Specimen RC-6

The properties of column RC-6 were the same as those of RC-5 except for spacing. Column RC-6 was confined with two No. 10 spirals at a spacing of 100 mm. This produced the same volumetric reinforcement ratio in both columns. Figure 3.28 illustrates the progression of damage observed at different stages of testing.

The yield displacement was established as $\Delta_y = 17 \text{ mm}$, coinciding with 1% lateral drift. The cover concrete began to spall in the form of buckling of the shell concrete in several places. Concrete crushing could be heard during each load cycle. No flexural cracks were visible. The longitudinal steel at the base reached 0.13% strain. The bars 125 mm above the footing were yielding with Gauge 3 recording 1.25% strain. Steel strains inside the footing were not available due to strain gauge damage. The maximum transverse strain recorded during this load stage was 0.08%, and was recorded by Gauge 6.

Further spalling of cover was observed upon reaching $2\Delta_y$ at 2% lateral drift. Large rectangular pieces of cover concrete fell off from the surface of the column. An increase in transverse steel strain was observed with the upper gauges approaching yielding. Cover spalling extended up to a third of the column before the end of this load stage. Although there was no strength degradation when P- Δ effects were ignored, approximately 35% loss in capacity was observed when P- Δ effects were considered. The column began to sway sideways near the end of the cycles at $2\Delta_y$.

The spalling of cover extended around the entire perimeter of the column and up to about half of its height during $3\Delta_y$ deformation cycles. The strain in flexural reinforcement exceeded yield at column-footing interface, and reached 2% at 125 mm above the base. The confinement reinforcement began to yield above the base of the column. Testing had to be aborted after only one half cycle at $4\Delta_y$ due to sudden buckling of some of the longitudinal bars. Complete spalling of the cover concrete extended to approximately $2/3$ of the column height before the test was stopped. The longitudinal steel at footing level had just yielded when testing was ceased. This implies that yield penetration into the foundation did not occur in this column.

Hysteretic force-displacement relationships with and without P- Δ effects are shown in Figures 3.29 and 3.30, respectively. The column resistance without P- Δ effects never dropped to 80% of its capacity, even at 4% lateral drift when the test was stopped. When P- Δ effects were included however, the drift capacity was only 1.5% at 20% strength decay. The base moment-rotation relationships for specimen RC-6 are presented in Figures 3.31 and 3.32. These results indicate that anchorage slip rotations made up only a negligible portion of the total hinge rotation, while the rotations due to flexure account for almost the total amount recorded. This conclusion agrees with the fact that no yield penetration into the footing was observed before the end of the test. The strain gauge data recorded during testing are presented in Appendix A.

3.2.7 Specimen RC-7

Column RC-7 was a duplicate of Column RC-3 and was tested to investigate the effect of axial compression on ductility of HSC columns. The applied axial compression was 925 kN, which corresponded to 50% of the load applied on Column RC-3. The resulting axial load level was equal to $0.21P_o$. The transverse reinforcement in this column consisted of a 1000 MPa plain steel spiral with 50 mm pitch. Damage sustained by the column at progressive stages of testing is presented in Figure 3.33.

The yield displacement Δ_y , determined during testing was 20 mm. No visible damage or cracking was observed at any point during this load stage. Furthermore, the strains in extreme longitudinal bars were limited to 0.38% at 125 mm above the footing. This was much lower than the values of strain ($>1.0\%$) reached in all other specimens tested under high axial compression. Yielding

of longitudinal bars at column-footing interface was not observed. Similarly, no yielding was observed in transverse steel. Several flexural cracks were observed at $2\Delta_y$. These cracks were mainly distributed near the base of the column. Spalling of the cover was initiated shortly after the first cycle at $2\Delta_y$, with several small pieces popping away from the column surface. Yielding of transverse reinforcement was then recorded with strains ranging from 0.03% to 0.25% in Gauges 5 through to 9. Longitudinal reinforcement strains increased rapidly during this load stage and yielding began to penetrate into the footing. The tensile strains reached 0.5% and 1.0% at footing level and 125 mm above the footing, respectively. By the end of the third cycle at $2\Delta_y$, spalling progressed up to a distance approximately equal to the cross sectional dimension. The lateral force capacity decreased by only 2% at this stage, even when P- Δ effects were considered, corresponding to a lateral drift of 2.4%.

Cover spalling progressed further during displacement cycles at $3\Delta_y$. The strength was reduced to 71% of its capacity after the first cycle of this deformation level when P- Δ effects were considered. No noticeable difference in behaviour was observed at $4\Delta_y$, with the exception of an increase in transverse steel strains near the base, where yielding was exceeded. The strain data for longitudinal steel indicated yield penetration deep into the footing. The strain in longitudinal reinforcement, 125 mm into the footing was 0.23%.

At $5\Delta_y$ (6% drift) the lateral force degradation was equal to 11% of the capacity without the secondary effects. Spalling continued at a very gradual rate as it extended further up the column. During the cycles of $6\Delta_y$, the longitudinal reinforcement strains inside the footing were always higher than 0.25% at 125 mm below the column-footing interface, indicating deeper yield penetration. At 50 mm above the column base the maximum strains recorded in confinement steel was approximately 0.50%. This shows that under lower axial load levels transverse reinforcement attains high strains at levels much closer to the base of the column than under higher axial loading. By the end of this load stage the cover concrete spalled off around the full perimeter of the column, between the footing and a distance about twice the cross-sectional dimension. Testing was terminated after one complete cycle at $7\Delta_y$, corresponding to a drift of 8.5%, when the lateral strength of the column decreased to 78% of its capacity without P- Δ effects.

Figures 3.34 and 3.35 show the hysteretic force-displacement relationships obtained from test data. The relationships indicate that the column dropped to 80% of its lateral strength capacity at a drift of 7.7%, disregarding P- Δ effects. When these effects were accounted for however, the drift capacity was observed to be 3.3%, at a 20% decay in its lateral force capacity. Hysteretic moment-total hinge rotation and moment-anchorage slip rotation relationships for this column are shown in Figures 3.36 and 3.37 respectively. Since yield penetration into the footing was observed, it can be concluded that anchorage slip rotation was a significant part of the total rotation. The above mentioned figures indicate that at least 1/8 of the total rotation was attributed to anchorage slip, resulting in 7/8 being caused by flexural rotation.

3.2.8 Specimen RC-8

Every parameter of Column RC-8 was the same as that of Column RC-3. The difference between the two specimens lied in the fact that Column RC-8 was confined with circular hoops instead of continuous spirals. The hoop ends were anchored into the core concrete with 90° hooks with 6 bar-diameter ($6d_b$) extensions. This was in agreement with the hook anchorage length prescribed by the CSA A23.3-1984 standard. The column was subjected to an axial load level of $0.43P_o$. The amount of column damage observed at different stages of lateral loading can be seen in Figure 3.38.

The yield displacement was determined during testing to be $\Delta_y = 18 \text{ mm}$. Some diagonal cracking was noticed at the sides of the column near the footing. Only one flexural crack was visible at this load stage, approximately 250 mm above the column-footing interface. Spalling of cover concrete started during the next two cycles. No yielding of longitudinal reinforcement was recorded at or below the footing level. However, at 125 mm above the base the strain in the extreme bar reached 1.1%. The maximum strain in the transverse steel at the end of $1\Delta_y$ was 0.09%, measured at 125 mm above the footing (Gauge 9).

The cover concrete started to spall extensively at $2\Delta_y$ (2.2% drift), with small pieces of concrete projecting away from the column. Transverse steel strain of 0.21% was recorded by Gauge 7, 125 mm above the base, indicating yielding of transverse reinforcement. The confinement steel below this level did not yield and values of strain remained below 0.02%. Longitudinal steel strains

increased only slightly during this stage. Spalling progressed very rapidly and by the end of this deformation level several large pieces of concrete had fallen away from the column up to at least a third of its height. There was no spalling observed on the side faces. Although no strength degradation was observed when P- Δ effects were ignored, considering them revealed a 25% reduction in strength.

Further increase in spalling was observed at $3\Delta_u$, extending over the entire perimeter of the column. Inelastic strains in confinement steel continued to increase quickly at about 125 mm above the base while the strains closer to the footing remained low at 0.04%. This observation is true for all the specimens tested under high axial compression. Crushing of concrete penetrated deeper into the confined core area at $4\Delta_u$, exposing the hoops. This condition revealed that some of the unsupported hoop segments, between the longitudinal bars, have almost completely straightened out in tension. The longitudinal reinforcement showed clear evidence of buckling. Data from the two vertical actuators verified this loss of stability by showing a slight amount of sudden column shortening. Furthermore, the column also began to lean to one side in the direction perpendicular to the load path. Strains of 0.50% were recorded in the transverse reinforcement at 100 mm from the column base. Near the footing however, the maximum confinement steel strain never went beyond 0.06%. Also, the longitudinal reinforcement never yielded at the base of the column or inside the footing, indicating that yield penetration had not occurred. Testing was concluded at the end of this stage because of the ever increasing instability of longitudinal reinforcement. Disregarding secondary effects, the lateral strength capacity of this column decreased by 10%.

The hysteretic force-displacement relationships are shown in Figures 3.39 and 3.40. The relationship without P- Δ effects shows that the column strength never dropped to 80% of its lateral capacity. However, the drift capacity at 20% reduction in the lateral resistance of this column was observed to be 2% when p- Δ effects were accounted for. Figures 3.41 and 3.42 contain the base moment-rotation relationships for this specimen. The results indicate that anchorage slip rotations make up for only a negligible portion of the total hinge rotation, and hence the total rotation was almost totally caused by flexure. This conclusion agrees with the fact that no yield penetration into the footing was observed.

3.2.9 Specimen RC-9

Column RC-9 was unique from all others in that it was built without concrete cover. Because sona tubes of a diameter necessary to eliminate the cover concrete were not available, the core diameter was instead increased to fit the tube. Since the outside diameter of this column is the same as in all other specimens (250 mm), the resulting increase in core area caused a slightly lower volumetric ratio of transverse steel than in comparable specimens confined with No. 10 spirals. However, all other properties, such as the concrete strength, confinement spacing, and the axial load level used, were identical to those of Column RC-5. The thin layer of concrete present around the reinforcement was approximately 2 to 3 mm in thickness. Figure 3.43 illustrates the extent of column damage observed at different stages of testing.

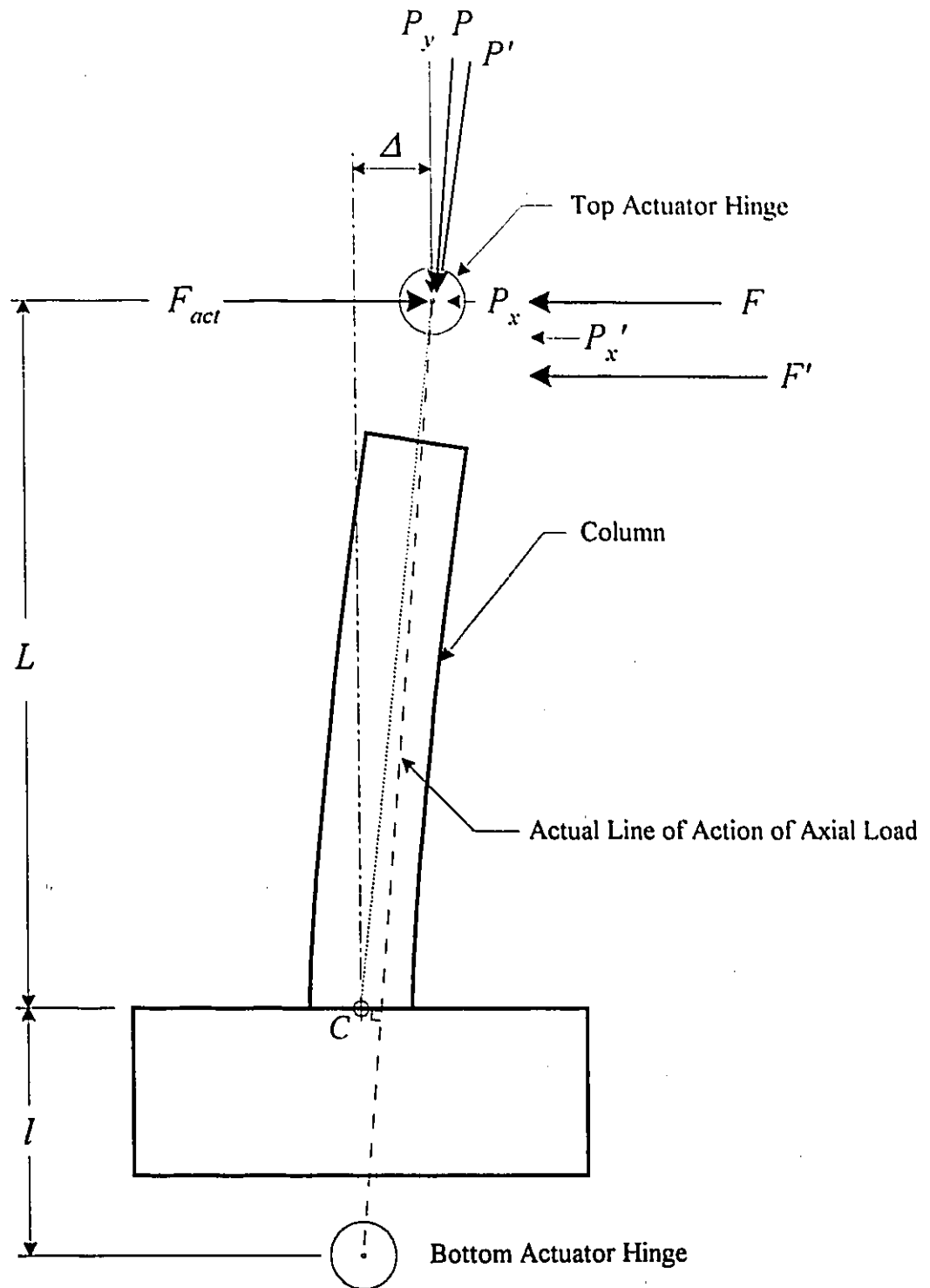
The yield displacement Δ_y was established to be 18 mm. No damage was observed during this load stage, and the maximum strain in transverse reinforcement was less than 0.06%, 100 mm above the footing. Yielding of longitudinal reinforcement was recorded at 125 mm above the footing where the tensile strain was 0.21%.

The $2\Delta_y$ load level (2.2% drift) marked a significant change in the behaviour. First of all, a large portion of the thin concrete shell simply fell off from both the front and rear sides of the column. Secondly, yielding was observed in the longitudinal reinforcement at the column base (0.22%), as well as in the confinement steel at 100 mm above the footing (Gauge 8). Strains in the spiral close to the footing remained very low at about 0.03%. Finally, by the end of the first cycle, the lateral force resistance of the column degraded by 20% of its capacity.

During the cycles of $3\Delta_y$, strains in the reinforcement increased significantly. The maximum transverse steel strain attained during this load level was 0.55%, while at the footing level it continued to be very low. The longitudinal bars at the base of the column exceeded tensile strains of 0.50%, indicating yield penetration into the footing. At 125 mm below the surface of the footing strains were just below yield. An abrupt change in the transverse steel strains was observed just after progressing into the $4\Delta_y$ stage. Strains in the spiral steel near the foundation, which up to this point remained very low, increased rapidly to values greater than 0.65% at 50 mm above the base (Gauge 6). Above this region all gauges ceased functioning as strains

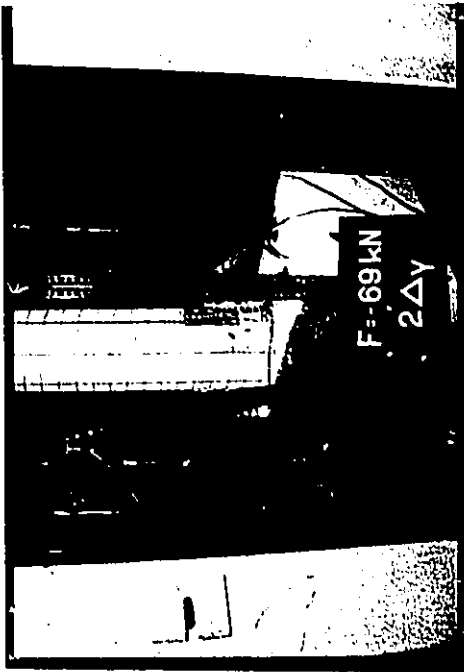
exceeded 1.0%. At $5\Delta_y$, more lateral expansion of the hinging region was noticed as the crushing of concrete projected deeper into the confined core. Although near the bottom of the column some of the transverse steel became completely exposed, it was not possible to see whether longitudinal bars began to buckle. Figures in Appendix A illustrate the distribution of strains along the longitudinal and transverse steel at different deformation stages. Testing continued until one cycle at $6\Delta_y$ was completed, corresponding to a lateral drift of 6.6%.

Figures 3.44 and 3.45 provide hysteretic force-displacement relationships obtained from test data, without and with P- Δ effects, respectively. The relationships indicate that the column never dropped to 80% of its lateral strength capacity when P- Δ effects were disregarded. Considering these effects, the drift capacity of this column was observed to be 2.2%, at 20% strength loss in lateral capacity. Hysteretic moment-total hinge rotation and moment-anchorage slip rotation relationships are shown in Figures 3.46 and 3.47, respectively. The figures indicate that approximately 1/8 of the total rotation was caused by anchorage slip, and 7/8 by flexural rotation.

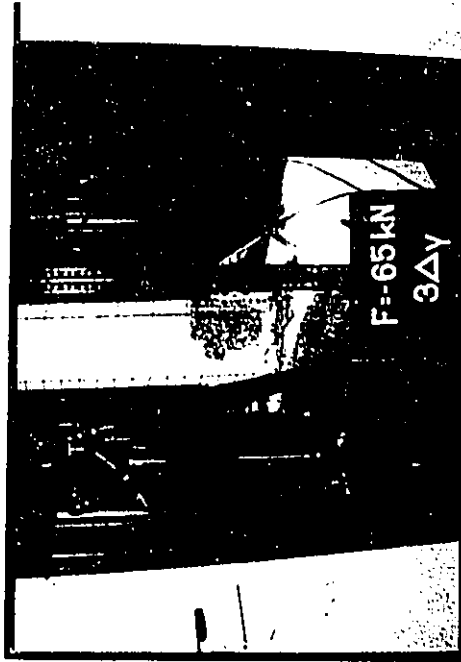


$L = 1645 \text{ mm}$; $l = 590 \text{ mm}$; Δ = lateral column displacement

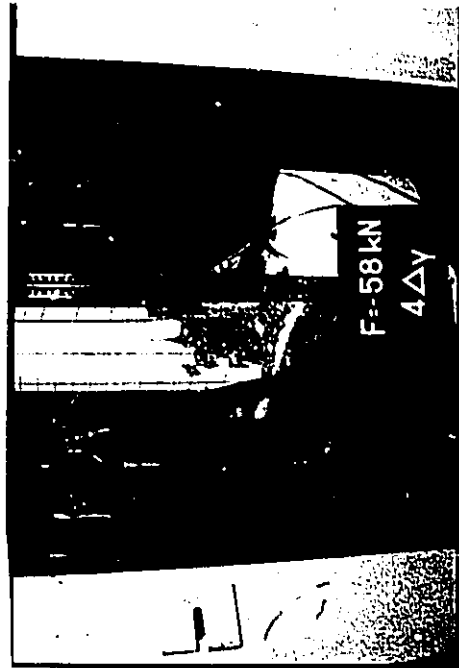
Figure 3.1: System of Forces Acting on a Typical Specimen



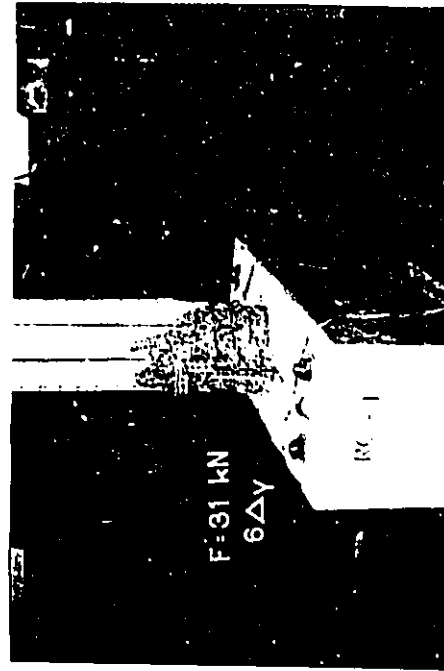
(a) $2\Delta_y$



(b) $3\Delta_y$



(c) $4\Delta_y$



(d) $6\Delta_y$

Figure 3.2: Extent of Damage in Column RC-1 at Different Stages of Testing

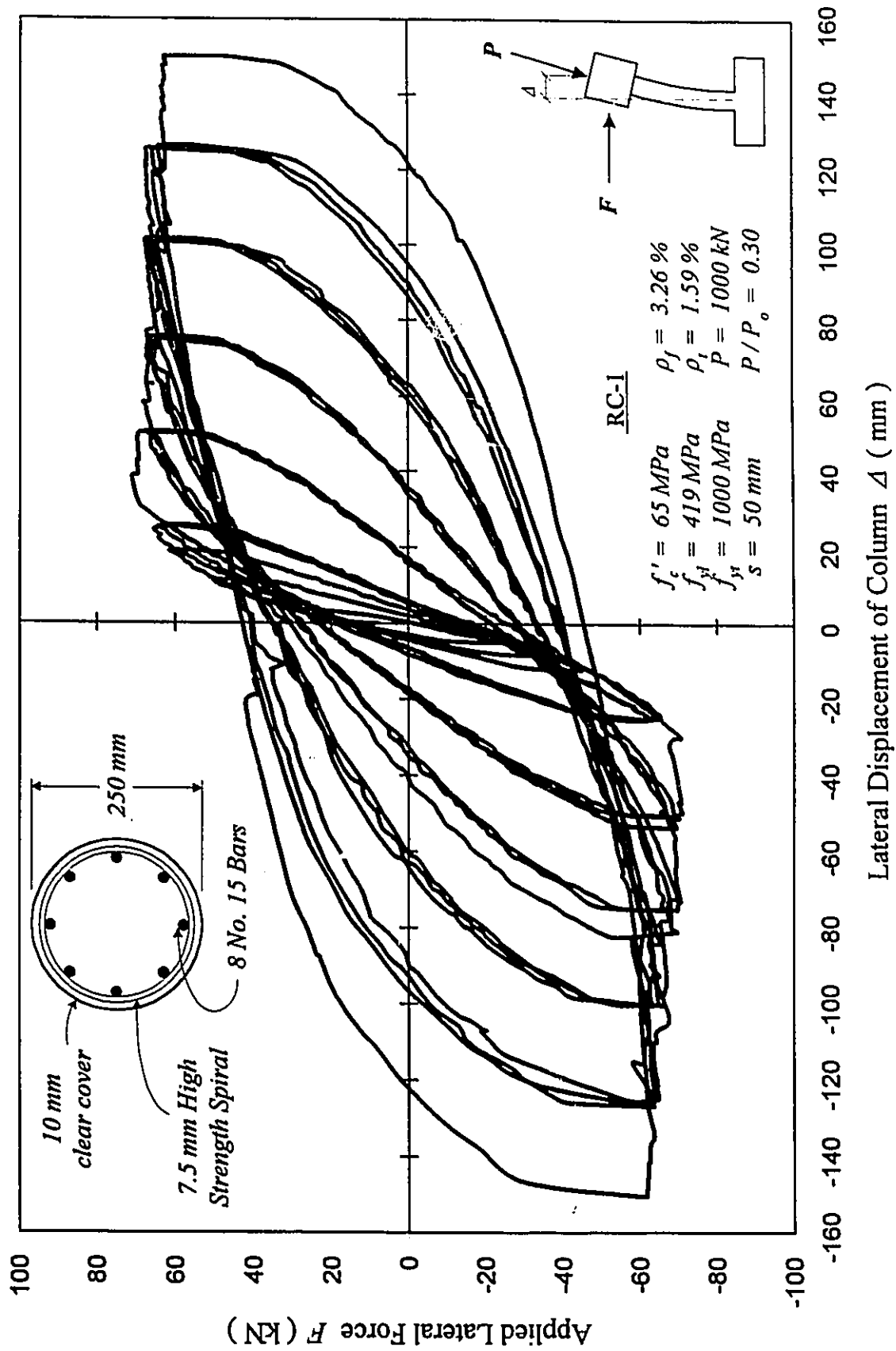


Figure 3.3: Force - Displacement Relationship for Column RC-1
 (Point of Application of Load - without $P-\Delta$ effect)

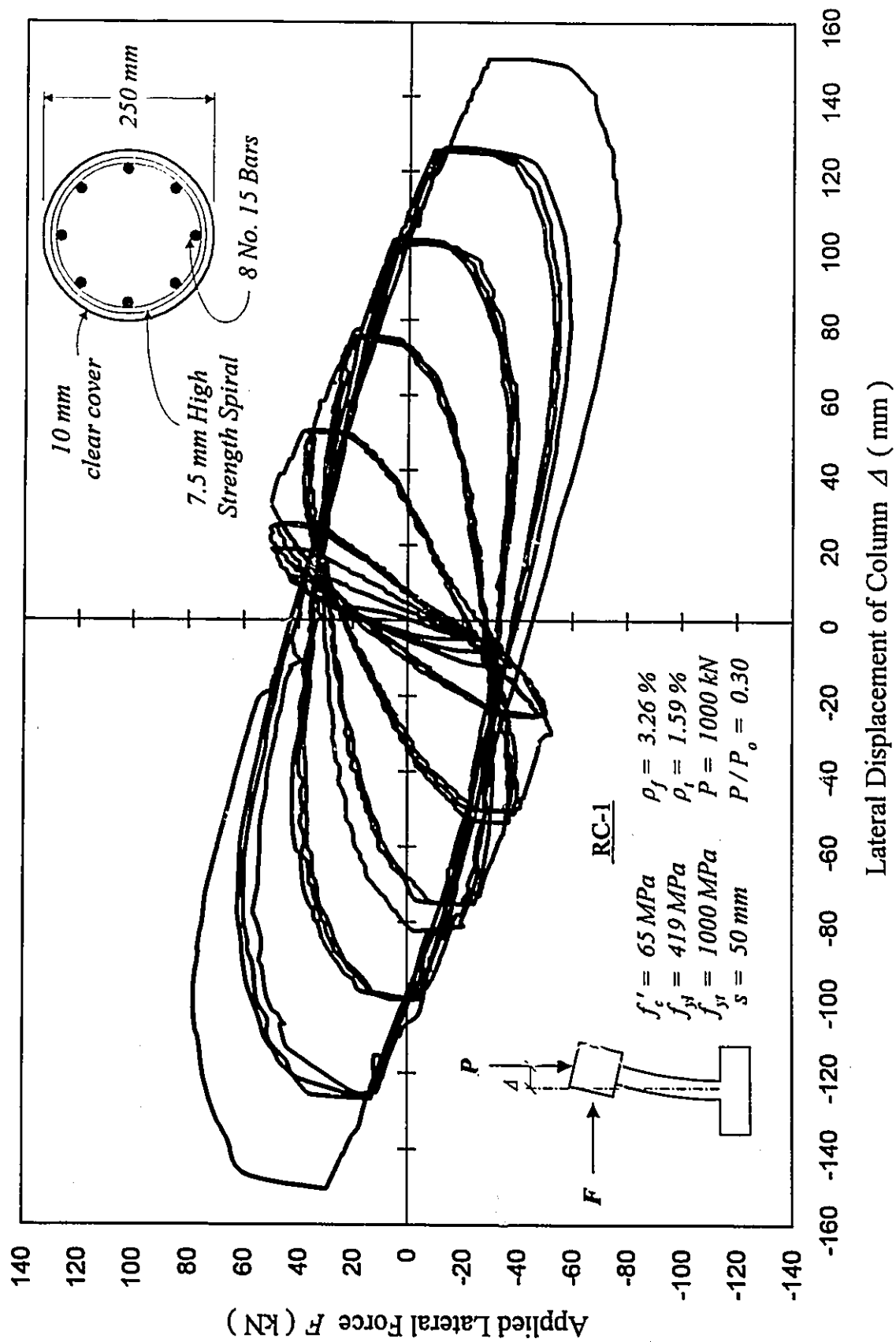


Figure 3.4: Force - Displacement Relationship for Column RC-1
(Point of Application of Load - with $P-\Delta$ effect)

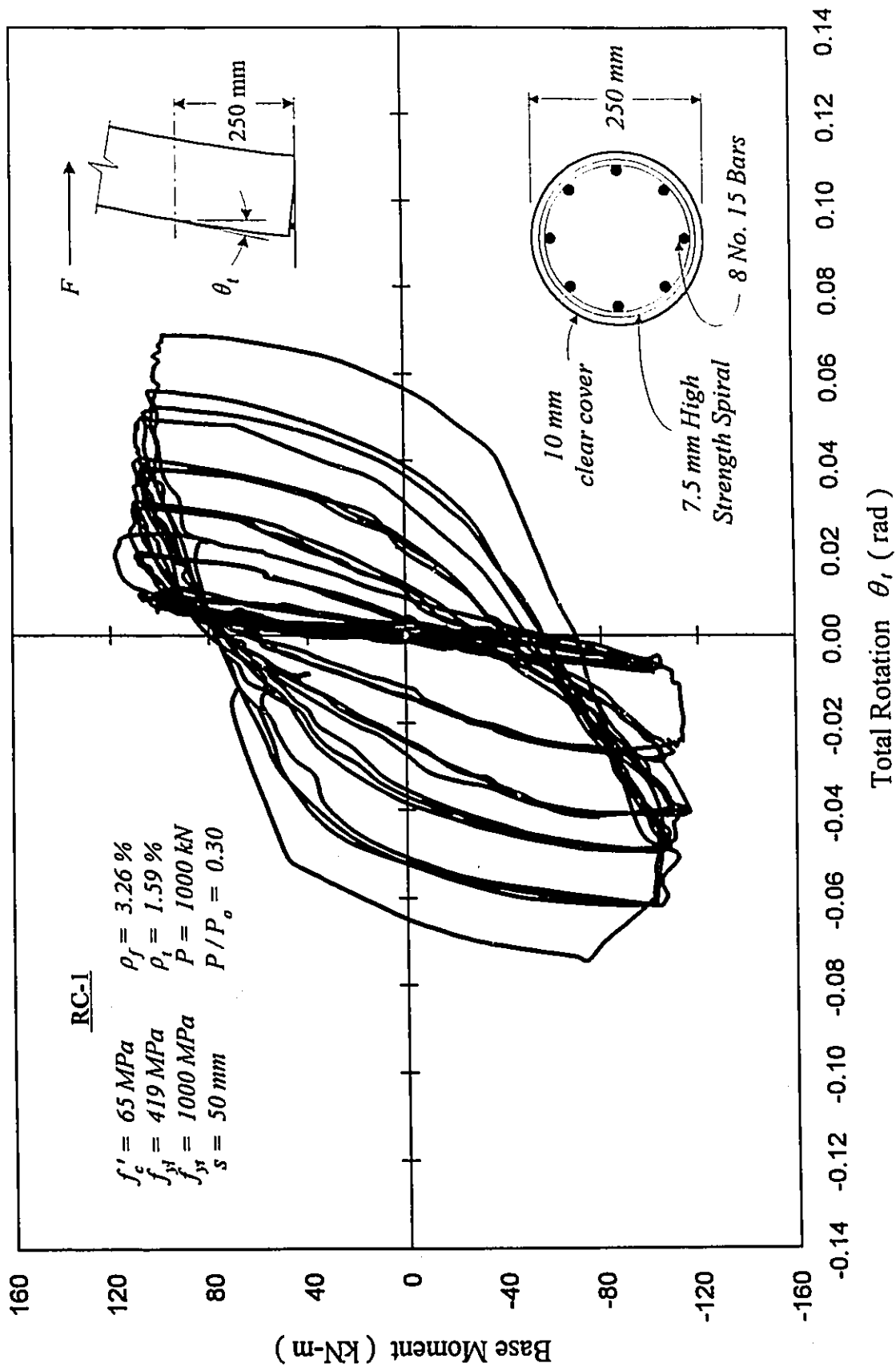


Figure 3.5: Base Moment - Total Rotation Relationship for Column RC-1
(Without $P-\Delta$ effects)

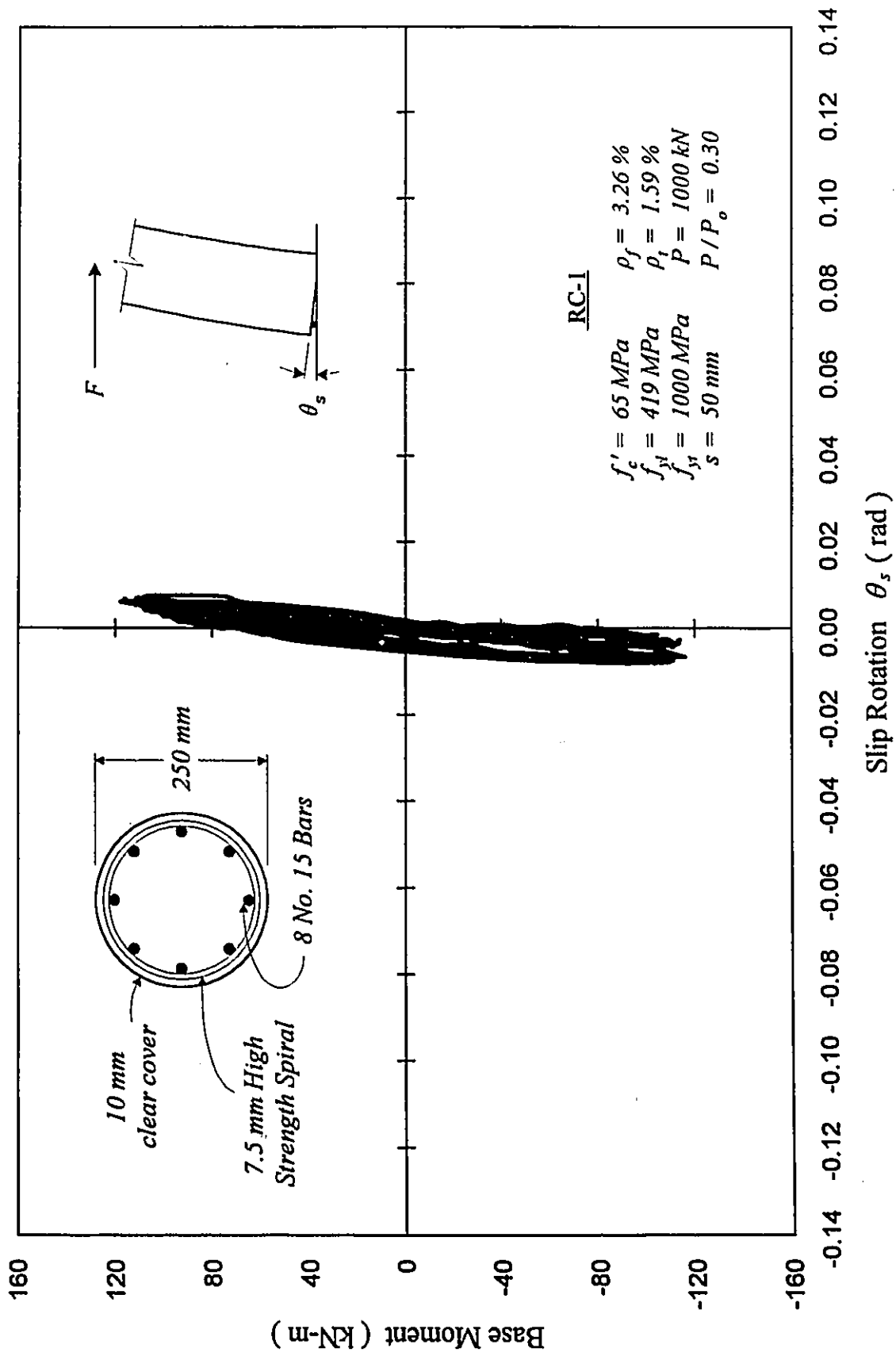
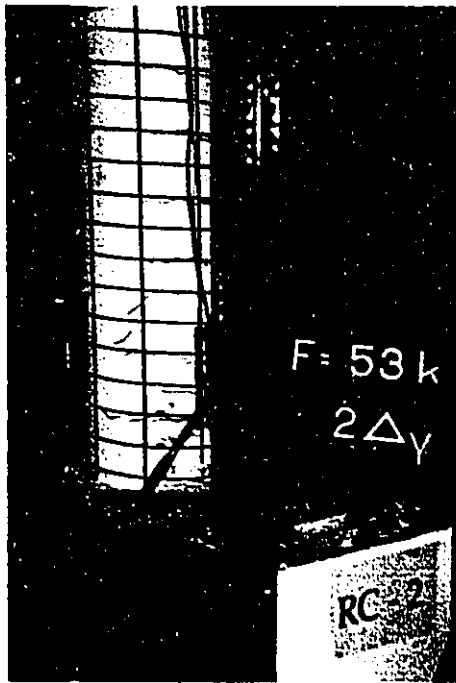


Figure 3.6: Base Moment - Slip Rotation Relationship for Column RC-1
(Without $P-\Delta$ effects)



(a) $2\Delta_y$



(b) $3\Delta_y$



(c) $5\Delta_y$



(d) $6\Delta_y$

Figure 3.7: Extent of Damage for Column RC-2 at Different Stages of Testing

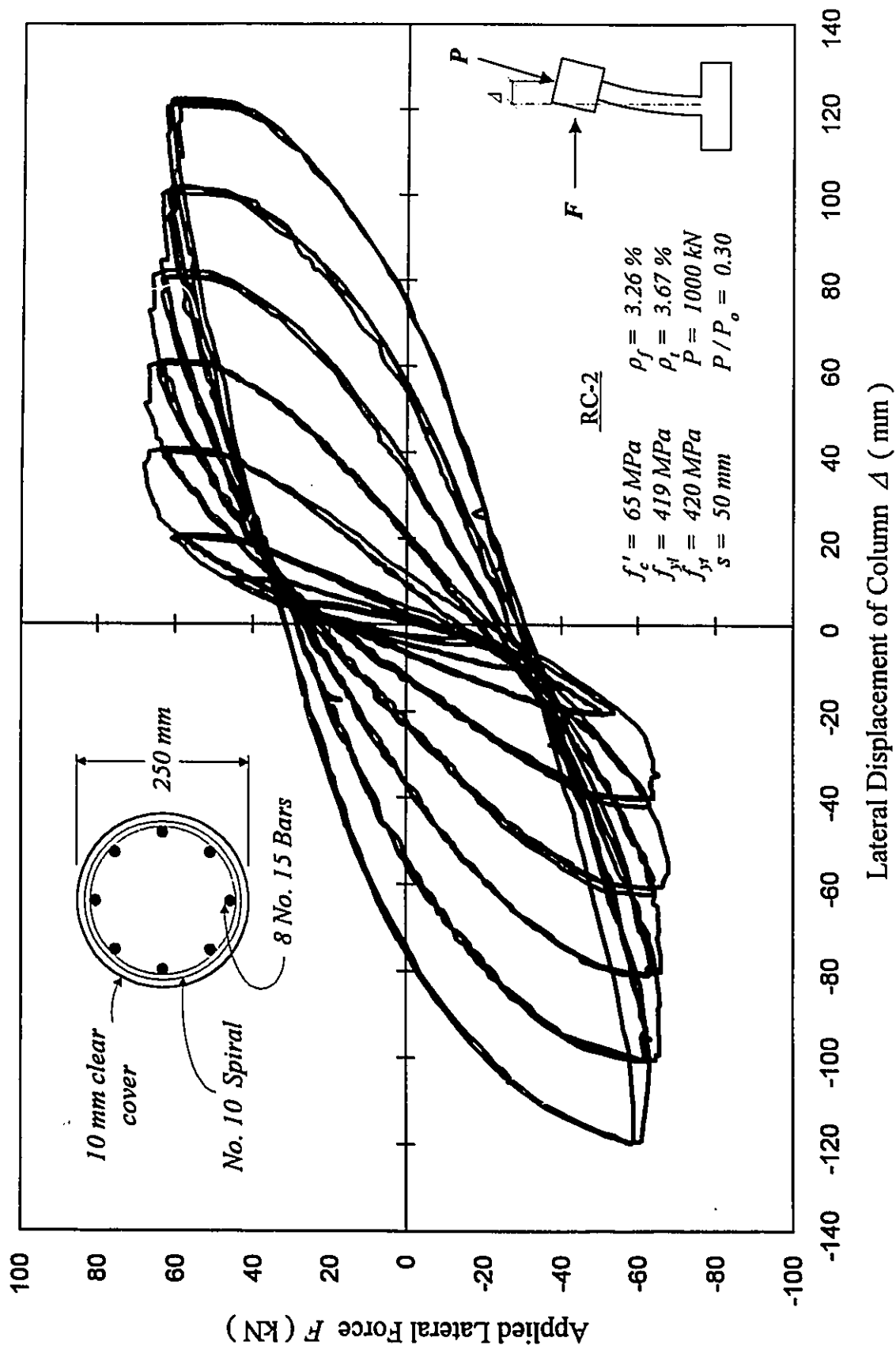


Figure 3.8: Force - Displacement Relationship for Column RC-2
(Point of Application of Load - without $P-\Delta$ effect)

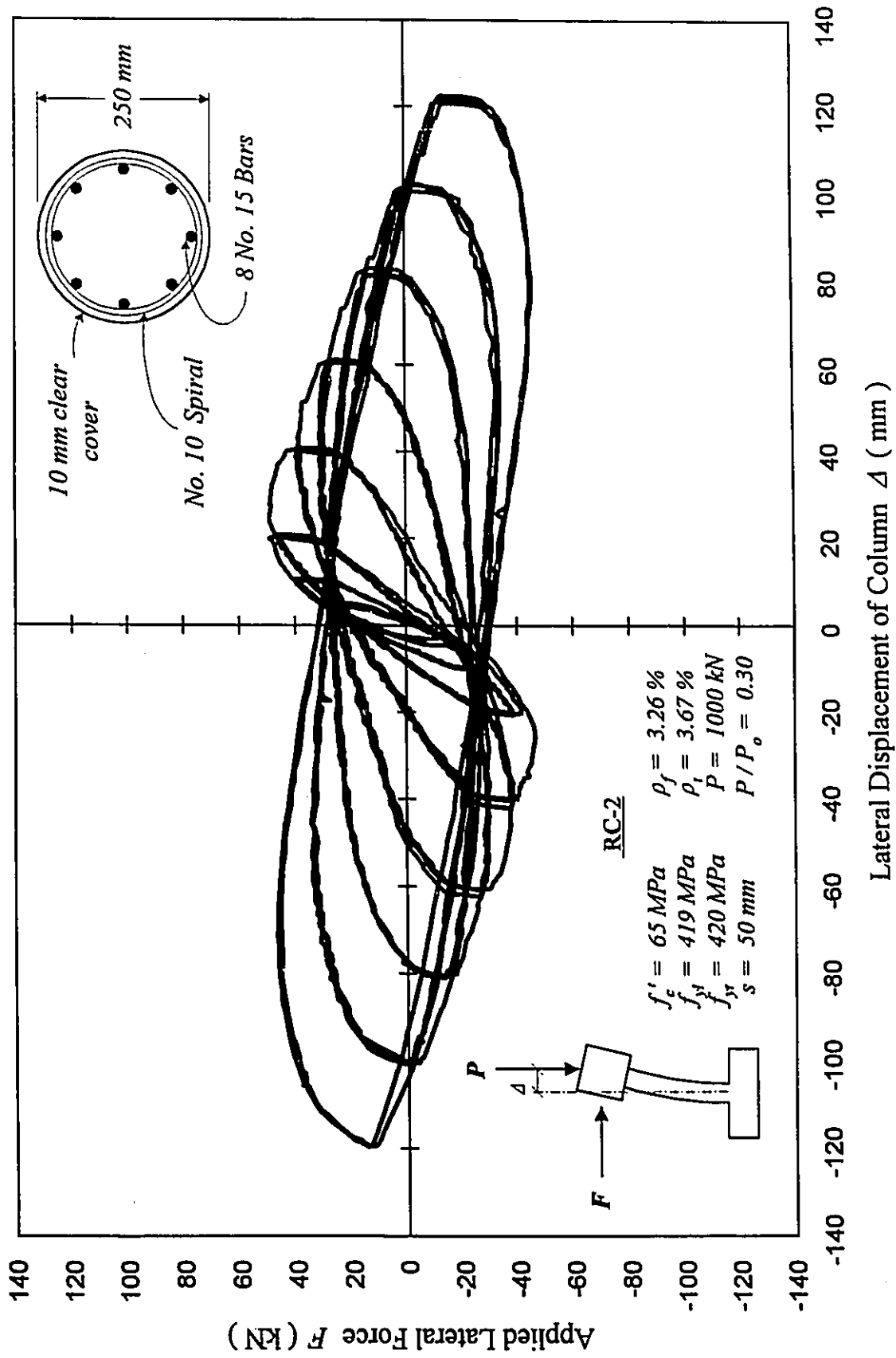


Figure 3.9: Force - Displacement Relationship for Column RC-2
(Point of Application of Load - with $P-\Delta$ effect)

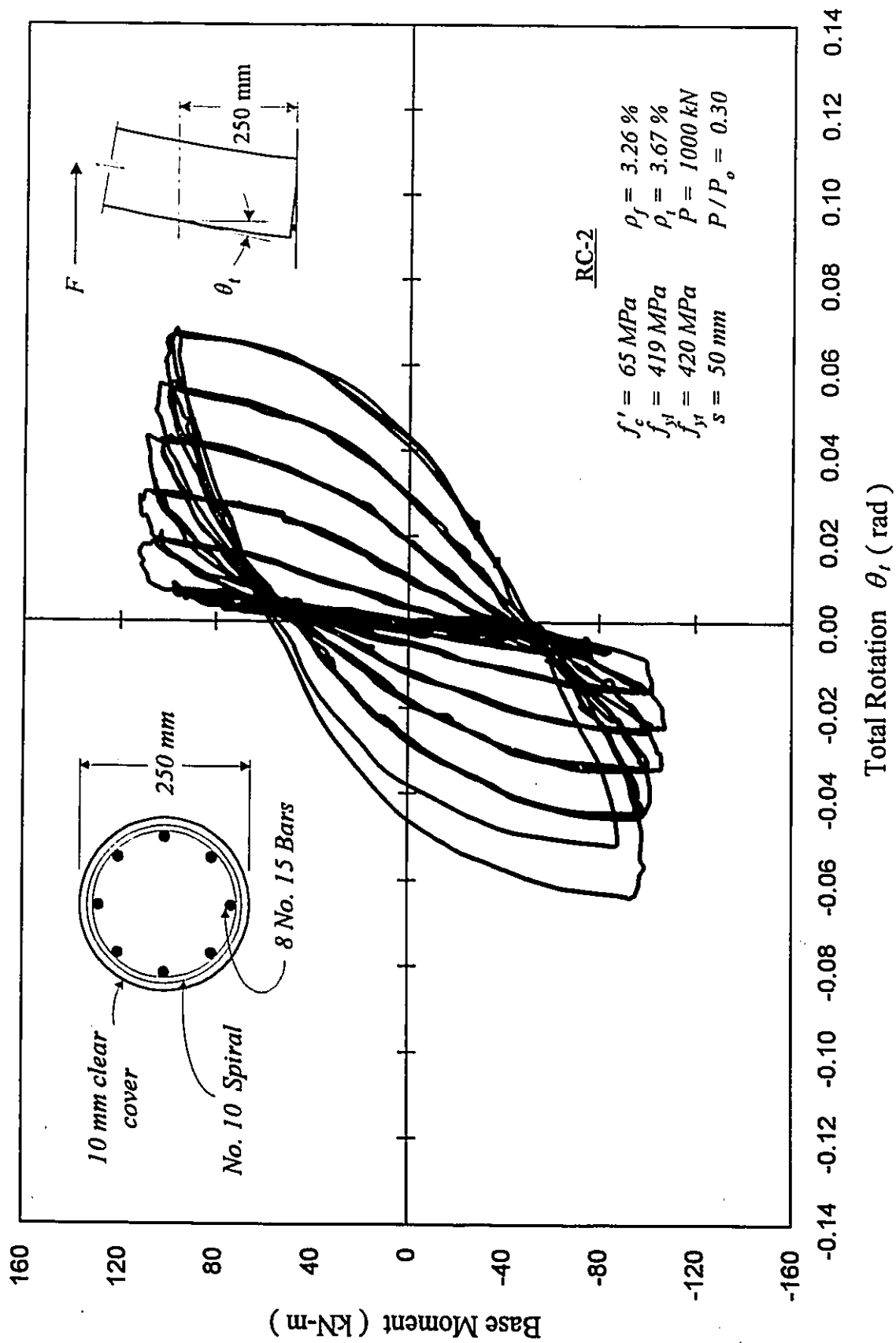


Figure 3.10: Base Moment - Total Rotation Relationship for Column RC-2
(Without $P-\Delta$ effects)

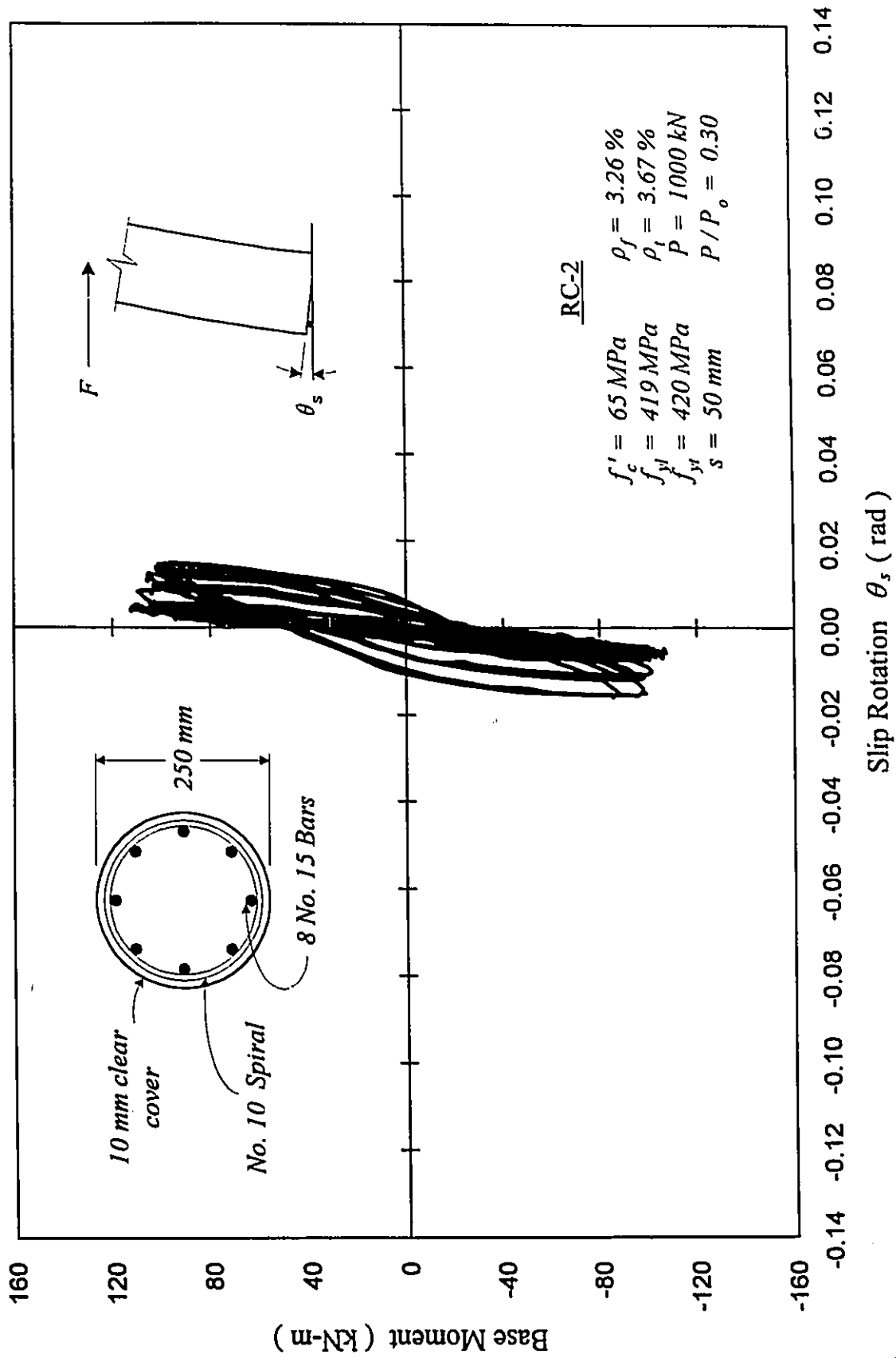
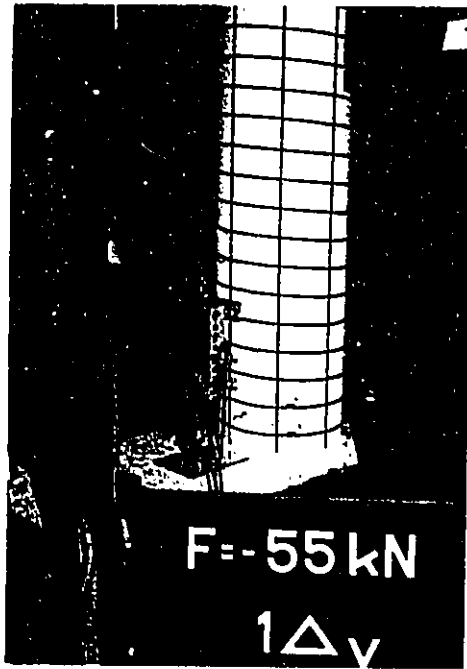


Figure 3.11: Base Moment - Slip Rotation Relationship for Column RC-2
(Without $P-\Delta$ effects)



(a) $1\Delta_y$



(b) $2\Delta_y$



(c) $3\Delta_y$



(d) $5\Delta_y$

Figure 3.12: Extent of Damage for Column RC-3 at Different Stages of Testing

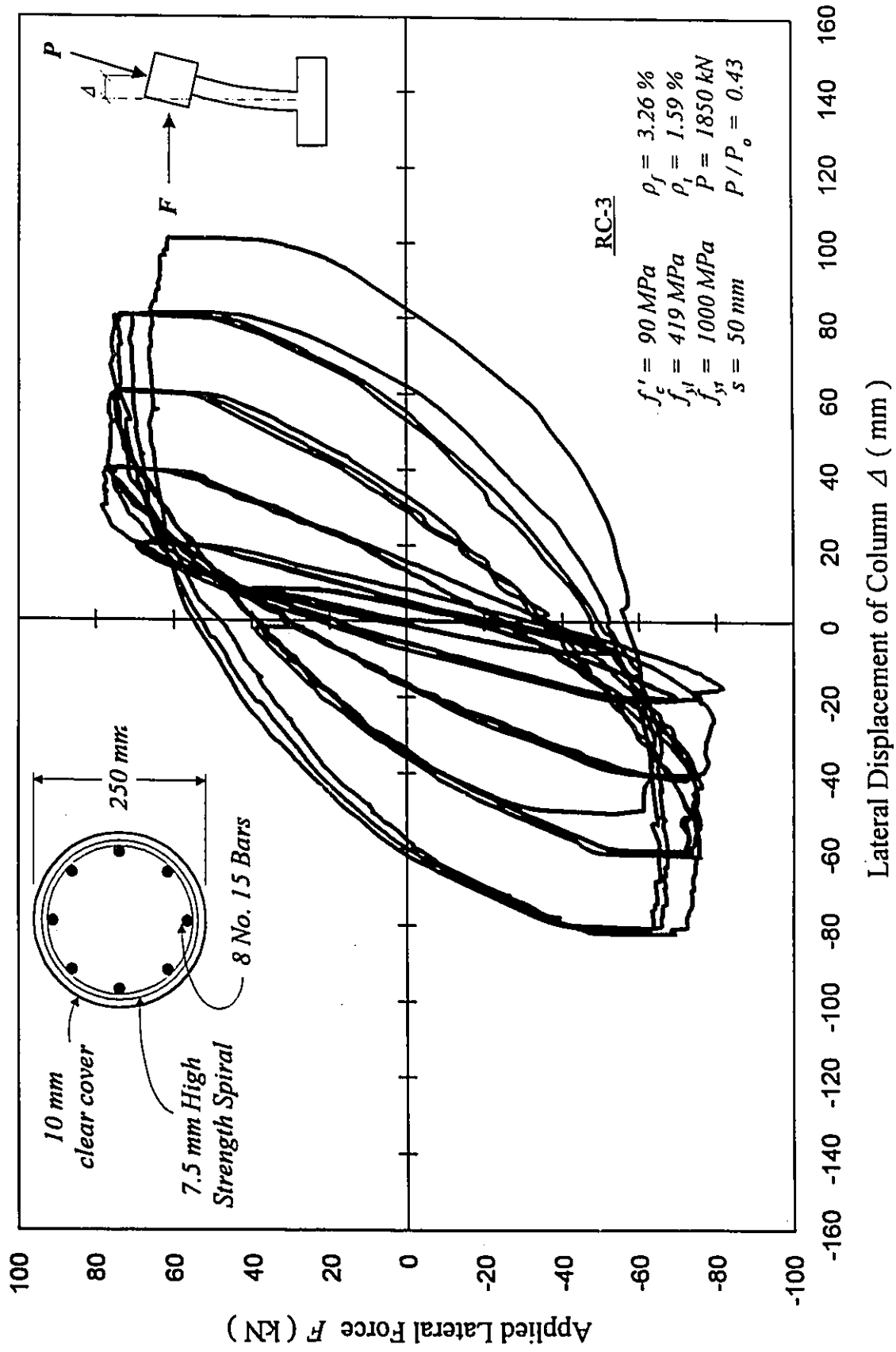


Figure 3.13: Force - Displacement Relationship for Column RC-3
(Point of Application of Load - without $P-\Delta$ effect)

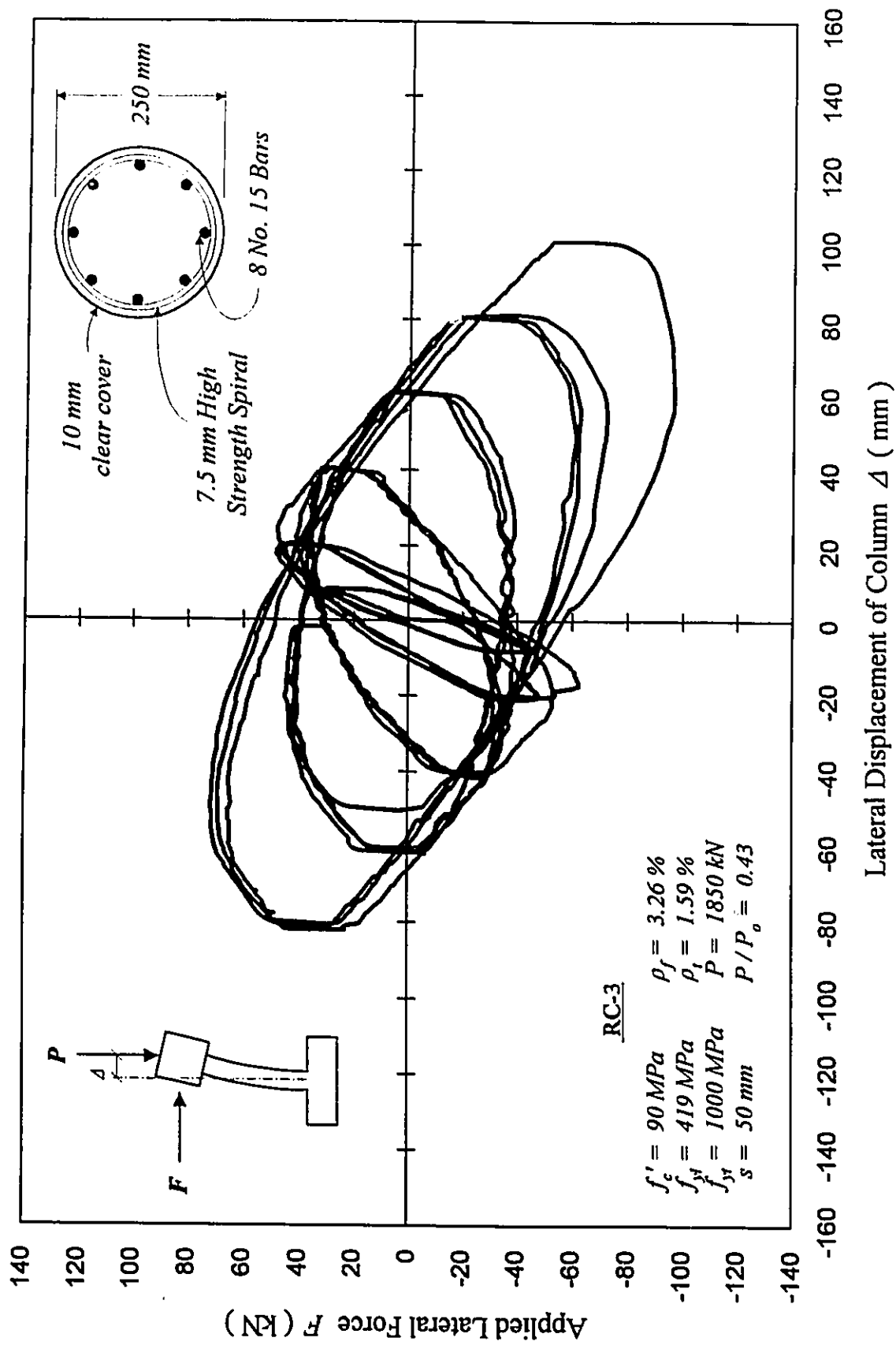


Figure 3.14: Force - Displacement Relationship for Column RC-3
(Point of Application of Load - with $P-\Delta$ effect)

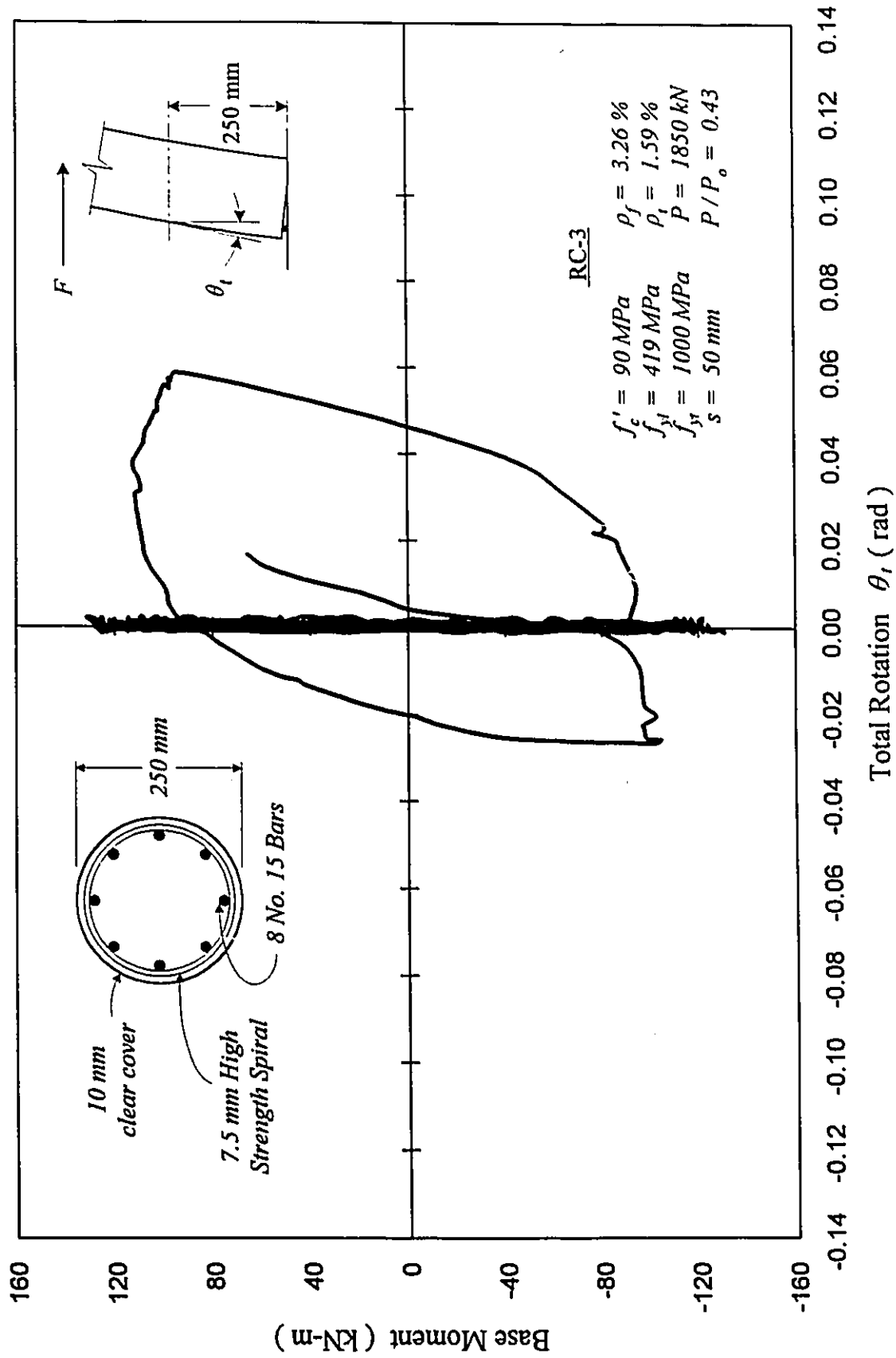


Figure 3.15: Base Moment - Total Rotation Relationship for Column RC-3
(Without $P-\Delta$ effects)

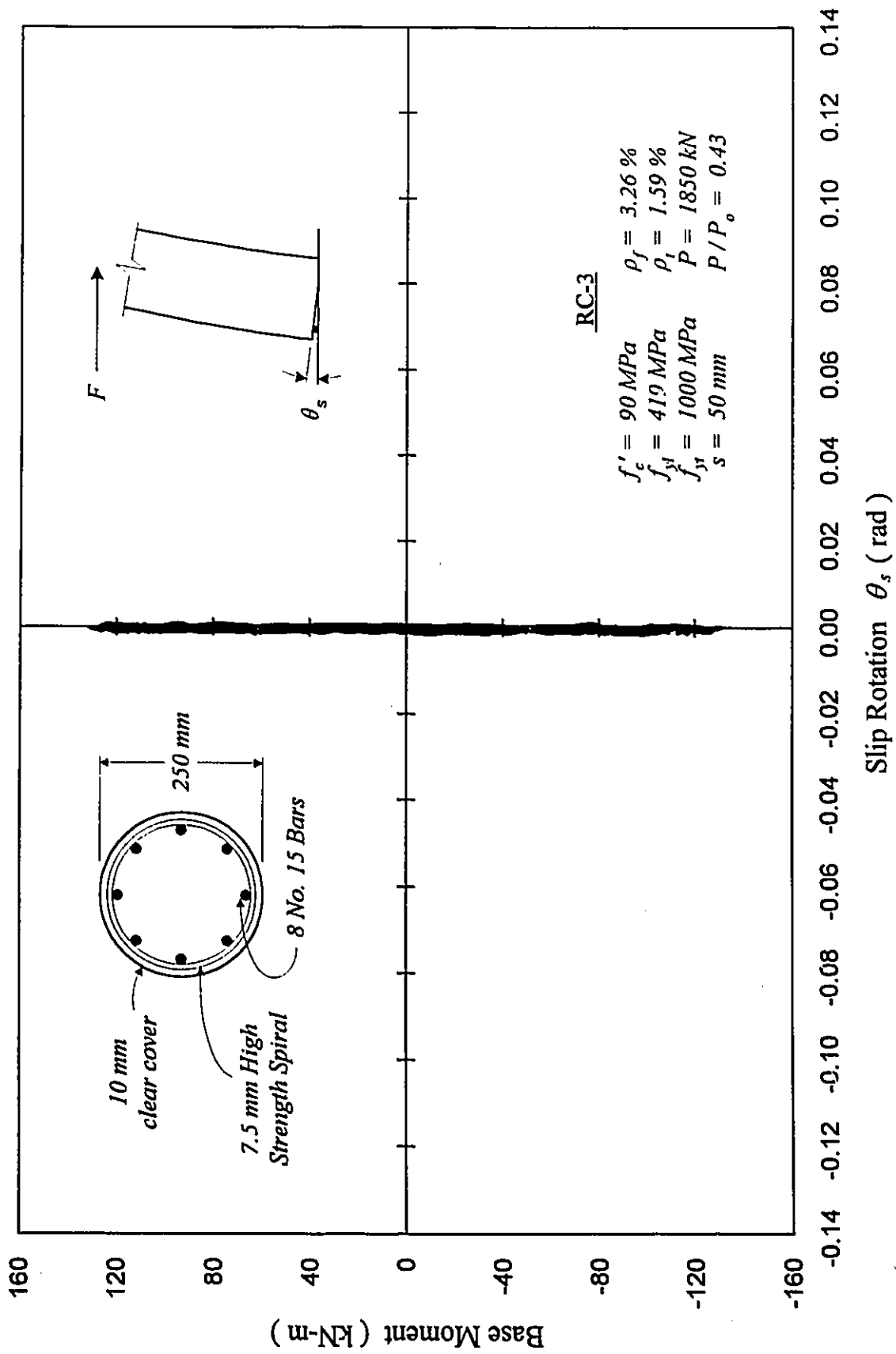
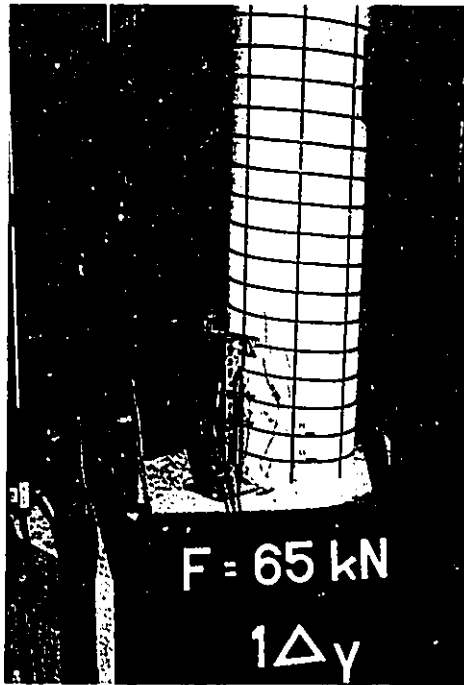
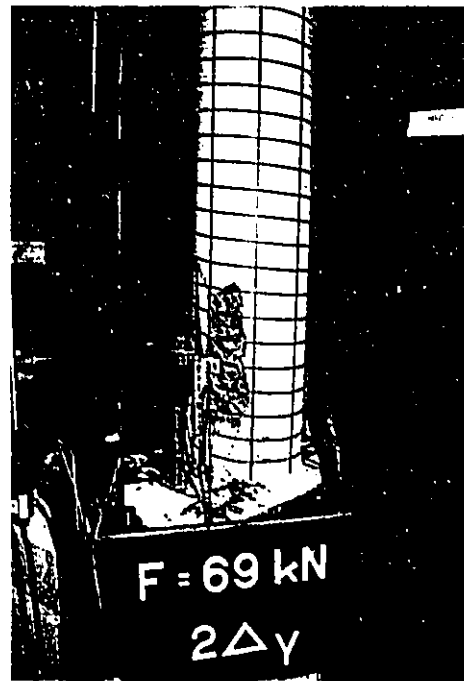


Figure 3.16: Base Moment - Slip Rotation Relationship for Column RC-3
(Without $P-\Delta$ effects)



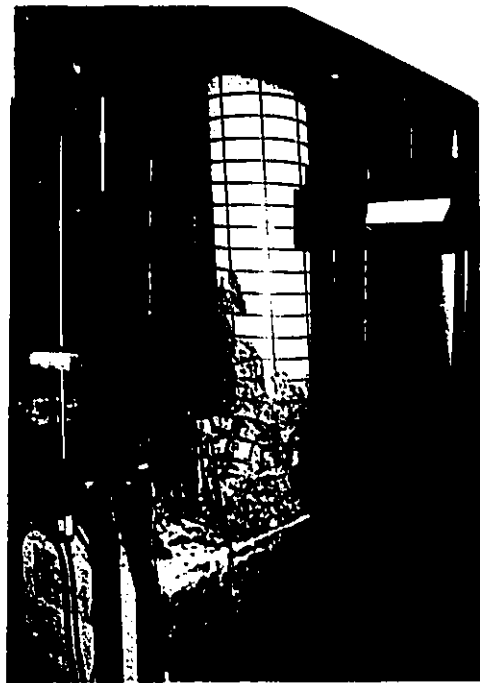
(a) $1\Delta_y$



(b) $2\Delta_y$



(c) $3\Delta_y$



(d) At Failure

Figure 3.17: Extent of Damage for Column RC-4 at Different Stages of Testing



(a) Front



(b) Front-Side



(c) Rear



(d) Spiral Rupture

Figure 3.18: Close-up Views of Column RC-4 Hinge Region After Failure

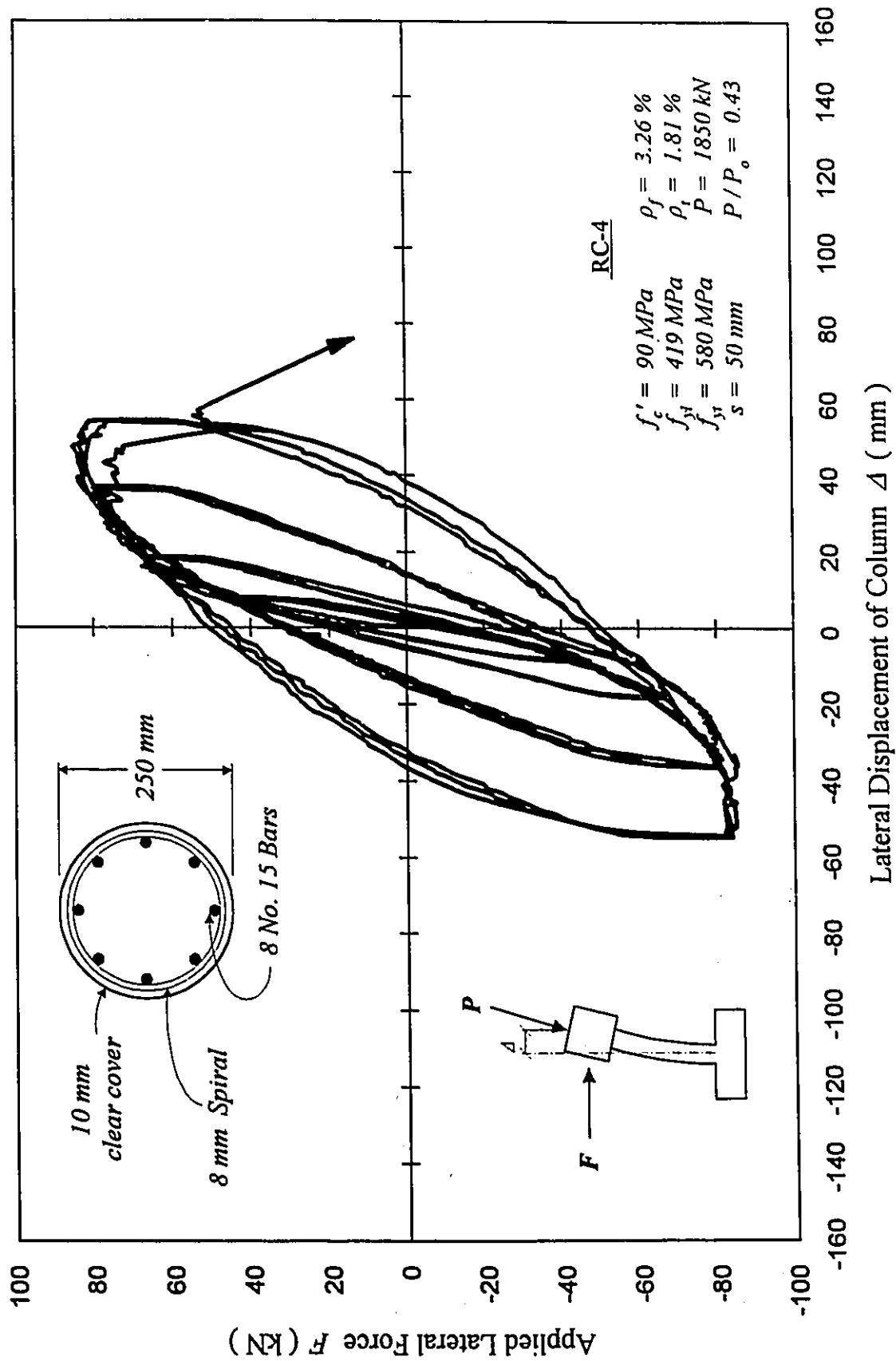


Figure 3.19: Force - Displacement Relationship for Column RC-4
(Point of Application of Load - without $P-\Delta$ effect)

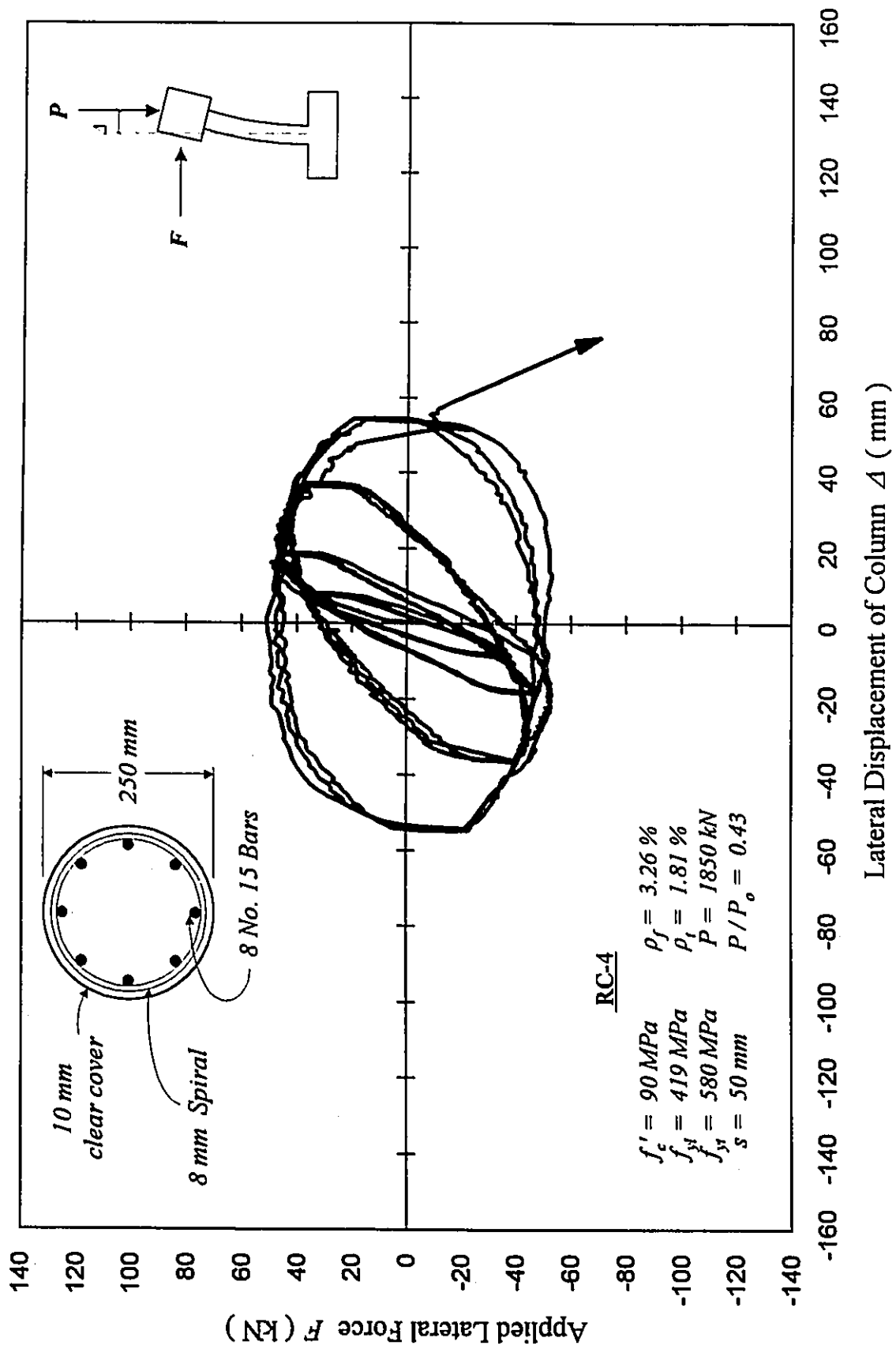


Figure 3.20: Force - Displacement Relationship for Column RC-4
(Point of Application of Load - with $P-\Delta$ effect)

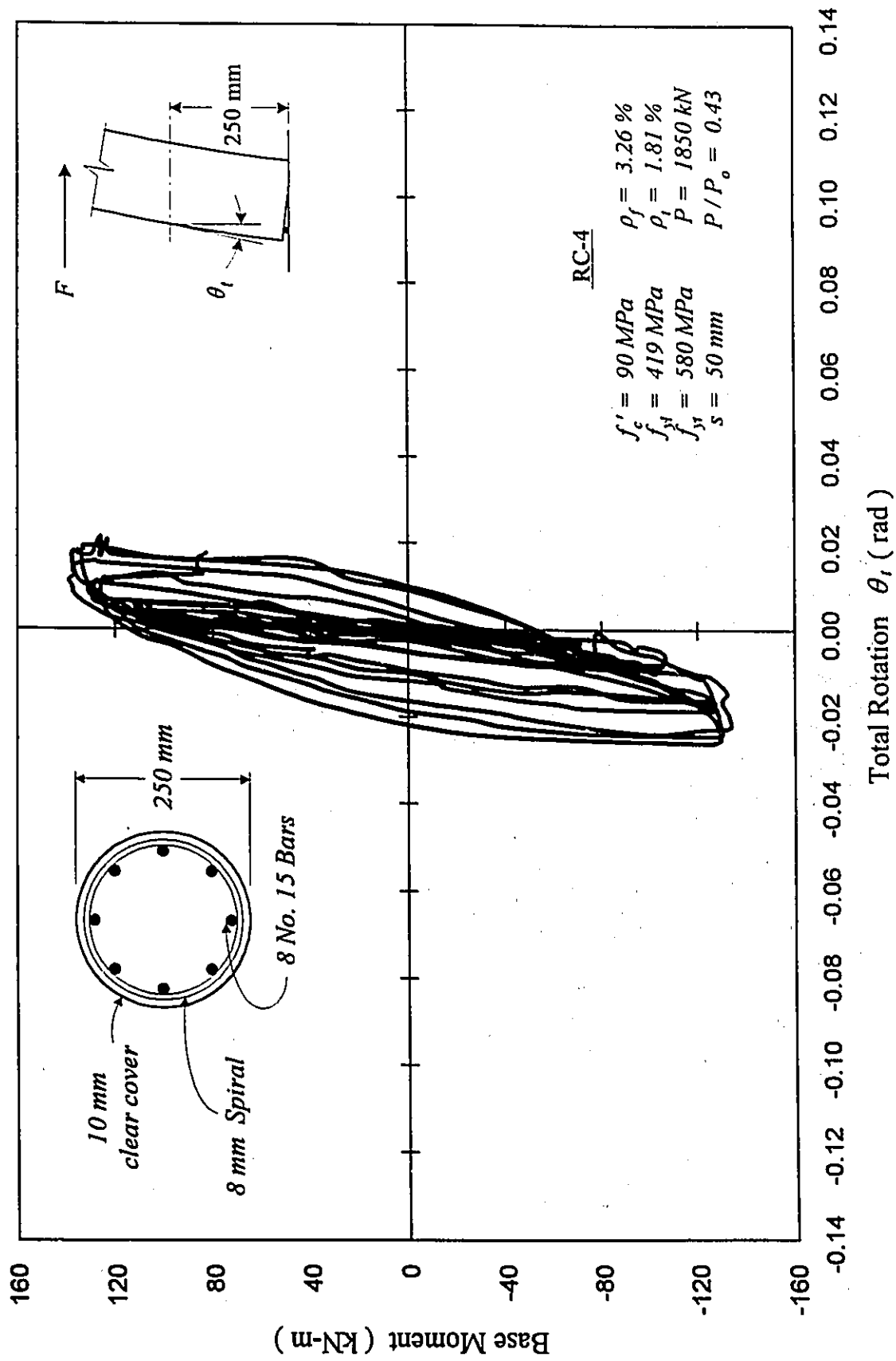


Figure 3.21: Base Moment - Total Rotation Relationship for Column RC-4
(Without $P-\Delta$ effects)

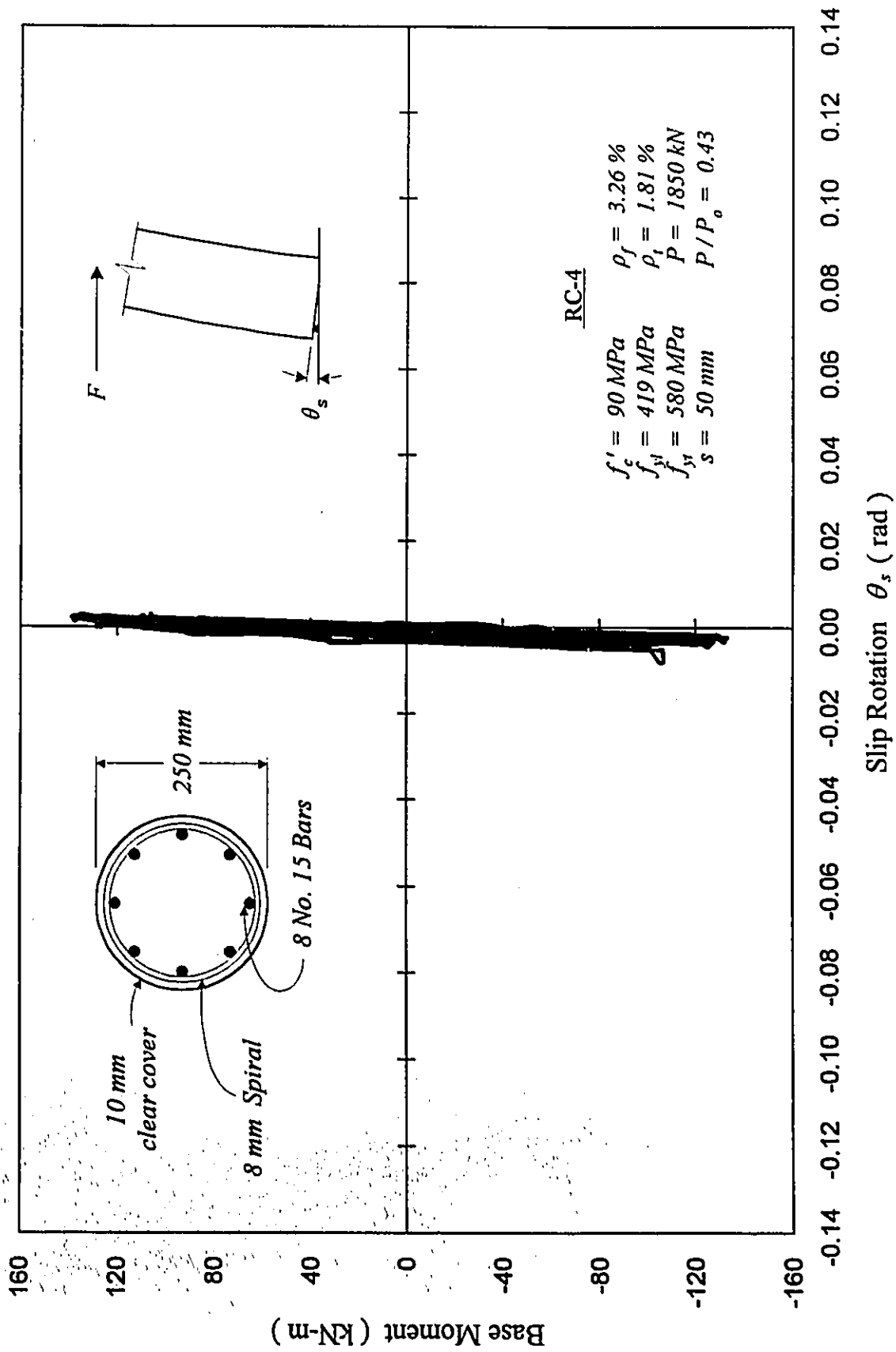
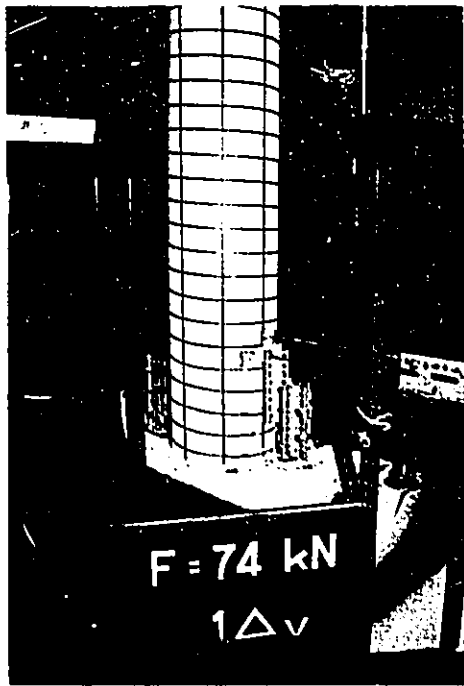
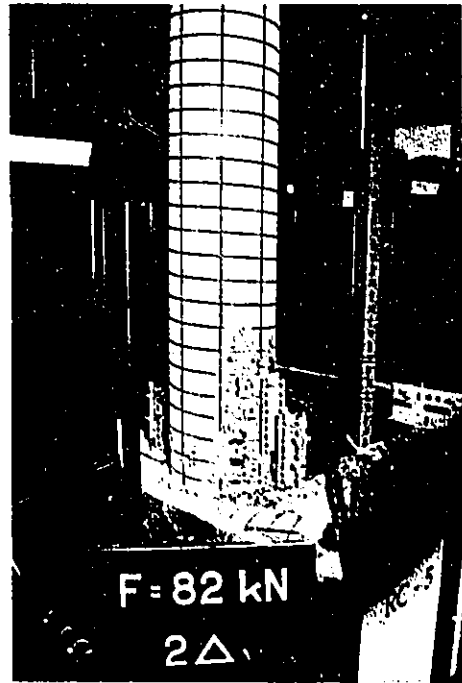


Figure 3.22: Base Moment - Slip Rotation Relationship for Column RC-4
(Without $P-\Delta$ effects)



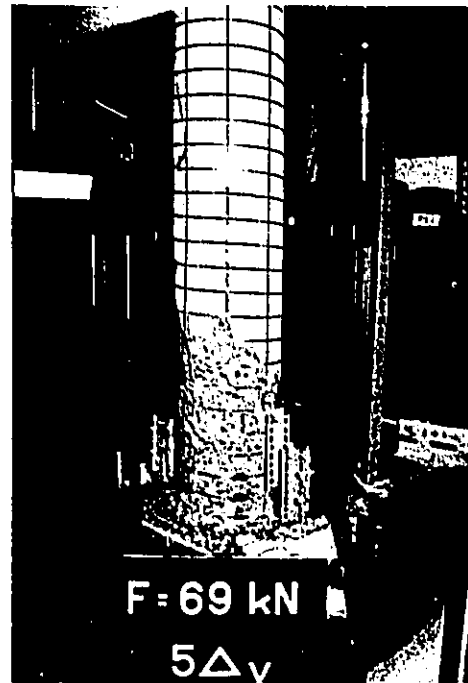
(a) 1Δ_y



(b) 2Δ_y



(c) 3Δ_y



(d) 5Δ_y

Figure 3.23: Extent of Damage for Column RC-5 at Different Stages of Testing

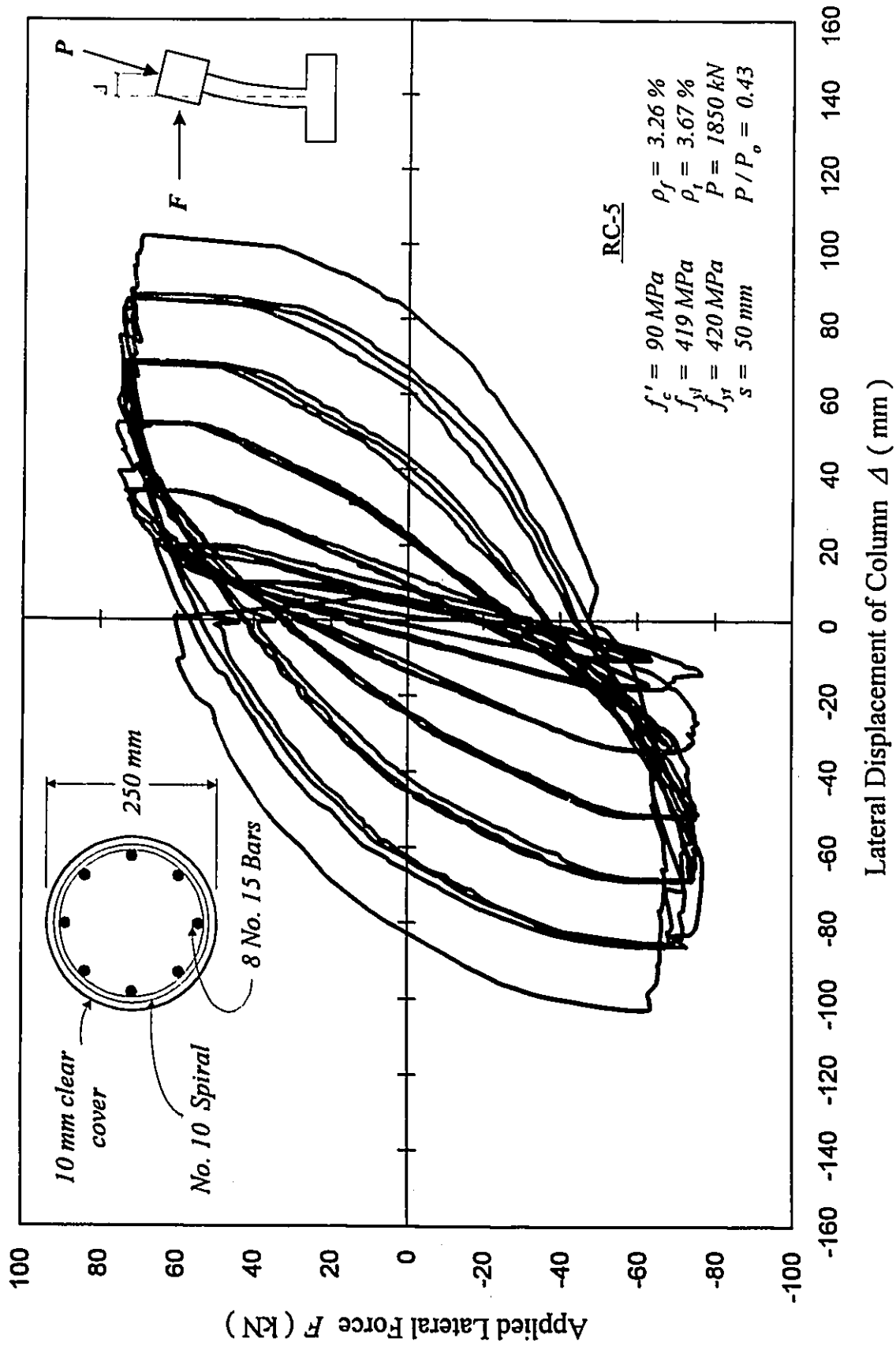


Figure 3.24: Force - Displacement Relationship for Column RC-5
(Point of Application of Load - without $P-\Delta$ effect)

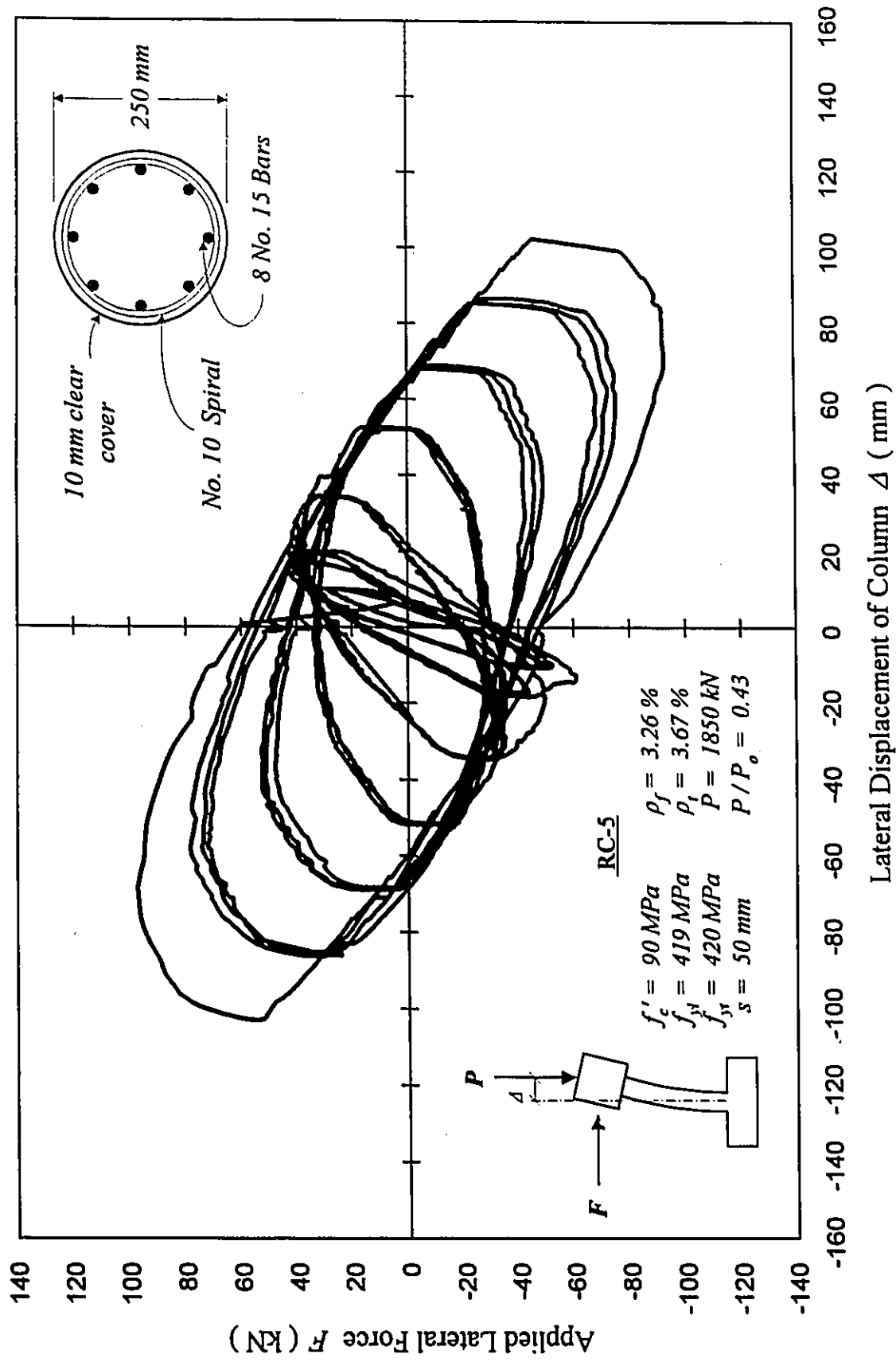


Figure 3.25: Force - Displacement Relationship for Column RC-5
 (Point of Application of Load - with $P-\Delta$ effect)

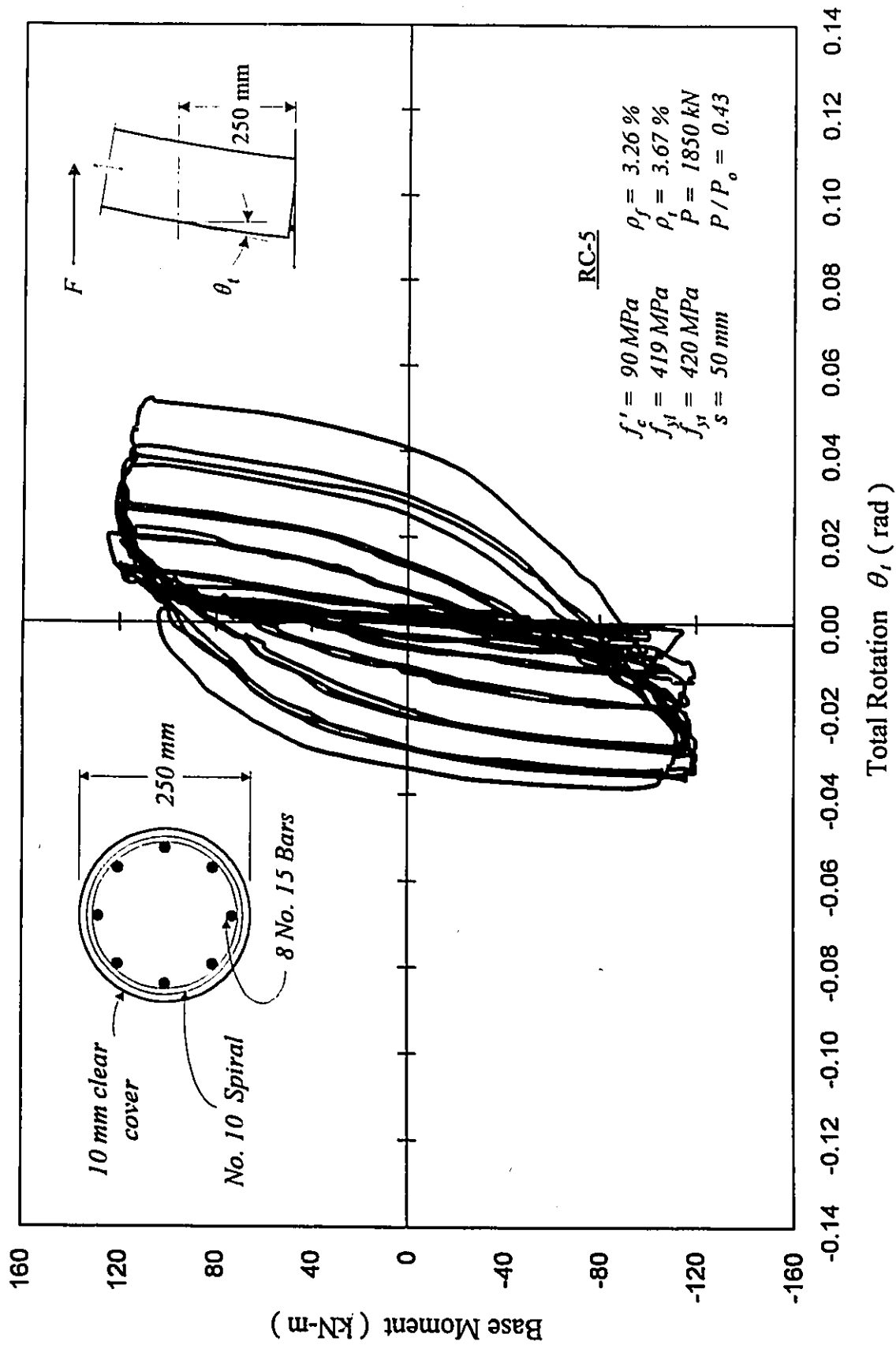


Figure 3.26: Base Moment - Total Rotation Relationship for Column RC-5
(Without $P-\Delta$ effects)

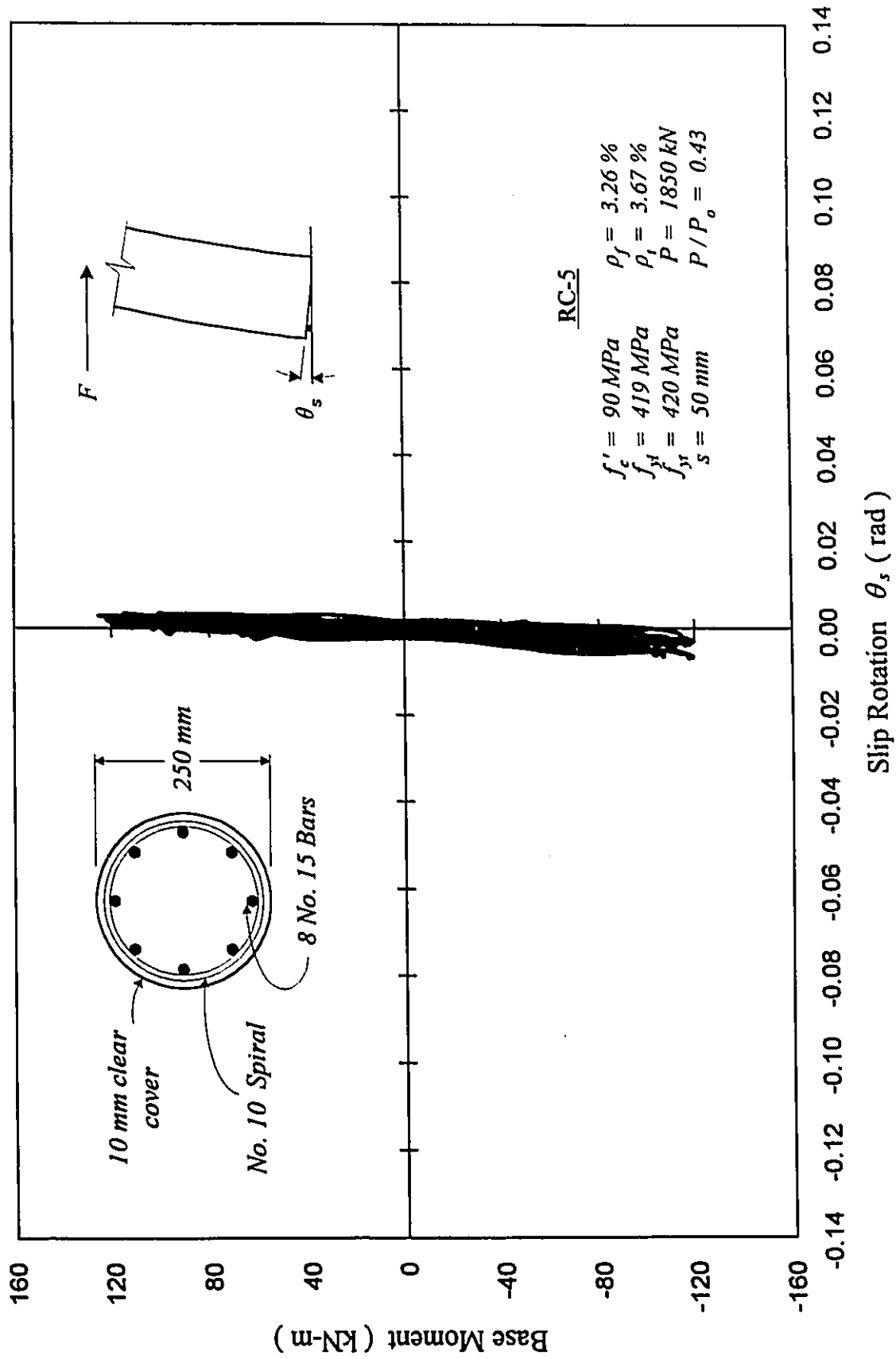


Figure 3.27: Base Moment - Slip Rotation Relationship for Column RC-5
(Without $P-\Delta$ effects)



(a) $1\Delta_y$



(b) $2\Delta_y$



(c) $3\Delta_y$



(d) $4\Delta_y$

Figure 3.28: Extent of Damage for Column RC-6 at Different Stages of Testing

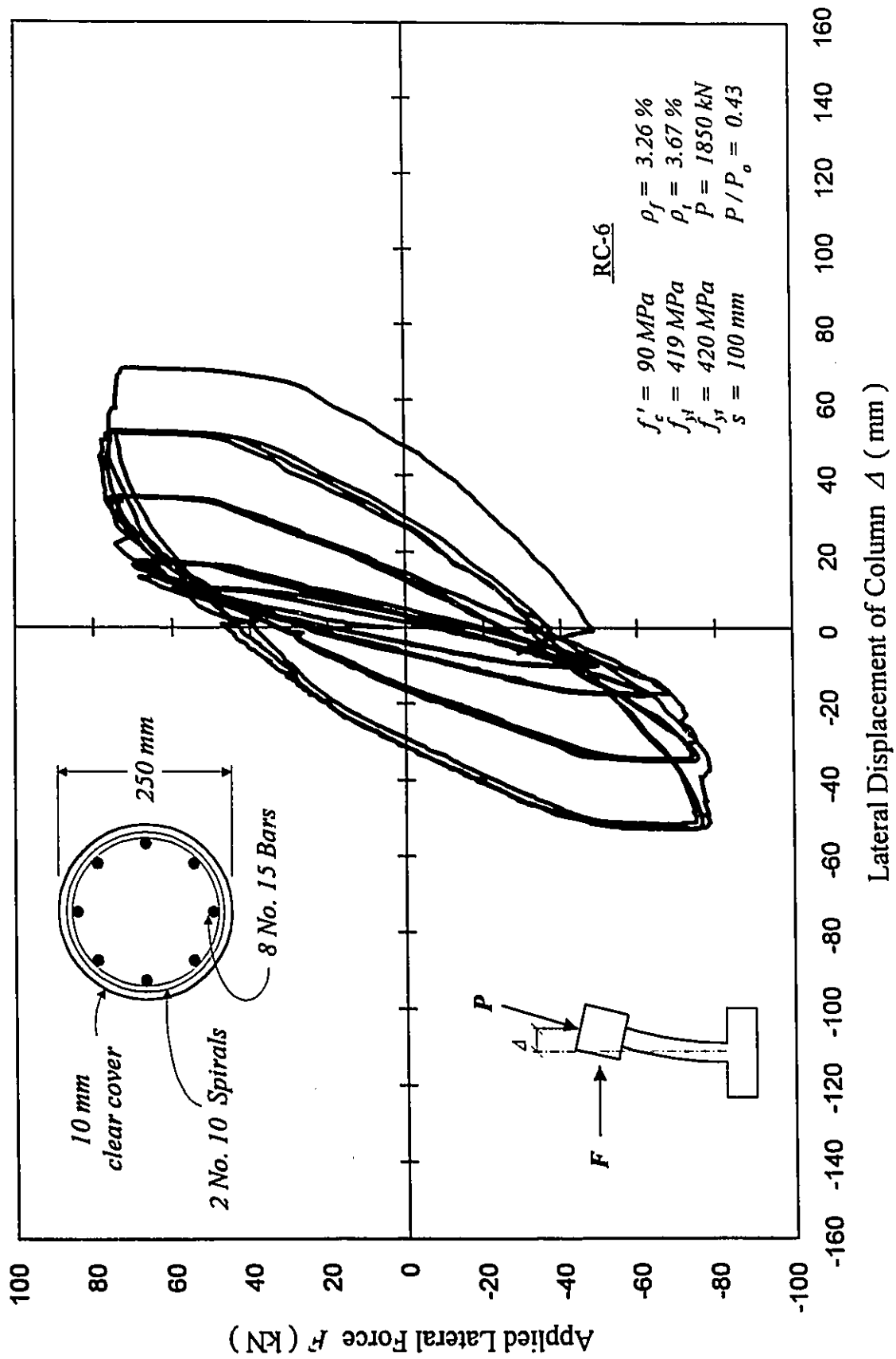


Figure 3.29: Force - Displacement Relationship for Column RC-6
(Point of Application of Load - without $P-\Delta$ effect)

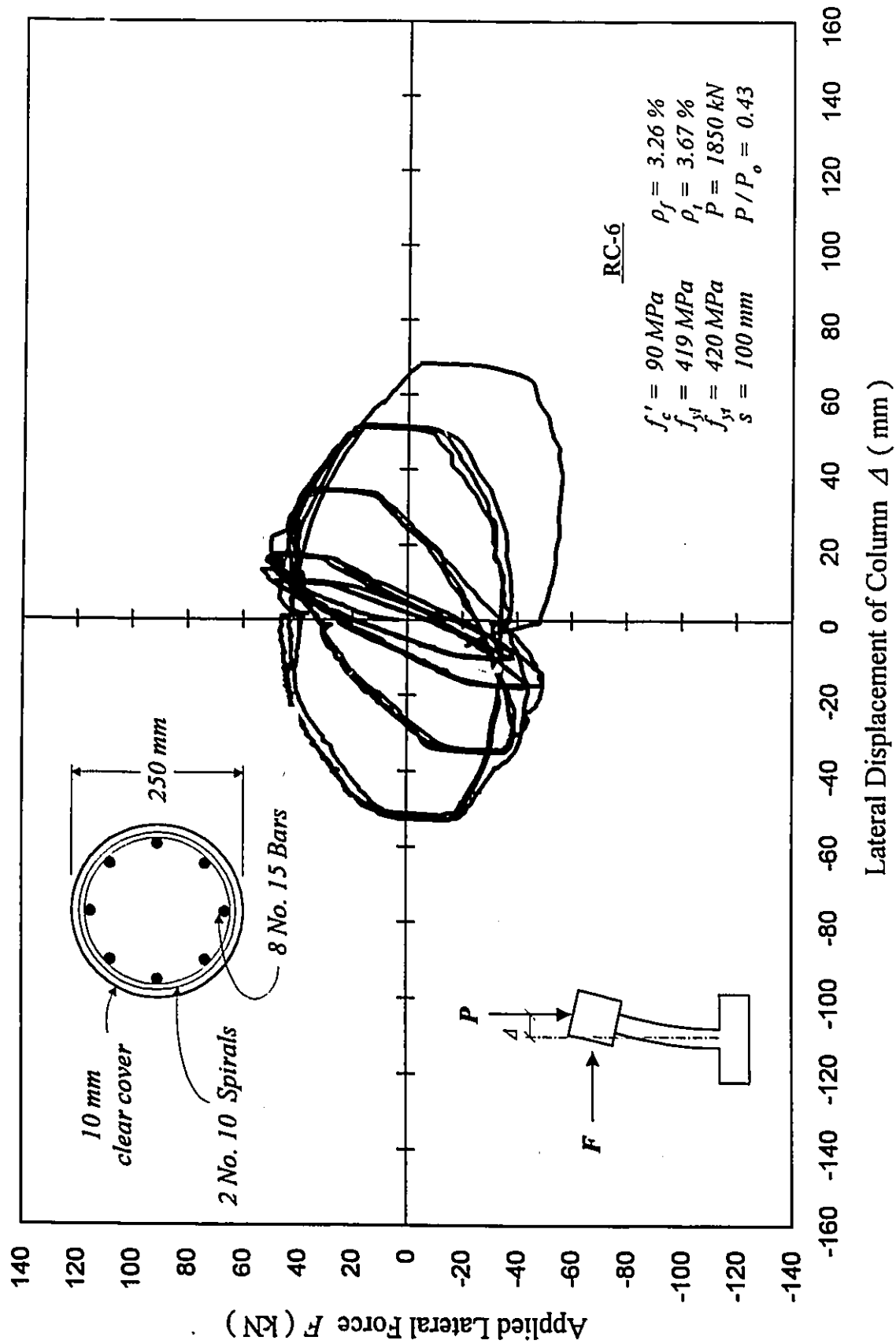


Figure 3.30: Force - Displacement Relationship for Column RC-6
(Point of Application of Load - with $P-\Delta$ effect)

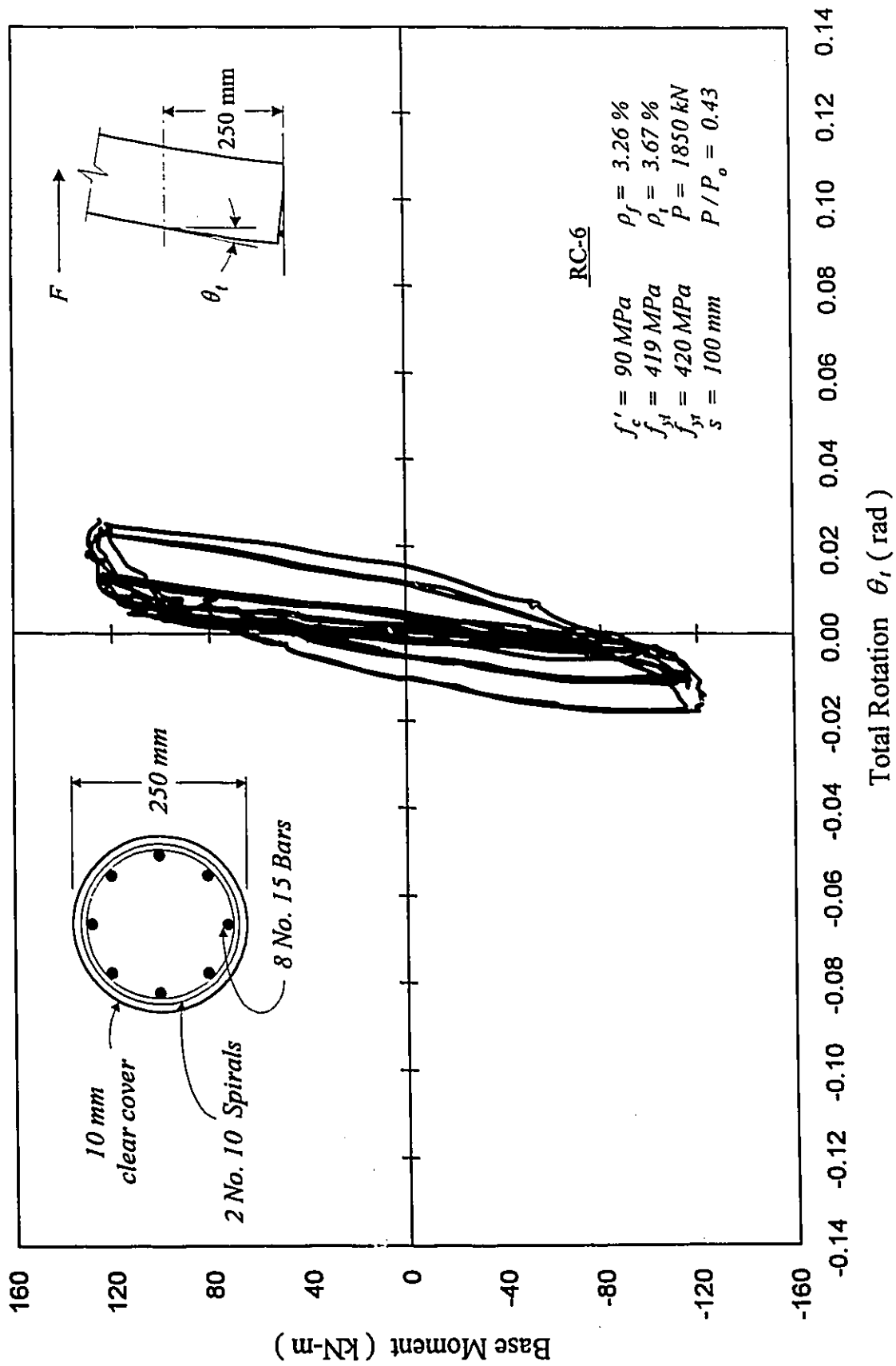


Figure 3.31: Base Moment - Total Rotation Relationship for Column RC-6
(Without $P-\Delta$ effects)

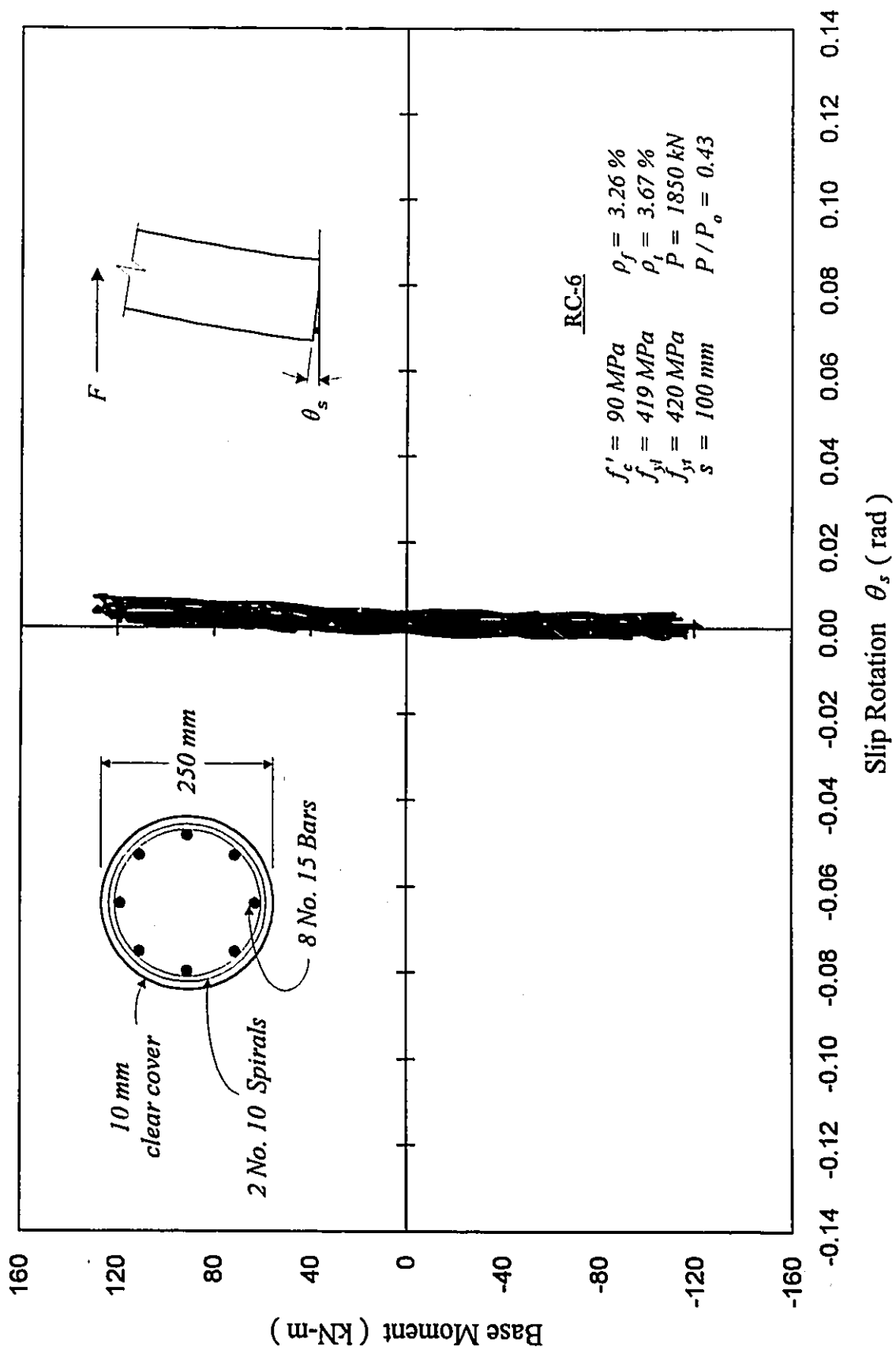
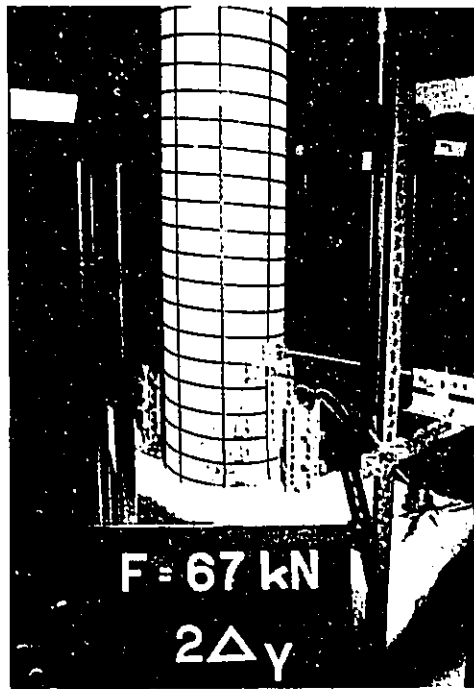


Figure 3.32: Base Moment - Slip Rotation Relationship for Column RC-6
(Without $P-\Delta$ effects)



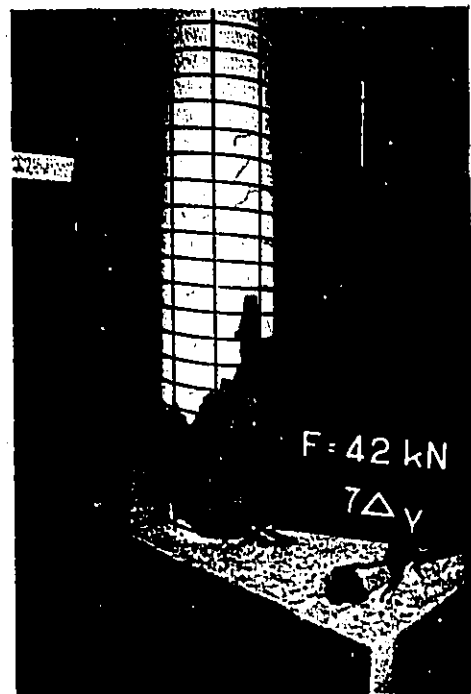
(a) $2\Delta_y$



(b) $3\Delta_y$



(c) $5\Delta_y$



(d) $7\Delta_y$

Figure 3.33: Extent of Damage for Column RC-7 at Different Stages of Testing

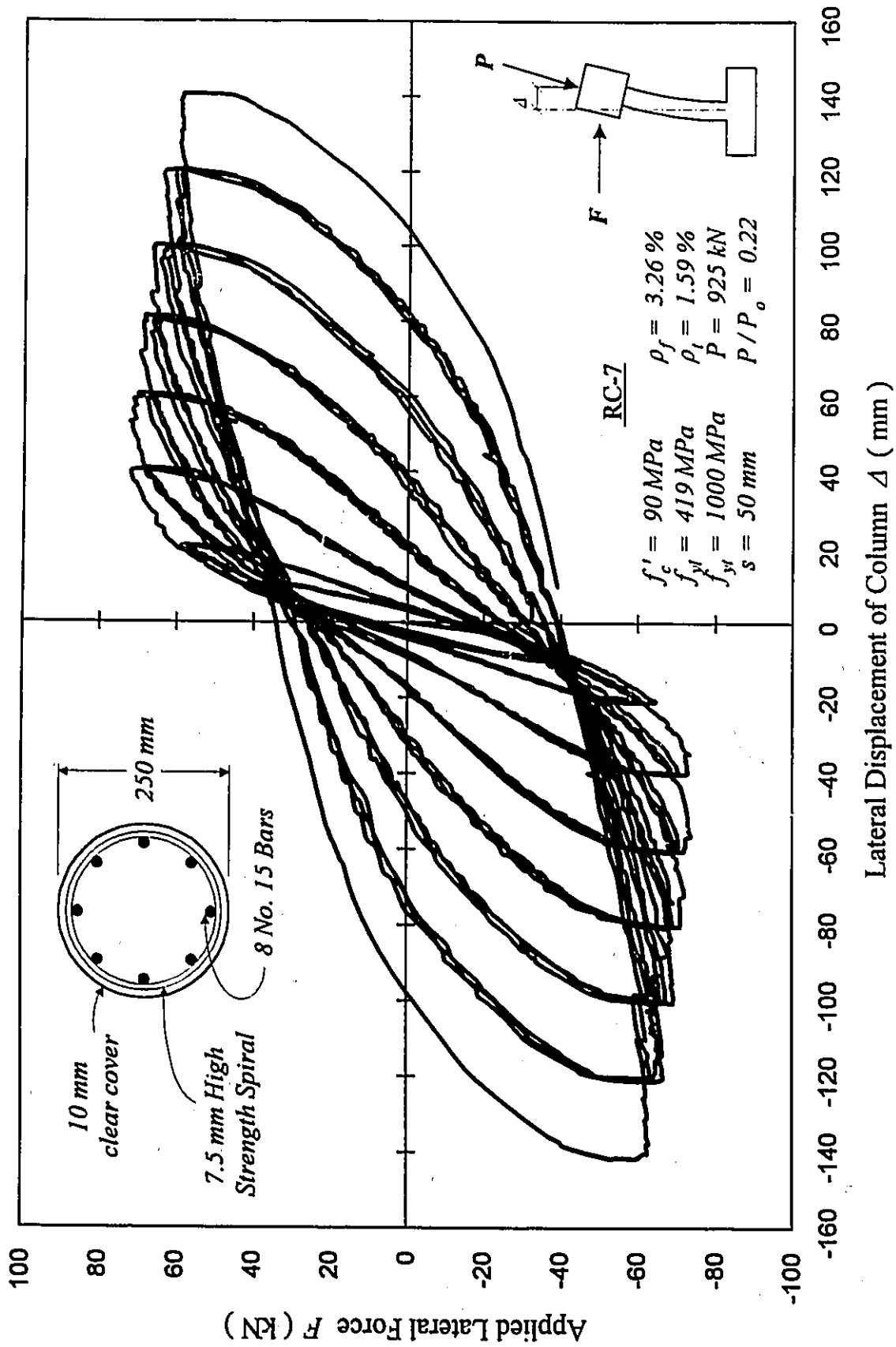


Figure 3.34: Force - Displacement Relationship for Column RC-7
 (Point of Application of Load - without $P-\Delta$ effect)

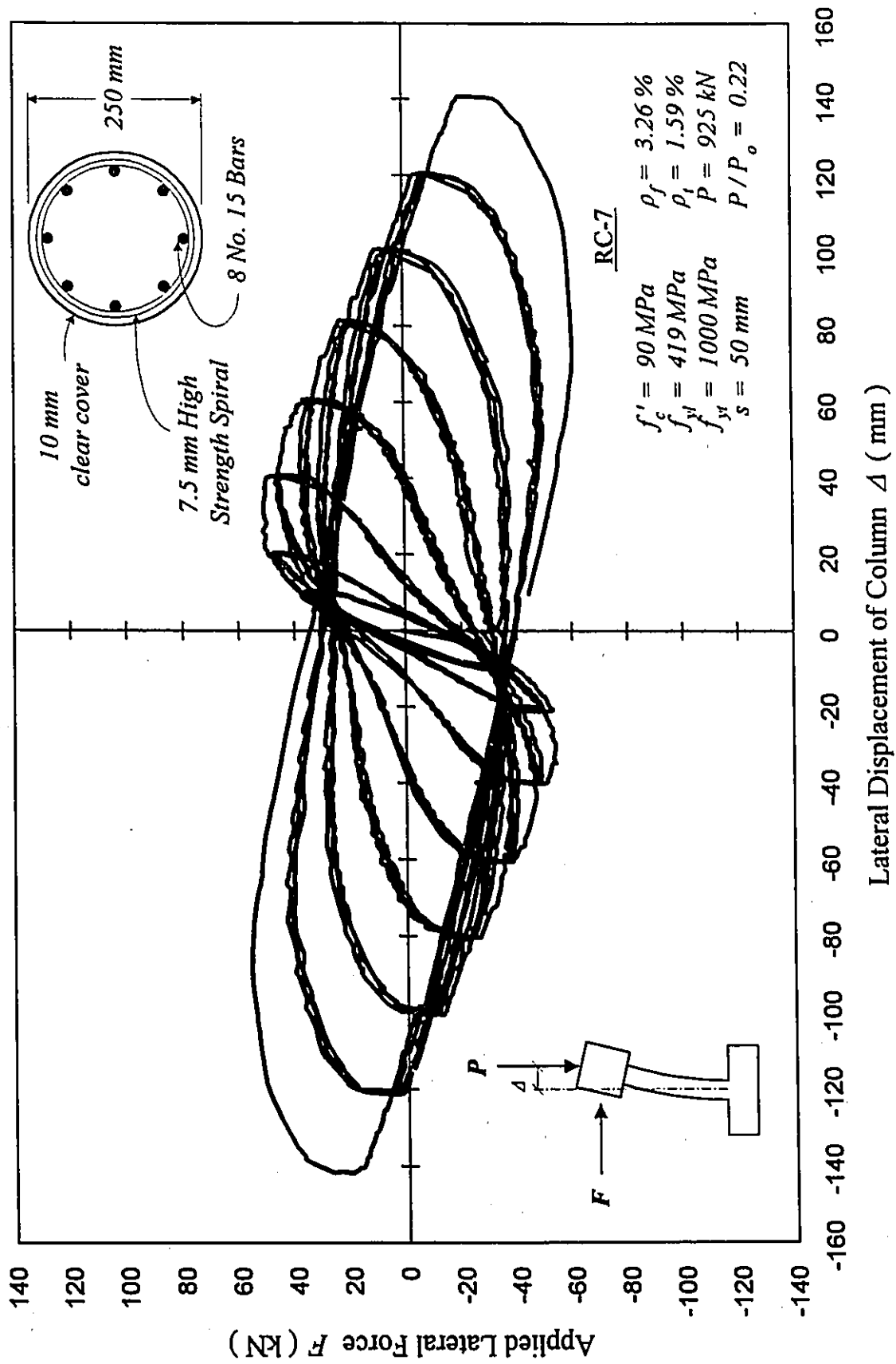


Figure 3.35: Force - Displacement Relationship for Column RC-7
(Point of Application of Load - with $P-\Delta$ effect)

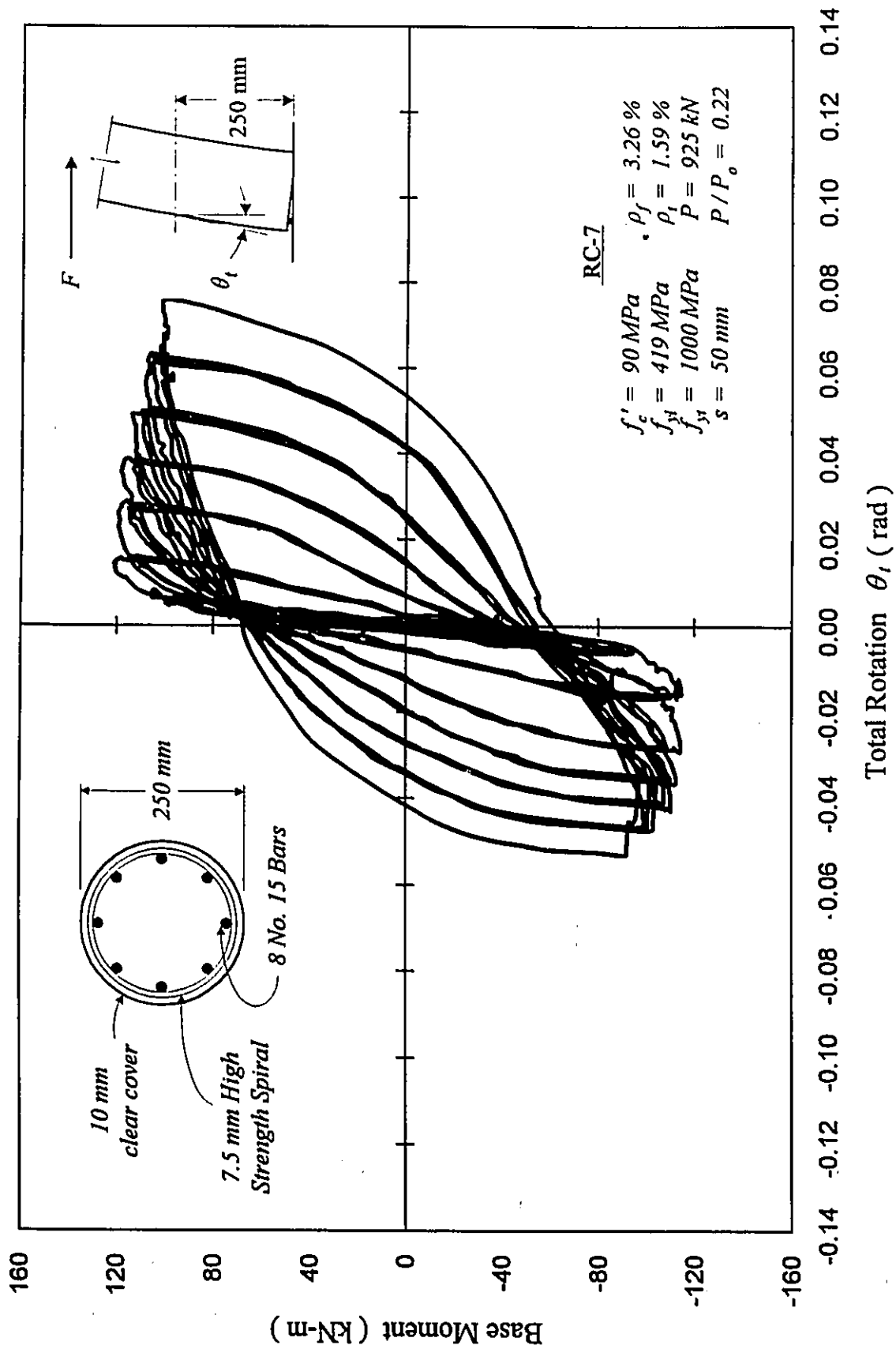


Figure 3.36: Base Moment - Total Rotation Relationship for Column RC-7
(Without $P-\Delta$ effects)

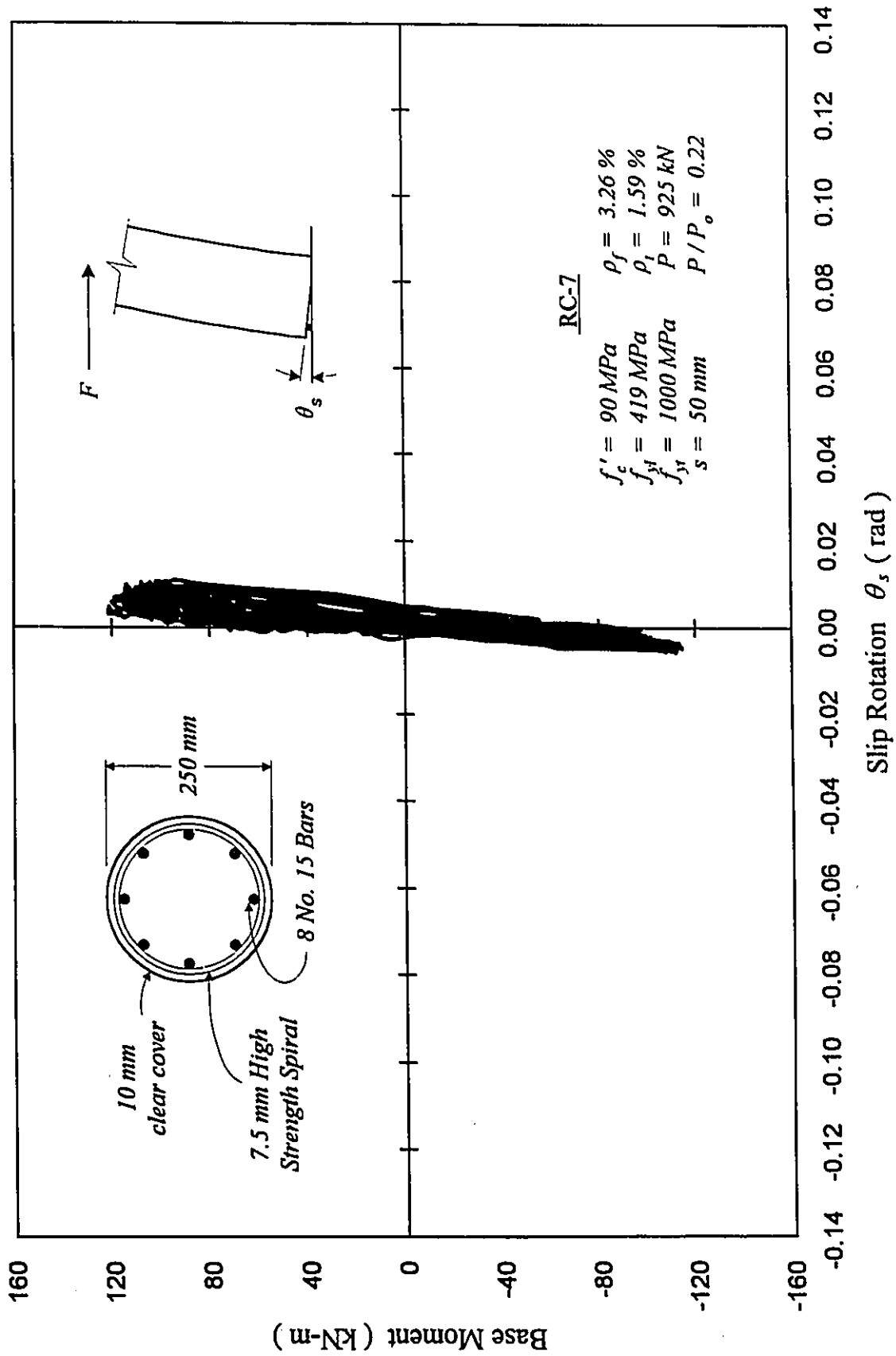
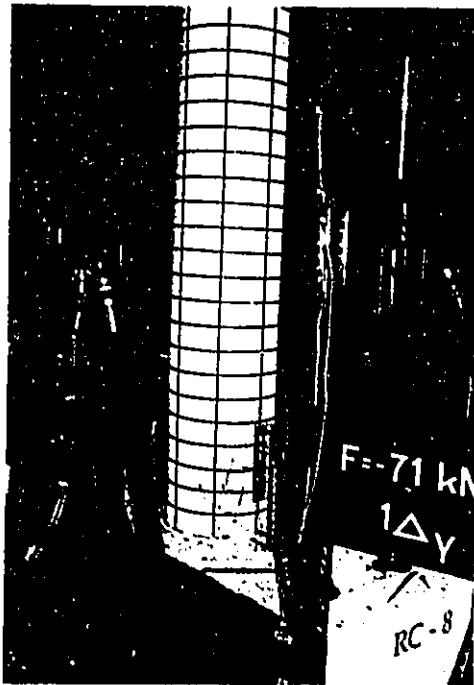
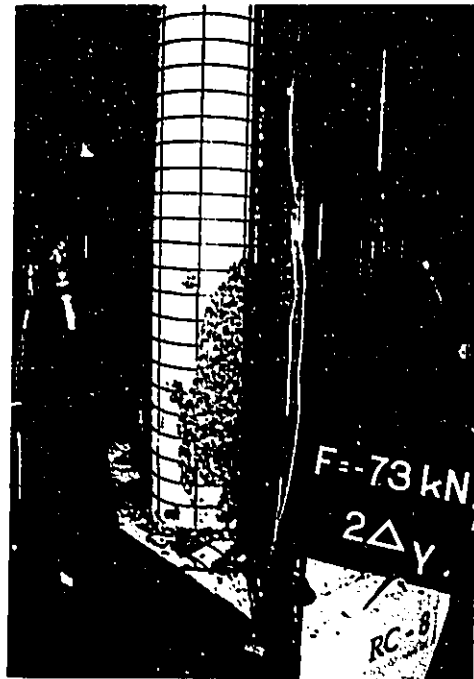


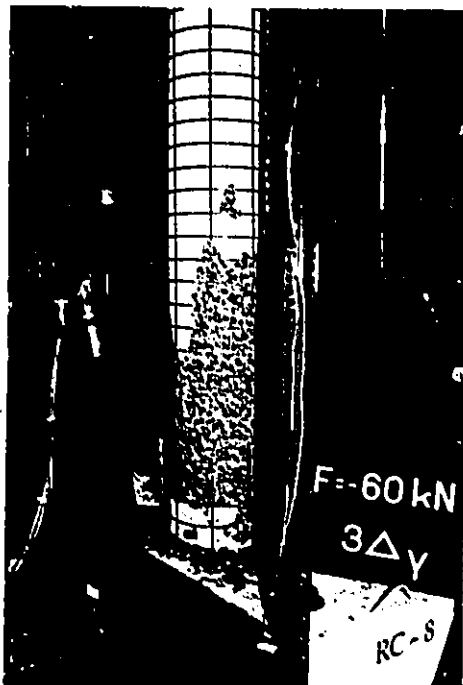
Figure 3.37: Base Moment - Slip Rotation Relationship for Column RC-7
(Without $P-\Delta$ effects)



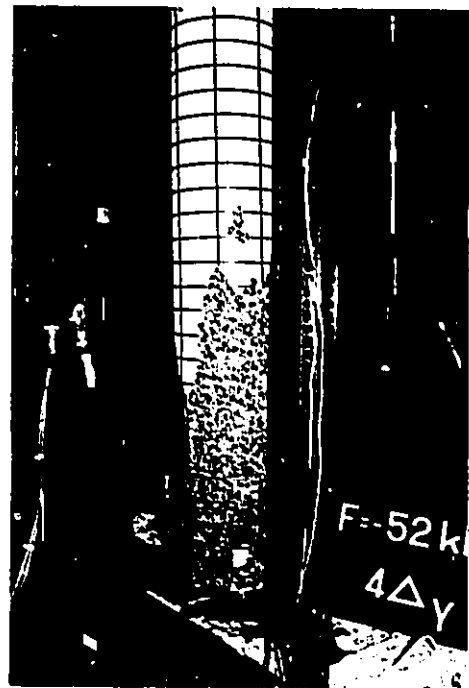
(a) $1\Delta_y$



(b) $2\Delta_y$



(c) $3\Delta_y$



(d) $4\Delta_y$

Figure 3.38: Extent of Damage for Column RC-8 at Different Stages of Testing

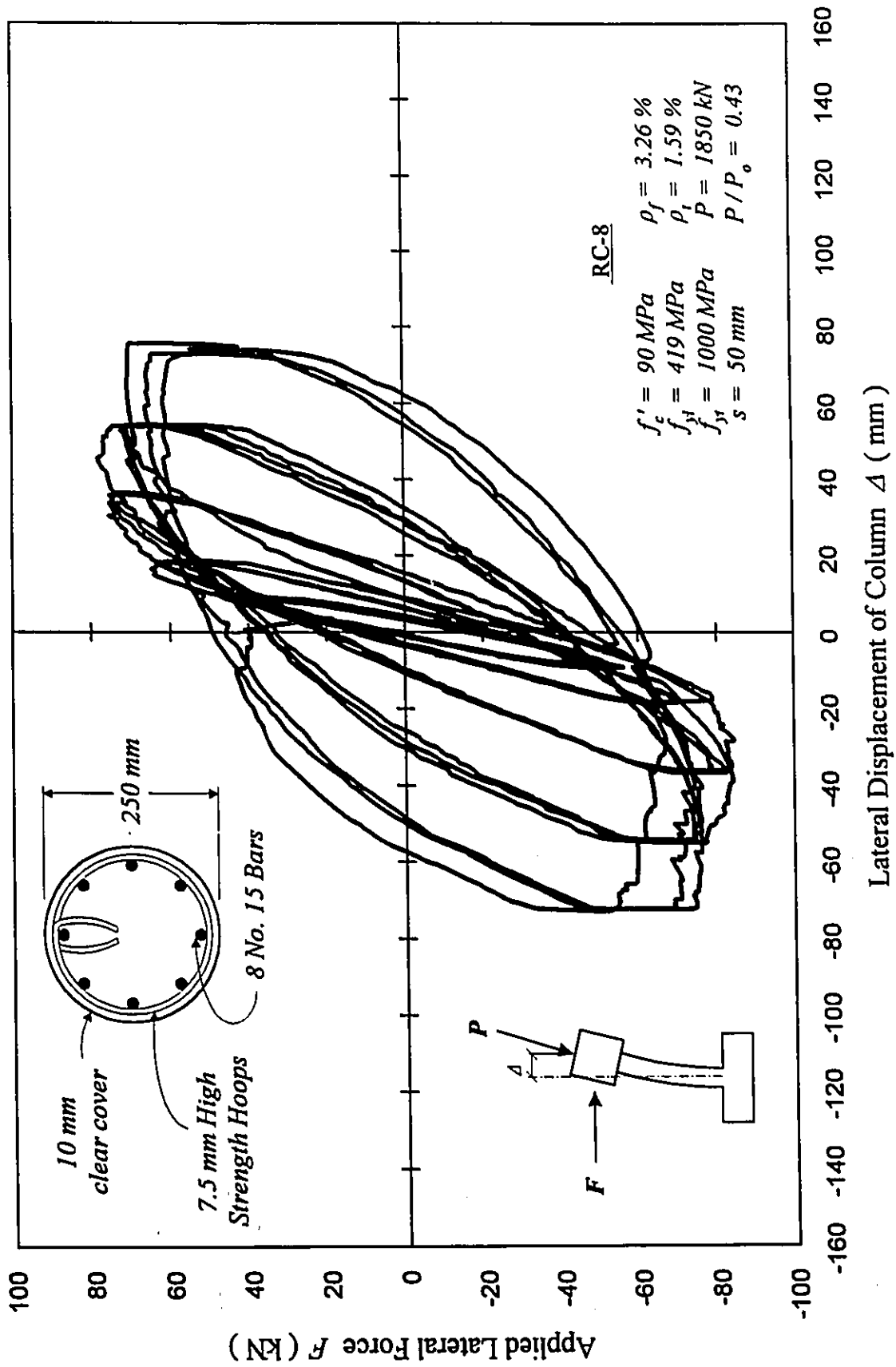


Figure 3.39: Force - Displacement Relationship for Column RC-8
(Point of Application of Load - without $P-\Delta$ effect)

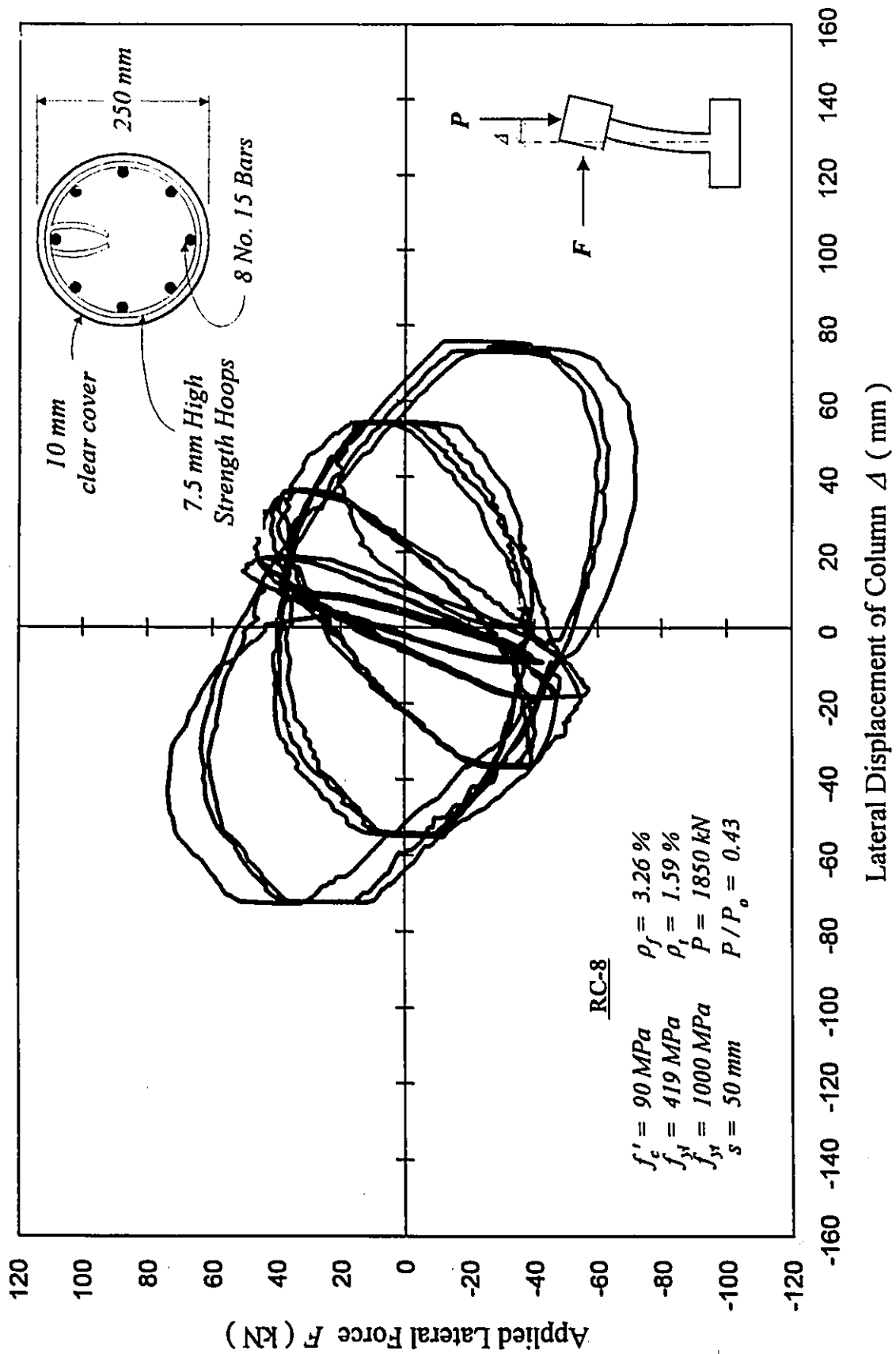


Figure 3.40: Force - Displacement Relationship for Column RC-8
(Point of Application of Load - with $P-\Delta$ effect)

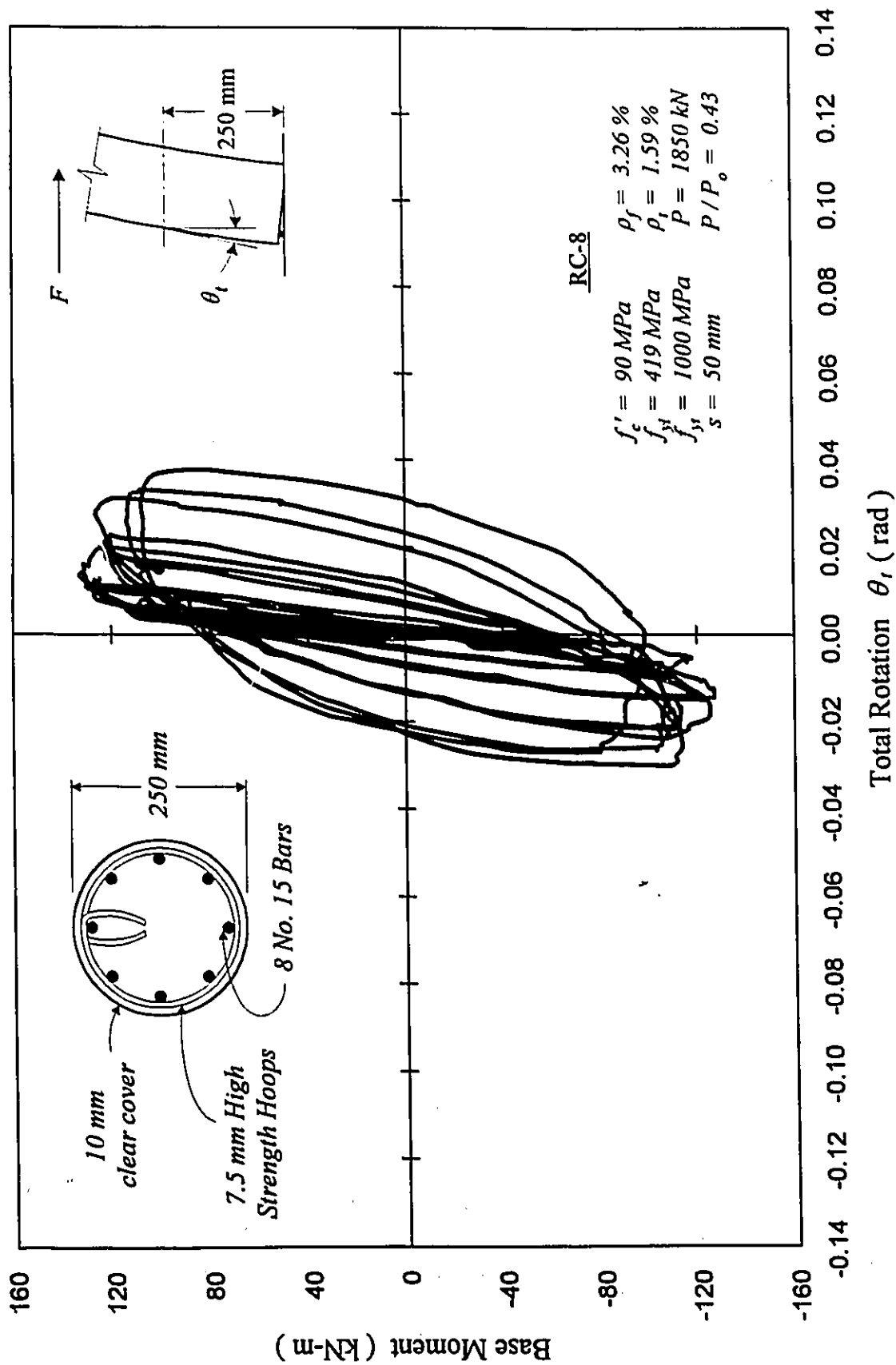


Figure 3.41: Base Moment - Total Rotation Relationship for Column RC-8
(Without $P-\Delta$ effects)

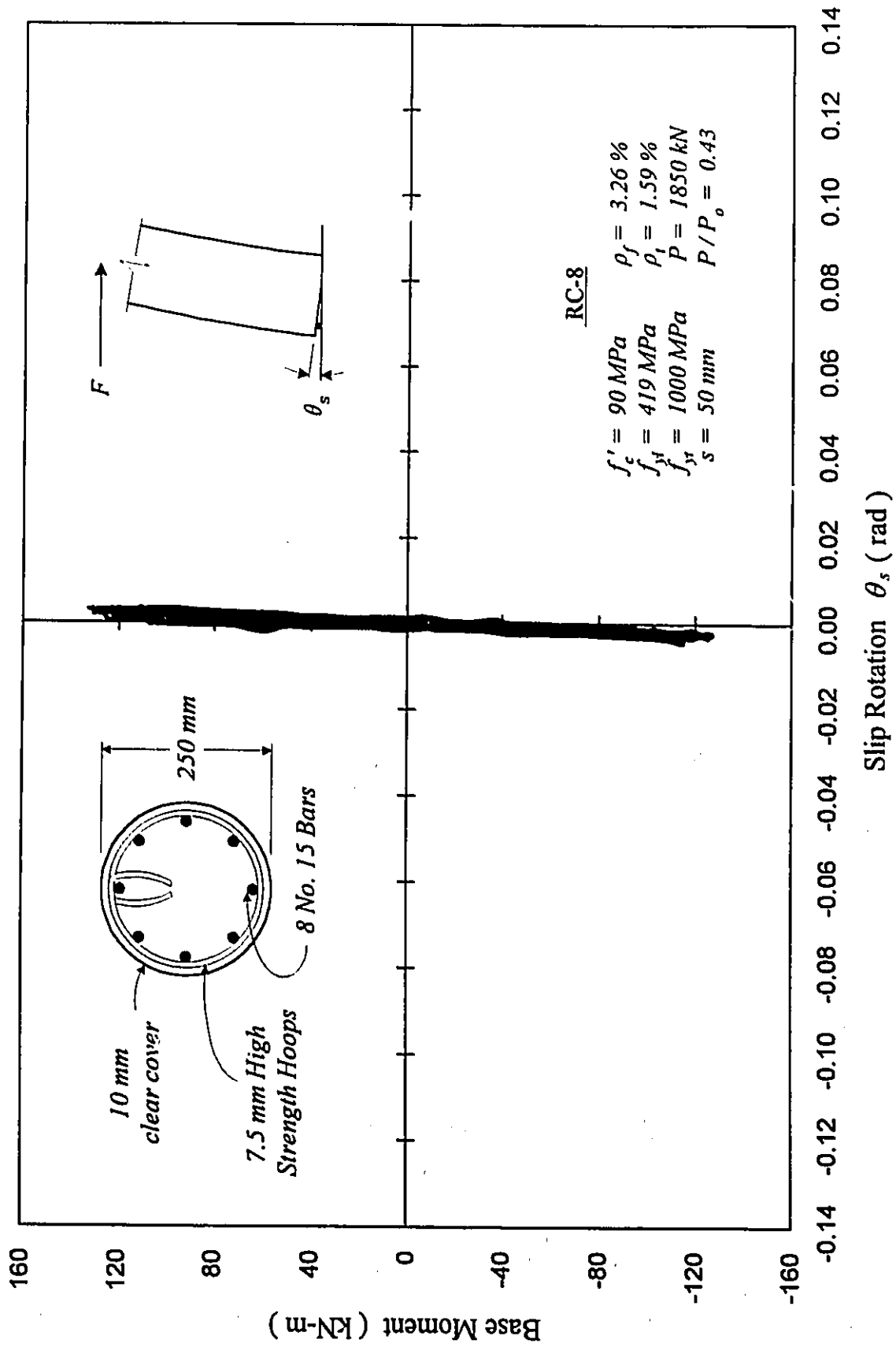
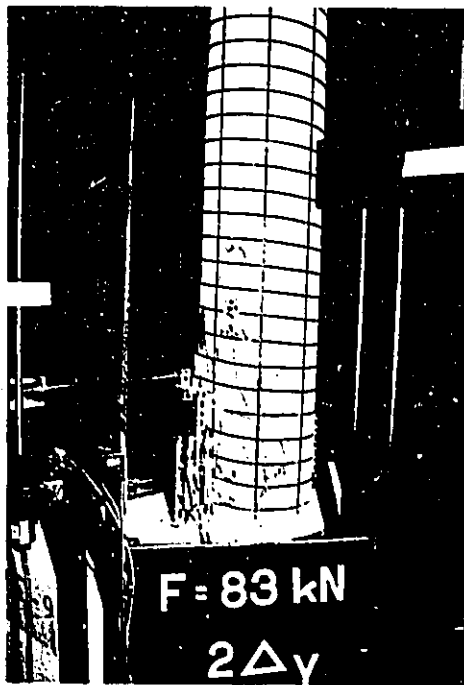


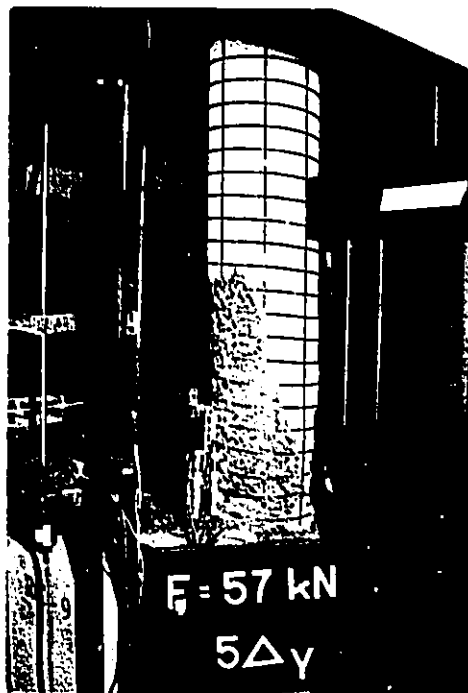
Figure 3.42: Base Moment - Slip Rotation Relationship for Column RC-8
(Without $P-\Delta$ effects)



(a) $2\Delta_y$



(b) $3\Delta_y$



(c) $5\Delta_y$



(d) $6\Delta_y$

Figure 3.43: Extent of Damage for Column RC-9 at Different Stages of Testing

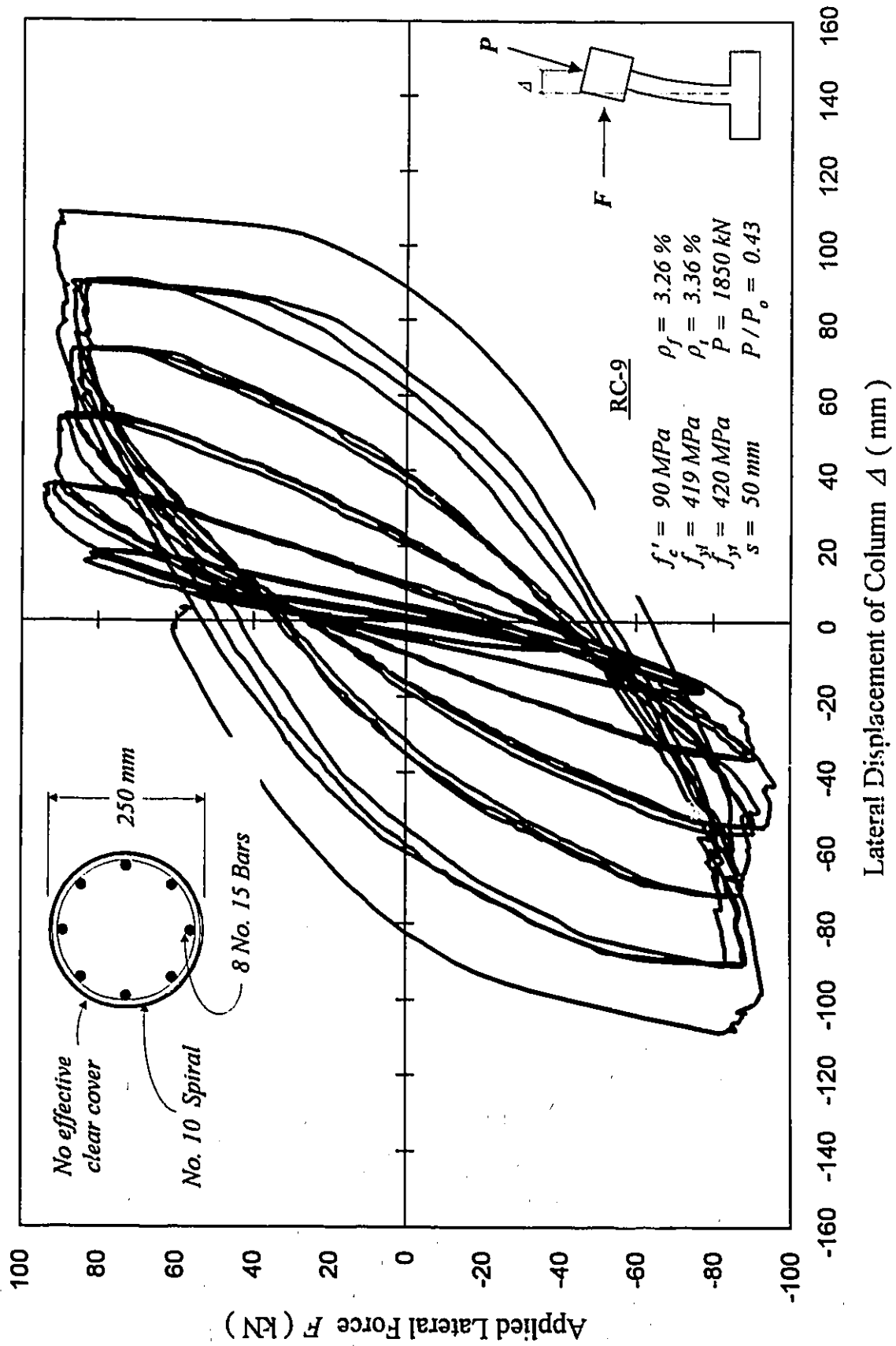


Figure 3.44: Force - Displacement Relationship for Column RC-9
(Point of Application of Load - without $P-\Delta$ effect)

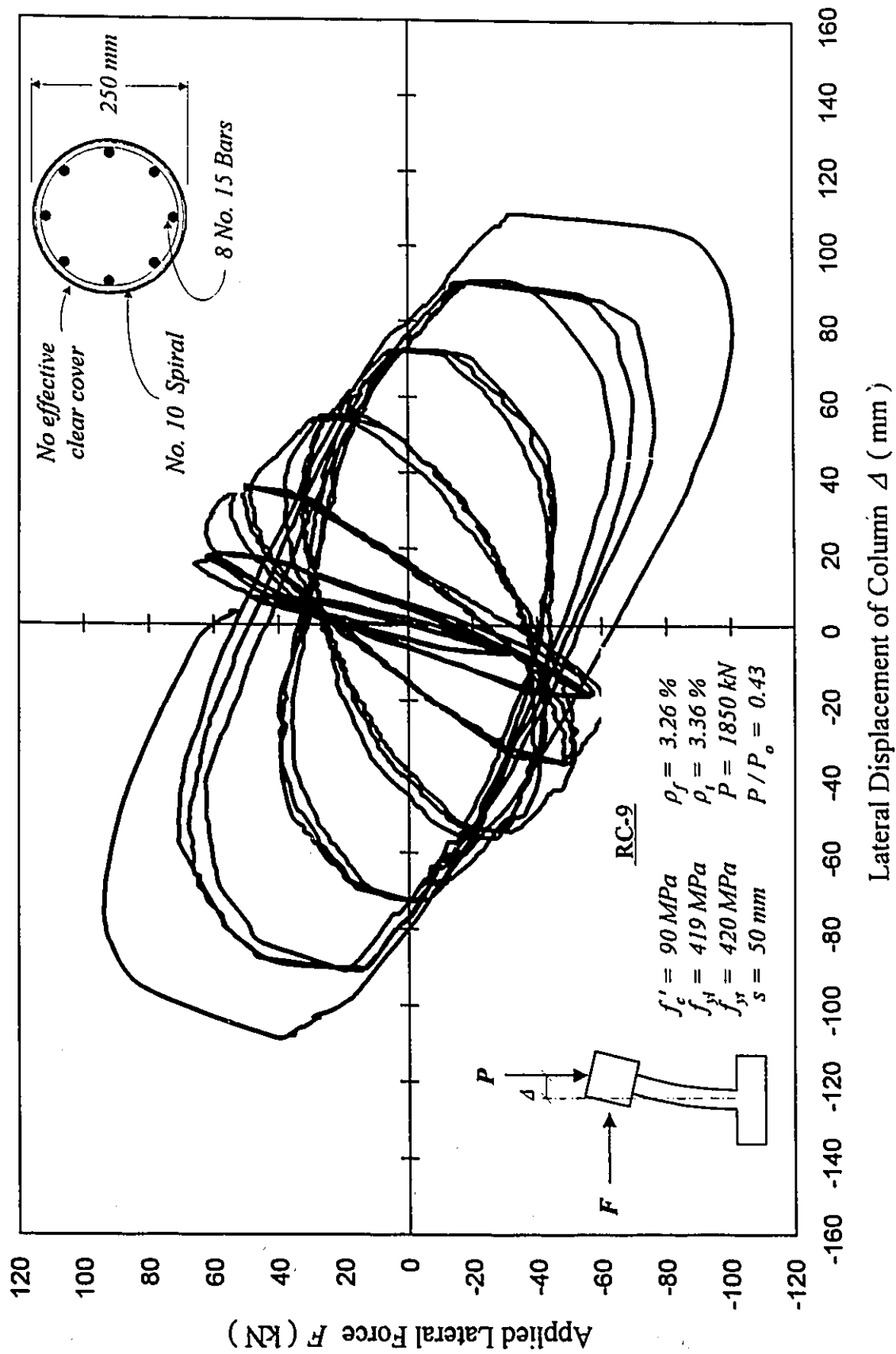


Figure 3.45: Force - Displacement Relationship for Column RC-9
 (Point of Application of Load - with $P-\Delta$ effect)

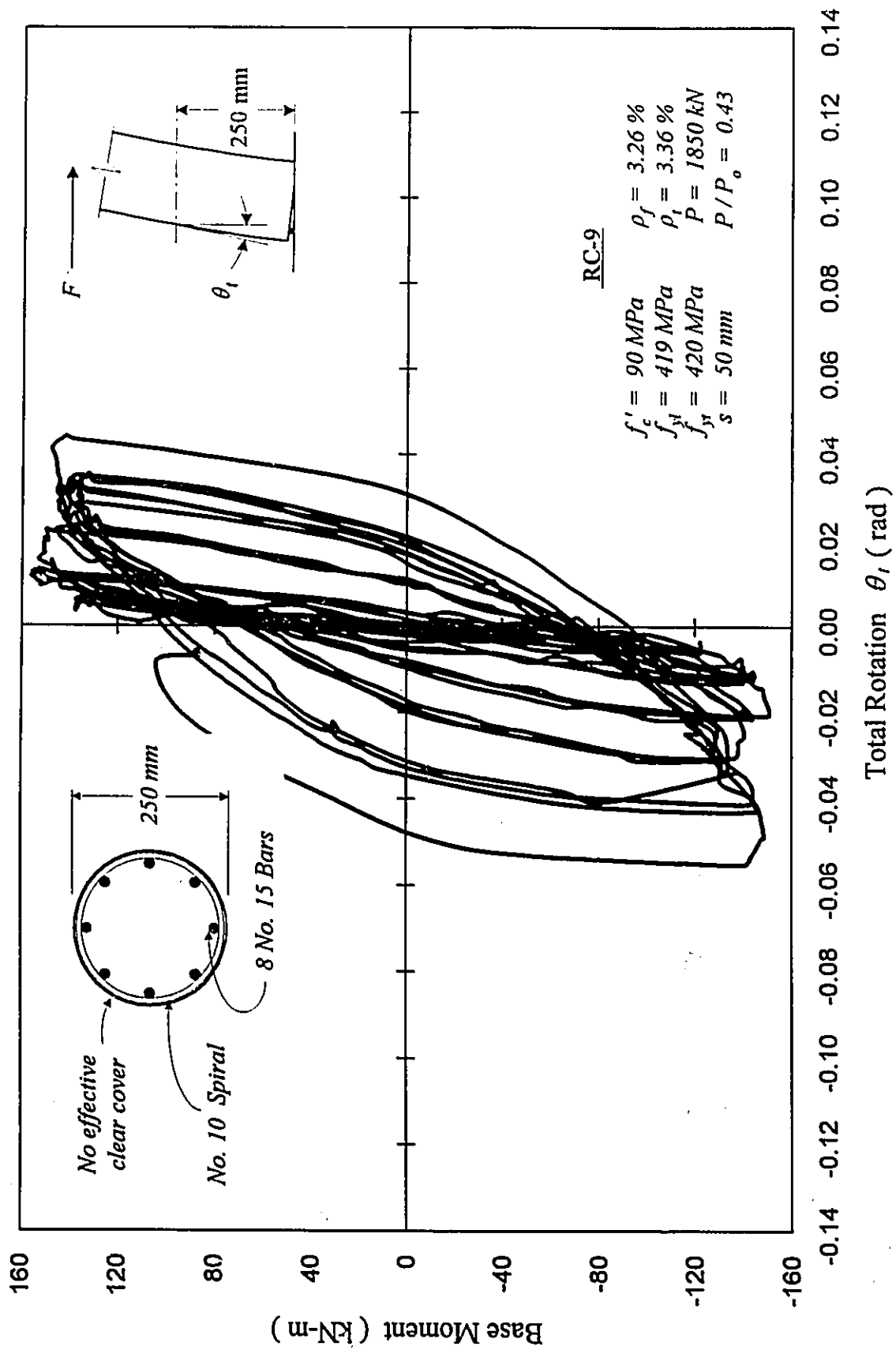


Figure 3.46: Base Moment - Total Rotation Relationship for Column RC-9
(Without $P-\Delta$ effects)

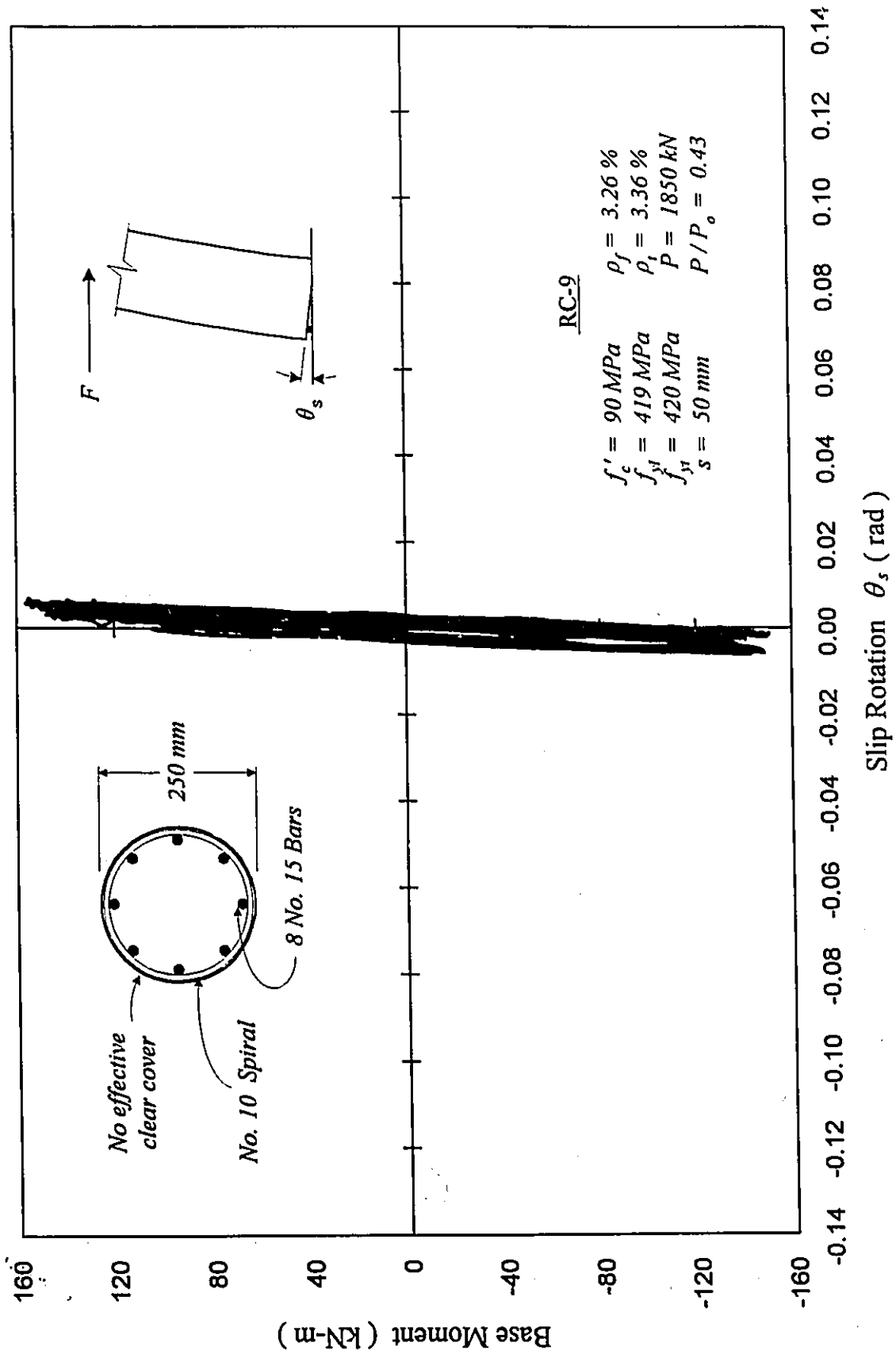


Figure 3.47: Base Moment - Slip Rotation Relationship for Column RC-9
(Without $P-\Delta$ effects)

CHAPTER 4

Effects of Test Parameters

4.1 General

The experimental data recorded during column tests were analysed and compared to assess the significance of test variables. The comparisons of columns with different test parameters are presented and discussed in this Chapter. The test data, in terms of force-displacement and moment-rotation hysteretic relationships are presented in Chapter 3. The comparison with analytical predictions are included in Chapter 5.

4.2 Test Parameters

The effects of test parameters were analysed by comparing the envelopes of hysteretic force-displacement relationships for pairs of companion specimens. In each pair of specimens the parameter under investigation was varied while all others were kept constant. The envelopes were normalized relative to peak loads to eliminate strength as a parameter. This permitted the investigation of test parameters on column ductility. The parameters considered in the experimental program are summarized below:

- 1) Compressive Strength of Concrete
- 2) Yield Strength of Transverse Reinforcement
- 3) Volumetric Ratio of Transverse Reinforcement
- 4) Spacing of Transverse Reinforcement
- 5) Axial Load Level
- 6) Type of Transverse Reinforcement (*Spiral vs. Hoop*)
- 7) Cover Concrete

4.2.1 Effect of Concrete Strength

Two sets of specimens were tested to investigate the effect of concrete compressive strength on column deformability. Each pair was confined with different grade of transverse steel, while each column in a pair was cast using different strength concrete. Column RC-1, with 65 MPa concrete, was companion to column RC-3 with 90 MPa concrete. There was a small difference in the level of axial compression applied on these two columns. RC-1 was tested under $0.30P_o$, whereas RC-3 was tested under $0.43P_o$. Both columns were confined with 1000 MPa plain spirals with a volumetric ratio of 1.59% . Force-displacement envelopes of the specimens are compared in Fig. 4.1. The results indicate that both columns exhibited ductile behaviour due to favourable confinement characteristics of high-strength steel. However, Column RC-3 with 90 MPa concrete developed a rapid strength degradation beyond 4.5% drift while Column RC-1 with 65 MPa was able to sustain its strength until 9% drift with little decay.

The other pair of columns considered were RC-2 and RC-5, with 65 MPa and 90 MPa concretes, respectively. These columns were confined with No. 10 spirals which resulted in a very high volumetric ratio of 3.67% . Therefore, both columns showed excellent inelastic deformability with very little strength decay, even in higher strength concrete column, throughout the test range. Fig. 4.2 illustrates that virtually no strength loss was observed until the end of testing. Hence, these columns did not illustrate any effect of concrete strength on ductility.

4.2.2 Effect of Yield Strength of Transverse Reinforcement

The effect of transverse steel strength on ductility was investigated by comparing Columns RC-3 and RC-4. Two grades of steel were used as transverse reinforcement, namely, 1000 MPa for column RC-3 and 580 MPa for column RC-4. The spirals were made of plain steel wires. The volumetric ratios of spirals for the two columns were slightly different from each other because of a 0.5 mm difference in diameter between the two spirals. Column RC-3 had a ratio of 1.59% while Column RC-4 had a ratio of 1.81% . All other parameters were identical in both columns, including concrete strength (90 MPa) and axial load level ($0.43P_o$).

The force-displacement envelopes of these columns are compared in Fig. 4.3. The results indicate

that column RC-3, confined with 1000 MPa steel, developed a higher deformability than Column RC-4 confined with 425 MPa steel. No significant strength degradation was observed in RC-3 until a lateral drift of 4.5%, whereas Column RC-4 failed in a brittle manner at 2.9% drift. Furthermore, the strain gauge data reported in Chapter 3 indicates that the high-strength confinement steel used in specimen RC-3 began yielding before a lateral drift of 2% was reached. This implies that the lateral expansion of high-strength concrete was sufficient to fully utilize the benefits of high-strength transverse reinforcement.

4.2.3 Effect of Volumetric Ratio of Transverse Reinforcement

The effect of volumetric ratio of transverse steel was studied by comparing the behaviour of two 90 MPa columns, RC-4 and RC-5, with volumetric ratios, ρ_t of 1.81% and 3.67%, respectively. Column RC-4 was confined with an 8 mm diameter spiral, while the transverse reinforcement of RC-5 consisted of a standard deformed No. 10 spiral. The remaining confinement parameters were common to both specimens. The normalized force-displacement envelopes for the two specimens are presented in Figure 4.4. The figure indicates that column RC-4 with a lower volumetric ratio behaved in an extremely brittle manner. After reaching its peak load at approximately 2.9% lateral drift, it showed a rapid strength decay to the point of failure at about 3.5% drift. In contrast, Column RC-5 showed virtually no strength degradation up to 4.6% drift, followed by a slow and gradual reduction in its lateral capacity. The figure clearly shows the impact of the volumetric ratio on ductility of high-strength concrete columns.

Further analysis of test data indicates a strong relationship between the volumetric ratio and grade of transverse reinforcement. This becomes evident when a column with low volumetric ratio of higher grade steel is compared with another column with a high volumetric ratio of lower grade steel. Figure 4.5 shows the force-displacement envelopes for Column RC-3 and RC-5. Column RC-5 with a high volumetric ratio of 3.67% shows a ductile behaviour. Column RC-3, on the other hand, also shows a ductile response even with a low volumetric ratio of 1.59%, because of the higher grade of 1000 MPa steel employed in this specimen. A companion column (RC-4) with approximately the same volumetric ratio but grade 425 MPa steel shows a brittle behaviour as illustrated in Fig. 4.4. These comparisons underline the importance of the product $\rho_t f_{yt}$ as a design parameter. Similarly, columns RC-1 and RC-2 with 65 MPa concrete are compared in Fig. 4.6.

These columns were confined with different volumetric ratios and grades of transverse steel. However, the product ρf_{yt} remained approximately constant, implying that they developed approximately the same hoop tension irrespective of the difference in volumetric ratio. The results indicate that these two columns behaved in a very similar manner, both showing ductile response. One may conclude from these comparisons that the high volumetric steel ratio requirement for high-strength concrete columns can be partly replaced with the use of higher grade transverse reinforcement.

4.2.4 Effect of Spacing of Transverse Reinforcement

Columns RC-5 and RC-6, with two different pitch of spiral steel, are compared in Fig. 4.7 to investigate the effect of spacing of transverse reinforcement on ductility of high-strength concrete columns. These columns were cast using 90 MPa concrete and were confined with 3.67% volumetric ratio of grade 420 MPa steel. The spiral pitch in Column RC-5 was 50 mm as compared to that of Column RC-6, which was 100 mm . To maintain a constant volumetric ratio with different pitch, Column RC-6 was confined with two No. 10 spirals (in double layer) while RC-5 had a single spiral. Both columns were tested under constant axial compression of $0.43P_o$.

The results indicate that deformability is reduced with increase in spiral pitch. Column RC-5 with 50 mm ($h/5$) spacing achieved ductile behaviour with little strength decay up to 6% drift, at which deformation level the test was stopped. Column RC-6 on the other hand, attained a drift of only 4% when it began to fail due to the buckling of longitudinal reinforcement. This clearly illustrates the importance of spacing on column ductility. Close spacing of transverse reinforcement improves concrete confinement, while also preventing premature buckling of longitudinal reinforcement.

4.2.5 Effect of Axial Load Level

The comparison of normalized force-displacement envelopes of columns RC-3 and RC-7, shown in Figure 4.8, is used to illustrate the effect of axial compression on column deformability. These two columns had identical characteristics except for the level of axial compression used during testing. Column RC-3 was tested under an axial load level of $0.43P_o$, while RC-7 was loaded to

$0.21P_o$, where P_o represents concentric column capacity. The figure indicates that while both specimens showed ductile behaviour, the deformability of Column RC-3 was reduced due to higher axial compression. Column RC-7, subjected to $0.21P_o$, developed 80% of its lateral force capacity at a drift of 7.7%. In contrast, Specimen RC-3, loaded to 43% of P_o , experienced 20% strength degradation at 5.9% lateral drift. It should be noted that the detrimental effect of high axial compression was not emphasized in this comparison, essentially because both columns were well confined with grade 1000 MPa transverse reinforcement.

4.2.6 Effect of Type of Circular Reinforcement

The difference between circular spirals and closed circular hoops was investigated to determine the significance of continuity in transverse reinforcement. Two columns were specifically designed to make this comparison possible. Columns RC-3 and RC-8 were identical in every respect with the exception that the former was confined with continuous spirals while the latter was confined with individual circular hoops. The volumetric ratio and yield strength of lateral steel were the same in both specimens. The ends of the hoops were made into standard 90° hooks, as recommended by CSA A23.3-1984, to prevent opening of their ends under lateral pressure.

The comparison of force-displacement envelopes for the two columns is presented in Figure 4.9. The figure indicates that both columns behaved approximately the same, while the specimen with spiral reinforcement showed only a slightly better deformability than the specimen confined by circular hoops. Almost no strength degradation was observed in Column RC-3 up to 4.5% drift. This was then followed by a rapid rate of decay to 80% of the column lateral load capacity at 5.9% drift. The envelope of Column RC-8, on the other hand, indicates a slow and gradual strength decay beyond the peak load, which occurred at 2.6% drift. The test had to be aborted when local buckling of longitudinal reinforcement was noticed at a drift of 4.5%. The damage observed after testing indicated that unsupported segments of the hoops, those between longitudinal bars, began to straighten out within the hinging region. This phenomenon provided an additional room to initiate the instability of longitudinal bars. This type of behaviour was not observed in columns confined with continuous spirals. Hence, while the improvement in deformability of spirally reinforced columns was only marginal, the effectiveness of hoop steel seemed to be reduced at later stages of inelastic behaviour by potential instability of longitudinal

reinforcement.

4.2.7 Effect of Cover Concrete

One of the columns tested (RC-9) was prepared without any cover concrete to investigate the effect of cover spalling on column deformability. This resulted in two companion columns to be compared, Column RC-5 and Column RC-9. The normalized force-displacement envelopes are presented in Figure 4.10. Column RC-5 had a clear cover of *10 mm* while no effective cover was provided in Column RC-9. The outside diameter of the two columns was the same and it was equal to *250 mm*. This implies that the confined core area of RC-9 was slightly larger than that of RC-5. The resulting difference in volumetric ratio of transverse reinforcement was considered to be negligible. The comparison indicates that the cover concrete in RC-5 did not affect the ductility of columns considered, although RC-5 showed some strength degradation at large deformations, while RC-9 maintained its capacity without any strength decay. However, a general conclusion cannot be reached due to the small proportion of cover concrete used in these specimens.

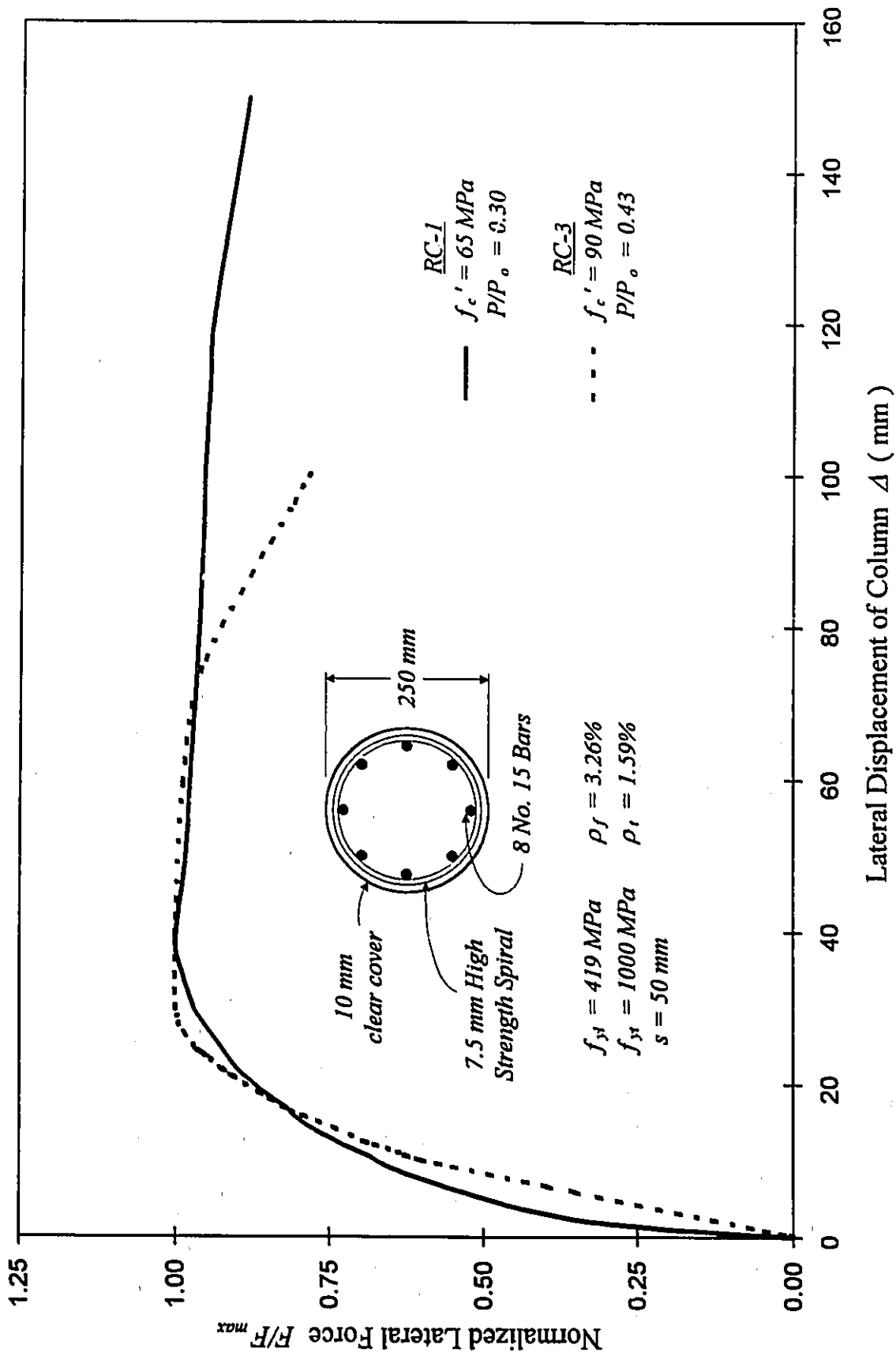


Figure 4.1: Normalized Force - Displacement Envelopes for Columns RC-1 and RC-3
 (Effect of Concrete Strength - without $P-\Delta$ effect)

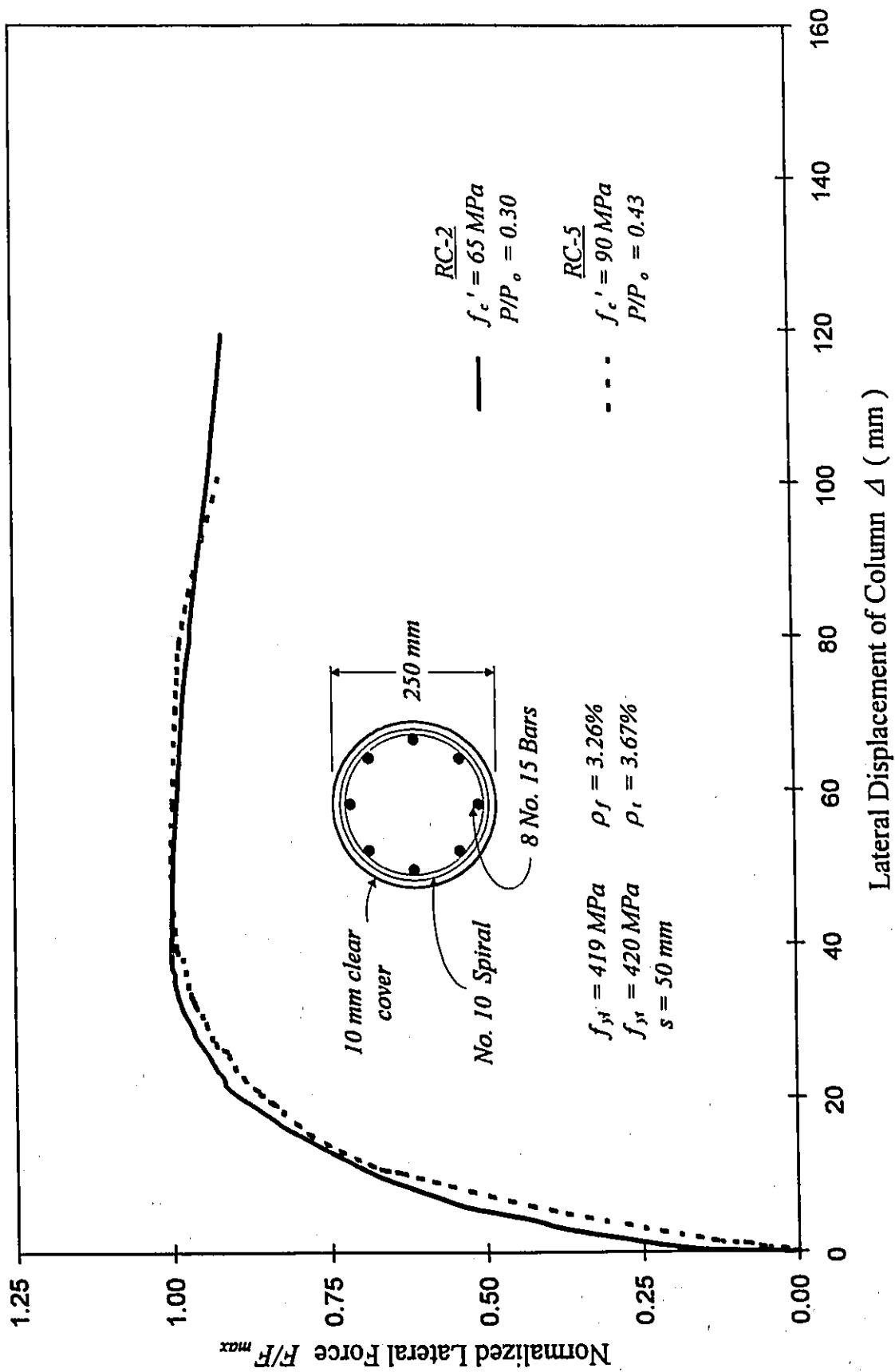


Figure 4.2: Normalized Force - Displacement Envelopes for Columns RC-2 and RC-5
(Effect of Concrete Strength - without $P-\Delta$ effect)

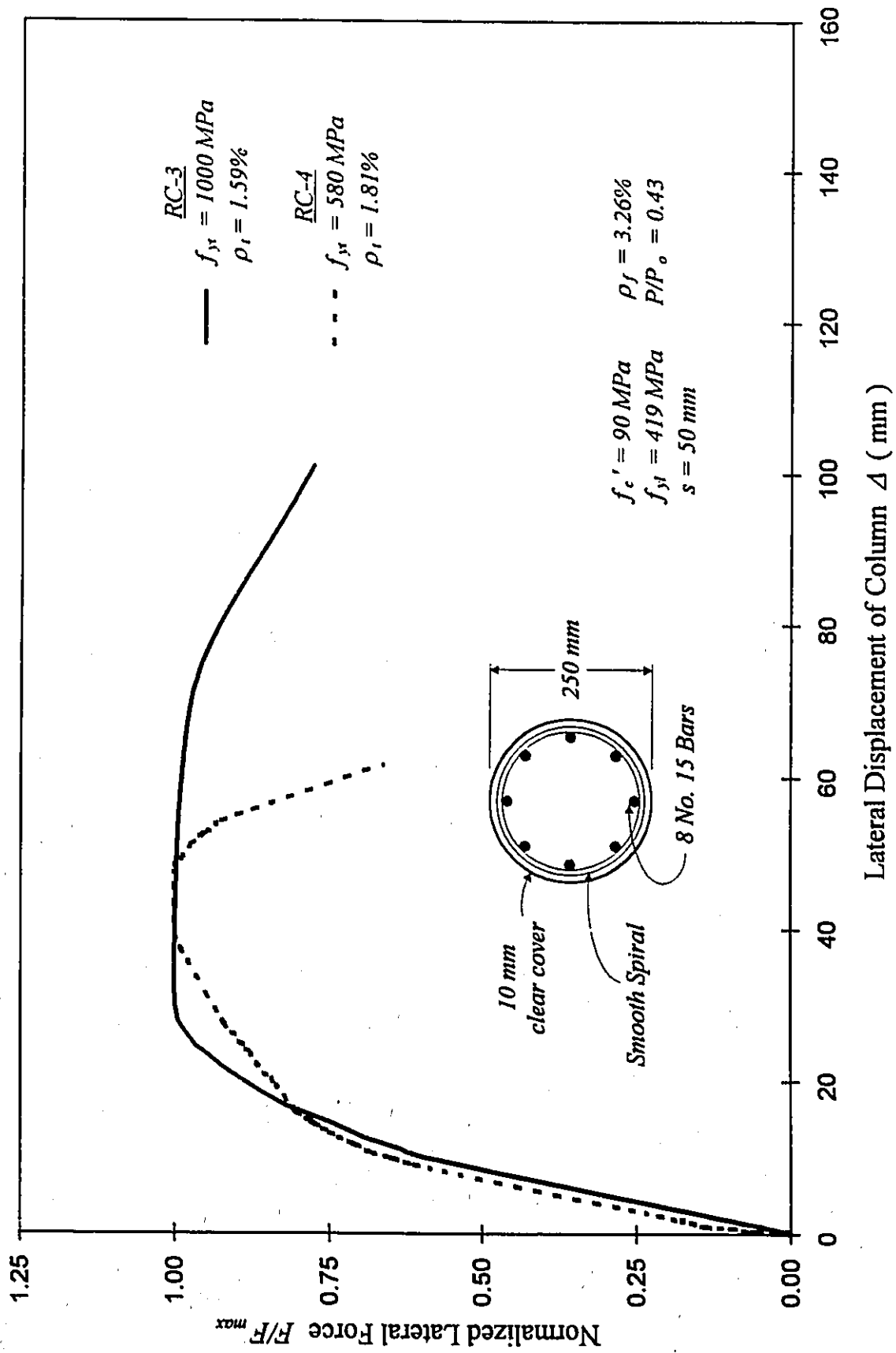


Figure 4.3: Normalized Force - Displacement Envelopes for Columns RC-3 and RC-4
 (Effect of Transverse Steel Yield Strength - without $P-\Delta$ effect)

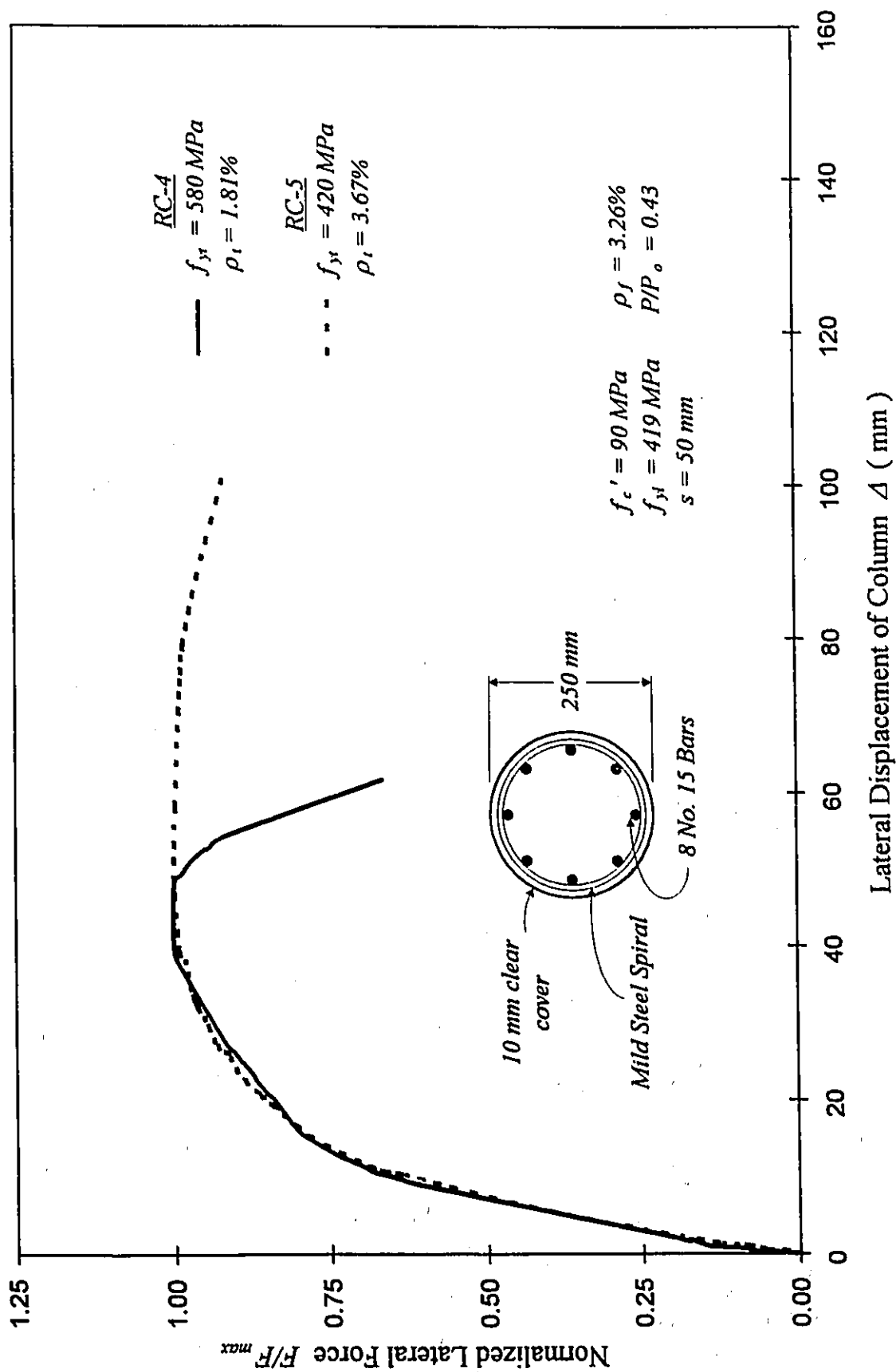


Figure 4.4: Normalized Force - Displacement Envelopes for Columns RC-4 and RC-5
 (Effect of Transverse Steel Volumetric Ratio - without $P-\Delta$ effect)

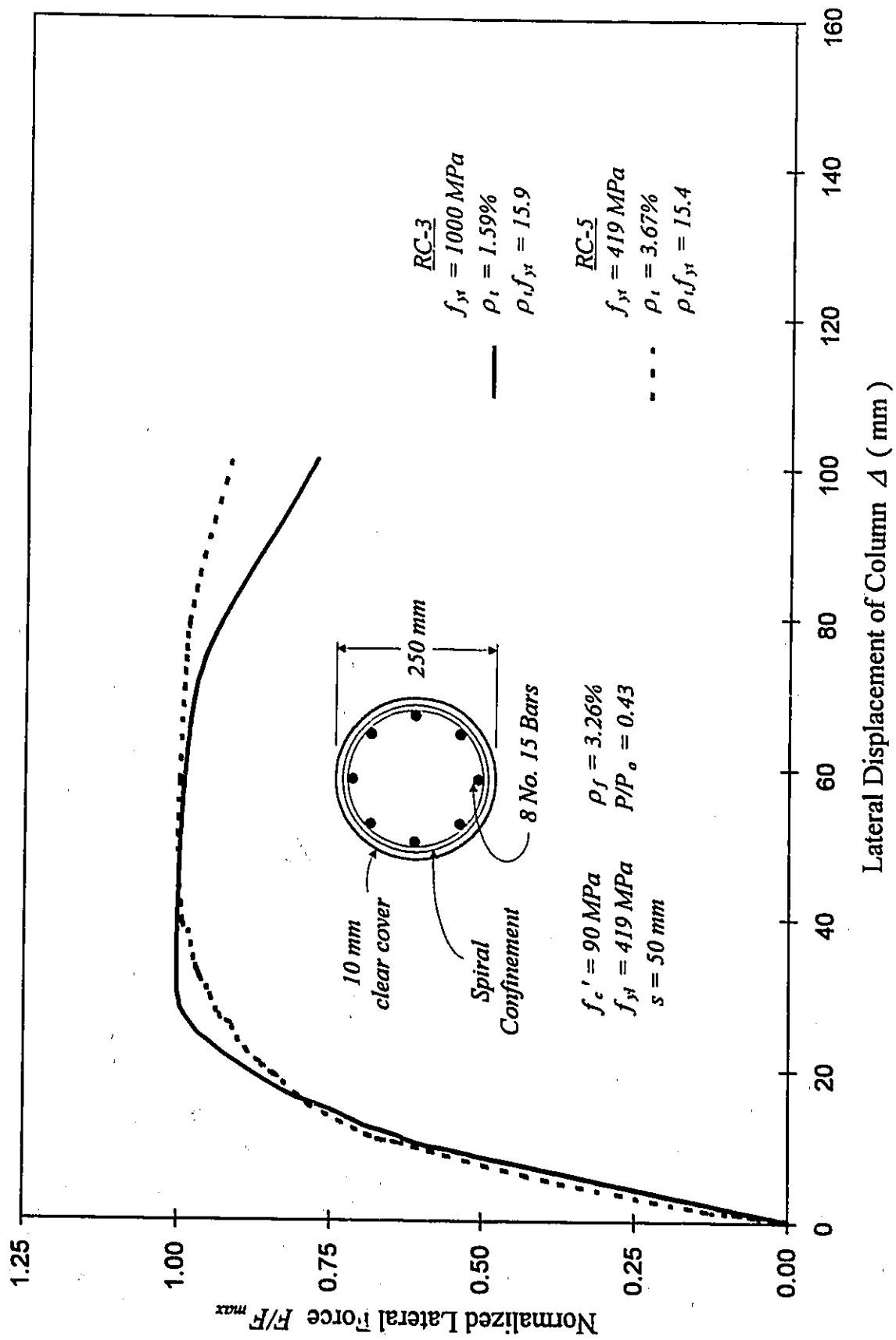


Figure 4.5: Normalized Force - Displacement Envelopes for Columns RC-3 and RC-5
 (Effect of Product $\rho_t f_y$ as a Design Parameter - without $P-\Delta$ effect)

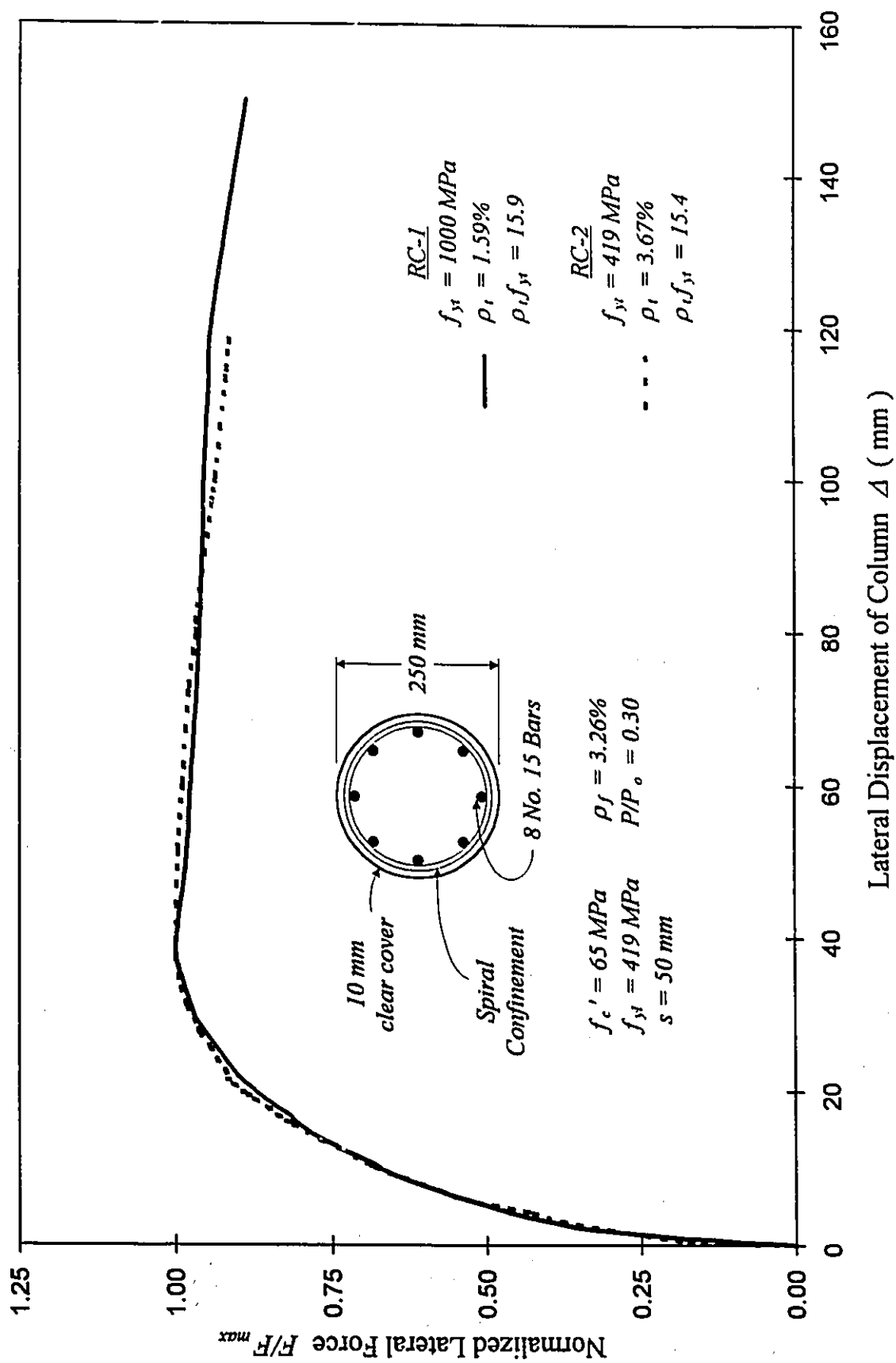


Figure 4.6: Normalized Force - Displacement Envelopes for Columns RC-1 and RC-2
 (Effect of Product $\rho_t f_{yt}$ as a Design Parameter - without $P-\Delta$ effect)

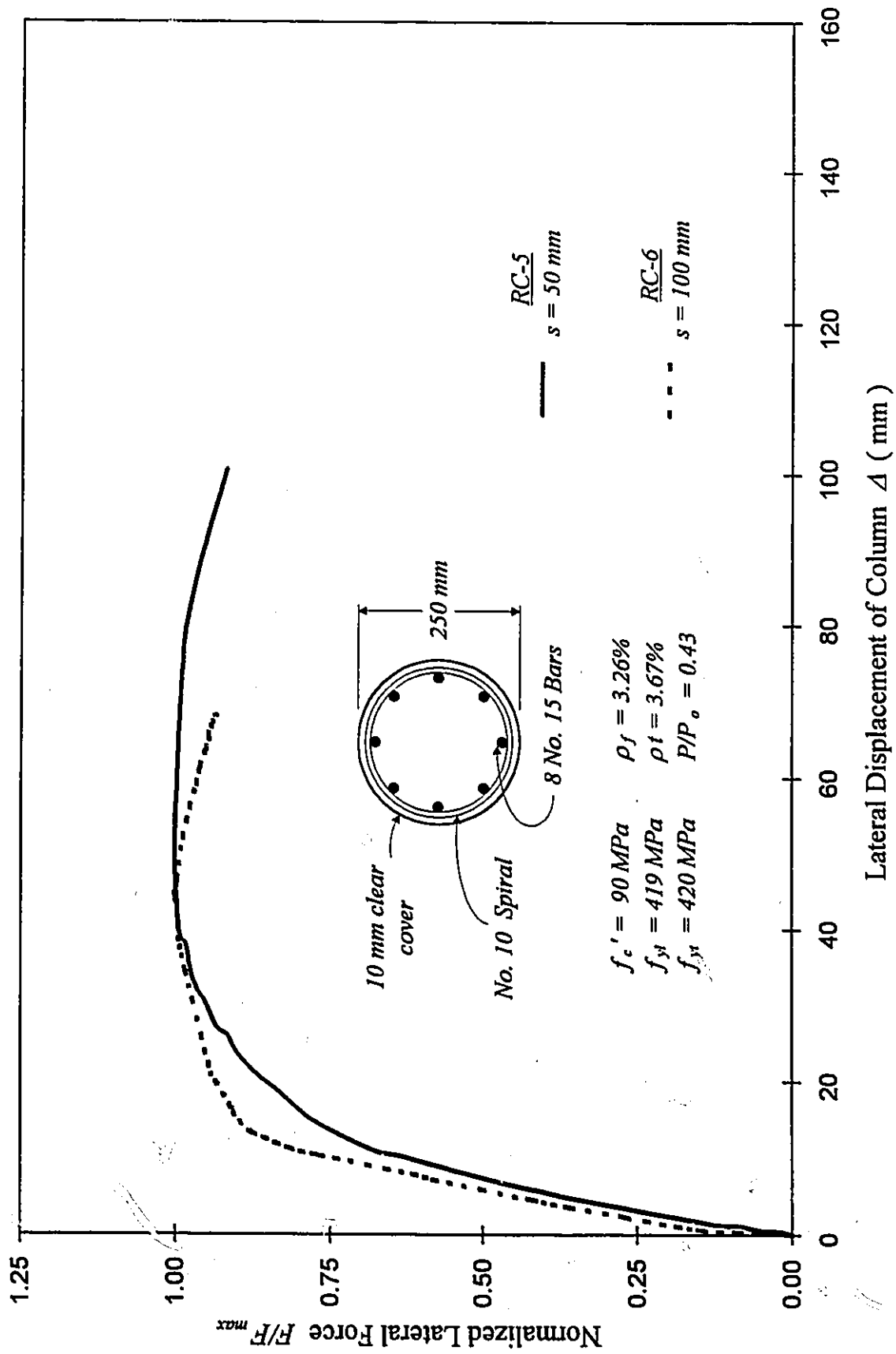


Figure 4.7: Normalized Force - Displacement Envelopes for Columns RC-5 and RC-6
(Effect of Transverse Reinforcement Spacing - without $P-\Delta$ effect)

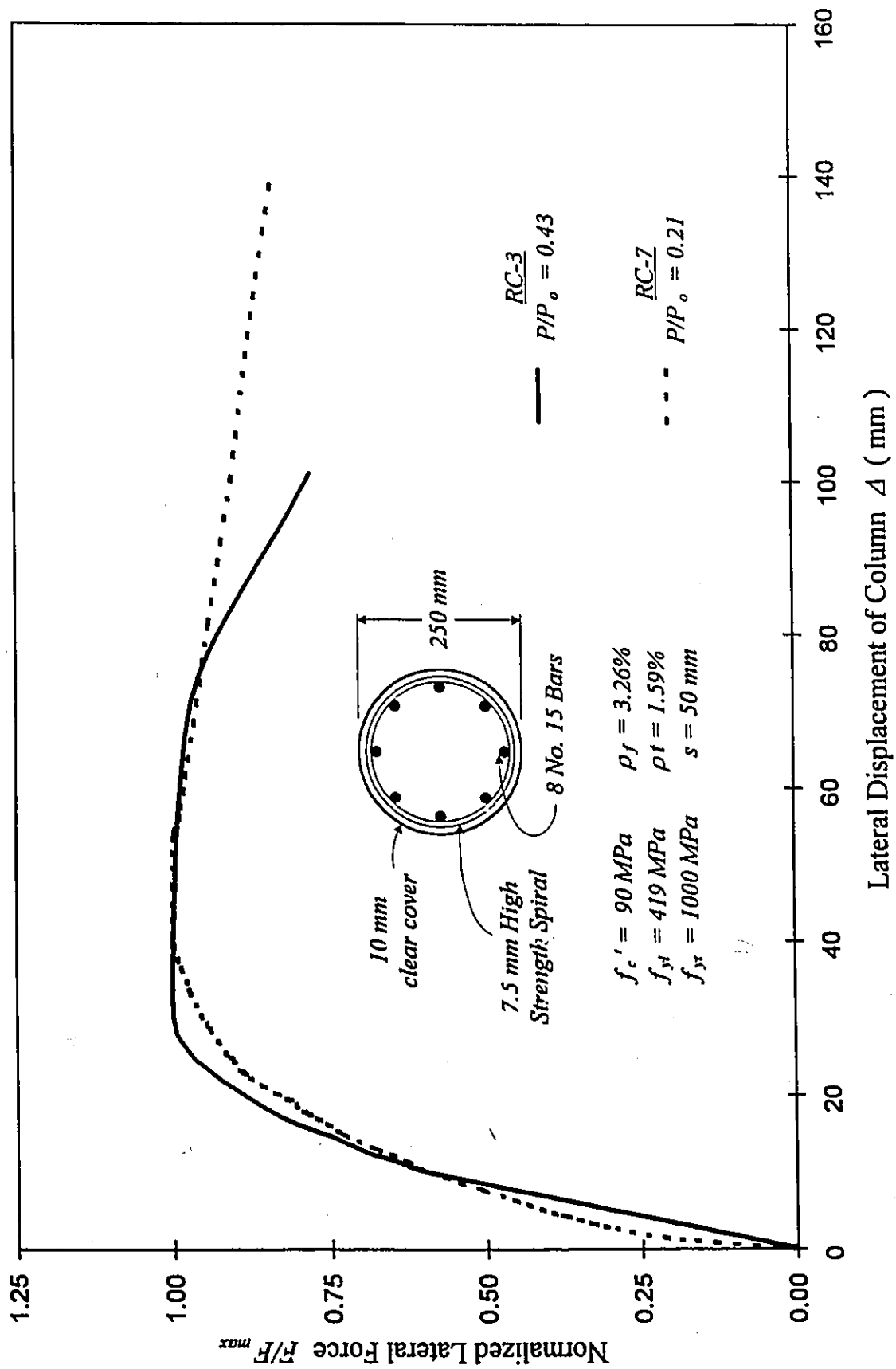


Figure 4.8: Normalized Force - Displacement Envelopes for Columns RC-3 and RC-7
 (Effect of Axial Load Level - without $P-\Delta$ effect)

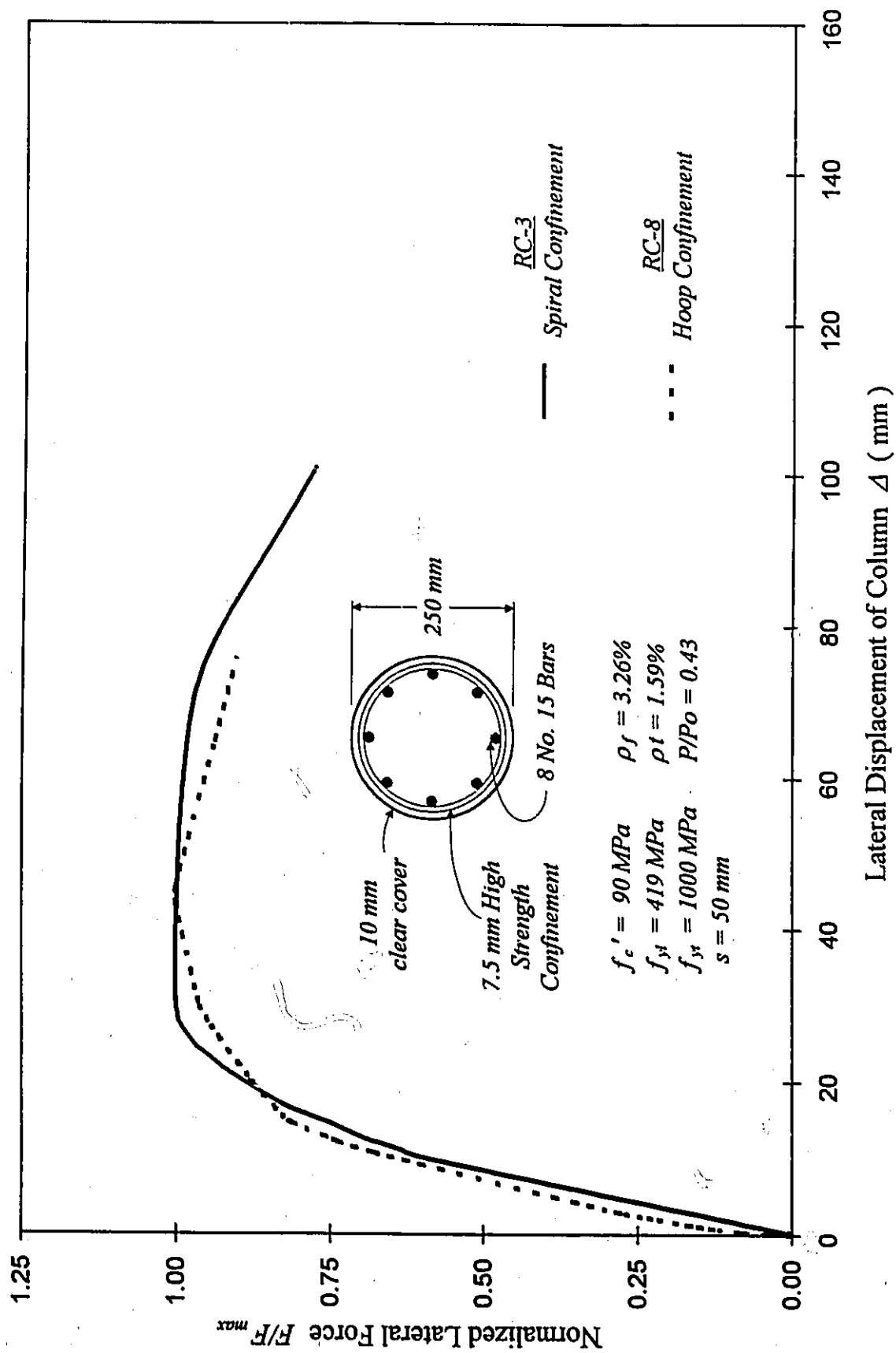


Figure 4.9: Normalized Force - Displacement Envelopes for Columns RC-3 and RC-8
 (Effect of Transverse Reinforcement Type - without P- Δ effect)

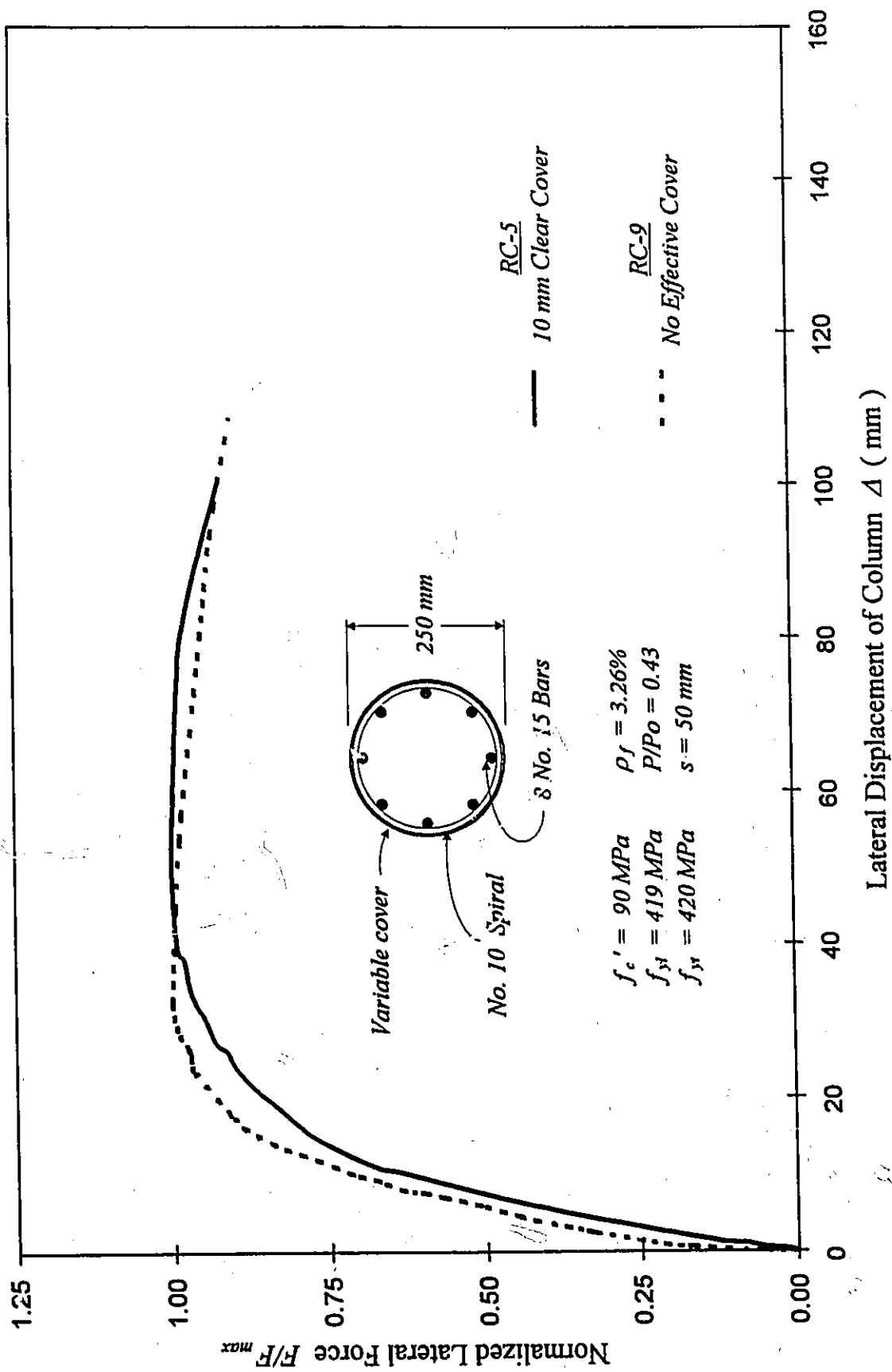


Figure 4.10 Normalized Force - Displacement Envelopes for Columns RC-5 and RC-9
(Effect of Cover Concrete - without $P-\Delta$ effect)

CHAPTER 5

Theoretical Analyses of Columns

5.1 General

The experimentally obtained test data was used to verify a recently developed analytical model for confined high-strength concrete and to explore the possibility of using the analysis procedures commonly used for normal-strength concrete columns in establishing force-displacement relationships of high-strength concrete columns. This was done by comparing the envelopes of force-displacement hysteretic relationships presented in Chapter 3 with those obtained analytically. The results, as well as the analysis procedure, are presented and discussed in this chapter.

5.2 Analytical Calculation of Force-Displacement Relationships

Theoretical force-displacement relationships were calculated for each of the nine columns tested. Well established analysis techniques and a recently developed material model were used to establish the theoretical relationships. The analyses were conducted by using a computer software that incorporated stress-strain models for concrete and steel, as well as an algorithm for progression of hinging in column critical regions. The analysis procedure employed consisted of sectional and member analyses, including the effects of flexure and anchorage slip, but ignoring shear deformations. A detailed discussion of these procedures is presented in the following sections.

5.2.1 Sectional Analysis

The first step in column analysis consisted of a plane sectional analysis. Plane sections were assumed to remain plane before and after bending, resulting in a linear distribution of strains across the depth of the section. Stress-strain relationships of concrete and steel were used to describe the distribution of stresses corresponding to the strains. A stress-strain model for confined high-strength concrete, recently proposed by Razvi and Saatcioglu [1995], was utilized to define the compression concrete. Experimentally obtained stress-strain relationships with strain hardening were used for reinforcing steel. The internal stresses and forces obtained from sectional analysis were used to calculate bending moments and curvatures at different strain profiles. This resulted in a moment-curvature relationship for column sections.

The confinement model for high-strength concrete, adopted in the sectional analysis, is an extension of the model originally developed by Saatcioglu and Razvi [1992] for normal-strength concrete. The model is based on equivalent uniform pressures generated by different types of confinement reinforcement and the resulting improvement in strength and ductility of confined concrete. The parameters of the model were established after reviewing an extensive amount of data from previous column tests. The model is applicable to columns of various shapes, incorporating the effects resulting from different amounts, grades, and spacings of transverse reinforcement. For the case of the circular columns analyzed herein, the lateral pressure caused by closely spaced spiral confinement is considered to be uniform around the core perimeter, as illustrated in Figure 5.1 (a). The actual lateral pressure is then calculated from hoop tension of spiral reinforcement, as shown in Figure 5.1 (b). The stress-strain model is illustrated in Figure 5.2.

5.2.2 Member Analysis

Moment-curvature relationships obtained from sectional analyses were used to compute deformations of columns as members. Two separate components of column deformation were computed. These were deformations caused by flexure and anchorage slip. Tests of columns under inelastic load reversals indicate that flexure dominant columns experience two components of deformations, i.e., flexure and anchorage slip [Saatcioglu and Ozcebe, 1989]. Flexural

deformations reflect bending of the member between the ends, and anchorage slip reflects deformations that take place due to the extension and/or slippage of main column reinforcement within the adjoining member. Deformations due to anchorage slip can be significant if the axial load is low. The columns tested in this research program were subjected high axial compression, with the exception of two that were loaded with relatively lower levels of axial load. Therefore, the effects of anchorage slip were more pronounced on those columns with lower axial compression.

5.2.2.1 Flexural Deformations

Flexural deformations were computed by first establishing the curvature distribution along the height of a column. While the curvature distribution within the ascending branch of moment-curvature relationship can be directly established from this relationship, the post peak behaviour requires consideration of plastic hinging and progression of hinging. Reinforced-concrete members that experience post peak deformations develop increased curvatures within the plastic hinge, while the moment resistance continue dropping. The increase in curvatures during strength degradation, and subsequent increase in moment resistance due to concrete confinement, as well as associated progression of hinging requires an algorithm to model this behaviour. Razvi and Saatcioglu [1994] developed an algorithm that has been verified extensively against tests of normal-strength concrete columns. This algorithm was adopted in the computer software used for column analysis, and is illustrated in Figure 5.3.

Member rotations were obtained by integrating the curvatures along the height. The first moment of the area under the curvature distribution gave column lateral displacements due to flexure. The procedure adopted for the computation of flexural displacement is based on accepted principles of strain compatibility and equilibrium, and is assumed to be applicable to high-strength concrete, with proper material models.

5.2.2.2 Deformations due to Anchorage Slip

Deformations due to anchorage slip consists of two components; i) extension and ii) slippage of reinforcement. Well anchored bars do not develop the slip deformation. However, the extension

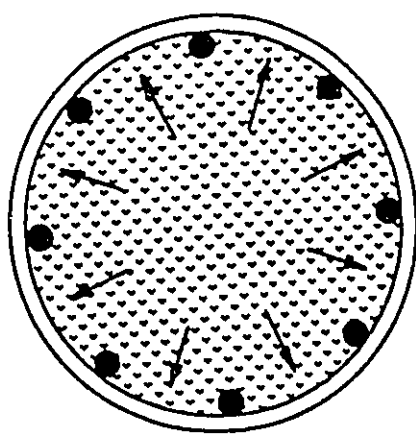
of reinforcement within the adjoining member is unavoidable if the critical section takes place at the end of the column. Penetration of yielding into the adjacent member results in inelastic bar extension, which, when accumulated can be significant at the interface. As stated earlier, however, the effect of bar extension can be negligibly small if the member is under high axial compression. Research data indicate that member deformations due to the extension of reinforcement become significant if the reinforcement at the interface reach strain hardening [Alsiwat and Saatcioglu, 1992]. The same behaviour was often observed during the experimental part of this research, and is very typical of reinforced concrete members subjected to flexure. Extension of embedded reinforcement is referred to here as "anchorage slip."

Deformations due to anchorage slip were computed on the basis of the analytical model developed by Alsiwat and Saatcioglu [1992]. The model is based on the construction of stress and strain diagrams along the embedment length of reinforcement. This is done by first conducting the sectional analysis of the critical section to obtain the stress and strain in reinforcement at that location, and using elastic and inelastic bond stresses to compute the length of each region within the embedment length. Figure 5.4 illustrates the strains and stresses along the length of reinforcement within the adjoining member. This model was however, developed for normal-strength concrete, and hence its applicability to high-strength concrete may be questioned. The bond stress between the steel and high-strength concrete can be different than that used for normal-strength concretes. However, this difference is believed to be small for the application considered in this research program.

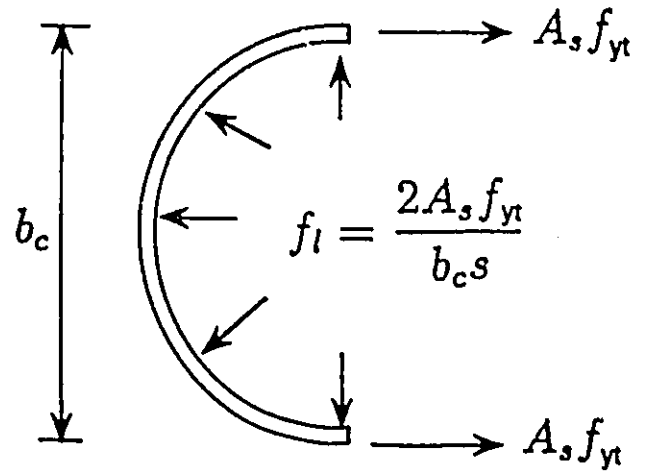
5.3 Comparison of Results with Experimental Data

A computer program, developed on the basis of the procedure described above was used to generate analytical force-displacement relationships for the high-strength concrete columns tested in this investigation. The analytically obtained force-displacement relationships were then compared with the force-displacement envelopes obtained from experimental data. The analytical curves were established for monotonically increasing lateral loading, while also accompanied by constant axial compression. The experimental curves on the other hand were obtained as envelopes of the hysteretic force-displacement relationships. It was therefore assumed that the

monotonic curves were representative of these envelope curves. This was shown by Fafitis and Shah [1985] and Saatcioglu, Salamat, and Razvi [1995] to be the case for normal strength concrete, and was assumed to be also true for high-strength concrete. The comparisons of the analytical and experimental results, for the nine columns tested, are presented in Figures 5.5 through to 5.14. The results of the comparisons show excellent agreement between the experimental and analytical curves. Furthermore, it may be concluded that the procedures utilized, and the analytical models adopted, are suitable for calculation of inelastic column behaviour, even if the column is made with high-strength concrete.



(a) Uniform Lateral Pressure



(b) Computation of Lateral Pressure from Hoop Tensile Force

Figure 5.1: Lateral Pressure in Circular Columns [Razvi, S., 1995]

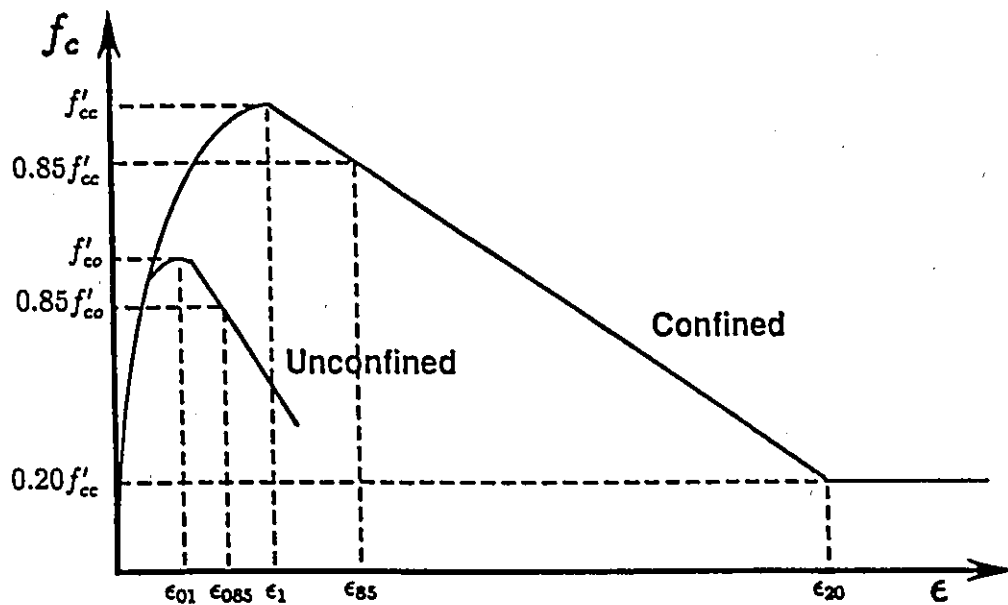


Figure 5.2: Model Stress-Strain Relationship for Confined Concrete [Razvi, S., 1995]

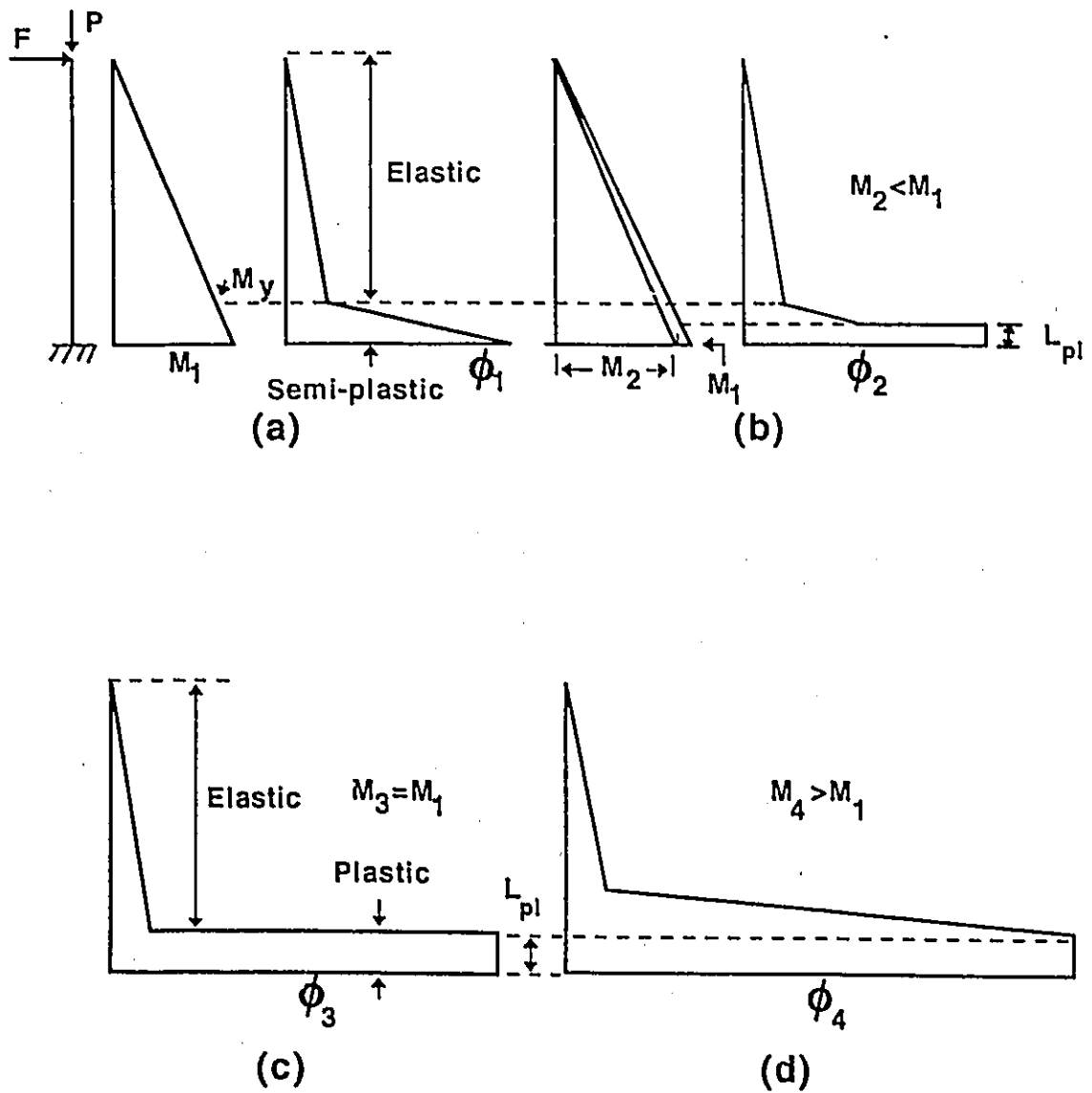


Figure 5.3: Progression of a Plastic Hinge [Razvi, S., 1995]

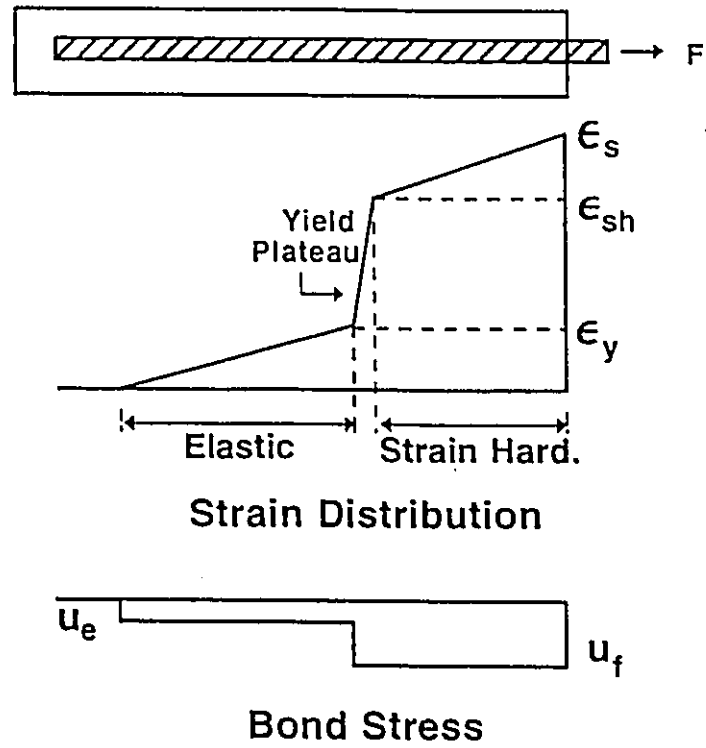


Figure 5.4: Strain and Bond Stress in Anchorage Reinforcement
 [Alsiwat, J. and Saatcioglu, M., 1992]

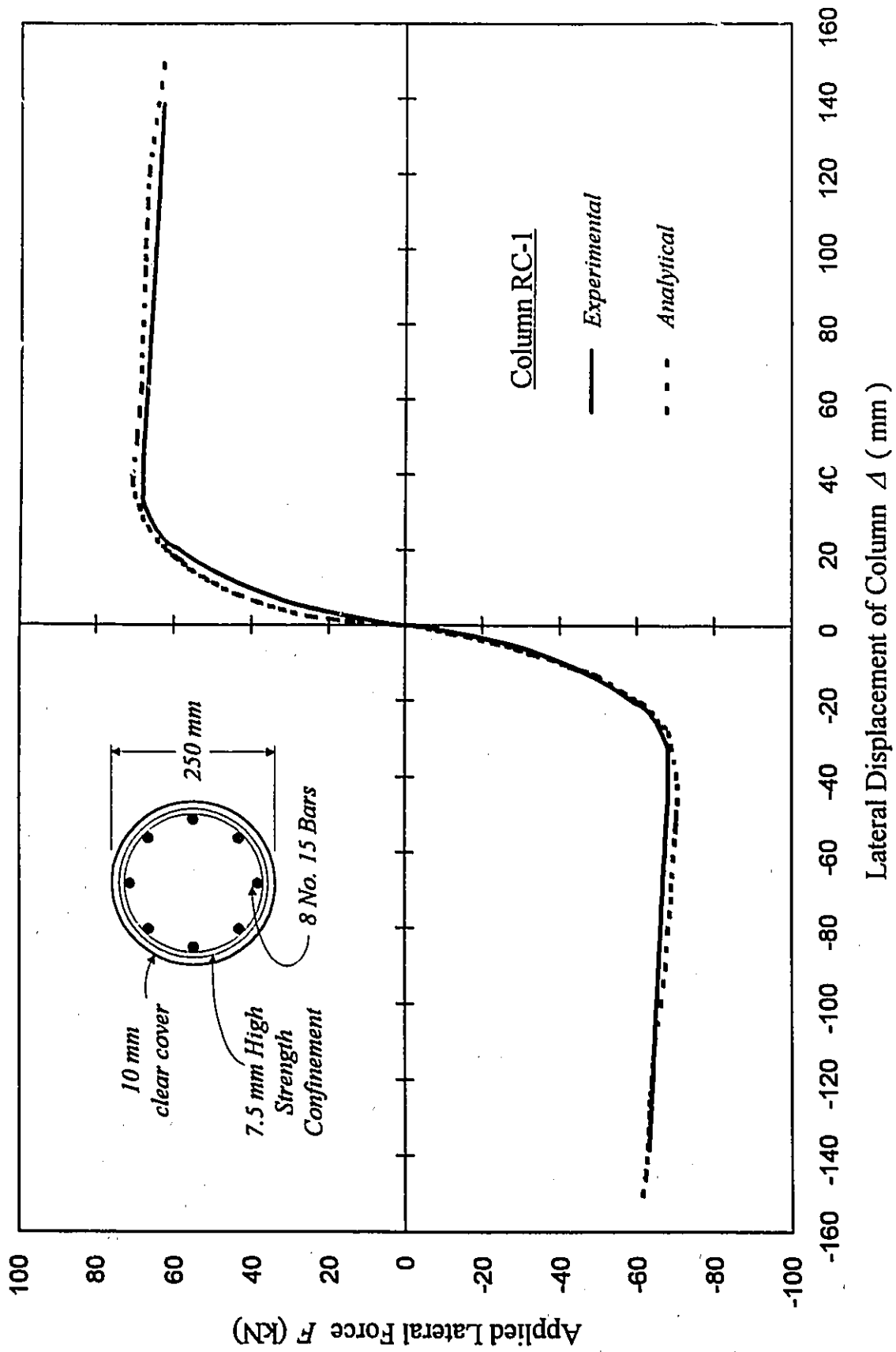


Figure 5.5: Comparison of Force - Displacement Relationships for Columns RC-1
(*Experimental and Analytical - Without P- Δ effect*)

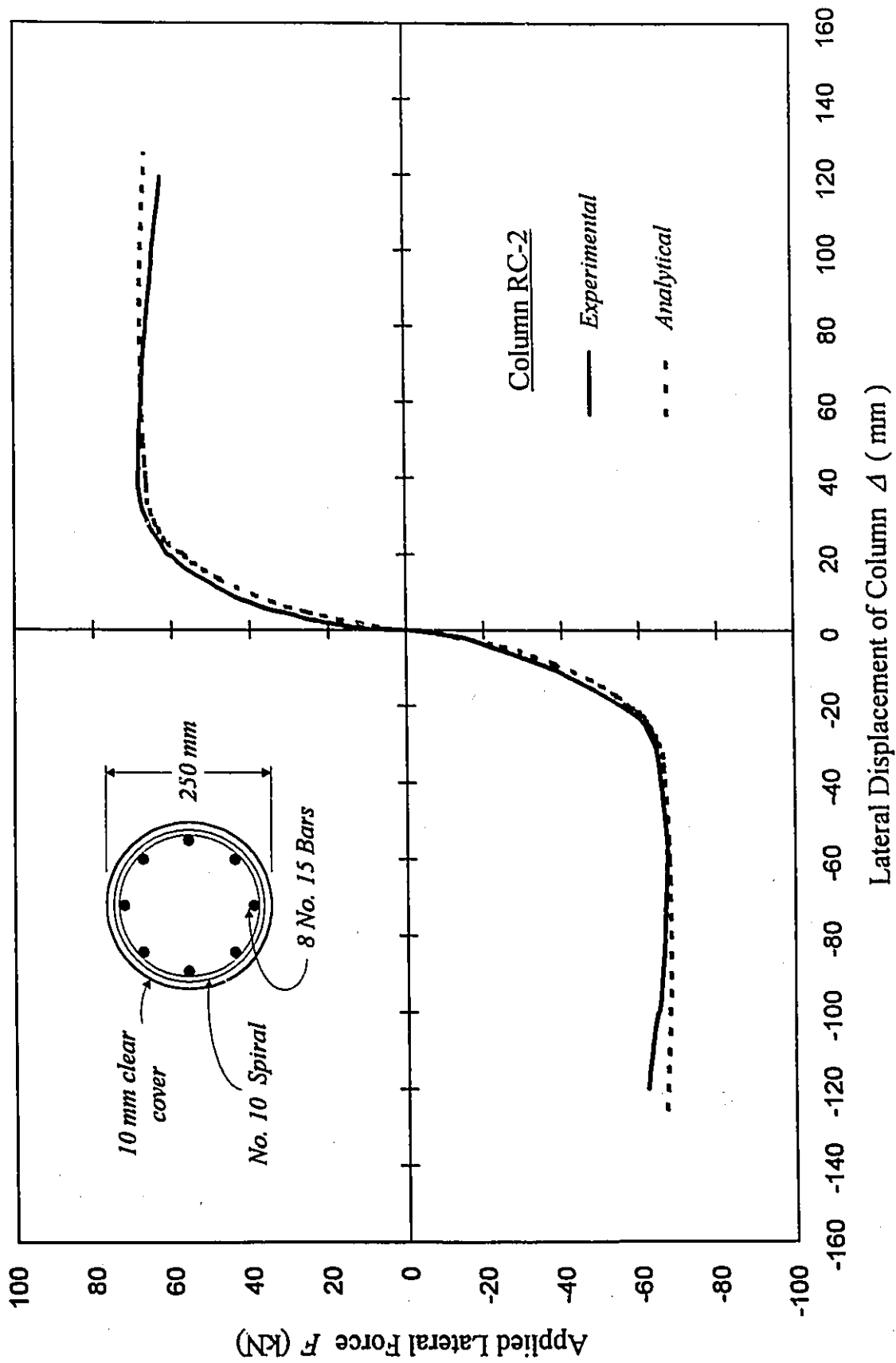


Figure 5.6: Comparison of Force - Displacement Relationships for Columns RC-2
(Experimental and Analytical - Without $P-\Delta$ effect)

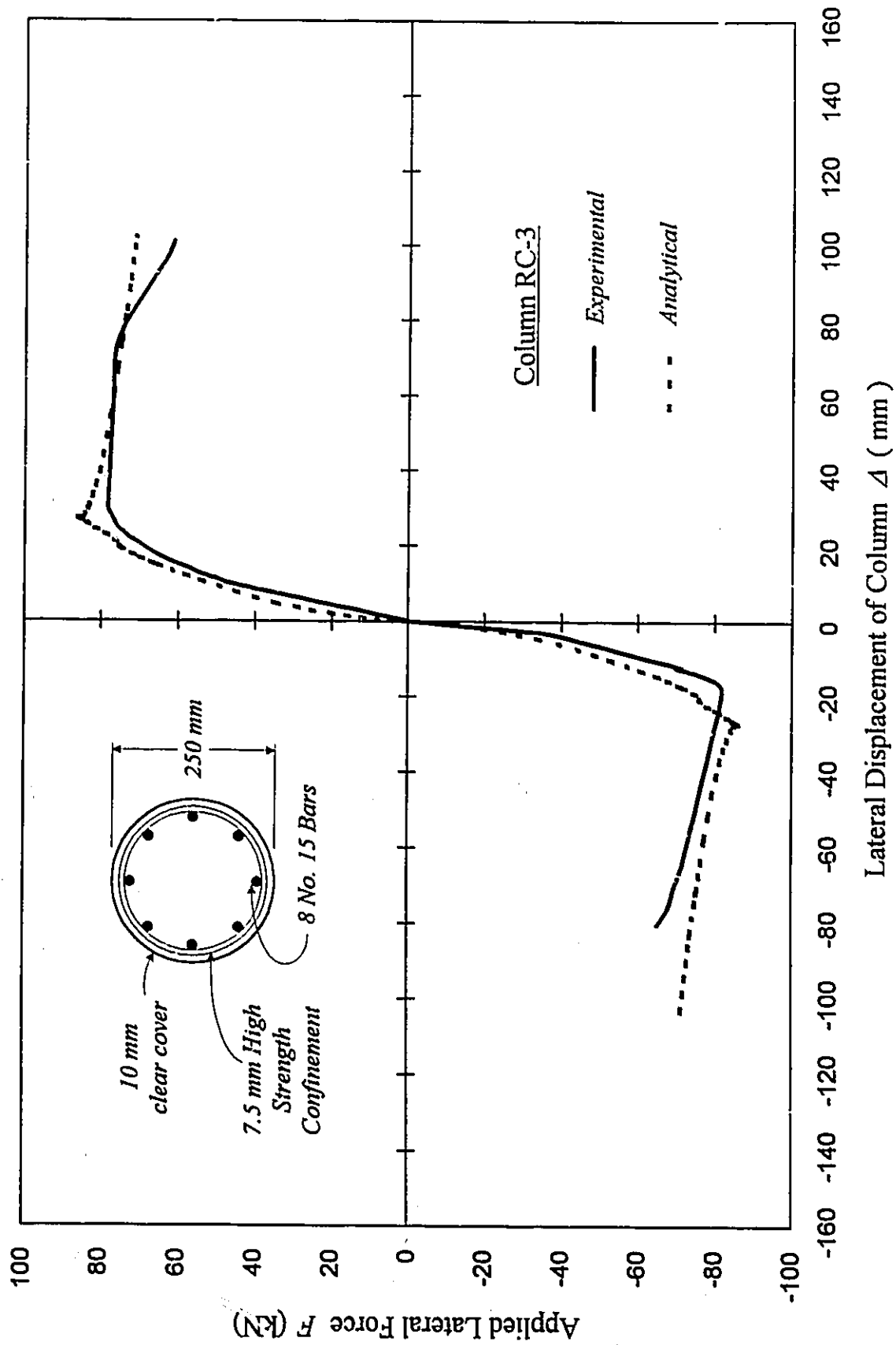


Figure 5.7: Comparison of Force - Displacement Relationships for Columns RC-3
(Experimental and Analytical - Without $P-\Delta$ effect)

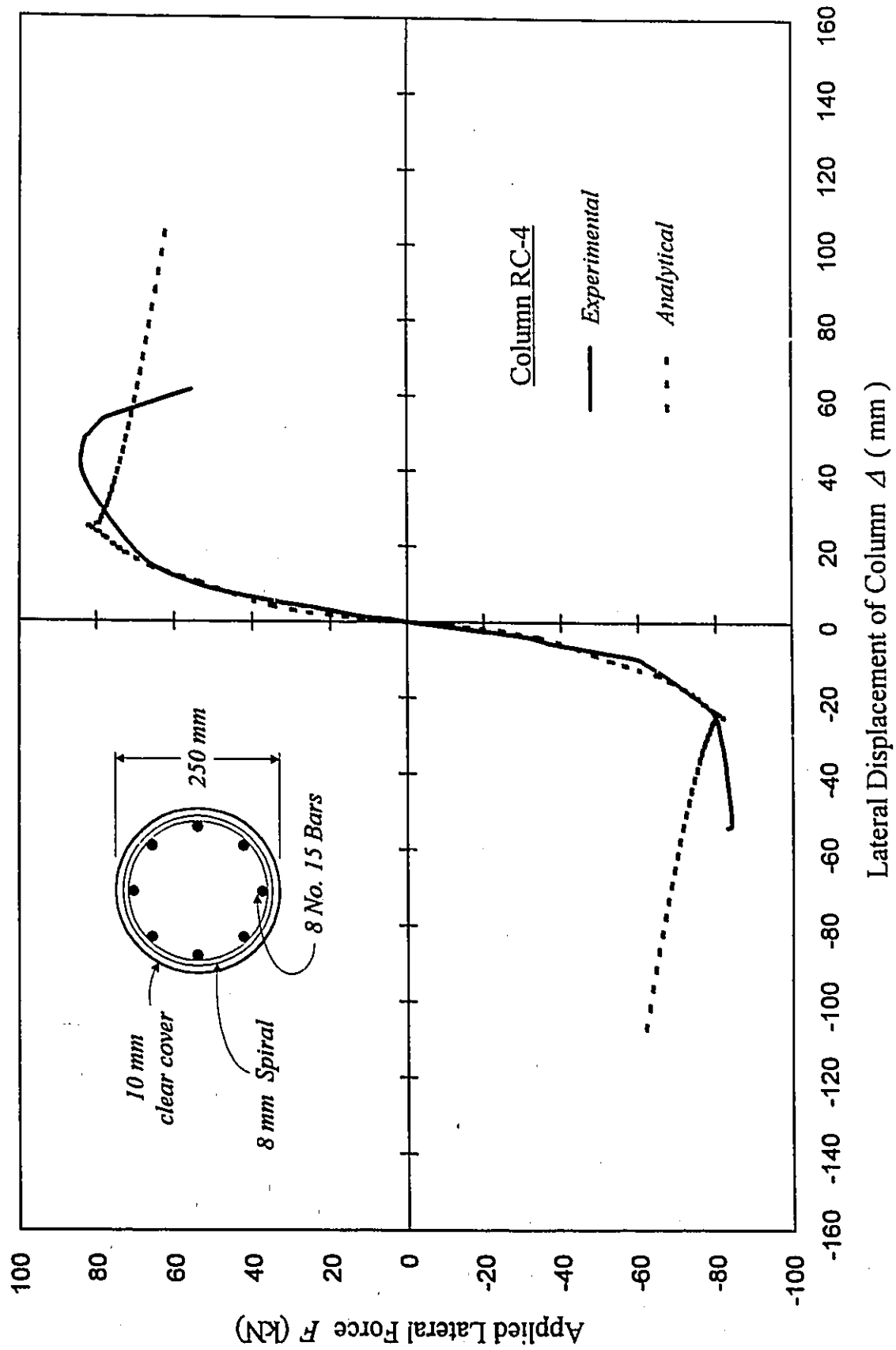


Figure 5.8: Comparison of Force - Displacement Relationships for Columns RC-4
 (Experimental and Analytical - Without $P-\Delta$ effect)

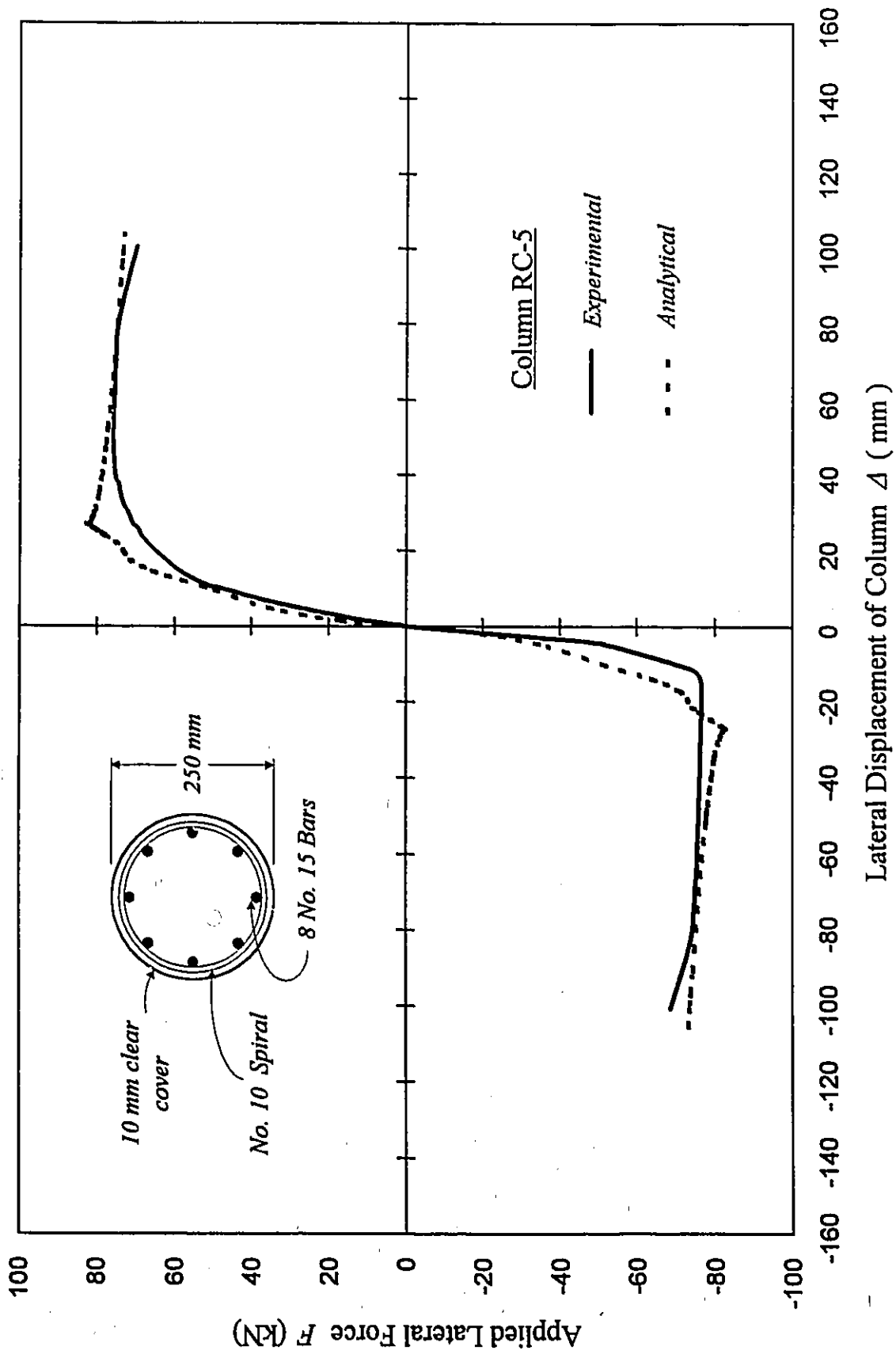


Figure 5.9: Comparison of Force - Displacement Relationships for Columns RC-5
(Experimental and Analytical - Without $P-\Delta$ effect)

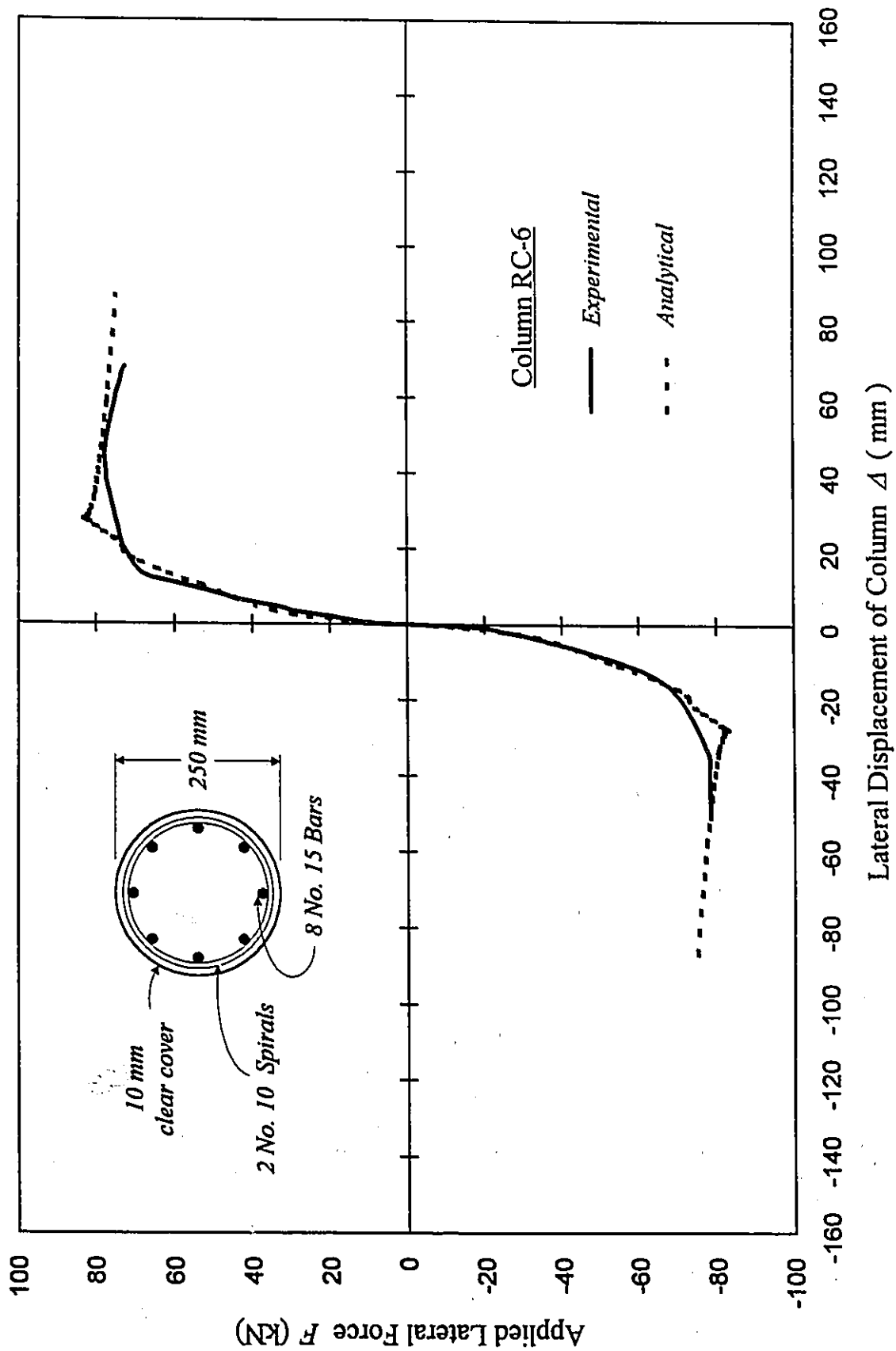


Figure 5.10: Comparison of Force - Displacement Relationships for Columns RC-6
(*Experimental and Analytical - Without $P-\Delta$ effect*)

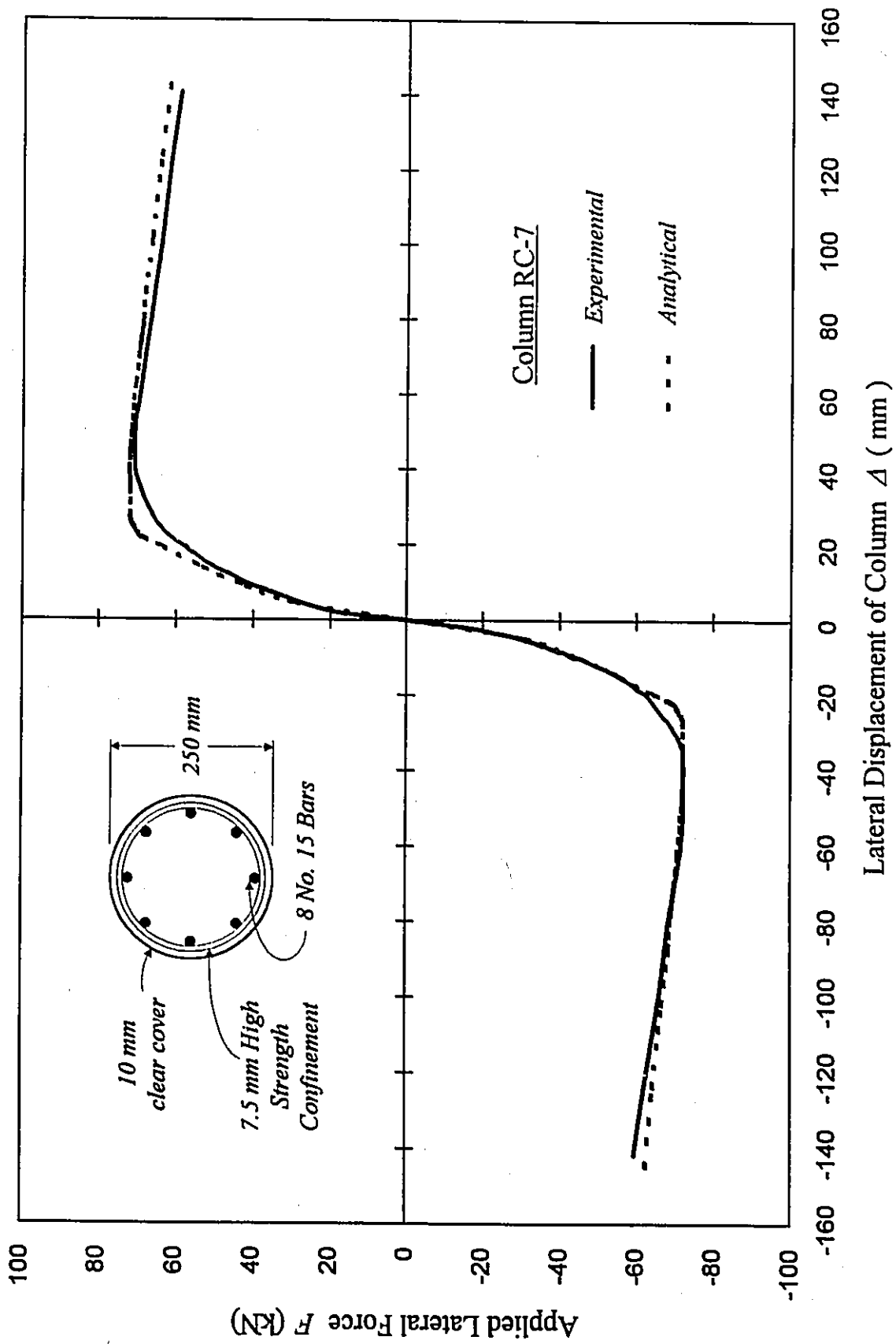


Figure 5.11: Comparison of Force - Displacement Relationships for Columns RC-7
 (Experimental and Analytical - Without $P-\Delta$ effect)

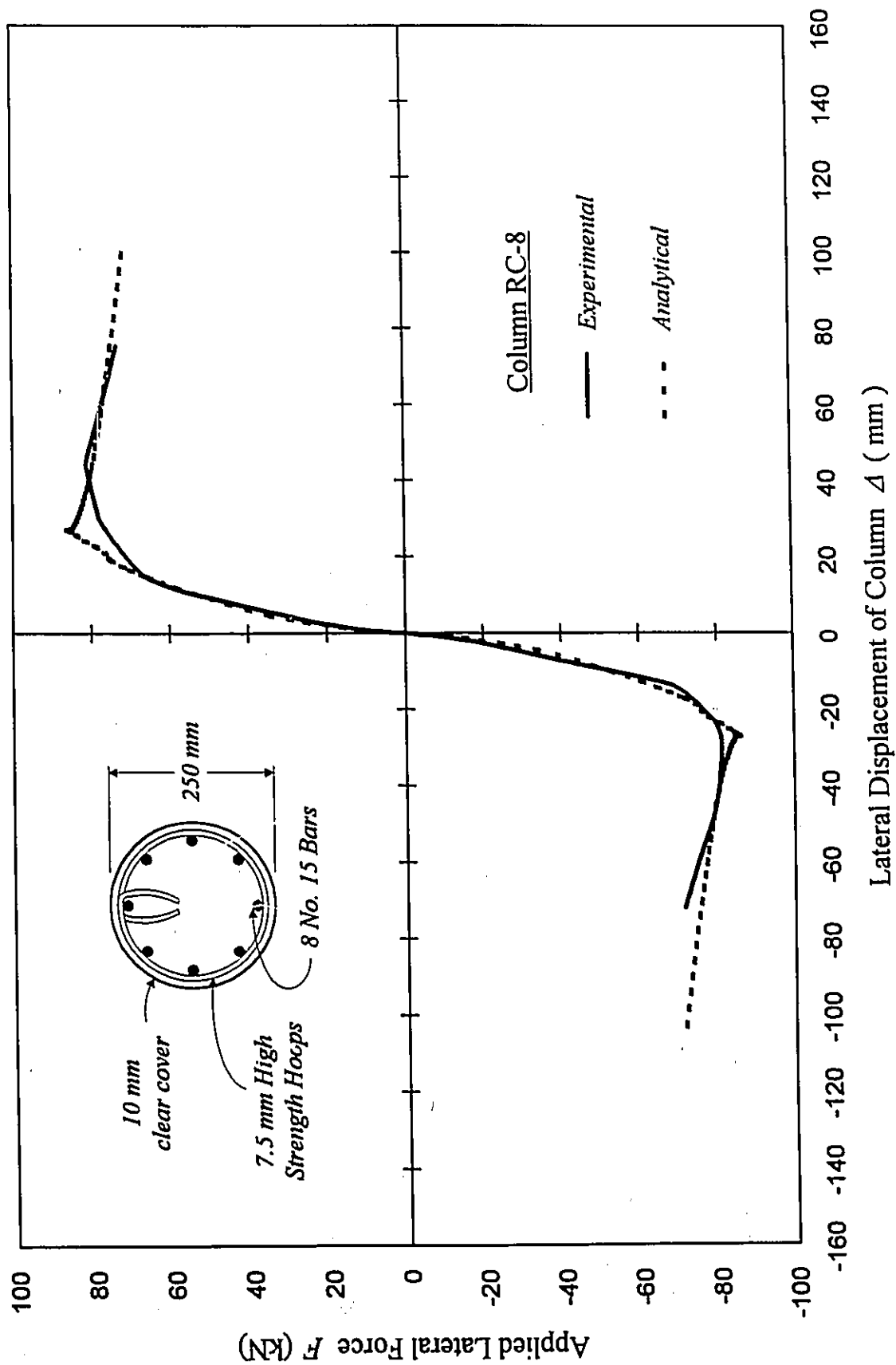


Figure 5.12: Comparison of Force - Displacement Relationships for Columns RC-8
(Experimental and Analytical - Without $P-\Delta$ effect)

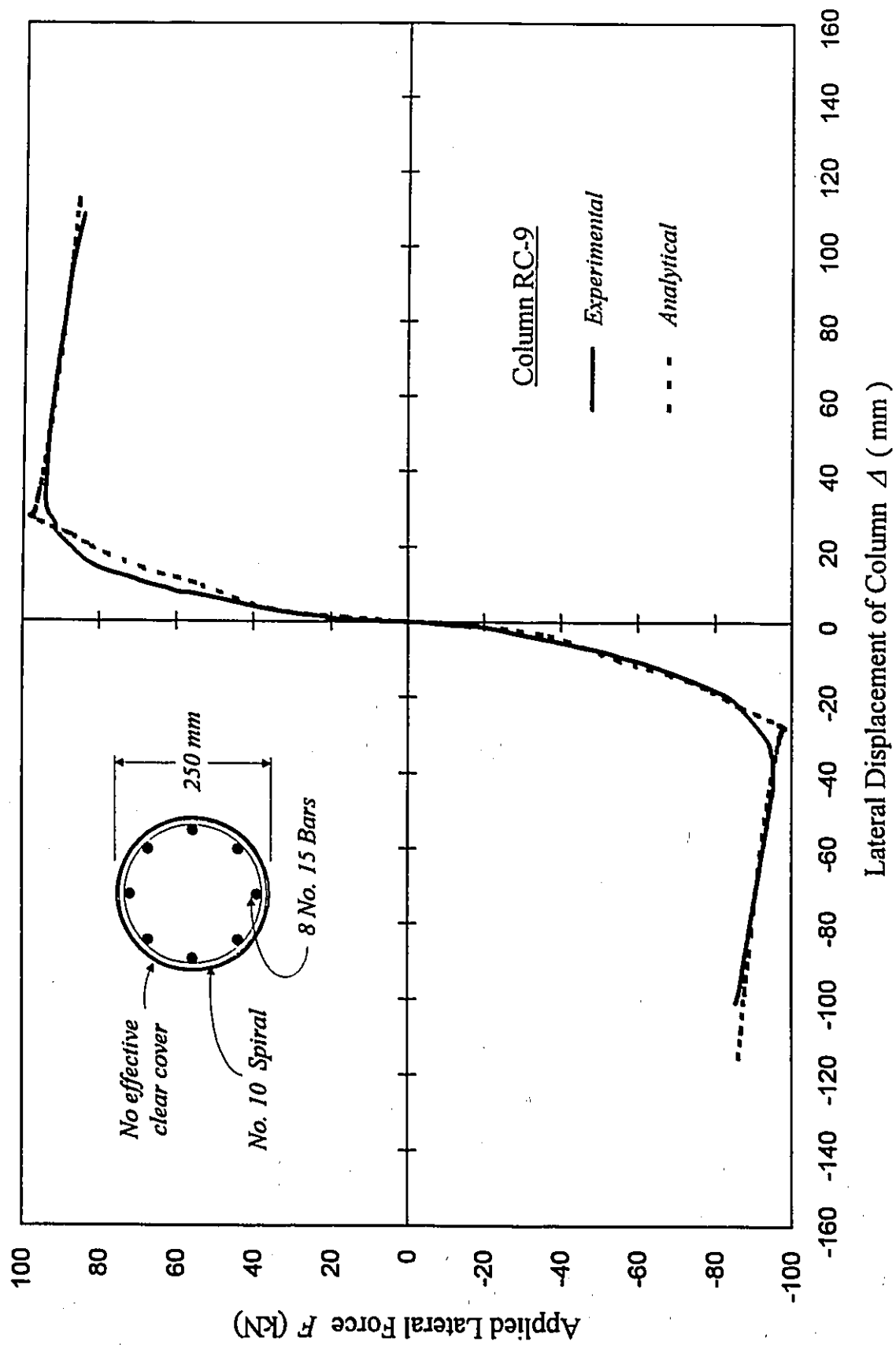


Figure 5.13: Comparison of Force - Displacement Relationships for Columns RC-9
(Experimental and Analytical - Without $P-\Delta$ effect)

CHAPTER 6

Summary and Conclusions

6.1 Summary

An experimental and analytical investigation was conducted to study the strength and ductility of circular high-strength concrete columns. A total of nine full scale columns were tested under combined axial compression and lateral load reversals. The specimens were designed to allow the investigation of various confinement parameters and their effects on column deformability. The parameters included the compressive strength of concrete, the volumetric ratio, yield strength, and spacing of transverse reinforcement, the level of axial load, the type of circular reinforcement, and cover concrete. Strength and deformation characteristics of columns were discussed. Hysteretic force-displacement and moment-rotation relationships were presented. Strain profiles along the longitudinal and transverse reinforcement were also recorded. The effects of test parameters were analyzed by comparing the force-displacement envelopes of companion columns.

Theoretical analyses of columns were carried out to obtain analytical force-displacement relationships. Moment-curvature analysis was conducted using a plane section analysis and a confined concrete model. Inelastic rotations and column displacements were computed from the distribution of curvatures along the height. Member response was computed by modelling the plastic hinge region and following an algorithm for the progression of hinging. The analysis provided inelastic displacements caused by flexure and anchorage slip. Force-displacement relationships obtained from the analytical computations were then compared with the envelopes of those recorded experimentally.

6.2 Conclusions

The following conclusions can be reached from the experimental and analytical research reported in this thesis:

- Deformability of reinforced concrete columns decreases with increasing concrete strength. The specimens cast from 90 MPa concrete showed considerably lower drift capacities than those made of 65 MPa concrete. Also, the crushing of concrete was more pronounced at lower deformations in higher strength concrete columns.
- It is possible to obtain ductile behaviour from high-strength concrete columns subjected to lateral deformation reversals, provided that they are suitably confined. The amount of steel needed for this purpose is generally higher than that required for normal-strength concrete columns. However, the additional steel requirement can be partially offset by the use higher grade confinement steel.
- High-strength confinement steel of up to 1000 MPa can be fully utilized in high-strength concrete columns. Strain gauge data indicated yielding of high strength reinforcement at approximately the peak resistance. This implies that the full benefits of high strength steel were utilized in the test columns. However, this may or may not be the case for reinforcement with higher than 1000 MPa grade. Columns with the same volumetric ratio but higher grade of reinforcement showed improved deformability as compared to those with lower grade steel.
- The volumetric ratio of transverse reinforcement is one of the most important parameters affecting deformability of high-strength concrete columns. The experimental investigation showed that Column RC-5 with a ρ_t of 3.67% exhibited no strength degradation up to a 4.6% drift. Column RC-4 with a ρ_t of 1.81% experienced rapid strength decay after the peak load and failed in a brittle manner at a drift of approximately 3%. Furthermore, Column RC-4 with low volumetric ratio failed due to the instability of longitudinal reinforcement after the confinement steel ruptured. Since the strength and volumetric ratio

of transverse steel are closely related in developing confinement pressure, it is best to consider them together in design. The test data indicated that $\rho_t f_y / f_c'$ can be used as a design parameter for confinement of circular high-strength concrete columns. Columns confined to have $\rho_t f_y / f_c'$ ratio of at least 15 showed good ductility characteristics, developing at least 6% lateral drift.

- The primary functions of proper confinement can be fulfilled with reduced spacing of transverse reinforcement. At a constant volumetric ratio, Column RC-5, with a 50 mm spacing of lateral steel developed a drift of 6%. In contrast, Column RC-6, with a spacing of 100 mm, began to fail at 4% drift, due to buckling of longitudinal reinforcement. Close pitch of spiral is one of the best ways of ensuring ductile behaviour in circular HSC columns.
- The effect of axial compression is to decrease column deformability. Although high axial compression increases column strength, it decreases ductility and increases the P- Δ effect. The P- Δ effect accelerates the strength decay under high axial compression. Therefore, the effects of secondary deformations must be considered in design.
- Spirally confined columns and the column confined with individual hoops showed similar behaviour. A marginally better performance was observed in the spirally reinforced columns at high deformation levels. This could be attributed to the stability of longitudinal reinforcement when supported by spirals. Although opening of the hoops was not observed during testing, the curved hoop segments, located between longitudinal reinforcement, was observed to straighten out, giving additional space to longitudinal bars to move and buckle at the high deformation range. It may be concluded that continuity of circular transverse reinforcement appears to have a positive influence in stabilizing longitudinal bars and improving confinement at high inelastic deformations.
- For the thickness range considered in this investigation, it appears that the cover concrete did not affect the deformability of HSC columns. However, this conclusion should not be generalized until additional data with different cover thicknesses are obtained.

- The force-displacement relationships obtained from inelastic flexural analysis correlate well with those obtained experimentally. This implies that the analytical methods commonly used for normal-strength concrete elements can be applied to high-strength concrete members, given that proper material and inelastic behaviour models are utilized. The recently proposed models, adopted in the analytical part of this thesis, produced good results.
- Experimental data obtained from longitudinal strain gauges indicates that, under high axial load levels, no penetration of yield into the footing occurred. Therefore, the rotations due to anchorage slip appeared to be negligible. However, two of the columns were tested under a relatively lower axial force. These columns developed some deformations due to anchorage slip, approximately equal to 13% of the total. The test data further indicated that reinforcement yield penetration and hence anchorage slip rotations were much smaller in columns confined with high-strength steel than in columns with lower strength transverse reinforcement.

6.3 Recommendations for Future Research

The following recommendations are made for future research:

- Tests of HSC columns confined with different grades of high yield-strength reinforcement are necessary to establish an upper strength limit for use in design.
- To better illustrate the effect of cover concrete on the ductility of HSC columns under simulated seismic loading, tests of columns with cover proportions similar to those used in practice, should be conducted.
- Further study is suggested to address the issue of transverse reinforcement continuity on the deformability of HSC columns.
- Experimental research is recommended to establish bond-slip characteristics of

reinforcement in high-strength concrete to establish anchorage slip deformations in this material.

Appendix A

Additional Test Data

The additional data collected during the column tests is summarized in the following section. The data is presented in graphical format and includes the hysteretic force-displacement relationships for each specimen, recorded below the loading beam as well as immediately above the potential hinging region. The data also includes the distribution of strains in the column reinforcement. The graphs are organised sequentially, where for each column the first and second figures contain the force-displacement relationships and the third and fourth contain the strain distributions in longitudinal and transverse steel, respectively.

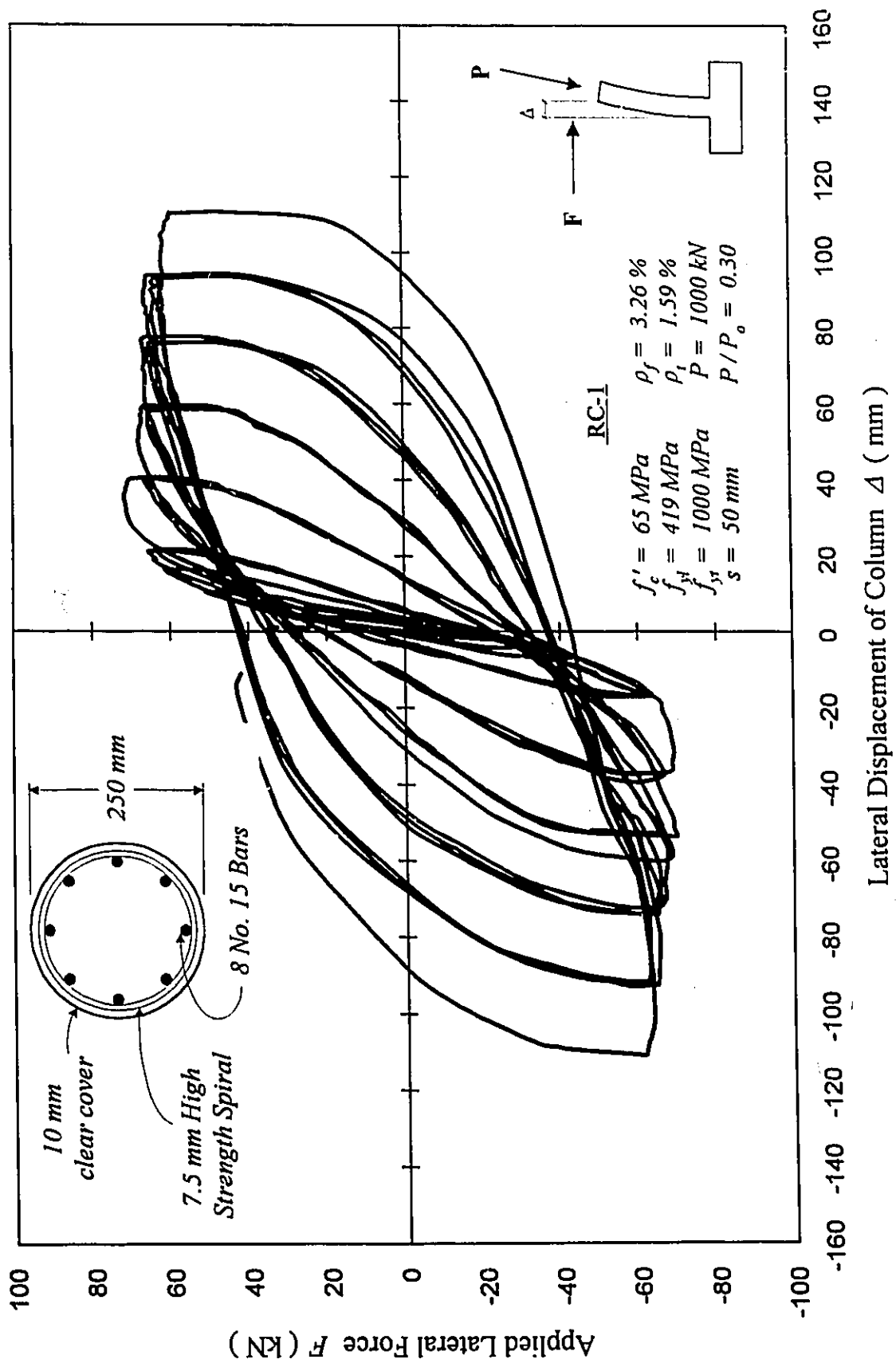


Figure A.1: Force - Displacement Relationship for Column RC-1
 (At Tip of Column - without P - Δ effect)

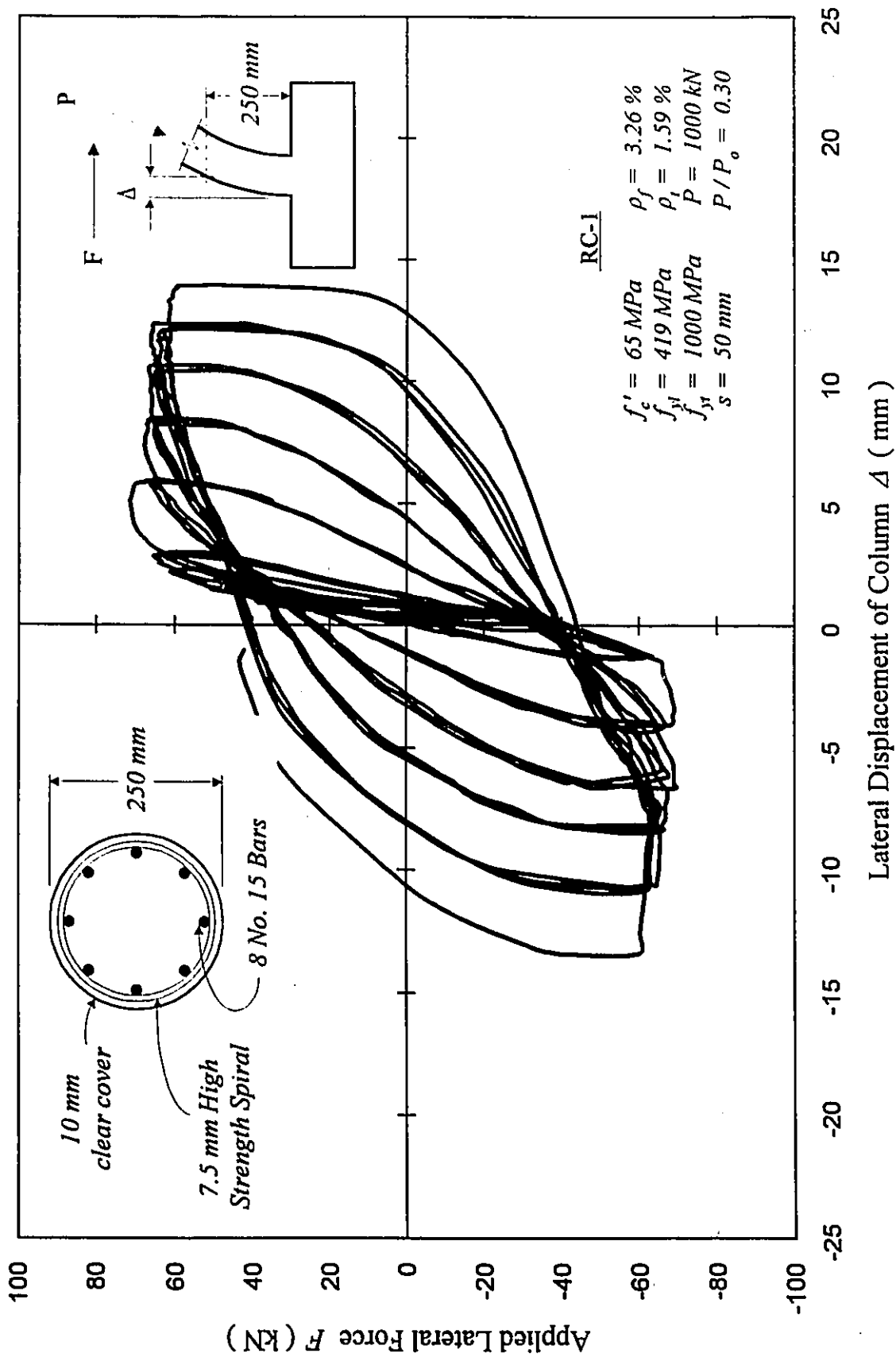


Figure A.2: Force - Displacement Relationship for Column RC-1
(At Top of Hinge Region - without P - Δ effect)

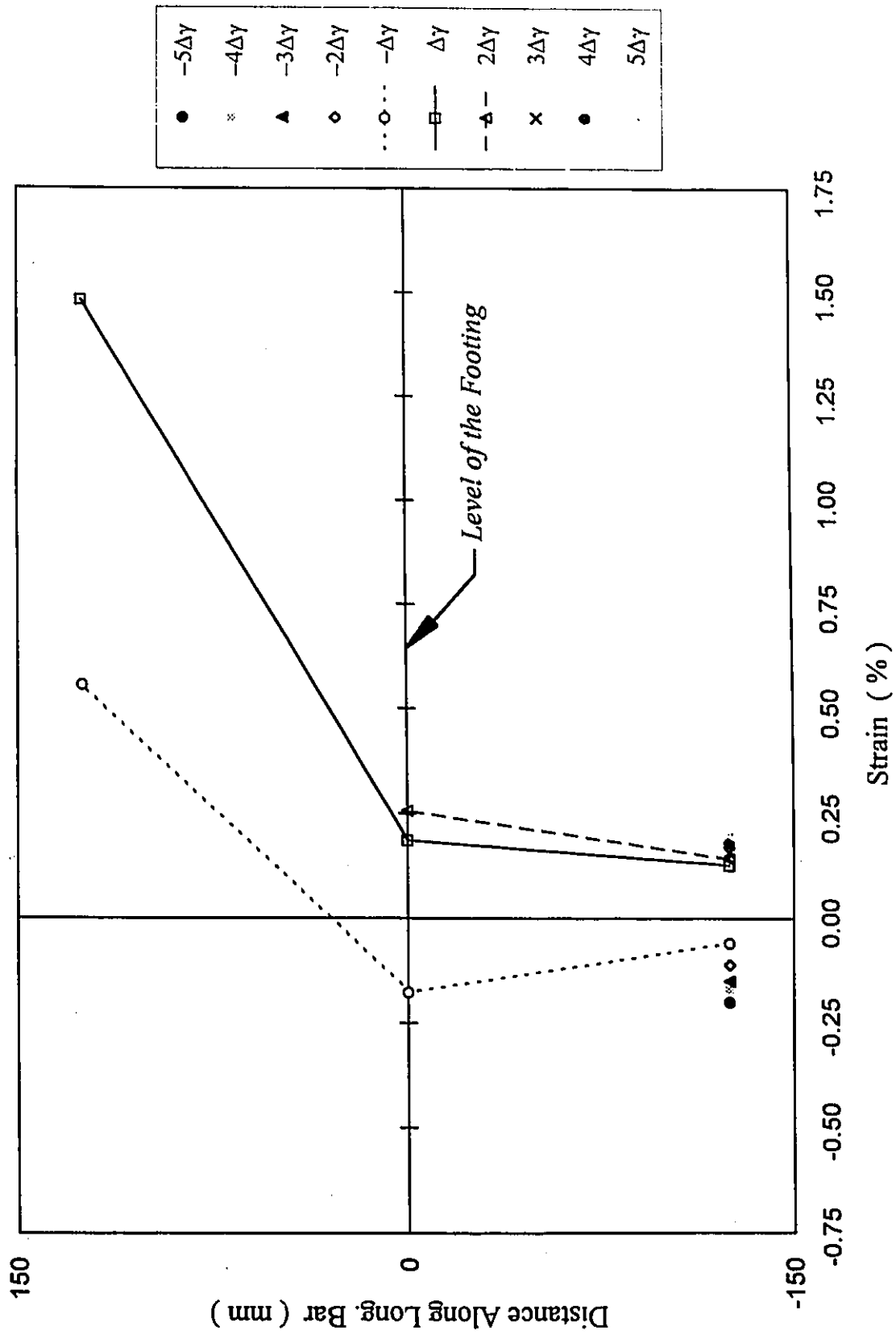


Figure A.3: Strain Distribution in Longitudinal Reinforcement for Different Δy Values
Column RC-1

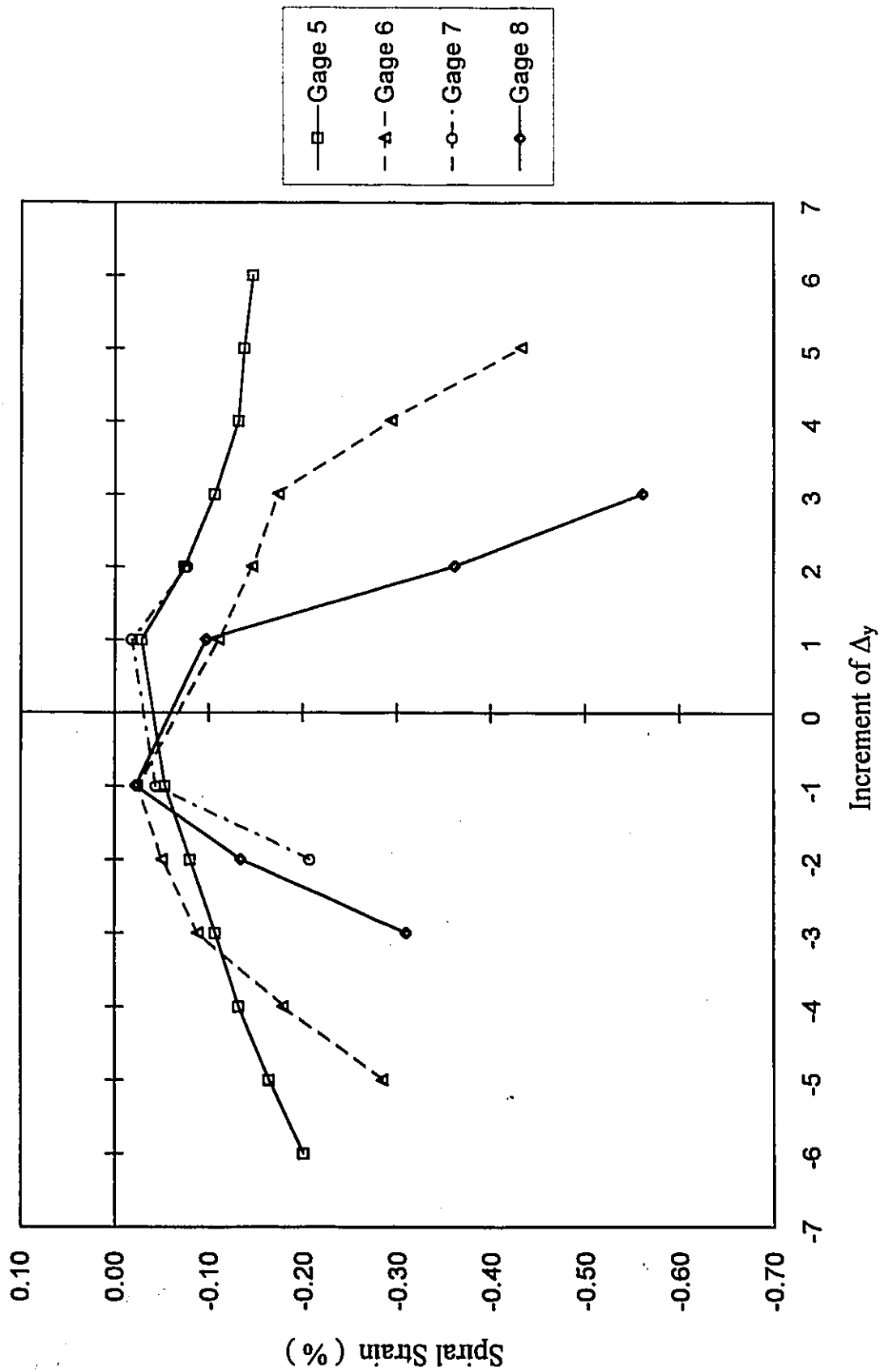


Figure A.4: Strain Distribution in Transverse Reinforcement for Different Δy Increments
Column RC-1

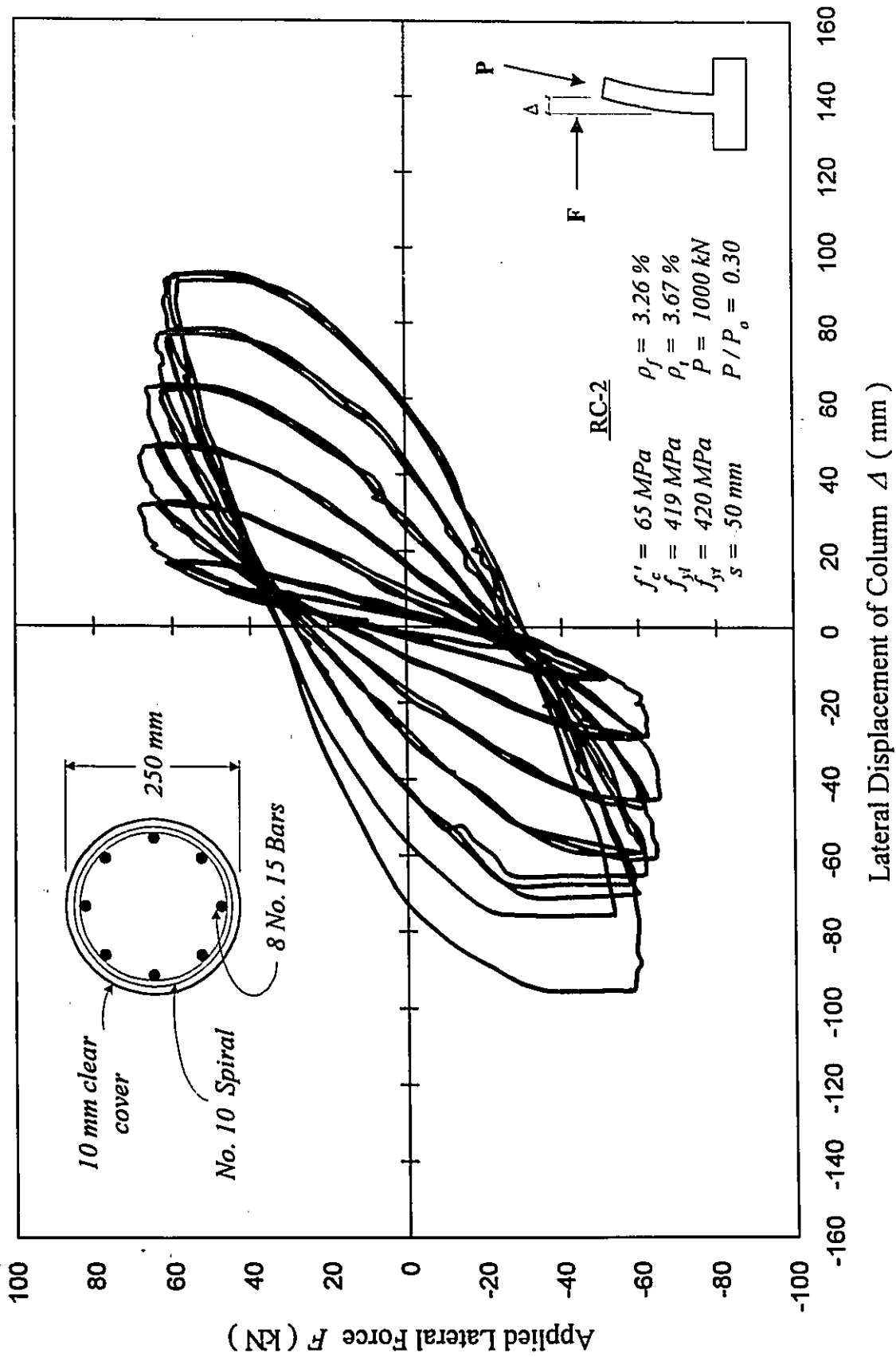


Figure A.5: Force - Displacement Relationship for Column RC-2
 (At Tip of Column - without $P-\Delta$ effect)

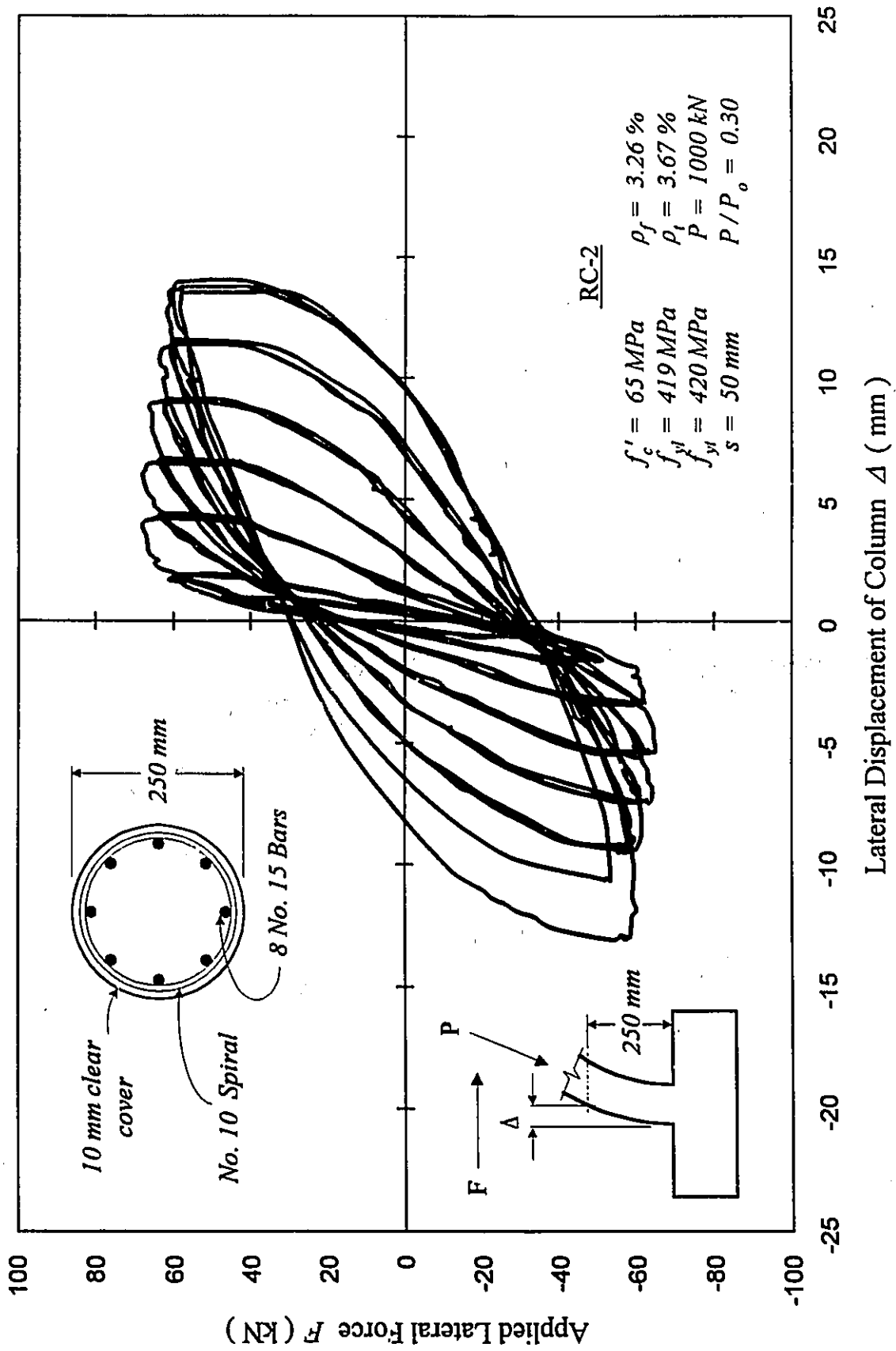


Figure A.6: Force - Displacement Relationship for Column RC-2
(At Top of Hinge Region - without P - Δ effect)

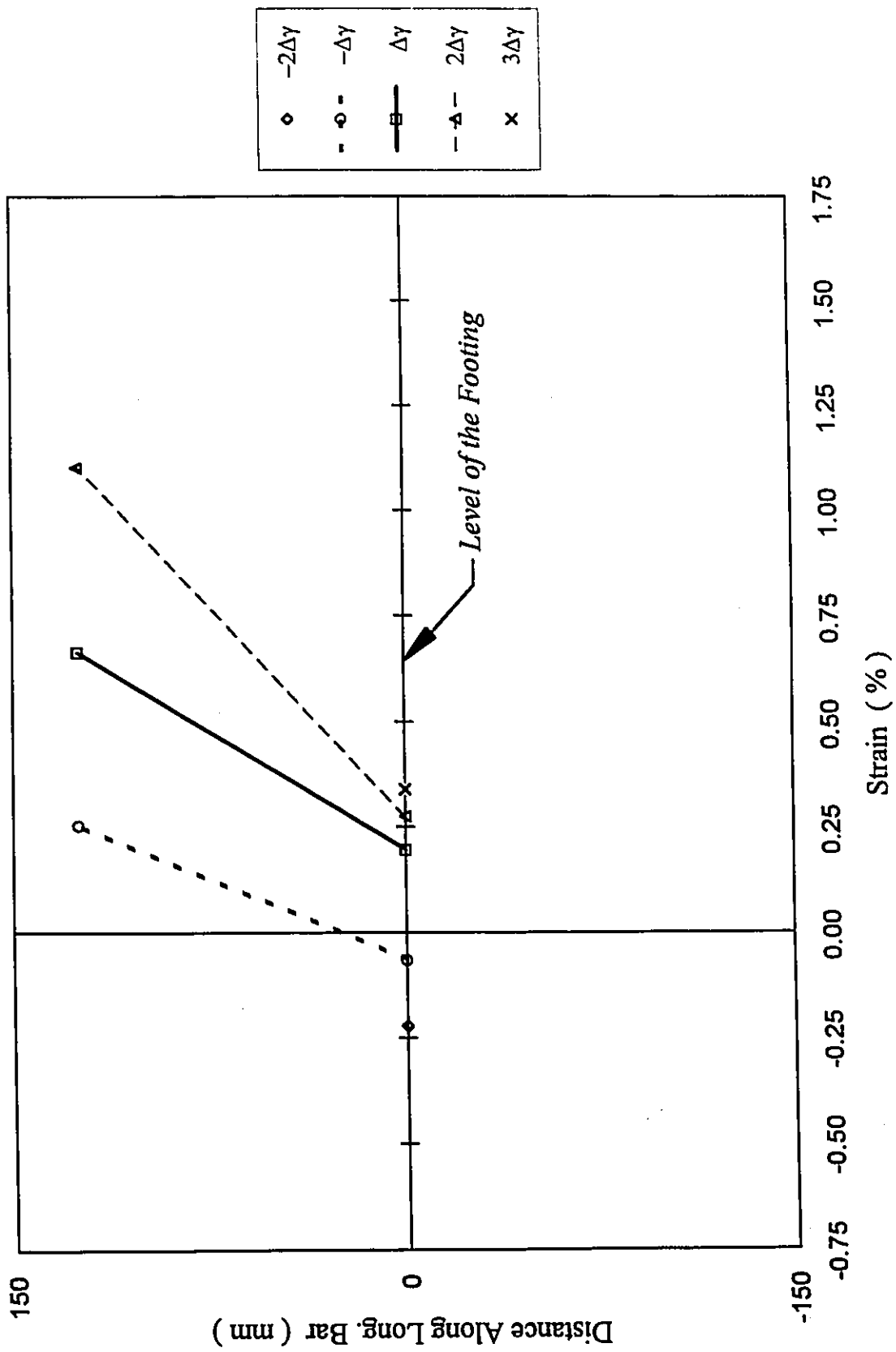


Figure A.7: Strain Distribution in Longitudinal Reinforcement for Different Δy Values
Column RC-2

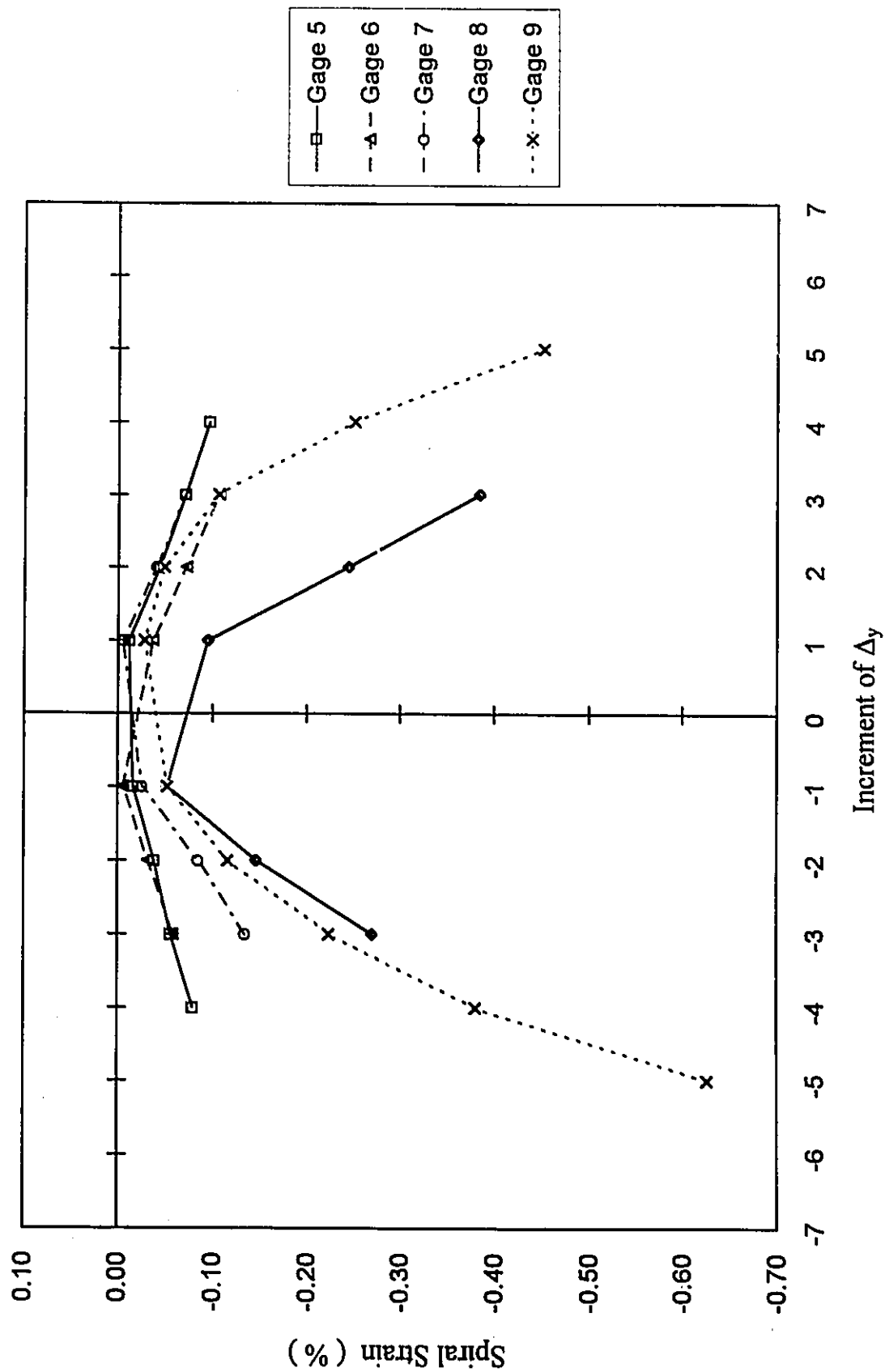


Figure A.8: Strain Distribution in Transverse Reinforcement for Different Δy Increments
Column RC-2

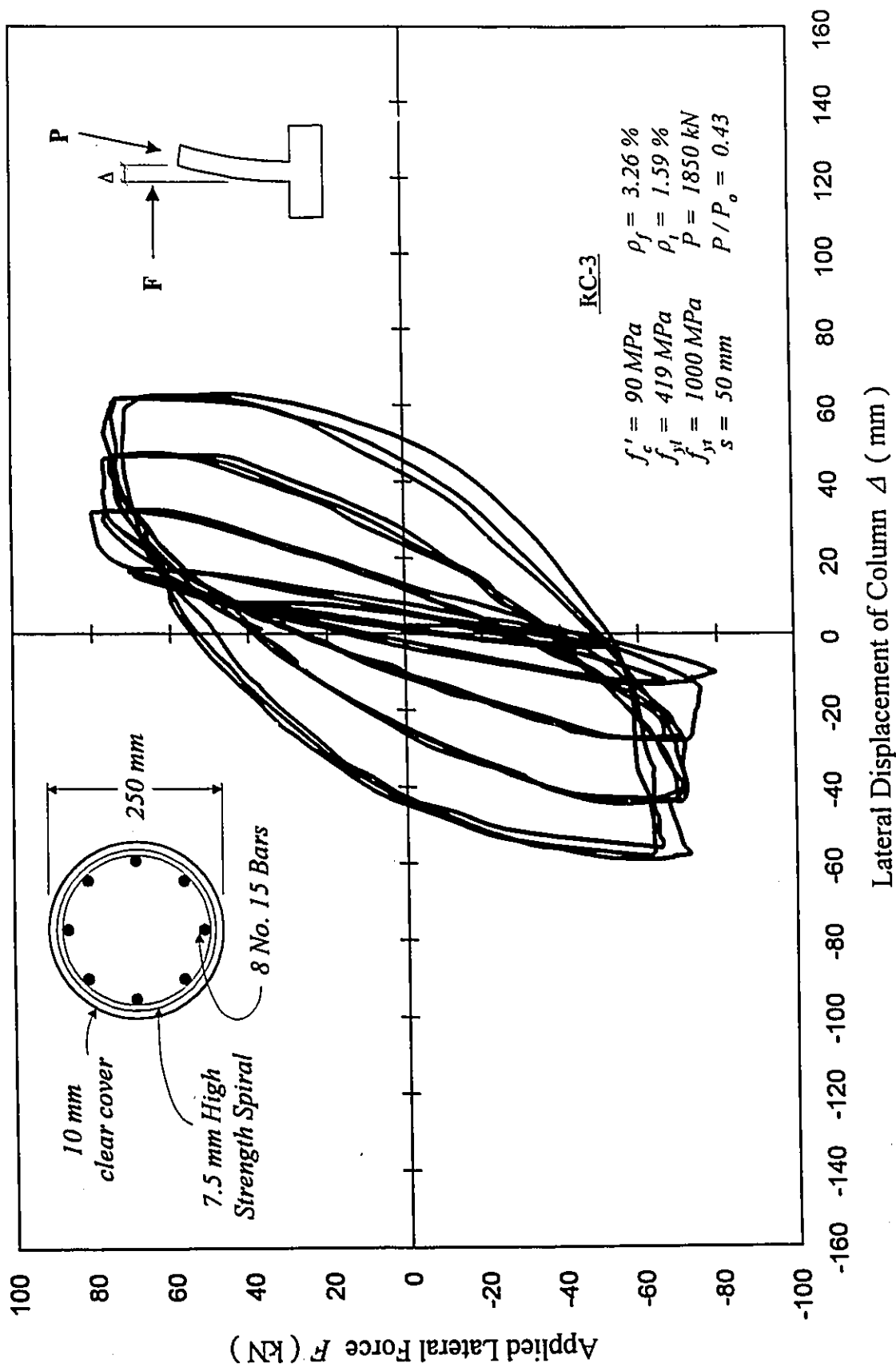


Figure A.9: Force - Displacement Relationship for Column RC-3
(At Tip of Column - without $P-\Delta$ effect)

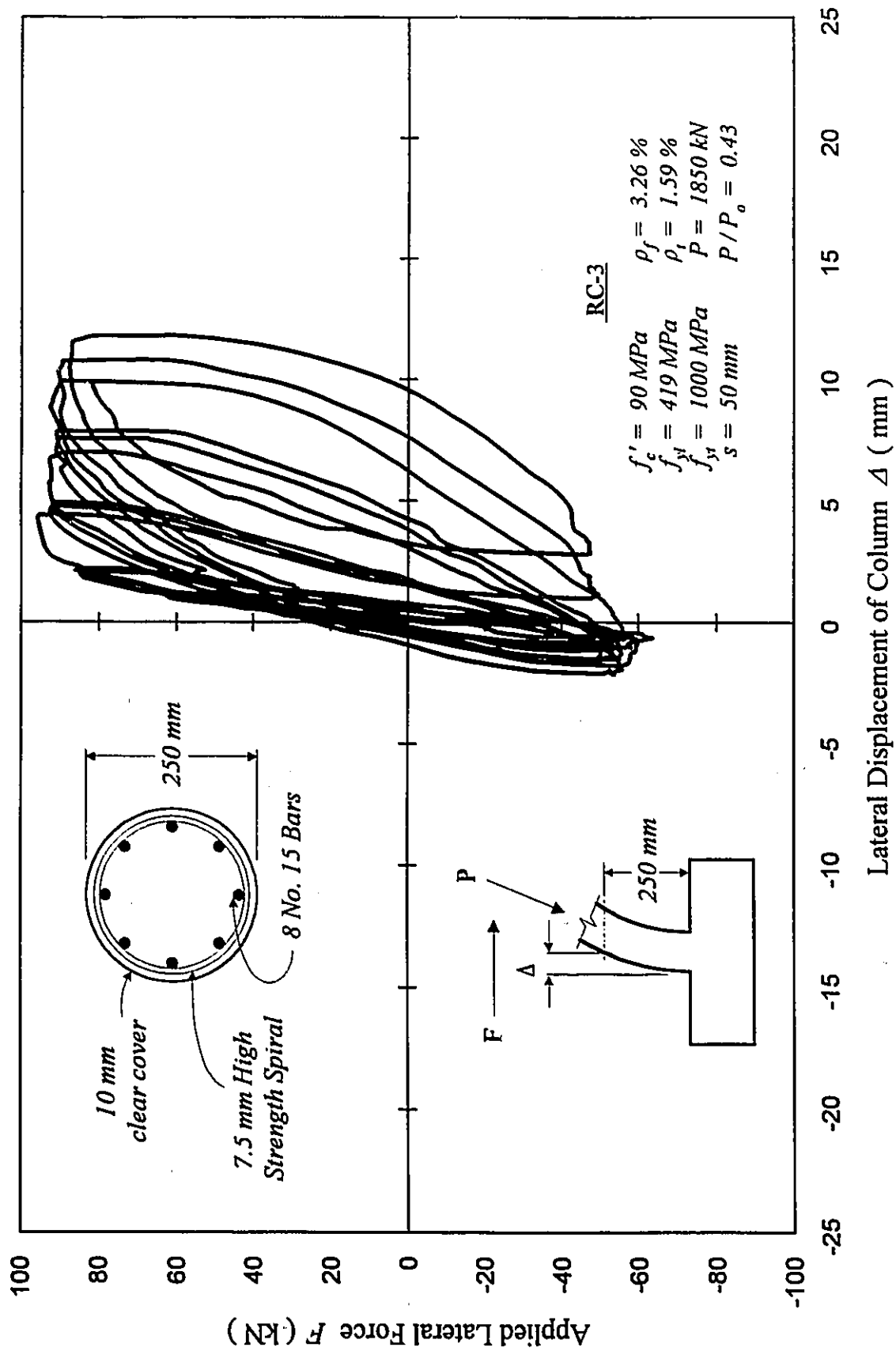


Figure A.10: Force - Displacement Relationship for Column RC-3
(At Top of Hinge Region - without $P-\Delta$ effect)

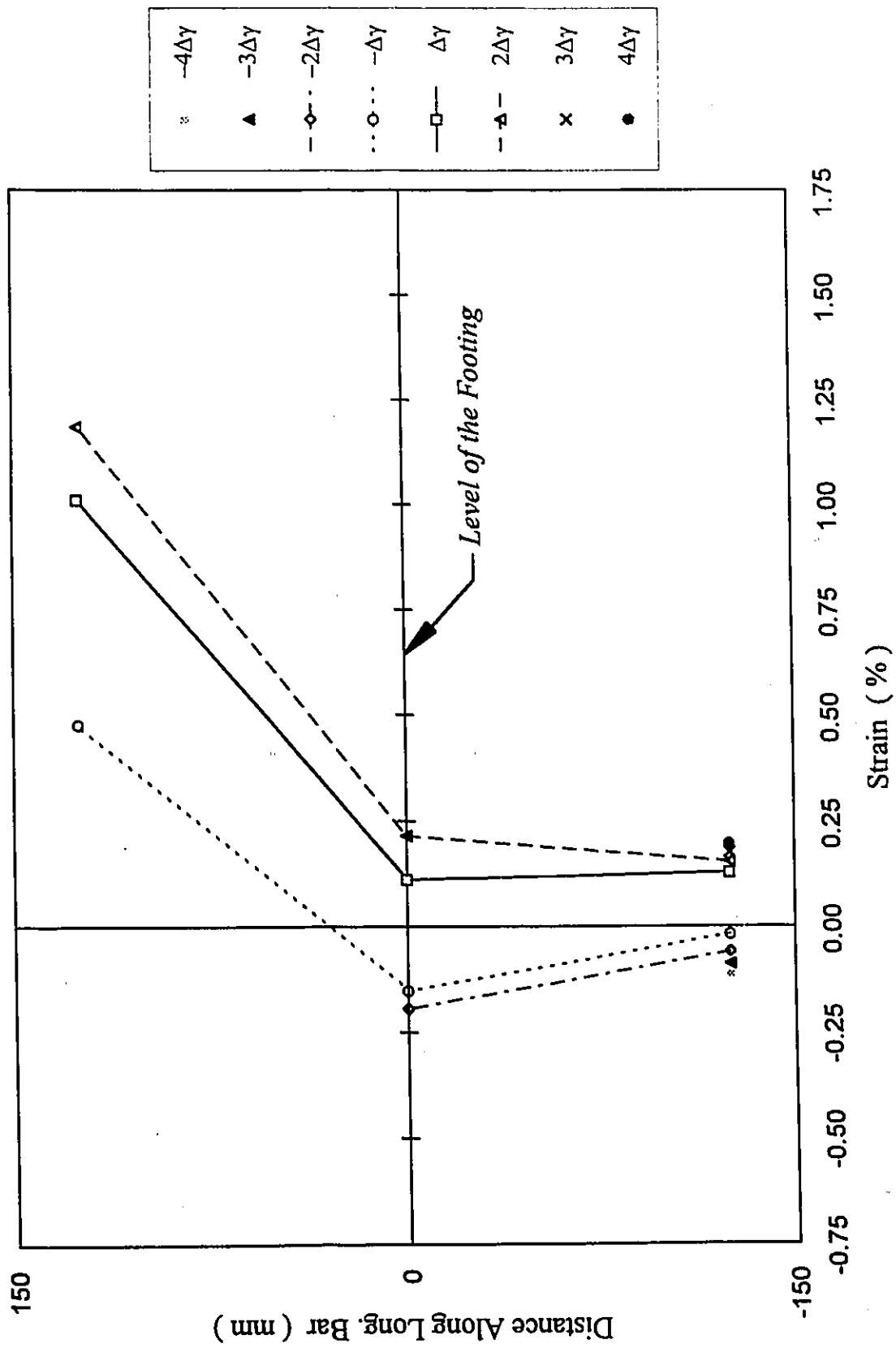


Figure A.11: Strain Distribution in Longitudinal Reinforcement for Different Δy Values
Column RC-3

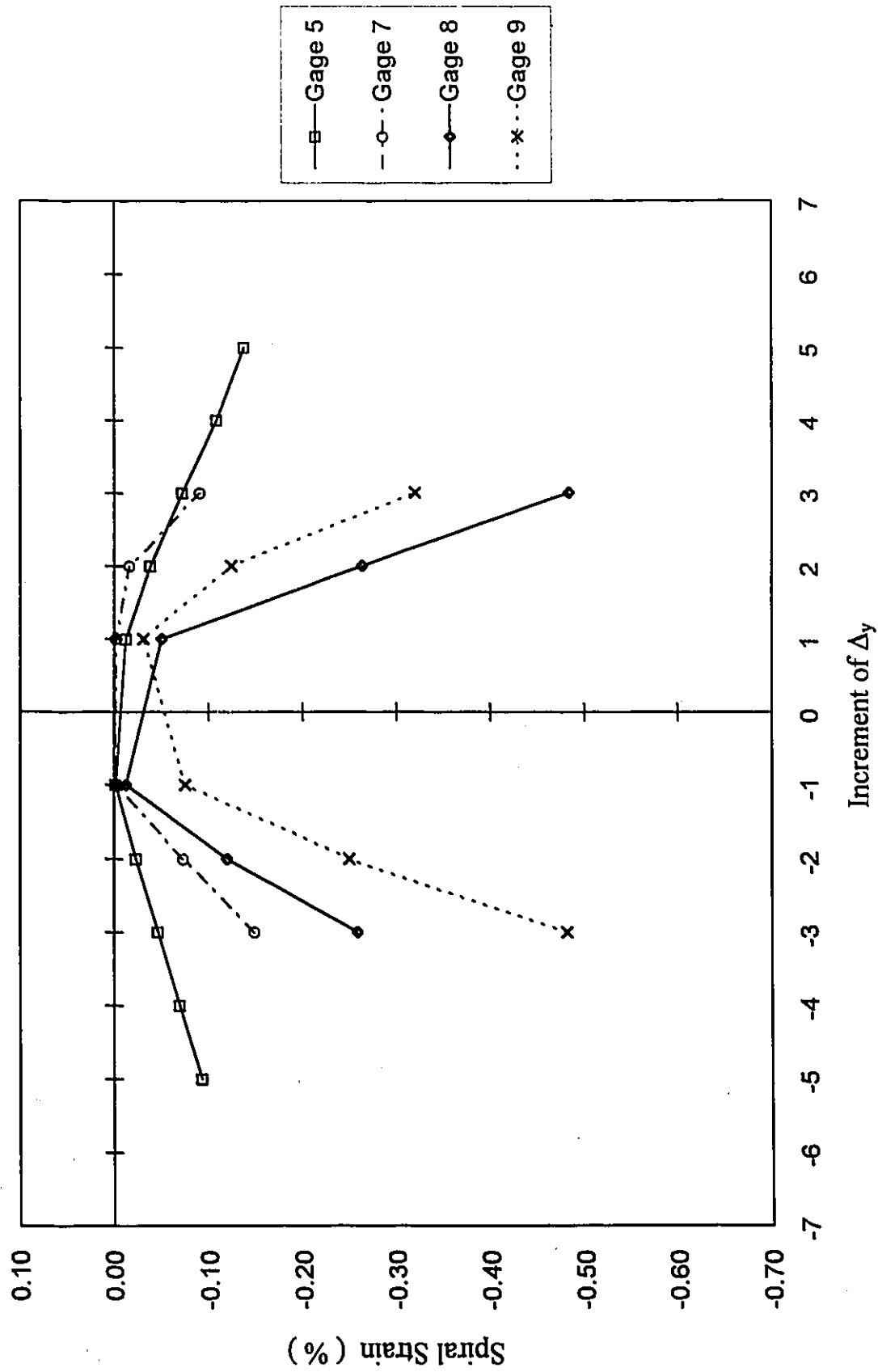


Figure A.12: Strain Distribution in Transverse Reinforcement for Different Δy Increments
Column RC-3

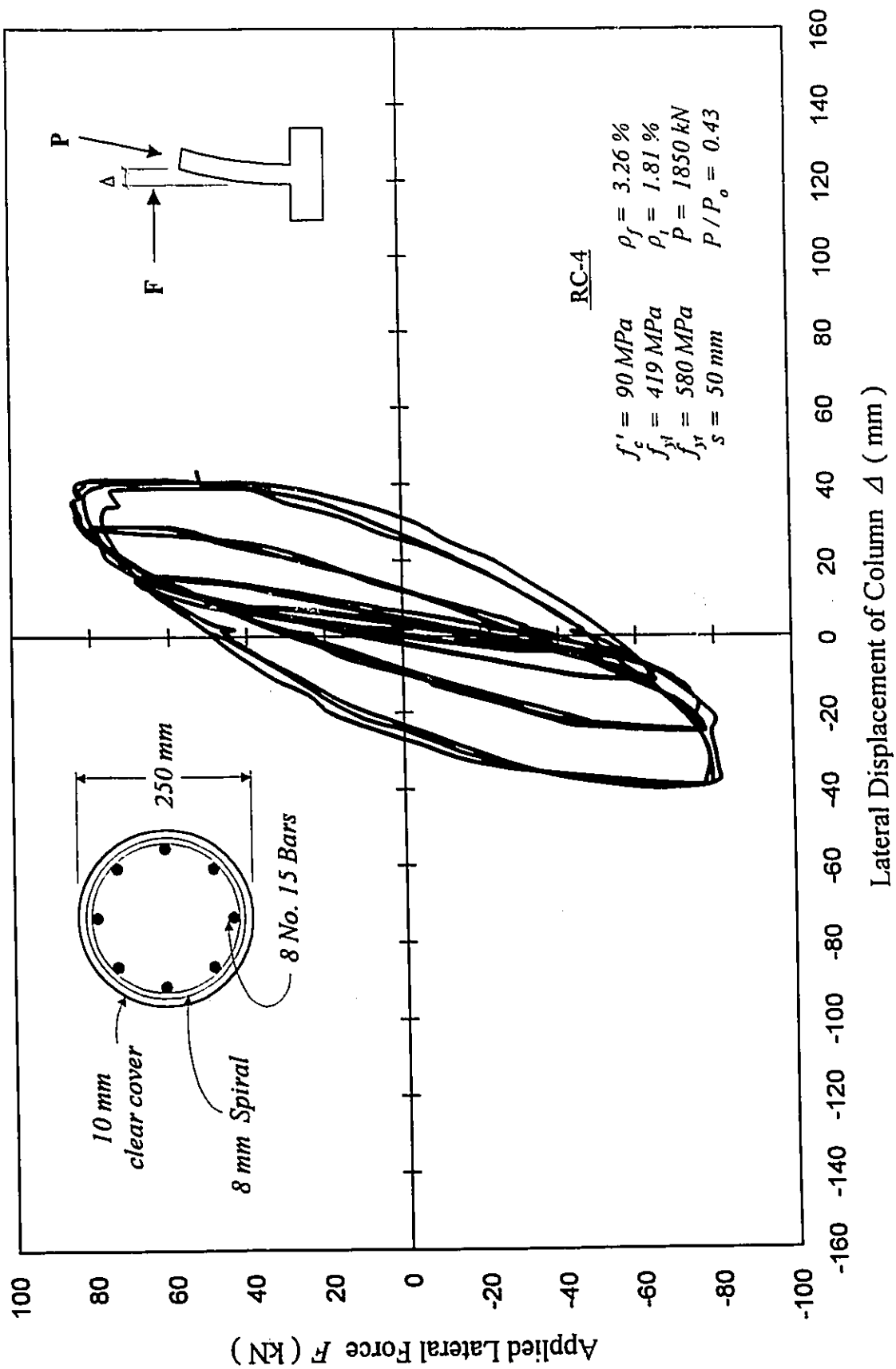


Figure A.13: Force - Displacement Relationship for Column RC-4
(At Tip of Column - without P - Δ effect) ;

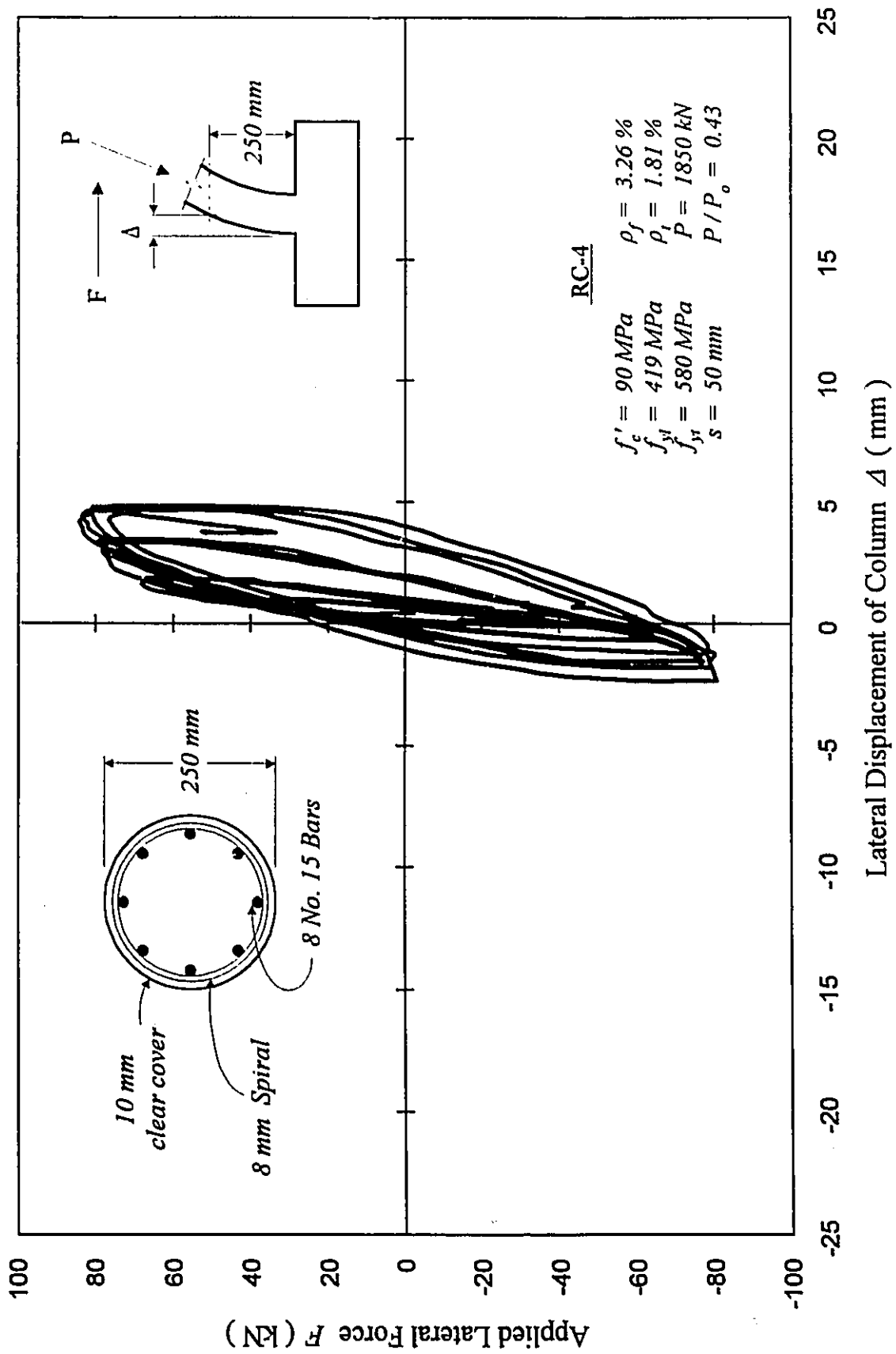


Figure A.14: Force - Displacement Relationship for Column RC-4
(At Top of Hinge Region - without P - Δ effect)

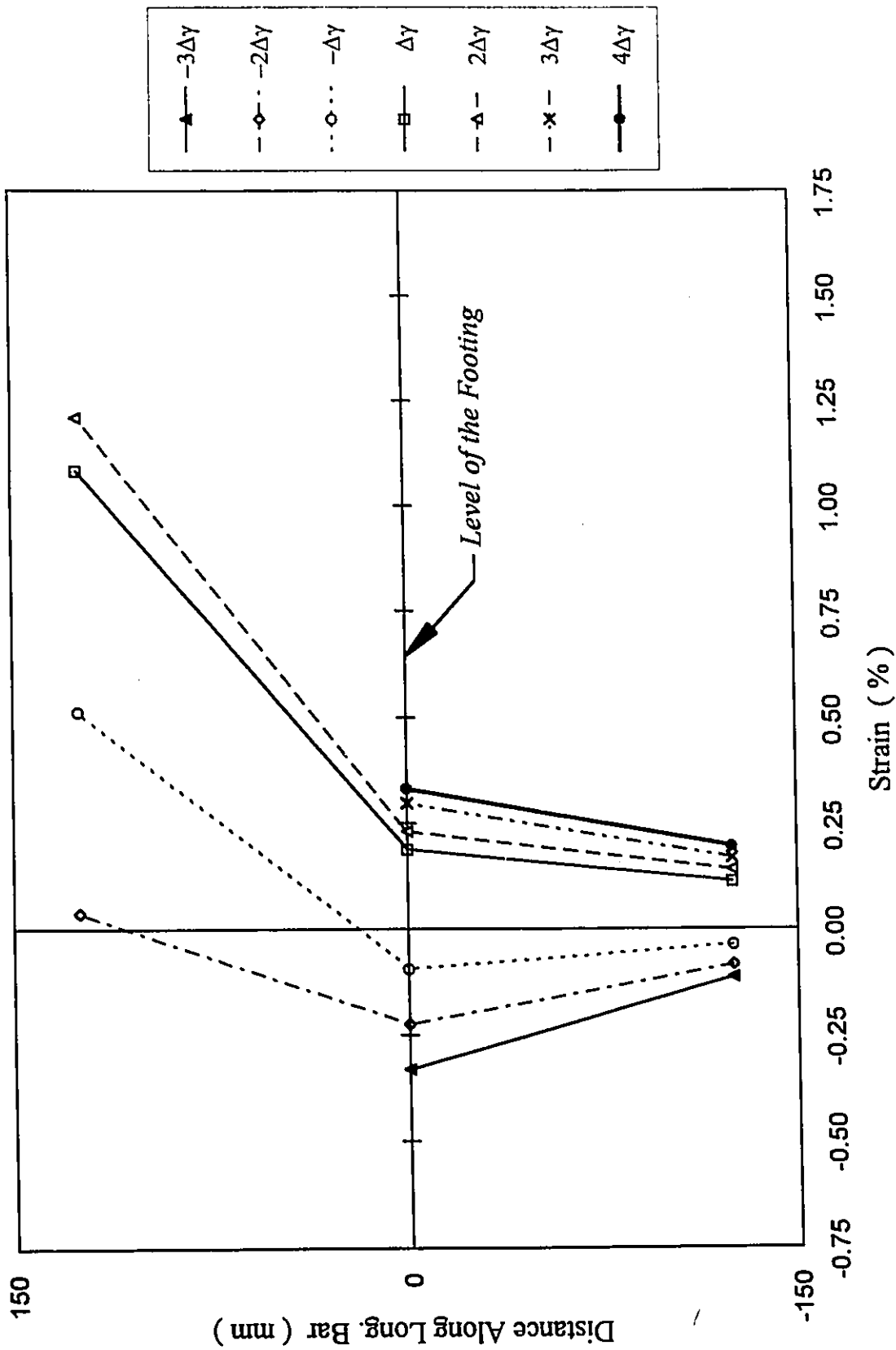


Figure A.15: Strain Distribution in Longitudinal Reinforcement for Different Δy Values
Column RC-4

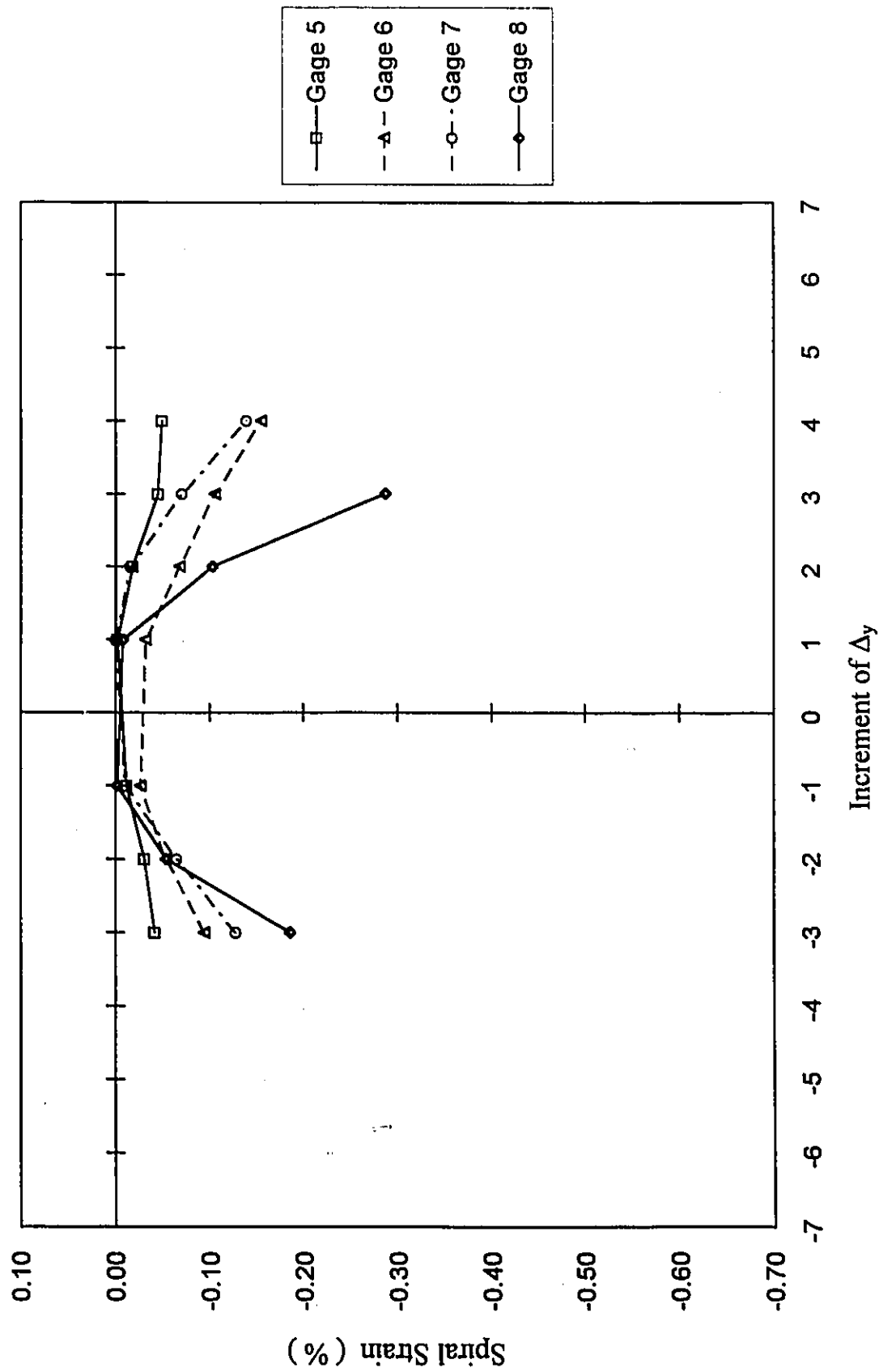


Figure A.16: Strain Distribution in Transverse Reinforcement for Different Δy Increments
Column RC-4

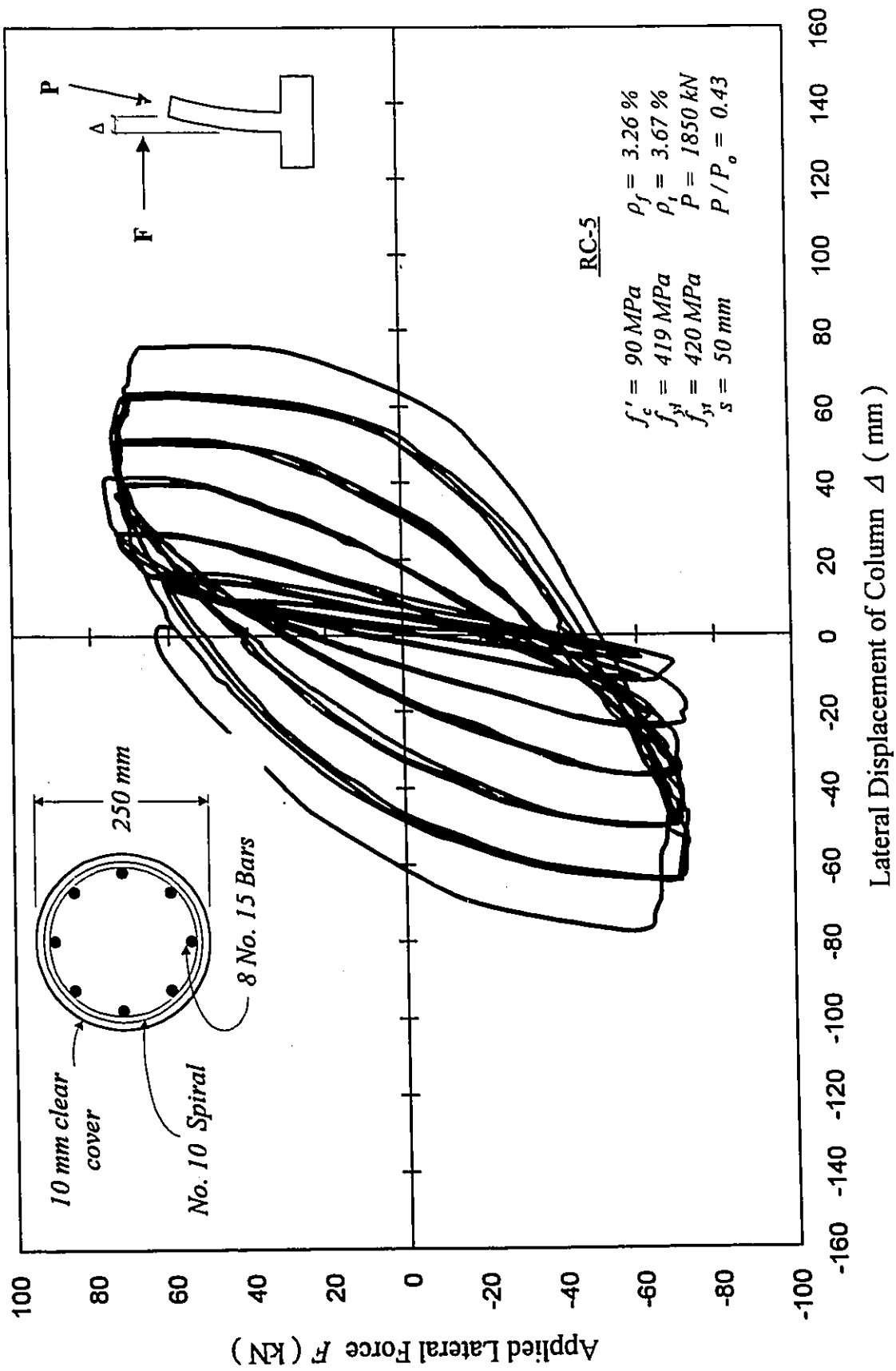


Figure A.17: Force - Displacement Relationship for Column RC-5
(At Tip of Column - without $P-\Delta$ effect)

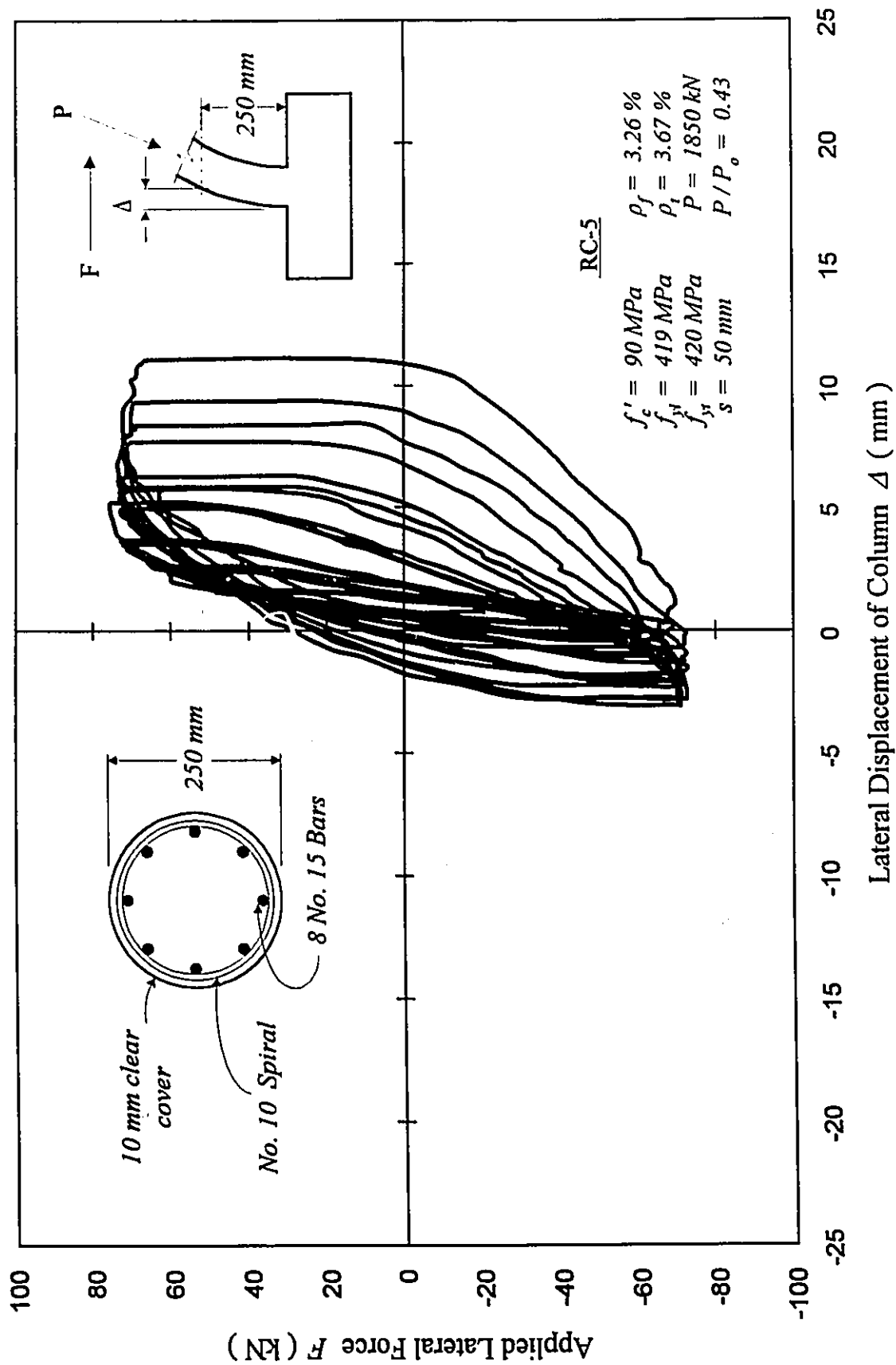


Figure A.18: Force - Displacement Relationship for Column RC-5
(At Top of Hinge Region - without $P-\Delta$ effect)

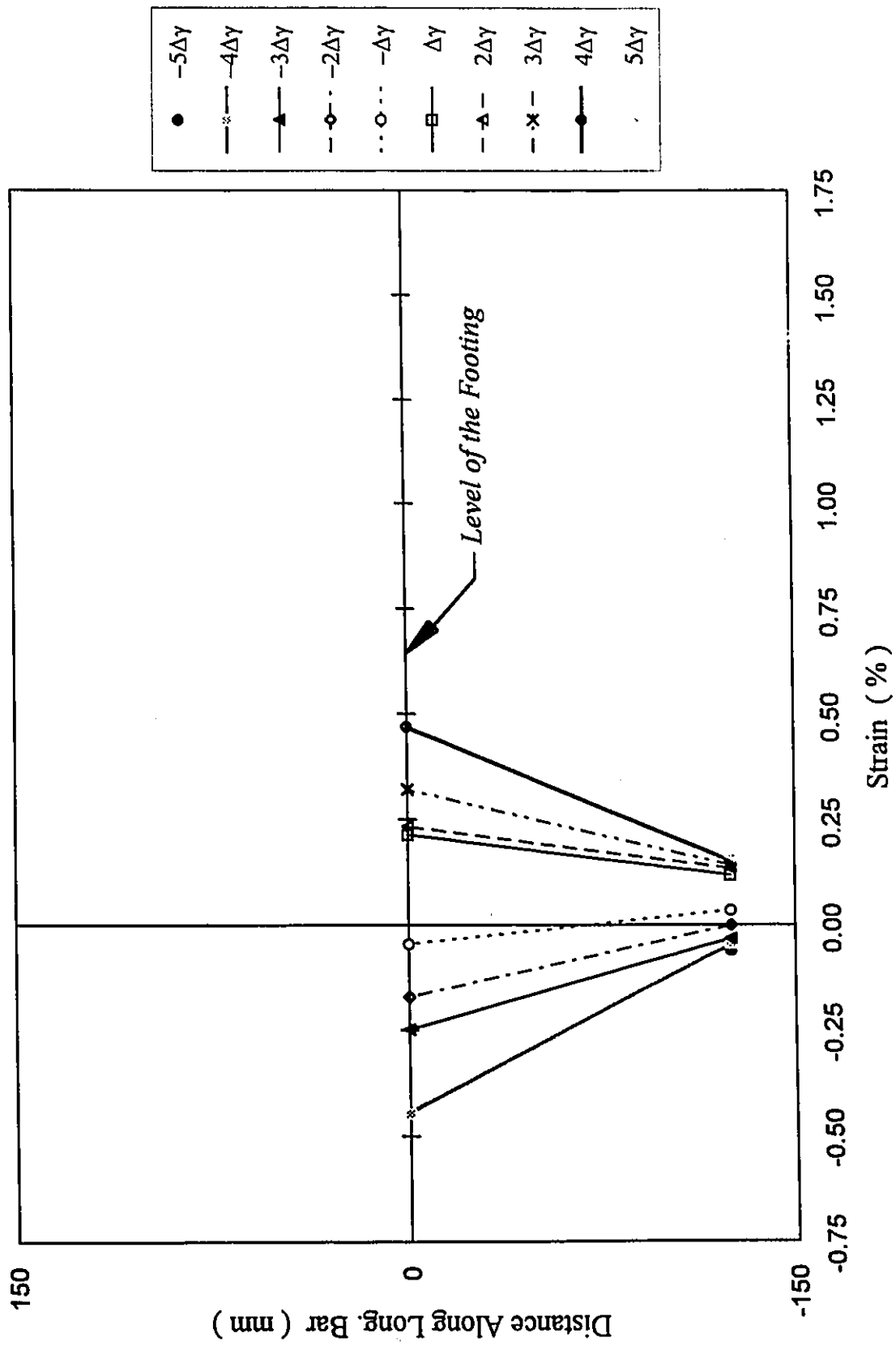


Figure A.19: Strain Distribution in Longitudinal Reinforcement for Different Δy Values
Column RC-5

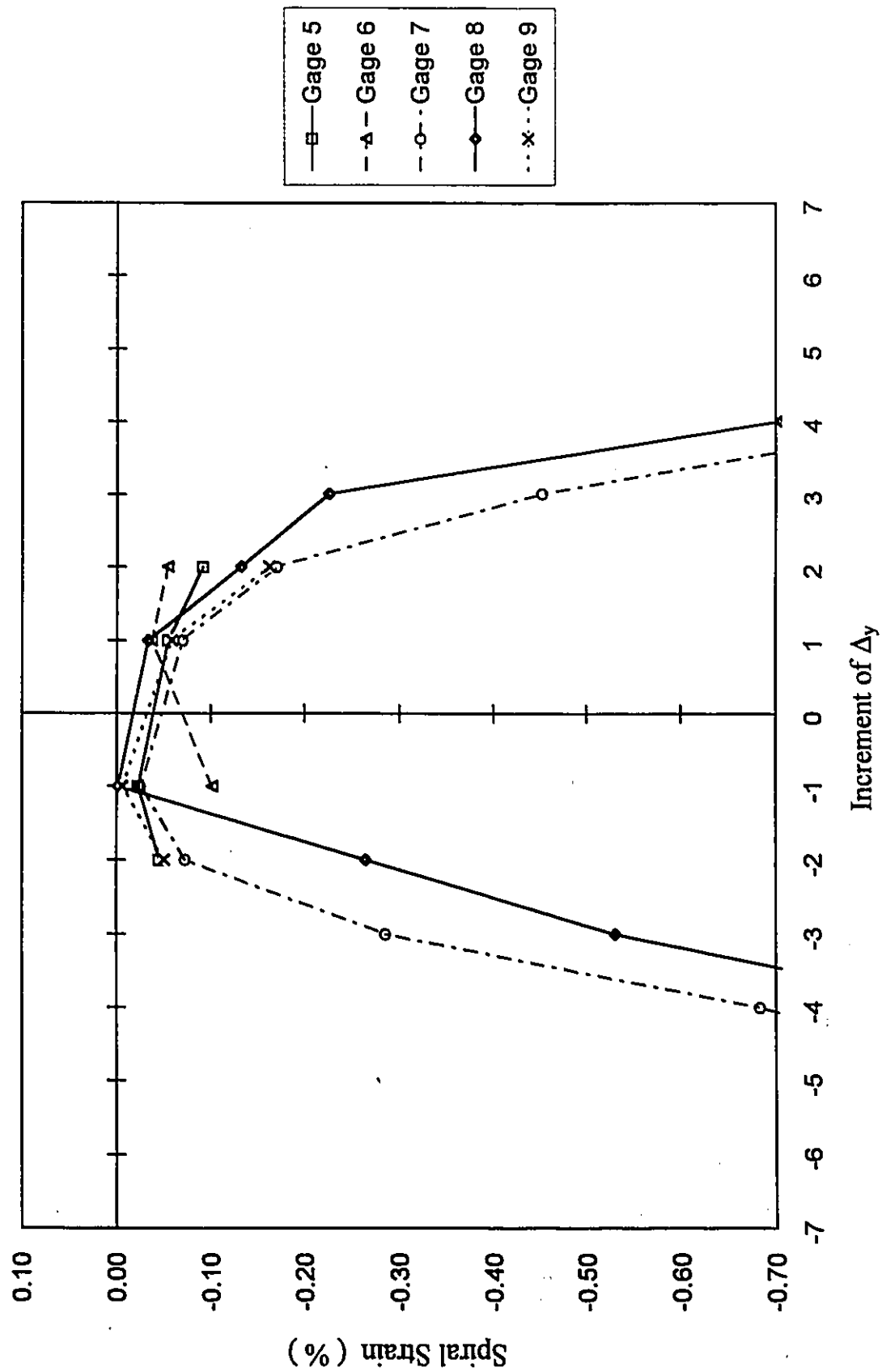


Figure A.20: Strain Distribution in Transverse Reinforcement for Different Δy Increments
Column RC-5

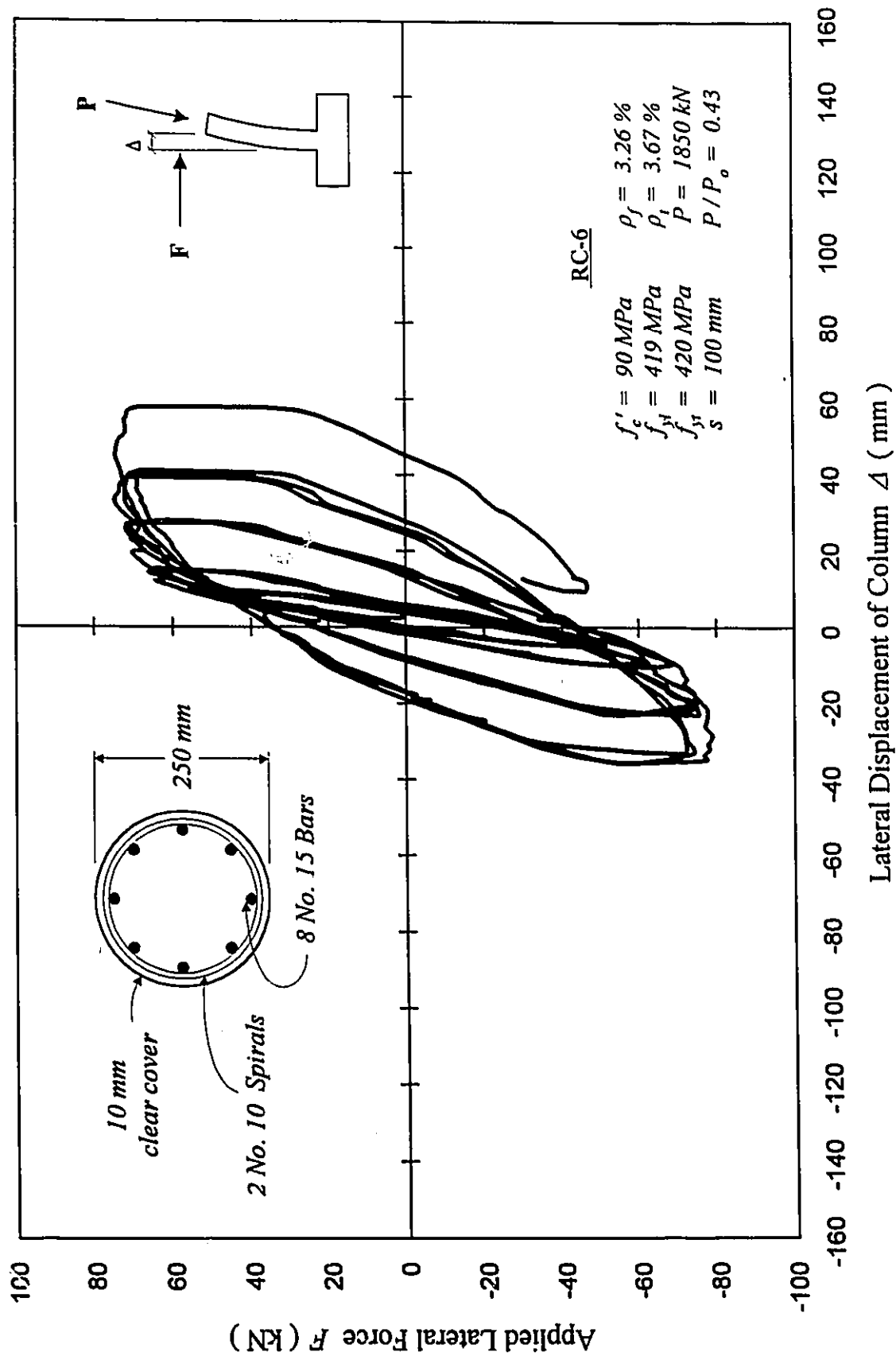


Figure A.21: Force - Displacement Relationship for Column RC-6
(At Tip of Column - without $P-\Delta$ effect)

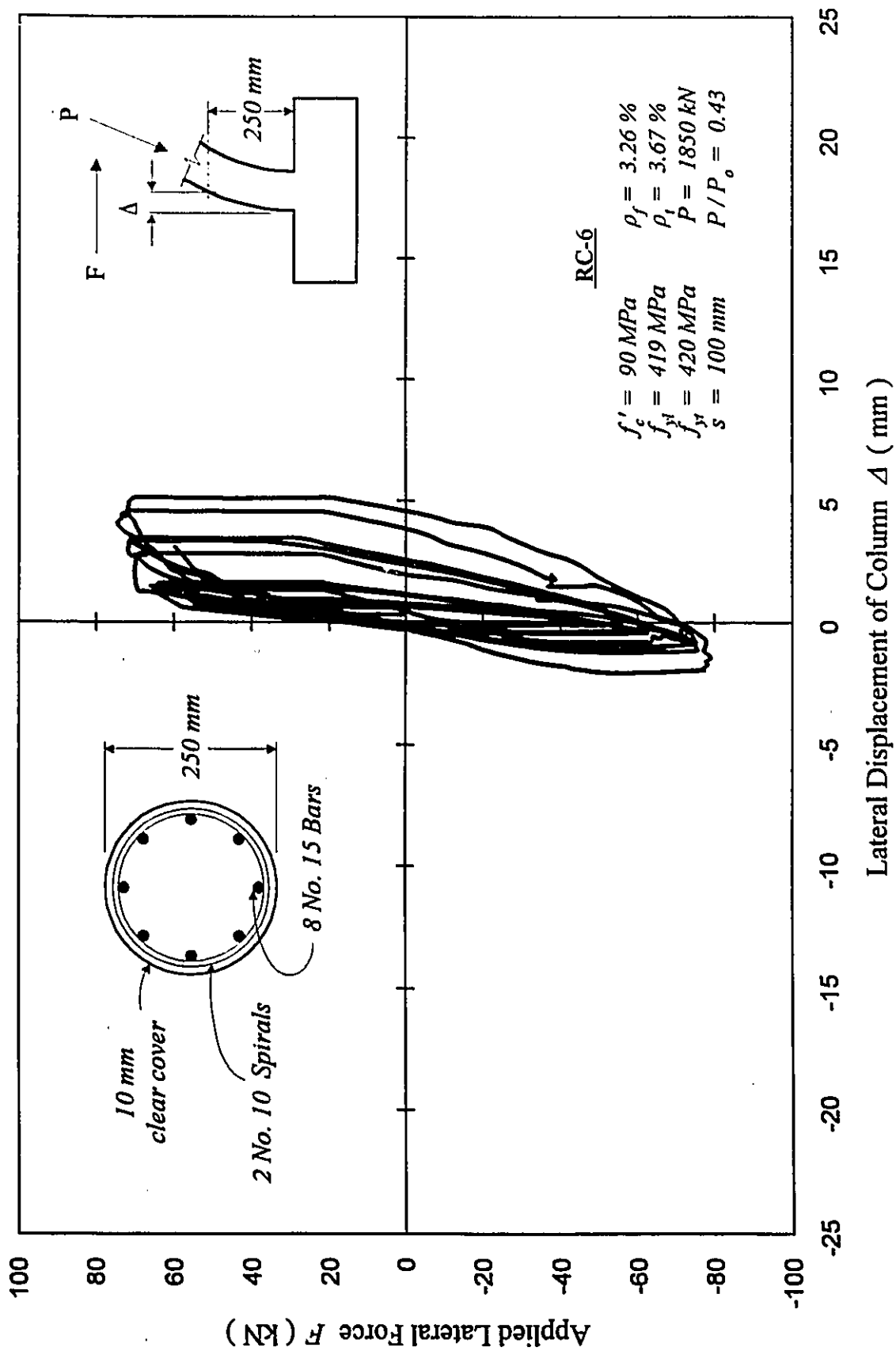


Figure A.22: Force - Displacement Relationship for Column RC-6
(At Top of Hinge Region - without $P-\Delta$ effect)

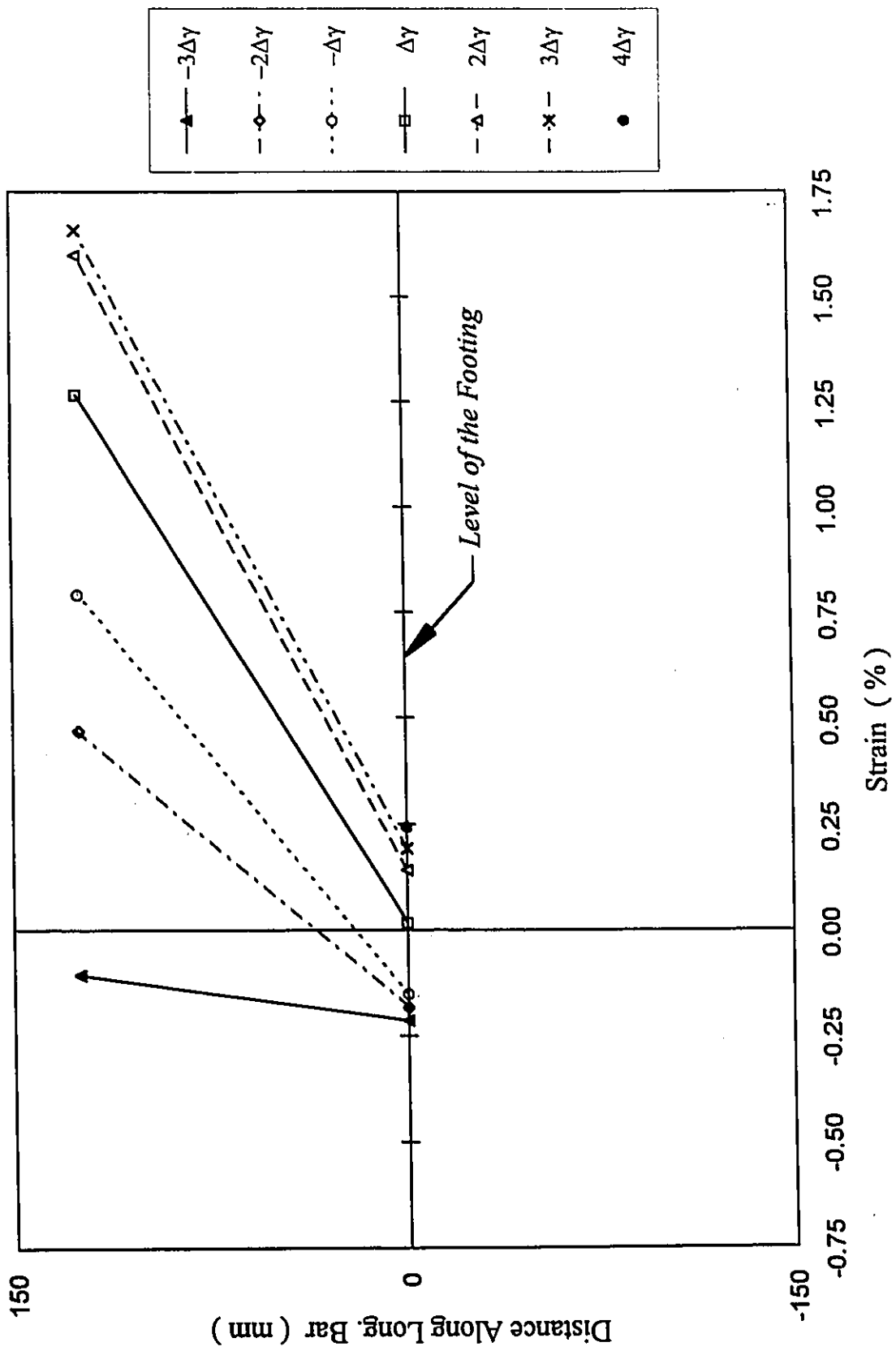


Figure A.23: Strain Distribution in Longitudinal Reinforcement for Different Δy Values
Column RC-6

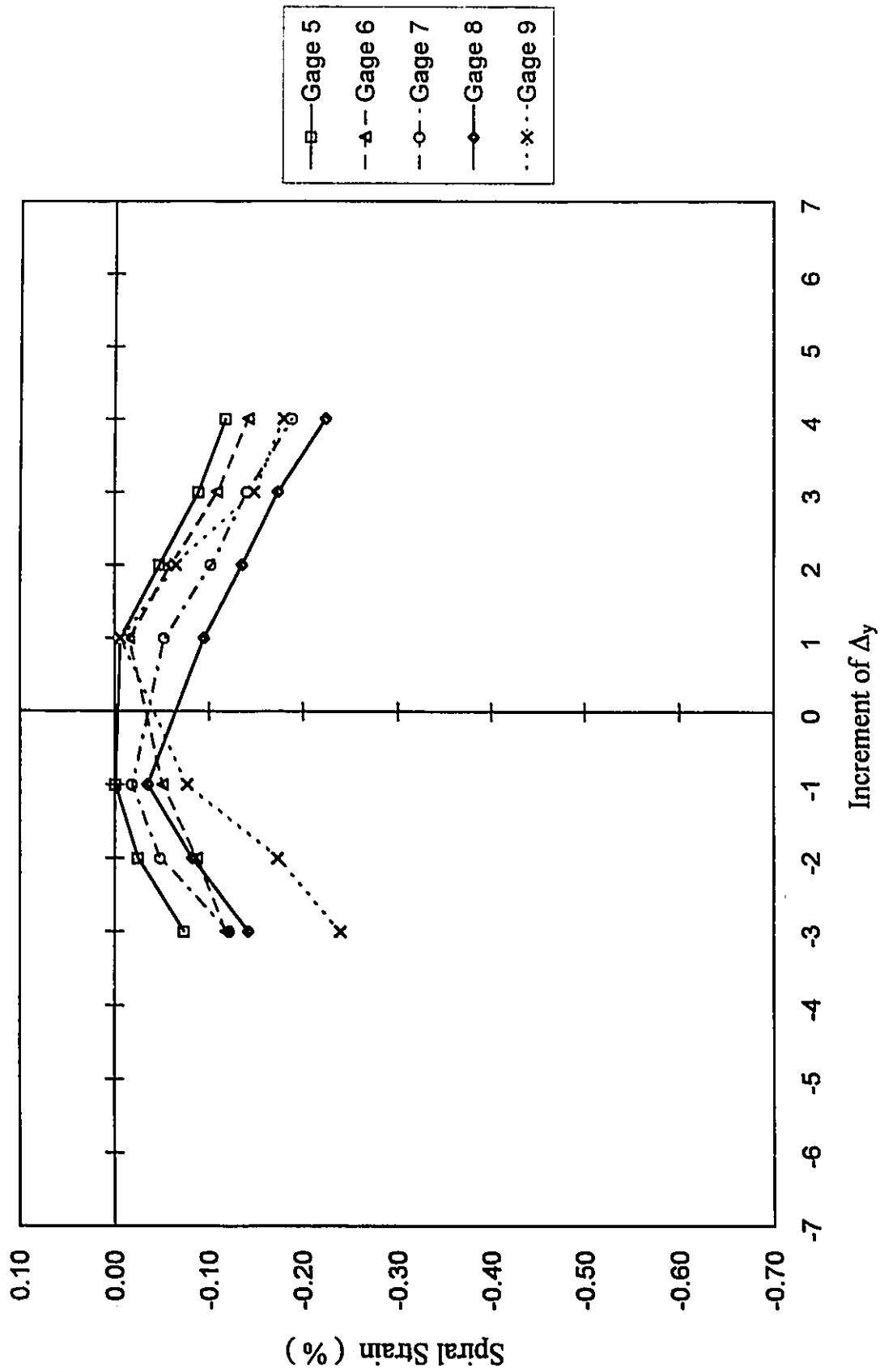


Figure A.24: Strain Distribution in Transverse Reinforcement for Different Δy Increments
Column RC-6

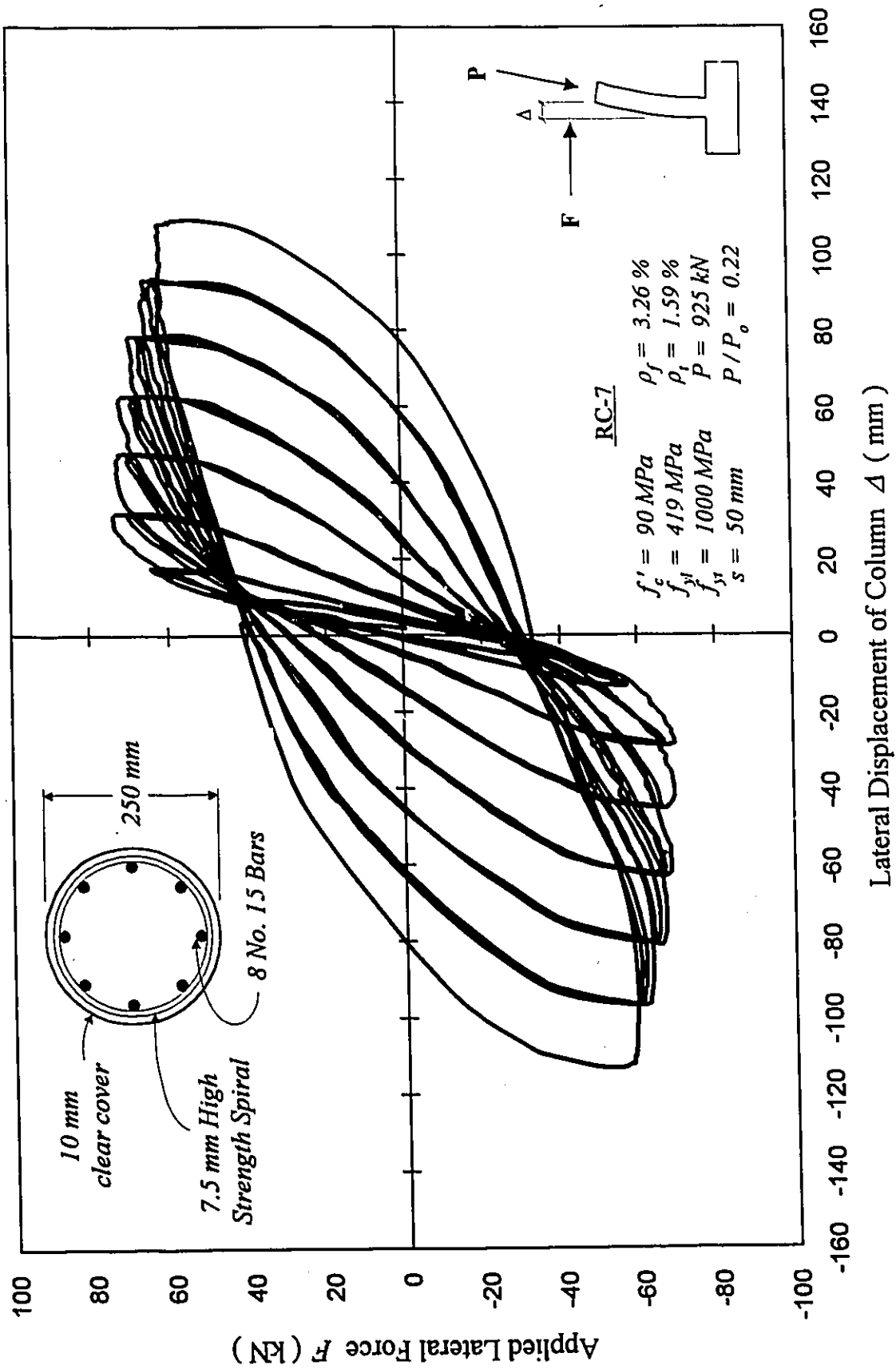


Figure A.25: Force - Displacement Relationship for Column RC-7
 (At Tip of Column - without P - Δ effect)

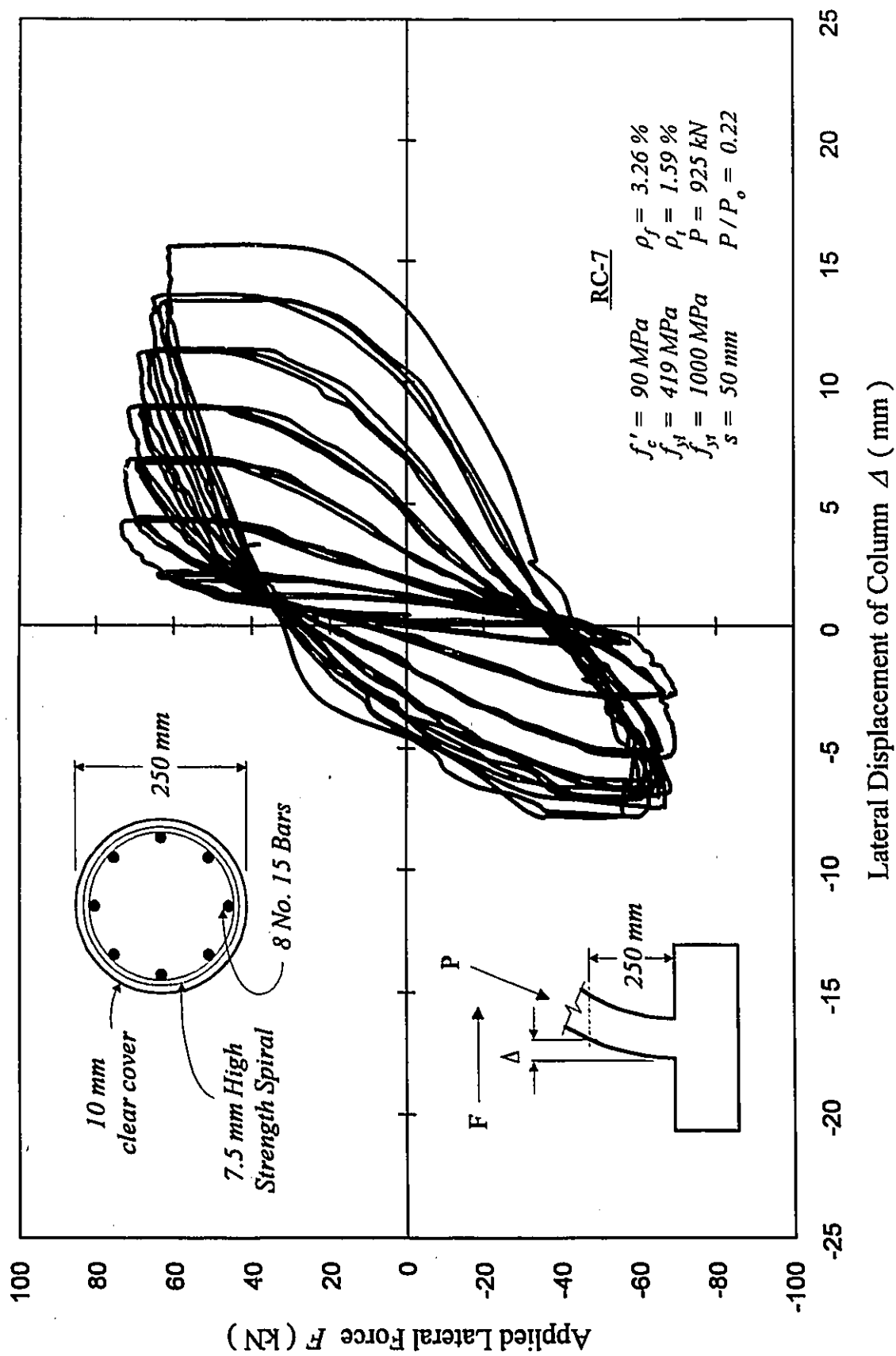


Figure A.26: Force - Displacement Relationship for Column RC-7
(At Top of Hinge Region - without $P-\Delta$ effect)

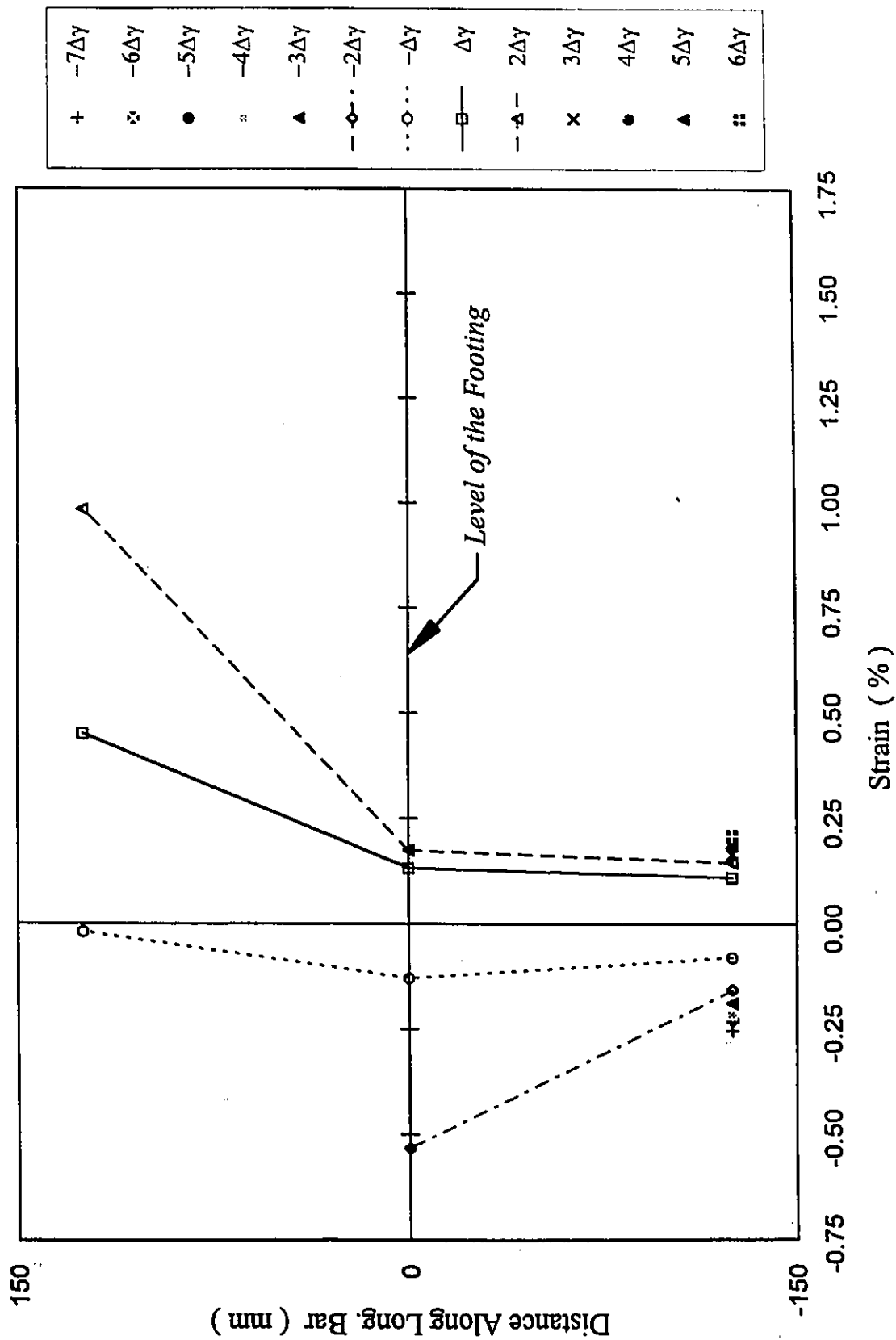


Figure A.27: Strain Distribution in Longitudinal Reinforcement for Different Δy Values
Column RC-7

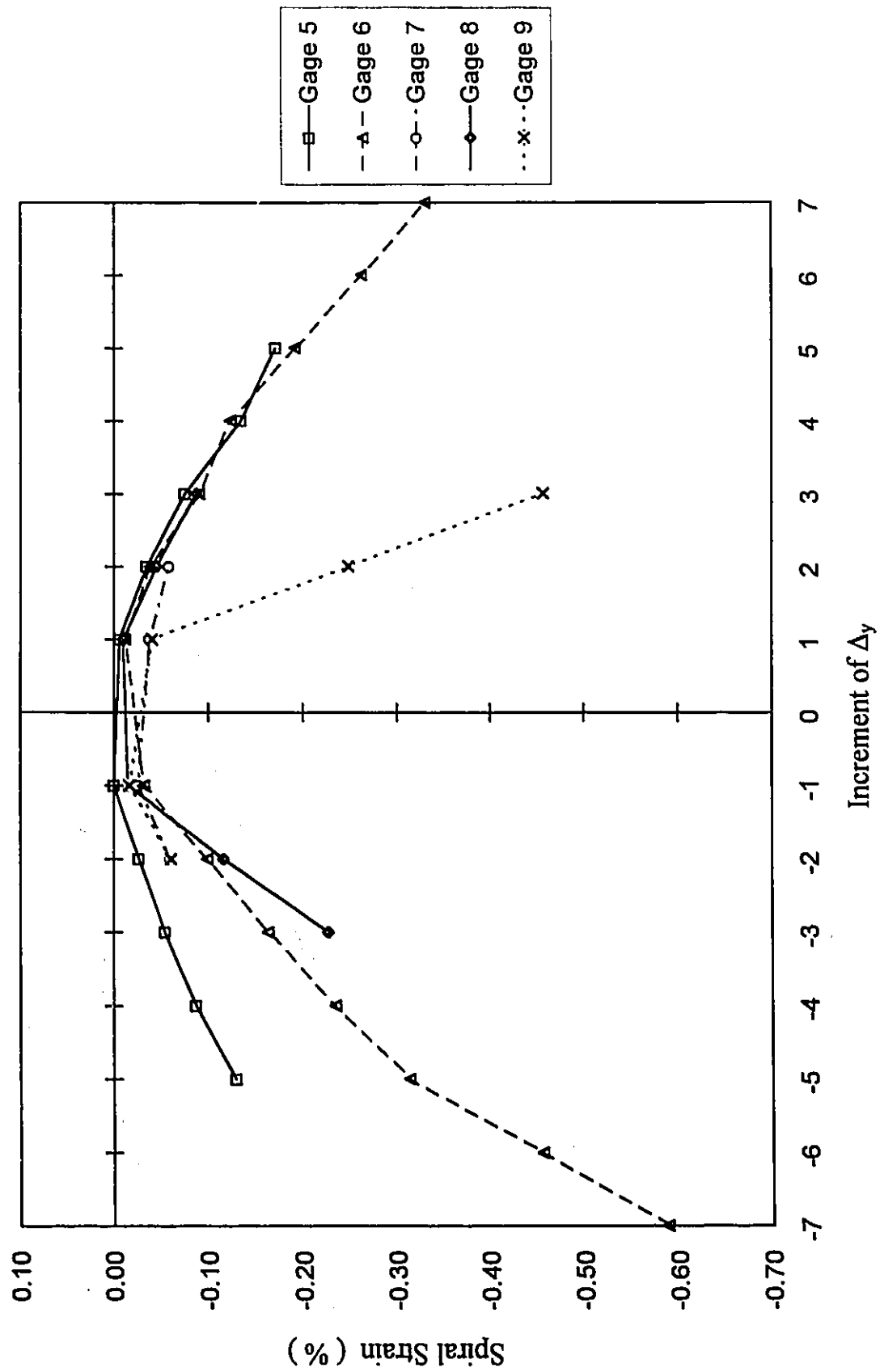


Figure A.28: Strain Distribution in Transverse Reinforcement for Different Δy Increments
Column RC-7

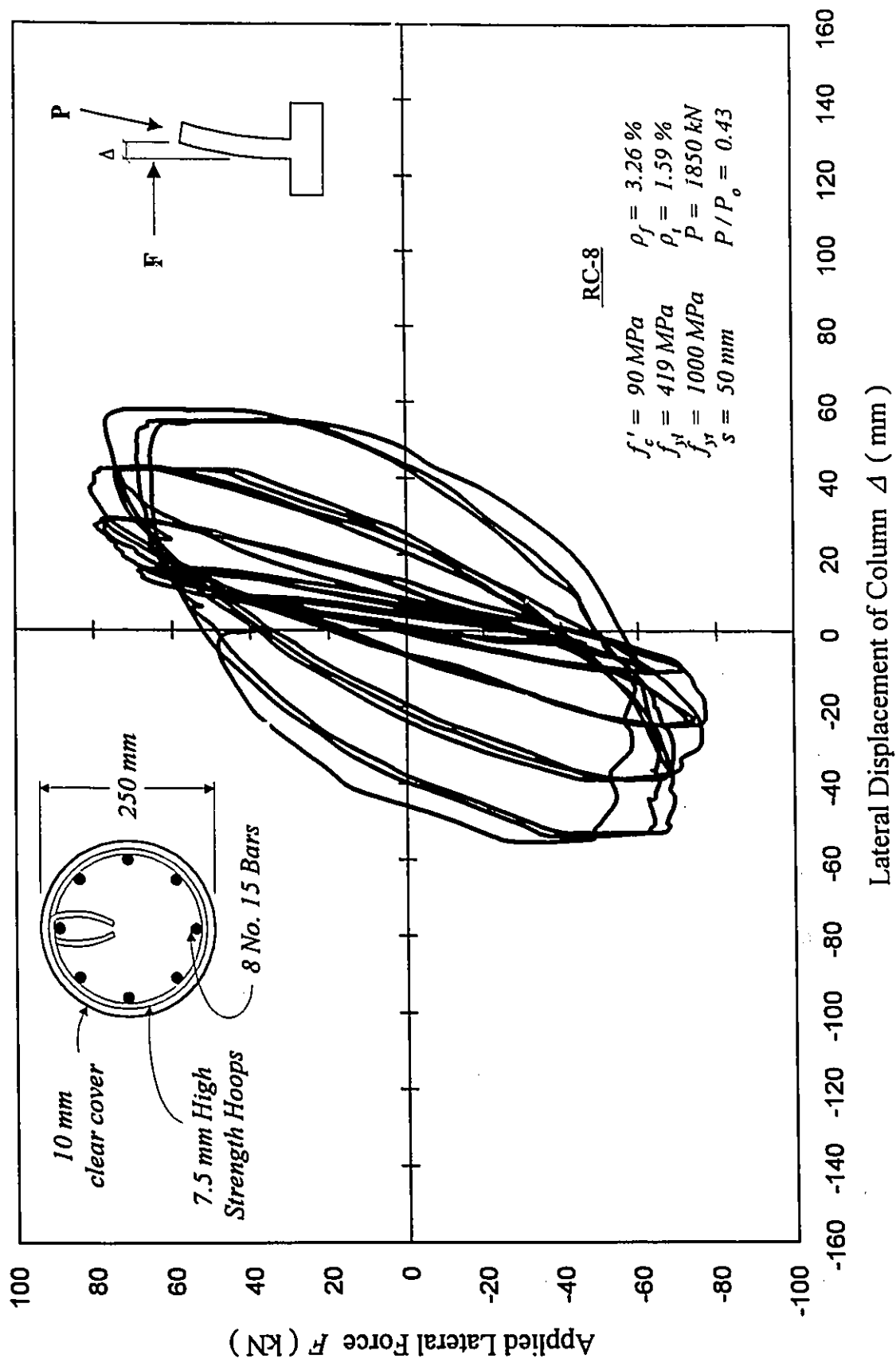


Figure A.29: Force - Displacement Relationship for Column RC-8
(At Tip of Column - without P - Δ effect)

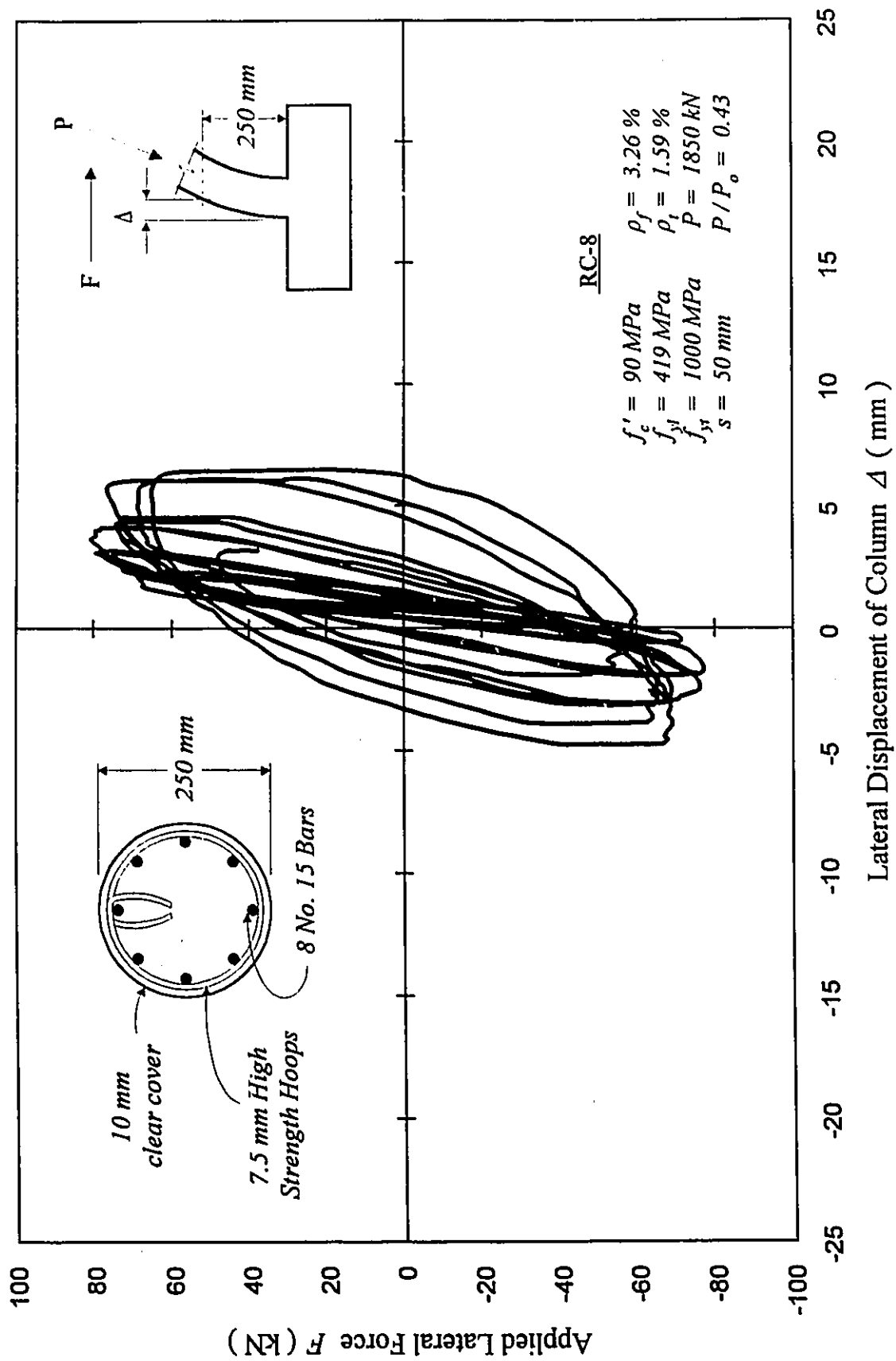


Figure A.30: Force - Displacement Relationship for Column RC-8
(At Top of Hinge Region - without $P-\Delta$ effect)

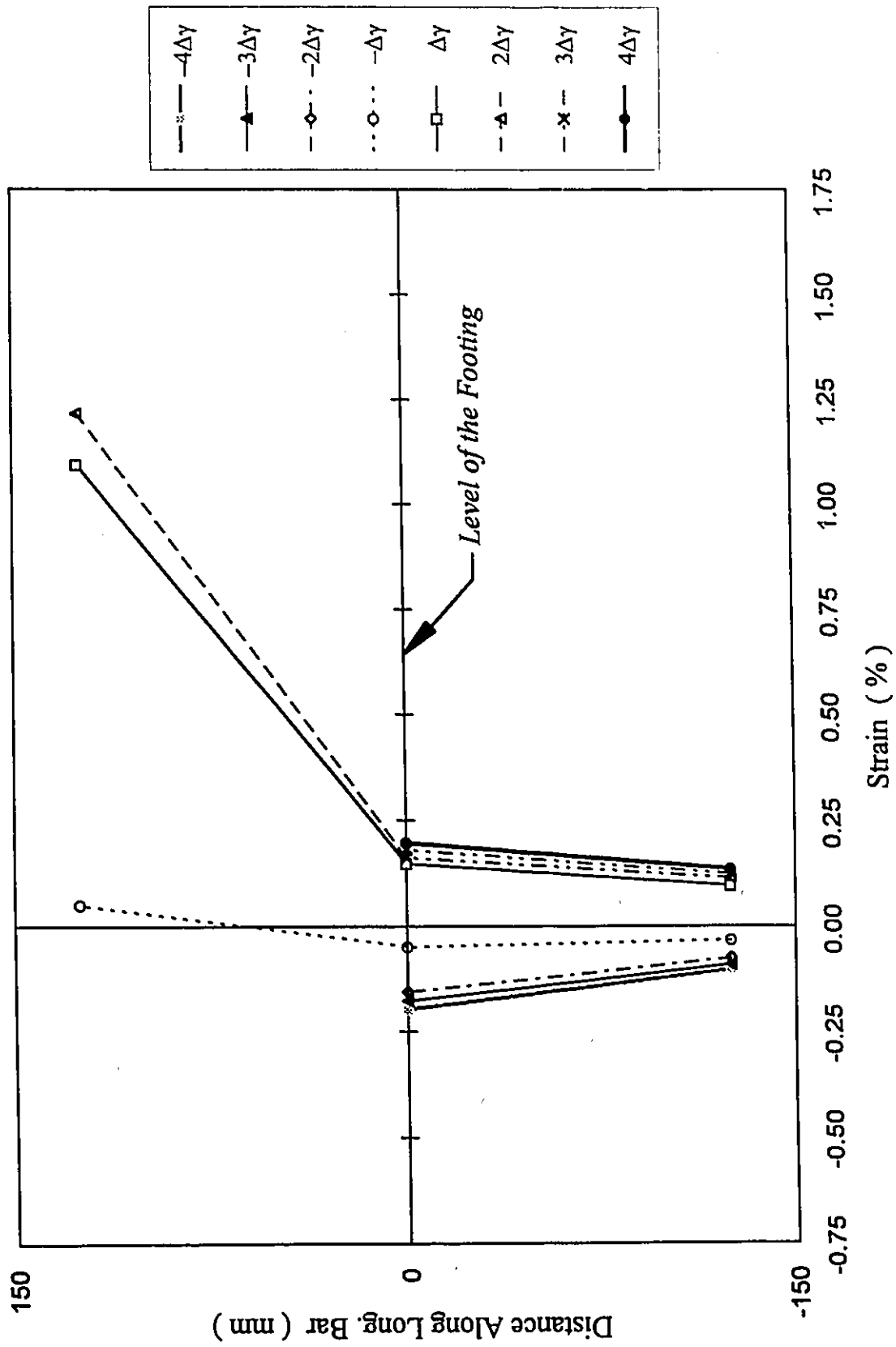


Figure A.31: Strain Distribution in Longitudinal Reinforcement for Different Δy Values
Column RC-8

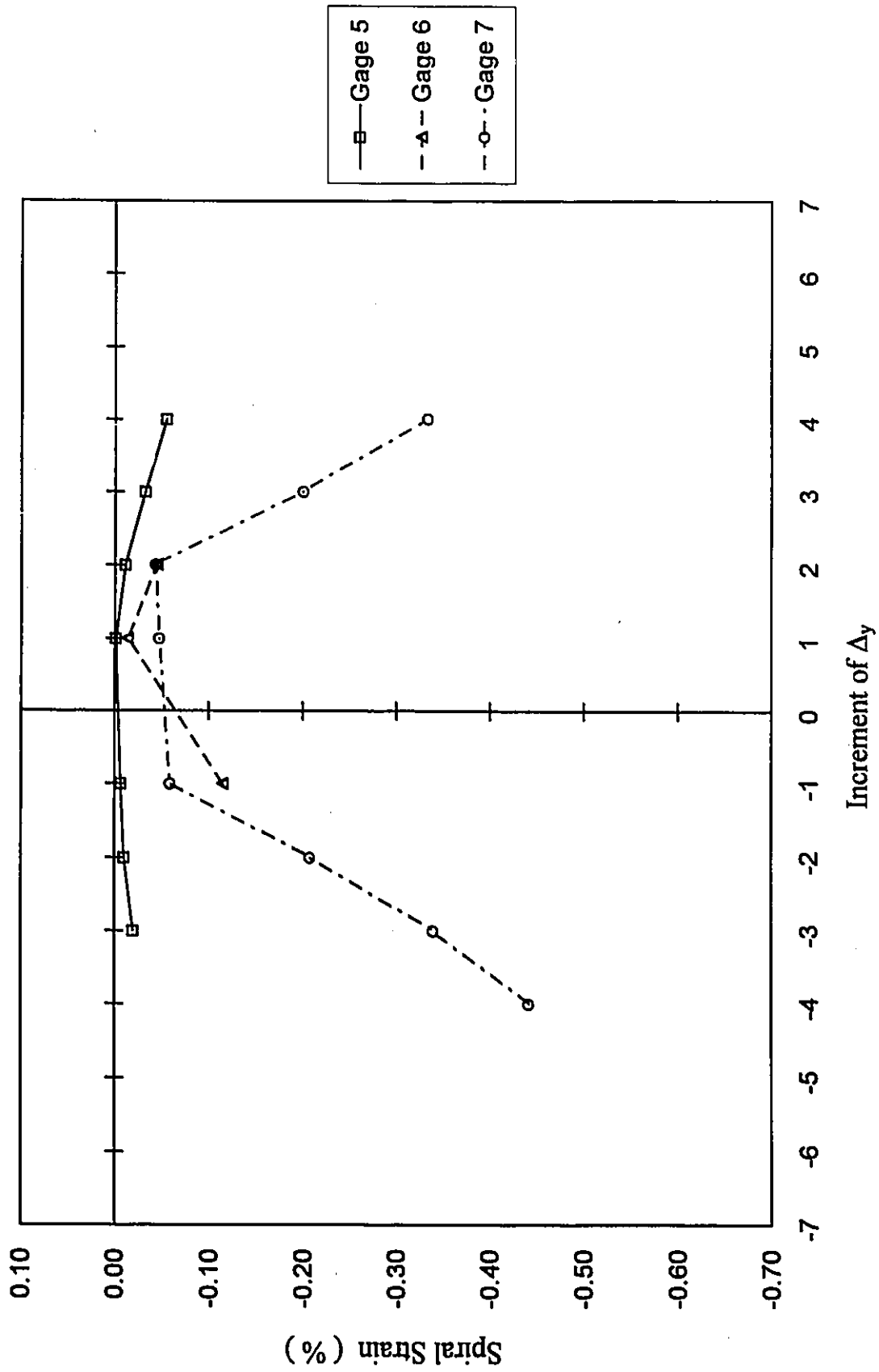


Figure A.32: Strain Distribution in Transverse Reinforcement for Different Δy Increments
Column RC-8

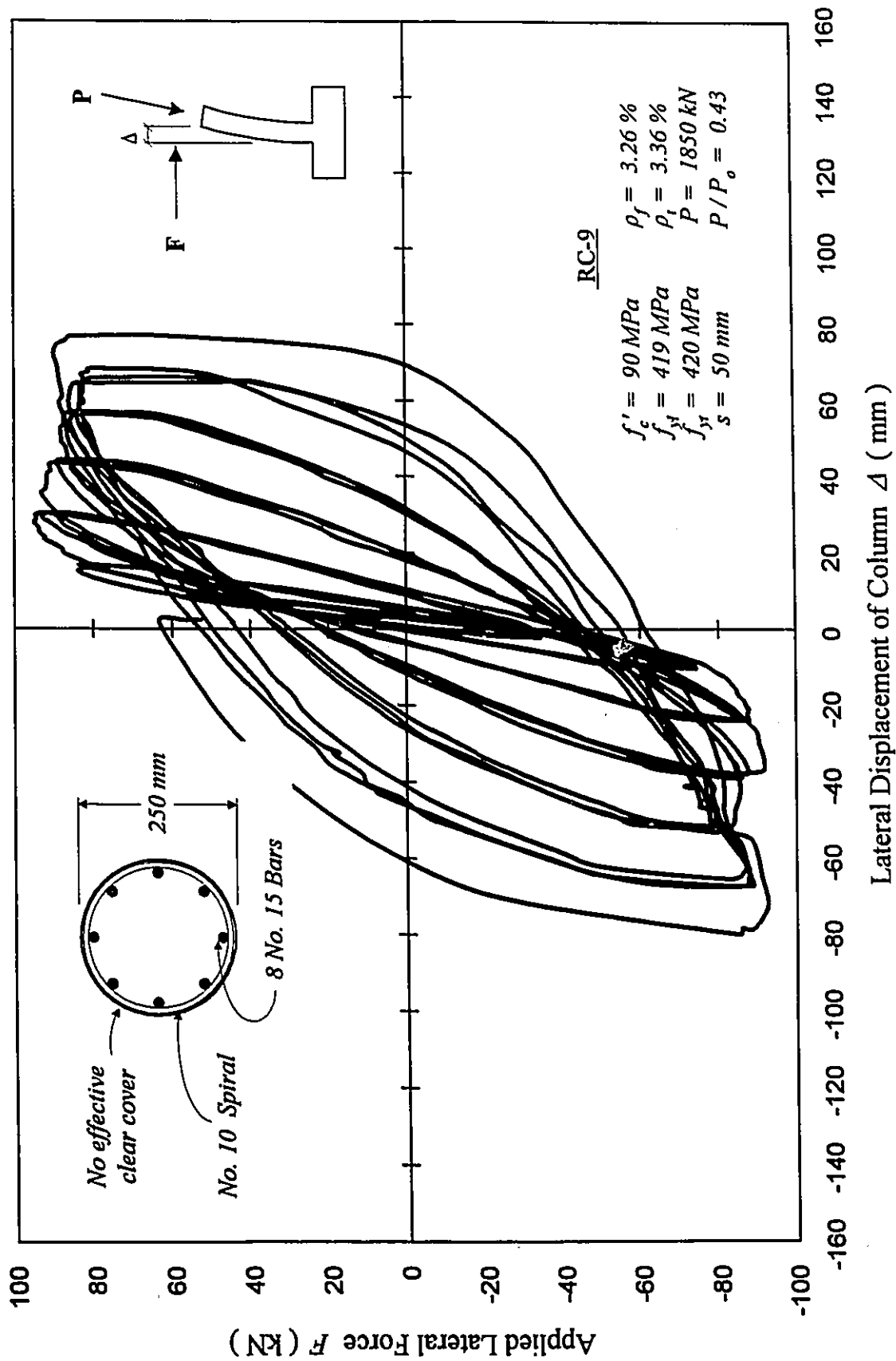


Figure A.33: Force - Displacement Relationship for Column RC-9
(At Tip of Column - without $P-\Delta$ effect)

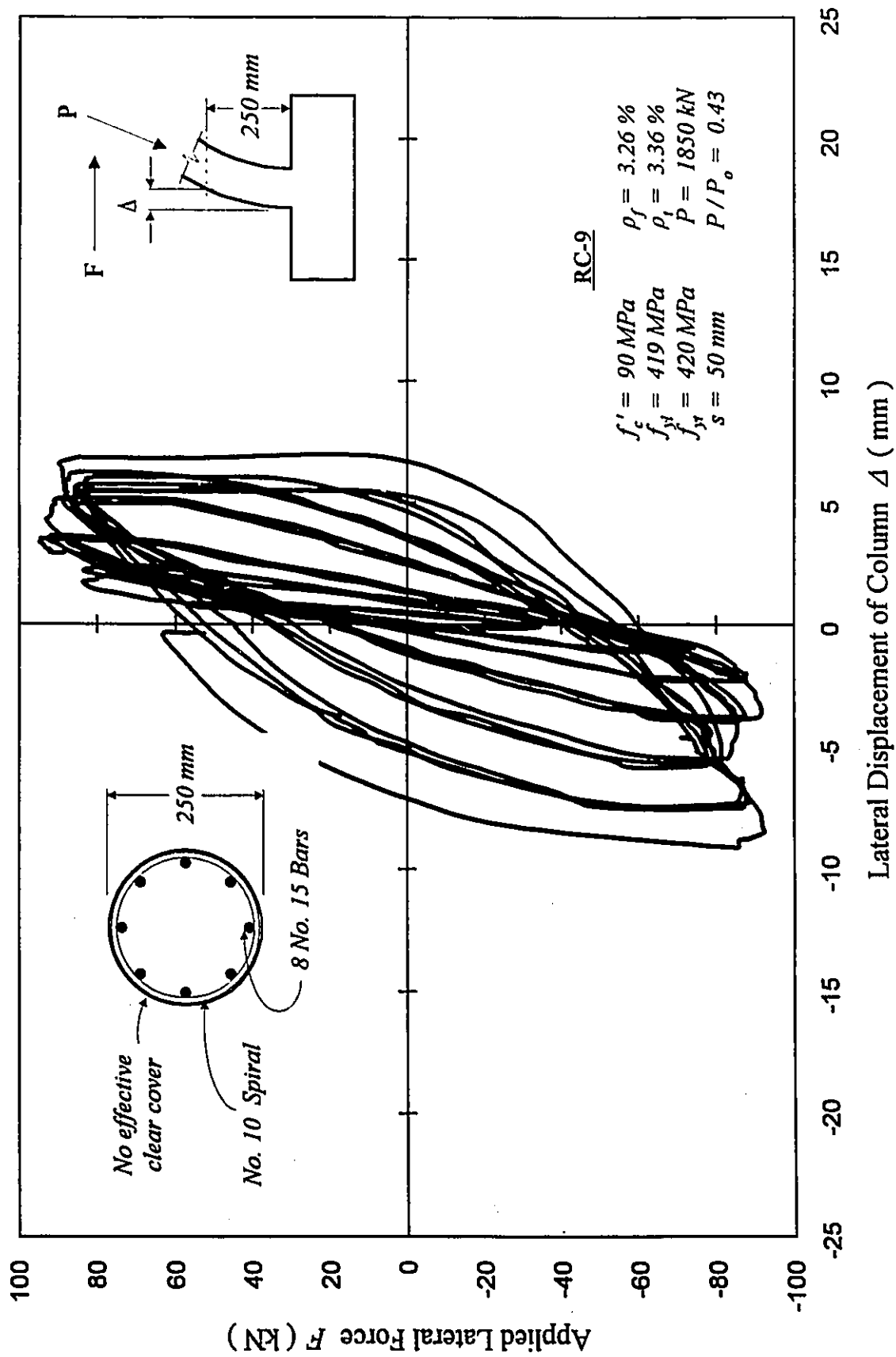


Figure A.34: Force - Displacement Relationship for Column RC-9
(At Top of Hinge Region - without $P-\Delta$ effect)

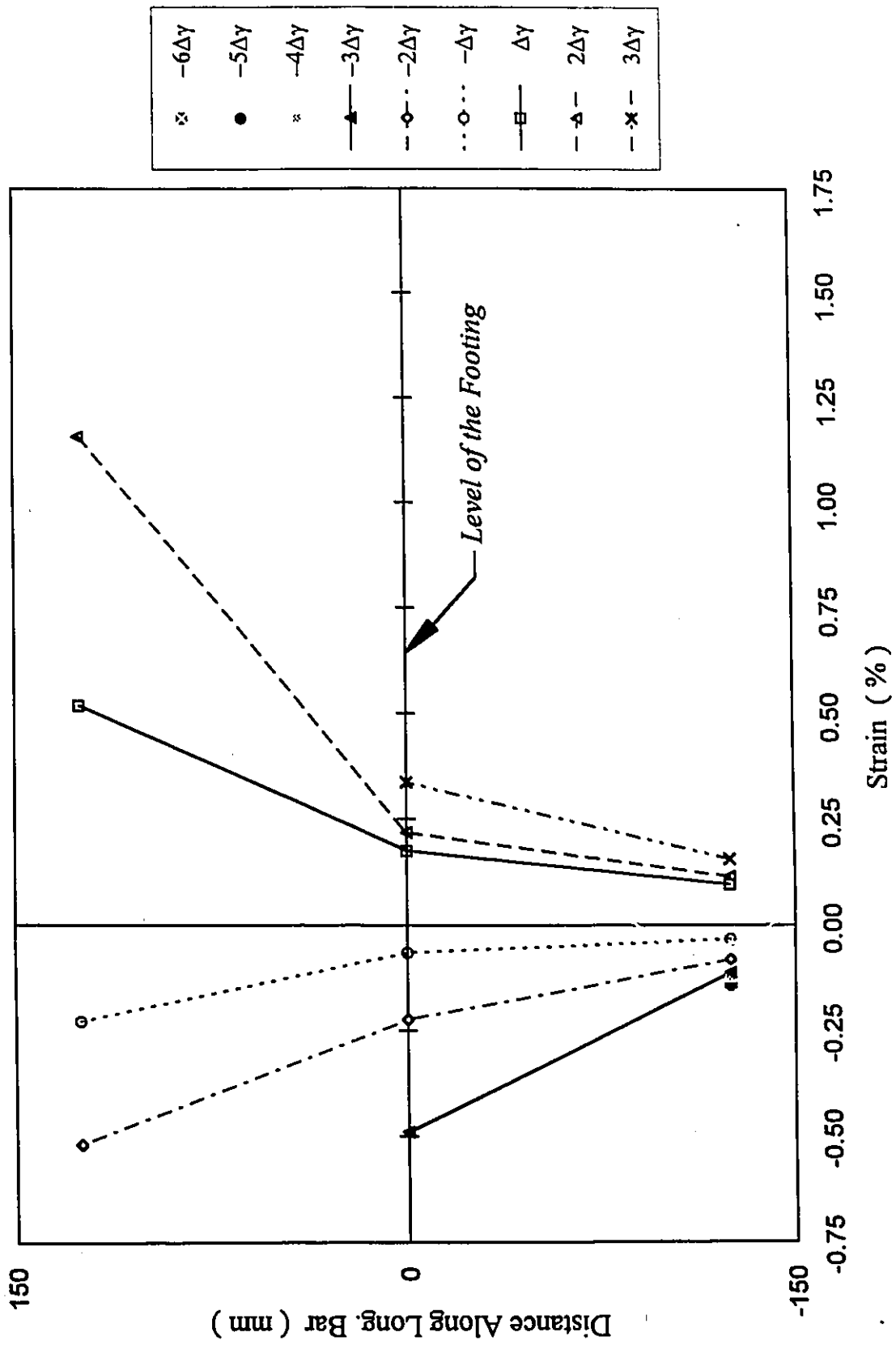


Figure A.35: Strain Distribution in Longitudinal Reinforcement for Different Δy Values
Column RC-9

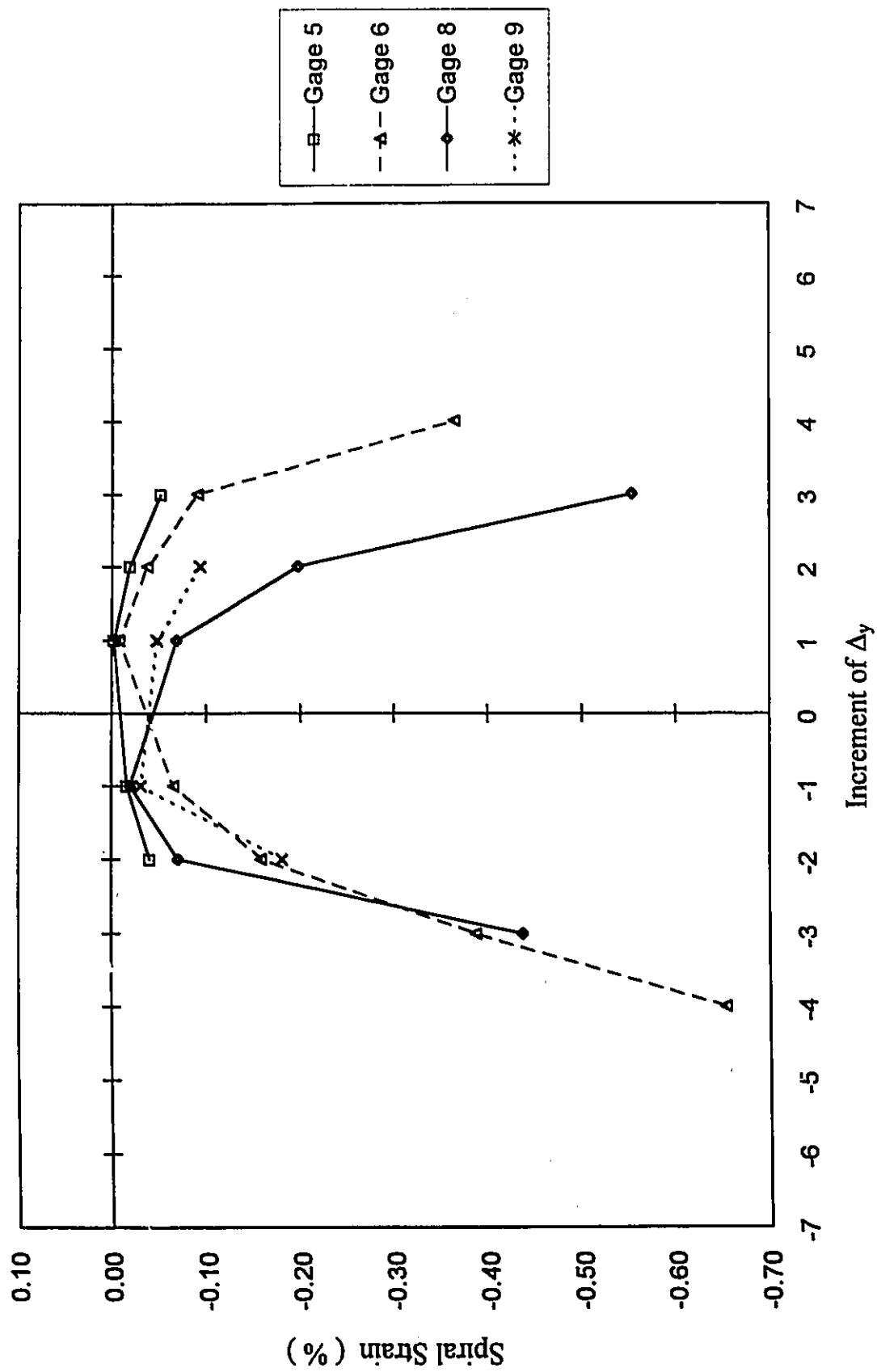


Figure A.36: Strain Distribution in Transverse Reinforcement for Different Δy Increments
Column RC-9

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