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FACULTÉ DE ÉTUDES SUPÉRIEURES  
ET POSTDOCTORALES

FACULTY OF GRADUATE AND  
POSTDOCTORAL STUDIES

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Ph.D.(Civil Engineering)

GRADE - DEGREE

Department of Civil Engineering

FACULTÉ, ÉCOLE, DÉPARTEMENT - FACULTY, SCHOOL, DEPARTMENT

TITRE DE LA THÈSE - TITLE OF THE THESIS

Concrete Columns and Beams Reinforced with FRP Bars and  
Grids under Monotonic and Reversed Cyclic Loading

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# **CONCRETE COLUMNS AND BEAMS REINFORCED WITH FRP BARS AND GRIDS UNDER MONOTONIC AND REVERSED CYCLIC LOADING**

By

**Mohammad-Kazem Sharbatdar**

A Thesis submitted to  
the Faculty of Graduate Studies and Research  
in partial fulfillment of  
the requirement for the degree of

**DOCTOR OF PHILOSOPHY**  
*In Civil Engineering\**

Department of Civil Engineering  
University of Ottawa  
Ottawa, Ontario, Canada

December 2003

\*The Ph.D. in Civil Engineering is a joint program with the  
Carleton University, administrated by the Ottawa-Carleton Institute for Civil Engineering

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## ***In the Name of God the Compassionate and the Merciful***

### **Acknowledgements**

I am thankful to the Almighty God for giving me the opportunity, wisdom, strength, support, guidance, and all blessing upon me to process and undertake this research. I wish to express my sincere appreciation to my thesis supervisor, Professor Murat Saatcioglu, for his guidance, valuable suggestions, encouragement, and support throughout the course of this research.

The scholarship provided by the Ministry of Science, Research, and Technology of I.R. of Iran is gratefully appreciated. Sincere thanks to colleagues and others who helped me in my research. Specifically, the author would like to express his gratitude to Professor Benmorkane of Sherbrooke University and Pultrall Inc. for their cooperation and donation of part of the FRP material used in this research program. Technical support given by the support staff of Machine Shop of the Faculty of Engineering is acknowledged. Special appreciations are extended to Mr. Muslim Majeed, Technical officer of the Structural Laboratory at the University of Ottawa for his assistance and continuous support during the course of this investigation, especially during the construction and testing of specimens.

Most importantly, I wish to extend my highest appreciations to my wife, Shahzad, for her encouragements, support, prayers, patience, sacrifice and assistance through my studies. I can never forget the pain she had to endure during her medical problems while I was doing my research. I wish her a healthy life in the years to come.

I wish the best to my beloved daughters, Nasim and Neshat, whose love gave us strength during difficult periods. The companionship of our fellow graduate students, friends and their families is much appreciated. I also would like to thank my family, my beloved mother, brother, sister and my in-laws for their encouragement, prayers, moral and emotional support.

## **Abstract**

The use of fiber reinforced polymer (FRP) reinforcement is currently being explored by the construction industry as a new and innovative material. A significant force behind this consideration is the superior performance of FRP material in corrosive environments. FRP possesses high strength-to-weight ratio, favorable fatigue strength, and low relaxation characteristics when compared with steel reinforcement, offering economically and structurally sound alternative for most applications. The majority of current FRP applications have been in the area of rehabilitation and strengthening of existing structures, although some attempts have been made for application to new construction.

While the FRP reinforcement has favorable characteristics for use in new construction, some fundamental questions remain to be answered, especially when used in concrete columns. The performance of FRP longitudinal reinforcement in compression, as well as in tension-compression reversals must be explored to assess its suitability to earthquake-resistant construction. Furthermore, the inherent brittleness of FRP reinforcement poses challenges for seismic applications where the dissipation of seismic induced energy is required through inelastic action. The use of FRP as transverse reinforcement poses additional challenges, since shear resistance and concrete confinement provided by such reinforcement have not been well established.

Limited previous research on the use of FRP reinforcement indicated possible application of FRP reinforcement in new concrete columns, although the bars were reported to experience severe reductions in strength when subjected to compression. The bars often maintained their effectiveness until the crushing strain of concrete. They could not maintain their resistance in tension subsequent to significant compression, as would be the case in reversed cyclic loading. FRP grids, on the other hand performed satisfactorily as tie elements to confine concrete. This research involved application of FRP grids as transverse reinforcement and FRP bars as longitudinal reinforcement in beams and columns of new concrete structures.

The research program consisted of four phases; (i) columns under eccentric monotonic loading (ii) reinforced cylinders under concentric compression (iii) columns under constant axial compression and lateral load reversals, and (iv) cantilever beams subjected lateral monotonic or cyclic loading. The results of tests conducted at the Structures Laboratory of the University of Ottawa indicate that CFRP reinforcement can be used effectively in new concrete members. CFRP reinforced columns subjected to eccentric loading were able to develop their expected moment capacities as governed by the crushing of concrete. All columns under cyclic loading sustained a lateral drift level exceeding 2% and 2.5% limits specified for earthquake resistant columns in the National Building Code of Canada (1995, 2005). Failure in these columns was initiated by the spalling of concrete cover, followed by the buckling of FRP bars in compression at approximately 0.45% to 0.60% compressive strain, and subsequent crushing of the core concrete. The failure of FRP bars in compression was due to instability. The stability failure of FRP bars in compression was different than that observed in steel reinforcing bars. The hysteretic relationship of flexure dominant FRP reinforced beams indicated stiffness degradation under cyclic loading due to the progressive cracking of concrete and associated softening in the member, developing approximately 3% lateral drift ratio prior to failure. This level of lateral drift may be considered to be sufficient for earthquake resistant construction. All the flexure dominant members developed their flexural capacities computed on the basis of plane-strain analysis.

A hysteretic model was developed on the basis of experimental observations for dynamic response history analysis of FRP reinforced concrete structures. Design information was also generated for reinforcement development length and shear resistance of FRP reinforced concrete elements.

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## Notations

|                   |                                                                                            |
|-------------------|--------------------------------------------------------------------------------------------|
| $A_c$             | : Core area of column                                                                      |
| $A_f$             | : Total area of transverse legs of each grid reinforcement in CSA 806-02                   |
| $A_{FRP}$ , $A_b$ | : Area of FRP bar computed from its nominal diameter                                       |
| $A'_{FRP}$        | : Gross area of FRP bar including the area occupied by sand coating                        |
| $A_{FH}$          | : Required area of transverse CFRP grid reinforcement for confinement                      |
| $A_{fprv}$        | : Minimum area of FRP shear reinforcement                                                  |
| $A_g$             | : Gross cross-sectional area of column                                                     |
| $A_s$ , $A_v$     | : Cross sectional area of steel lateral reinforcement in the direction of loading          |
| $\sum A_s$        | : Total area of combined transverse reinforcement in both cross-sectional directions       |
| $a$               | : Shear span                                                                               |
| $a = \beta_1 c$   | : Depth of rectangular stress block measured from the extreme compressive fibre            |
| $B$               | : Core dimension of a square column                                                        |
| $b$ , $b_w$       | : Effective web width                                                                      |
| $b_c$             | : Core dimension measured center to center of perimeter tie                                |
| $C_c$             | : Force developed in compression concrete                                                  |
| $C_f$             | : Compressive force developed in FRP bar                                                   |
| $c$               | : Distance from the extreme compression fibre to neutral axis                              |
| $c$               | : Center-to-center distance between longitudinal reinforcement in Sheikh and Uzumeri Model |
| $d$               | : Effective depth of section                                                               |
| $d_{cs}$          | : Distance defined in Equation (6.24) for FRP bar development length                       |
| $d_b$             | : Nominal diameter of FRP bar                                                              |
| $E_c$             | : Modulus of elasticity of concrete                                                        |
| $E_f$ , $E_v$     | : Modulus of elasticity of CFRP grids in tension                                           |

|                 |                                                                                            |
|-----------------|--------------------------------------------------------------------------------------------|
| $E_{fc}$        | : Modulus of elasticity of FRP bar in compression                                          |
| $E_{frp}$ $E_F$ | : Modulus of elasticity of FRP bar in tension                                              |
| $E_s$           | : Modulus of elasticity of steel                                                           |
| $f_c$           | : Compressive stress in concrete                                                           |
| $f_{cc}$        | : Confined concrete stress                                                                 |
| $f'_c$          | : Cylinder compressive strength of concrete                                                |
| $f_c''$         | : Compressive strength of concrete in Kent and Park model                                  |
| $f'_{cc}$       | : Confined concrete strength                                                               |
| $f'_{co}$       | : Unconfined strength of concrete                                                          |
| $f_F$           | : Design stress in FRP tension reinforcement at ultimate limit state                       |
| $f_{fb}$        | : Failure strength at stirrup bends                                                        |
| $f_{Fh}$        | : Maximum tensile strength of FRP transverse reinforcement                                 |
| $f_{frp}$       | : Stress in FRP bars at failure                                                            |
| $f_{frpu}$      | : Ultimate tensile strength of FRP bars                                                    |
| $f_{fu}$        | : Maximum tensile strength of FRP bars in ACI 440-01                                       |
| $f_{F.U}$       | : Ultimate strength of FRP shear reinforcement.                                            |
| $f_l$           | : Lateral confinement pressure in Saatcioglu and Razvi Model                               |
| $f_{le}$        | : Equivalent lateral pressure                                                              |
| $f_{fv}$        | : Stress in FRP shear reinforcement                                                        |
| $f'_s$          | : Steel stress at peak concrete strength                                                   |
| $f_{st}$        | : Stress in transverse reinforcement at peak concrete stress in Saatcioglu and Razvi Model |
| $f_u$           | : Design tensile strength of FRP bars in ISIS draft                                        |
| $f_{yh}$        | : Transverse steel reinforcement                                                           |
| $h_c$           | : Cross-sectional dimension of column core in the direction of loading                     |
| $h''$           | : Core dimension in Kent and Park Model                                                    |
| $K$             | : Strength enhancement coefficient in Kent-Park and Saatcioglu-Razv Models                 |

|                           |                                                                                      |
|---------------------------|--------------------------------------------------------------------------------------|
| $K_c$                     | : Coefficient in Equation (4.1) for rectangular grids                                |
| $K_s$                     | : Strength enhancement factor in Sheikh and Uzumeri Model                            |
| $K_U$                     | : Correction coefficient defined in Equation (1.13)                                  |
| $K_p'$                    | : Equivalent correction coefficient in Equation (1.13)                               |
| $k_1$                     | : Variable in Saatcioglu and Razvi Model                                             |
| $k_2$                     | : Confinement efficiency coefficient in Saatcioglu and Razvi Model                   |
| $k_1, k_2, k_3, k_4, k_5$ | : Modification factors for development length of FRP bars in tension                 |
| $k_1$                     | : Bar location factor                                                                |
| $k_2$                     | : Concrete density factor                                                            |
| $k_3$                     | : Bar size factor                                                                    |
| $k_4$                     | : Bar fibre factor                                                                   |
| $k_5$                     | : Bar surface profile factor                                                         |
| $l_d$                     | : Development length of FRP bars in tension                                          |
| $l_{bf}$                  | : Basic development length of FRP bars in tension                                    |
| $M_f$                     | : Factored moment resistance calculated at ultimate limit state                      |
| $N_c$                     | : Concentric column capacity                                                         |
| $N_u$                     | : Ultimate applied axial load                                                        |
| $n$                       | : Number of parabolas that define unconfined concrete in Sheikh and Uzumeri Model    |
| $n_1$                     | : Number of bars in reinforcement arrangement                                        |
| $P_f$                     | : Factored applied axial load with zero end eccentricity (applied concentric load)   |
| $P_{ro}$                  | : Factored axial resistance under zero end eccentricity (concentric load resistance) |
| $P_o, P_n$                | : Column concentric capacity in compression                                          |
| $P_{oc}$                  | : Core capacity of concrete under concentric compression                             |
| $s$                       | : Spacing of shear reinforcement                                                     |
| $s_l$                     | : Spacing of transverse legs or crossties within the cross-sectional plane           |

|                     |                                                                                                     |
|---------------------|-----------------------------------------------------------------------------------------------------|
| $T_f$               | : Tensile force developed in FRP bar                                                                |
| $T_v$               | : Tensile strength of shear reinforcement at bend section                                           |
| $u_f$               | : Bond stress between the FRP bars and concrete                                                     |
| $V_c$               | : Diagonal tension capacity of concrete in reinforced concrete                                      |
| $V_{cf}$            | : Concrete shear capacity of FRP reinforced flexural members                                        |
| $V_f$               | : Factored shear force at ultimate limit state.                                                     |
| $V_r$               | : Factored shear resistance                                                                         |
| $V_s$               | : Contribution of steel stirrups to shear resistance                                                |
| $V_{SF}$            | : Factored shear resistance provided by FRP shear reinforcement                                     |
| $V_{u1}$            | : Ultimate shear strength corresponding to the rupturing of shear reinforcement in diagonal tension |
| $V_{u2}$            | : Ultimate shear strength corresponding to compression crushing of concrete in diagonal compression |
| $Z_m$               | : Coefficient defining the descending slope in Kent and Park Model                                  |
| $\alpha$            | : Angle between reinforcement and member axis                                                       |
| $\alpha_1$          | : Coefficient for rectangular stress block defining the width                                       |
| $\beta_1$           | : Coefficient for rectangular stress block defining the depth                                       |
| $\delta$            | : Design lateral drift ratio                                                                        |
| $\Delta_y$          | : Displacement at the end of the elastic response,                                                  |
| $\Delta_u$          | : Displacement corresponding to ultimate failure moment                                             |
| $\varepsilon$       | : Strain in concrete, used in Saatcioglu and Razvi Model                                            |
| $\varepsilon_0$     | : Strain at peak stress of concrete                                                                 |
| $\varepsilon_{01}$  | : Strain at peak stress of unconfined concrete                                                      |
| $\varepsilon_{085}$ | : Strain at 85% of strength beyond the peak of unconfined concrete                                  |
| $\varepsilon_1$     | : Strain at peak stress of confined concrete in Saatcioglu and Razvi Model                          |

|                       |                                                                                             |
|-----------------------|---------------------------------------------------------------------------------------------|
| $\varepsilon_{85}$    | : Strain at 85% of strength beyond the peak of confined concrete, Saatcioglu Model          |
| $\varepsilon_c$       | : Strain at extreme compressive fibre                                                       |
| $\varepsilon_{c,frp}$ | : Buckling strain of FRP bars                                                               |
| $\varepsilon_{cu}$    | : Ultimate strain in concrete in compression                                                |
| $\varepsilon_F$       | : Recorded tensile strain in FRP grids at or near diagonal crack                            |
| $\varepsilon_{frpu}$  | : Ultimate strain in FRP bars in tension in extreme layer                                   |
| $\varepsilon'_F$      | : Compressive strain in FRP reinforcement                                                   |
| $\varepsilon_{s1}$    | : Strain at the beginning of peak stress of confined concrete, Sheikh and Uzumeri Model     |
| $\varepsilon_{s2}$    | : Strain at the end of peak stress of confined concrete, Sheikh and Uzumeri Model           |
| $\varepsilon_{sf}$    | : Strain in shear reinforcement                                                             |
| $\varepsilon_t$       | : Tensile strains in FRP reinforcement                                                      |
| $\phi_c$              | : Capacity reduction factor for concrete.                                                   |
| $\phi_F$              | : Capacity reduction factor for FRP reinforcement                                           |
| $\lambda$             | : Modification factor for concrete mass density.                                            |
| $\mu_\Delta$          | : Displacement ductility ratio                                                              |
| $\mu_\phi$            | : Curvature ductility ratio                                                                 |
| $\rho_{frp}$          | : Reinforcement ratio ( $\rho_{frp} = \frac{A_{frp}}{b_w d}$ )                              |
| $\rho_w$              | : Tension reinforcement ratio of steel bars                                                 |
| $\rho_w$              | : Ratio of shear reinforcement ( $\frac{A_v}{b_w s}$ )                                      |
| $\rho'_w$             | : Equivalent shear reinforcement ratio $\{ (\frac{E_v}{E_s}) \rho_w \}$                     |
| $\rho_t$              | : Longitudinal reinforcement ratio                                                          |
| $\rho_s$              | : Volumetric ratio of transverse reinforcement in confined concrete models                  |
| $\rho_{s,max}$        | : Maximum steel longitudinal reinforcement ratio ( $\rho_{frp} = \frac{A_{s,max}}{b_w d}$ ) |

# *Chapter 1*

## **Introduction and Previous Research**

### **1.1 Introduction**

#### **1.1.1 General**

The use of fiber reinforced polymer (FRP) reinforcement in new concrete construction is currently being explored as replacement for steel reinforcement. The major driving force behind this effort is the superior performance of FRPs in corrosive environments. FRPs possess high strength-to-weight ratio, favorable fatigue strength, and low relaxation characteristics when compared with steel reinforcement, offering economically and structurally sound alternative in most applications. They also have high electro-magnetic transparency. The low weight (density) of the material is an asset in cage assembly and placement, while its sensitivity to bending makes it vulnerable to premature failures during construction handling. An important feature for structural applications is its high strength, while exhibiting linearly elastic stress-strain characteristics without any ductility.

FRPs may be manufactured from carbon, glass or aramid fibers. They have circular or square cross-sections, with or without surface roughness or deformations. Carbon fiber reinforced polymer (CFRP) is more expensive than glass fiber reinforced polymer (GFRP) and aramid fiber reinforced polymer (AFRP), but has higher strength and stiffness characteristics. The elastic modulus of CFRP reinforcement can approach and even exceed that of steel, and may be considered to be suitable for use in concrete structures. The elastic modulus of GFRP bars can be as low as 1/6 of the value of CFRP. GFRPs may suffer from adverse effects of the alkalinity of cement in concrete. On the other hand GFRPs have larger elongation at failure.

Different CFRP bars have different physical properties. The tensile strength of some CFRP bars can reach 2450 MPa, with an elastic modulus of 150 GPa. The compression strength,

based on limited previous research, is usually assessed to be approximately 30-50% of its tensile strength. The maximum tensile (rupturing) strain varies between 1% and 3% (average values ranging between 1.5% to 2%). The weight to stiffness ratio is approximately 1/5 the value for conventional steel reinforcement.

The majority of current FRP applications have been in the area of rehabilitation and strengthening of existing structures, utilizing FRP sheets, although some attempts have been made to use them in new construction. A few pilot projects have been implemented in actual practice involving the use of FRP re-bars and grids in concrete beams, bridge decks and slabs. FRP bars and grids have not been used in practice as column reinforcement.

While FRP reinforcement has many favorable characteristics for use in new construction, some fundamental questions remain to be answered. The performance of FRP as longitudinal column reinforcement has not been established, especial in compression. As well, the behaviour of bars in tension-compression reversals, as in the case of reinforcement in earthquake resistant elements, has not yet been explored. Furthermore, the inherent brittleness of FRP reinforcement poses challenges for seismic applications from the point of view of energy dissipation through plastic hinges. The use of FRP transverse reinforcement presents additional challenges, as shear resistance and concrete confinement characteristics of such reinforcement have not been well established. Only limited research has been conducted in Japan and North America on the use of FRP reinforcement in columns, both as longitudinal and transverse reinforcement.

### **1.1.2 Research Needs**

The corrosion of steel reinforcement in concrete and the resulting deterioration of structures prompted research on FRPs as potential reinforcement for concrete. The results of previous research indicate that FRP reinforcement may be used in new concrete structural elements, although the bars experience severe reduction in strength under compression. Furthermore, the FRP reinforcement that has failed in compression cannot maintain its tensile resistance under reversed cyclic loading. On the other hand, experimental research on FRP grids used as



column ties indicates that the grids can be employed as tie reinforcement for concrete confinement (Saatcioglu and Grira, 2001).

There is limited research on the behaviour of concrete structural elements reinforced with FRP re-bars. Experimental research is needed to verify the applicability of newly developed FRP bars under different stress conditions. In particular, columns under monotonically increasing compression, and reversed cyclic loading need to be investigated establish their behaviour. Furthermore, analytical research is needed to develop reliable mathematical models for FRP materials as well as design and detailing techniques for structural elements. Specifically, there is need to research the following areas:

- Behaviour of FRP bars in concrete when subjected to tension, compression, and tension-compression cycles.
- Assessment of seismic performance of concrete members reinforced with FRP reinforcement.
- Behaviour of FRP reinforced concrete columns and beams and their deformability within the inelastic range of concrete.
- Development of some design guidelines for concrete columns and beams reinforced with FRP reinforcement.
- Development of reliable material models for FRP reinforcement and concrete confined by such reinforcement, as well as hysteretic models for FRP reinforced concrete elements under reversed cyclic loading.

### **1.1.3 Objective**

The primary objective of the proposed research is to investigate the behaviour of FRP reinforced concrete structural elements and to develop design and detailing requirements for FRP reinforced concrete beams and columns under gravity and seismic loading. The objective also includes generation of test data on FRP reinforced structural elements, while developing behavioral material and element models.

#### **1.1.4 Scope**

The objective of proposed research is realized within the following scope:

- Review of previous research on behaviour of concrete elements reinforced with FRP reinforcement.
- Investigation of the behaviour of FRP reinforcement and concrete confined by such reinforcement.
- Design, construction, instrumentation, and testing of 21 large-scale FRP reinforced concrete structural members. The specimens include 5 large-scale FRP reinforced concrete columns under eccentric loading, 10 large-scale FRP reinforced concrete columns under constant axial compression and lateral deformation reversals, and 6 large-scale FRP reinforced concrete beams under simulated seismic loading.
- Evaluation and interpretation of test data with respect to specific design and detailing parameters.
- Development of an analysis technique for FRP reinforced concrete beams and columns subjected to gravity and seismic loads.
- Investigation of analytical relationships for stress-strain relationships of FRP reinforcement, and concrete confined by such reinforcement.
- Development of a mathematical model for hysteretic behaviour of FRP reinforced members under reversed cyclic loading.
- Development of shear and flexural design procedures for FRP reinforced concrete elements under bending, shear, and axial load.

## **1.2 Previous Research**

### **1.2.1 General**

The use of FRP reinforcement in new concrete construction is a new concept that is currently being researched. Therefore, previous research in the area is limited. The following sections provide a summary of available research on the application of FRP longitudinal bars and pre-shaped FRP ties/stirrups or grids. Of the experimental work reported in available literature on

the topic, the majority involves simple beams under monotonic point loads and columns under monotonic concentric or eccentric loading. Very little research has been conducted on the behaviour of FRP reinforced concrete columns subjected to lateral load reversals.

Composite Fiber Plastic materials were introduced to the concrete industry in early fifties. Rubinsky and Rubinsky (1954) used fiberglass in pre-stressed concrete structural elements. Since several FRP products have become commercially available in the market after the introduction of new manufacturing procedures, FRP reinforcement products often appear in the form of cables, bars, planar (2-D) grids, three-dimensional (3-D) cages, sheet materials, plates and strips.

The following sections provide an overview of previous research, discussed under the headings of; i) transverse FRP reinforcement, ii) longitudinal FRP reinforcement, and iii) design recommendations.

### **1.2.2 Transverse FRP Reinforcement**

The behaviour of FRP transverse reinforcement has been investigated by studying; i) members reinforced with steel longitudinal bars tied with FRP transverse reinforcement, ii) externally applied FRP transverse reinforcement on steel reinforced concrete members, and iii) members with FRP longitudinal and FRP transverse reinforcement.

Nagasaka and Fukuyama (1993) carried out several tests on simply supported beams reinforced with steel longitudinal bars and FRP stirrups. The beams failed either by the failure of FRP stirrups at 90-degree bends or crushing of diagonal concrete struts. It was reported that the tensile strength of rectangular FRP stirrups formed curved sections is smaller than that of straight bars, probably because of stresses induced during the manufacturing process or due to the stress concentration effect. The tensile strength was observed to decrease to 30 to 80% of the strength of straight FRP elements. It was concluded that the specimens failed due to crushing of the concrete had more ductile behaviour than those specimens failed due to stirrup rupturing, this stiffness reduction was not dependent on the type of FRP stirrup so much as it dependent on the amount of stirrups. The increase in

stirrup ratio,  $\rho_w$  improved the stiffness reduction by controlling diagonal cracking. Also, the ultimate shear capacity increased approximately linearly with  $\sqrt{\rho_w E_w}$  and the shear capacity was affected by the axial rigidity of stirrups.

Grira and Saatcioglu (2000) designed, constructed and tested full-size columns confined by NEFMAC-CFRP grids. The grids were made by impregnating continuous carbon fibers with vinyl ester resin. The columns had a 350 mm square cross-section and a 1645 mm shear span. They were reinforced with 8 or 12 #20 (19.5 mm diameter) steel bars as longitudinal reinforcement and CFRP grids as transverse reinforcement, having either 4-cells or 9-cells. The columns were tested under simulated seismic loading, and exhibited ductile response when properly confined by the grids. The failure of these columns was caused mostly by crushing of compression concrete. In few columns the FRP grids failed at their joints, only after fulfilling their functions as confinement reinforcement. The ultimate tensile strength and modulus of elasticity of FRP grids were 1480 MPa and 83000 MPa, respectively. The columns were confined following the spacing requirements of ACI 318-95. They showed improved inelastic deformability of up to 5% to 7% lateral drift ratio prior to experiencing 20% degradation in moment resistance. The experimental investigation indicated that FRP grids could be used as column confinement reinforcement for improved seismic performance. The grids were reported to offer ease and speed in construction, savings in materials, and improved performance.

Nagasaka et al. (1993) tested thirty-five half-scale concrete beams under monotonically increased shear force, reinforced for shear with rectangular FRP spirals or closed hoops. They reported that the ultimate shear capacity of beams was governed by either the tensile rupturing of stirrups or the crushing of diagonal compression struts. When diagonal tension governed the behaviour, the shear capacity depended on the amount and type of FRP used.

Taniguchi and Mutsuyoshi (1993) investigated the confining effects of spiral FRP reinforcement on square and circular specimens. The specimens either had a 150 mm square or a 150 mm circular cross-section. All the specimens were of 300 mm height. The transverse FRP reinforcement was manufactured from carbon fibers resulting in either 5 mm diameter strands or 5.3 mm diameter bars. It was observed that the rate of strength decay in specimens

confined with circular reinforcement was smaller than that of specimens confined with rectangular reinforcement. Also, the rate of strength decay beyond yielding was smaller in confined specimens with lower strength concrete. Steel and FRP provided the same level of confinement despite differences in their characteristics under dynamic loading. It was also concluded that FRP transverse reinforcement was effective as confinement reinforcement when used with sufficiently small pitch. This finding led to testing of an entire frame, consisting of FRP reinforced elements confined with FRP transverse reinforcement. During testing the initial cracks were observed near the column base at 0.5% lateral drift. These cracks were essentially flexural cracks and progressively increased within the potential hinging region up to 2% drift ratio. At this deformation level, the column cover concrete started to crush. The overall resistance of frame continued to increase up to 5% lateral drift. It was concluded that FRP reinforced frames may be designed and built without much difficulty.

Some researchers investigated the effect of longitudinal bars on shear capacity. Tottori and Wakui (1993) mentioned that the contribution of longitudinal FRP bars to shear capacity of beams was about 70% of that of equal diameter steel reinforcement.

Alsayed and Al-Sallaoom (1997) tested 21 full-size concrete beams designed to fail in shear. The beams either had steel or GFRP reinforcement, used either longitudinally or transversely, or in combination. The researchers proposed modifications to the current ACI 318(2002) approach for shear design. The formula suggested for beams with longitudinal and transverse GFRP bars was as follows:

$$V_n = \frac{V_c}{2} + \frac{V_F}{2} \quad (1.1)$$

where the concrete contribution was;

$$V_C = \frac{1}{6} \sqrt{f_c'} b d \quad (1.2)$$

and the FRP grid contribution was;

$$V_F = A_F f_F \frac{d}{s} \quad (1.3)$$

In the above expressions  $f'_c$ , specified compressive strength of concrete, and  $f_{fr}$ , tensile strength of FRP transversare reinforcement, to be specified in MPa;  $b$  and  $d$  in mm; and  $V_c$  and  $V_{fr}$  in Newtons. Parameters of  $d$  and  $s$  are respectively effective depth of section and spacing of grids. The presence of GFRP bars as longitudinal reinforcement reduced the contribution of concrete by 50%, while the use of GFRP transverse reinforcement reduced the contributions of stirrups by 50%, relative to steel reinforced members.

Zhao and Maruyama (1995) tested beams reinforced with longitudinal and transverse FRP bars. Their results indicated that the longitudinal reinforcement ratio,  $\rho_l$  did not have an appreciable influence on shear capacity and the results were consistent with those predicted by current codes.

Sonobe and Kanakubo (1995) tested concrete specimens reinforced with stirrups and diagonal FRP bars. It was reported that the shear strength increased with the amount of diagonal reinforcement. In their test specimens both longitudinal and diagonal bars carried approximately the same level of stress when the shear strength was attained.

Razaqpur and Mostofinejad (1999) conducted an experimental study to investigate the shear behavior of continuous concrete beams reinforced with FRP reinforcement. They tested four 6.5m long two span continuous beams, reinforced with longitudinal FRP bars and transverse reinforcement consisting of either steel stirrups or CFRP grids. The modulus of elasticity of CFRP grids was approximately 50% of that of steel stirrups. The shear reinforcement ratio  $A_s/s$  for steel and FRP were 2.0 and 1.75 mm, respectively. The specimens were monotonically loaded at the center of each span until failure. The researchers reported that the beams failed in flexure. CFRP grids performed as well as steel stirrups, developing tensile strains of up to 0.4%.

### **1.2.3 Longitudinal FRP Reinforcement**

The majority of FRP reinforcement in previous research was in the form of longitudinal bars in simple or continuous flexure dominant beams. Only limited research was conducted on

columns with FRP longitudinal bars. These research programs included column tests under monotonic loading.

Kawai et al. (1991) and Nakano et al. (1993) tested simply supported concrete beams reinforced with FRP longitudinal and transverse reinforcement. Different parameters were considered in tests, including; i) type of FRP reinforcement, ii) ratio of FRP longitudinal reinforcement, and iii) type of concrete. The researchers computed sectional moment-curvature relationships using plane-section analysis and concluded that the analysis produced reasonably accurate predictions of bending characteristics of beams. They also concluded that the conventional methods used for flexural analysis of steel-reinforced concrete are applicable to FRP reinforced concrete beams, so long as an allowance is made for FRP material properties.

Weichen (2001) conducted 9 tests to investigate the performance of concrete beams reinforced with FRP bars. Simple beams with 1.8 m span were cast with tensile and compressive FRP bars. Based on the results of 9 concrete beam tests he concluded that the flexural capacity, crack characteristics and deflection of FRP reinforced members could be calculated in the same manner as steel-reinforced concrete structures, so long as the yield strength of reinforcement is replaced by a rational value for FRP strength.

Salib et al. (2001) presented a comprehensive analytical model to evaluate the strength of concrete beams reinforced with FRP bars and the corresponding modes of failure, including the premature failure of beam due to dowel failure. The model takes into account parameters such as the crack path geometry, the crack width, and properties of FRP bars in both longitudinal and transverse directions. The contribution of stirrups is also obtained based on the number and tensile strain of stirrups crossing the failure crack. They concluded that the predictions were in good agreement with experimental results. The researchers also indicated that the current design guidelines for FRP-reinforced structures over-estimate beam strength significantly since they disregard the dowel failure and the contribution of shear reinforcement.

Meng et al. (2001) presented a method to determine upper and lower limits of reinforcement ratio for CFRP reinforced concrete beams. They also employed numerical simulations to compare results. The lower limit was found based on the assumption that compression concrete crushes prior to FRP failure. This implies that the reinforcement ratio  $\rho$  of CFRP bars should be  $\rho \geq \rho_{\min} = \rho_b$ . Considering the available ductility of concrete and the high price of CFRP bars, the upper limit of reinforcement ratio is determined such that the failure occurs in CFRP bars. The researchers concluded that the section dimension has little influence on the reinforcement ratio and the ratio depends mainly on concrete strength. Their recommendation for the maximum ratio, as obtained from regression analysis, is shown below:

$$\rho_{\max} (\%) = 0.08 f_c' (MPa) \quad (1.4)$$

The performance of FRP bars in compression has been investigated by a number of researchers experimentally. Wu (1990) reported that the compressive strength of FRP was lower than its tensile strength. Accordingly, the compressive strength of GFRP, CFRP and AFRP bars were 55%, 78% and 20% of their tensile strengths, respectively. Kobayashi and Fujisaki (1995) reported similar reductions in strength. These researchers conducted material tests on FRP bars, as well as FRP reinforced column tests under monotonic and reversed cyclic loading. The bars were made out of carbon, aramid and glass fibers. The material tests were conducted on FRP bars subjected to externally applied compression while the bars were embedded in concrete. They reported that the compressive capacity of carbon, aramid and glass FRP bars were 30 to 50%, 10% and 30 to 40% of their tensile capacities, respectively. The same bars were subjected to incrementally increasing axial tension-compression reversals. They were subjected to either 10 or 30 cycles at each load level. AFRP and GFRP bars were affected significantly by cyclic loading, showing a reduction of 20 to 50% in their monotonic compressive capacities. CFRP bars did not show any significant reduction of the cyclic load. In all cases the bars failed in compression and the tension capacity was not affected by reversing load. The researchers also tested small-scale concrete columns having 200 mm square cross-section and 650 mm height under uniaxial compression. The columns were reinforced with CFRP grids. The hoops consisted of glass, aramid or carbon FRP. The



results indicated that the strain compatibility was maintained up to the crushing of concrete. The columns developed their concentric capacities at about 0.25 to 0.4% compressive strains. The FRP continued straining up to 1.0% to 1.8% in compression, beyond the peak column load, before they failed in compression. It was suggested that the concentric capacity of FRP reinforced concrete columns could be computed conservatively by ignoring the contribution of FRP and using the gross area of column section multiplied by 85% of concrete strength. The recorded experimental strength values showed a scatter of test data between this recommended analytical value and 120% of the same value. The researchers also indicated that under strain gradient, plane section analysis could be employed with failure governed by concrete crushing. The failure of FRP in the compression zone was reported not to influence the column behaviour, however those that suffered compression failure were not expected to resist subsequent tension under reversed cyclic loading.

Paramanantham and Daniali (1993) investigated the behaviour of FRP reinforced concrete columns under concentric and eccentric loading, by testing small-scale specimens. Longitudinal and tie reinforcement were produced by pultrusion and were made of vinyl ester resin and E-glass fibers. The longitudinal bars had an area of  $200 \text{ mm}^2$ , whereas the ties had  $70 \text{ mm}^2$  cross-sectional areas. The tensile strength of FRP reinforcement varied between 550 MPa and 590 MPa, with corresponding elastic modulus of 38000 MPa and 41000 MPa, respectively. A total of 15 FRP reinforced concrete blocks were tested to investigate the concentric capacity of specimens as affected by tie spacing and bar buckling. The tie spacing used was 100 mm, 150 mm, and 200 mm. It was concluded that the concentric capacity of specimens was very similar to each other, though those with 100 mm spacing showed better control of bar buckling while developing slightly improved strength. The FRP bars were able to contribute to the load carrying capacity by developing stresses corresponding to 0.003 strains. This constituted about 20% to 30% of the ultimate strength of FRP. Additional 16 columns were tested under eccentric loading. The columns under eccentric loading were subjected to different axial force and bending moment combinations. They all failed in a compression crushing mode. Column behaviour was characterized on the basis of the ratio of slopes of moment-deflection relationships. The relationships were first idealized as curves having three segments. The first two segments were linear, and the last segment was non-

linear. The slopes of the first and second segments were defined as  $S_1$  and  $S_2$ , respectively. When the ratio of  $S_1/S_2$  varied between 1.25 and 1.85, the failure was defined as compression failure. When the ratio varied between 2.22 and 2.37, the failure was defined as compression with tensile cracking. When the ratio increased to vary between 5.5 and 6.0, the failure was defined as compression-flexure failure. The researchers also concluded that a tie spacing of 100 mm produced the best results. The maximum tensile stress measured in FRP bars was about 70% of their ultimate tensile strength in direct tension. The experimental results were compared with a moment axial force interaction diagram derived analytically on the basis of the plane section analysis. The comparison showed that the experimental strength values were consistently higher than those computed analytically, except for four columns, which showed slightly lower capacities. It was also shown that the compression-controlled portion (i.e., above balanced load) of interaction diagrams was similar for both FRP reinforced and steel reinforced columns, though the tension governing portions were different.

A comparative study was conducted by Alsayed et. al (1999) between steel and FRP reinforced concrete columns. The researchers tested 450 mm by 150 mm rectangular columns with 1200 mm column height under concentric compression. A total of 15 columns were tested with different combinations of steel and GFRP reinforcement, including all GFRP reinforced columns. The longitudinal FRP reinforcement consisted of 15.7 mm diameter re-bars and the FRP ties consisted of 6.35 mm diameter bars. The results indicated that the FRP reinforcement resulted in approximately 10% reduction in concentric column capacity as compared to the steel reinforcement. The analytical expression commonly used to compute concentric capacities of columns with steel reinforcement was found to produce reasonably good values for FRP reinforced columns when the effect of longitudinal FRP reinforcement was accounted for at the crushing strain level of concrete.

Fukuyama and Masuda (1995) designed, built and tested a half scale model of a three-story FRP reinforced concrete frame. The frame was subjected to lateral load reversals at the third story mid-height level. The frame had two 3.5 m spans and 1.8 m story height. The main reinforcement in beams, columns, and slabs were braided aramid-fiber bars with 67000 MPa elastic modulus. The bars were coated with silica sand to improve their bond characteristics. The columns had a 350 mm square cross-section with 20 longitudinal bars, each having 90

mm<sup>2</sup> areas. The beams had a 200 mm x 400mm cross-section with total 18 longitudinal bars, each 90 mm<sup>2</sup> area. Initial cracking was observed near the column base at 0.5% lateral drift. These cracks were essentially flexural cracks, and progressively increased within the potential hinging region, up to 2% drift ratio. At this deformation level, the column concrete started to crush. The overall resistance of frame continued to increase up to 5% lateral drift. It showed that the frame remained elastic and had a stable hysteretic curve. Deformation at the time of rupture of main reinforcement in the beam was 2.2 (sometimes 1.5) times the value at concrete crushing. Residual deformations in FRP reinforced concrete frame were small. Therefore, its rehabilitation was easy. It was reported that lateral drift under ultimate limit-state and maximum crack width under serviceability limit-state controlled the design. It was concluded that FRP reinforced frames may be designed and built without much difficulty, but additional research was needed to establish many aspects of these structures. It was mentioned that earthquake-resistant concrete structures reinforced with FRP bars are feasible, but it would be better to use densely populated small diameter bars.

Researchers in the past have also concentrated on investigating the use of FRP grids as longitudinal slab reinforcement. Grid shaped FRP reinforcement has been developed to prevent deterioration of concrete structures caused by corrosion. The 2-D grids were designed to control surface cracks in slabs and pavements. NEFMAC<sup>TM</sup> reinforcement grids were used in experimental slabs in practice. Rahman and Taylor (1992) used the Finite Element Method to estimate the service load deflections of a slab reinforced with FRP grids. Schmeckpeper (1992) carried out one-way strips of slab, reinforced with FRP or steel rebars. Rahman et al. (1993) investigated the suitability of NEFMAC<sup>TM</sup> grids for reinforcing concrete elements such as flat slabs. NEFMAC<sup>TM</sup> grids were also used by Banthia et al. (1995) to test three small-size concrete slabs. The same grids were also used in structural walls. Fujisaki and Kobayashi (1988) built and tested three concrete shear walls reinforced with NEFMAC<sup>TM</sup> grids.

#### **1.2.4 Design Recommendations**

Complete design provisions for FRP reinforced concrete structures have not yet been developed because of lack of sufficient research data. The variability of FRP rebar properties

and lack of design information have discouraged many designers from using FRP reinforcement in practice. Some papers, particularly those written by Japanese researchers recommended design expressions for shear and flexure. It was also indicated that the design expressions proposed for shear capacity of steel reinforced beams were also applicable to FRP reinforced elements. The previous publications consistently indicated that more research was needed to develop standardized design provisions.

A review of design recommendations and design expressions proposed by various researchers for shear, flexure, and axial compression are presented below. Also included are available provisions of existing codes and standards on FRP reinforced concrete structures.

#### **1.2.4.1 Recommended Design Provisions for Flexural Capacity**

Previous research indicated that the majority of design code provisions for steel reinforced concrete are also applicable to FRP-reinforced concrete, so long as appropriate allowable stresses are used for tension and compression capacities of FRP reinforcement. Japanese Design Guideline of FRP Reinforced Concrete (1997) recommends the following assumptions for the computation of sectional forces within the cross section:

1. The strain in FRP reinforcement and concrete across the depth of cross section are proportional to the distance from the neutral axis.
2. The relationship between the stress and strain of FRP bars remains elastic under tension. Resistance of FRP bars in compression is ignored.
3. Nonlinearity of compressive stress-strain relationship of concrete is taken into consideration as appropriate. A rectangular stress distribution may be assumed for compression concrete, such as that recommended by ACI 318 (2002) for steel-reinforced concrete.
4. The area of FRP reinforcement is assumed to be concentrated at the centroid of the cross section of each FRP reinforcement.

Canadian Standards Association S806-02 (2002) proposes a similar method of flexural design as that recommended in the Japanese Standard. The ultimate strain in extreme

concrete compression fibre is assumed to be 0.0035. The compressive stress of concrete,  $\alpha_1 \phi_c f'_c$ , is assumed to be uniformly distributed as a rectangular stress block within a distance  $a = \beta_1 c$ , measured from the maximum compressive fiber, where  $c$  is distance from the extreme compression fibre to neutral axis. The coefficients  $\alpha_1$  and  $\beta_1$  can be computed from the following expressions;

$$\begin{aligned}\alpha_1 &= 0.85 - 0.0015 f'_c \geq 0.67 \\ \beta_1 &= 0.97 - 0.0025 f'_c \geq 0.67\end{aligned}\tag{1.5}$$

The Canadian Network of Centres of Excellence on Intelligent Sensing for Innovation Structures- ISIS Canada, ISIS-M04-00 Draft, 2000, recommended strain compatibility analysis for the analysis of FRP-reinforced members identical to Japanese and Canadian Design codes. Three possible modes of flexural failure have been considered for a concrete section reinforced with FRP bars; (i) Balanced failure in which rupture of FRP and crushing of concrete occurring simultaneously (ii) Compression failure in which concrete crushes while the FRP remains in the elastic range with a strain smaller than ultimate strain, and (iii) Tension failure in which the rupture FRP bars occur prior to crushing of concrete. The reinforcement ratios were presented for three different cases with recommending some expressions for two parameters  $\alpha_1$  and  $\beta_1$  as given in S806-02 (2002). The following equation expresses the balanced failure reinforcement ratio:

$$\rho_{frpb} = \alpha_1 \beta_1 \frac{f'_c}{f_{frpu}} \left( \frac{\epsilon_{cu}}{\epsilon_{cu} + \epsilon_{frpu}} \right)\tag{1.6}$$

Where  $f_{frpu}$  = ultimate tensile strength of FRP, MPa

$\epsilon_{cu}$  = ultimate strain in concrete in compression, 0.0035

$\epsilon_{frpu}$  = ultimate strain in FRP in tension in extreme layer

Whenever the section is over-reinforced in only one layer, crushing of concrete occurs prior to rupture of the FRP bars. In this case, the stress in the FRP bars at failure  $f_{frp}$  which is smaller than the tensile strength, can be calculated from:

$$f_{frp} = 0.5 E_{frp} \varepsilon_{cu} \left[ \left( 1 + \frac{4 \alpha_1 \beta_1 f'_c}{\rho_{frp} E_{frp} \varepsilon_{cu}} \right)^{1/2} - 1 \right] \quad (1.7)$$

in which  $\rho_{frp}$ , reinforcement ratio, is equal  $\rho_{frp} = \frac{A_{frp}}{bd}$  and  $E_{frp}$  is tensile modulus elasticity of FRP reinforcement. The depth of neutral axis,  $c$ , is calculated from equilibrium and then the moment resistance of the section can be calculated. Tensile failure is caused by rupture of FRP bars, so the FRP bars strain is  $\varepsilon_{frpu} = \frac{f_{frpu}}{E_{frp}}$  and corresponding strain  $\varepsilon_c$  at the extreme compressive fibre will be less than 0.0035. Therefore, the traditional rectangular block cannot be used for distribution of compressive stress in the concrete zone, and  $\alpha_1$  and  $\beta_1$  should be found for  $\varepsilon_c$  varying up to 0.0035 (from suggested tables and figures). The best iteration should be chosen in order to satisfy the following equilibrium equation,  $T=C$ , and then calculate the moment:

$$\alpha_1 f'_c \beta_1 c b = A_{frp} \varepsilon_{frpu} E_{frp} \quad (1.8)$$

Finally, this draft recommended that the FRP compressive bars should be ignored when calculating flexural capacity of the section.

#### 1.2.4.2 Recommended Design Provisions for Shear

Tottori and Wakui (1988) suggested the following design expressions for shear strength of FRP reinforced concrete beams.

$$V = V_c + V_s \quad (1.9)$$

$$V_c = 200 f_c^{\frac{1}{3}} (\rho_w \frac{E_f}{E_s})^{\frac{1}{3}} d^{\frac{-1}{4}} [0.75 + \frac{1.4}{(\frac{a}{d})}] b_w d \quad (1.10)$$

$$V_s = (A_{sf} E_{sf} \frac{\epsilon_{sf}}{S}) (\sin \alpha + \cos \alpha) Z \quad (1.11)$$

Where;

a : Shear span (m)

d : Effective Depth(m)

V<sub>c</sub> : Concrete contribution to shear resistance in kN

Z : d / 1.15

α : Angle between reinforcement and member axis

ε<sub>sf</sub> : Strain in shear reinforcement (ε<sub>sf</sub> = 0.01 for diagonal tension failure)

The shear design procedure recommended in “Design Guidelines for FRP Reinforced Concrete Building,” prepared by several Japanese researchers (1997) is presented below for ultimate shear strength of beams and columns.

$$V_U = \min (0.8 V_{U1}, 0.9 V_{U2}) \quad (1.12)$$

Where;

$$V_{u1} = b_w j \left[ \frac{0.115 K_u K'_p (f'_c + 180)}{\frac{M}{V d} + 0.12} + 2.7 \sqrt{\rho_w T_V} \right] \quad (1.13)$$

$$V_{u2} = b_w j \left[ \frac{0.115 K_u K'_p (f'_c + 180)}{\frac{M}{V d} + 0.12} + 2.7 \sqrt{\rho'_w T_V} \right] \quad (1.14)$$

Where;  $V_{u1}$  and  $V_{u2}$  correspond to the rupturing strength of shear reinforcement due to diagonal tension and compression crushing of concrete due to diagonal compression, respectively, in kgf.

$\rho_w$  = Volumetric ratio of shear reinforcement.

$\rho'_w$  = Equivalent shear reinforcement ratio.

$$\rho'_w = \left( \frac{E_v}{E_s} \right) \rho_w$$

$E_v$  = Young's modulus for FRP shear reinforcement (kgf/cm<sup>2</sup>)

$T_v$  = Tensile strength of shear reinforcement at bend section (kgf/cm<sup>2</sup>)

$$j \approx \frac{7}{8} d$$

$K_{Uj}$  = Correction coefficient dependent on  $d$  [ $K_{Uj} = 0.72$  when  $d \geq 40$  cm ]

$K'_p$  = Equivalent correction coefficient dependent on  $\rho_t$

$$K'_p = 0.82(100\rho_t \frac{E_f}{E_s})^{0.23}$$

The Canadian Standards Association, CSA S806-02 (2002) includes a method for shear design in flexural regions. The factored shear resistance,  $V_r$  for FRP grids is computed by the following expression:

$$V_r = V_c + V_{SF} \leq V_c + 0.6 \lambda \phi_c \sqrt{f'_c} b_W d \quad (1.15)$$

For sections with minimum amount of transverse reinforcement, or for members with an effective depth not exceeding 300 mm, factored shear resistance provided by concrete is given as follows:

$$V_c = 0.035 \lambda \phi_c [f'_c \rho_W E_F \frac{V_F}{M_F} d]^{1/3} b_W d \quad (1.16)$$

$$0.1 \lambda \phi_c \sqrt{f'_c} b_W d \leq V_c \leq 0.2 \lambda \phi_c \sqrt{f'_c} b_W d \quad (1.17)$$



The factored shear resistance provided by FRP shear reinforcement is specified as:

$$V_{SF} = \frac{0.4 \phi_F A_V f_{FU} d}{s} \quad (1.18)$$

Where;

- $\lambda$  : Factor to modify for concrete density.
- $\phi_c$  : Capacity reduction factor for concrete.
- $\rho_w$  : Longitudinal reinforcement ratio ( $\frac{A_{fp}}{b_w \cdot d}$ ).
- $V_F$  : Factored shear resistance calculated in accordance with ultimate limit state design.
- $M_F$  : Factored moment resistance calculated in accordance with ultimate limit state design.
- $\phi_F$  : Capacity reduction factor for FRP reinforcement
- $A_V$  : Area of shear reinforcement perpendicular to the axis of member within distance  $s$
- $f_{FU}$  : Ultimate strength of FRP shear reinforcement.

Nanni (2001) pointed out that the concrete shear capacity,  $V_{cf}$  of FRP reinforced flexural members can be derived from the ACI 318 expression for  $V_c$  (shear strength of a steel-RC cross section provided by concrete), by multiplying the ratio of axial stiffness of the FRP reinforcement ( $\rho_f E_f$ ) to that of steel reinforcement ( $\rho_s E_s$ ). For typical steel yield strength of 420 MPa and  $\rho_s$  taken as  $0.5\rho_{s,max} = 0.375\rho_b$ ,  $V_{cf}$  is given as following formula ( $V_{cf}$  cannot be larger than  $V_c$ ).

$$V_{cf} = \frac{\rho_f E_f}{90 \beta_1 f_c} V_c \leq V_c \quad (1.19)$$

He also pointed out that the method used to calculate shear contribution of steel stirrups,  $V_s$ , is also applicable to FRP shear reinforcement with the provision that the stress level in FRP,  $f_{fv}$ , is limited to control shear crack widths, maintain shear integrity of concrete and

avoiding failure at stirrup bends,  $f_{fb}$ . The ultimate design stress in shear reinforcement is given by

$$f_{fv} = 0.002E_f \leq f_{fb} \quad (1.20)$$

The Canadian Network of Centres of Excellence on Intelligent Sensing for Innovation Structures- ISIS Canada, ISIS-M04-00 Draft, 2000, recommended some expressions for shear resistance of FRP-reinforced beams. There are few differences between CSA S806-02 (2002) and this draft, the shear resistance attributed to concrete presented at CSA, was multiplied by one factor, thus the following expression was presented:

$$V_c = 0.2 \lambda \phi_c \sqrt{f'_c} b_w d \sqrt{\frac{E_{frp}}{E_s}} \quad (1.21)$$

Where  $E_s$  is modulus of elasticity of steel taken as 200 GPa. The minimum area of FRP shear reinforcement was suggested to calculate as following expression:

$$A_{frpv} = 0.06 \sqrt{f'_c} \frac{b_w s}{0.4 f_{frpvu}} \quad (1.22)$$

To control the shear crack width and avoid shear failures of FRP-reinforced beams, the strain in the stirrups at service load is limited to 0.002.

#### **1.2.4.3. Recommended Design Provisions for Members under Combined Axial Compression and Bending**

Paramanantham and Daniali (1993) investigated the behaviour of FRP reinforced concrete columns. The researchers presented equations for concentric and eccentric capacities of concrete columns reinforced with GFRP:

Under concentric loading:

$$P_n = 0.85 f'_c (A_g - A_f) + 0.003 E_f A_f \quad (1.23)$$

Under eccentric loading:

$$\begin{aligned} P_n &= C_c + C_f - T_f \\ C_c &= 0.85 f'_c \beta_1 \chi b \\ C_f &= \frac{A_f \epsilon'_F E_{fc}}{2} \\ T_f &= \frac{A_F \epsilon_t E_f}{2} \end{aligned} \quad (1.24)$$

where;  $\epsilon_t$  and  $\epsilon'_F$  are tensile and compressive strains in FRP reinforcement, respectively. It was observed that the failure of FRP reinforcement in compression, even for AFRP reinforcement, did not occur below 0.2 - 0.3% strain. Sonobe Y., et al (1997) in the Japanese "Design Guidelines of FRP Reinforced Concrete Building Structures" suggested that the effect of compressive strength of FRP could be neglected. This results in the following expressions for concentric column capacity  $N_c$ :

$$N_c = 0.85 f'_c A_g \quad (1.25)$$

The strength enhancement in concrete caused by confinement of hoop reinforcement is recognized in the Japanese guidelines for circular columns. Accordingly, the following expression is suggested for compressive strength of columns:

$$N_c = [0.12(100\rho_w)^2 + 1] f'_c A_g \quad (1.26)$$

$\rho_w$  : Shear reinforcement ratio( $\frac{A_v}{b_w s}$ ).

$A_v$  : Area of shear reinforcement within distance s.

## ***Chapter 2***

# **CFRP Reinforced Concrete Columns under Eccentric Loading**

### **2.1 General**

Five pin ended large-scale CFRP-reinforced concrete columns were designed, constructed and tested in Phase I of the experimental investigation. The test parameters considered were the eccentricity of loading, spacing of transverse grids reinforcement, and arrangement of longitudinal reinforcement. The following sections present the properties of specimens test variables considered and test results.

### **2.2 Test Specimens**

The geometry of column specimens tested in Phase I is illustrated in Figure 2.1. The specimens had a 230 mm square cross-section with either 4 or 8 bar arrangements. LeadLine CFRP bars with 8 mm diameter were used as longitudinal reinforcement, and NEFMAC CFRP grids were used as transverse reinforcement. The details of CFRP reinforcement are presented in Section 2.3. Figure 2.2 illustrates the arrangement of reinforcement and assembly of cages. The formwork was prepared from plywood. The reinforcement cages were placed in the formwork along with four bolts placed at the top for assembly of steel end plates for testing. This is illustrated in Figure 2.3. The specimens were cast horizontally using ready-mix concrete from a local supplier, and then cured by wet burlap and plastic sheet covers. Figure 2.4 shows the casting of specimens. The curing continued until standard cylinder tests indicated that the required strength was attained. Table 2.1 provides a summary of specimen properties and test variables for Phase I columns.

## **2.3 Material Properties**

### **2.3.1 Concrete**

Normal Portland Cement concrete was ordered from a local ready-mix supplier to cast the columns in the Structures Laboratory of the University of Ottawa. Standard concrete cylinders were cast from each batch to establish the 28 day-strength of concrete, as well as the strength on the day of column tests. In all cases the 28-day strength of concrete was higher than the specified design strength of 30 MPa. Maximum aggregate size used in all batches was 10 mm and the slump of concrete was 100 mm. The mix proportions of a cubic meter of concrete are specified in Table 2.2. Material characteristics are specified in Table 2.3.

Single-use molds were used to cast 150 mm by 300 mm standard cylinders to perform the standard compression tests. The cylinders were cast in three layers of equal volume, and compacted by rodding 25 strokes after each layer, in accordance with the CSA Test Method A23.2-3c (ASTM C192-20a). They were cured with wet burlaps and plastic sheets immediately after demolding. The curing was done together with test specimens having the same concrete, under identical environmental conditions. The cylinders were tested after 7, 14, 21 and 28 days, as well as during the period of column and beam tests. They were capped prior to testing with high strength sulphur compound, conforming to ASTM C617, to ensure the uniform application of load. A 2200 kN-capacity Fourney hydraulic testing machine was used to test cylinders in accordance with ASTM method C39 and establish the stress-strain relationships of concrete, since this machine could be connected to the data acquisition system. A compressometer with three-point contacts was used with two Linear Voltage Displacement Transducers (LVDTs) to measure concrete compressive strains. The results are summarized in Table 2.4 and Figure 2.5.

### **2.3.2 CFRP Bars**

LeadLine CFRP bars were used as longitudinal column reinforcement in Phase I specimens. These bars were developed by Mitsubishi Chemical Group in Japan, which pioneered research and development of CFRP for use in the construction industry. The rods were

produced by pultrusion of longitudinally oriented carbon fibers. This process resulted in higher strength and improved properties than twisted carbon fiber cables of the same diameter, which is also used in the construction industry as prestressing strands. LeadLine was produced for application to civil engineering structures, marine, chemical, and non-magnetic applications and for ground anchors and structural tiebacks. The properties of LeadLine bars used in the current investigation are presented in Table 2.5; the stress-strain relationship in tension, as provided by the manufacturer and a photo of a bar are shown in Figure 2.6 (a).

The stress-strain relationship of FRP bars in compression is difficult to establish by tests since the stability of bars must be maintained during testing. Therefore, samples having lengths equal to 2 times the bar diameter were tested under direct compression until failure. The stress-strain relationships in compression for LeadLine are plotted in Figure 2.6 (b). The average modulus of elasticity in compression was 15,000 MPa. The failure stress in compression was approximately 275 MPa. These correspond to 11% of tensile strength. The corresponding compressive failure strain was approximately 1.8% and measured by LVDT connected to testing machine. The failure in direct compression tests of bars was caused either by delamination of fibers and crushing of resin or by splitting of bars longitudinally.

### **2.3.3 CFRP Grids**

CFRP transverse reinforcement used in the current investigation consisted of NEFMAC grids. NEFMAC was developed by Shimizu Corporation of Japan and is manufactured in Canada by Autocon Composites Inc. of Weston, Ontario, under a license from Shimizu Corporation.

NEFMAC is manufactured by forming flat or curved grids through a pin-winding process, similar to filament winding. These grids are manufactured either from glass or carbon fibers, impregnated with vinyl ester resin. The product was reported to have a specific gravity of 1.4 and a modulus of elasticity of 100,000 MPa, by the manufacturer.

The NEFMAC<sup>TM</sup> grids used in the current investigation had carbon fibers. Those used in Phase I columns consisted of cross CFRP reinforcement elements with a 6 x 8 mm rectangular cross-section. The cross elements formed four equal-size square openings with an outside dimension of 200 mm. These cross elements protruded by about 6 mm beyond the grid perimeter at each exterior grid joint. Figure 2.7 illustrates the schematic view and test photographs of CFRP grids.

Coupons were cut from sample grids to establish their stress-strain relationships, experimentally. The cross legs of grids were cut to obtain coupons such that a grid joint, with overlapping carbon fibers was located in the test region. The coupons were cast in steel pipes at the ends, using a special epoxy. The ends had rough surface, created by cutting the crossing elements. This was essential to improve the friction resistance of coupons inside the steel pipes during testing. Figure 2.8(a) shows coupons before, during and after testing. Figure 2.8(b) depicts the stress-strain relationships of NEFMAC grids obtained experimentally through tension coupon tests. The coupons ruptured in tension at maximum strength of 1230 MPa with modulus elasticity of 76335 MPa.

## **2.4 Instrumentation**

The columns were well instrumented for displacement, rotation and strain measurements. A Linear Variable Differential Transducer (LVDT) was used to measure column mid-span deflection. A light steel frame mounted on the laboratory floor supported the LVDT. Additional LVDTs were placed vertically within the mid-span region to measure axial deformations on tension and compression sides of each column to obtain the average strain distribution. The LVDTs were mounted on two threaded rods, 300 mm apart. These rods had been cast in column concrete, extending into the core, to enable continued readings after the spalling of cover concrete. The locations of LVDTs are shown in Figure 2.9.

Electric resistance strain gauges were placed on transverse grids and longitudinal bars to measure the variation of strains during testing. The strain gauges on grids would indicate hoop tension under lateral expansion of compression concrete, and the gauges on longitudinal reinforcement would provide additional data to establish strain profiles. A total

of eight strain gauges were used in each column. Four of the strain gauges were mounted on the longitudinal bars and the remaining four on the mid-height grids. The strain gauge locations are illustrated in Figure 2.10.

The test data were recorded by a data Acquisition System that consisted of a Personal Computer, a Control Unit and a power supply.

## **2.5 Test Set-Up and Test Procedure**

A Tinius Olsen Universal Testing Machine with a 1500 kN capacity was used to test the columns. Figure 2.11 illustrates the overall test set-up and the test machine. The specimens were first externally confined at the ends by means of steel brackets, specially manufactured for this purpose. This would prevent premature failure of columns at the ends, outside the test region. The brackets were made of steel angles and were bolted together as illustrated in Figure 2.12 (a). Steel end plates were used to provide the required level of eccentricity. The eccentricity was changed by means of adjustable V-blocks. The end plates were bolted to the columns through threaded rods that had been cast in concrete.

The specimens were prepared for testing by placing the end brackets and end steel plates. They were carefully positioned in the machine for proper level of eccentricity. The load was applied through the universal testing machine. Figure 2.12(b) shows typical columns before loading. Load and deformation measurements were taken in small increments as the applied load was increased. The strain gauge readings were monitored during testing. The overall behaviour of specimens was manually recorded and the crack patterns were observed. As the peak load was approached, the application of load was slowed down due to the relatively brittle failure of FRP reinforced members. The readings were taken at a faster rate as deformations increased faster under approximately constant load, followed by subsequent reduction in load.

## **2.6 Test Results**

Test results, including observations during testing are presented and discussed in this chapter. Photographs taken at different stages of loading illustrate the performance of each column.



The behaviour of columns is presented in terms of moment-mid height deflection relationships. The moments plotted in these relationships were computed from recorded test data by multiplying the applied axial load with total mid-span eccentricity that consisted of the initial end eccentricity plus the mid-height column deflection.

Additional test data are presented in the form of moment-curvature and moment-reinforcement strain relationships. Columns were instrumented to measure two sets of longitudinal (vertical) elongation readings within the mid-height region. This region was the critical region for flexure since the columns were pin (hinge) supported at the ends. The first set of readings gave the elongation of tension fiber on one side and the other set gave the elongation of compression fiber on the other side. The gauge length used to mount the LVDTs was 300 mm. These readings were taken within the most disturbed region of columns because of concrete crushing. Hence, the LVDTs were removed before the crushing of concrete, resulting in lack of data at this very high level of deformations.

Moment-strain relationships for longitudinal and transverse CFRP reinforcement are also presented. Four corner longitudinal bars in each column were instrumented with strain gauges at the critical section at column mid-height. The strain gauges ceased to function at different stages of loading either due to material failure or due to detachment of gauges from reinforcement. Therefore, the data presented may not be complete in reflecting the behaviour of FRP reinforcement near the end of testing. A total of four strain gauges were placed on two grids within the column mid-height region for strain measurements.

The columns were able to develop their capacities predicted based on plane section analysis under combined bending and axial compression. The peak load was attained when the cover concrete showed signs of spalling. The longitudinal and transverse CFRP reinforcement remained intact at peak load resistance. Further increase in applied load resulted in the crushing of concrete, accompanied by rapid strength decay. The crack widths on the tension side were smaller than that expected in steel reinforced concrete at yield. The cover on the compression side spalled off completely shortly after the peak load, exposing CFRP reinforcement. As the longitudinal bars started bending and buckling, the load resistance

dropped gradually. There was no sign of material damage in CFRP bars at this stage in terms of delamination of fibers or failure of the resin.

The following sections provide observations made during individual column tests.

### **2.6.1 Column CFS1**

Column CFS1 was reinforced with 4 - 8 mm diameter LeadLine CFRP longitudinal bars that resulted in 0.38% reinforcement ratio. The column was confined with 4-cell CFRP grids. The grid spacing was 115 mm, which approximately corresponded to twice the maximum spacing allowed in ACI 318 (2002). The column was tested under monotonically increasing eccentric loading with an end eccentricity of 60 mm. The maximum moment recorded was 74.1 kN.m, including the effect of mid-height deflection. The test was continued until a maximum mid-height deflection of 31 mm was recorded.

Observations during testing indicated that initial flexural cracking started at approximately 500 kN load within the middle 220 mm portion of column. The cracks formed at the middle two grid locations near mid-height. The flexural cracking spread to the middle half of the column as the load was increased to 1000 kN, though they remained as hairline cracks. The failure was initiated by the crushing of concrete on the compression side, shortly after this stage of loading, at approximately 1020 kN of load, corresponding to maximum compressive fiber strain of 0.0032. The column failed abruptly shortly thereafter when concrete crushed completely. There was no failure observed in CFRP reinforcement. Neither the tension bars ruptured nor the compression bars buckled/failed. Figure 2.13 illustrates the observed performance of column CFS1 at selected stages of loading. Figure 2.14 shows recorded test data.

Strain gauge readings indicated that the maximum-recorded strains in FRP bars at the end of test were 0.58% in tension and 0.42% in compression. The maximum-recorded strain in FRP grids was 0.34% in tension. Both bars and grids were unloaded after the crushing of concrete, prior to reaching their failure strengths. Figures 2.14(b) and 2.14(c) show the strain gauge data recorded on longitudinal bars and grids, respectively. The moment-strain relationship for the extreme concrete fiber and moment-curvature relationship for the critical column section

are shown in Figures 2.14(d) and 2.14(e), respectively. Deformations in these relationships were obtained from LVDT readings. Maximum tensile and compressive strains of extreme concrete fibers were 0.21% and 0.32%, respectively, while the maximum curvature was approximately  $80 \times 10^{-6}$  (1/mm).

### **2.6.2 Column CFS2**

Column CFS2 was identical to CFS1, except for the end eccentricity, which was equal to 75 mm. The maximum moment recorded was 76.9 kN.m, including the effect of mid-height deflection. The test was continued until a maximum mid-height deflection of 26 mm was recorded.

Observations during testing indicated that initial flexural cracking started at approximately 500 kN load within the middle 300 mm portion of column. The cracks formed at the middle two grid locations near mid-height. The flexural cracking spread to the middle half of column as the load was increased to 860 kN, though they remained as hairline cracks. The column reached 1000 kN peak load resistance without significant spalling of concrete and damage to FRP reinforcement. Subsequent increase in load resulted in gradual crushing of concrete. The failure was not as brittle as that of the companion column (CFS1) tested under a smaller end eccentricity. Figure 2.15 illustrates the observed performance of column CFS2 at selected stages of loading. This column was the first to test in Phase I and some technical problems were encountered during testing. The hydraulic pump was shut down unexpectedly a number of times, which resulted in the unloading, and reloading of column. This is illustrated in the moment-displacement relationship shown in Figure 2.16. The same figure also includes other recorded test data.

Strain gauge readings indicated that FRP bars experienced a maximum of 0.45% tensile strain and 0.4% compressive strain at the end of test, whereas the FRP grids experienced a maximum of 0.05% tensile strain. Figures 2.16(b) and 2.16(c) show the strain gauge data recorded on longitudinal bars and grids, respectively. The moment-strain relationship for the extreme concrete fiber and moment-curvature relationship for the critical column section are shown in Figures 2.16(d) and 2.16(e), respectively. Deformations in these relationships were

obtained from the LVDT readings. Maximum tensile and compressive strains of the extreme concrete fibers were 0.47% and 0.55%, respectively, while the maximum curvature was approximately  $87 \times 10^{-6}$  (1/mm).

### **2.6.3 Column CFS3**

Column CFS3 was reinforced with 4 - 8 mm diameter LeadLine CFRP longitudinal bars that resulted in 0.38% reinforcement ratio. The column was confined with 4-cell CFRP grids. The grid spacing was 57 mm, which approximately corresponded to the maximum spacing allowed by ACI 318 (2002). The column was tested under monotonically increasing eccentric loading with an end eccentricity of 60 mm. The maximum moment recorded was 83.1 kN.m, including the effect of mid-height deflection. The test was continued until a maximum mid-height deflection of 21 mm was recorded.

Observations during testing indicated that initial flexural cracking started at approximately 500 kN load within the middle 310 mm portion of column. The cracks formed at the middle two grid locations near mid-height. The flexural cracking spread to the middle half of the column as the load was increased to 1000 kN, though they remained as hairline cracks. The maximum load resisted by the column was 1200 kN. It continued deforming beyond the peak load resistance with little crushing of cover concrete. As deformations increased the column failed gradually due to the crushing of concrete followed by the failure of compression bars. The compression bars were forced to buckle once the lateral restraint provided by concrete diminished due to spalling. These bars were subjected to higher compressive strains as compared to those in Column CFS1 because of the enhanced confinement of the core concrete. The CFRP bars were unable to develop the kind of plasticity often observed in buckling steel bars, which show unstable and excessively deformed shapes. The bars ruptured when forced to deform in compression. The failure type observed was similar to bar fracture in tension, as they were forced to bend and fail under bending. No damage was observed in tensile bars, which maintained their integrity until after the end of the test. Figure 2.17 illustrates the observed performance of column CFS3 at selected stages of loading. The test data are plotted in Figure 2.18.

Strain gauge readings indicated that FRP bars experienced a maximum of 0.62% tensile strain and 0.70% compressive strain at the end of test, whereas the FRP grids experienced a maximum of 0.22% tensile strain. Figures 2.18(b) and 2.18(c) show the strain gauge data recorded on longitudinal bars and grids, respectively. The moment-strain relationship for the extreme concrete fiber and moment-curvature relationship for the critical column section are shown in Figures 2.18(d) and 2.18(e), respectively. Deformations in these relationships were obtained from the LVDT readings. The maximum tensile and compressive strains of extreme concrete fibers at peak capacity were 0.31% and 0.36%, respectively, and the latter value increased up to 0.55% at the end of the test due to the close spacing of grids. The maximum curvature was approximately  $118 \times 10^{-6}$  (1/mm) based on the LVDT readings.

#### **2.6.4 Column CFS4**

Column CFS4 was identical to CFS3, except for the end eccentricity, which was equal to 75 mm. The maximum moment recorded was 83 kN.m, including the effect of mid-height deflection. The test was continued until a maximum mid-height deflection of 30 mm was recorded.

Observations during testing indicated that initial flexural cracking started at approximately 500 kN load within the middle 300 mm portion of column. The cracks formed at the middle two grid locations near mid-height. The flexural cracking spread to the middle half of the column as the load was increased to 900 kN, though the cracks were in the form of hairline cracks. The maximum load carried by this column was 960 kN. It was able to sustain post-peak deformations with little crushing of cover concrete. The failure of the column was initiated by the crushing of concrete, which lead to the failure of compression bars. These bars were able to sustain high compressive strains due to the close spacing of transverse reinforcement, but fractured under bending when they were forced to buckle. The type of failure observed was similar to those of Column CFS3. However, this time the test machine was not stopped immediately after the failure of compression bars, which lead to the rupturing of tension bars, splitting the entire column into two pieces at the end of the test. Figure 2.19 illustrates the observed performance of column CFS4 at selected stages of loading. Figure 2.20 shows recorded test data.

Strain gauge readings indicated that FRP bars experienced a maximum of 0.8% tensile strain and 0.70% compressive strain at the end of test, whereas the FRP grids experienced a maximum of 0.3% tensile strain. Figures 2.20(b) and 2.20(c) show the strain gauge data recorded on longitudinal bars and grids, respectively. The moment-strain relationship for the extreme concrete fiber and moment-curvature relationship for the critical column section are shown in Figures 2.20(d) and 2.20(e), respectively. Deformations in these relationships were obtained from the LVDT readings. Maximum tensile and compressive strains of the extreme concrete fibers were 0.55% and 0.4%, respectively, while the maximum curvature was approximately  $125 \times 10^{-6}$  (1/mm).

#### **2.6.5 Column CFS5**

Column CFS5 was reinforced with 8 - 8 mm diameter LeadLine CFRP longitudinal bars that resulted in 0.76% reinforcement ratio. The column was confined with 4-cell CFRP grids. The grid spacing was 115 mm, which approximately corresponded to twice the maximum spacing allowed by ACI 318 (2002). The column was tested under monotonically increasing eccentric loading with an end eccentricity of 75 mm. The maximum moment recorded was 89.5 kN.m, including the effect of mid-height deflection. The test was continued until a maximum mid-height deflection of 34 mm was recorded.

Observations during testing indicated that initial flexural cracking started at approximately 500 kN load at the middle four grid locations near mid-height. The flexural cracking spread to the middle half of the column as the load was increased to approximately 1000 kN, though the crack were in the form of hairline cracks. The load resistance increased up to 1050 kN, beyond which deformations increased with little degradation of strength and accompanying crushing of the cover concrete. The failure of the column was attributed to the crushing of concrete, followed by the failure of extreme compression and tension bars. The compression bars failed when they were forced to buckle and the tension bars ruptured. The middle layer of reinforcement present in this column prevented the splitting of the entire column into two pieces, unlike some of the column failures observed earlier. Figure 2.21 illustrates the observed performance of column CFS5 at selected stages of loading. Figure 2.22 shows recorded test data.

Strain gauge readings indicated that FRP bars experienced a maximum of 0.82% tensile strain and 0.81% compressive strain at the end of test, whereas the FRP grids experienced a maximum of 0.55% tensile strain. Figures 2.22(b) and 2.22(c) show the strain gauge data recorded on longitudinal bars and grids, respectively. The high level of compressive strain recorded in longitudinal bars is of significance, since it shows that CFRP bars can develop as high as 1200 MPa compressive stress without failure, assuming their elastic modulus in compression is the same as its value in tension. The moment-strain relationship for the extreme concrete fiber and moment-curvature relationship for the critical column section are shown in Figures 2.22(d) and 2.22(e), respectively. Deformations in these relationships were obtained from the LVDT readings. Maximum tensile and compressive strains of the extreme concrete fibers were 0.55% and 0.51%, respectively, while the maximum curvature was approximately  $122 \times 10^{-6}$  (1/mm).

## **2.7 Effects of Test Parameters**

### **2.7.1 Spacing of Grids**

The effect of grid spacing was evaluated by comparing the results of columns CFS1 and CFS3 as well as CFS2 and CFS4. The results are illustrated in Figure 2.23 and indicate that CFS3 and CFS4 with reduced grid spacing showed improved deformability. However, the effect was more pronounced in columns with low end eccentricity since the contribution of concrete to overall response in these columns would be more dominant, showing the improvement in concrete confinement caused by reduced grid spacing. Similarly, the moment capacity improved with reduced grid spacing, resulting in 12% and 8% increases in moment resistances for columns tested under low and high end eccentricities, respectively. Examination of the reinforcement strain data further indicates that the maximum compressive strains recorded in CFS3 with closer grid spacing was approximately 55% higher than that recorded in CFS1 with wider grid spacing.

### **2.7.2 Eccentricity**

The effect of end eccentricity was evaluated by comparing the results for columns CFS1 with CFS2 and columns CFS3 with CFS4. The comparisons for moment-displacement relationships are illustrated in Figure 2.24. The results indicate that columns subjected to higher end eccentricity were subjected to higher bending, which resulted in increased flexural cracking and softening of the members. This can be seen in the earlier part of the curves, where columns with higher end eccentricity show reduced slope, signifying reduced stiffness. However, these columns also showed more ductile response, as expected. Reinforced concrete elements typically behave in a more ductile manner under reduced axial compression. The improvement in post-peak deformations was more pronounced in column CFS4 with closely spaced grids, due to the improved confinement of concrete.

Comparison of reinforcement strain data for columns tested under different levels of end eccentricity indicates that those subjected to higher end moments developed significantly higher tensile and compressive strains in longitudinal reinforcement. The increase in strains ranged between approximately 20% and 70%.

### **2.7.3 Reinforcement Arrangement**

The effect of reinforcement arrangement on column behaviour was evaluated by comparing the results for Columns CFS2 and CFS5 in Figure 2.25. Column CFS5 had twice the longitudinal reinforcement, well distributed around the perimeter of column. Therefore, it developed about 16% higher moment capacity, while showing initially stiff behaviour. The post-peak behaviour of this column was also improved, resulting in enhanced deformability as compared to the companion column with only 4 corner bars. A contributing factor to the post-elastic behaviour was the enhancement of concrete confinement mechanism resulting from the better distribution of longitudinal reinforcement.

The comparison of reinforcement strain data indicated that Column CFS5 developed approximately 100% to 140% more tensile and compressive strains in FRP longitudinal reinforcement, reaching about 0.8% strain in both tension and compression without failure.



## 2.8 Ductility

The experimental data was further analyzed to assess inelastic deformability of columns.

This was done by computing displacement and curvature ductility ratios,  $\mu_\Delta$  and  $\mu_\phi$ , respectively defined by Pauly and Priestly (1991) and given as below:

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} \quad (2.1)$$

$$\mu_\phi = \frac{\phi_u}{\phi_y} \quad (2.2)$$

where  $\Delta_y$  and  $\Delta_u$  are column displacements at mid-height corresponding to the initial peak moment resistance and ultimate failure moment, respectively. In a multi-storey multi-bay building frame, it is not realistic to expect failure as soon as the peak moment resistance is attained. In these frame elements, some level of strength degradation may be tolerated during a severe seismic activity as the structure develops plastic hinges and dissipates seismic induced energy without jeopardizing the overall strength and stability of the structure. Therefore, the ultimate displacement used in ductility ratio calculations is taken as displacement corresponding to 20% strength decay in moment resistance beyond the peak value. The displacement quantities used in finding ductility ratios are analogous to yield and ultimate displacements in steel reinforced concrete members. Similarly, the curvature values  $\phi_y$  and  $\phi_u$  corresponds to the onset of crushing in extreme compression fiber and curvature corresponding to 20% strength decay in moment resistance beyond the peak moment. These definitions of ductility ratios are illustrated schematically in Figure 2.26. Figures 2.27 and 2.28 show computed displacement and curvature ductility ratios for the columns tested in this phase of the experimental program. The results show that the displacement ductility ratio ranged between 1.8 and 3.2. Columns with widely spaced grids showed displacement ductility ratios of 1.8 and 2.0, while those with closely spaced grids showed slightly improved ductility ratios of 2.5 and 3.2 due to improved concrete confinement. Column CFS5, with an 8-bar arrangement showed somewhat improved ductility ratio of 2.3, in spite of the wide spacing of grids used. In general, columns with increased end eccentricities

developed slightly higher displacement ductility ratios. Similar observations can also be made for curvature ductility ratios, which varied between 3.2 and 5.6. These results provide insight into inelastic deformability of FRP reinforced columns, which are not expected to have significant deformability due to the lack of inelasticity in FRP reinforcement.

The experimentally obtained ductility ratios are compared with those obtained from similar tests of steel-reinforced concrete columns, conducted by Saatcioglu et al (1995) in Table 2.6. These columns either had 8 or 12 longitudinal reinforcement with either perimeter ties or perimeter ties in combination with one or two crossties in each direction. The results indicate that steel reinforced concrete columns developed displacement ductility ratios of 2.1 to 5.4, depending on the reinforcement arrangement and eccentricity of loading. Steel reinforced columns with 8 bars supported by perimeter ties only, developed ductility ratios of 2.1 and 2.2 when the tie spacing was approximately equal to half the cross-sectional dimension. When an 8-bar column with crossties was tested, similar to Column CFS5 of the current investigation, the displacement ductility ratio obtained was 3.1, as opposed to 2.3 of the CFRP reinforced column.

## **2.9 Conclusion**

The following conclusions can be drawn from the columns tested in Phase I of the experimental investigation:

- CFRP reinforced columns are able to develop their moment capacities as governed by the crushing of concrete. CFRP reinforcement used as column compression reinforcement maintained its integrity and load resistance until after the surrounding concrete crushed and spalled off beyond the peak moment resistance. CFRP grids were able to fulfill their functions as transverse reinforcement in terms of providing lateral support to compression bars and confining compression concrete.
- CFRP bars were measured to sustain up to 0.8 % compressive strain without failure after crushing of concrete in compression. The same bars developed up to 1% tensile strain at the end of test. Compressive strains of bars corresponding to peak moment resistance were 0.33% and 0.36% in 4-bar columns when the grid spacing was  $h/2 =$

115 mm and  $h/4 = 58$  mm, respectively. Similarly, the strain was 0.4% in an 8-bar column with a grid spacing of  $h/2 = 115$  mm.

- Failure of compression bars was observed only after crushing of the concrete. When the tie (grid) spacing was high, the bars developed up to approximately 0.45% and 0.8% compressive strain, in 4-bar and 8-bar arrangement, and then were forced to buckle. Because CFRP bars do not have high bending capacity, they broke when forced to buckle, without showing the kind of plastic deformations that are observed in bending and buckling steel re-bars. The failure of CFRP bars in compression was in the form of breaking the bars into two by bending, as if the bar was cut. There was no sign of delamination of fibers in compression. The failure was also different from the rupturing failure in direct tension, which often showed randomly fractured carbon fibers of different length within the failure region.
- The longitudinal CFRP bars were able to develop high tensile strains required to maintain sectional equilibrium under increased bending as the concrete on the compression side was gradually crushing. None of the columns tested failed due to the tension failure of CFRP bars. Tension failure of CFRP bars were observed in only two columns after the compression bars failed and the columns lost their stability.
- CFRP reinforced columns do develop some limited ductility, initiated by the gradual crushing of concrete. Though limited in numbers, the comparison of ductility ratios with those of steel reinforced columns, at 0.25h to 0.30h end eccentricity range, indicated that CFRP reinforced columns were able to sustain approximately 2/3 the ductility ratios observed in companion steel reinforced columns.
- The examination of test parameters indicated that the expected trends, as observed in steel reinforced columns were also observed in CFRP reinforced columns, i.e., the deformability increased with reduced tie (grid) spacing, increased number of longitudinal bars and increased end eccentricity.

Table 2.1. Properties of Phase I Specimens (Columns under Eccentric Loading)

| Specimen<br>Label | $f_{co}$<br>MPa | Reinf<br>Arrang. | $\rho$<br>(%) | Eccentricity<br>(mm) | L<br>(mm) |
|-------------------|-----------------|------------------|---------------|----------------------|-----------|
| CFS1              | 30              | 4-8 mm bar       | 0.36          | 60                   | 1680      |
| CFS2              | 30              | 4-8 mm bar       | 0.36          | 75                   | 1680      |
| CFS3              | 30              | 4-8 mm bar       | 0.36          | 60                   | 1680      |
| CFS4              | 30              | 4-8 mm bar       | 0.36          | 75                   | 1680      |
| CFS5              | 30              | 8-8 mm bar       | 0.72          | 75                   | 1680      |

Table 2.2: Mix Proportion of a Cubic Meter of Concrete

|                             |         |
|-----------------------------|---------|
| Specified Concrete Strength | 30 MPa  |
| Cement                      | 204 kg  |
| Slag                        | 83 kg   |
| Aggregate                   | 1130 kg |
| Sand                        | 873 kg  |
| Water                       | 132 kg  |

Table 2.3: Properties of Concrete and Constituent Materials

|                             |                    |
|-----------------------------|--------------------|
| Specified Concrete Strength | 30 MPa             |
| Slump                       | 80 mm              |
| Type of Cement              | Type 10 Lafarge    |
| Type of Slag                | St. Mary's         |
| Type of Aggregate           | Crushed lime Stone |
| Type of Sand                | Gatineau Sand      |
| Maximum Aggregate Size      | 10 mm              |

Table 2.4: Variation of Concrete Strength with Age in Phase I Specimens  
(Columns under Eccentric Loading)

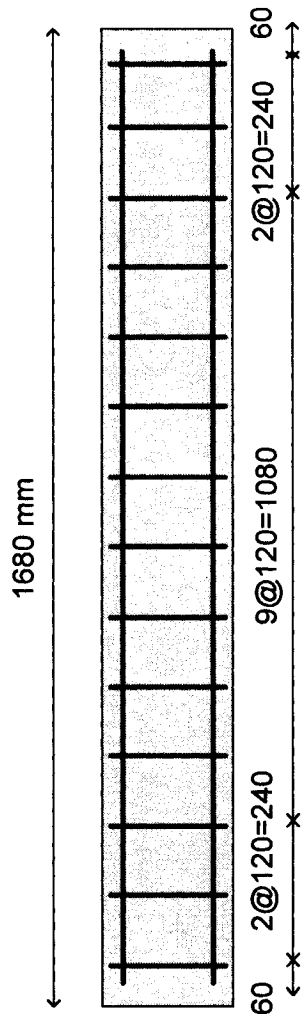
| Compressive Strength of Concrete in MPa |        |         |         |          |
|-----------------------------------------|--------|---------|---------|----------|
| Age                                     | 7 days | 14 days | 28 days | 10 weeks |
| Cylinder 1                              | 29.1   | 37.4    | 47.3    | 54.5     |
| Cylinder 2                              | 28.8   | 37.3    | 47.2    | 53.7     |
| Cylinder 3                              | 30.5   | 38.1    | 47.3    |          |
| Average                                 | 29.5   | 37.6    | 47.25   | 54.1     |

Table 2.5: Properties of LeadLine CFRP Bars

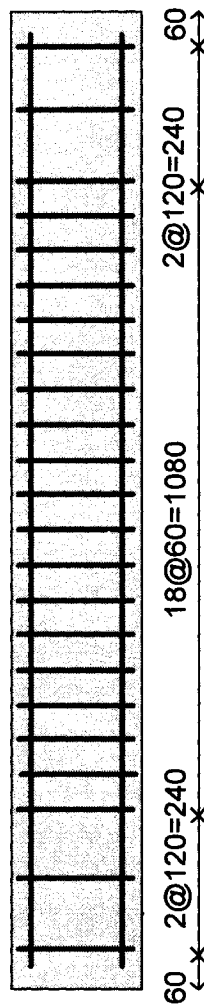
|                  |               |                       |                                       |
|------------------|---------------|-----------------------|---------------------------------------|
| Diameter         | 8 mm          | Matrix                | Epoxy Resin                           |
| Tensile Strength | 2550 Mpa      | Fiber Volume Fraction | 65%                                   |
| Youngh's Modulus | 147000 Mpa    | Elongation            | 1.6%                                  |
| Unit Weight      | 77 g/m        | Specific Gravity      | 1.6                                   |
| Relaxation Ratio | 2-3% at 20° C | Thermal Expansion     | $0.7 \times 10^{-6} / ^\circ\text{C}$ |

Table 2.6 : Ductility Values of Steel Reinforced Concrete Columns  
under Eccentric Loading [Obtained from Saatcioglu et al .(1995) ]

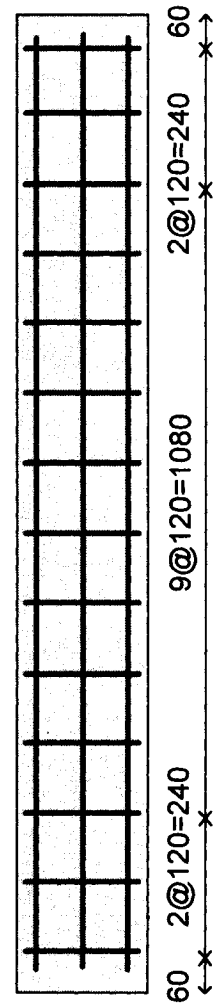
| Column | Eccentricity<br>e (mm) | Tie Spacing<br>S (mm) | No. of<br>Bars | Cross Tie | Moment<br>$M_u$ (kN.m) | $\mu_\Delta$ | $\mu_\phi$ |
|--------|------------------------|-----------------------|----------------|-----------|------------------------|--------------|------------|
| C5     | 75                     | d/4                   | 8              | yes       | 68                     | 4.2          | 5.4        |
| C6     | 75                     | d/4                   | 12             | yes       | 83                     | 4.6          | 6.4        |
| C7     | 60                     | d/2                   | 8              | no        | 50                     | 2.2          | 2.5        |
| C8     | 60                     | d/2                   | 8              | yes       | 55                     | 2.5          | 4.5        |
| C9     | 60                     | d/2                   | 12             | yes       | 60                     | 3.9          | 4.2        |
| C10    | 75                     | d/2                   | 8              | no        | 60                     | 2.1          | 4.6        |
| C11    | 75                     | d/2                   | 8              | yes       | 56                     | 3.1          | 6          |
| C12    | 75                     | d/2                   | 12             | yes       | 69                     | 4.1          | 5.4        |



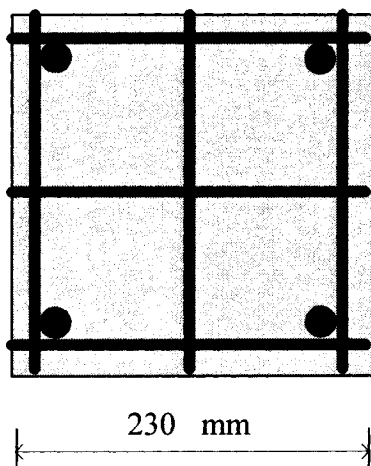
(a) Columns CFS1 and CFS2



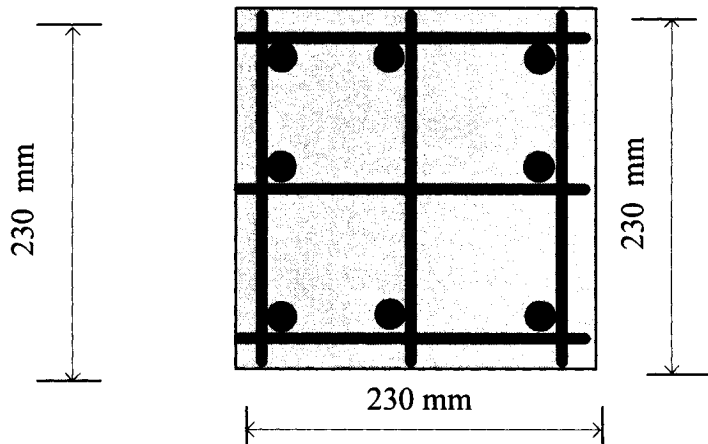
(b) Columns CFS3 and CFS4



(c) Column CFS5



(d) 4-Bar Arrangement



(e) 8-Bar Arrangement

Figure 2.1: Geometric Details of Phase I Columns Under Eccentric Loading

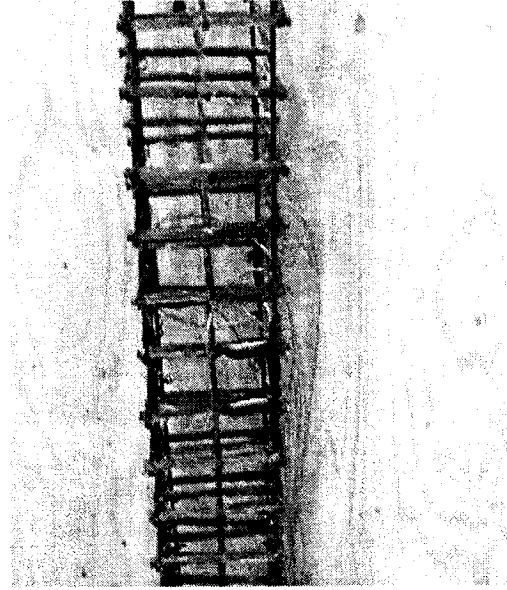
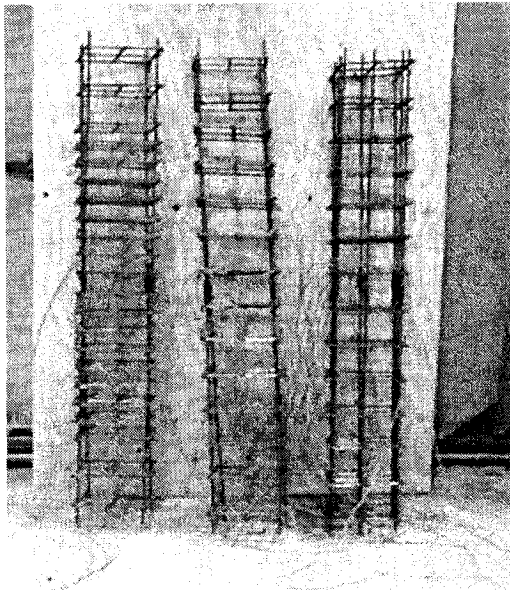
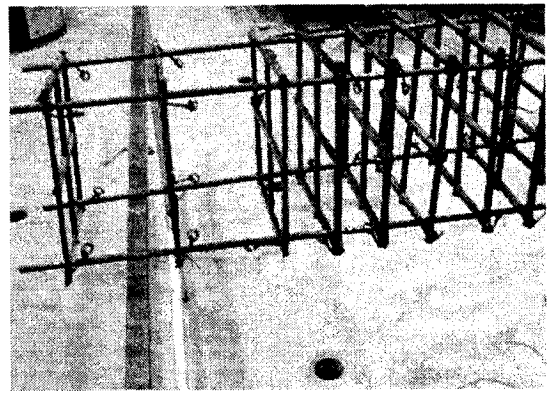
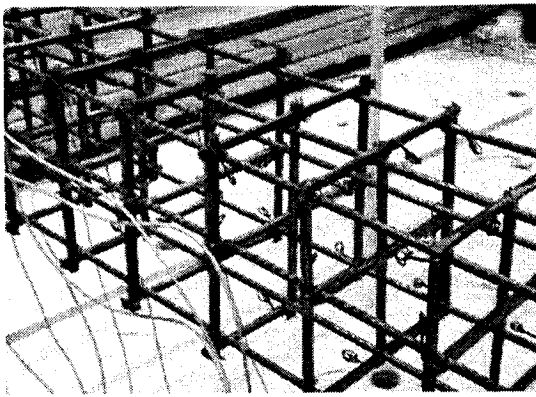
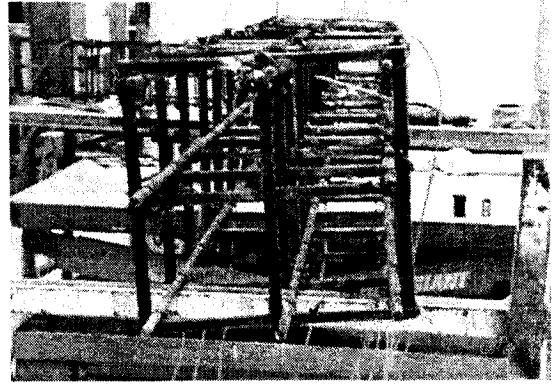
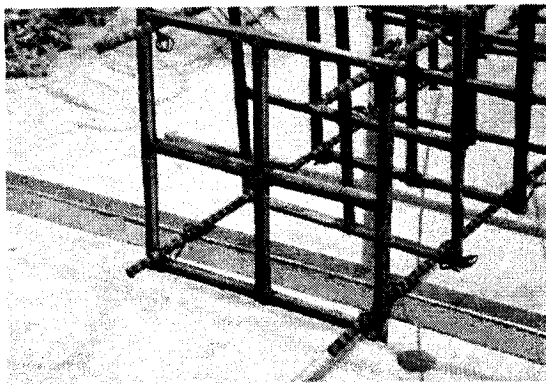


Figure 2.2: Arrangement of Reinforcement and Assembly of Cages for Phase I Specimens

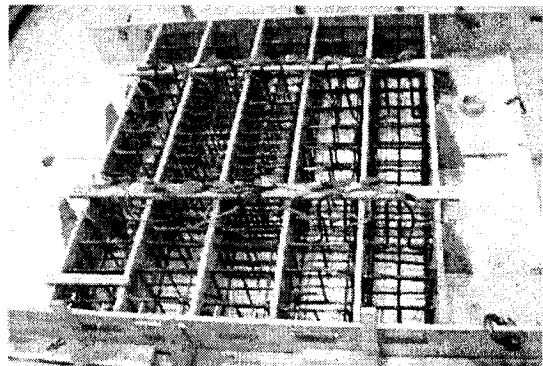
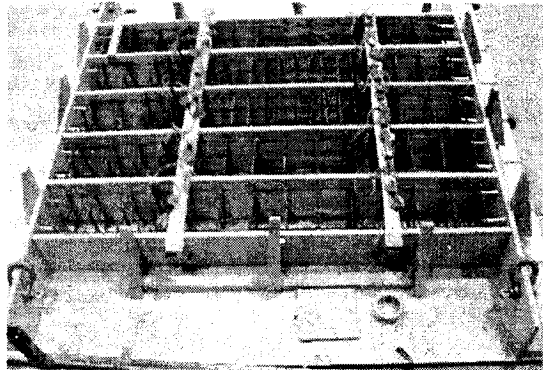
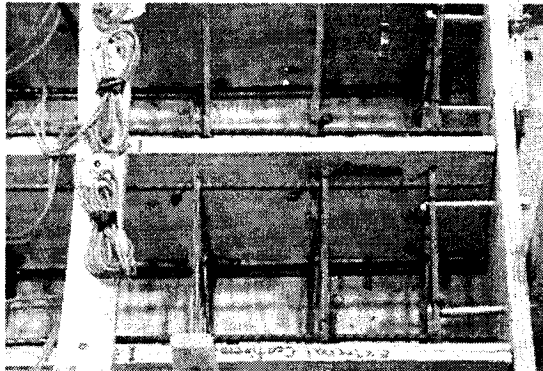
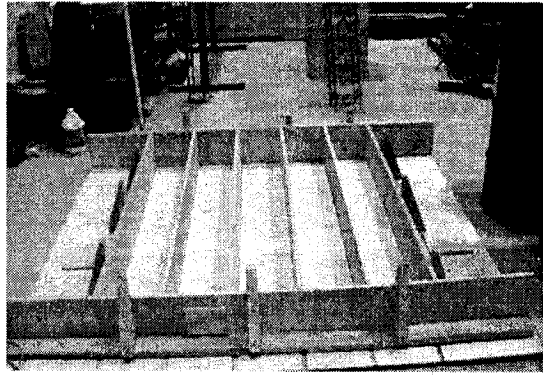


Figure 2.3: Preparation of Formwork and placement of Cages for Phase I Columns



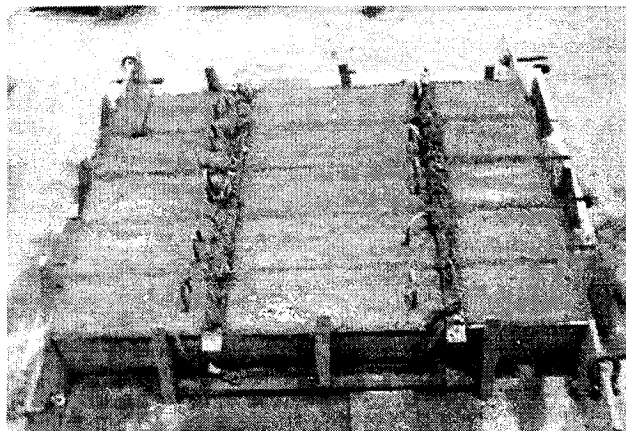
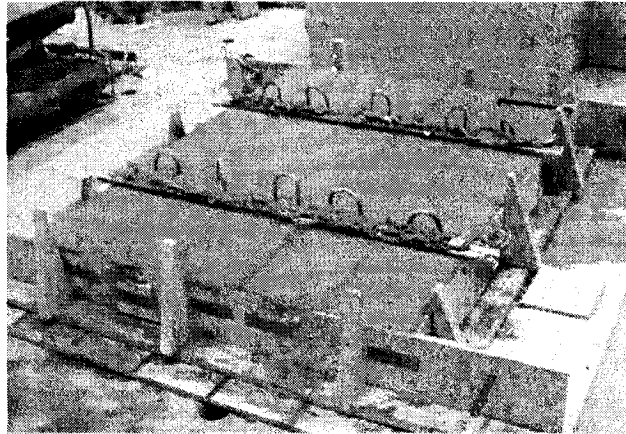
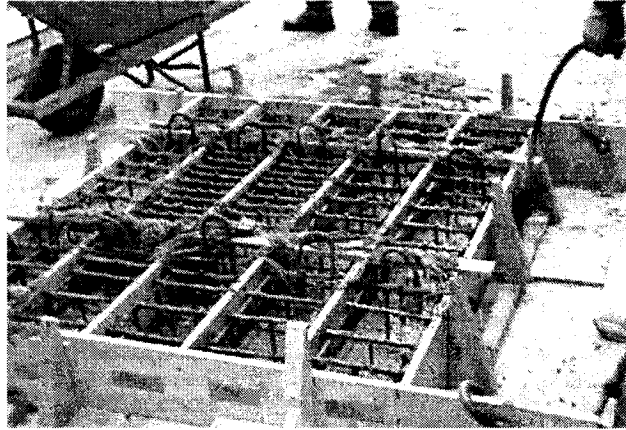


Figure 2.4: Casting of Phase I Column Specimens

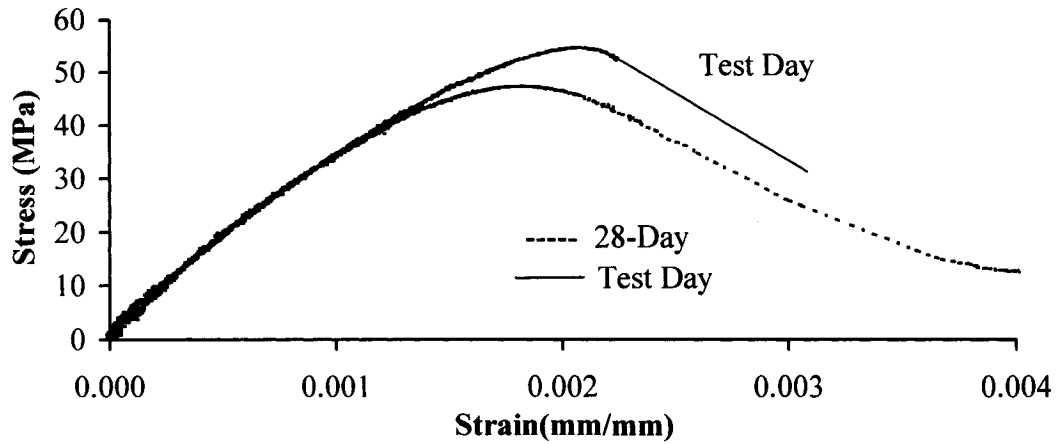
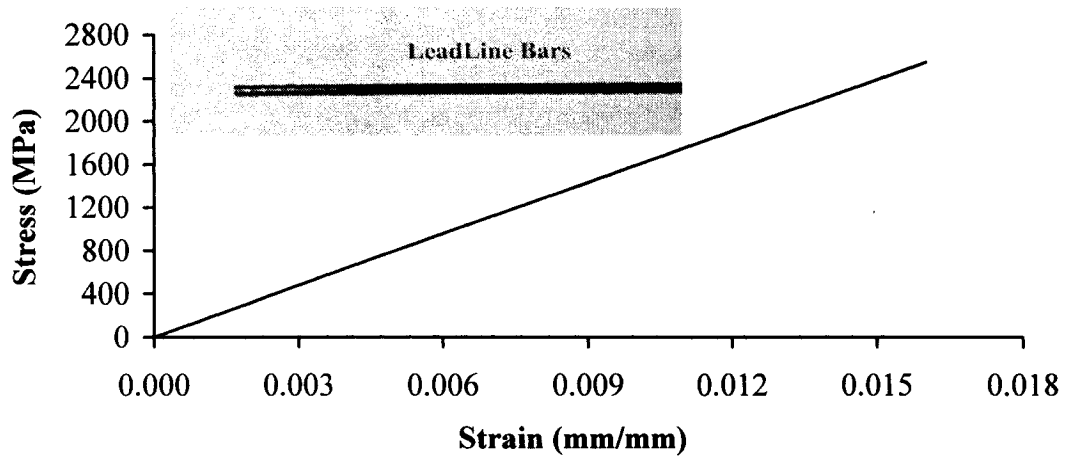
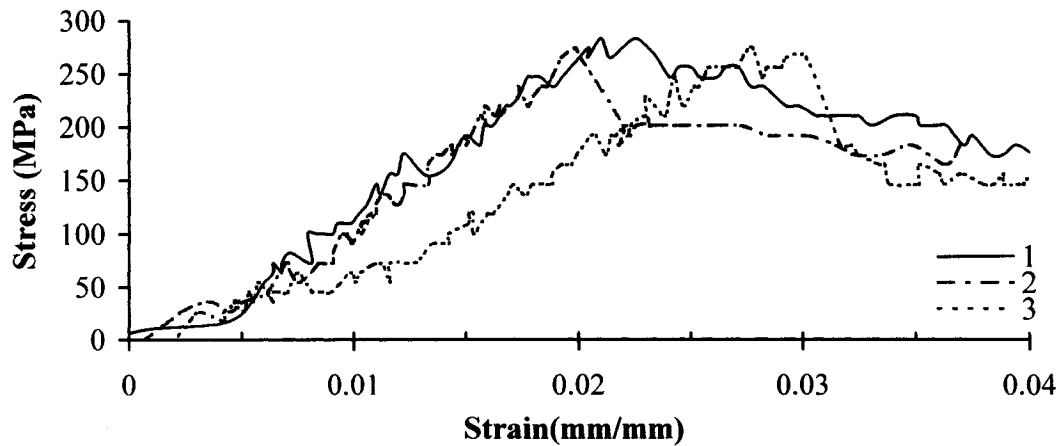


Figure 2.5: Stress-Strain Relationship for Plain Concrete in Phase I Specimens (Columns under Eccentric Loading)



(a) Tensile Stress-Strain Relationship



(b) Compressive Stress-Strain Relationship ,  $l/d=2$

Figure 2.6: Tensile and Compression Stress-Strain Relationship for LeadLine CFRP Bars

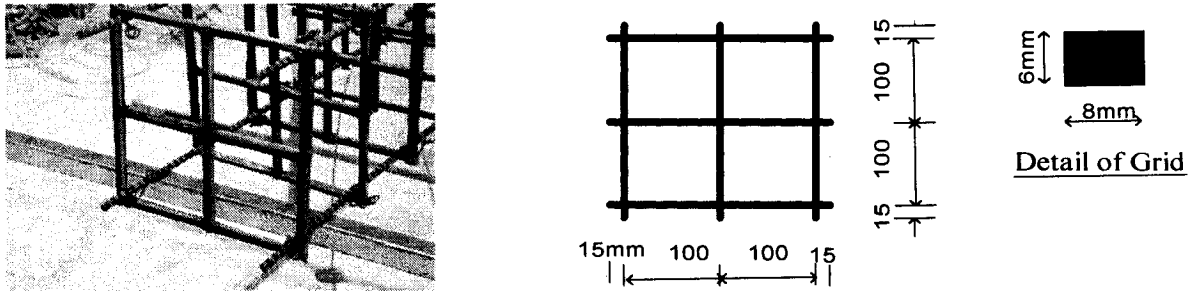
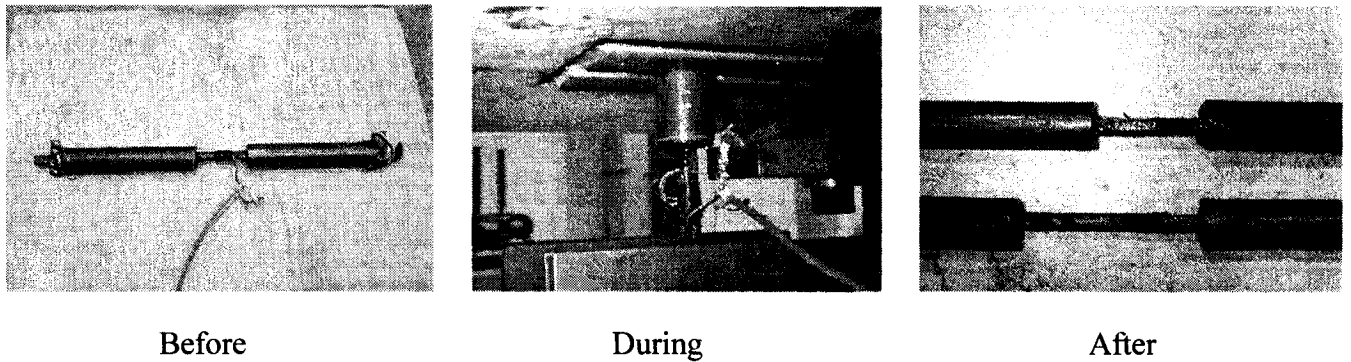
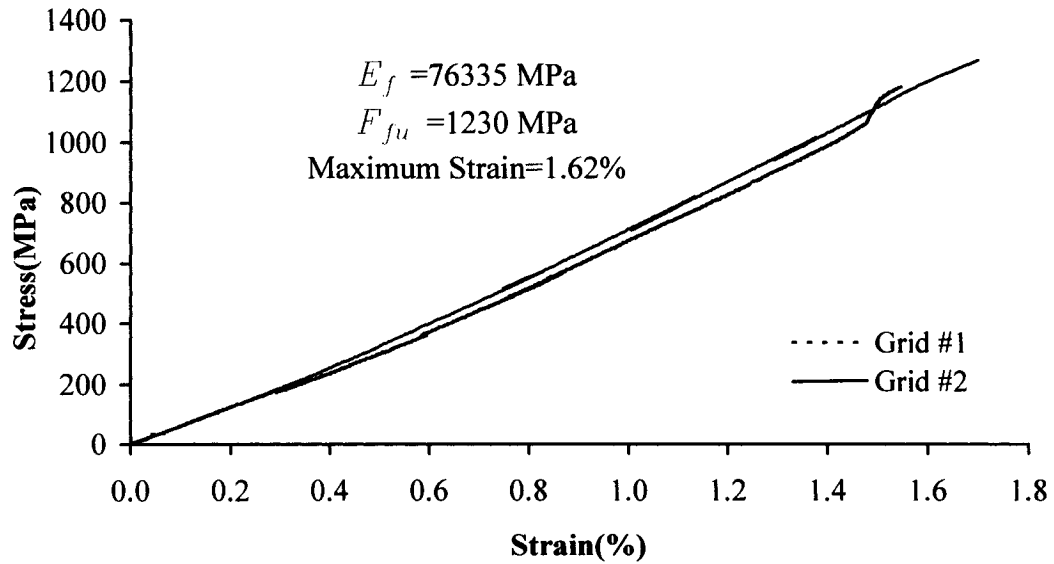


Figure 2.7: CFRP Grids Used in Phase I Specimens



(a) Coupons Before, During, and After Testing



(b) Stress-Strain Relationship

Figure 2.8: Coupon Tests of NEFMAC Grids

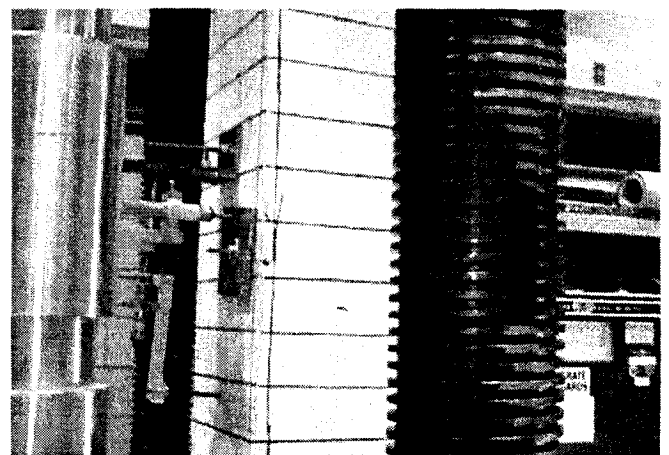
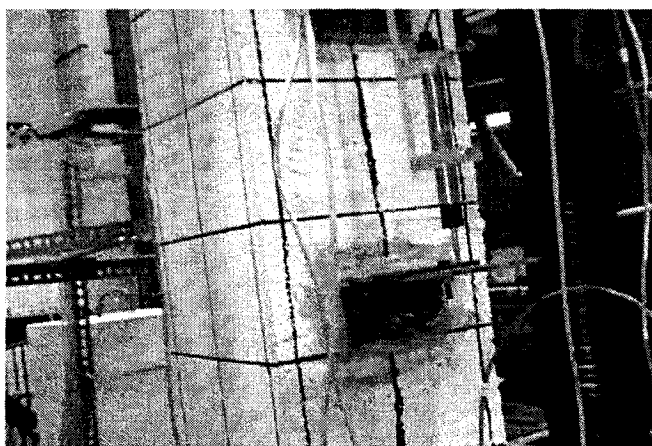
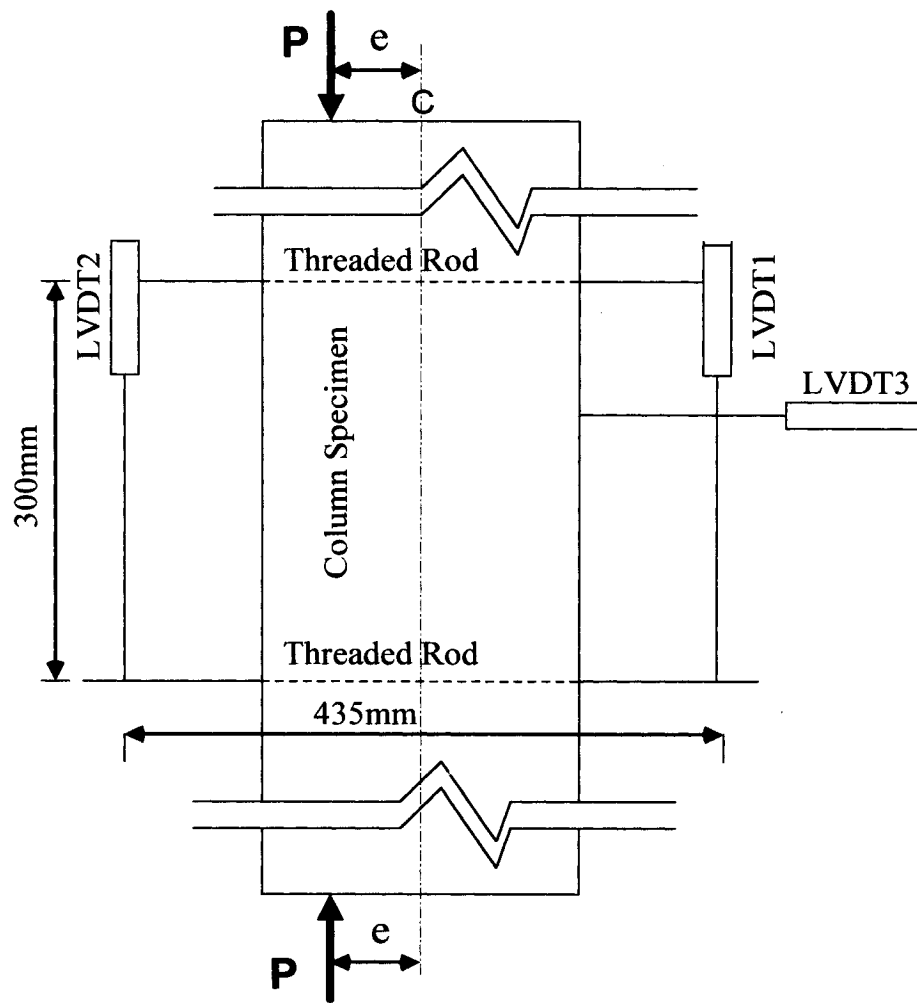


Figure 2.9 : Location of Vertical and Horizontal LVDTs on Phase I Columns Under Eccentric Loading

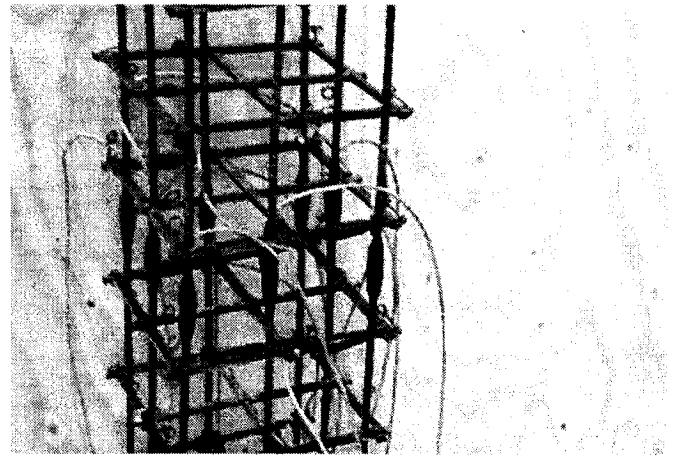
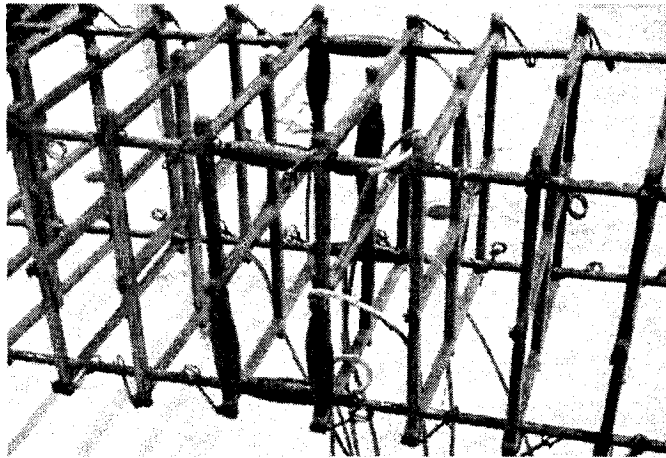


Figure 2.10: Strain Gauge Positions on Grids and Longitudinal Bars for Phase I Columns

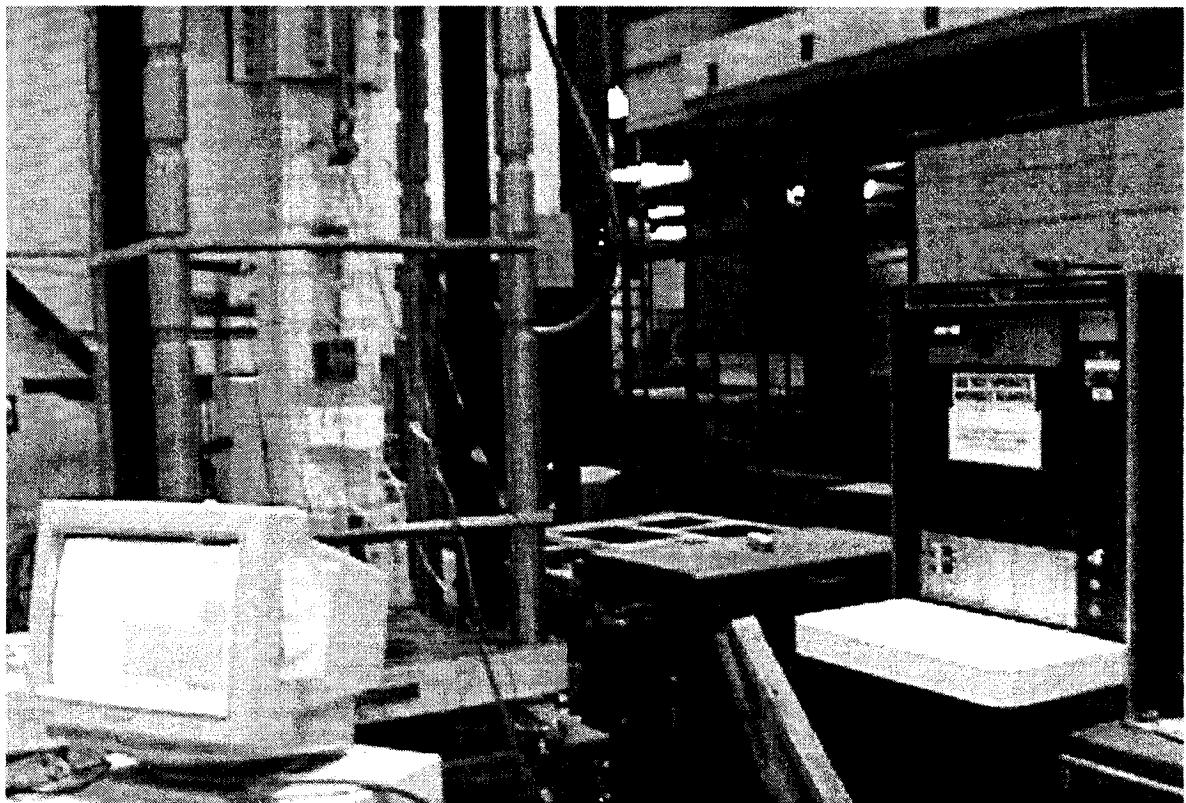
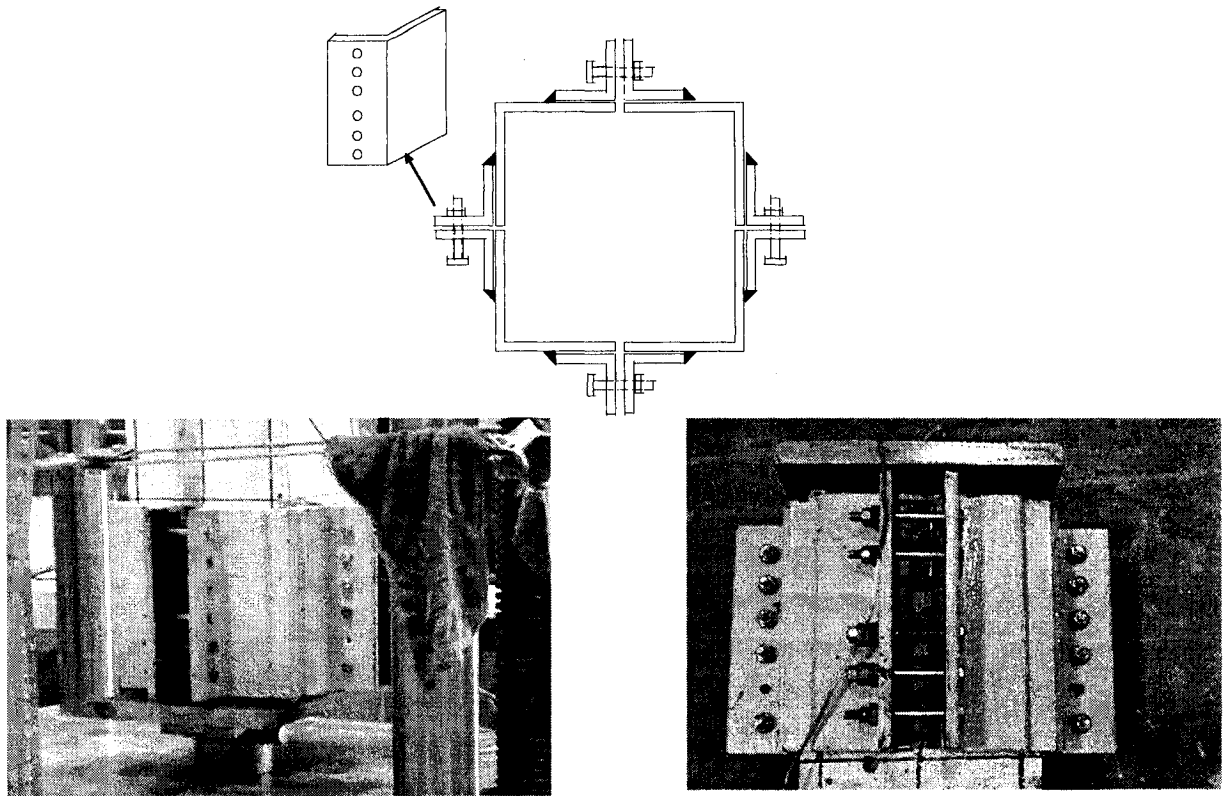
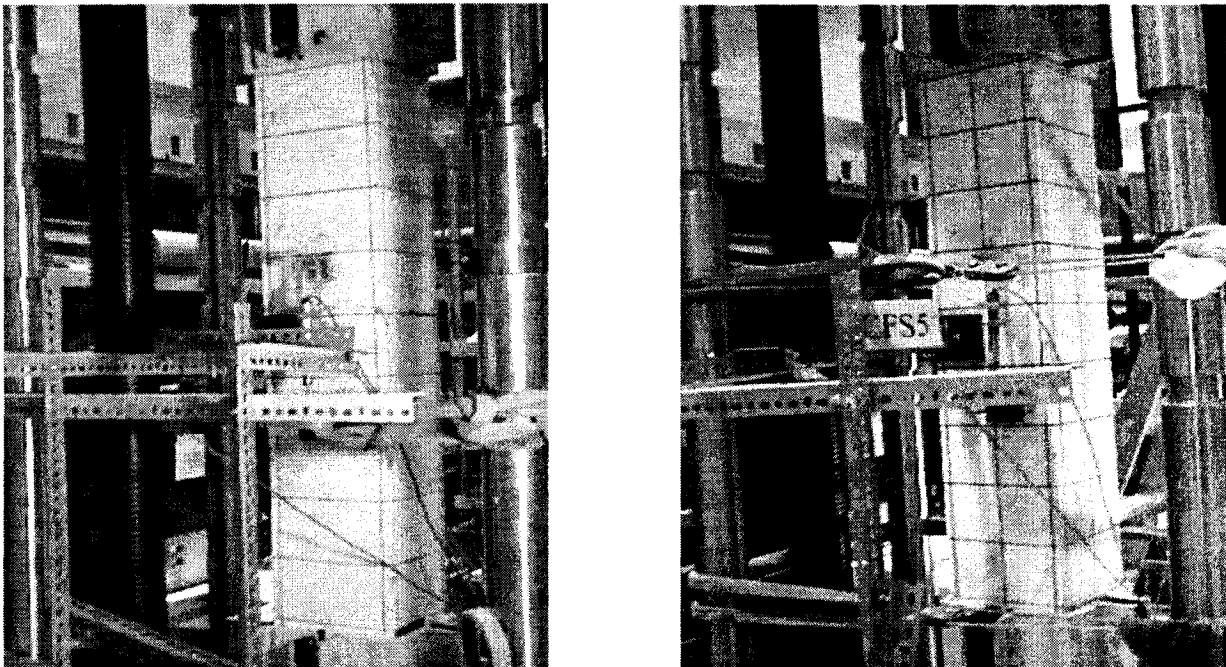


Figure 2.11: Test Set-Up and Test Machine for Phase I Columns Under Eccentric Loading

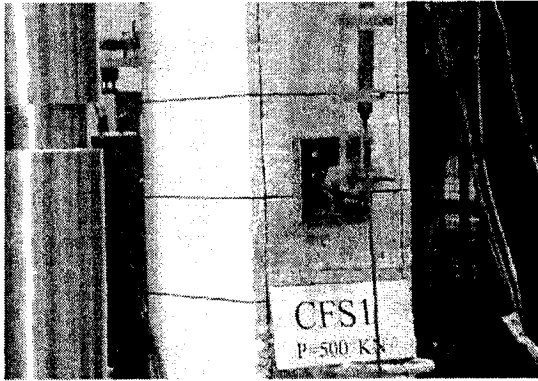


(a) External End Confinement



(b) Specimens Before Loading

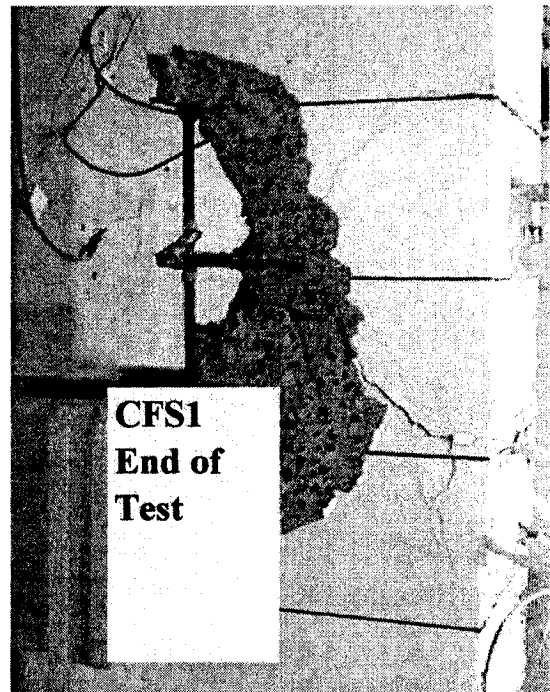
Figure 2.12: Test Specimens for Phase I Columns under Eccentric Loading



(a) At 500 kN Load

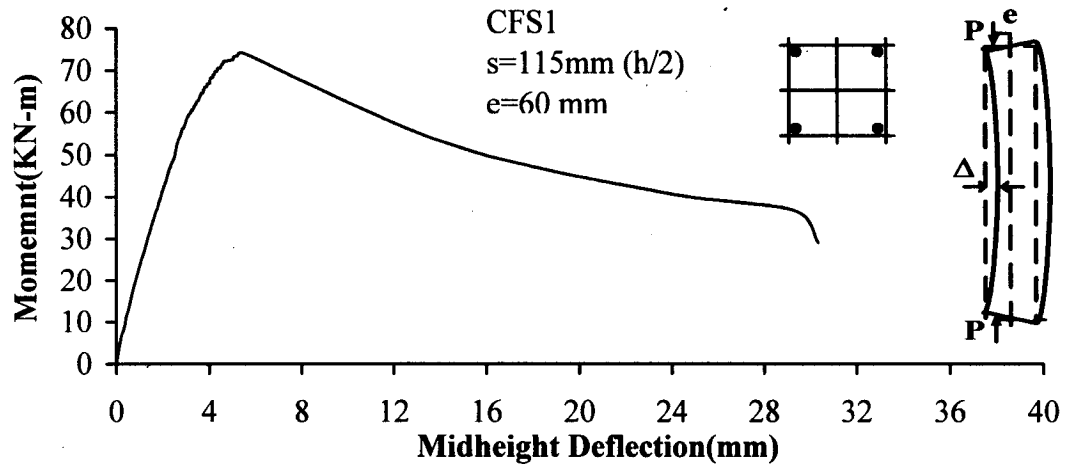


(b) At 1000 kN Load

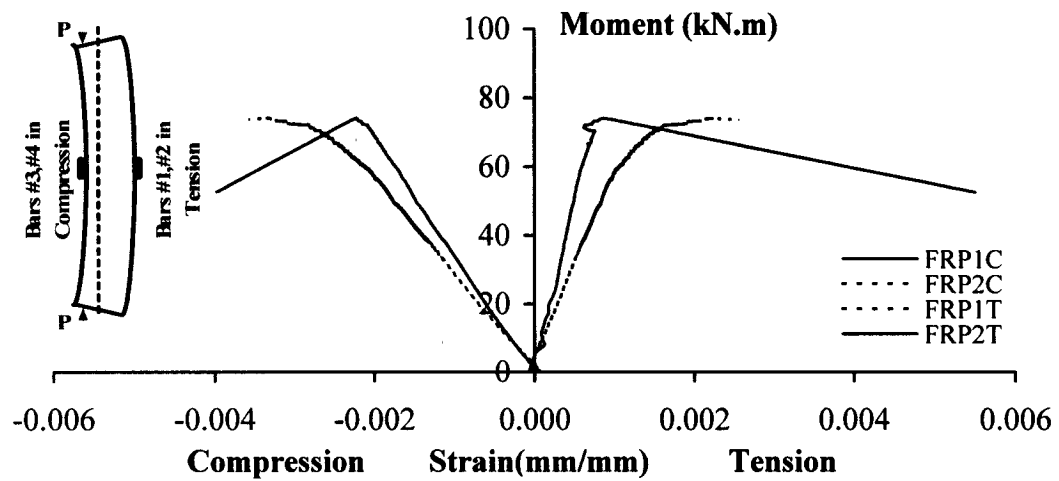


( c ) After Crushing

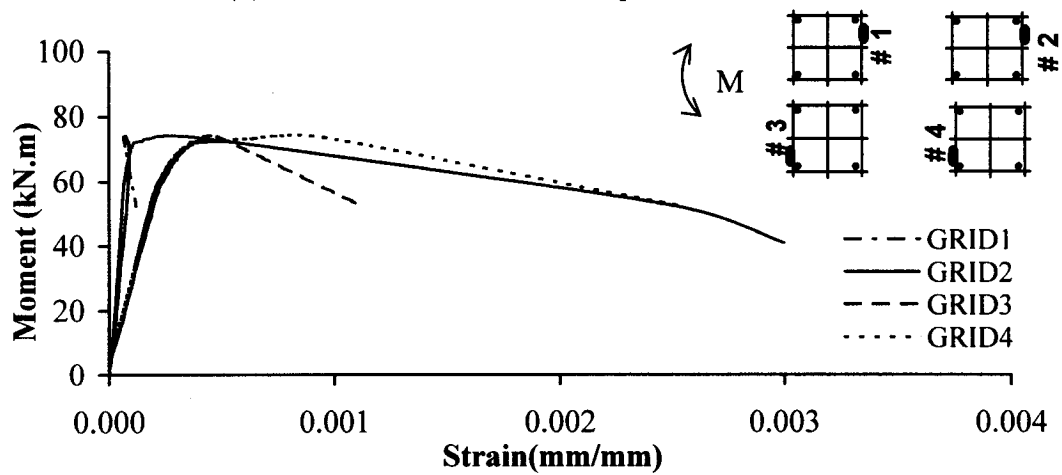
Figure 2.13: Column CFS1 at Different Stages of Loading



(a) Moment- Midheight Deflection Relationship



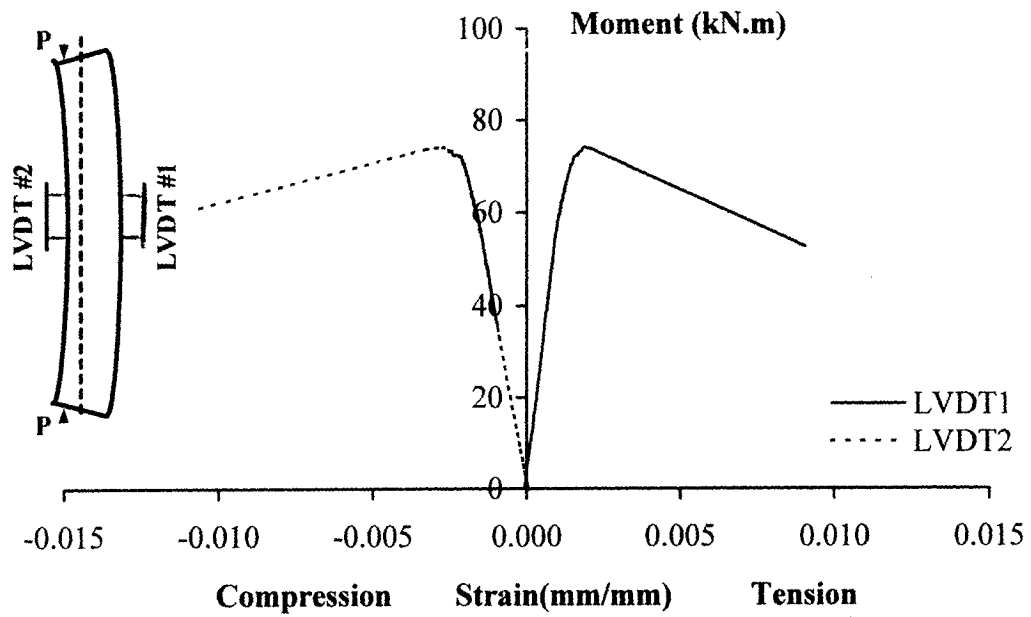
(b) Moment-Strain Relationship of CFRP Bars



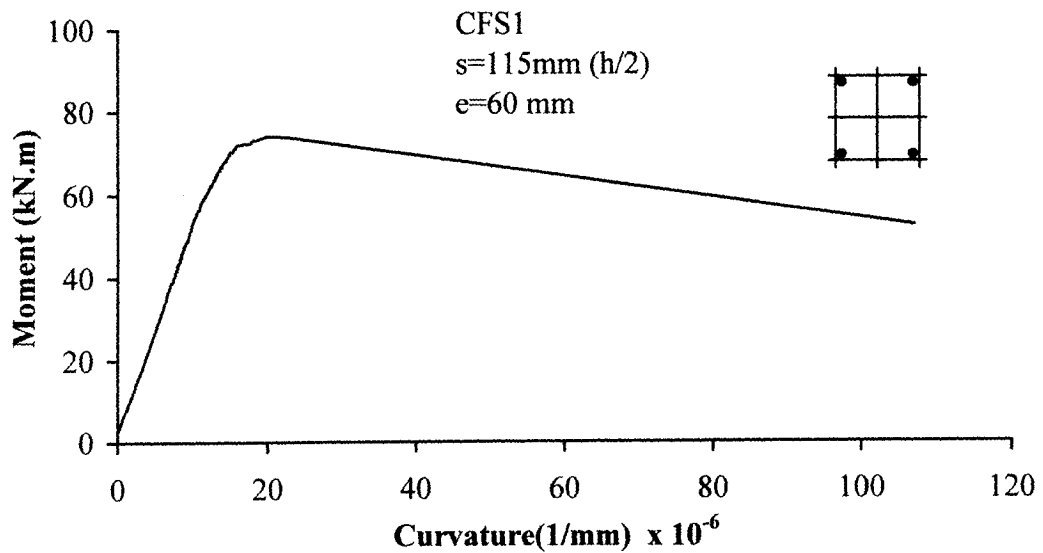
(c) Moment-Strain Relationship of CFRP Grids

Figure 2.14: Test Results of Column CFS1



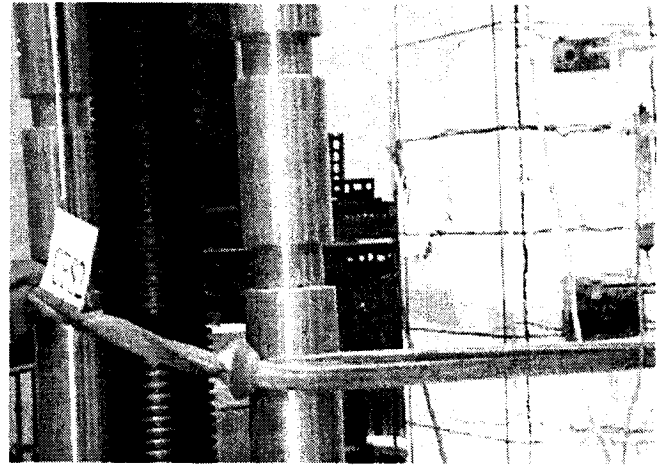
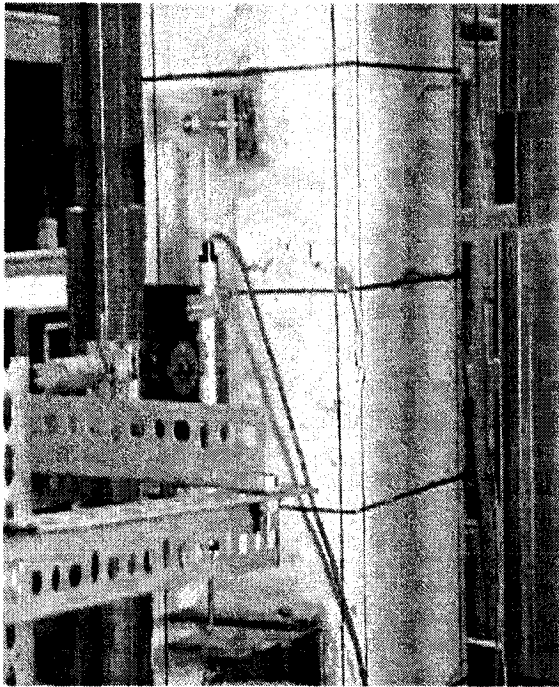


(d) Moment- Strain Relationship of Extreme Fiber of Concrete

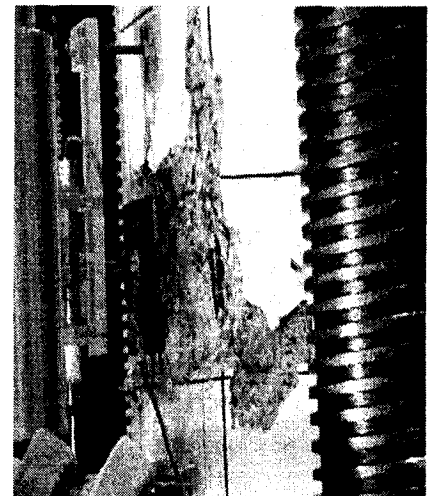


(e) Moment-Curvature Relationship

Figure 2.14(Cont.): Test Results of Column CFS1

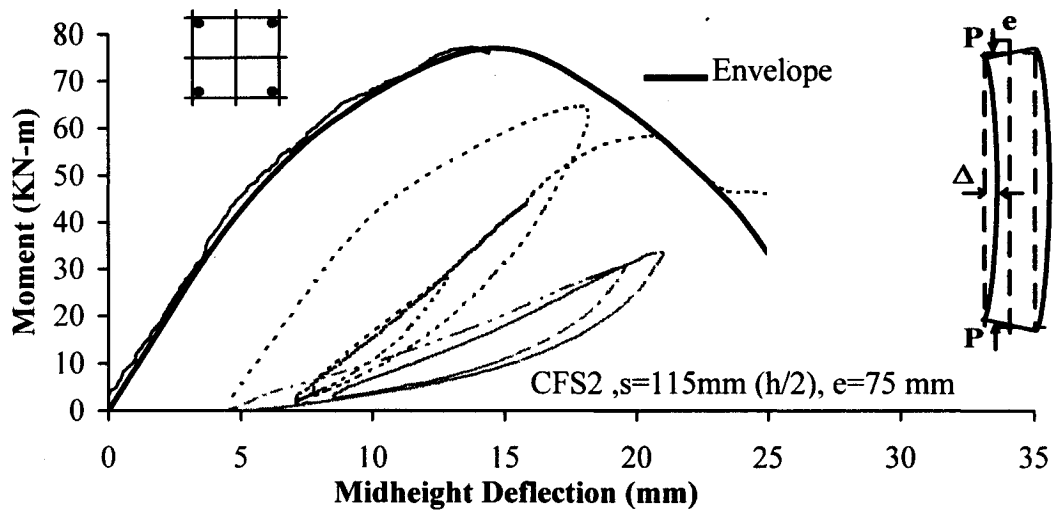


(a) At 500 kN Load

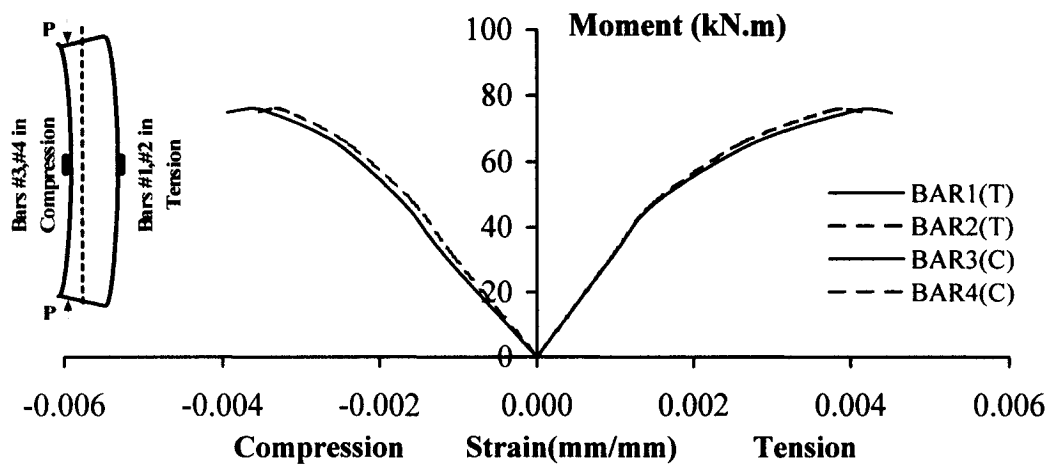


(b) After Crushing

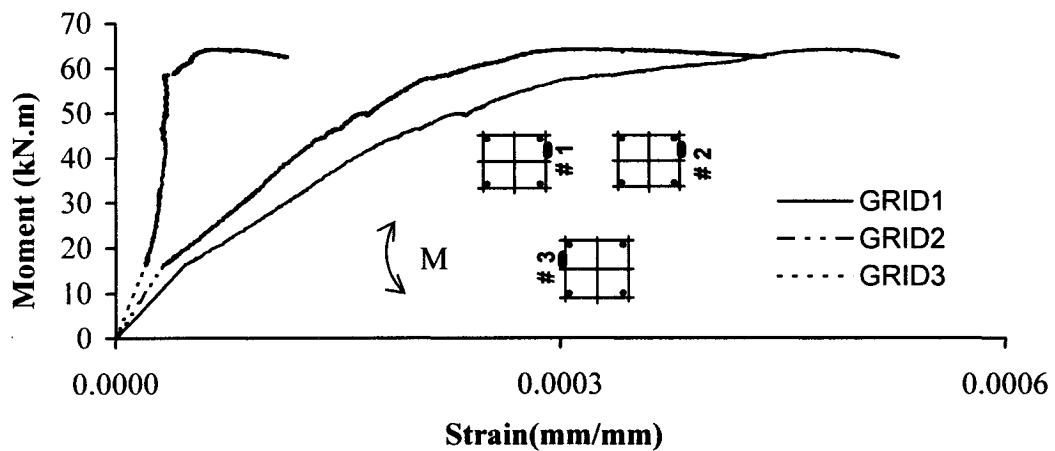
Figure 2.15: Column CFS2 at Different Stages of Loading



(a) Moment- Midheight Deflection Relationship

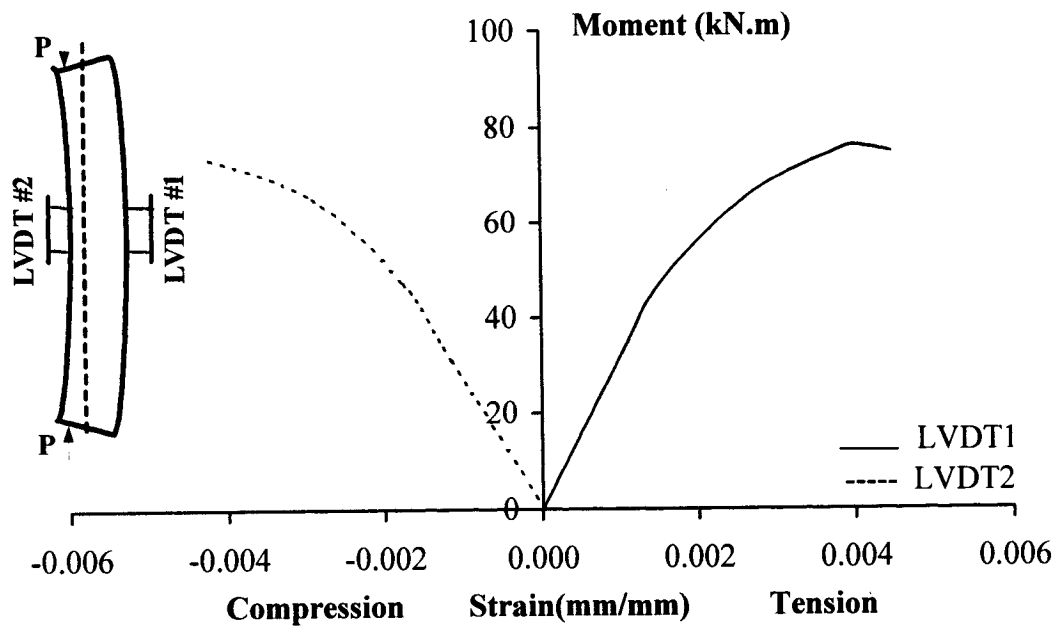


(b) Moment-Strain Relationship of CFRP Bars

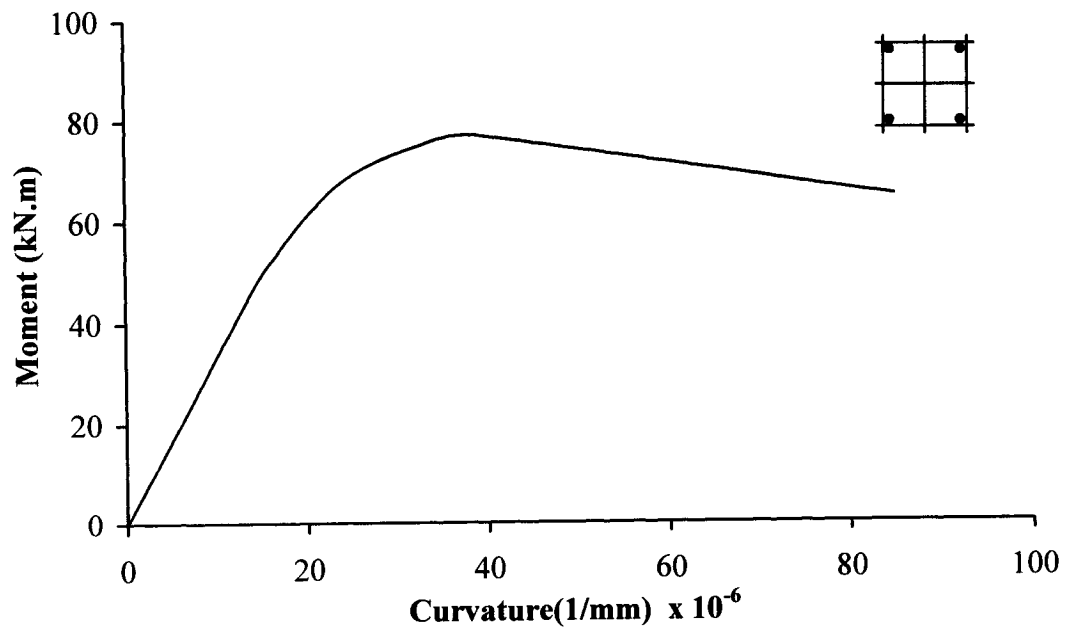


(c) Moment-Strain Relationship of CFRP Grids

Figure 2.16: Test Results of Column CFS2

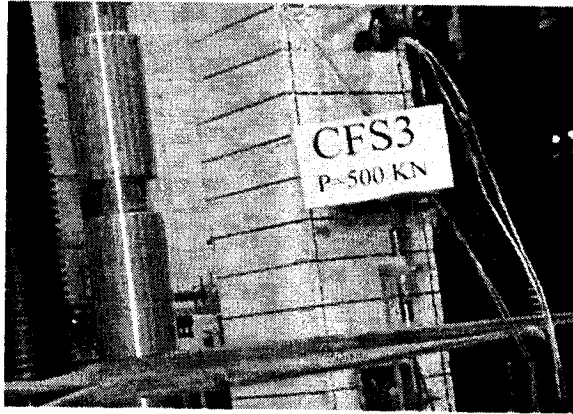


(d) Moment- Strain Relationship of Extreme Fiber of Concrete

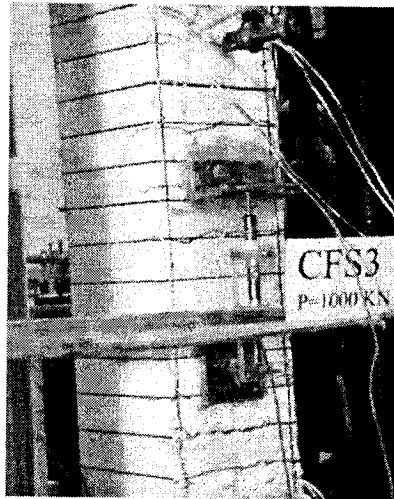


(e) Moment-Curvature Relationship

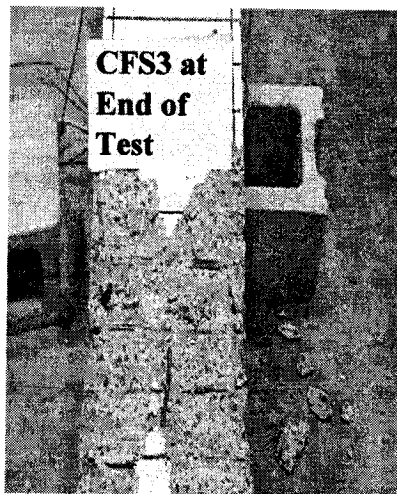
Figure 2.16(Cont.): Test Results of Column CFS2



(a) At 500 kN Load

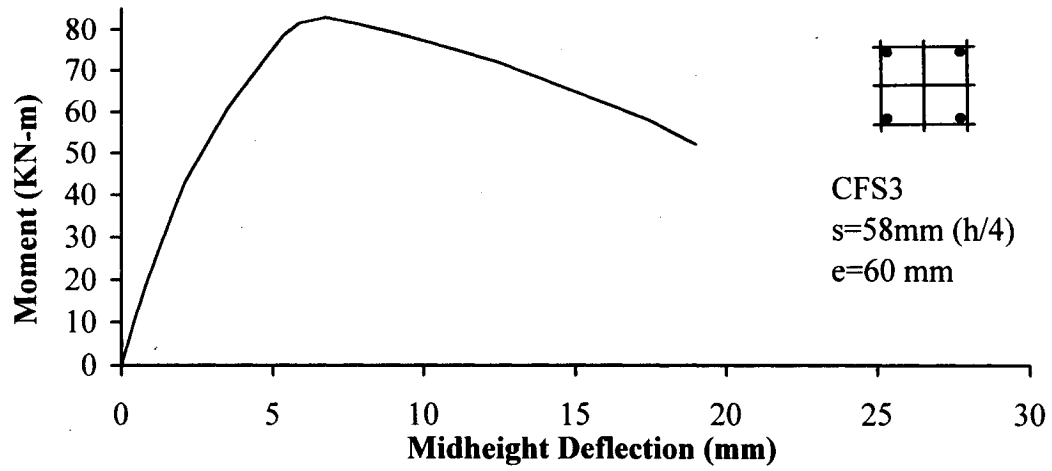


(b) At 1000 kN Load

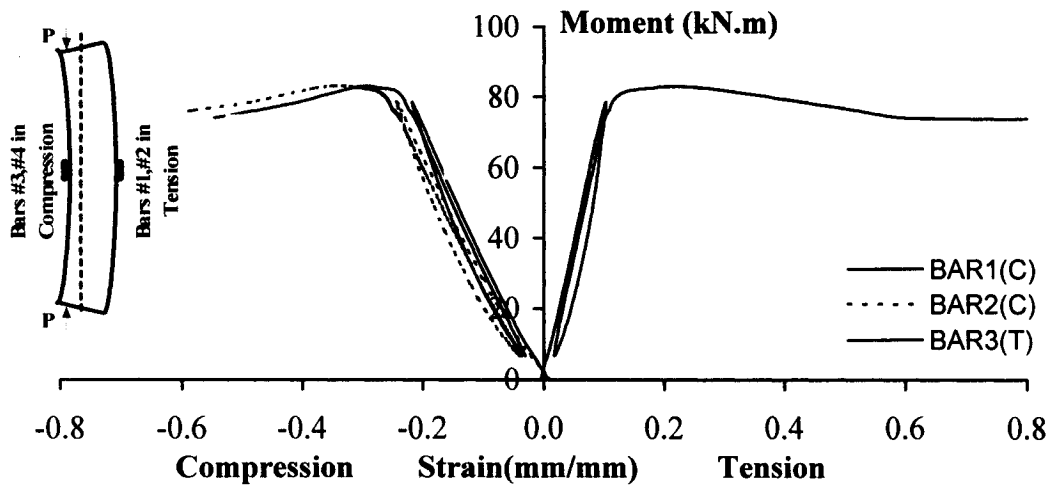


(c) After Failure

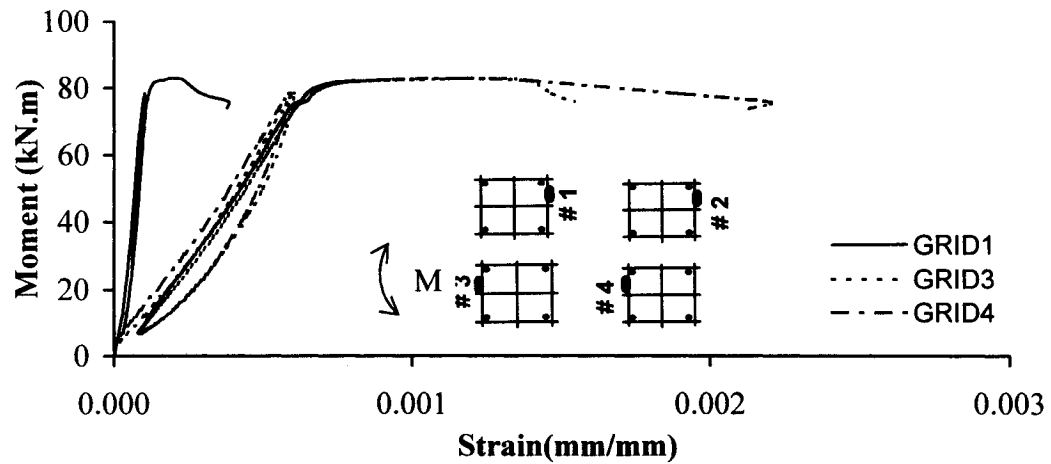
Figure 2.17: Column CFS3 at Different Stages of Loading



(a) Moment- Midheight Deflection Relationship

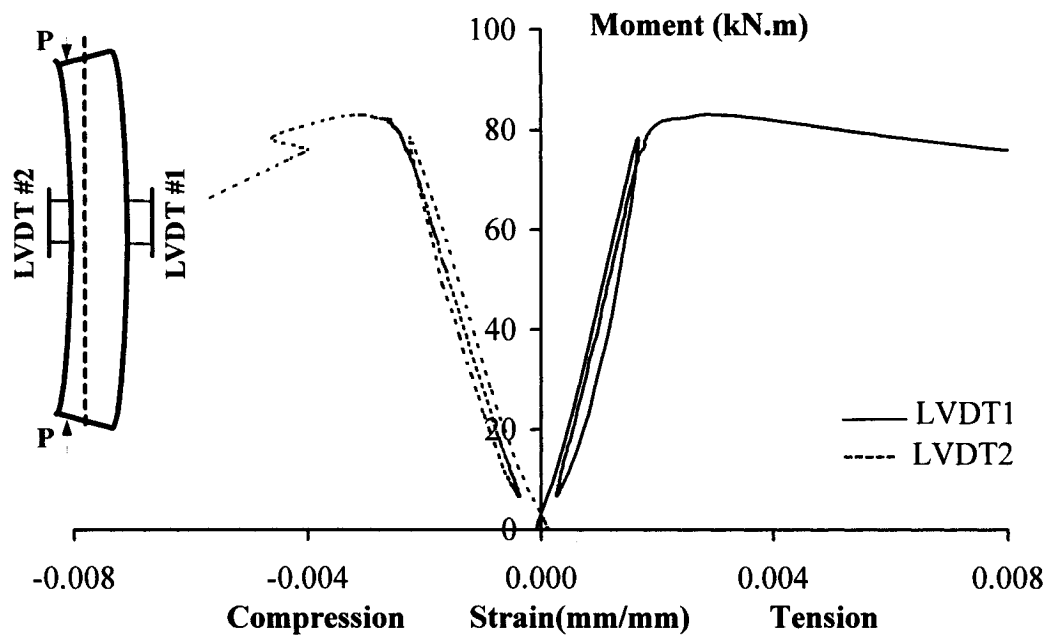


(b) Moment-Strain Relationship of CFRP Bars

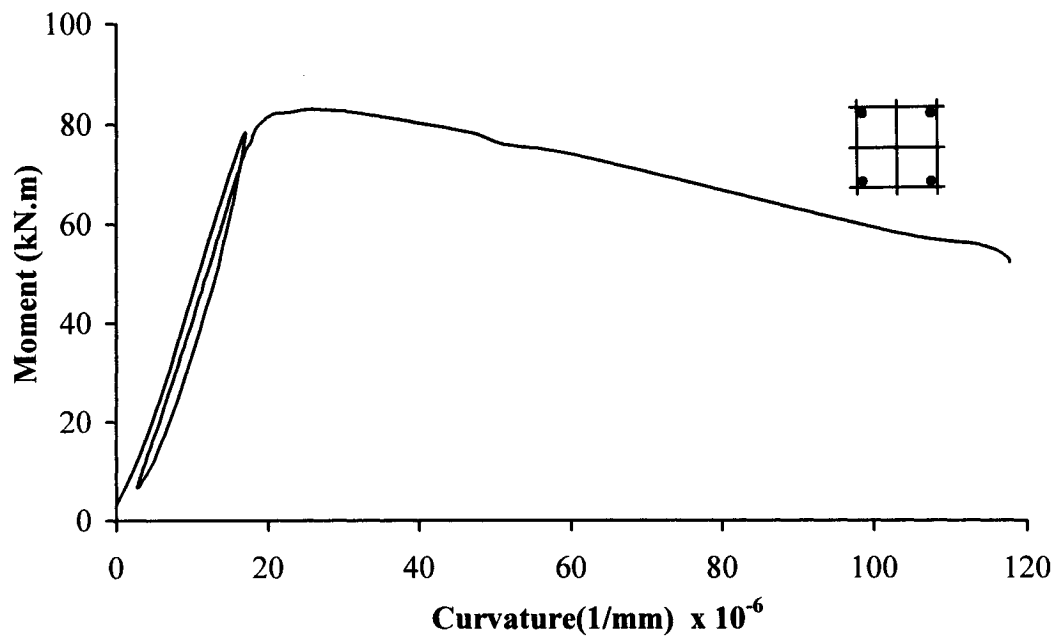


(c) Moment-Strain Relationship of CFRP Grids

Figure 2.18: Test Results of Column CFS3

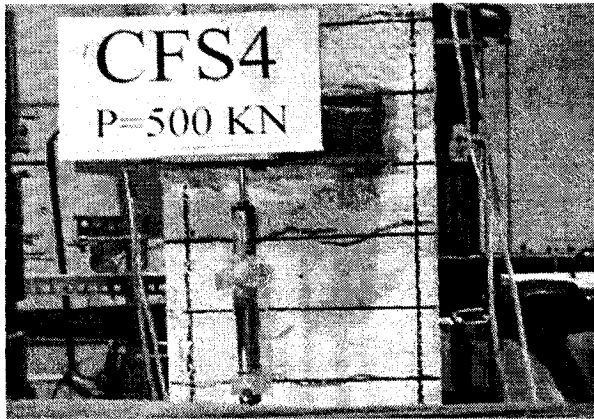


(d) Moment- Strain Relationship of Extreme Fiber of Concrete



(e) Moment-Curvature Relationship

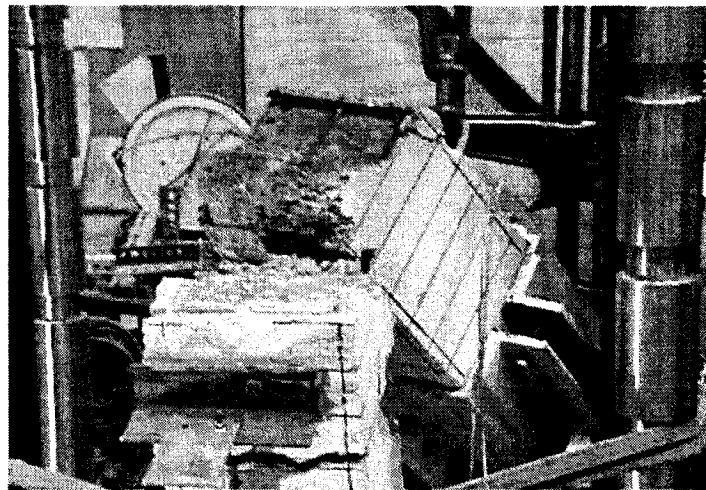
Figure 2.18(Cont.): Test Results of Column CFS3



(a) At 500 kN Load



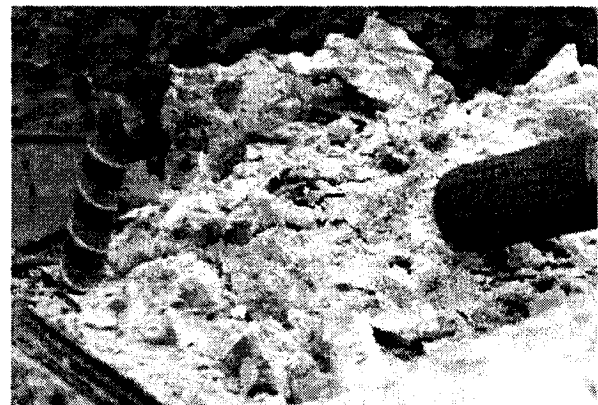
(b) At 800 kN Load



(c) After Crushing



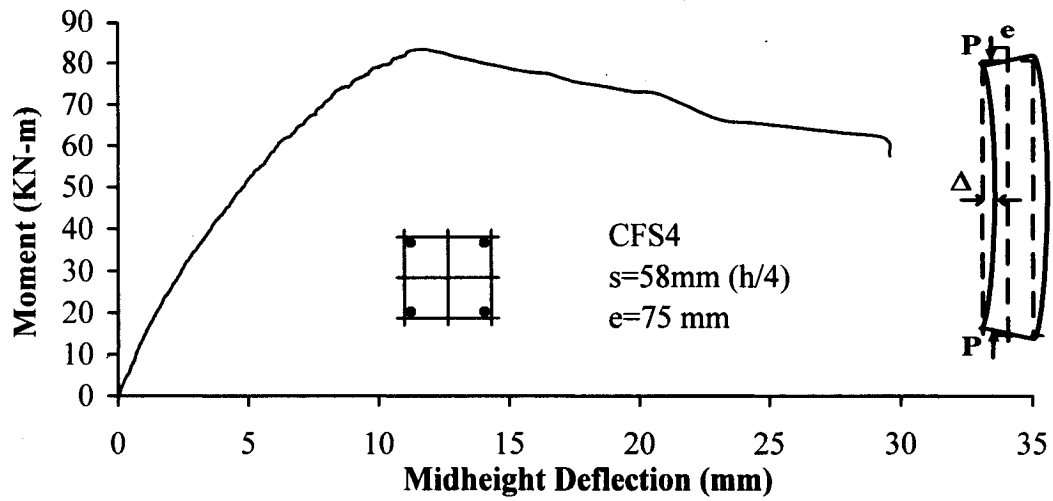
(e) Rupturing in Tension



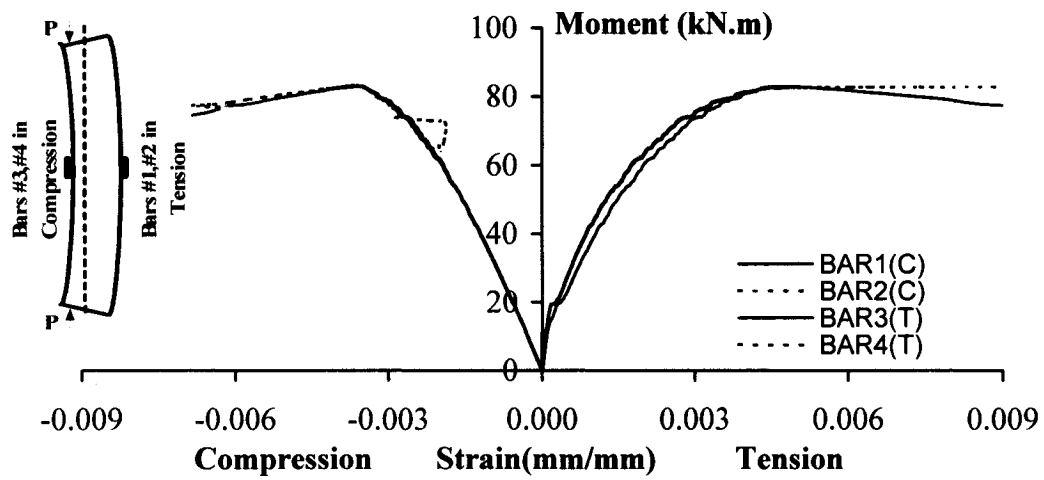
(f) Buckling in Compression

Figure 2.19: Column CFS4 at Different Stages of Loading

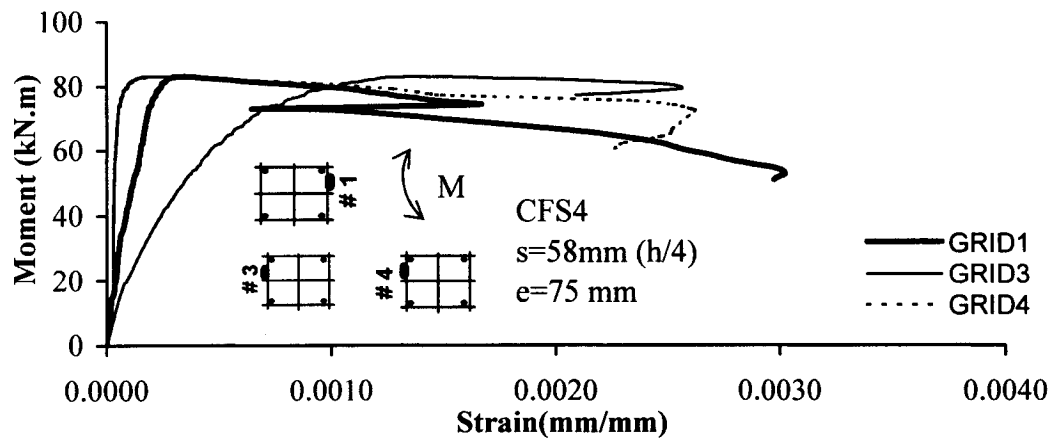




(a) Moment- Midheight Deflection Relationship

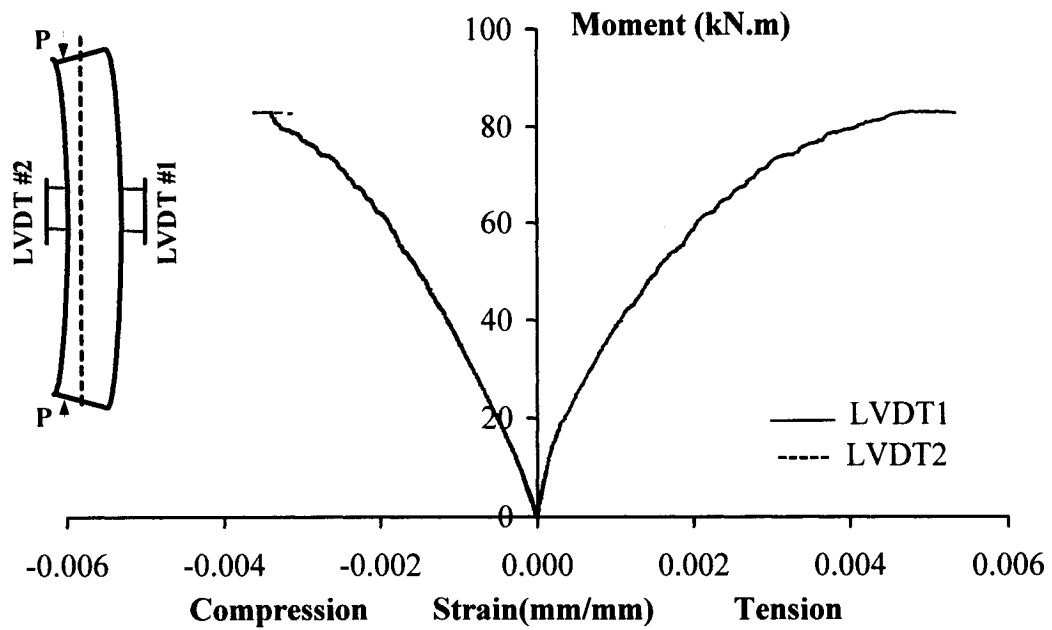


(b) Moment-Strain Relationship of CFRP Bars

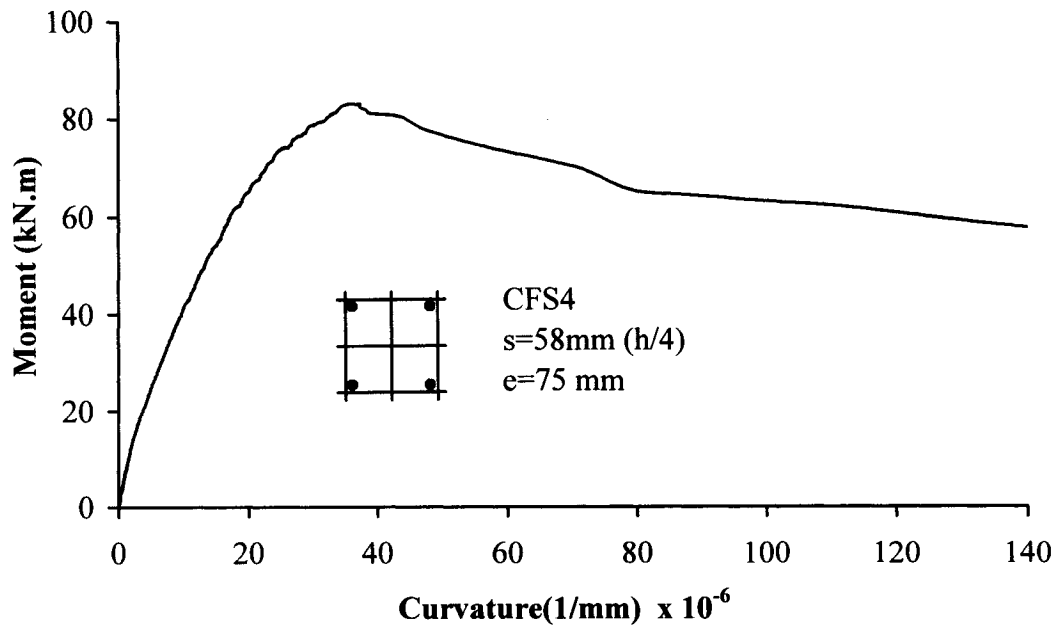


(c) Moment-Strain Relationship of CFRP Grids

Figure 2.20: Test Results of Column CFS4

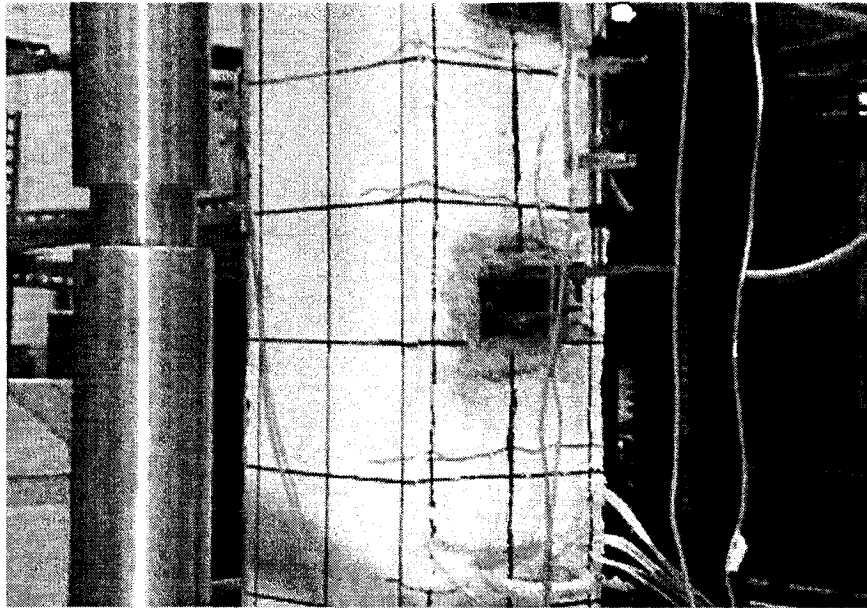


(d) Moment- Strain Relationship of Extreme Fiber of Concrete

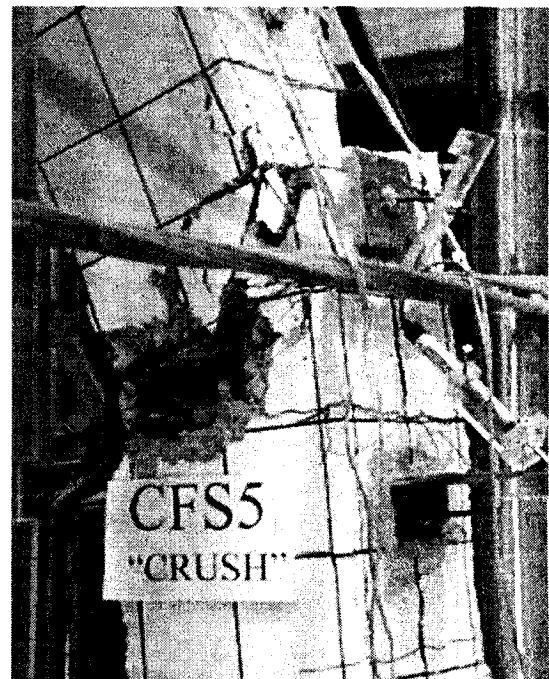
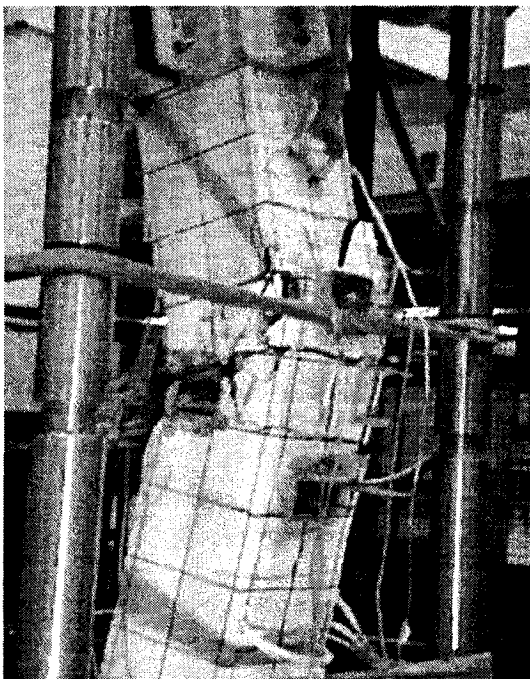


(e) Moment-Curvature Relationship

Figure 2.20(Cont.): Test Results of Column CFS4

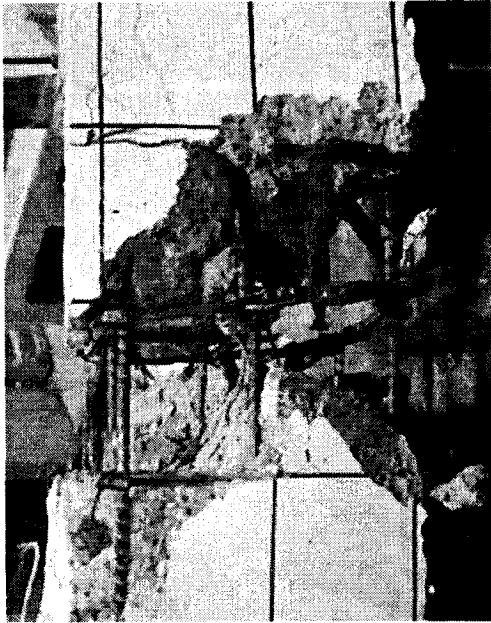


(a) At 500 kN Load



(b) After Crushing

Figure 2.21 : Column CFS5 at Different Stages of Loading



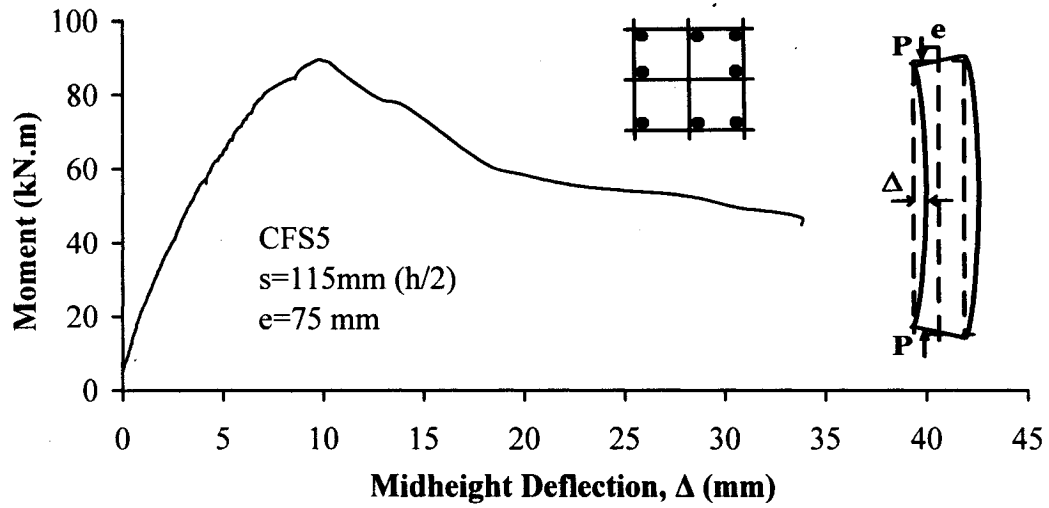
( c ) FRP Bars After Failure of Column



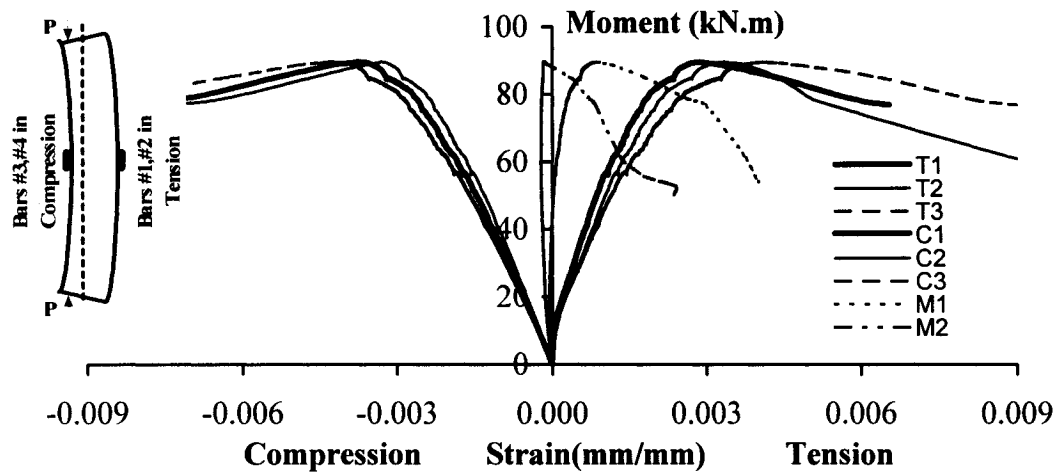
(d) Tension Rupturing of Bars

(e) Buckling of FRP Bars

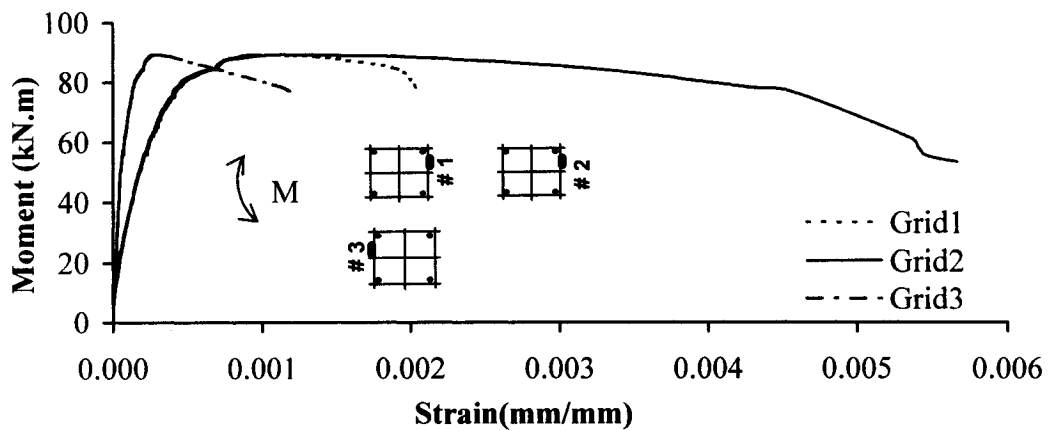
Figure 2.21(Cont.) : Column CFS5 at Different Stages of Loading



(a) Moment- Midheight Deflection Relationship

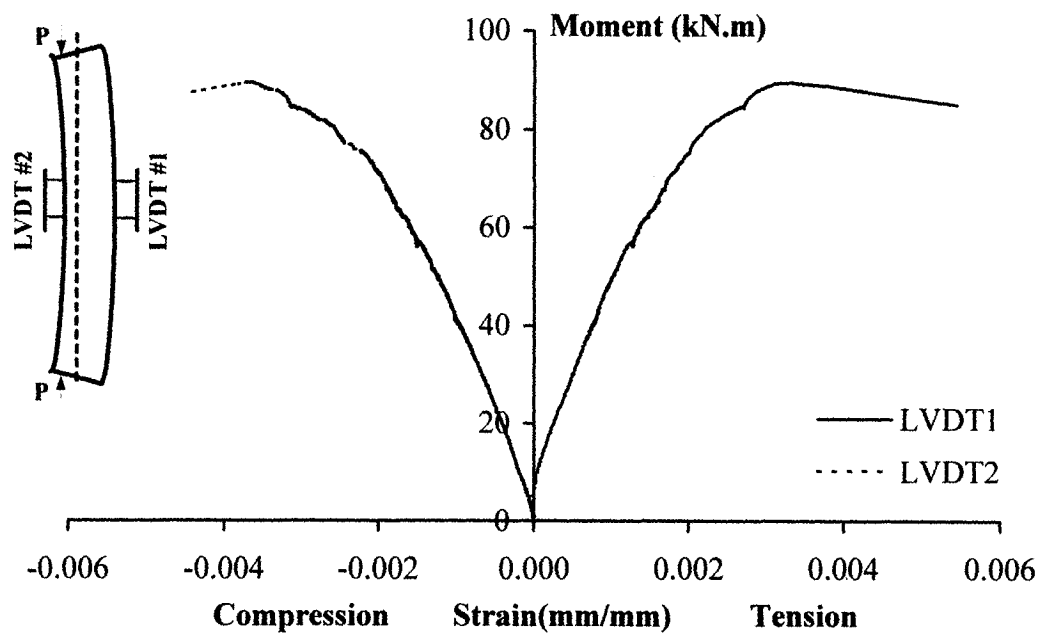


(b) Moment-Strain Relationship of CFRP Bars

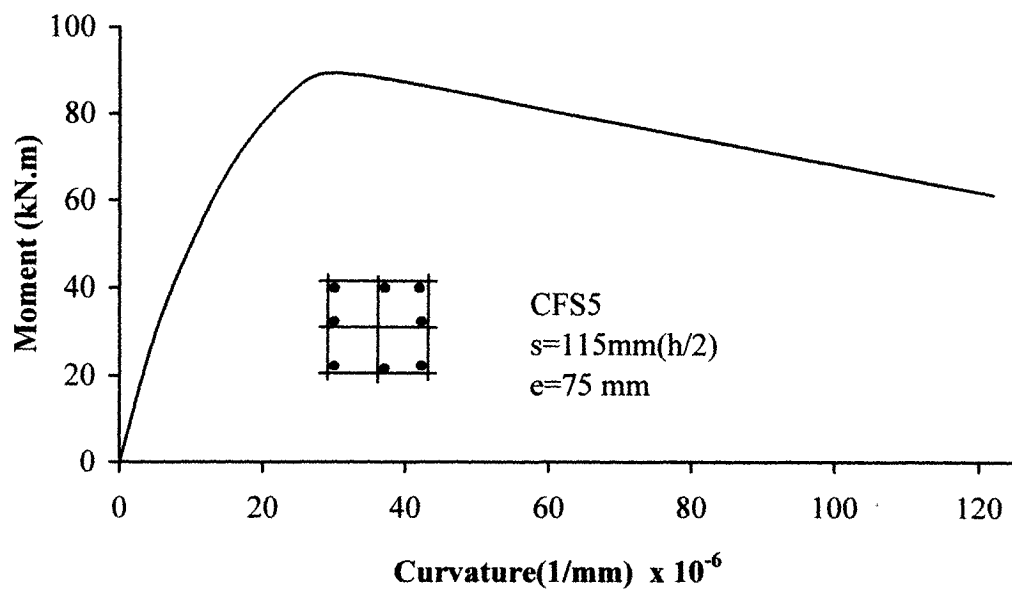


(c) Moment-Strain Relationship of CFRP Grids

Figure 2.22 : Test Results of Column CFS5

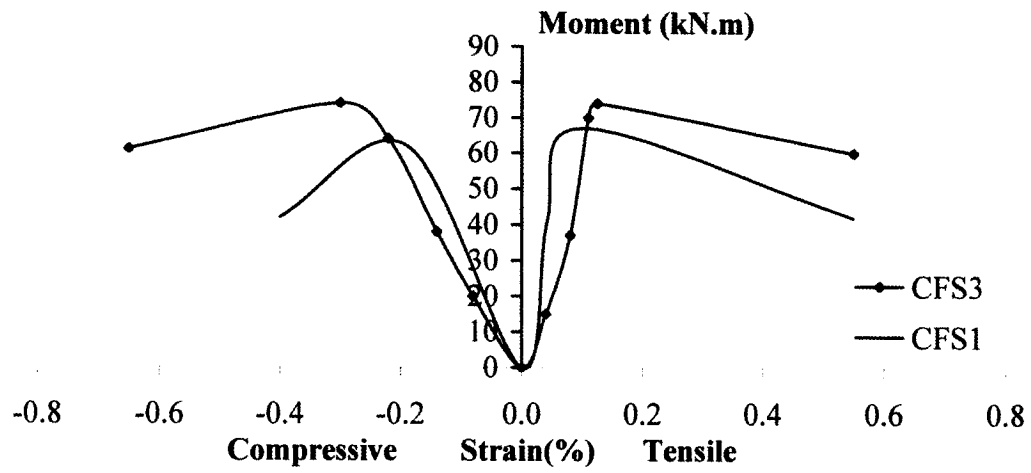
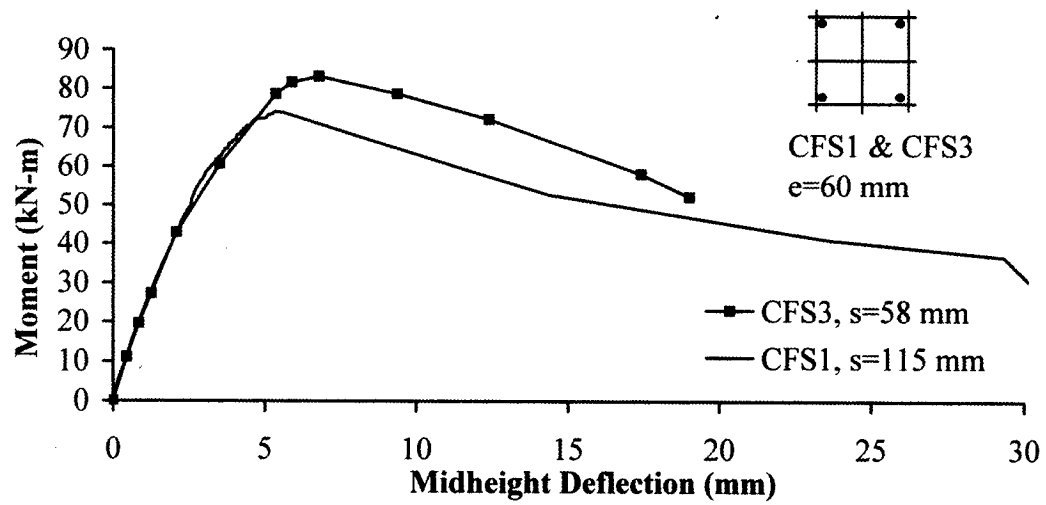


(d) Moment- Strain Relationship of Extreme Fiber of Concrete

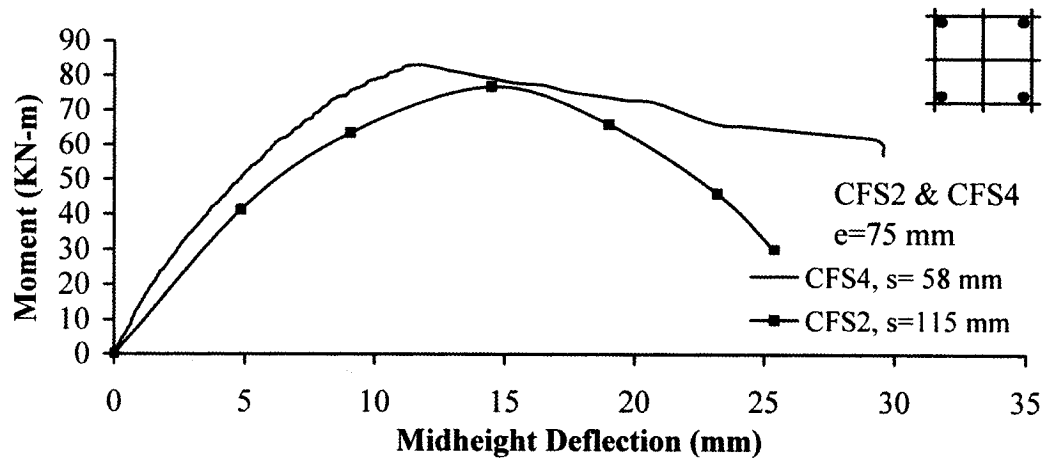


(e) Moment-Curvature Relationship

Figure 2.22 (Cont.): Test Results of Column CFS5

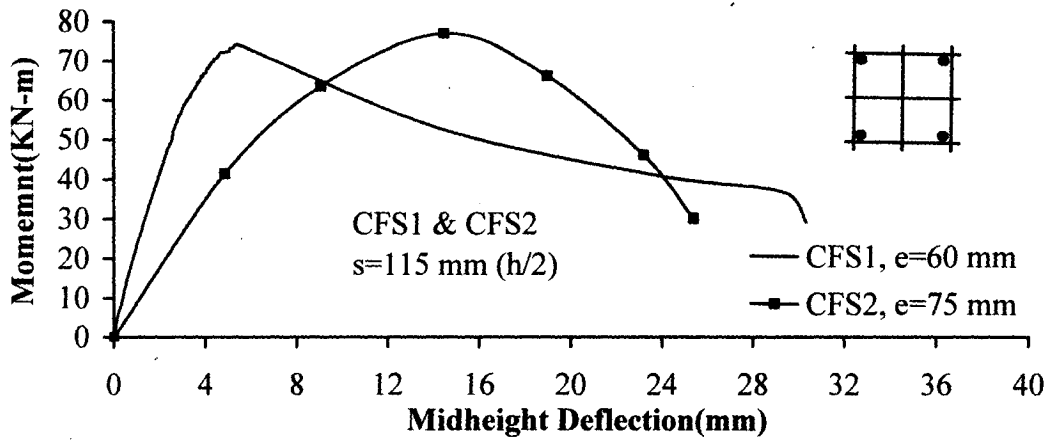


(a) Columns CFS1 with CFS3



(b) Columns CFS2 with CFS4

Figure 2.23: Comparison of Results for Effect of Spacing of Grids



(a) Columns CFS1 with CFS2

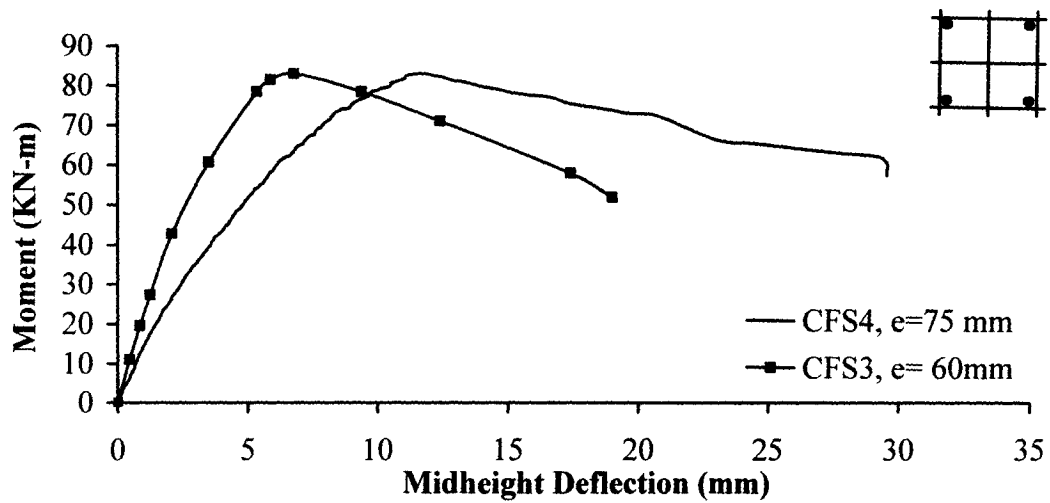


Figure 2.24: Comparison of Results for Effect of Eccentricity

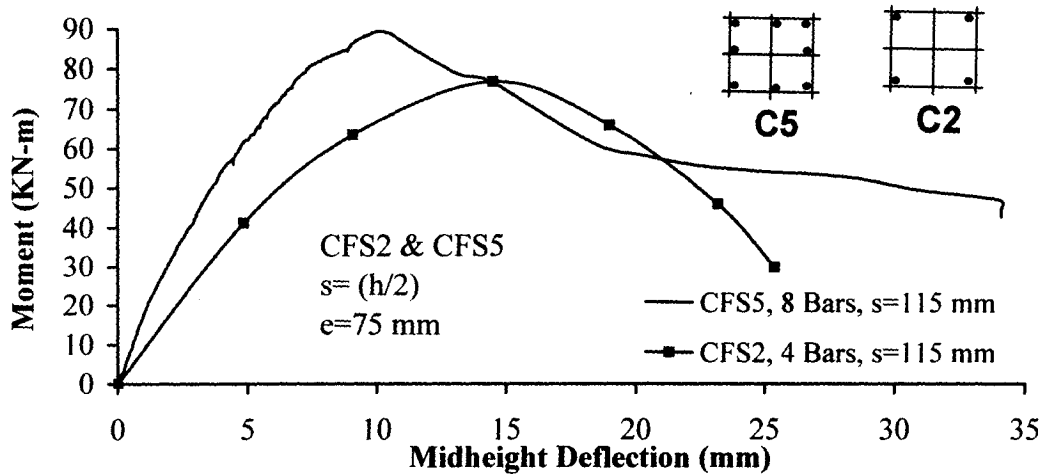


Figure 2.25: Comparison of Results of CFS2 & CFS5 for Effect of Arrangement



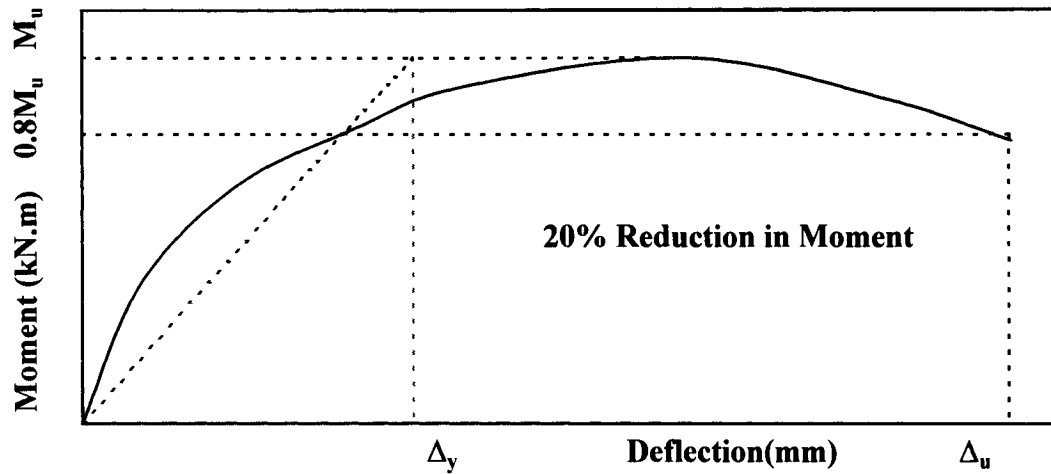
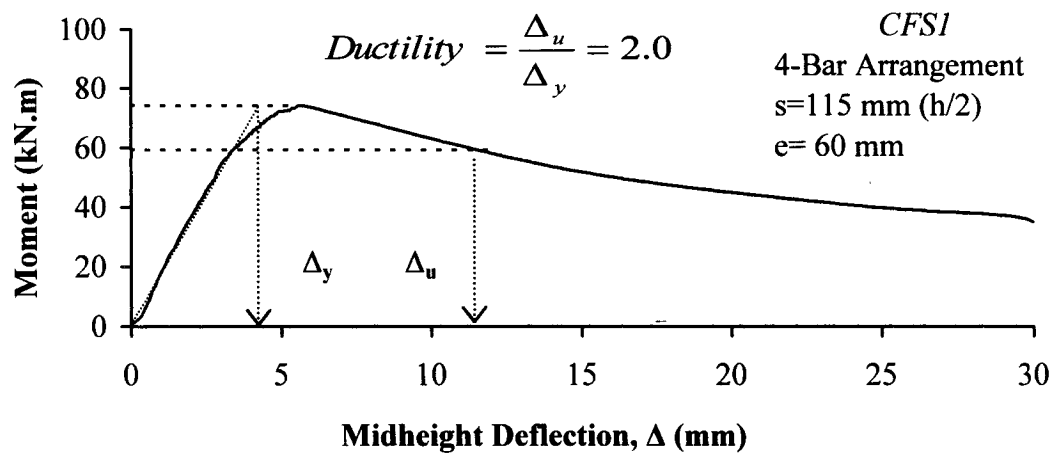
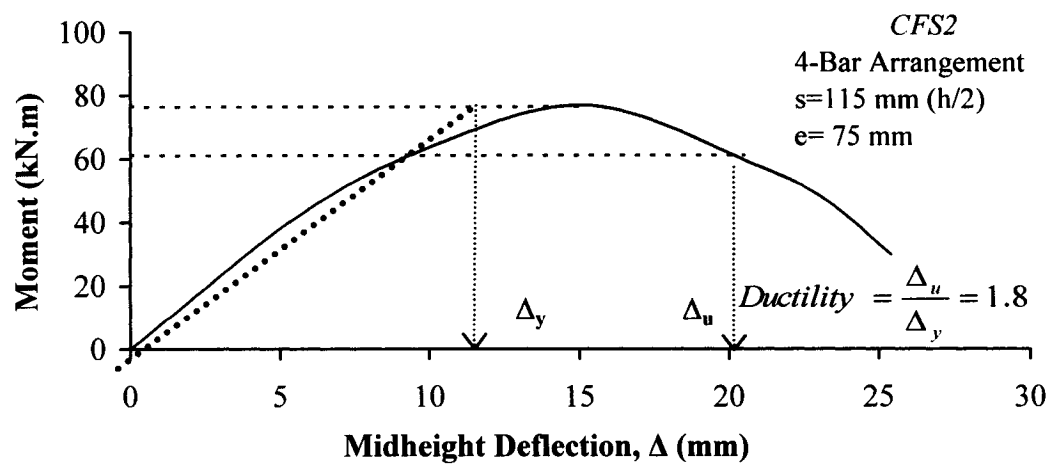


Figure 2.26: Definition of Ductility

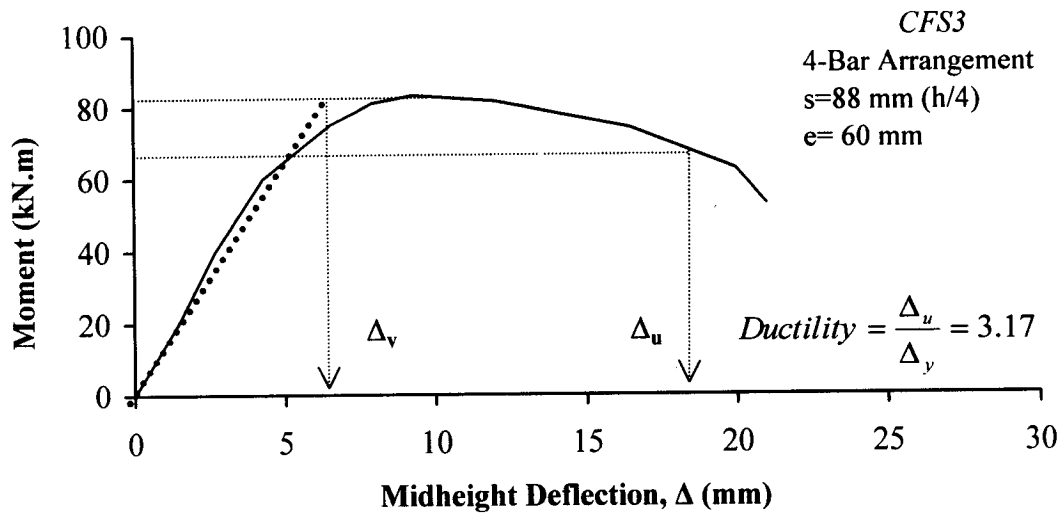


(a) Column CFS1

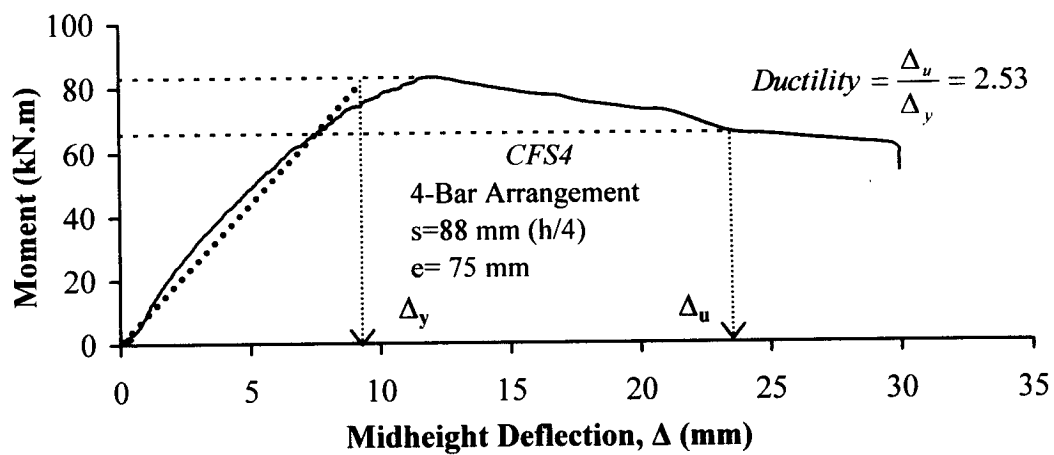


(b) Column CFS2

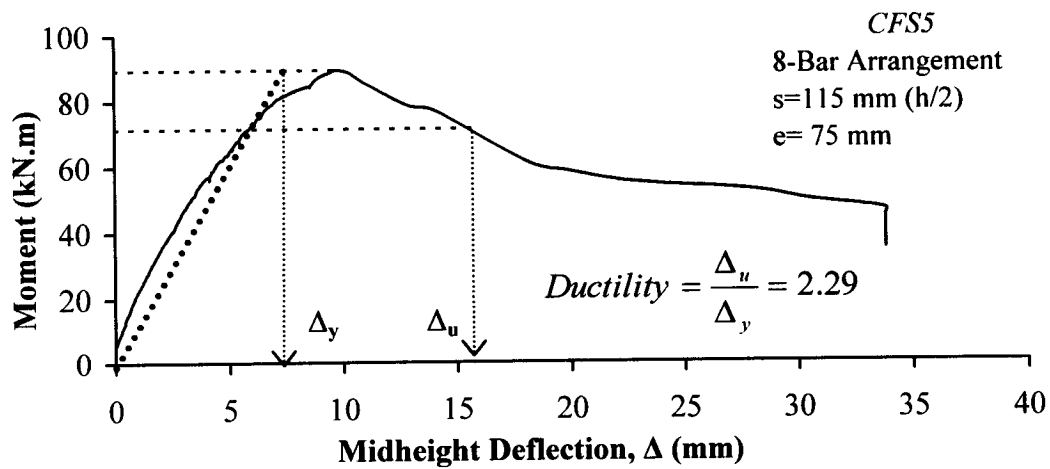
Figure 2.27: Experimental Displacement Ductility of Eccentric Loading Columns



(c) ColumnCFS3



(d) ColumnCFS4



(e) ColumnCFS5

Figure 2.27(Cont): Experimental Displacement Ductility of Eccentric Loading Columns

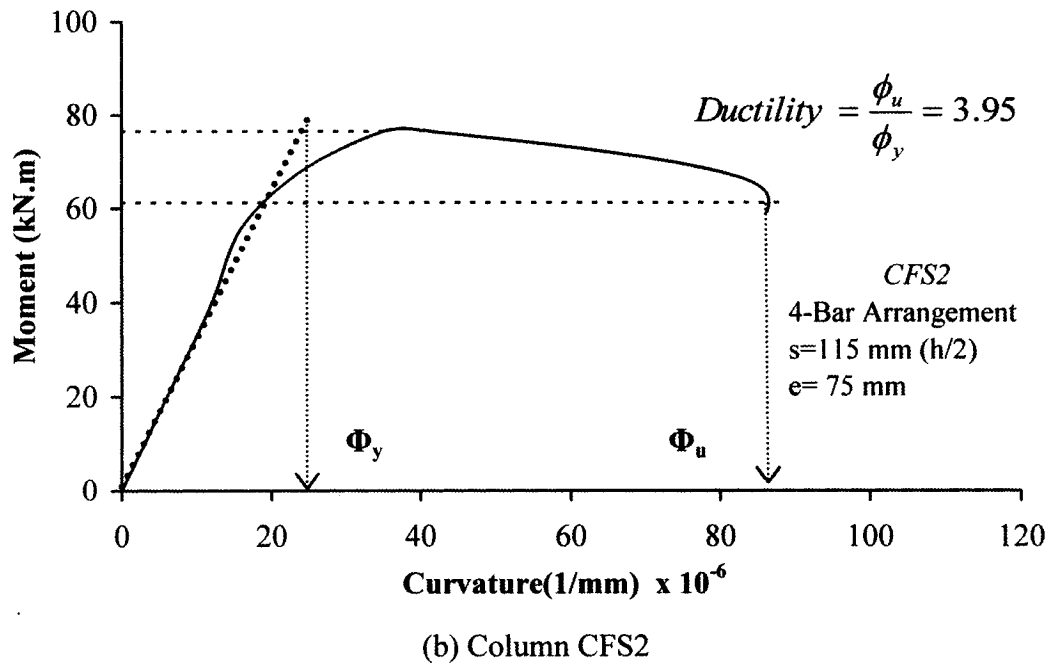
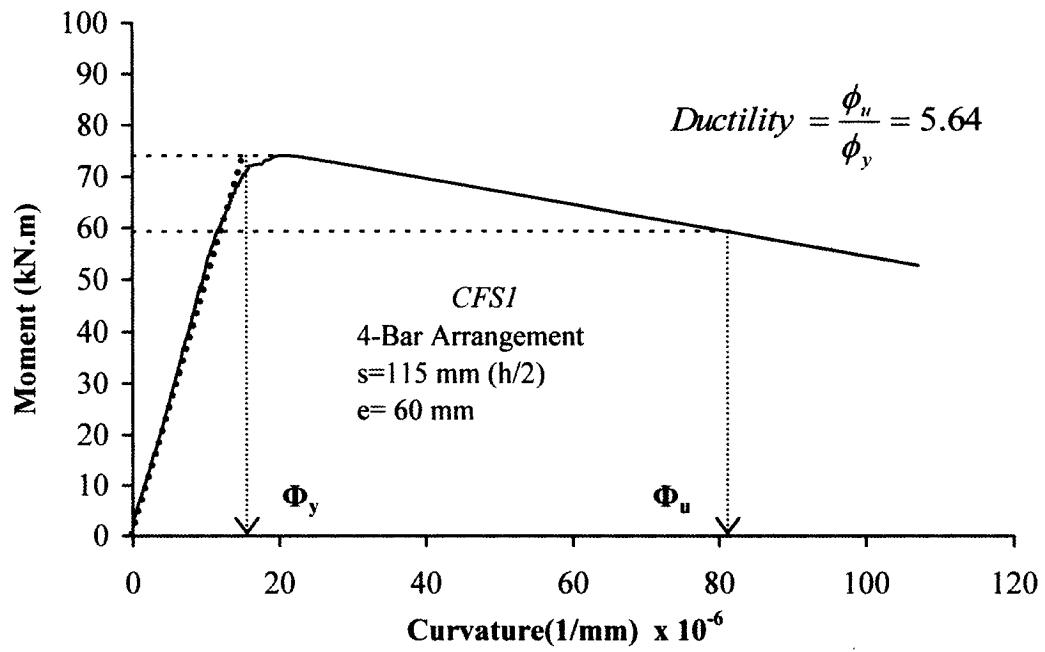


Figure 2.28: Experimental Curvature Ductility of Eccentric Loading Columns

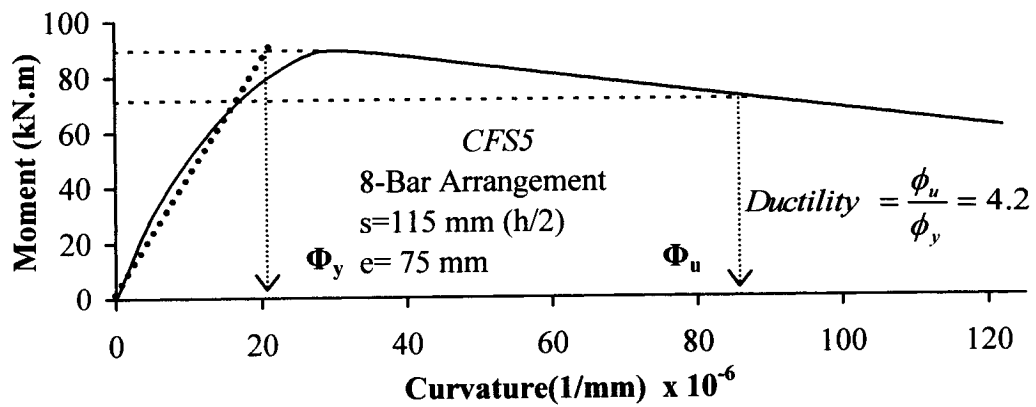
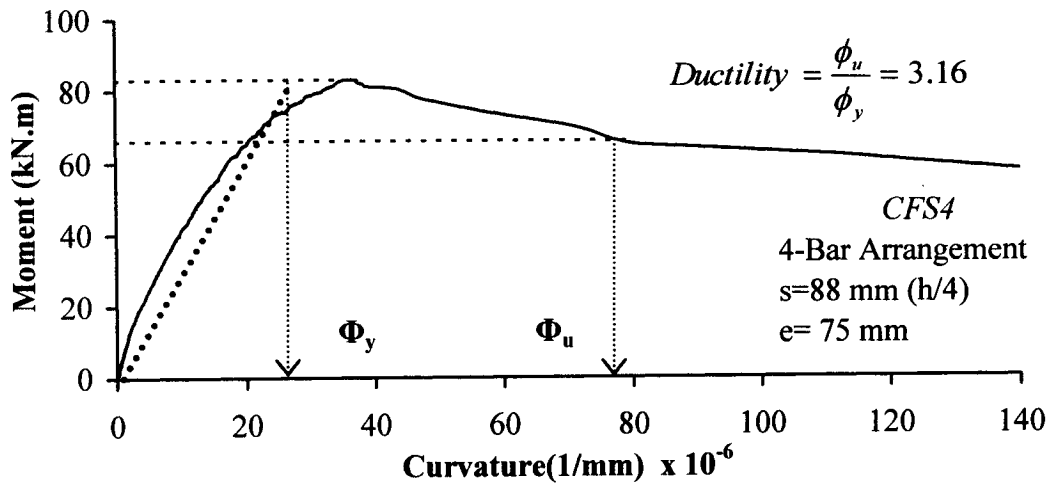
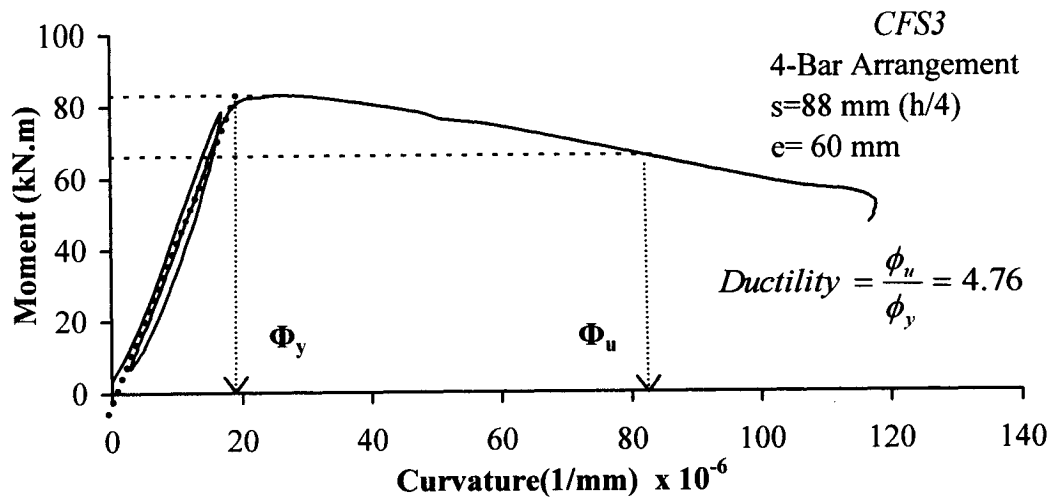


Figure 2.28(Cont): Experimental Curvature Ductility of Eccentric Loading Columns

## *Chapter 3*

# **CFRP Reinforced Concrete Cylinders Under Concentric Loading**

### **3.1 General**

A total of 11 cylinders were reinforced with two different types of CFRP bars as representatives of small-scale columns, forming Phase II of the experimental investigation. They were tested under monotonically increasing concentric compression until failure. Two main parameters were investigated, consisting of the spacing of transverse grids as lateral reinforcement, and the number and arrangement of longitudinal bars. The primary purpose of these small-scale specimens was to investigate the behaviour of CFRP bars in simple uniaxial compression. The details are presented in the following sections.

### **3.2 Test Specimens**

Reinforced cylinders of Phase II were prepared and tested in two stages. The first series consisted of eight standard cylinders with 150 mm diameter and 300 mm height, reinforced with 8-mm Leadline CFRP bars. The concrete strength was 47 MPa. Six of the cylinders had NEFMAC CFRP grids, cut to have single cells and used as transverse ties. The grid reinforcement had a 6 mm by 8 mm cross-section with either a 76 mm or 38 mm spacing along the height of cylinders. The other two had the longitudinal bars placed in the centre of cylinders, without any perimeter tie.

Three additional cylinders were reinforced with 9.5 mm diameter CFRP Pultrall bars to form the second series. These specimens were reinforced with either 8 or 12 bars tied by circular steel hoops made from 6.3 mm diameter smooth reinforcement with 100 mm spacing to control the stability of longitudinal bars. The concrete used had 35 MPa compressive strength. Figure 3.1 illustrates the details of reinforced cylinders tested in Phase II.

### **3.3 Material Properties**

Normal Portland Cement concrete batches were ordered from a local ready-mix supplier to cast eight cylinders in the first series, along with Phase I columns. The concrete used to cast the second series of cylinders was designed and mixed in the Concrete Laboratory. Maximum aggregate size used in all batches was 10 mm and the slump of concrete was 100 mm. The mix proportions of a cubic meter of concrete and material characteristics are specified in Tables 2.4 and 2.5 for the ready-mix concrete. A similar mix design and material properties were used in mixing the in-house concrete used for casting the second series of cylinders.

Plain concrete cylinders were cast from each batch of concrete to establish the stress-strain relationship, as well as monitoring strength gain at different ages. The cylinders were cast and tested as specified in the CSA Test Method A23.2-3c (ASTM C192-20a). They were cured with wet burlaps and plastic sheets immediately after demolding. The curing was done together with test specimens having the same concrete under identical environmental conditions. The cylinders were tested after 7, 14, 21 and 28 days. They were capped prior to testing with high strength sulphur compound, conforming to ASTM C617, to ensure the uniform application of load. A 2200 kN-capacity Fournery hydraulic testing machine was used to test cylinders in accordance with ASTM method C39, since this machine could be connected to the data acquisition system. A compressometer with three-point contacts was used with two Linear Variable Displacement Transducers (LVDTs) to measure concrete compressive strains. The results are summarized in Tables 3.1 and Figures 3.2.

The CFRP LeadLine bars and CFRP NEFMAC grids used in this phase of the investigation were identical to those used in Phase I columns, whose properties are discussed in detail in

Chapter 2, and will not be repeated here. The only other type of CFRP reinforcement used was Pultrall bars. These bars had a nominal diameter of 9.5 mm and an average tensile strength of 1480 MPa. The stress-strain relationship was linear up to failure with an elastic modulus of 120,000 MPa. The details of Pultrall bars and their properties can be found in Chapter 4 since the same bars were used in Phase III and IV tests reported in subsequent chapters where a closer examination of Pultrall bar properties is made, including coupon tests.

### **3.4 Instrumentation & Test Procedure**

Electric resistance strain gauges were placed on longitudinal bars. Two or three strain gauges were used in each specimen. The cylinders were tested using a concrete cylinder-testing machine. The specimens were capped with a high strength sulphur compound two hours prior to testing. A compressometer with three-point contacts was placed around each cylinder for longitudinal strain measurements. The axial deformations were measured within 200 mm gauge length using two Linear Variable Displacement Transducers (LVDTs) mounted on the compressometer on two opposite sides of cylinders. Load and deformation readings were recorded using a high-speed data acquisition system. The specimens were tested under monotonically increasing concentric compression until failure. Figures 3.3 and 3.4 illustrate the detail of FRP reinforced cylinders and the test process.

### **3.5 General Observations and Test Results**

#### **3.5.1 Test Series #1**

Reinforced cylinders in the first series were labelled CFC1 through CFC8. Cylinders CFC1 and CFC2 had 3 or 4 - 8 mm diameter LeadLine CFRP bars, respectively, placed in the centre of cylinders without any transverse reinforcement. This resulted in 0.85% and 1.15% reinforcement ratios. The photographs of CFC2 before and after the test are shown in Figure 3.5. The same figure also illustrates the performance of LeadLine bars inside the concrete after the removal of crushed concrete, exposing reinforcement at the end of testing. As can be observed, there was no distress in the bars and the bars performed well until after crushing of

the concrete. The load-strain relationships recorded by externally placed LVDTs and internally placed reinforcement strain gauges are plotted in Figure 3.6. The results indicate that the average concrete strain and the strain measured on CFRP bars show similar behaviour. The maximum-recorded compressive strain in CFRP bars varied approximately between 0.3% and 0.4%, confirming that the bars were effective in resisting compression until after the crushing of concrete.

Cylinders CFC3, CFC4, and CFC5 were reinforced with four LeadLine bars distributed around the perimeter of section, forming a square cage and producing 1.15% reinforcement ratio. Single cell CFRP grids placed at 75 mm spacing enclosed them. Figure 3.7 shows the photographs of Cylinder CFC3 before and after the test. The same figure also illustrates the performance of bars inside the concrete, at the end of testing having removed the crushed concrete, exhibiting that the bars remained intact without any sign of failure. The load-strain relationships recorded by externally placed LVDTs and internally placed reinforcement strain gauges are plotted in Figure 3.8. The results indicate that the average concrete strain and the strain measured on CFRP bars show similar behaviour up to peak load resistance. Beyond the peak, however, the average concrete strain increases with decreasing stress, while some of the bars unload at a faster rate than concrete. This can be explained by the loss of bond between the concrete and the CFRP bars, and the lack of transfer of applied load from concrete to reinforcement. The maximum-recorded compressive strain in CFRP bars was 0.38%, confirming that the bars were effective in resisting compression until after the crushing of concrete.

Cylinders CFC6, CFC7, and CFC8 had the same reinforcement arrangement as the previous three columns, with four bars supported by CFRP grids. This time, however, the spacing of grids was reduced to 38 mm. The load-strain relationships recorded by externally placed LVDTs and internally placed reinforcement strain gauges are plotted in Figure 3.9. The results indicate that the average concrete strain and the strain measured on CFRP bars show similar behaviour up to peak load resistance. Beyond the peak, however, the average concrete strain increased with decreasing stress, while the bars ceased to sustain additional compressive strains. As was the case in previous cylinder tests, this can be explained by the



loss of bond between the concrete and CFRP bars, and the lack of transfer of applied load from concrete to reinforcement. The stress-strain relationship of concrete showed slightly improved deformability beyond the peak, as compared to those discussed earlier, due to the improved concrete confinement provided by closely spaced transverse reinforcement. The maximum-recorded compressive strain in CFRP bars was 0.45%, confirming that the bars were able to sustain higher strains than the bars in previous cylinders due to the improved overall behaviour.

An important outcome of cylinder tests was the behaviour of CFRP bars in concentric compression. The bars maintained their integrity until after the failure of cylinders, without any sign of damage or delamination of carbon fibers, implying that they can be used as compression reinforcement until the surrounding concrete develops crushing.

Further evaluation of test results was made at peak cylinder resistance. The results for all cylinders are presented in Table 3.2 in terms of specimen properties, their load resistances and concrete strengths computed based on the gross area, core area and net area found by discounting the area occupied by FRP reinforcement. It was observed during testing that specimens with cover did not always develop complete spalling. There were signs of spalling in most cases with vertical splitting cracks forming on the surface, indicating less and less load being carried by the cover. Therefore, the effective area of concrete at peak load resistance varied between the gross area and the confined core plus some of the cover that had not spalled off completely. While it is difficult to quantify these values exactly, it can be seen in Table 3.2 that the results do reflect the effects of confinement and partial spalling of cover, while also illustrating the contribution of FRP bars to load resistance. The plain concrete strength of 47 MPa should be compared with average concrete strength in reinforced cylinders of 37.5 MPa based on gross area, which is too low; and 65.5 MPa based on core area, which is too high for the three identical cylinders with 75 mm (D/2) grid spacing. The contribution of Leadline FRP bars to overall cylinder resistance was 2%. A similar comparison can be made for confined cylinders with 38 mm (D/4) grid spacing, for which the average strength of core concrete, 70 MPa was 7% higher than that of the previous three specimens with 75 mm (D/2) grid spacing.

### 3.5.2 Test Series #2

The cylinders in Series 2 were labelled as CFC9 through CFC11 and were reinforced with either 8 or 12 CFRP Pultrall bars with a 9.5 mm nominal bar diameter. This resulted in 3.2 % or 4.8 % reinforcement ratios. Two cylinders had 8 bars and one cylinder had 12 bars. Figure 3.10 and 3.11 illustrate the photographs of two 8-bar cylinders before and after testing. Figure 3.12 shows the stress-strain relationship for two different plain concretes for these cylinders. Figures 3.13 and 3.14 show load-strain relationships for CFRP reinforced concrete cylinders. The strain gauge data indicated that the concrete and FRP had the similar behaviour until 0.2% strain, which corresponds to the onset of crushing in concrete. Subsequently, the FRP bars took little additional strain (increasing to approximately 0.26 %) while the concrete was able to sustain up to approximately 0.75% strain in compression before it completely crushed.

One of the objectives of testing reinforced cylinders was to investigate the behaviour and failure mode of CFRP bars in concentric compression. The Pultrall bars used maintained their integrity until after the failure of cylinders, without any sign of damage or delamination of carbon fibers, implying that they can be used as compression reinforcement until the surrounding concrete develops crushing.

Further evaluation of test results was made at peak cylinder resistance. The results for the three cylinders tested in Series #2 are included in Table 3.2 in terms of specimen properties, their load resistances and concrete strengths computed based on the gross area, core area and net area found by discounting the area occupied by FRP reinforcement. The cylinders in this series had only 15 mm clear cover. This thickness of concrete shell is very thin as compared to the cover thickness used in actual structural members. The shell concrete was observed to spall off at or near the peak load, almost entirely. Therefore, the concrete strength values based on the net area of the core reflect the actual behaviour more closely. The plain concrete strength of 26 MPa should be compared with average concrete strength of the two 8-bar reinforced cylinders of 36.7 MPa based on the net area of core concrete. This comparison indicates that the core concrete was confined even though there was large tie spacing of 100 mm, while the FRP reinforcement resisted 9.5% of total applied load. Similar comparisons

can be made for CFC11, which had 12 Pultrall bars. In this case the core concrete strength was computed to be 38.1 MPa, based on the core area, about 40% above the expected concrete strength of 26.7 MPa. The contribution of Pultrall CFRP bars to overall load resistance was 12%. It should be noted, however, that the contribution of bars to load resistance was computed from measured strains by using elastic modulus determined by tension coupon tests. This modulus of elasticity may be lower in compression, as further discussed in Chapter 6 where analytical models are developed for material behaviour.

### 3.5.3 Effect of Reinforcement

The test data was analyzed in an effort to shed more light into the behaviour of reinforced cylinders under concentric compression, particularly the role of CFRP bars. This was felt necessary because the strength of plain concrete turned out to be higher than that of reinforced cylinders. The concrete in the entire gross cross-sectional area provided the concentric load resistance of plain concrete cylinders. Therefore, the strength was computed by dividing the applied load by the gross cross-sectional area of a cylinder. Reinforced cylinders, on the other hand, consisted of concrete and longitudinal FRP bars. Some specimens had transverse reinforcement and other did not. Those with transverse reinforcement often suffered from cover spalling near the peak, leaving mostly the core concrete effective in resisting load. In this case the applied load is carried by the core concrete, with contributions from the longitudinal reinforcement and some from unspalled cover. This point is illustrated in Table 3.2 by calculating concrete strength on the basis of gross area and core area, recognizing that in the majority of cases the behaviour was in between the two extreme cases. This was perhaps not the case for the three cylinders tested in Series 2, all of which had a thin concrete cover of 15 mm thickness and observed to have suffered complete cover spalling at peak load.

In Table 3.2, the core area was computed as the area enclosed by the centreline of perimeter ties. The area of FRP bars ( $A_{FRP}$ ) was computed from their nominal diameters. However, for Pultrall bars, where the bars were coated with sand, the contribution of sand to strength was neglected. This resulted in the consideration of a gross area for Pultrall bars, including the area occupied by sand coating ( $A'_{FRP}$ ). The results presented in Table 3.2 indicate that the

FRP bars did resist compression until after the crushing of concrete and provided load resistance proportional to their area. The contributions to overall load resistance were computed using recorded strains and elastic modulus in compression. These resulted in 2% to 12% contributions, depending on the type of FRP bar used and the percentage of reinforcement in the section.

An important outcome of these small-scale specimen tests was the observed behaviour of FRP bars in concentric compression. This was one of the first tests conducted in this investigation and the behaviour of the composite material under compression was questionable. It was consistently observed, however, that the bars did not suffer any failure, neither in the form of material failure nor stability failure. In all cases they maintained their integrity until after the crushing of concrete. The strains recorded in compression ranged between 0.30% and 0.45% without any failure.

One of the objectives of the current investigation was to examine the effectiveness of FRP bars as reinforcement and also to assess bar behaviour in compression. There was some evidence of the possibility of CFRP bars developing lower elastic modulus in compression, as compared to those obtained by tension coupon tests. However, at this stage, the experimental evidence was not sufficient to suggest a reduced value. However, this aspect of bar behaviour is examined in detail in Chapter 6, in light of all the other test data compiled from all four phases of the experimental program.

Table 3.1: Variation of Concrete Strength with Age in Phase II Specimens  
(Reinforced Cylinders)

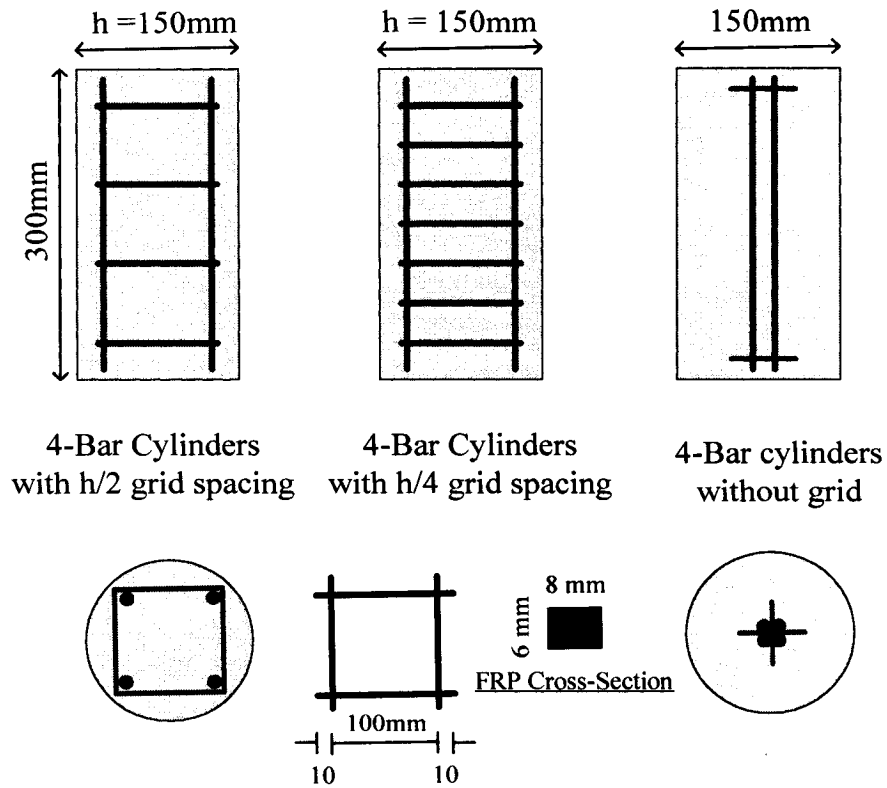
| Compressive Strength of Concrete in MPa |        |         |         |
|-----------------------------------------|--------|---------|---------|
| Age                                     | 7 days | 14 days | 28 days |
| <i>Series#1</i>                         |        |         |         |
| Cylinder 1                              | 29.1   | 37.4    | 47.3    |
| Cylinder 2                              | 28.8   | 37.3    | 47.2    |
| Cylinder 3                              | 30.5   | 38.1    | 47.3    |
| Average                                 | 29.5   | 37.6    | 47.25   |
| <i>Series#2</i>                         |        |         |         |
| Cylinder 1                              | 19.5   | 24.6    | 28.5    |
| Cylinder 2                              | 18.8   | 25.2    | 28.0    |
| Cylinder 3                              | 18.4   | 24.0    | 28.5    |
| Average                                 | 18.9   | 24.6    | 28.3    |

Table 3.2: Comparison of Concrete Strengths of Plain and Reinforced Cylinders of Phase II Specimens Based on Gross and Core Areas of Sections, Including the Contribution of FRP Bars.

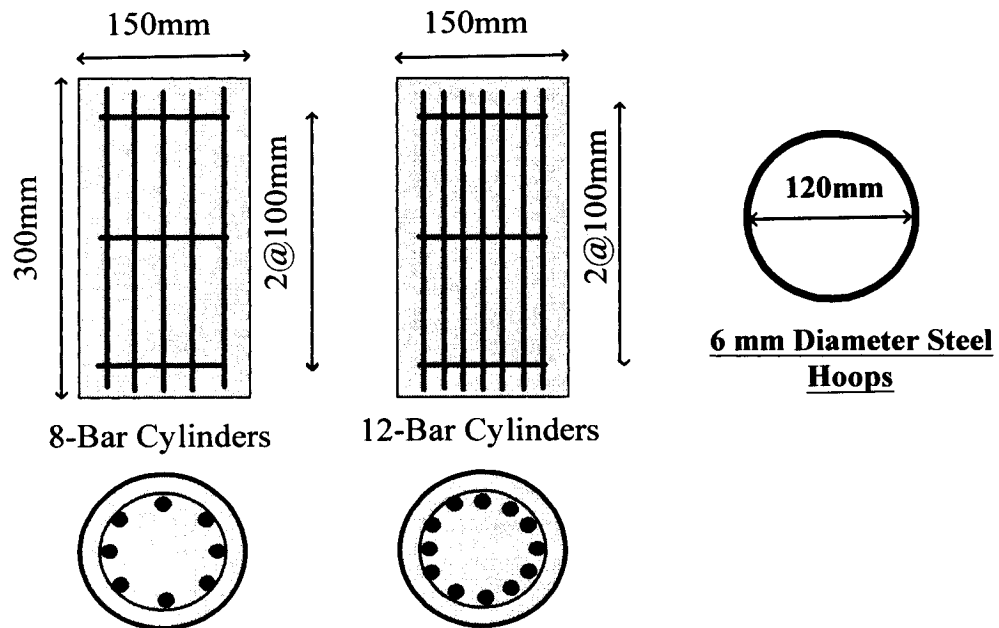
| Cylinder Labels | Arrangement      | Tie Spacing (mm) | Plain Concrete |          | Reinforced Cylinders        |                            |          |                      |                        |                                                   |                                                  |
|-----------------|------------------|------------------|----------------|----------|-----------------------------|----------------------------|----------|----------------------|------------------------|---------------------------------------------------|--------------------------------------------------|
|                 |                  |                  | $f'_c$ (MPa)   | $P$ (kN) | $\frac{P}{A_{Gross}}$ (MPa) | $\frac{P}{A_{Core}}$ (MPa) | $P$ (kN) | $\epsilon_{FRP}$ (%) | $A_{FRP} f_{FRP}$ (kN) | $\frac{P - A_{FRP} f_{FRP}}{A_{Gross} - A_{FRP}}$ | $\frac{P - A_{FRP} f_{FRP}}{A_{Core} - A_{FRP}}$ |
| Series # 1      |                  |                  |                |          |                             |                            |          |                      |                        |                                                   |                                                  |
| CFC1            | 3-Bar, at Centre | No Tie           | 47             | 850      |                             |                            | 595      | 0.25                 | 8.3                    | 32.7                                              |                                                  |
| CFC2**          | 4-Bar, at Centre | No Tie           | 47             | 850      |                             |                            |          |                      |                        |                                                   |                                                  |
| CFC3            | 4-Bar            | 75               | 47             | 850      | 35                          | 65                         | 650      | 0.29                 | 12.8                   | 35.6                                              | 64.9                                             |
| CFC4            | 4-Bar            | 75               | 47             | 850      | 35                          | 64                         | 640      | 0.235                | 13.4                   | 35.1                                              | 64                                               |
| CFC5            | 4-Bar            | 75               | 47             | 850      | 40                          | 70.5                       | 705      | 0.24                 | 10.6                   | 38.8                                              | 70.8                                             |
| CFC6            | 4-Bar            | 38               | 47             | 850      | 41                          | 73.7                       | 737      | 0.27                 | 11.9                   | 40.6                                              | 67.1                                             |
| CFC7            | 4-Bar            | 38               | 47             | 850      | 41.5                        | 75                         | 750      | 0.33                 | 14.6                   | 41.1                                              | 74                                               |
| CFC8            | 4-Bar            | 38               | 47             | 850      | 37.5                        | 68                         | 685      | 0.31                 | 13.7                   | 37.5                                              | 68.4                                             |
| Series # 2      |                  |                  |                |          |                             |                            |          |                      |                        |                                                   |                                                  |
| CFC9            | 8-Bar            | 100              | 26.1           | 472      | 25.4                        | 37.4                       | 450      | 0.25                 | 42                     | 23.6                                              | 35.6                                             |
| CFC10           | 8-Bar            | 100              | 26.1           | 472      | 26.5                        | 39                         | 480      | 0.27                 | 45.4                   | 25.2                                              | 37.9                                             |
| CFC11           | 12-Bar           | 100              | 26.7           | 484      | 27                          | 40                         | 498      | 0.24                 | 60.5                   | 25.3                                              | 38.1                                             |

Note: \*  $f_{FRP} = \epsilon_{FRP} \cdot E_{FRP}$ , Modulus of elasticity of LeadLine and Pultrall bars in compression are assumed to be 15% and 25% of that in tension.

\*\* Problems were encountered during testing this cylinder. The load resistance was lower than expected.



(a) Concrete Cylinders Reinforced with Leadline CFRP Bars and NERMAC Grids



(b) Concrete Cylinders Reinforced with Pultral CFRP Bars and Steel Hoops

Figure 3.1: Geometric Details of Phase II Specimens (FRP Reinforced Cylinders)

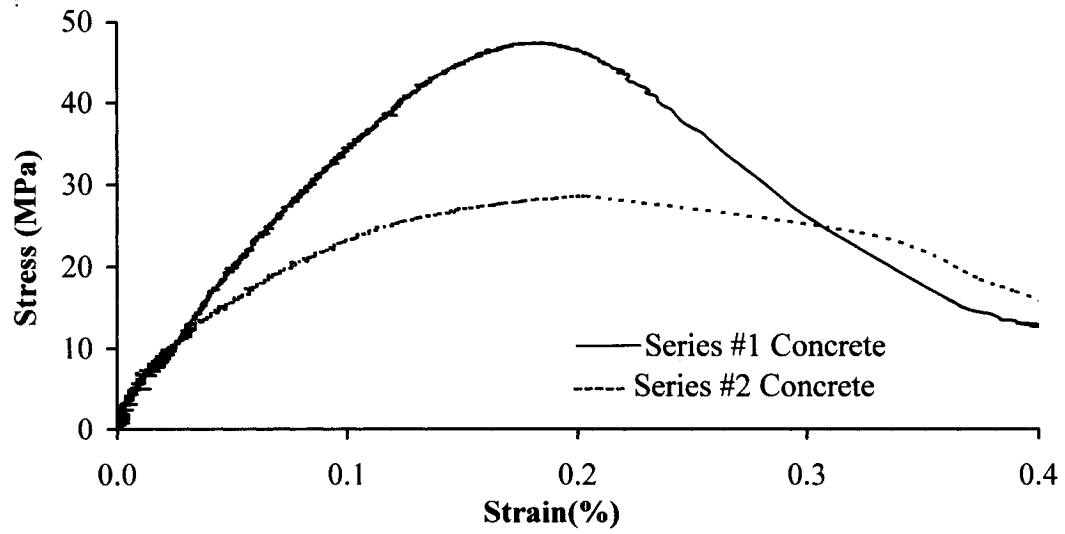
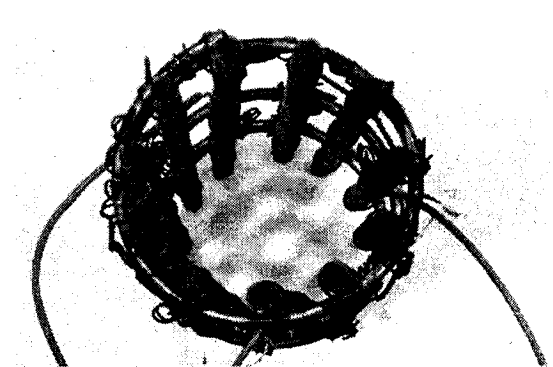
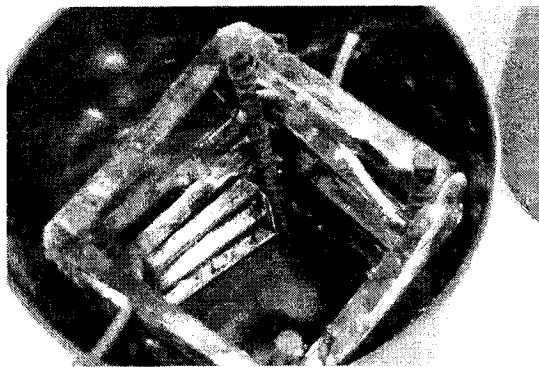
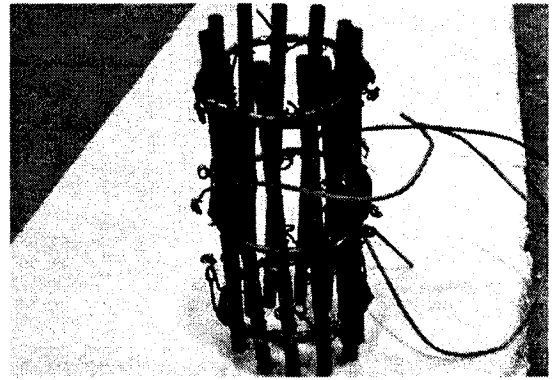
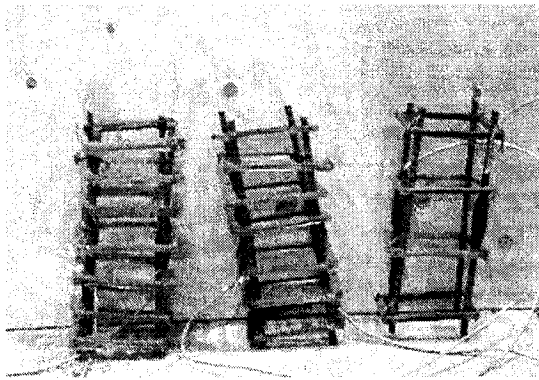


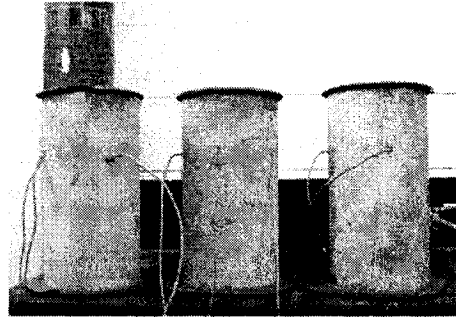
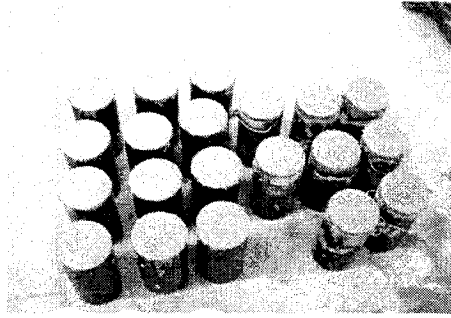
Figure 3.2: Stress-Strain Relationship for Plain Concrete in Phase II Specimens (Reinforced Cylinders)



(a) Preparation of Reinforcement Cages

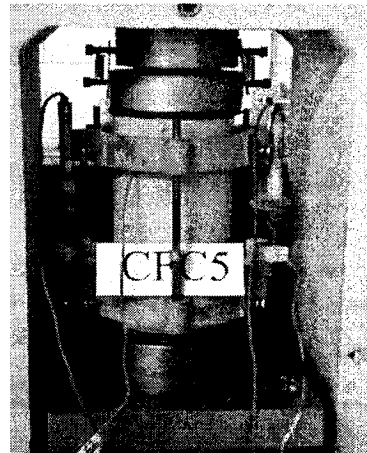
Figure 3.3: Preparation and Casting of FRP Reinforced Cylinders



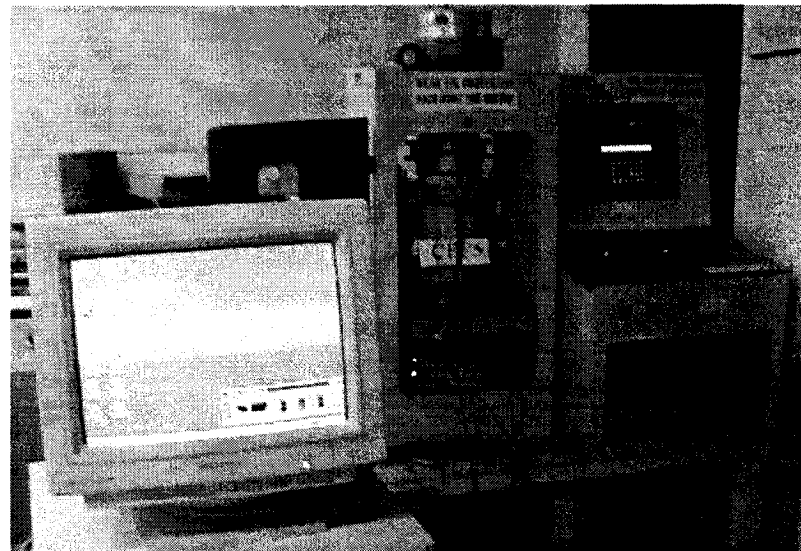


(b) Casting of FRP Reinforced Cylinders

Figure 3.3(Cont.): Preparation and Casting of FRP Reinforced Cylinders



(a) Cylinders Ready for Testing



(b) Cylinder Testing Machine and Data Acquisition

Figure 3.4: Cylinder Tests

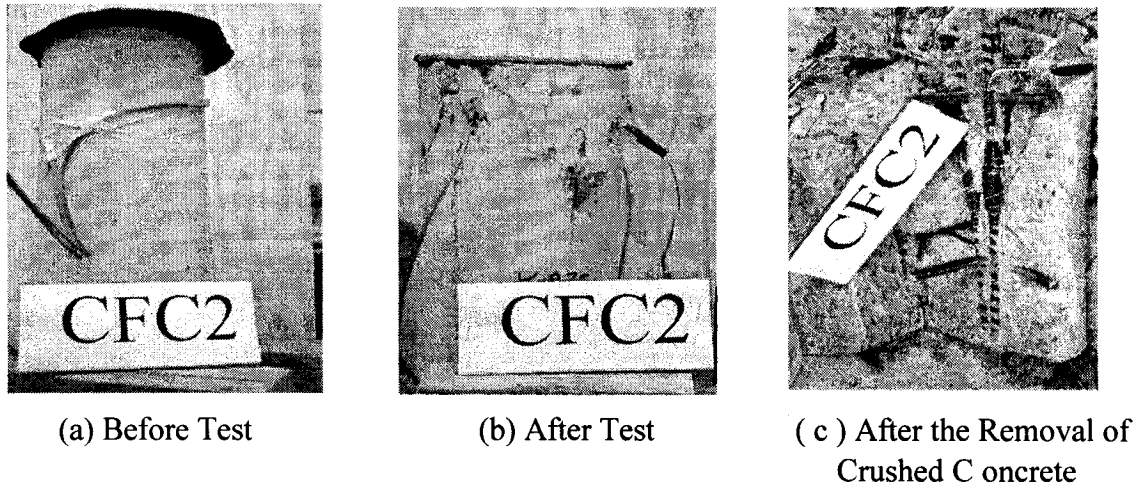


Figure 3.5: Photographs of CFC2 Before and After the Test

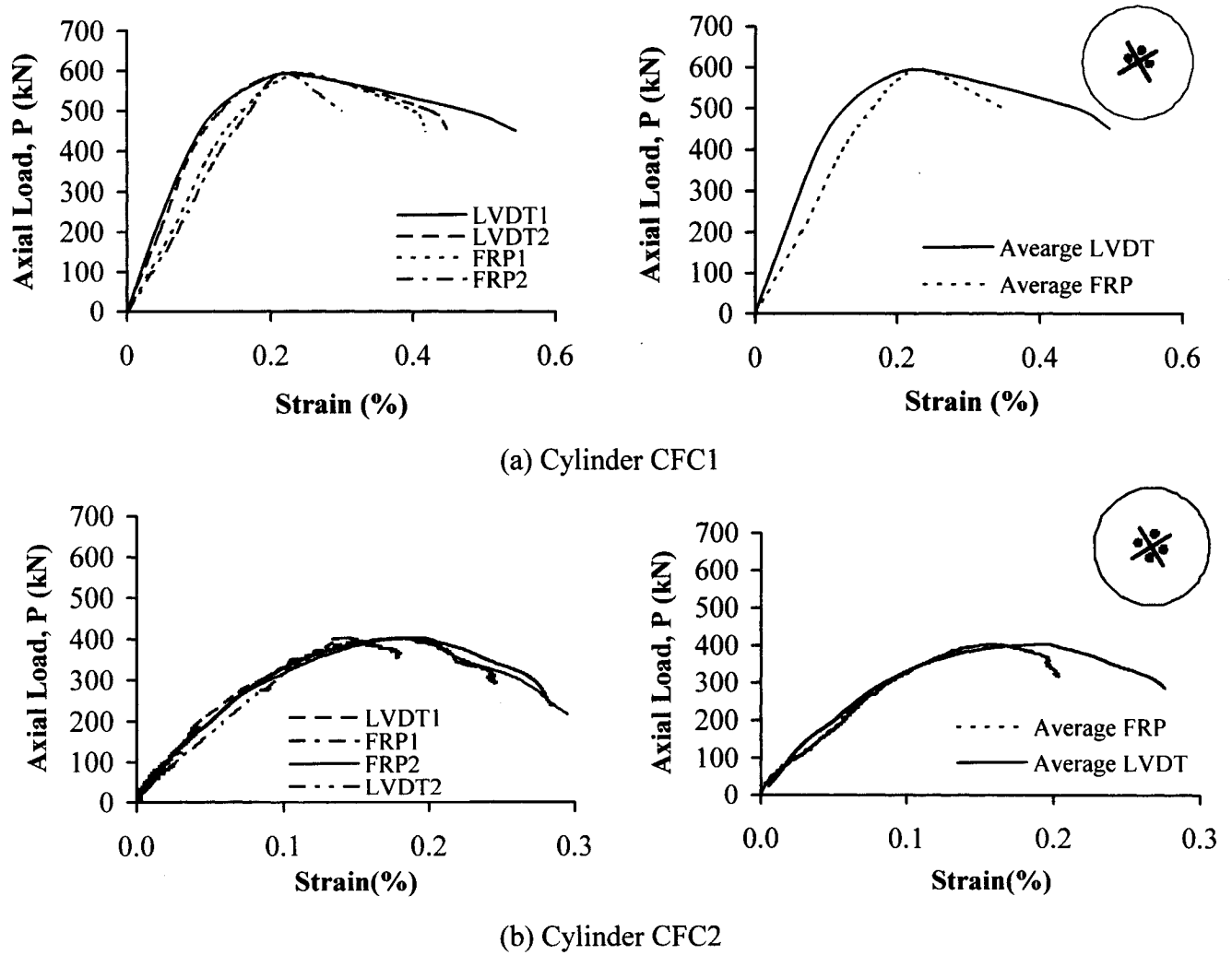


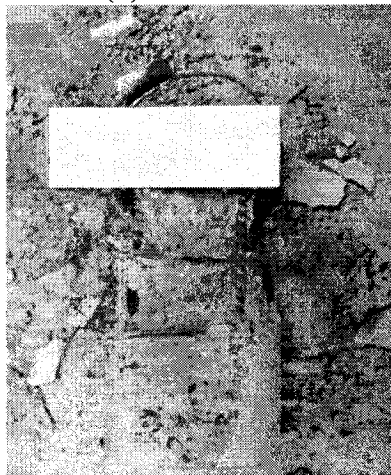
Figure 3.6: Load-Strain Relationships for Reinforced Cylinders, CFC1 , CFC2



(a) Before Test

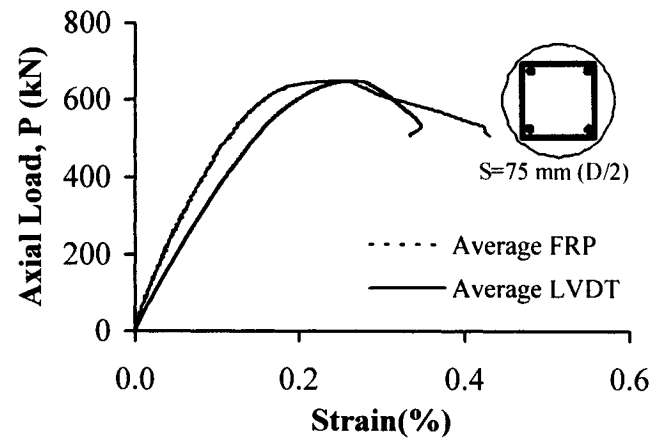
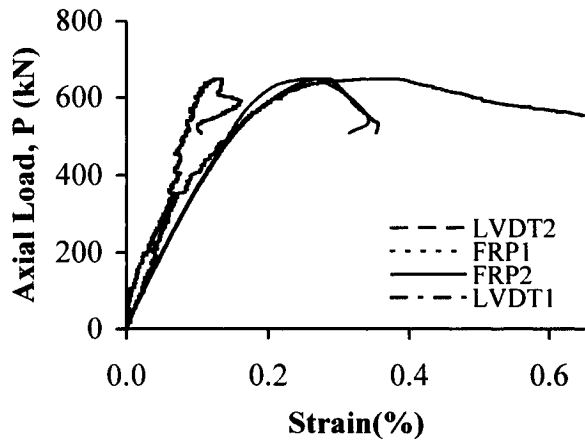


(b) After Test

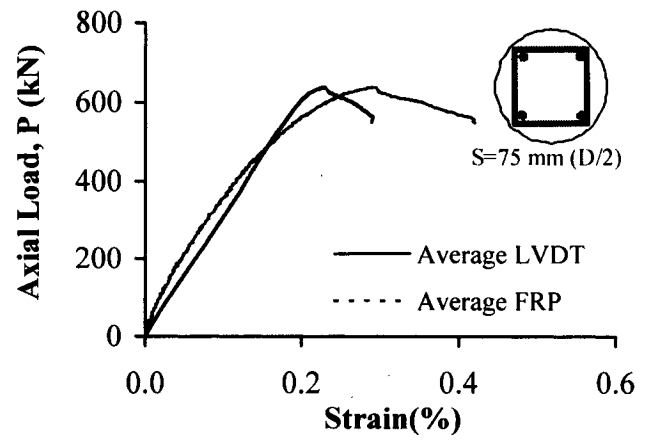
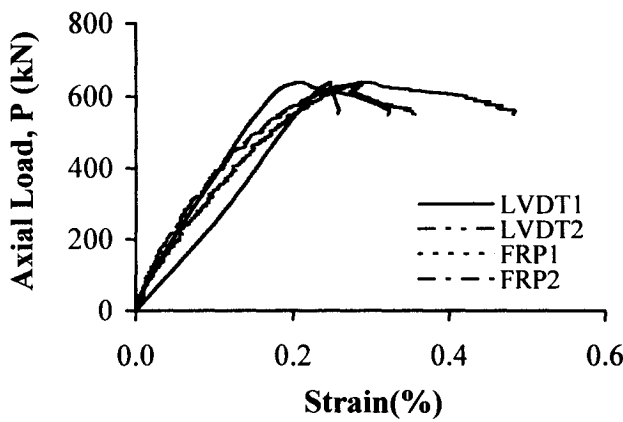


( c ) After the Removal of Crushed Concrete

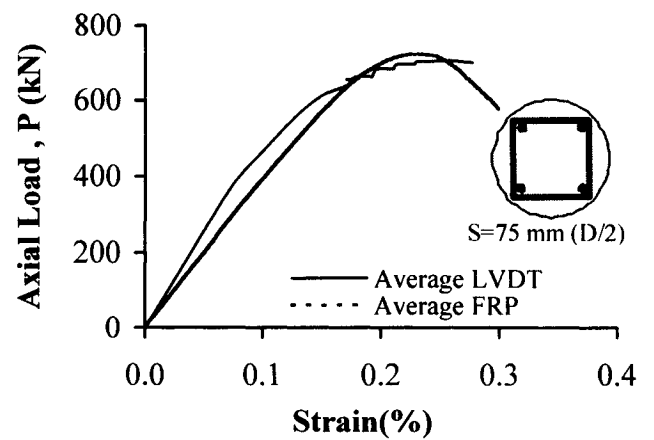
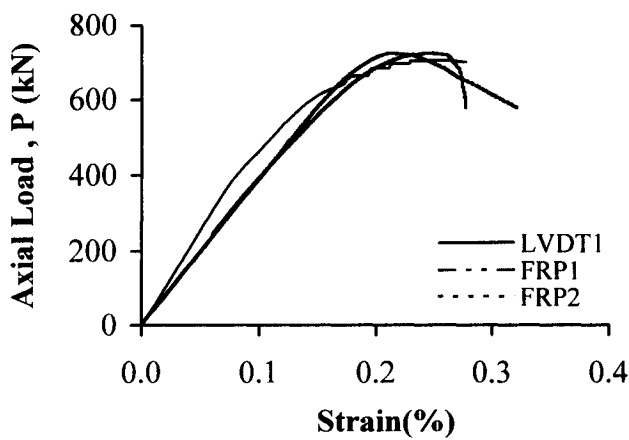
Figure 3.7: Photographs of CFC3 Before and After the Test



(a) Cylinder CFC3

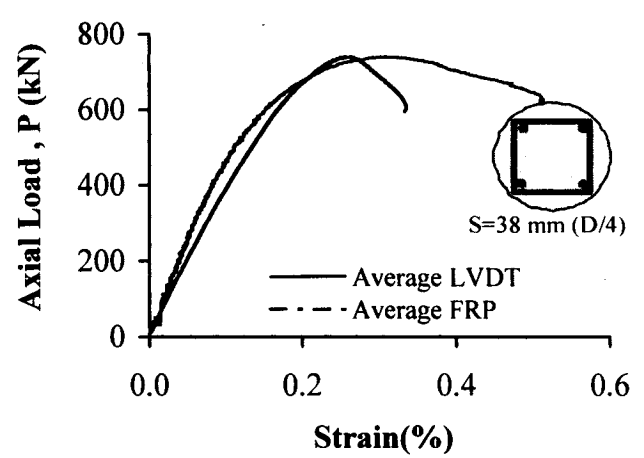
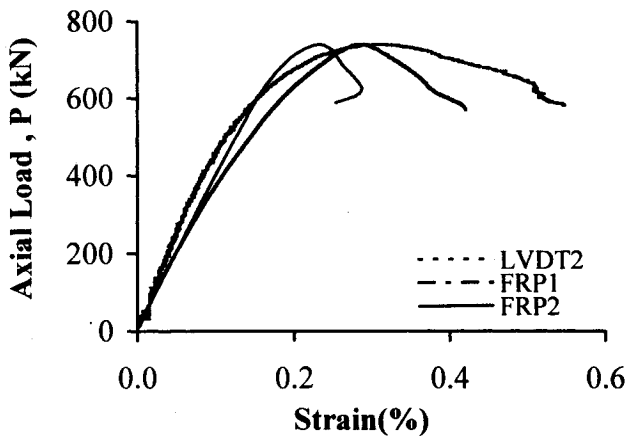


(b) Cylinder CFC4

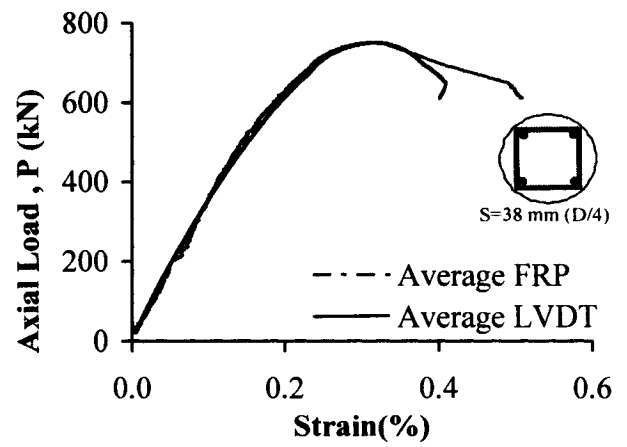
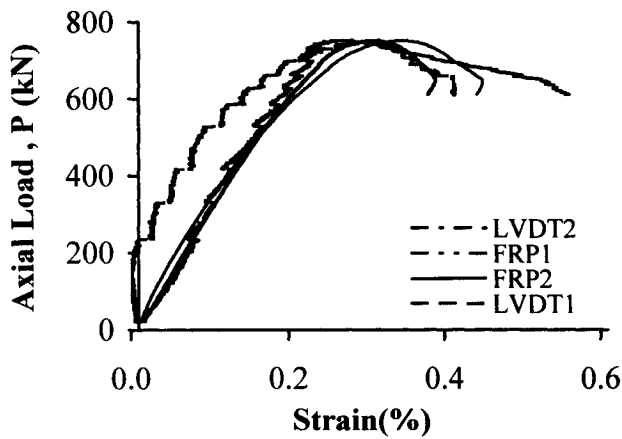


(c) Cylinder CFC5

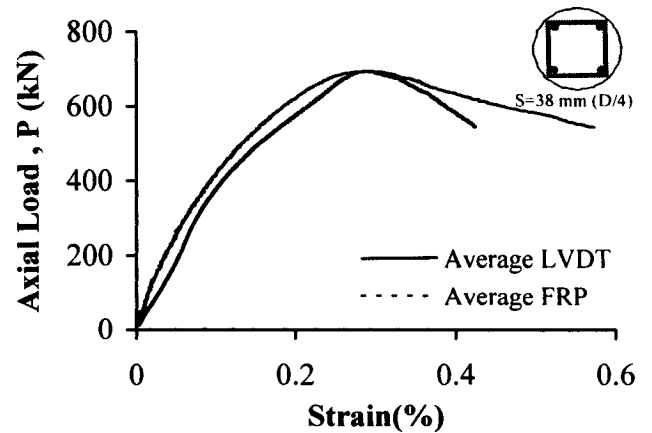
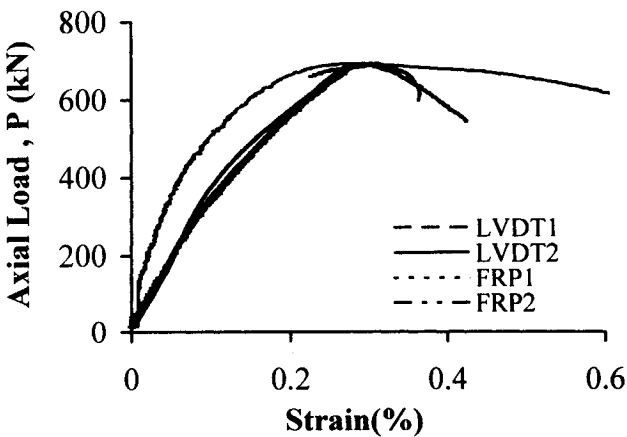
Figure 3.8: Load-Strain Relationships for Reinforced Cylinders (CFC3,CFC4 and CFC5)



(a) Cylinder CFC6

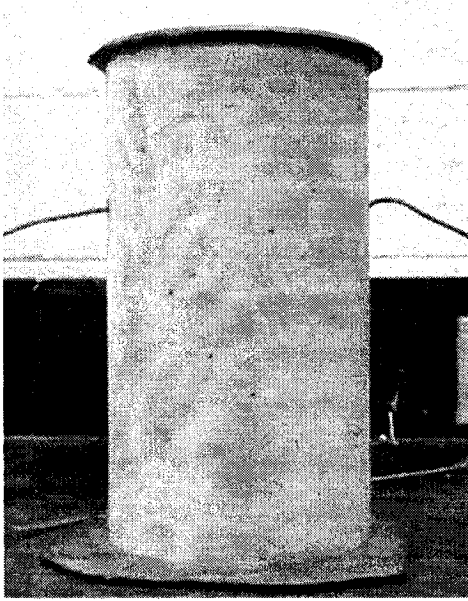


(b) Cylinder CFC7

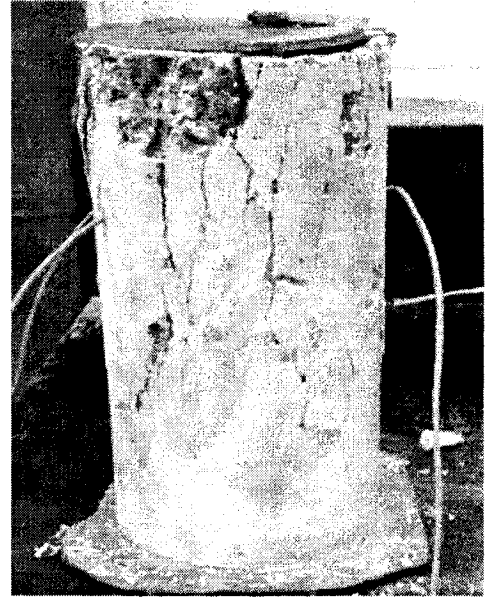


(c) Cylinder CFC8

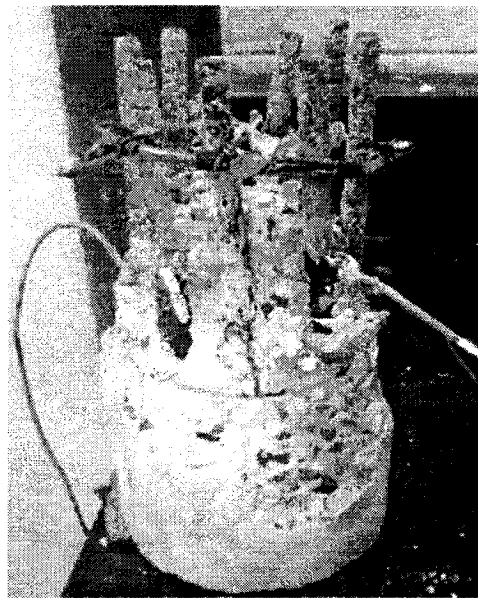
Figure 3.9: Load-Strain Relationships for Reinforced Cylinders (CFC6, CFC7 and CFC8)



(a) Before Test

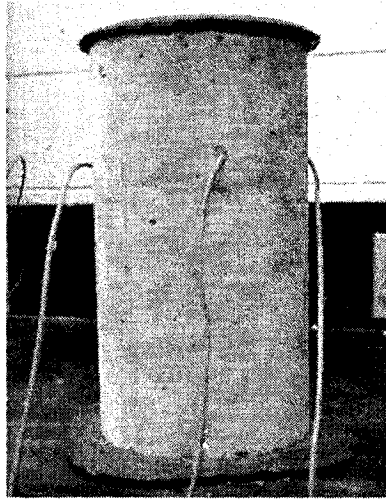


(b) After Test



(c) After Crushing

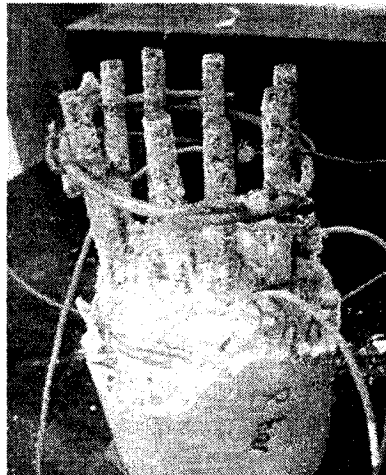
Figure 3.10: 8-Bar Reinforced Cylinder #1, Before and After Test



(a) Before Test

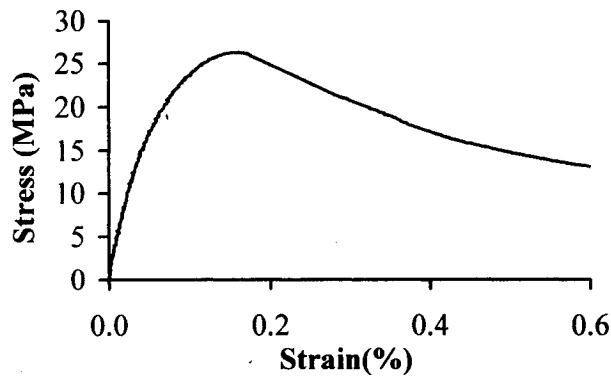


(b) After Test

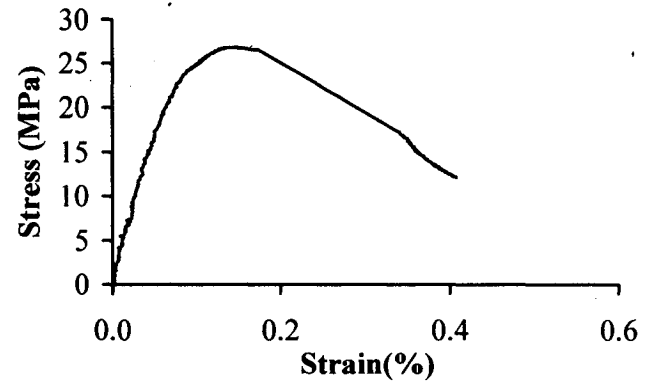


(c) After Crushing

Figure 3.11: 8-Bar Reinforced Cylinder #2, Before and After Test

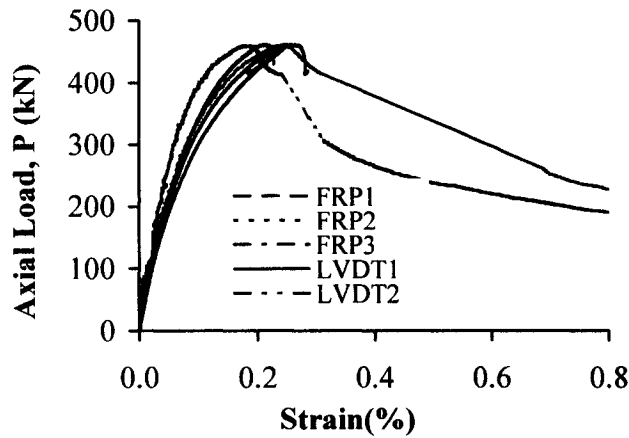


(a) Cylinder CFC9 and CFC10

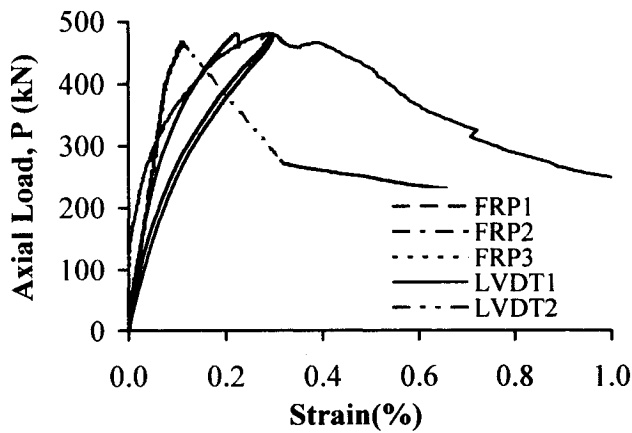
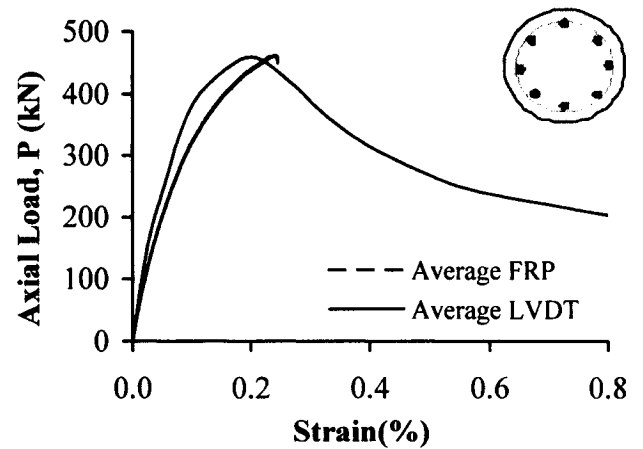


(b) Cylinder CFC11

Figure 3.12: Stress-Strain Relationships for Plain Concrete Series #2 Cylinders



(a) Cylinder CFC9



(b) Cylinder CFC10

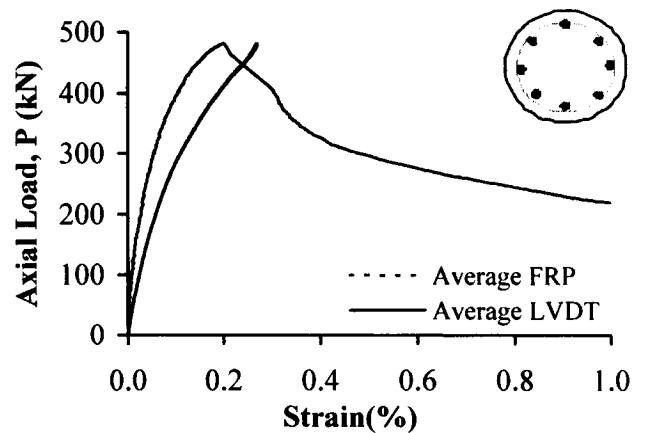


Figure 3.13: Load-Strain Relationships for Reinforced Cylinders, CFC9, CFC10





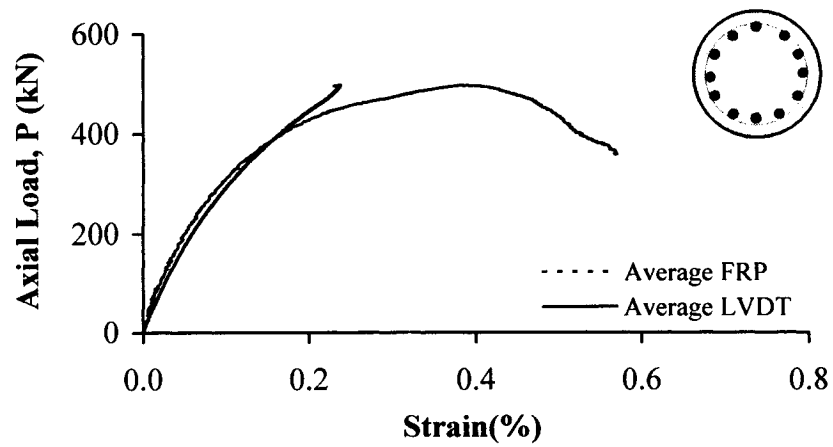
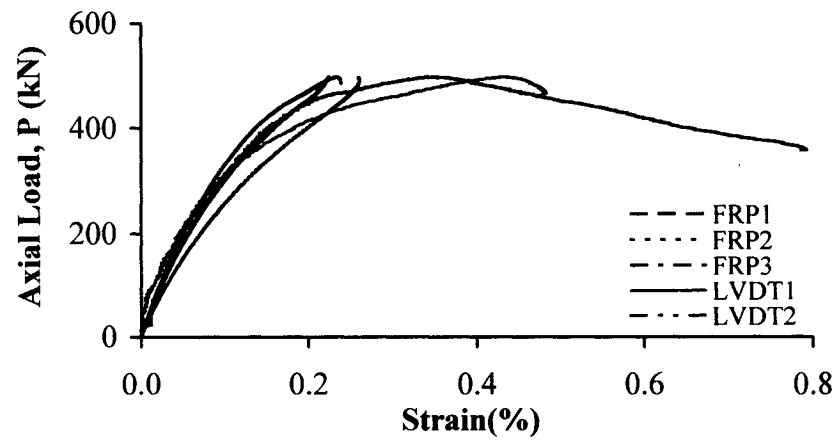
(a) Before Test



(b) After Test



(c) After Crushing



(d) Load-Strain Relationship

Figure 3.14: Photos and Load-Strain Relationships for Reinforced Cylinder CFC11

## ***Chapter 4***

# **CFRP Reinforced Concrete Columns under Reversed Cyclic Loading**

### **4.1 General**

First-storey columns in multi-storey buildings are known to be critical against earthquake loading. Full-size columns, representing part of a first storey building column, between the footing and point of inflection, were designed, built and tested as Phase III of experimental research. The columns were tested under a constant axial load and incrementally increasing lateral deformation reversals, simulating seismic loading. This section describes the details of column specimens, test set-up, instrumentation and procedure, as well as test results.

### **4.2 Test Specimens**

Ten cantilever columns were prepared for testing in Phase III. The columns had a 355 mm square cross-section with either a 1000 mm or a 1900 mm height, which resulted in 1280 mm, or 2180 mm shear span, respectively, when the top-loading beam is considered and the column height was measured to the point of application of the lateral force. The two heights were selected to represent shear dominant and flexure dominant columns. Figure 4.1 illustrates the geometric details of columns.

The longitudinal reinforcement consisted of 9.5 mm diameter Pultrall CFRP bars. Two reinforcement ratios were used; 0.46% for the eight-bar arrangement and 0.69% for the twelve-bar arrangement. The longitudinal bars extended into the footing, which had a depth of 520 mm. The length of FRP bars inside the footing was approximately 470 mm. Clear concrete cover was 20 mm in all cases, measured from the face of the column to the outer surface of NEFMAC grids, which were used as transverse reinforcement. Grid spacing was

either 88 mm or 175 mm. Four-cell grids were used with 8 longitudinal bars, and 9-cell grids were used with 12 bars. The test parameters included the arrangement of longitudinal reinforcement, grid spacing, shear span and the level of axial load. Table 4.1 provides a summary of column properties considered in the current group of specimens, labelled as Phase III columns. Figure 4.2 provides a schematic view of the reinforcement arrangements used in each column.

## **4.3 Material Properties**

### **4.3.1 Concrete**

Normal Portland Cement concrete batches were ordered from a local ready-mix supplier to cast test specimens in the Structures Laboratory of the University of Ottawa. Standard concrete cylinders were cast from each batch to establish the 28-day strength of concrete, as well as the strength during the period testing. In all cases the 28-day strength of concrete was higher than the specified design strength of 30 MPa. The actual concrete strength during the period of column tests was determined by standard cylinder tests to be 38 MPa. Maximum aggregate size used in all batches was 10 mm and the slump of concrete was 100 mm. The mix proportions of a cubic meter of concrete are specified in Table 2.2 and the material characteristics are indicated in Table 2.3, as the same concrete discussed for Phase I specimens was also specified for the columns in the current phase.

Standard molds were used to cast 150 mm by 300 mm cylinders to perform compression tests, as discussed in Section 2.3.1. The results are summarized in Table 4.2 and Figure 4.3.

### **4.3.2 CFRP Bars**

Pultrall CFRP bars with a nominal diameter of 9.5 mm were used as longitudinal reinforcement. Pultrall Inc. in Thetford Mines, Quebec, manufactured the bars, illustrated in Figure 4.4. The manufacturing process involved combining pultrusion and on-line coating. The bars were made from high strength carbon fibers and durable vinyl ester resin. The carbon fibers impart strength to the bar while the vinyl ester resin imparts excellent corrosion resistance in harsh chemical and alkaline environments. Bar surface was sand-coated to

improve bond between the bar and the surrounding concrete. The sand coating resulted in a slight increase in bar diameter, from 9.5 mm to approximately 12.0 mm. However, the latter value has not been used in area and stress calculations, as the sand coating does not contribute to strength.

Tensile properties of Pultrall bars were established through laboratory testing. A number of standard coupon test procedures were developed and specified recently for FRP reinforcement (ACI 440H 2000, CSA 2002 S806-00). Typical coupon test specimens of CFRP Pultrall bars, consistent with the procedure specified in CSA Standard S806 (2000), are shown in Figure 4.5. The coupons were 1200 mm long, as shown in Figure 4.6. The 400 mm segment at each end was placed in a steel tube, to be able to grip on the coupon by the jaws of the machine during tension testing. The tubes had 25.5 mm inner diameter and 3.3 mm thickness. Several photos, illustrating the preparation of test coupons are shown in Figure 4.7. Two PVC centralizers with central holes, having an outer diameter equal to the inner diameter of the steel tube, were used to position the coupon in the centre. This is shown in Figure 4.7(c). These centralizers were secured in the steel tube with silicon. The steel tubes were then filled with specially mixed grout. The grout was manually mixed in the laboratory, consisting of three ingredients; i) epoxy resin, ii) hardener and iii) fine sand. The amount of hardener was 1/5 of the weight of epoxy resin, and the weight of sand was equal to one half the combined weights of resin and hardener. The grout was allowed to harden for at least 3-4 hours. A 270 kN capacity universal testing machine was used for testing, as shown in Figure 4.8. An LVDT with a gauge length of 200 mm was attached to the specimen to measure longitudinal deformations. Precision strain gauges were also placed on the coupons.

Some of the coupon tests were conducted at Sherbrooke University's Laboratory, since Pultrall developed the bars with assistance received from this laboratory. Figure 4.9 (a) depicts tensile stress-strain relationships of Pultrall CFRP bars as established by coupon tests at Sherbrooke University. Figure 4.9 (b) shows the stress-strain relationship obtained by a coupon test at the University of Ottawa. Tensile strengths were calculated using the nominal bar diameter of 9.5 mm. The average tensile strength and modulus of elasticity obtained from three coupons tested at Sherbrooke University were 1420 MPa and 119,000 MPa, respectively, while the values obtained from the University of Ottawa test were 1470 MPa

and 125,000 MPa for tensile strength and elastic modulus, respectively. The Sherbrooke University had reported the mechanical properties of 20 CFRP coupons earlier. Accordingly, the tensile strength was reported to vary between 1480 and 1590 MPa and the modulus of elasticity had a range of 128,000 MPa to 145,000 MPa.

The stress-strain relationship of FRP bars in compression is difficult to establish by tests for the reasons explained in Section 2.3.2 in connection with LeadLine bars. Therefore, samples having lengths equal to 2 to 5 times the bar diameter were tested under direct compression until failure. The stress-strain relationships in compression are plotted in Figure 4.9 (c). The average modulus of elasticity in compression was 23,000 MPa, these values correspond to 20% of elastic modulus in tension. The elastic modulus in compression may be thought to be representative of the modulus of resin alone, ignoring the contribution of fibers to compression. It should be recalled that Pultrall bars had vinyl ester resin with 60% fiber volume fraction. The failure stress in compression varied between 240 MPa and 310 MPa, these correspond to 16% to 21% of tensile strength. The corresponding compressive failure strain varied between 1% and 1.3% for Pultrall bars. The failure in direct compression tests of bars was caused either by delamination of fibers and crushing of resin or by splitting of bars longitudinally. The former type of failure was observed when the compression bar sample was 3 to 5 times the bar diameter and the latter failure type was observed when the sample length was twice the bar diameter. However, the failure type did not appear to affect compressive strength and failure strain.

#### **4.3.3 CFRP Grids**

CFRP transverse reinforcement used in the current investigation consisted of NEFMAC<sup>TM</sup> grids. NEFMAC<sup>TM</sup> is manufactured by forming flat or curved grids through a pin-winding process, similar to filament winding. Grids are manufactured either from glass or carbon fibres, impregnated with vinyl ester resin. The product used in the current phase of experimental research was reported to have a specific gravity of 1.4 and a modulus of elasticity of 100,000 MPa, by the manufacturer. The grids had a square configuration with 300 mm out-to-out dimension. Two types of grids were employed as transverse tie

reinforcement; i) 6 x 8 mm rectangular cross-section forming four equal-size square openings, and ii) 6 x 8 mm rectangular cross-section forming nine equal-size square openings. Figure 4.10 illustrates the schematic view and test photographs of CFRP grids. The stress-strain relationships obtained from coupon tests in Phase I, illustrated in Figure 2.8, can be used in the current phase of the experimental program, since the grids were of the same type and batch.

#### **4.4 Preparation of Specimens and Instrumentation**

Reinforcement cages were assembled first by tying Pultrall bars and NEFMAC grids together. The footing for each specimen was heavily reinforced with steel reinforcement, as shown in Figure 4.11 to prevent premature failure in the footing. Plywood formwork was prepared separately for casting the columns and footings in two stages. Figure 4.12 illustrates the formwork for footings. Ready-mix concrete was used to cast the footings first. The footing-column interface (construction joint) was intentionally left rough. The columns were cast vertically to simulate the actual construction practice a few weeks after the casting of footings. The column cages and stages of casting are shown in Figures 4.13 and 4.14. Wet burlap and plastic sheets cured the columns for at least two weeks.

The specimens were mounted on a heavily reinforced T-shaped foundation, which was secured on the laboratory strong floor and allowed for the assembly of vertical actuators on either side. The foundation had four holes that accommodated 63 mm diameter bolts to secure column specimens. Figure 4.15 illustrates the foundation. They had four holes, two on each side of the footing, to allow full fixity of columns to the foundation by means of 800 MPa, 63 mm steel threaded bolts.

A steel loading beam assembly was secured on the columns before they were connected to actuators. Four or eight Grade 800 MPa, 20 mm bolts were used for this purpose, depending if the column was a short or a long column, since shear was critical in short column bolts. The bolts had been cast in concrete at column top, vertically protruding from the column.

The specimens were instrumented with Linear Variable Differential Transducers (LVDTs), for displacement and rotation measurements. A Temposonic LVDT was placed horizontally

at the point of application of horizontal load to measure the column top displacement. A light aluminum frame was built around the column to attach the Tempononic LVDT so that the measured displacements would be relative to the column footing. Four additional LVDTs were placed vertically near the critical section of column to measure the rotations of hinging region, as well as those caused by anchorage slip (extension of longitudinal reinforcement within the footing). These LVDTs were placed in pairs, one on each side, perpendicular to the direction of loading. One pair was placed at 40 mm and the other at 355 mm from the footing. While the difference between the readings of the two latter LVDTs on either side divided by their horizontal distance would give the total rotation of hinging region, the readings of the former two LVDTs at the bottom of the column would give the rotation due to anchorage slip. The difference between the total hinge rotation and the anchorage slip rotation would give the flexural rotation of the hinging region. Figure 4.16 illustrates the locations of LVDTs.

Electric resistance strain gauges were placed on FRP bars and grids to measure strains in longitudinal and transverse reinforcement. Four longitudinal bars were instrumented with strain gauges at the extreme compression and tension fibers of column footing interface. One of the bars also had strain gauges 160 mm above and 160 mm below the column-footing interface to measure the variation of strains along the length of the bar within the critical region for flexure. The first two or three grids in each column, immediately above the footing, were also instrumented with strain gauges. Two grids were instrumented when the grid spacing was equal to half the column sectional dimension ( $h/2$ ) and three were instrumented when the grid spacing was reduced to a quarter of column dimension ( $h/4$ ). These grids were labeled as the first, second and third, the first grid being the closest to the footing. Figures 4.17 and 4.18 illustrate the locations of strain gauges. All the instrumentation, including the load cells and the LVDTs of MTS actuators were connected to data acquisition systems and MTS controller for data collection. Two microcomputers were used to control the data acquisition systems and the MTS controller. The Load Control and Data Acquisition Systems are shown in Figure 4.19.

## **4.5 Test Setup and Loading Program**

Three 1000 kN capacity servo controlled MTS actuators were used to apply the loads. Two of the actuators were positioned vertically to apply constant axial compression during testing. They were connected to a rigid base that had been fixed on the laboratory strong floor at one end, and to a steel-loading beam at the other end. The third actuator was positioned horizontally between the steel loading beam and the lateral support system. The maximum stroke of the horizontal actuator was 500 mm, which allowed horizontal displacement of up to  $\pm 250$  mm relative to the neutral position. The actual stroke and load during testing were monitored and recorded by two independent data acquisition systems. Figure 4.20 illustrates the details of the test set-up. Figure 4.21 shows the sequence of a typical column installation and test preparation in the laboratory. The general view of test set-up is depicted in Figure 4.22. The hydraulic pressure was supplied by a 33 GPM pump. The same pump supplied oil to the vertical actuators. The pressure was controlled through a MTS servo-valve for early actuator. The detail of the hydraulic power system is illustrated in Figure 4.23. A schematic view of the Network Diagram for the test system is shown in Figure 4.24.

The axial load was applied first and was maintained at a constant level through the test. The horizontal load was applied in the deformation control mode. Lateral deformation reversals were applied\ starting with three elastic cycles at 0.5% lateral drift, which approximately corresponded to the displacement at first flexural cracking, followed by three cycles at 1% drift. The subsequent stages of loading included three cycles at each of the incrementally increasing drift level. The loading program followed is illustrated in Figure 4.25.

## **4.6 Test Results**

Observed behaviour of columns during column tests is presented and discussed in this section. Photographs taken at selected stages of loading illustrate the performance of each column. The hysteretic behaviour is presented both in terms of force-displacement and moment-drift relationships. The moments plotted in these relationships are computed from recorded test data as the net lateral force times the shear span plus the axial force times the horizontal displacement (P-Delta moment).



Hysteretic moment-total rotation, moment-slip rotation, and moment-flexural rotation relationships are also presented and discussed in this chapter. These readings were taken within the potential hinging region, as explained in Section 4.5. The hinging region was defined as the column segment between the column-footing interface and the section 355 mm above the footing (equal to the column cross-sectional dimension). The first set of readings gave the total rotation of assumed hinging region relative to the footing. This set of readings consisted of rotations mostly due to flexure and also due to anchorage slip. The rotations due to anchorage slip were measured and formed the second set of readings. These readings were taken as the rotation of a column section near the column-footing interface relative to the footing. Ideally, these readings should have reflected the rotation of the critical column section at the interface. However, the LVDTs used required some gauge length to be positioned on the column. Hence, they were mounted on a section approximately 40 mm above the interface. Therefore, these readings also included flexural rotations of the lowest 40 mm segment of the column and hence are not accurate representation of anchorage slip. Furthermore, these readings were taken within the most disturbed region of columns because of concrete crushing. Hence, additional errors may have occurred especially near the end of testing when significant damage was observed in the column-hinging region. The relatively small magnitude of anchorage slip readings provides a good indication of the significance of this deformation component in columns. The difference between the total and the anchorage slip rotations gives flexural rotations.

Even though the anchorage slip caused in steel reinforced columns is closely related to the level of yield penetration that occurs into the footing, the slip rotation may also be important in FRP reinforced columns due to the elastic extension of lower modulus FRP bars and potential bar slippage in footings. The slippage of bars becomes noticeable only if the embedment length is not sufficient to develop the required stress in reinforcement. It may be less significant in columns subjected to high axial compression. All of the columns except for two short columns were subjected to high axial compression. Therefore, the anchorage slip component reported later in the Chapter formed only a small portion of the total rotation.

The variation of reinforcement strains in longitudinal and transverse reinforcement is presented in the form of hysteretic force-strain relationships. Each column had four longitudinal bars instrumented with a total of 6 strain gauges to measure the variation of strain in CFRP bars. The strain gauges ceased to function at different stages of loading either due to excessive tension or compression. Hence, the data presented does not represent the behaviour of FRP bars until the end of a test.

An important design parameter used in explaining column deformability is the degree of confinement in concrete. This quantity becomes especially important in dealing with FRP reinforced concrete elements since deformability in such members can only be provided by the confinement of compression concrete rather than the ductility of reinforcement. A reference quantity for concrete confinement adopted in this Chapter is that recommended by the recently published CSA Standard S806-02, entitled "Design and Construction of Building Components with Fibre-Reinforced Polymers". Accordingly, the required area  $A_{FH}$  of transverse CFRP grid reinforcement shall be calculated as specified below unless a larger amount is required for shear:

$$A_{FH} = 14 s h_c \frac{f'_c}{f_{FH}} \left( \frac{A_g}{A_c} - 1 \right) \frac{\delta}{\sqrt{K_c}} \frac{P_f}{P_{ro}} \quad (4.1)$$

Where;  $\left( \frac{A_g}{A_c} - 1 \right) \geq 0.3$

$P_f$  = Factored applied axial load with zero end eccentricity (applied concentric load).

$P_{ro}$  = Factored axial resistance under zero end eccentricity (concentric load resistance).

$A_g$  = Gross cross-sectional area of column.

$A_c$  = Core area of column core measured to the centerline of the perimeter grid.

$\delta$  = Design lateral drift ratio, shall not be less than 3%.

$f'_c$  = Specified compressive strength of concrete

$f_{FH} = 0.004 E_f$

$E_f$  = Elastic modulus of CFRP grids in tension.

$h_c$  = Cross-sectional dimension of column core in which direction  $A_{FH}$  is calculated.

$K_c = 0.15 \frac{h_c}{s}$  for rectangular grids.

$s$  = Spacing of transverse grids.

The maximum spacing of FRP grids, permitted by CSA S806-02 is the lesser of one-quarter of member dimension or 150 mm. In addition, a restriction on spacing is placed as 6 times the diameter of longitudinal bars. However, the latter requirement is intended for the stability of steel reinforcement, since the current CSA S806-02 does not permit the use of FRP bars in columns due to lack of research in the area. The FRP bars typically have smaller cross-sections because of their higher strength. The applicability of the latter restriction to FRP bars requires further research. The former spacing requirements result in a maximum grid spacing of 88 mm for the columns tested in the current phase of research.

The columns tested in Phase III were subjected to a constant axial load, which was established as a percentage of column concentric compressive capacity,  $P_o$ . This quantity may be computed as shown below, by superimposing the contributions of concrete and FRP reinforcement and assuming the failure under concentric compression occurs at 0.002 strain.

$$P_o = 0.85 f'_c (A_g - A'_{FRP}) + 0.002 E_{fc} A_{FRP} \quad (4.2)$$

$A_{FRP}$  and  $E_{fc}$  are the total core area and compressive elastic modulus of longitudinal FRP bars.  $A'_{FRP}$  is the total area of FRP reinforcement including the area of sand coating.

It should be noted that the Pultrall bars had a layer of sand coating around the bars. The thickness of this coating was measured to be approximately 1.0 mm. Any contribution to column strength by the sand coating has been neglected in computing the column concentric capacity based on Equation (4.2). Therefore, the area of FRP bar was computed based on 11.5 mm diameter for the purpose of computing net cross-sectional area of concrete, and 9.5 mm diameter in computing the contribution of FRP reinforcement to column capacity.

Average values of elastic modulus for Pultrall bars and NEFMAC grids were established to be 115000 MPa and 77000 MPa, respectively.

#### **4.6.1 Results and Observations**

##### **4.6.1.1 Column CFCL1**

Column CFCL1 was reinforced with 8 - 9.5 mm diameter Pultrall CFRP longitudinal bars that resulted in 0.46% reinforcement ratio. The bars were tied with 4-cell CFRP grids. The amount of transverse reinforcement ( $144 \text{ mm}^2$ , in each cross-sectional direction) was equal to 33% of that required by CSA A806-02 for concrete confinement, based on the FRP strength at 0.004 strain. The spacing of grids was 175 mm, which was approximately equal to twice the maximum spacing required by CSA S806-02. The column was tested under a constant compressive force of 30% of its concentric capacity. The hysteretic relationships recorded during the test, and shown in Figure 4.26, indicate that the column showed stable hysteresis loops up to 1.5% drift ratio, followed by significant and sudden strength decay due to the crushing of concrete and the stability failure of compression bars. This was expected because of the wider spacing of grids used, which resulted in poor concrete confinement and inadequate support to compression reinforcement.

Observations during testing indicated that initial flexural cracking started at 0.5 % and 1% drift cycles within the lower 425 mm portion of column at the second and third grid locations, on both sides of column. Some hairline diagonal tension cracks were also observed on side faces. Significant concrete crushing was observed near the base at about 1.5% drift ratio as the flexural cracks propagated toward the side faces. The crushing of concrete was more pronounced in one of the corners, as illustrated in Figure 4.27. This column was the first to test and was not supported in the direction perpendicular to the horizontal load. At this late stage of loading, the column was observed to experience out of plane deformations, resulting in increased unintended biaxial bending. This was accompanied by sudden strength degradation. The reinforcement was exposed in the critical region and the compression bars started to fail due to buckling. The appearance of bar failure was more like a bar broken into two pieces by bending or a bar cut by a cutter, rather than material failure in compression or

tension which would show either delamination or rupturing of individual fibers. Figure 4.27 illustrates the performance of column CFCL1 at selected stages of loading.

Strain gauge readings indicated that FRP bars experienced a maximum of 0.65% tensile strain and 0.50% compressive strain at the end of test, whereas the FRP grids experienced a maximum of 0.3% tensile strain. Figures 4.28 and 4.29 show the hysteretic behaviour of recorded strains in FRP bars and grids, respectively.

Moment-rotation hysteretic relationships, recorded during the test, are shown in Figure 4.30. The results indicate that about 40% of total hinging region rotation could be attributed to anchorage slip, and the remaining 60% to flexure. The results indicate higher anchorage slip rotations than those usually observed in steel-reinforced concrete columns, even though the FRP bars do not yield. This can be explained by the low elastic modulus of FRP, which produces high elastic elongations of the bars outside the column, within the embedment length in the footing. Further discussion of deformation components is given in Chapter 6, where the comparison of experimental and analytical deformations is made.

#### **4.6.1.2 Column CFCL2**

Column CFCL2 was companion to CFCL1, reinforced with 8 - 9.5 mm CFRP longitudinal bars that resulted in 0.46% reinforcement ratio. The column was confined with 4-cell CFRP grids, having reduced spacing of 87 mm, relative to CFCL1. This level of spacing is equal to the maximum spacing allowed by CSA S806-02. The area of transverse reinforcement in each cross-sectional direction was  $144 \text{ mm}^2$ , which corresponded to almost the area required by CSA S806-02 for concrete confinement (0.9 times that required by the CSA Standard). The axial load applied was equal to 30% of column concentric capacity.

The column showed better behaviour than the companion column CFCL1. The reduction in grid spacing increased the effectiveness of concrete confinement and the stability of compression reinforcement, enhancing inelastic deformability. The hysteretic relationships shown in Figure 4.31 indicate that the column showed stable hysteresis loops up to 4% drift ratio with little strength decay. When the column was forced to develop 5% drift ratio it

failed at about 4.5% drift level, initiated by the instability of compression FRP bars, followed by gradual crushing of concrete. An interesting observation that can be made on the hysteretic behaviour of FRP reinforced concrete, relative to steel reinforced concrete, is the stiffness during loading, unloading and reloading. Due to the relatively low elastic modulus of FRP bars, the member developed reduced stiffness during loading. However, because there was no yielding in reinforcement, the member did not experience permanent plastic deformations in reinforcement, unlike steel reinforcement. Hence, the unloading curves aimed approximately to the origin of force-deformation relationships, depicting reduced stiffness, rather than almost parallel unloading slope to the elastic slope often observed in flexure dominant steel reinforced concrete elements. This implies reduced area under hysteresis loops and equally reduced energy dissipation capacity of FRP reinforced members. However, inelastic drift level was able to reach as high as 3% drift ratio without a significant decay in strength, which is a level well above the expected inelastic design drift ratios used in practice.

Observations during testing indicated that the initial flexural cracking started during 1% drift cycles at about the third grid location, 250 mm above the footing. During 2% drift cycles, additional cracks formed at the fifth, fourth, and first grid locations, during the first, second, and third cycles, respectively. No shear cracks were observed. Some vertical cracks formed in one of the corners of column concrete during 2% drift cycles, signifying the beginning of spalling. The column was cycled until 3% drift ratio, with increasing cracking, but without significant strength decay. Displacements continued to increase with gradual drop in load resistance as the compression concrete crushed. The compression bars buckled at approximately 4.5% drift level, resulting in wide flexural cracks upon load reversal. Figure 4.32 illustrates the performance of Column CFCL2 at selected stages of loading.

Strain gauge readings indicated that the FRP bars experienced a maximum of 1.2% tensile strain, approaching approximately their tension capacity observed in coupon tests, while developing up to 0.55% compressive strains at the end of test. The FRP grids experienced a maximum of 0.55% tensile strain. Hysteretic behaviours of recorded strains are presented in Figures 4.33 and 4.34 for longitudinal and transverse reinforcement, respectively.

Moment-rotation hysteretic relationships recorded during testing are shown in Figure 4.35. There were locations at which the LVDT readings showed jumping after the column reached its maximum capacity. The recorded data after this point may not be reliable. The curves are plotted up to the maximum load resistance. The results indicate that about 35% of total hinging region rotation could be attributed to anchorage slip, and the remaining 65% to flexure.

#### **4.6.1.3 Column CFCL3**

Column CFCL3 was reinforced with 12 - 9.5 mm diameter CFRP longitudinal bars that resulted in 0.7% reinforcement ratio. It was confined with 9-cell CFRP grids. The grid spacing was 175 mm, which was approximately twice the spacing allowed by CSA S806-02. The amount of transverse FRP reinforcement in each cross-sectional direction was 192 mm<sup>2</sup>. This was 47% of that required by CSA S806-02 for concrete confinement. The column was subjected to a constant compressive force of 27% of its concentric capacity. The hysteretic relationships shown in Figure 4.36 indicate that the column showed stable hysteresis loops up to approximately 2% drift ratio, but developed significant strength decay during the cycles at 3% drift ratio, caused by the crushing of concrete. The reinforcement was intact at this stage of loading. The column failed when it was forced to develop 4.5% drift ratio due to the stability failure of bars in compression and increased crushing of concrete.

Observations during testing indicated that flexural cracking started at 0.5% and 1% drift cycles within the lower 800 mm portion of column. During 3% drift cycles, some of the flexural cracks propagated into the side faces with inclination, forming flexure-shear cracks. Cover concrete within the lower 200 mm portion of column started to crack and expand laterally, separating from the core. Cover concrete spalled off during 3.5% and 4% drift cycles. Stability failure of all compression bars was observed when the column was forced to develop 4.5% drift ratio, as the compression concrete continued to crush. Figure 4.37 illustrates the performance of Column CFCL3 at selected stages of loading.

Strain gauge readings indicated that FRP bars experienced a maximum of 0.76% and 0.5% strain in tension and compression at the end of test, respectively. The FRP grids experienced

a maximum of 0.31% tensile strain. Hysteretic behaviour of strains recorded during testing are presented in Figures 4.38 and 4.39.

Moment-rotation hysteretic relationships, recorded within the potential plastic hinge region, are shown in Figure 4.40. The results indicate that about 21% of total rotation was caused by anchorage slip and the remaining 79% by flexure. The anchorage slip was attributed to the elastic extension of FRP reinforcement inside the footing.

#### **4.6.1.4 Column CFCL4**

Column CFCL4 was companion to CFRP3, reinforced with 12 - 9.5 CFRP longitudinal bars that resulted in 0.7% reinforcement ratio. The column was confined with 9-cell CFRP grids, providing 192 mm<sup>2</sup> area of transverse reinforcement in each cross-sectional direction. This corresponded to approximately 1/3 more reinforcement than that required by CSA S806-02 for concrete confinement. The grid spacing was 87 mm, which was equal to the maximum spacing allowed by CSA S806-02. The column was subjected to a constant compressive force, equal to 27% of its concentric capacity. The hysteretic relationships shown in Figure 4.41 indicate that the column showed stable hysteresis loops up to 3% drift ratio, but developed significant strength decay during 4% drift cycles. It failed when forced to resist the first cycle at 5% drift ratio due to the crushing of concrete, followed by the buckling of all compression bars. The observation made earlier on the nature of hysteretic relationships for FRP reinforced columns can also be repeated for this column, especially because it showed high inelastic deformability, i.e., reduced unloading slopes of loops aiming towards the origin. This is a significant difference that differentiates hysteretic relationships of steel reinforced concrete elements with steel developing permanent plastic deformations during inelastic response, from those of FRP reinforced members which do not have the ability to experience plastic deformations.

Observations during testing indicated that flexural cracking started during 1 % drift cycles at second third and fourth grid locations from the base. Additional flexural cracks formed during 2% drift cycles within the bottom 800 mm segment. At 3% drift, crack widths became wider while vertical cracks started to appear within the cover concrete, signifying imminent



spalling. Cover concrete started to spall off lower than 3% drift cycles and continued spalling up to 4% drift ratio, at which time the bottom portion of cover had spalled off completely, exposing reinforcement. Subsequently, the compression bars buckled, leading to the failure of column at 5% drift ratio. Figure 4.42 illustrates the performance of Column CFCL4 at selected stages of loading.

Strain gauge readings indicated that FRP bars experienced a maximum of 0.93% and 0.55% strains in tension and compression at the end of test, respectively. The FRP grids experienced a maximum of 0.5% tensile strain. Hysteretic behaviour of recorded strains in bars and grids are presented in Figures 4.43 and 4.44, respectively.

Moment-rotation hysteretic relationships, recorded during testing, are shown in Figure 4.45. The results indicate that approximately 30% of total rotation within the potential plastic hinge region was caused by anchorage slip, while the remaining 70% was caused by flexure. Further discussion of these deformation components is made in Chapter 6 where analytical computations are presented.

#### **4.6.1.5 Column CFCL5**

Column CFCL5 was identical to CFCL1 except for its reduced length of 1000 mm, resulting in a shear span of 1280 mm, increasing the shear to moment ratio. It was reinforced with 8 - 9.5 CFRP longitudinal bars that resulted in 0.46% reinforcement ratio. The column was confined with 4-cell CFRP grids. The area of transverse reinforcement provided was 144 mm<sup>2</sup> in each cross-sectional direction. This amount of reinforcement was approximately equal to 29% of the amount required by CSA S806-02 for concrete confinement. The grid spacing was 175 mm, which was approximately equal to twice the maximum spacing allowed by CSA S806-02. The column was tested under a constant compressive force of 33% of its concentric capacity. The hysteretic relationships shown in Figure 4.46 indicate that the column showed stable hysteresis loops up to 2% drift ratio, but significant strength decay occurred at 3% drift ratio. This was expected because of the larger vertical spacing of grids that resulted in a low volumetric ratio of transverse reinforcement and insufficient concrete confinement and inability to stabilize the compression bars.

Observations during testing indicated that flexural cracking started at 1% drift during the first cycle, forming at the second and third grid locations from the base. A diagonal crack was observed during the second cycle of 1% drift ratio. The cracked region extended up to 650 mm from the base during the cycles corresponding to 2% drift ratio. In addition, new diagonal cracks formed on the side faces. The spalling of concrete cover was observed near the footing during 3% drift cycles. The buckling sound of FRP bars in compression was heard as the column was deformed to 3% drift ratio for the first time. Upon reversal of deformations, the bars on the other side failed in compression. Figure 4.47 illustrates the performance of column CFCL5 at selected stages of loading.

Strain gauge readings indicated that FRP bars experienced a maximum of 0.61% and 0.57% strain in tension and compression at the end of test, respectively. The FRP grids experienced a maximum of 0.61% tensile strain. This is an increase of a factor of 2 when compared with the recorded transverse strain of 0.3% in Column CFCL1, the companion column with about twice the shear span. This increase is attributed to the increased shear force applied on Column CFCL5. Figures 4.48 and 4.49 show the hysteretic behaviour of recorded strains in bars and grids, respectively.

Moment-rotation hysteretic relationships, recorded during testing, are shown in Figure 4.50. The results indicate that, for the range the rotations were recorded, approximately 10% of total was caused by anchorage slip and the remaining 90% by flexure.

#### **4.6.1.6 Column CFCL6**

Column CFCL6 was companion to CFRP5 with reduced grid spacing. The longitudinal reinforcement consisted of 8 - 9.5 CFRP bars, which resulted in 0.46% reinforcement ratio. The column was confined with 4-cell CFRP grids, providing 144 mm<sup>2</sup> of transverse reinforcement in each cross-sectional direction. This amount was 7% higher than that required by CSA S806-02 for concrete confinement. The grid spacing was 87 mm, which was equal to the maximum spacing allowed by CSA S806-02. The column was subjected to a constant compressive force of 33% of its concentric capacity. The reduction of grid spacing

in this column was expected to increase the effectiveness of confinement. The hysteretic relationships shown in Figure 4.51 indicate that the column showed stable hysteresis loops up to 2% drift ratio, but developed gradual strength decay thereafter, with little decay at 3% drift ratio. The strength decay was substantial during the 4% drift cycles. The column failed during first loading towards 4% drift, at about 3.5% drift ratio due to the buckling of compression bars and crushing of concrete. The column had reduced deformability when compared with the companion column with a longer shear span (CFCL2), because of increased contribution of shear and the resulting deterioration of compression concrete within the inelastic range of deformations. However, the improvement in behaviour relative to Column CFCL5, which had the same shear span but wider grid spacing, was limited.

Observations during testing indicated that flexural cracking started during 1% drift cycles. The cracks first formed within the lower 340 mm segment above the footing. The cracks increased during 2% drift cycles, covering approximately the lower half of column. Some diagonal cracks were observed on side faces. Vertical cracks also started to form in the lower region of cover concrete, indicating the initiation of cover spalling. Some parts of concrete spalled off near the end of 2% drift cycles. Significant spalling was observed during 3 % drift cycles on one side of the column, followed by the other side. The column failed at about 3.5% drift due to the compressive failure of bars on one side, followed by the bars on the other side upon load reversal. Figure 4.52 illustrates the performance of column CFCL6 at selected stages of loading.

Strain gauge readings indicated that FRP bars experienced a maximum of 0.77% and 0.62% strains in tension and compression at the end of test, respectively. The FRP grids experienced a maximum of 0.53% tensile strain. This was somewhat smaller as compared to Column CFCL5 which has wider grid spacing. The hysteretic behaviour of recorded strains for bars and grids are presented in Figures 4.53 and 4.54, respectively.

Moment-rotation hysteretic relationships, recorded during testing are shown in Figure 4.55. The results indicate that about 21% of total rotation was caused by anchorage slip and the remaining 79% by flexure.

#### 4.6.1.7 Column CFCL7

Column CFCL7 was identical to CFRP6 except for the reduced level of axial compression. The constant axial load applied was reduced by one half, to 17% of column concentric capacity. The transverse reinforcement used was  $144 \text{ mm}^2$  which was approximately 50% more than that required by CSA S806-02 for concrete confinement. The grid spacing was 87 mm, which was equal to the maximum spacing allowed by CSA S806-02. The hysteretic relationships shown in Figure 4.56 indicate that the column behaved in an extremely ductile manner showing very little decay in moment resistance up to 4% drift. The column failed during the first cycle towards 5% drift ratio, at about 4.5% drift due to the crushing of concrete and instability of bars under compression.

Observations during testing indicated that flexural cracking started at 1% drift cycles at the third and fourth grid levels above the footing. A diagonal crack formed on a column side face during 1% drift cycles. Additional flexural and diagonal tension cracks formed during 2% drift ratio cycles. There were more diagonal tension cracks observed in this specimen than all others tested previously. This could be attributed to the lower level of axial compression applied. Concrete at one of the corners started developing vertical cracks, indicating the initiation of cover spalling. Further increase in cracks was observed during 3% drift cycles, extending beyond 400 mm from the base. The cover concrete started spalling off during the first cycle at 4% drift. One of the bars on the north side ruptured in tension. A loud sound was heard during the next cycle, indicating buckling of a bar in compression on the south side. At this point a portion of corner concrete had already spalled off. Additional rupturing of tension bars was observed on the south side during the last cycle at 4% drift ratio and the first loading towards 5% drift ratio, resulting in severe strength degradation. Figure 4.57 illustrates the performance of column CFCL7 at selected stages of loading.

Strain gauge readings indicated that FRP bars experienced a maximum of 1.2% and 0.66% strains in tension and compressive at the end of test, respectively. The FRP grids experienced a maximum of 0.43% tensile strain. Hysteretic behaviour of recorded strains of bars and grids are presented in Figures 4.58 and 4.59, respectively.

Moment-rotation hysteretic relationships recorded during testing are shown in Figure 4.60. The results indicate that at peak load approximately 60% of total rotation was caused by anchorage slip and the remaining 40% by flexure.

#### **4.6.1.8 Column CFCL8**

Column CFCL8 was companion to Column CFCL3, except for its shorter length of 1000 mm. This resulted in a shear span of 1280 mm and associated increase in shear force reversals. The longitudinal reinforcement consisted of 12 - 9.5 mm diameter CFRP bars, resulting in 0.7% reinforcement ratio. The column was confined with 9-cell CFRP grids. This resulted in 192 mm<sup>2</sup> of transverse reinforcement in each cross-sectional direction. This amount was equal to 43% of the amount required by CSA S806-02 for concrete confinement. The grid spacing was 175 mm, which was approximately twice the maximum spacing permitted by CSA S806-02. The column was subjected to a constant compressive force of 30 % of its concentric capacity. The hysteretic relationships shown in Figure 4.61 indicate that the column showed stable hysteresis loops up to 2% drift ratio, but developed severe strength decay during the second cycle at 3% drift. The column failed during pulling at the second cycle of 3% drift due to the instability of FRP bars in compression and crushing of concrete.

Observations during testing indicated that flexural cracking started at 1% drift cycles at locations of the first three grids from the base. Some hairline diagonal cracks were also observed at this drift level. The cracks became wider during the first cycle at 2% drift. During the third cycle at 2% drift, the concrete cover started to spall off. Furthermore, some FRP bars buckled under compression. Large pieces of spalled concrete cover were observed during 3% drift cycles. This triggered the buckling of additional compression bars, resulting in rapid strength degradation. Four bars on each side were completely buckled at the second cycle of 3% drift. Figure 4.62 illustrates the performance of column CFCL8 at selected stages of loading.

Strain gauge readings indicated that FRP bars experienced a maximum of 0.84% and 0.63% strains in tension and compression at the end of test, respectively. The FRP grids experienced a maximum of 0.56% tensile strain. Hysteretic behaviours of recorded strains during testing are presented in Figures 4.63 and 4.64.

Moment-rotation hysteretic relationships recorded during testing are shown in Figure 4.65. The results indicate that about 30% of total rotation was caused by anchorage slip and the remaining 70% by flexure.

#### **4.6.1.9 Column CFCL9**

Column CFCL9 was companion to CFRP8 and was reinforced with 12 - 9.5 mm diameter CFRP bars, which resulted in 0.7% reinforcement ratio. It was confined with 9-cell CFRP grids. The area of transverse reinforcement in each cross-sectional direction was 192 mm<sup>2</sup>. This was 50% more than the amount required by CSA S806-02 for concrete confinement. The spacing of grids was 88 mm, which was equal the maximum spacing permitted by CSA S806-02. The column was subjected to a constant compressive force of 30% of its concentric capacity. It showed better behaviour than the companion column CFCL8. The reduction in grid spacing increased the effectiveness of confinement, while also providing additional capacity for shear. The hysteretic relationships shown in Figure 4.66 indicate that the column showed stable hysteresis loops up to 3% drift ratio. However, it developed significant strength decay during the cycles at 4 % drift ratio. The column failed during the second cycle of 4 % drift ratio due to the stability failure of longitudinal bars, followed by the crushing of core concrete.

Observations during testing indicated that flexural cracking started at 1% drift cycles. Three horizontal cracks formed within a portion 380 mm above the column-footing interface. Two hairline diagonal cracks were observed at 1% drift ratio. More flexural cracks formed when the column deformed to 2% lateral drift at grid locations, within 380 mm from the footing. Similarly, additional diagonal cracks formed within the lower 350 mm segment of column, starting from the column footing interface. The flexural cracks begun to form within a region 400 mm above the footing at 3% drift cycles. Some of the extreme compression bars failed due to instability when the column was pushed during the second cycle at 3% drift. The horizontal crack at the first grid location widened significantly upon load reversal. The remaining bars in extreme compression layer failed in instability when the column was

loaded to 4% lateral drift ratio. This resulted in a significant drop in lateral load resistance. Figure 4.67 illustrates the performance of Column CFCL9 at selected stages of loading.

Strain gauge readings indicated that FRP bars experienced a maximum of 0.9 % tensile strain and 0.66% compressive strain at the end of test, respectively. The FRP grids experienced a maximum of 0.77% tensile strain. The relatively higher strains observed in compression and transverse reinforcement is indicative of better control of the stability of compression bars by the grids, which delayed bar buckling. The hysteretic behaviours of strains in longitudinal and transverse reinforcement are presented in Figures 4.68 and 4.68, respectively.

Moment-rotation hysteretic relationships, recorded during testing, are shown in Figure 4.70. The results indicate that approximately 30% of total rotation was caused by anchorage slip, and the remaining 70% by flexure.

#### **4.6.1.10 Column CFCL10**

Column CFCL10 was identical to CFRP9, except for the level of axial compression under which it was tested. The column was subjected to a reduced axial force of 15% of its concentric capacity. The reduction in axial compression improved the deformability of column. The hysteretic relationships shown in Figure 4.71 indicate that the column behaved in a ductile manner showing virtually any decay in moment resistance up to 4% drift ratio. It developed 25% decay in moment resistance at 5.5% lateral drift. The column failed during the first cycle at 6 % drift ratio due to the failure of all compression bars and the crushing of concrete.

Observations during testing indicated that flexural cracking started at 1 % drift cycles at grid locations within the lower 400 mm segment, except for the first grid level. During 2% drift cycles, new flexural cracks formed within the bottom 600 mm segment of column. In addition hairline cracks were observed on the cover, near the footing. A diagonal crack was observed at 550 mm from the footing during the third cycle at 2% drift ratio. During 3% drift cycles, the width of flexural cracks increased, cover concrete started to spall off around corner bars, and more diagonal cracks formed. Some of the extreme compression bars failed at 4% drift ratio due to instability. This resulted in significant strength degradation. During

the last cycle at 4% drift, the cover concrete spalled off, exposing grids and broken compression bars. All of the extreme layer of FRP bars failed in compression on both sides when the column was cycles at 5% drift ratio, leaving only the middle layer of reinforcement to provide resistance. The test was stopped at this stage of loading. Figure 4.72 illustrates the performance of Column CFCL10 at selected stages of loading.

Strain gauge readings indicated that FRP bars experienced a maximum of 1.25% tensile strain and 0.64% compressive strain at the end of test, whereas the FRP grids experienced a maximum of 0.53% tensile strain. Though some longitudinal bars failed in tension before the end of test, the final failure of the column was probably due to buckling of bars in compression and tensile failure of other bars. Hysteretic behaviour of recorded strains of bars and grids during testing are presented in Figures 4.73 and 4.74, respectively.

Moment-rotation hysteretic relationships, recorded during testing, are shown in Figure 4.75. The results indicate that approximately 38% of total rotation was caused by anchorage slip and the remaining 62% by flexure. The contribution of anchorage slip rotations in this column was somewhat higher than the companion column CFCL9 due to the reduced axial compression.

#### **4.7 Effects of Test Parameters**

The significance of tests parameters is assessed in this section by evaluating the test results presented in the previous section, where hysteretic force-deformation relationships are plotted. The effects of test parameters on strength and deformability are investigated by comparing the envelopes of hysteretic relationships of companion columns.

The test parameters considered in this phase of the experimental study are as follows:

- 1- Spacing of Transverse Reinforcement (CFRP Grids)
- 2- Arrangement of Longitudinal CFRP Bars
- 3- Axial Load Level
- 4- Shear Span



#### **4.7.1 Effect of Spacing of Transverse Reinforcement (Grids)**

Spacing of FRP transverse reinforcement is an important parameter that affects the seismic behaviour of a FRP reinforced concrete column. Transverse reinforcement provides shear resistance, confines concrete and provides stability to compression bars. Effects of two levels of grid spacing were investigated by testing four long and four short columns.

Columns CFCL1 and CFCL2 had an 8-bar arrangement, with a long shear span to promote flexural behaviour. They had two different grid spacings. Figure 4.76 shows the comparison of the envelopes for force-deformation hysteretic relationships. While the increase in flexural capacity due to reduced grid spacing was limited to 16%, the improvement in lateral drift capacity was approximately 200%. Similarly, the comparison of Columns CFCL3 and CFCL4 with a 12 bar arrangement, shown in Figure 4.76, indicates a similar trend. However, the improvements observed are not as high as those for the previous case, perhaps because of the lower-than-expected strength and deformability of CFCL1, which may be attributed to the out-of-plane bending observed in this column near the end of testing. The improvements observed in columns with close spacing of grids can be explained by the improved stability of compression bars and enhanced concrete confinement.

The force-deformation envelopes for four short columns are compared in Figure 4.77. The column strength was not affected in any significant way by reducing the grid spacing. However, the columns with closer grid spacing had improved deformability. The lateral drift capacity of Column CFCL9 was approximately 30% more than that of Column CFCL8 with wide grid spacing. It was observed that shear was not critical in the short columns tested. Even the widely spaced grids used in some of the columns were sufficient in providing adequate shear resistance. The presence of axial compression in columns, unlike beams discussed in Chapter 5, helped increase shear resistance by delaying the formation of diagonal tension cracks. Hence, the improvement observed in columns with close grid spacing is attributed to the improved stability of compression bars and concrete confinement rather than increased shear resistance.

#### **4.7.2 Effect of Arrangement of Longitudinal FRP Bars**

The arrangement of longitudinal reinforcement affects concrete confinement. In steel-reinforced concrete columns, reinforcement cages consisting of closely spaced longitudinal and transverse reinforcement provide better resistance to lateral expansion of compression concrete, improving their inelastic deformability. The effect of reinforcement arrangement was investigated in this phase of the experimental program by testing columns with 8-bar and 12-bar arrangements. The comparison of force-displacement envelopes for long and short columns is illustrated in Figures 4.78 and 4.79, respectively. Columns with 12-bar arrangements exhibit slightly improved strength and deformability. While the improvements are small in all cases, this effect is more pronounced in flexure dominant long columns.

#### **4.7.3 Effect of Axial Load Level**

It has been observed in steel reinforced concrete columns that column deformability decreases with increasing axial compression. The effect of axial load was investigated in this phase of research by testing columns with different levels of constant axial compression. Two pairs of short columns, one with an 8-bar arrangement and the other with a 12-bar arrangement were subjected to two levels of axial compression prior to the application of inelastic deformation reversals. The force-deformation envelopes are compared in Figure 4.80. Columns CFCL6 and CFCL7 were loaded to about 33% and 17% of their concentric load capacity,  $P_o$ , respectively. Column CFCL6, under higher axial compression experienced 25% more lateral force and moment capacity when compared with Column CFCL7. Furthermore, the deformability of the column was reduced when compared with Column CFCL7, which developed 50% higher drift ratio. Similarly, Column CFCL9 with 30%  $P_o$  had 13% higher moment capacity than Column CFCL10 with 15%  $P_o$  as axial load. The latter column showed 50% more inelastic deformability than the former column, confirming earlier observations.

#### **4.7.4 Effect of Column Shear Span**

Two column lengths (shear spans) were considered in the experimental program. The intent was to promote flexural behaviour in long columns, while promoting shear behaviour in short columns. The envelopes of force-deformation relationships are compared in Figure 4.81. Because none of the columns failed in shear, there was no distinct effect of shear span on flexural capacity. However, the rate of strength degradation was faster in short columns, caused by higher shear stress reversals, which produced more diagonal tension cracking and crisscrossing of these cracks, deteriorating concrete at higher deformation levels.

#### **4.8 Ductility**

A measure of inelastic deformability in reinforced concrete structural elements is ductility ratio. The ductility ratio is expressed as the ratio of maximum inelastic deformation to deformation at yield. These can be sectional or member deformations, such as curvatures or rotations and displacements. In FRP reinforced concrete, the concept of yielding has to be clarified as FRP materials show linear elastic behaviour up to failure, without the ability to yield, though research is underway at various institutions to emulate steel yielding in FRP bars through the use of hybrid fibers and/or geometric alignment of fibers. However, almost all FRP reinforcement currently available for use in the construction industry lacks yielding and associated plastic behaviour.

In the absence of material yielding, inelastic force-deformation relationship of FRP reinforced concrete can only be attained when the behaviour is governed by the crushing of concrete, and the concrete is sufficiently confined to develop substantial inelasticity prior to crushing. A column under high axial compression may develop a point in the force-deformation relationship at which the deformations increase at a very high rate with little increase in force or moment resistance. This level of deformation may be labeled as “yield deformation.” It occurs as the compression concrete begins to crush, while the tension FRP reinforcement remains elastic. If the concrete is sufficiently confined, the member may develop inelastic deformations that are significantly higher than its yield deformation. For such structural members one can talk about a ductility ratio, which can be used as a useful

numerical tool to express the level of inelasticity. The displacement ductility factor  $\mu_{\Delta}$ , reflects the overall member deformability and is defined as follows:

$$\mu_{\Delta} = \frac{\Delta_u}{\Delta_y} \quad (4.3)$$

Where,  $\Delta_y$  and  $\Delta_u$  are displacements at yield and ultimate, respectively. For seismic design of a multi-storey, multi-bay structure, it is usually acceptable to tolerate some strength decay in some of the members, while the overall structure continues to resist seismic loading. In this research program 20% strength decay has been taken as an acceptable level of decay before a member is declared to have failed. Therefore, the deformation at 20% strength decay is used as the ultimate deformation. This is shown in Figure 4.82.

Table 4.3 provides a summary of displacement ductility ratios for the columns tested in Phase III of the experimental program. The results indicate that the displacement ductility ratio varied between 2.7 and 3.9. An earlier experimental investigation, also conducted at the Structures Laboratory of the University of Ottawa by Grira and Saatcioglu (1988), involving similar columns but reinforced with steel bars and FRP grids, was examined for comparison. Table 4.4 provides ductility ratios obtained from this test program. The results indicate that for the same reinforcement arrangements as those considered in FRP reinforced columns, the steel reinforced columns developed ductility ratios ranging between 2.5 and 4.6. These values indicate similar ductility ratios can be attained in steel reinforced and FRP reinforced columns, especially if the axial load is at or above the balanced load.

An alternative to displacement ductility ratio is the curvature ductility ratio, which is easier to compute, as inelastic sectional analysis of a flexure dominant element can be carried out with relative ease. The curvature ductility ratio,  $\mu_{\phi}$ , is given as follows:

$$\mu_{\phi} = \frac{\phi_u}{\phi_y} \quad (4.4)$$

Where,  $\phi_y$  and  $\phi_u$  are the curvatures at yield and ultimate, respectively. In the experimental program, average curvatures were measured and recorded within the potential plastic hinging region. This is explained in Section 4.4. Because these curvature values gave average values within the gauge length, where the gauge length was equal to  $h = 355$ , the moments use to plot moment-curvature relationships were also average moment within the same length. This implied taking moments at  $h/2$  since the variation of moment was linear. This is shown in Figures 4.85 and 4.86 for long and short columns, respectively, for those specimens where the readings at high deformation range were available. The curvature ductility ratios are also tabulated in Table 4.3. The results indicate that the curvature ductility ratios for long and short columns varied between 3.2 and 4.75.

## 4.9 Conclusion

The following conclusions can be drawn from the column tests conducted in Phase III:

- The majority of columns were tested under sufficiently high levels of axial compression to be above the balanced load, with the exception of two, which were subjected to lower levels of axial loads. Therefore, their behaviour was governed by the crushing of concrete, prior to the rupturing of FRP reinforcement in tension. The failure in these columns was initiated by the spalling of concrete cover, followed by the buckling of FRP bars in compression and subsequent crushing of the core concrete.
- The failure of FRP bars in compression was due to instability. Although some bars experienced up to 0.7% compressive strain at the end of test, other bars at the same level buckled at the lower compressive strain followed by strength decay. Some bars buckled at approximately 0.39% to 0.44% when the transverse grid spacing was equal to 175 mm, corresponding to 18 times the bar diameter. When the grid spacing was reduced to 88 mm, corresponding to 9 times the bar diameter, the buckling compressive strain recorded varied between 0.49% and 0.54%. The strain values at failure indicates full compatibility with the surrounding concrete until the lateral restraint provided by concrete is lost due to the spalling of cover concrete and the

crushing of core concrete. This implies that the FRP bars were able to provide resistance in compression; until after the surrounding concrete crushes.

- The stability failure of FRP bars in compression was different than that observed in steel reinforcing bars. Unlike steel bars, the FRP bars were not able to deform plastically, developing significant bending before they became unstable. Instead, no sign of bending was observed and the bars broke into pieces when they became unstable, as if they were cut at the failure section. The failure section resembled a clean cut by a cutter, rather than delamination or rupturing of fibers. This can be explained by the lack of resistance of FRP bars against bending. No sign of material failure was observed in compression.
- Tension FRP reinforcement ruptured only in two columns where the level of axial compression was below balanced load. The tension failure in these bars was limited to a few bars, and the final column failure was attributed to the buckling of compression bars and the crushing of concrete. The type of tension failure was very similar to that observed in coupon tests, where the failure took place within approximately 150 mm segment of the bar, with fibers rupturing randomly at different locations, displaying clear delamination of ruptured fibers from the resin.
- None of the columns suffered from shear failure, including those with short shear spans.
- All the columns sustained a lateral drift ratio higher than the 2% and 2.5% limits specified for earthquake resistant columns in the National Building Code of Canada (1995, 2005). The hysteretic relationships indicated progressive stiffness degradation due to concrete cracking, and inelasticity due to gradual concrete crushing. However, the shape of hysteresis loops was distinctly different than those of steel reinforced columns. The unloading branches of hysteresis loops for FRP reinforced concrete columns were aiming towards the origin, due to the elastic behaviour of FRP bars in tension.
- The FRP reinforced concrete columns tested in the current phase of the experimental program had limited inelastic deformability with displacement ductility ratios ranging between 2.7 and 3.9 depending on the level of axial compression, degree of confinement and stability of compression bars as governed by grid spacing.

Table 4.1. Properties of Phase III Specimens  
(Columns under Reversed Cyclic Loading)

| Specimen<br>Label | $f_{co}$<br>MPa | Reinforcement<br>Arrangement | $\rho$<br>(%) | $s$<br>(mm) | $L$<br>(mm) | $P$<br>(kN) | $P/P_o$<br>(%) |
|-------------------|-----------------|------------------------------|---------------|-------------|-------------|-------------|----------------|
| CFCL1             | 38              | 8-9.5 mm bar                 | 0.46          | 175         | 1900        | 1255        | 30             |
| CFCL2             | 38              | 8-9.5 mm bar                 | 0.46          | 88          | 1900        | 1255        | 30             |
| CFCL3             | 38              | 12-9.5 mm bar                | 0.70          | 175         | 1900        | 1115        | 27             |
| CFCL4             | 38              | 12-9.5 mm bar                | 0.70          | 88          | 1900        | 1115        | 27             |
| CFCL5             | 38              | 8-9.5 mm bar                 | 0.46          | 175         | 1000        | 1358        | 33             |
| CFCL6             | 38              | 8-9.5 mm bar                 | 0.46          | 88          | 1000        | 1358        | 33             |
| CFCL7             | 38              | 8-9.5 mm bar                 | 0.46          | 88          | 1000        | 679         | 17             |
| CFCL8             | 38              | 12-9.5 mm bar                | 0.70          | 175         | 1000        | 1220        | 30             |
| CFCL9             | 38              | 12-9.5 mm bar                | 0.70          | 88          | 1000        | 1220        | 30             |
| CFCL10            | 38              | 12-9.5 mm bar                | 0.70          | 88          | 1000        | 627         | 15             |

Table 4.2: Variation of Concrete Strength with Age in Phase III Specimens  
(Columns under Reversed Cyclic Loading)

| Compressive Strength of Concrete in MPa |        |         |         |         |              |
|-----------------------------------------|--------|---------|---------|---------|--------------|
| Age                                     | 7 days | 14 days | 21 days | 28 days | During Tests |
| <i>Long Columns</i>                     |        |         |         |         |              |
| Cylinder 1                              | 22.5   | 23.4    | 27.8    | 27.7    | 38           |
| Cylinder 2                              | 20.5   | 23.8    | 30.4    | 28.5    | 37           |
| Cylinder 3                              | 22.8   | 25.9    | ---     | 27.8    | 36           |
| Average                                 | 22.0   | 24.4    | 29.1    | 28.0    | 36.7         |
| <i>Short Columns</i>                    |        |         |         |         |              |
| Cylinder 1                              | 24.7   | 33.3    | 33.6    | 37.6    | 41           |
| Cylinder 2                              | 22.9   | 33.4    | 32.9    | 36.2    | 39           |
| Cylinder 3                              | 28.2   | 32.4    | 33.2    | 37.2    | -----        |
| Average                                 | 25.2   | 33.1    | 33.2    | 37      | 40           |

Table 4.3: Experimental Deformation Ductility Ratios for Columns in Phase III

| Column Label | Drift at 20% Decay | Displacement Ductility | Curvature Ductility |
|--------------|--------------------|------------------------|---------------------|
| CFCL1        | 1.5                | 1.9                    | -----               |
| CFCL2        | 3.4                | 3.9                    | -----               |
| CFCL3        | 3.5                | 2.85                   | 3.75                |
| CFCL4        | 4.5                | 3.38                   | 4.75                |
| CFCL5        | 2.8                | 2.7                    | 3.2                 |
| CFCL6        | 3.5                | 3.3                    | 4.2                 |
| CFCL7        | 4.2                | 3.2                    | 3.9                 |
| CFCL8        | 3.0                | 2.7                    | 3.4                 |
| CFCL9        | 4.0                | 3.2                    | 3.4                 |
| CFCL10       | 5.4                | 2.9                    | 3.3                 |

Table 4.4: Displacement Ductility Ratio for Concrete Columns Reinforced with Steel Bars and CFRP Grids ( Adopted from M. Grira, 1998)

| Column Label | Steel bar Arrangement | FRP Grids (mm <sup>2</sup> ) | s (mm) | P/P <sub>o</sub> (%) | Drift at 20% Decay (%) | Displacement Ductility Ratio (%) |
|--------------|-----------------------|------------------------------|--------|----------------------|------------------------|----------------------------------|
| FRP-1        | 8#20                  | 6x10(4-cell)                 | 152    | 39                   | 2                      | 2.5                              |
| FRP-2        | 8#20                  | 6x10(4-cell)                 | 76     | 39                   | 4.1                    | 4                                |
| FRP-3        | 8#20                  | 6x10(4-cell)                 | 76     | 20                   | 6                      | 4.6                              |
| FRP-6        | 12#20                 | 6x10(9-cell)                 | 76     | 20                   | 7                      | 4.6                              |
| FRP-9        | 12#20                 | 6x8(9-cell)                  | 76     | 39                   | 4                      | 3.4                              |
| FRP-10       | 12#20                 | 6x8 (4-cell)                 | 76     | 20                   | 6                      | 3.8                              |
| FRP-11       | 8#20                  | 8x10(4-cell)                 | 76     | 36                   | 3                      | 3                                |
| FRP-12       | 12#20                 | 8x10(9-cell)                 | 76     | 38                   | 4                      | 3.4                              |



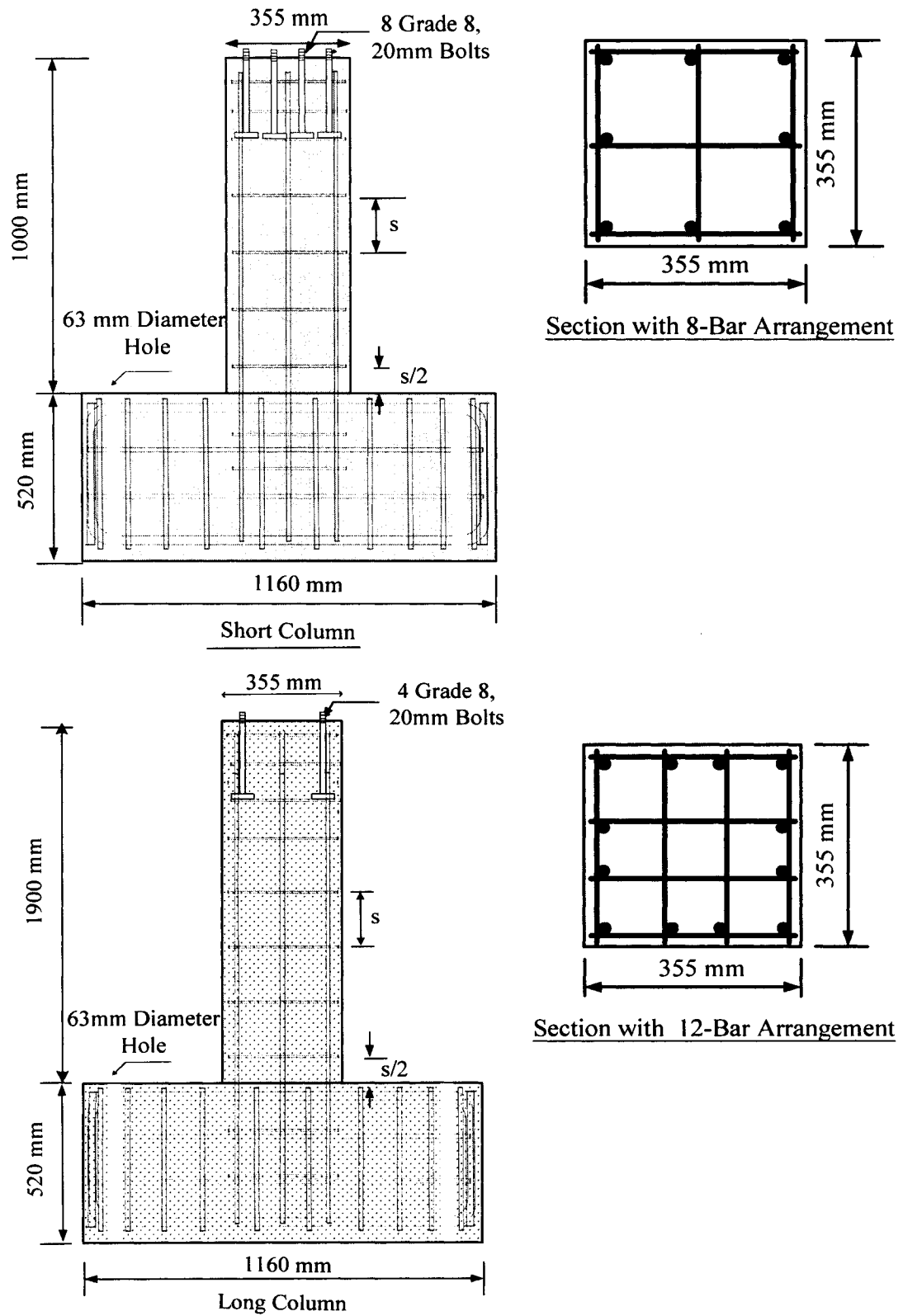
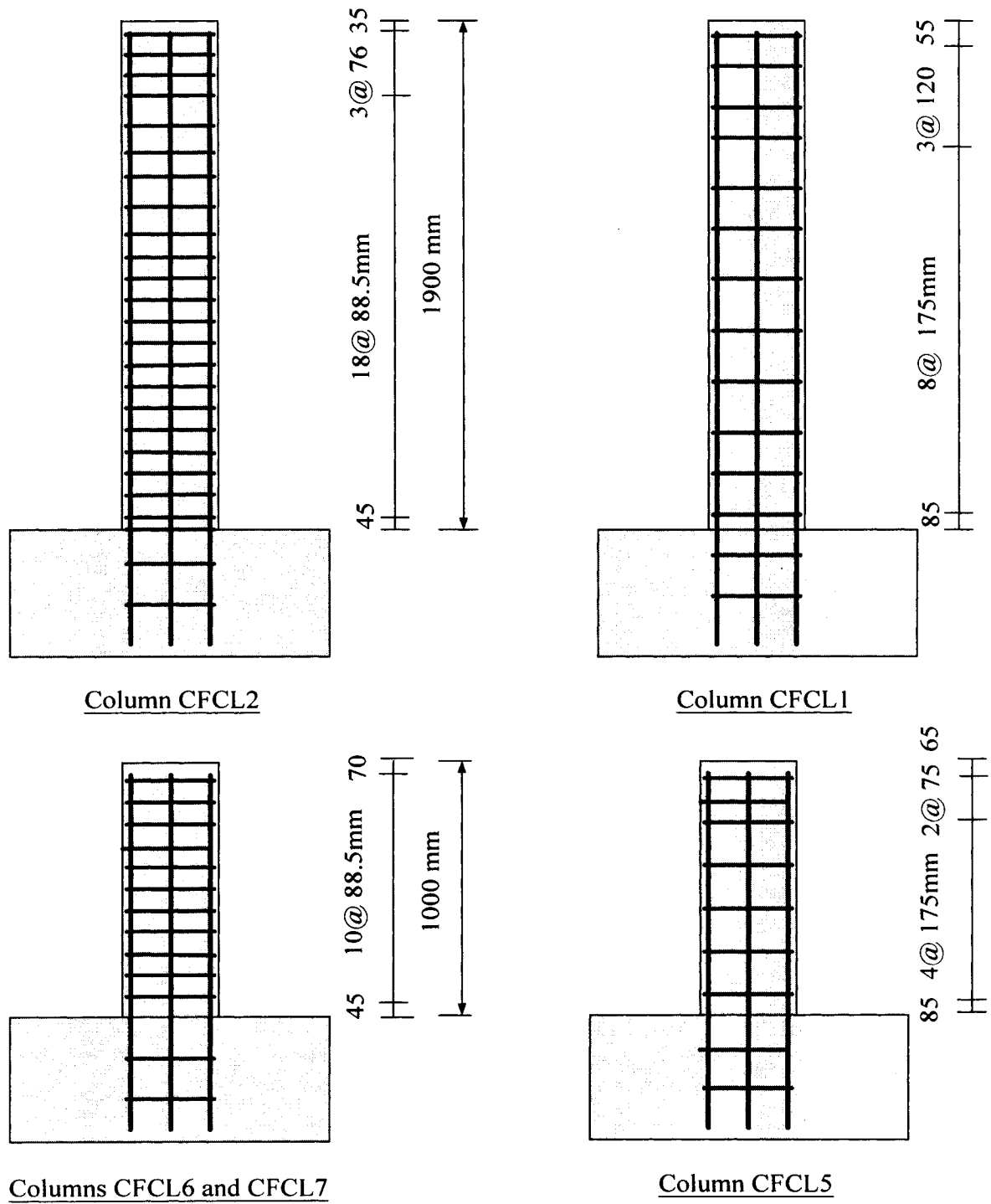
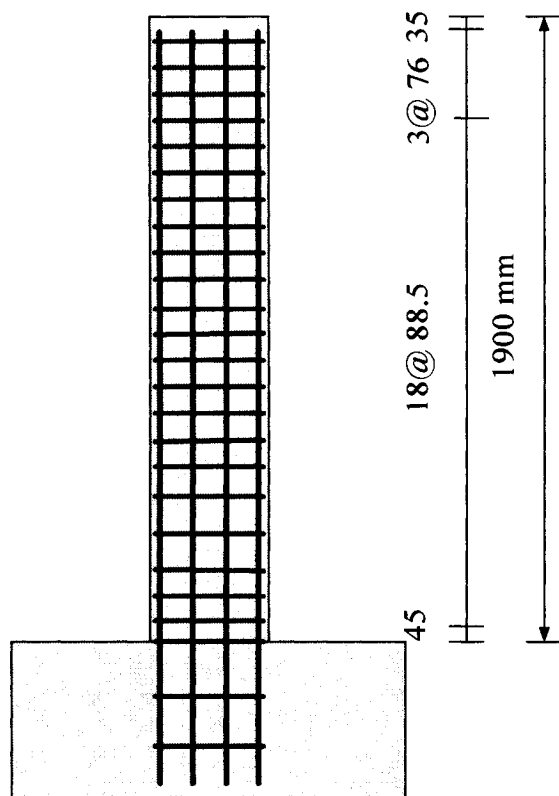


Figure 4.1: Geometric Details of Phase III Specimens  
(Columns Tested under Reversed Cyclic Loading)

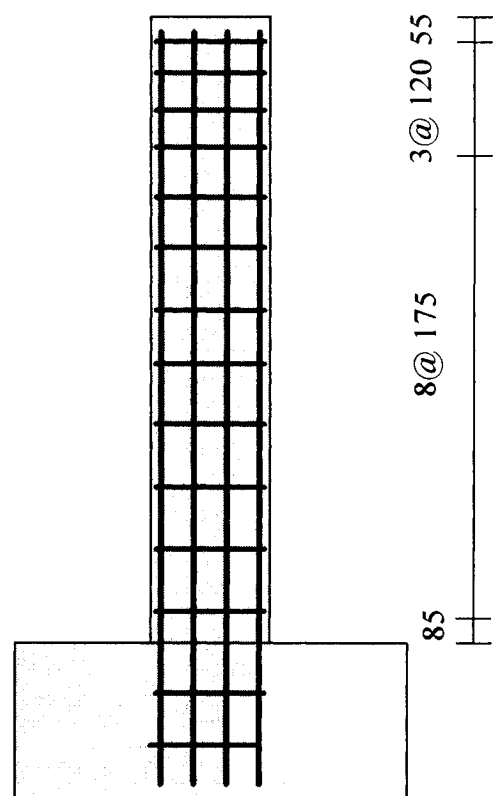


(a) Elevation Views of Columns with 8 Bars

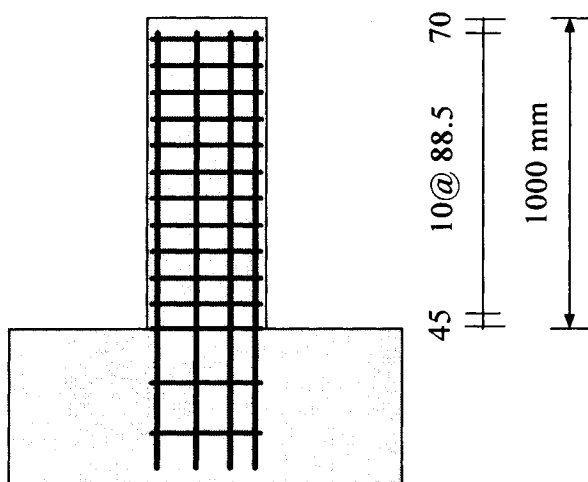
Figure 4.2 : Columns Tested under Reversed Cyclic Loading in Phase III



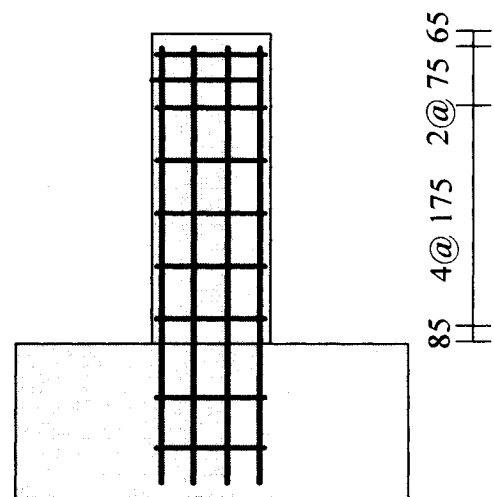
Column CFCL4



Column CFCL3



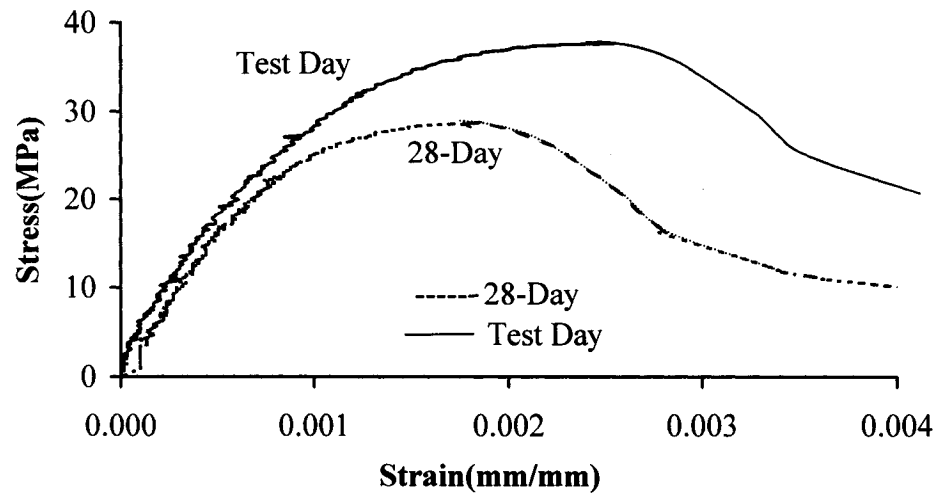
Columns CFCL9 and CFCL10



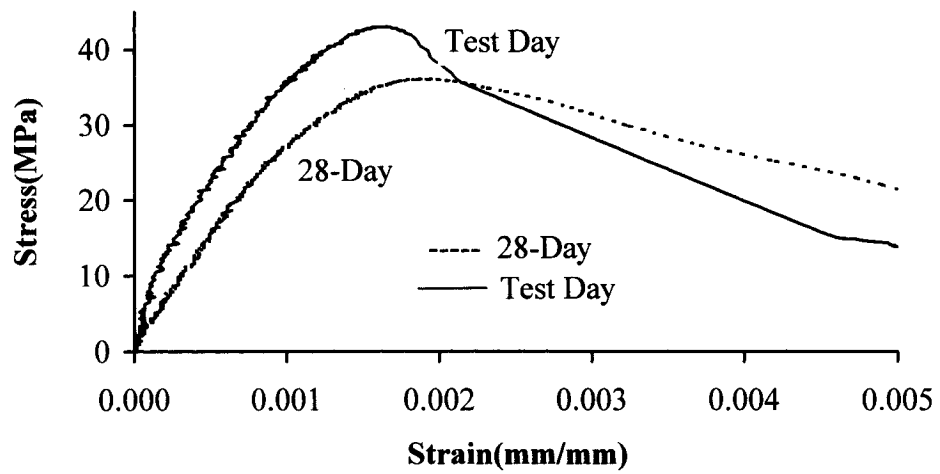
Column CFCL8

(b) Elevation Views of Columns with 12 Bars

Figure 4.2 (Cont.): Columns Tested under Reversed Cyclic Loading in Phase III



(a) Long Columns



(b) Short Columns

Figure 4.3: Stress-Strain Relationship for Plain Concrete in Phase III  
(Columns Under Reversed Cyclic Loading)

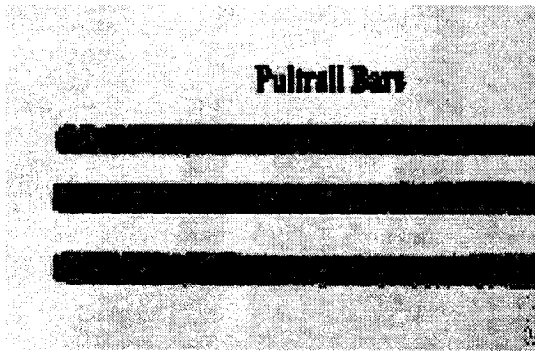


Figure 4.4: Pultrall CFRP Bars Used in Phase III Specimens

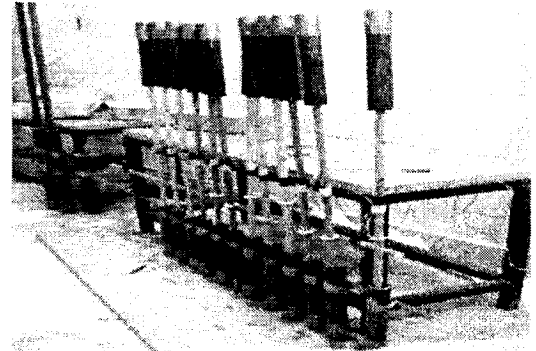


Figure 4.5: Typical Tensile Test Specimens Prior to Testing

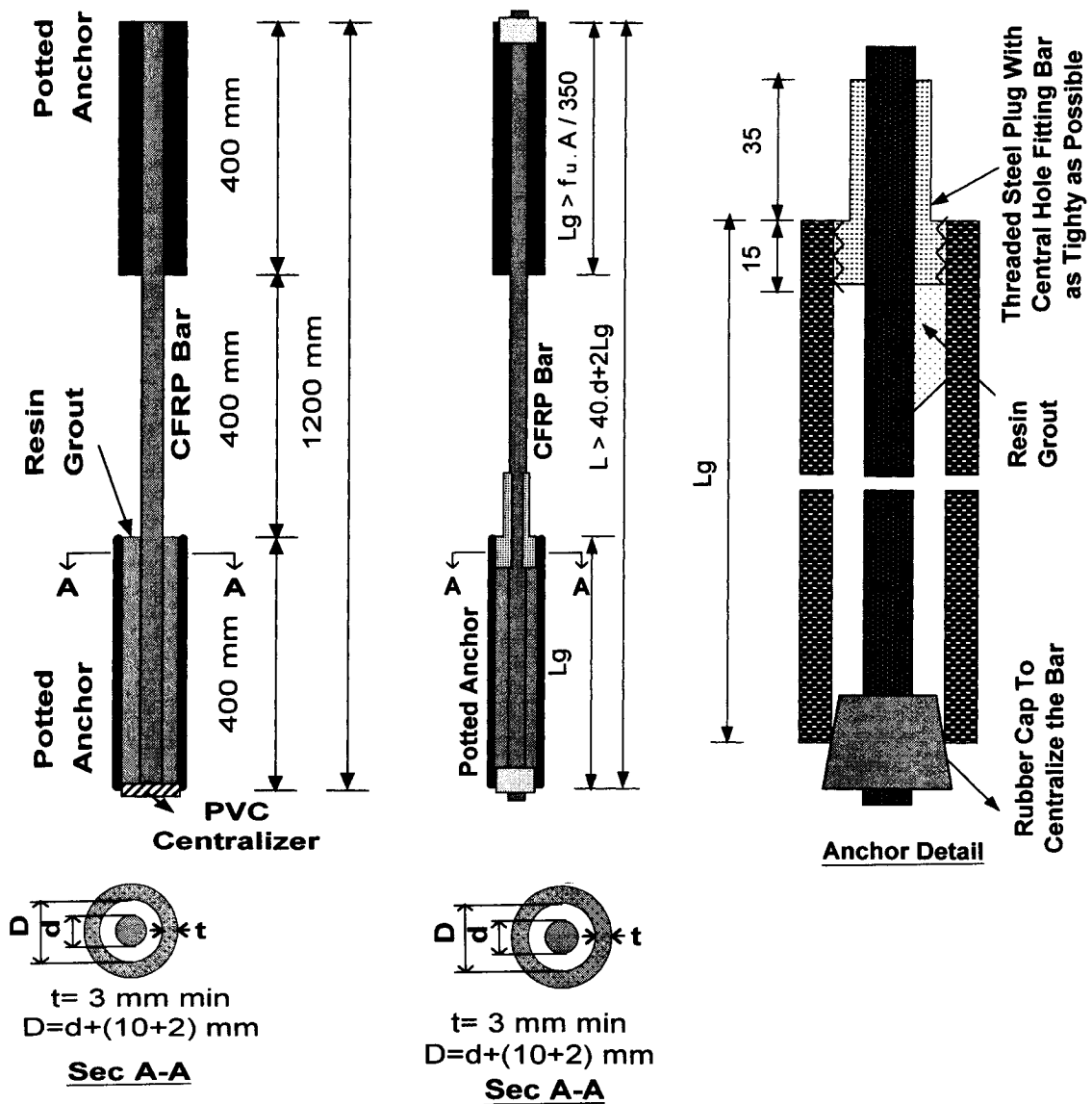
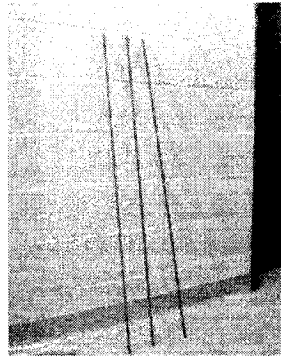
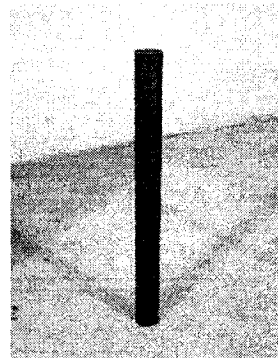


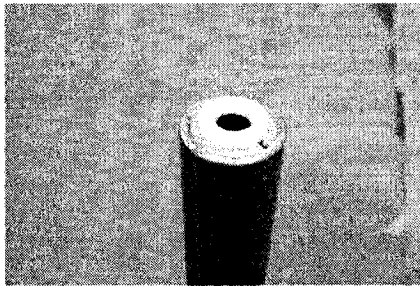
Figure 4.6: Schematic View of Tensile Test Specimens



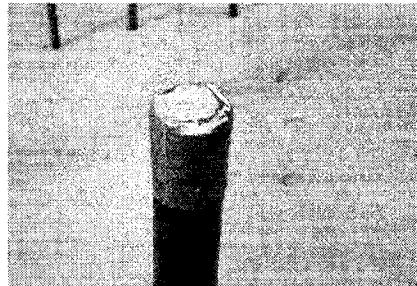
(a) CFRP Test Specimens



(b) Steel Tube for Anchorage



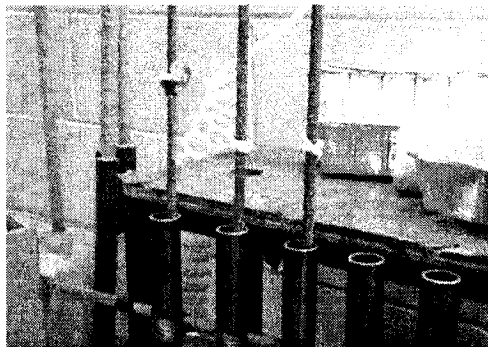
( c ) PVC Centralizers



(d) Tape at the End of Pipe



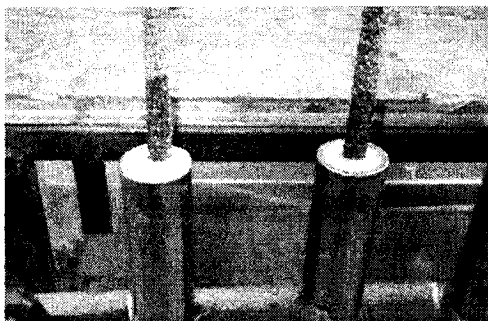
(e)Vertically Placed Tubes



(f) Placement of Bars



(g) Pouring of Grout

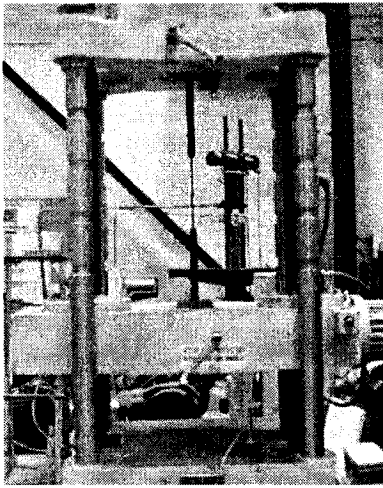


(h) Centering the Bars



( I ) Tubes at other End

Figure 4.7: Preparation of Tensile Coupons

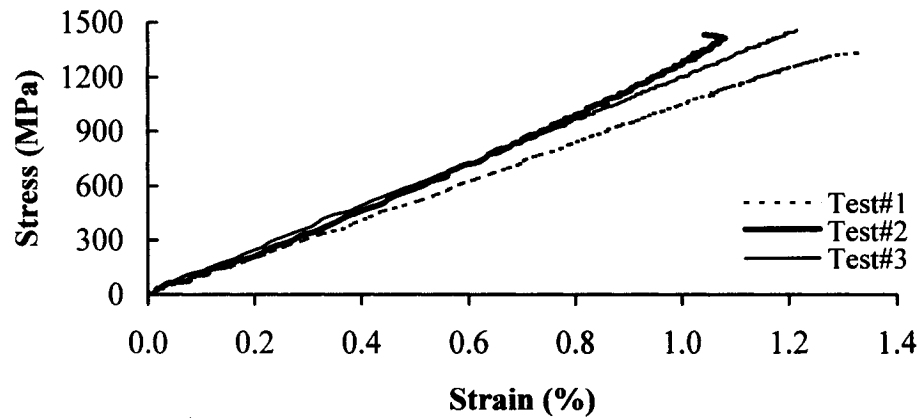


(a) General View

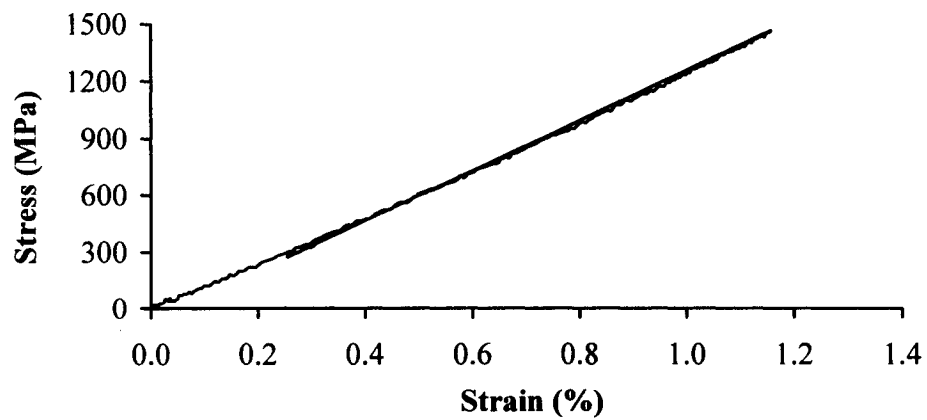


(b) Close-up View

Figure 4.8: Tensile Test Set-Up of CFRP Bars

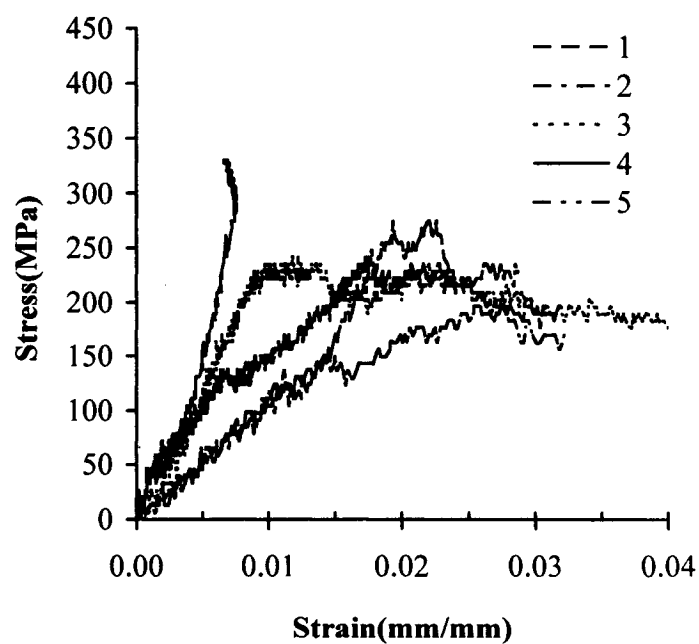
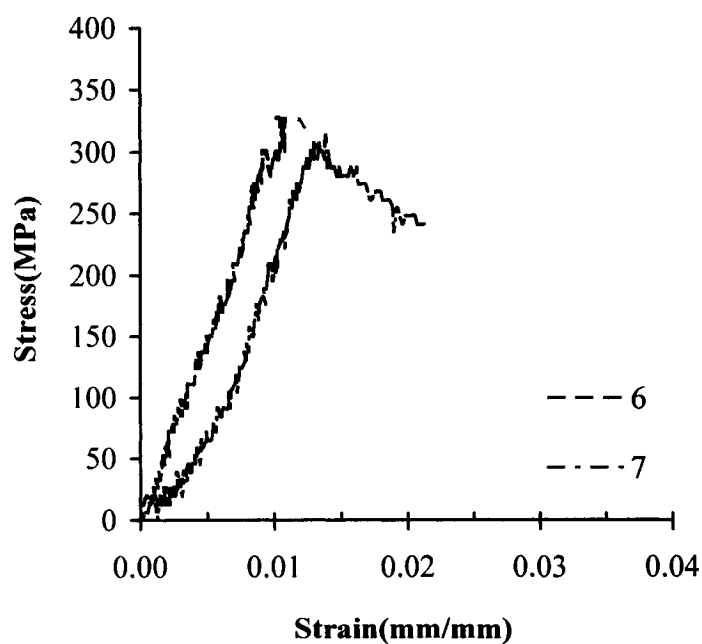
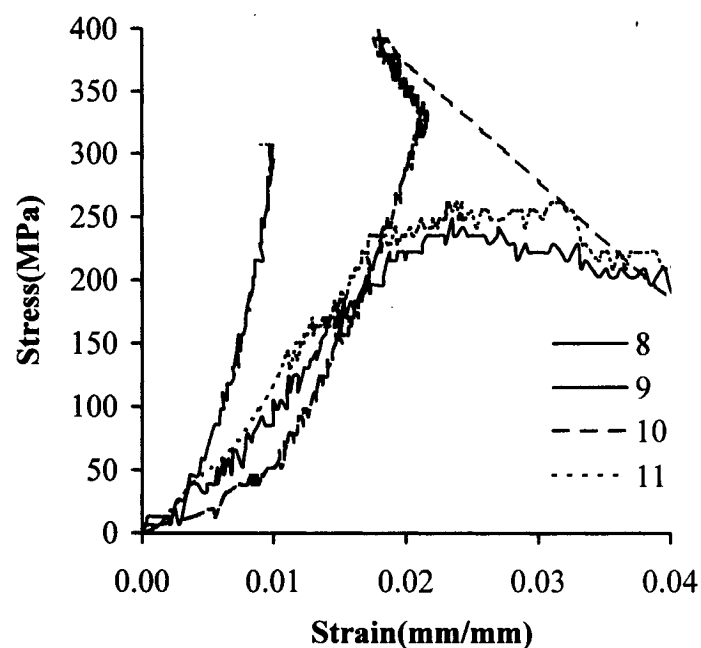
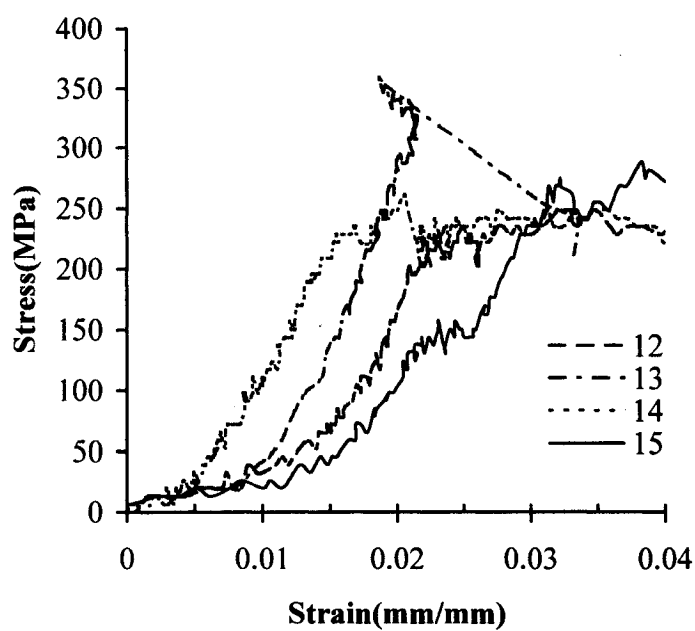


(a) Tension Tests Conducted at Sherbrooke University



(b) Tension Tests Conducted at the University of Ottawa

Figure 4.9: Stress-Strain Relationships for Pultrall CFRP Bars



(c) Compression Test of Coupons with Different  $l/d$  Ratios

Figure 4.9 (Cont.): Stress-Strain Relationships for Pultrall CFRP Bars



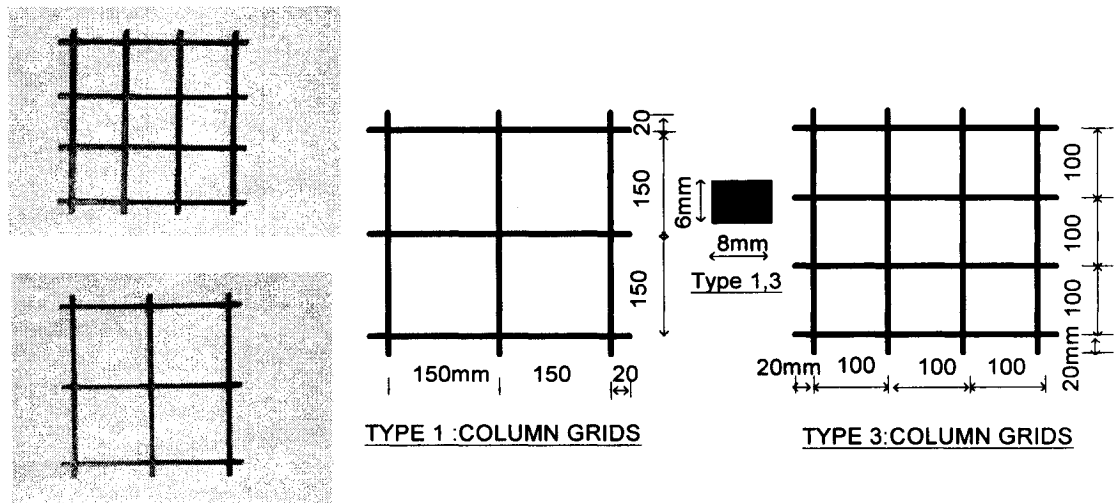


Figure 4.10: Schematic View of CFRP Grid Sections Used in Phase III Specimens

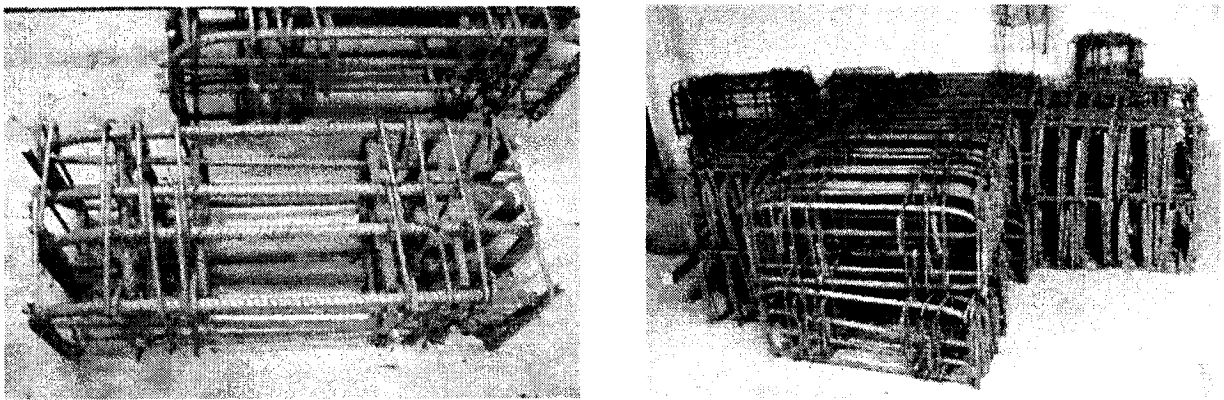


Figure 4.11: Reinforcement Cages for Column Footings

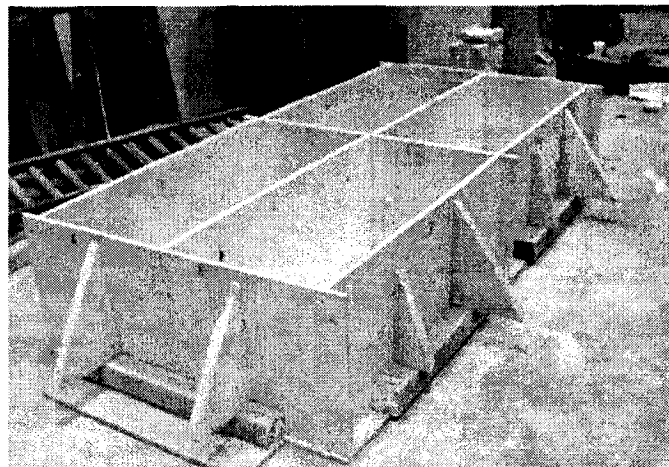


Figure 4.12: Formwork for Column Footings

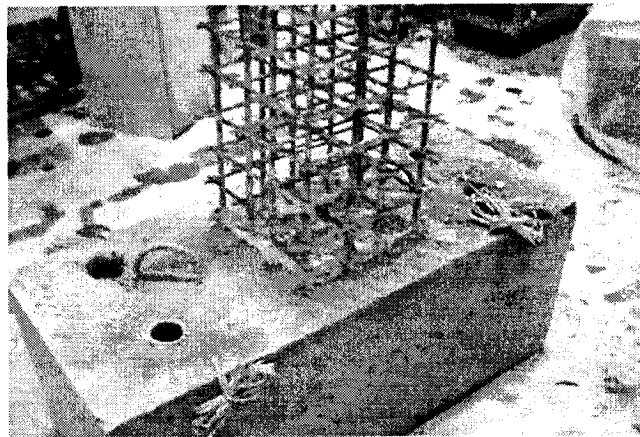
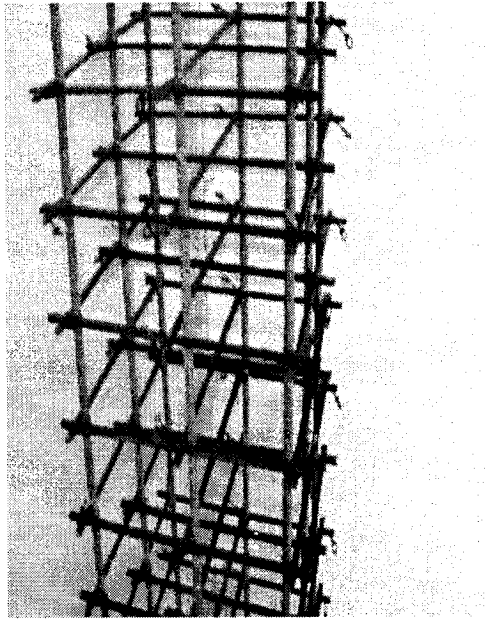
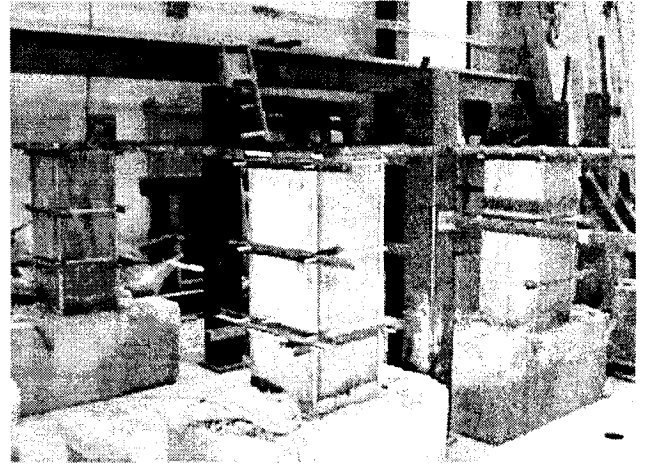
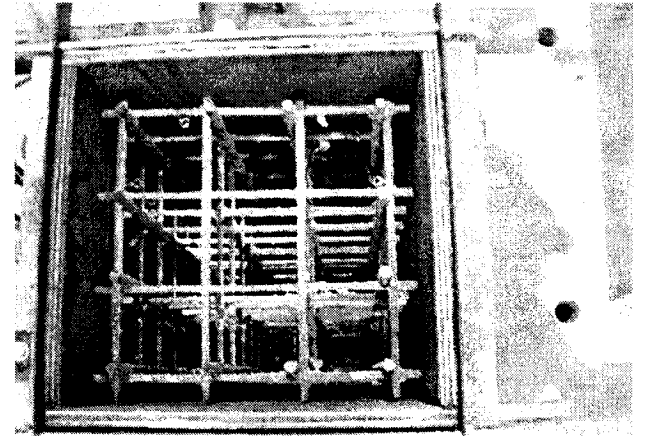
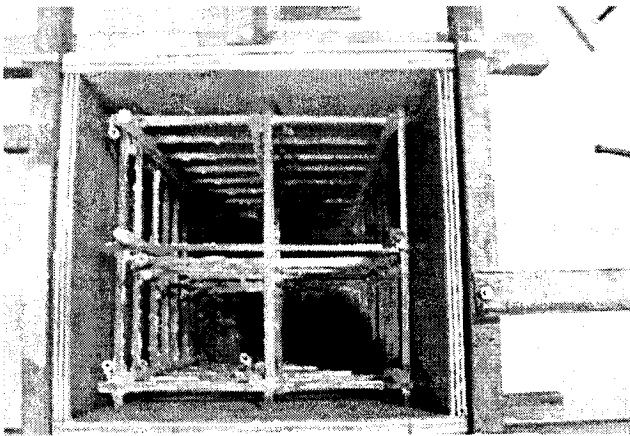


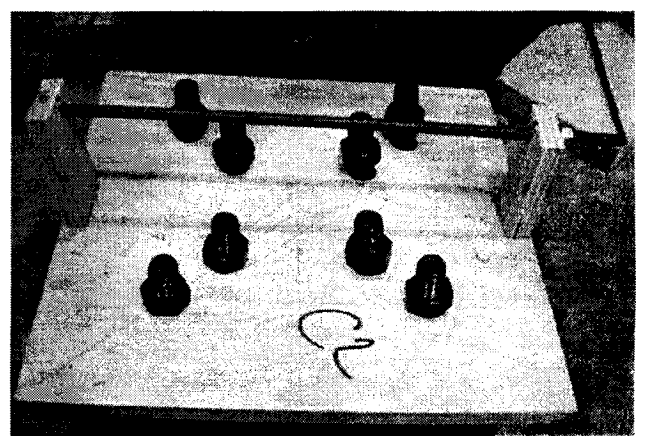
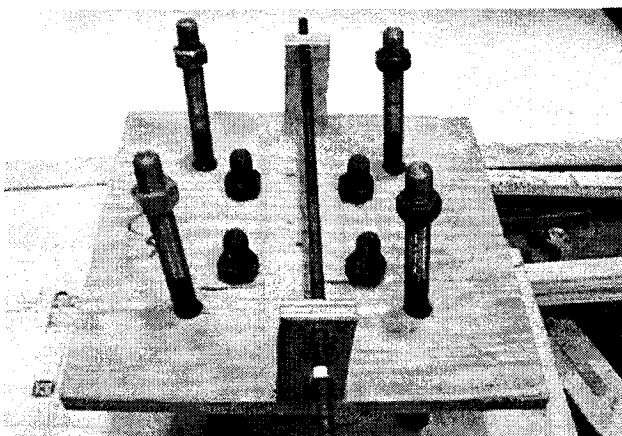
Figure 4.13: Column FRP Cages and Casting of Footings



(a) Formwork for Long and Short Columns



(b) Top Views of Columns with 8-Bar and 12-Bar Arrangements



(c) Bolts at Column ends, Cast in for Attachemnt of Loading Beams

Figure 4.14: Formwork and Preparation for Column Concrete Casting

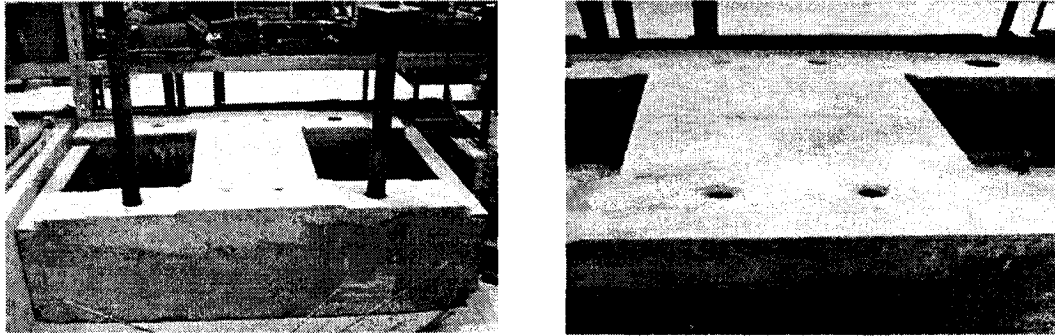


Figure 4.15 Heavily Reinforced Foundation

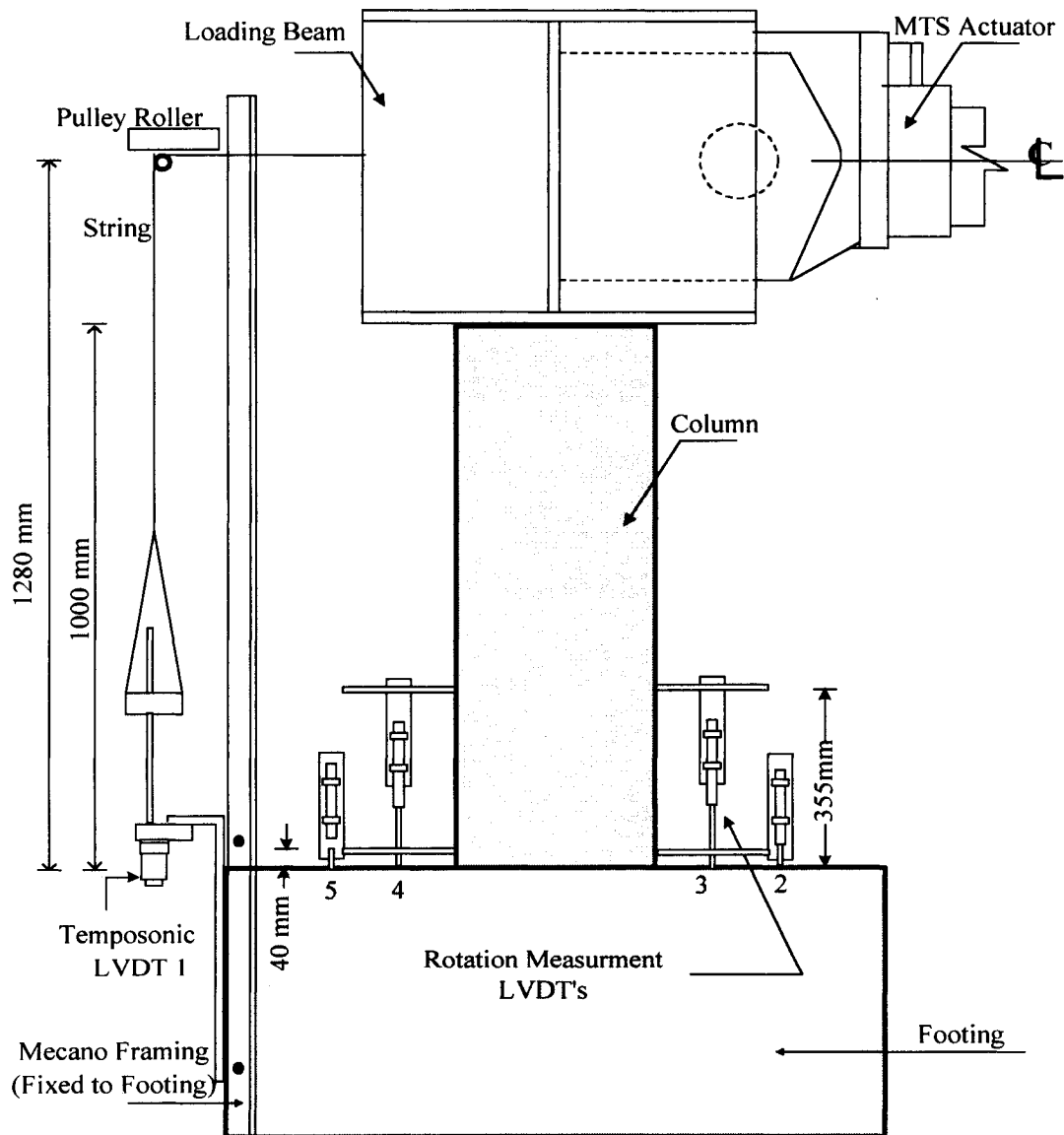


Figure 4.16: Locations of LVDTs Used in Phase III Specimens

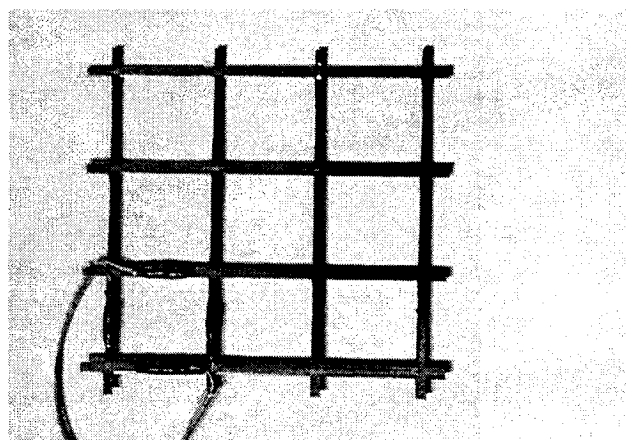
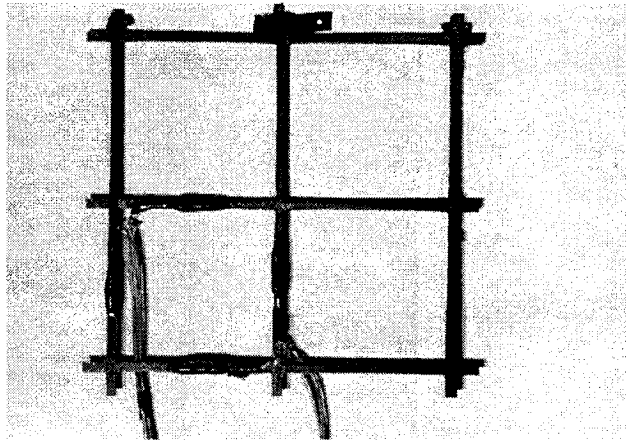
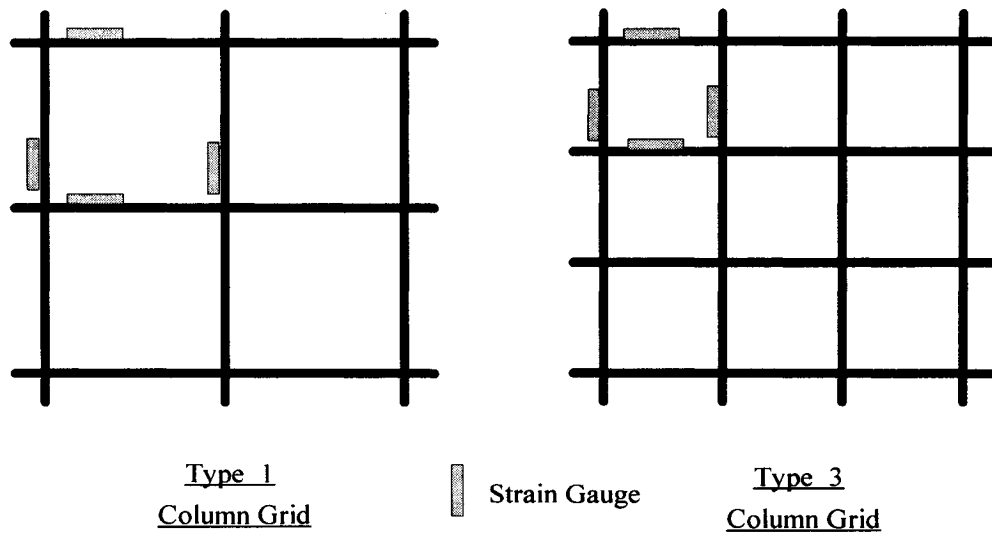


Figure 4.17: Strain Gauge Locations on CFRP Grids Used in Phase III Columns

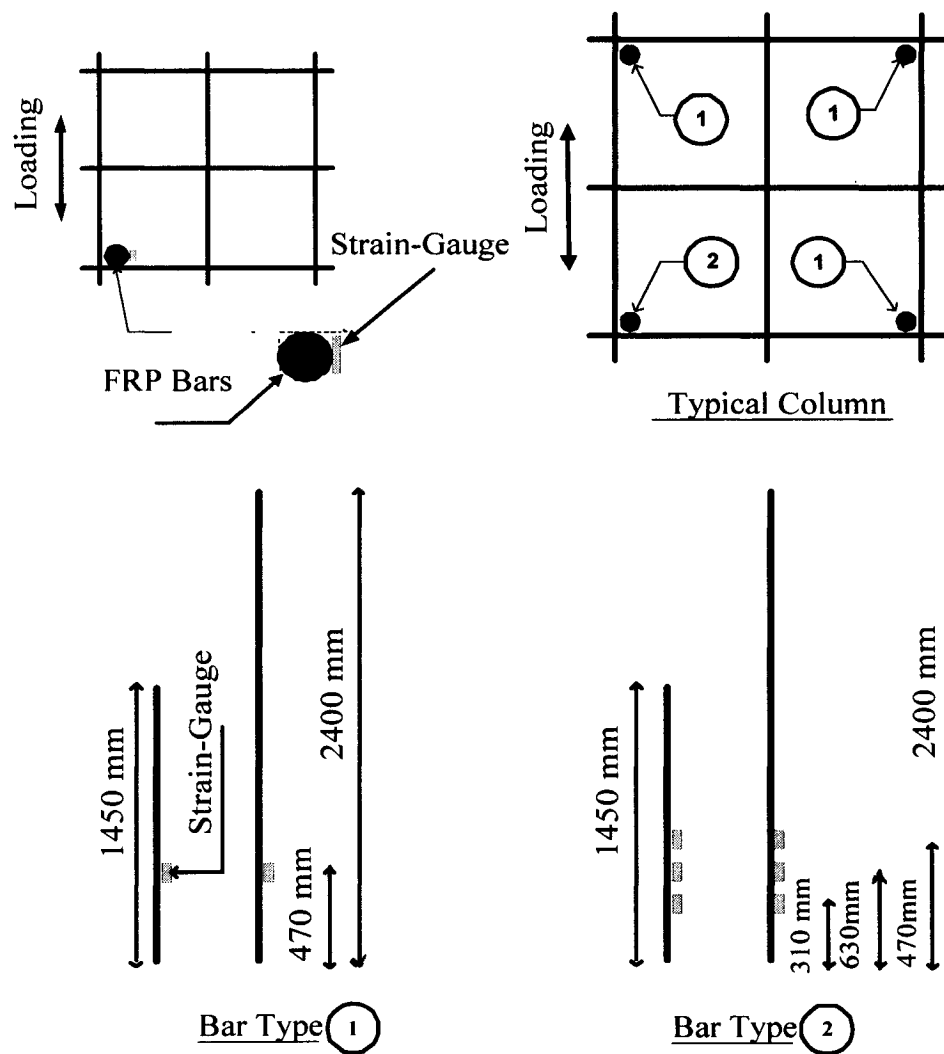
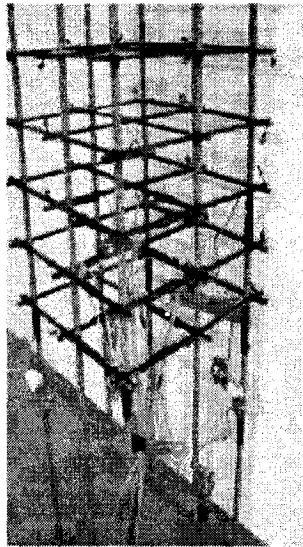
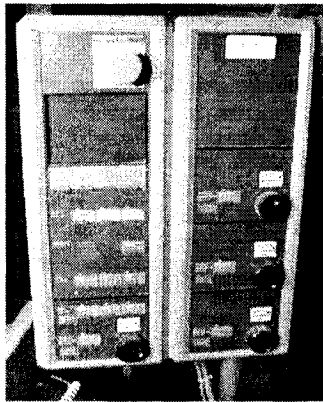
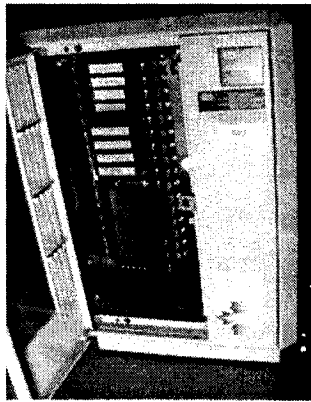


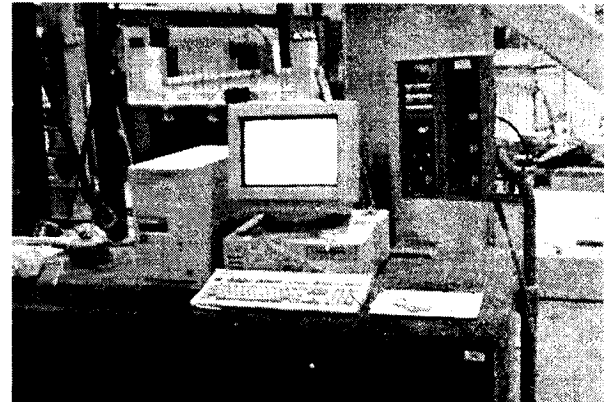
Figure 4.18: Locations of Strain Gauges on CFRP Bars Used in Phase III Columns



(a) Load Control Panel



(b) MTS Controller



(c) Overall View

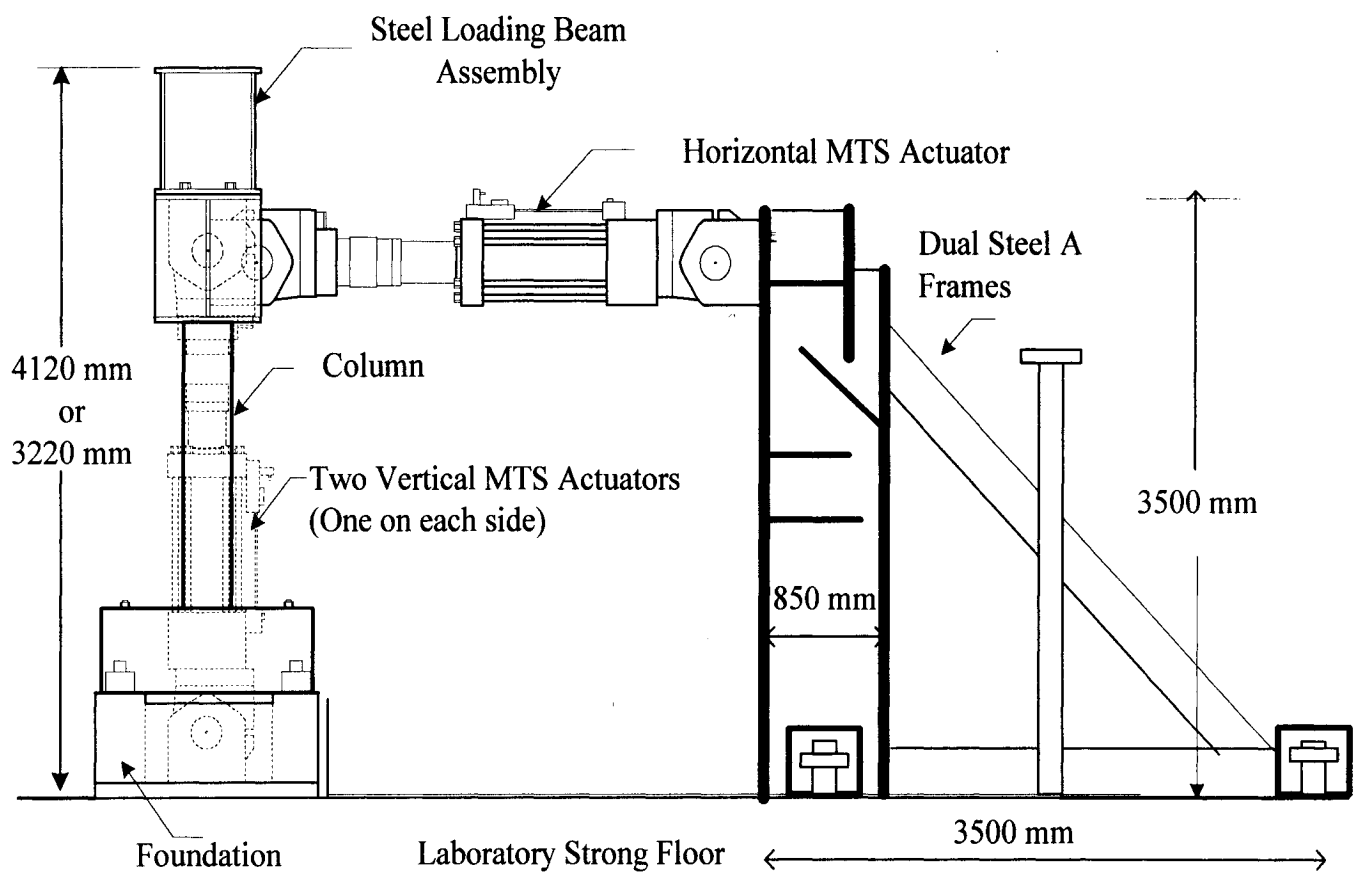
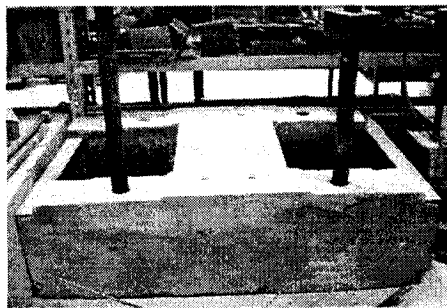
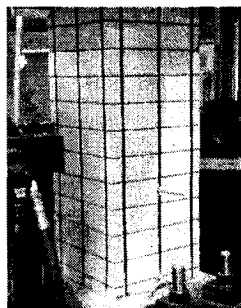


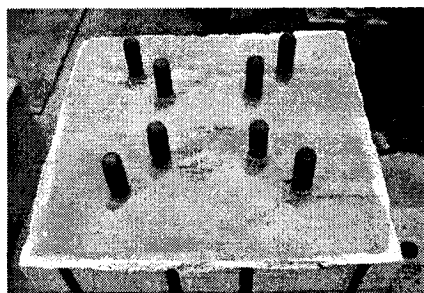
Figure 4.20: Test Set-up Used in Phase III Column Tests



(a) Fixing Foundation to Laboratory Floor



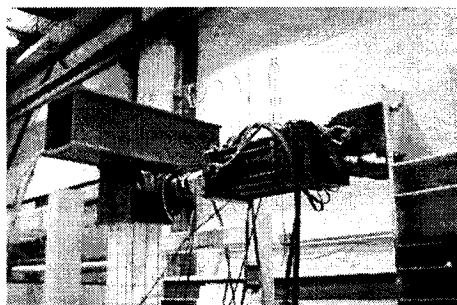
(b) Connecting a Test Column to Foundation



(c) Bolts Protruding to Fix Loading Beam



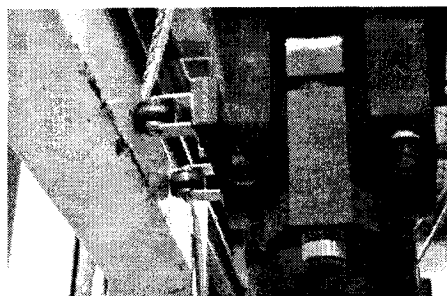
(d) Connecting the Loading Beam to Column



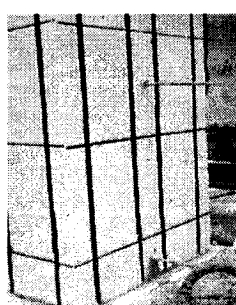
(e) Connecting the Horizontal Actuator



(f) Connecting Vertical Actuators



(g) Rollers to Prevent Out-of-Plane Movements



(h) Embedded Bars for LVDT Installation

Figure 4.21: Preparation of Column Specimens for Testing



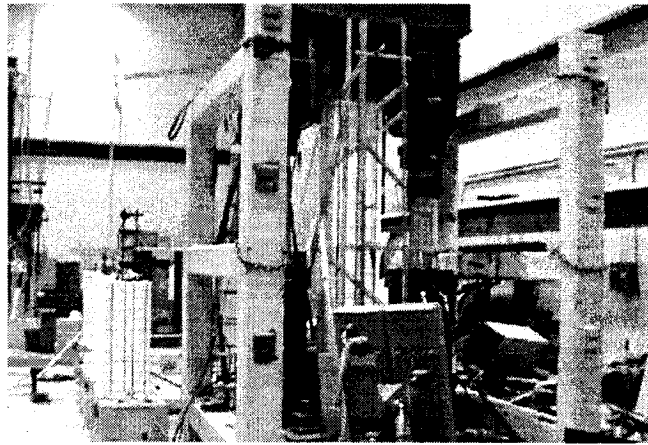
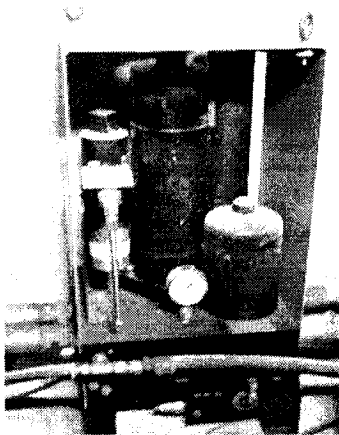
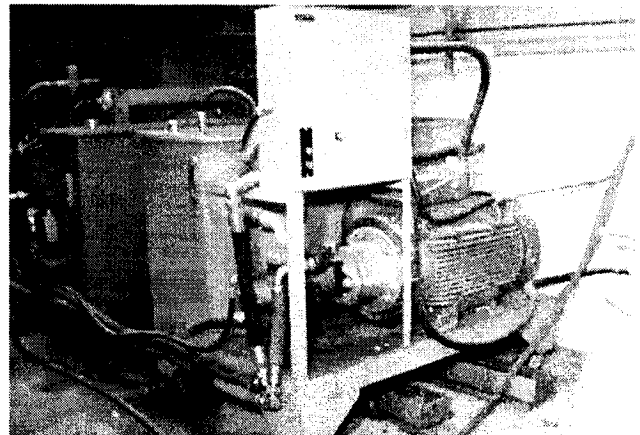


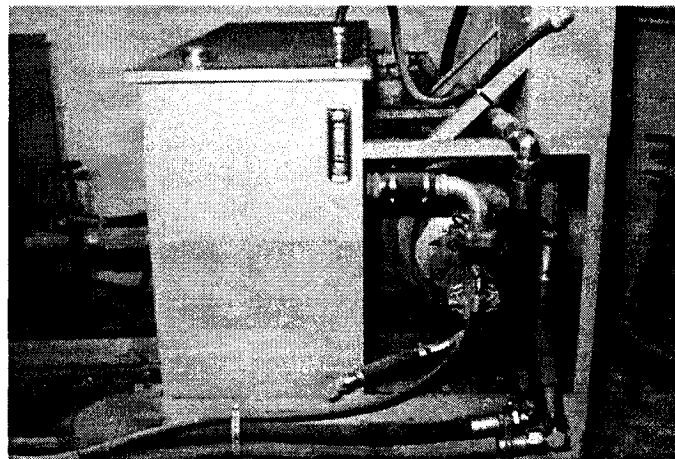
Figure 4.22: A Test Column Ready for Testing



(a) Hydraulic Manifold



(b) Hydraulic Pump



(c) Hydraulic Power System

Figure 4.23: Hydraulic Pump and Supply for the MTS System

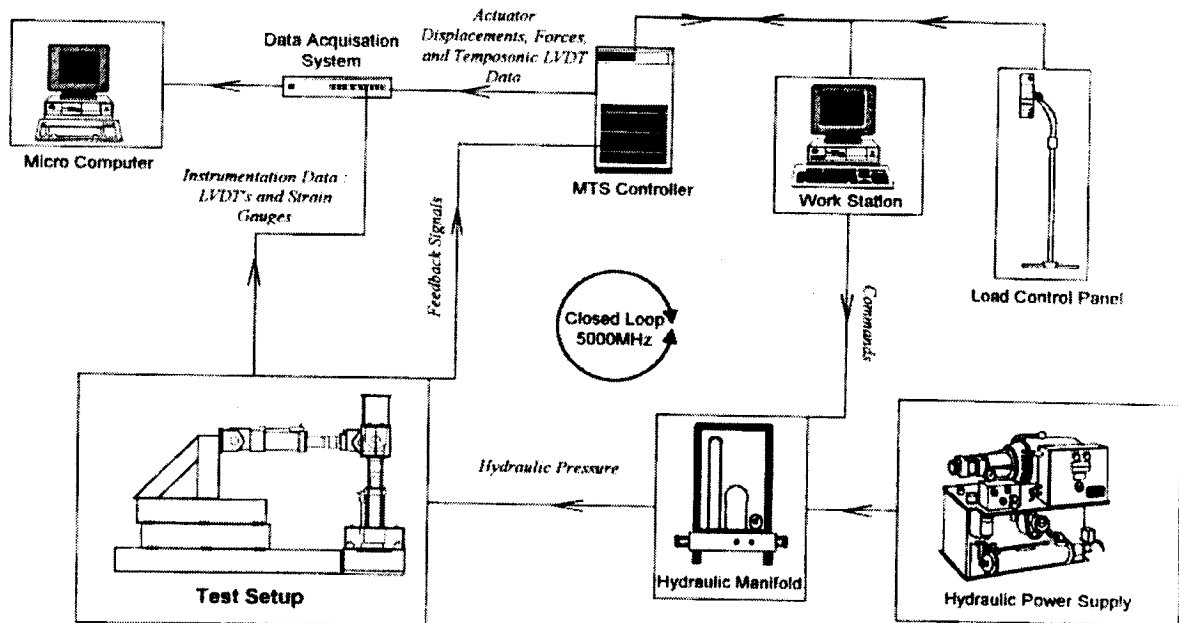


Figure 4.24: Schematic Diagram for Test Network

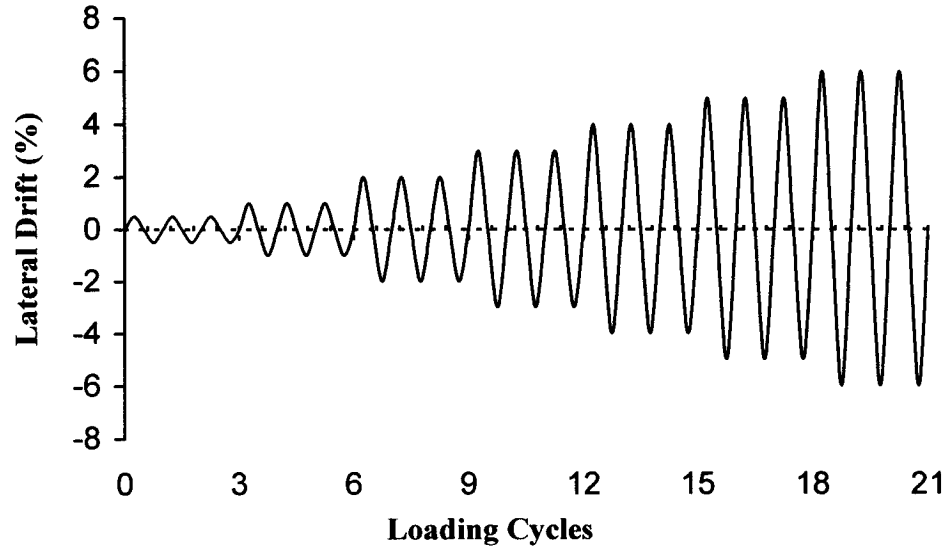
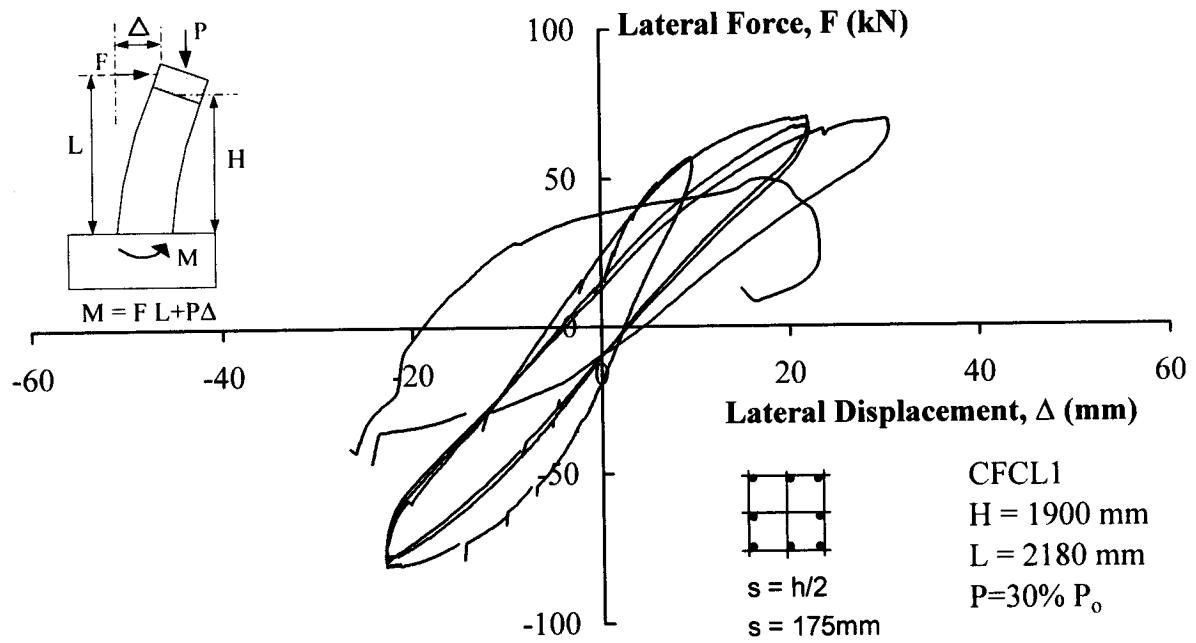
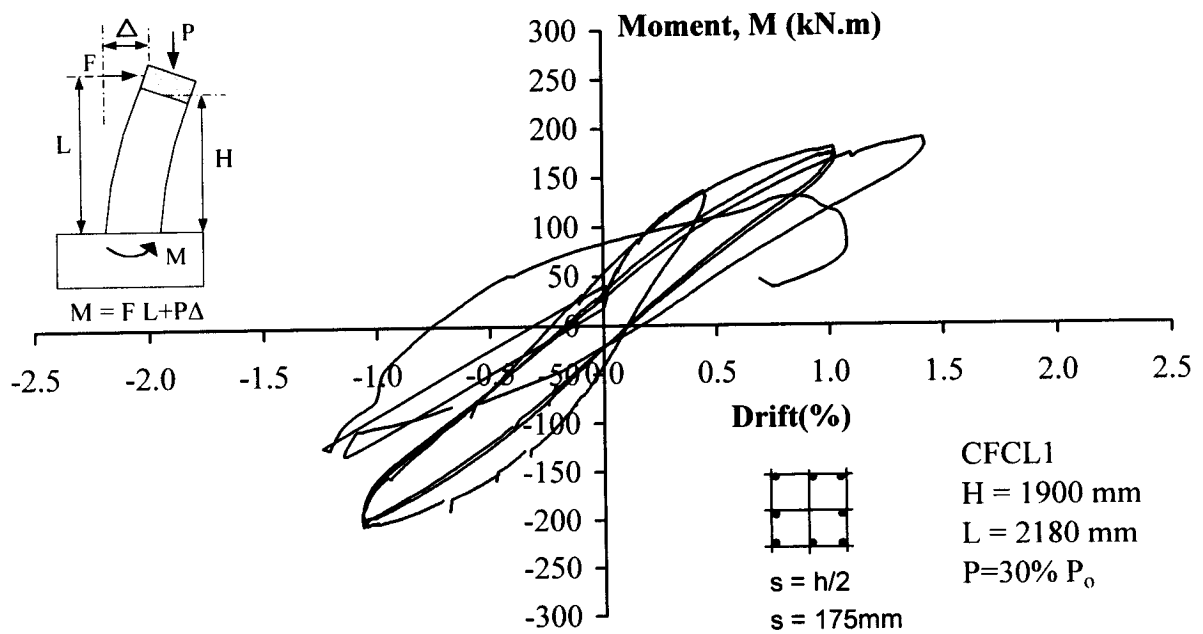


Figure 4.25: The Deformation History for Cyclic Lateral Loading

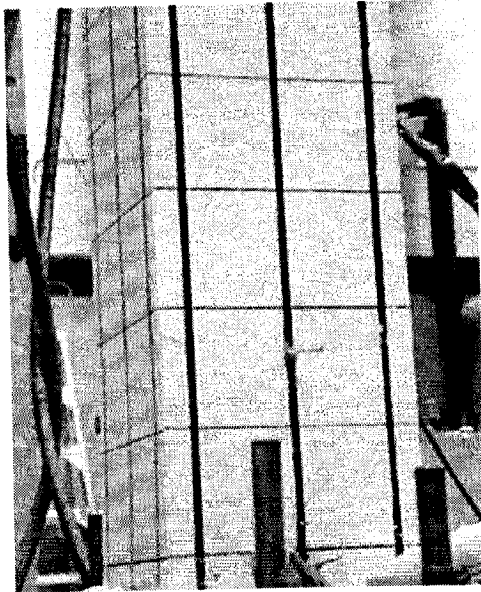


(a) Lateral Force-Lateral Displacement Relationship

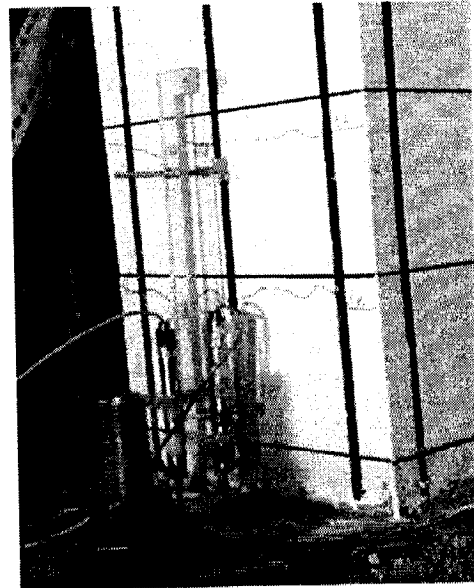


(b) Moment-Drift Relationship

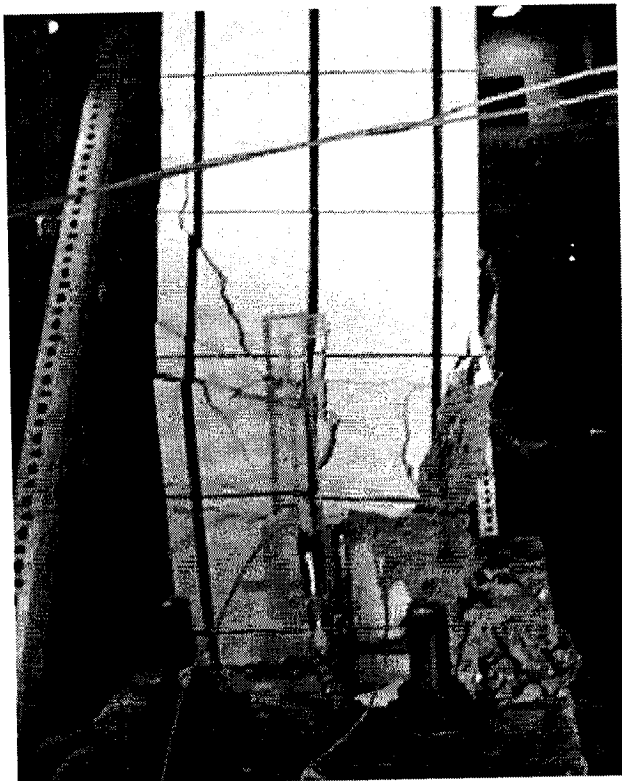
Figure 4.26: Hysteretic Behavior for Column CFCL1



(a) Before Loading



(b) At 1% Drift



( c ) Column Failure

Figure 4.27: Observed Damage in Column CFCL1 at Selected Stages of Loading

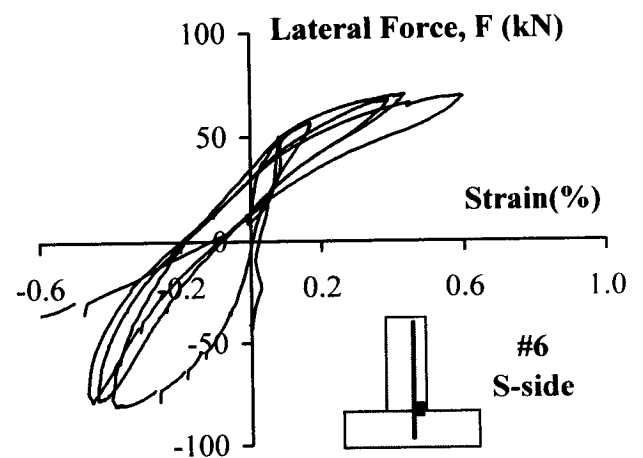
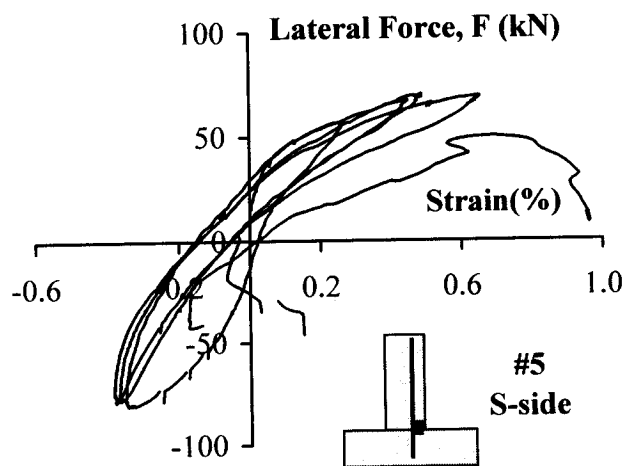
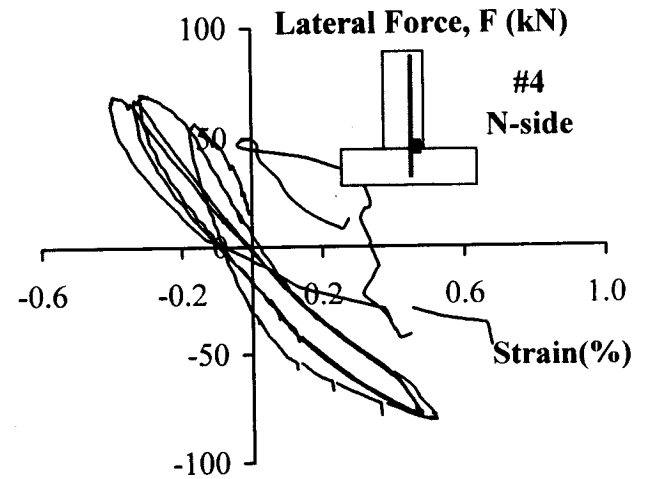
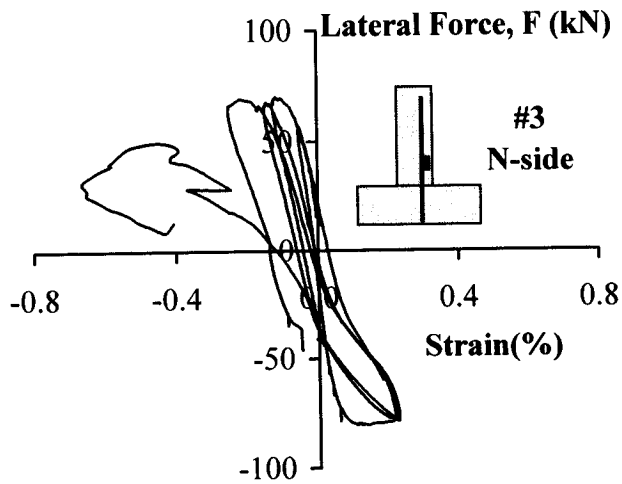
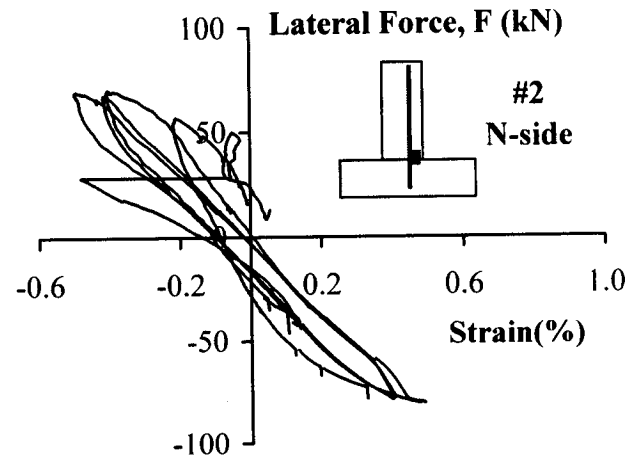
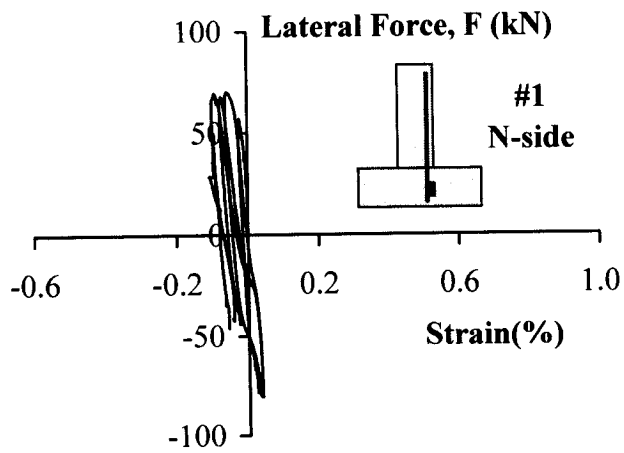


Figure 4.28: Bar Strain Readings in Column CFCL1

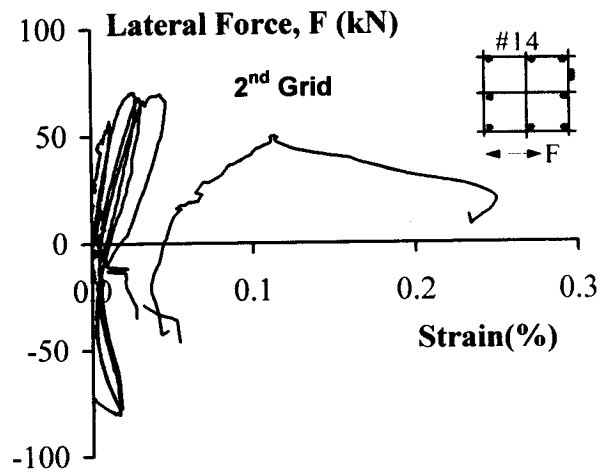
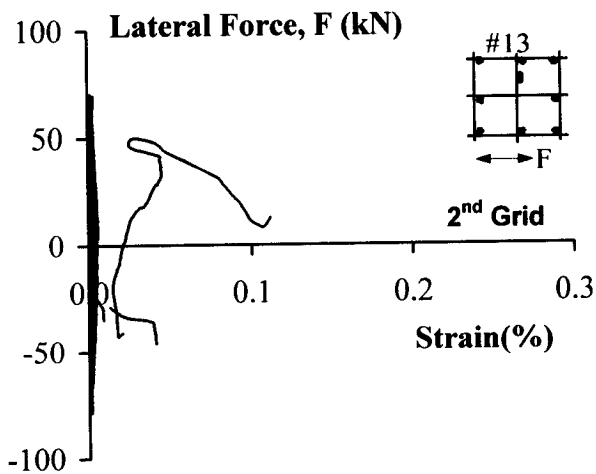
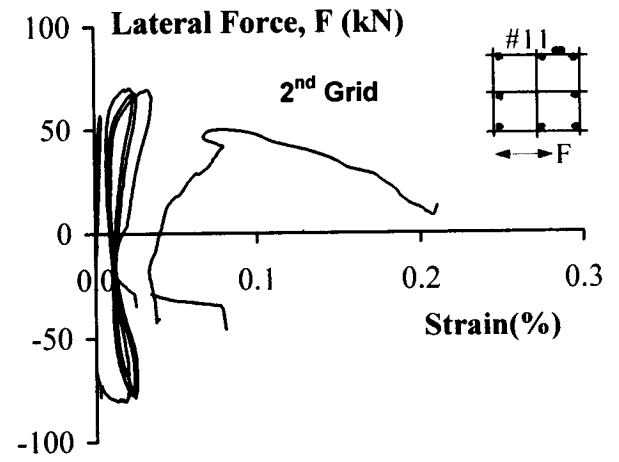
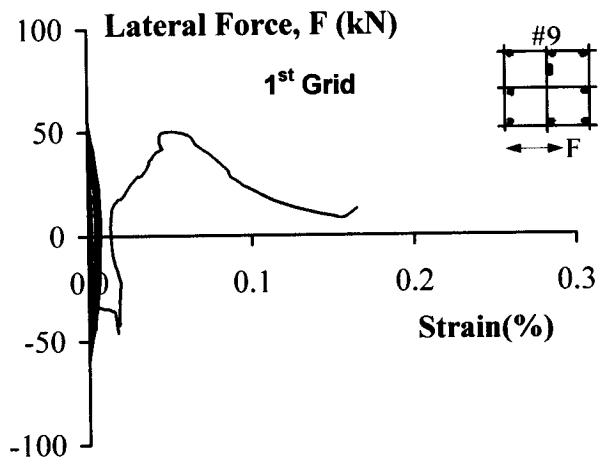
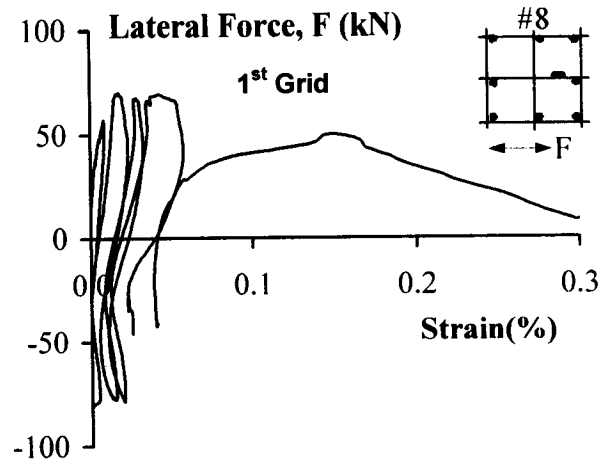
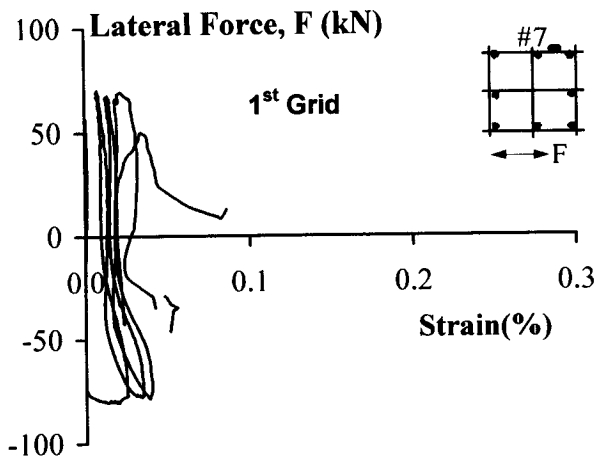
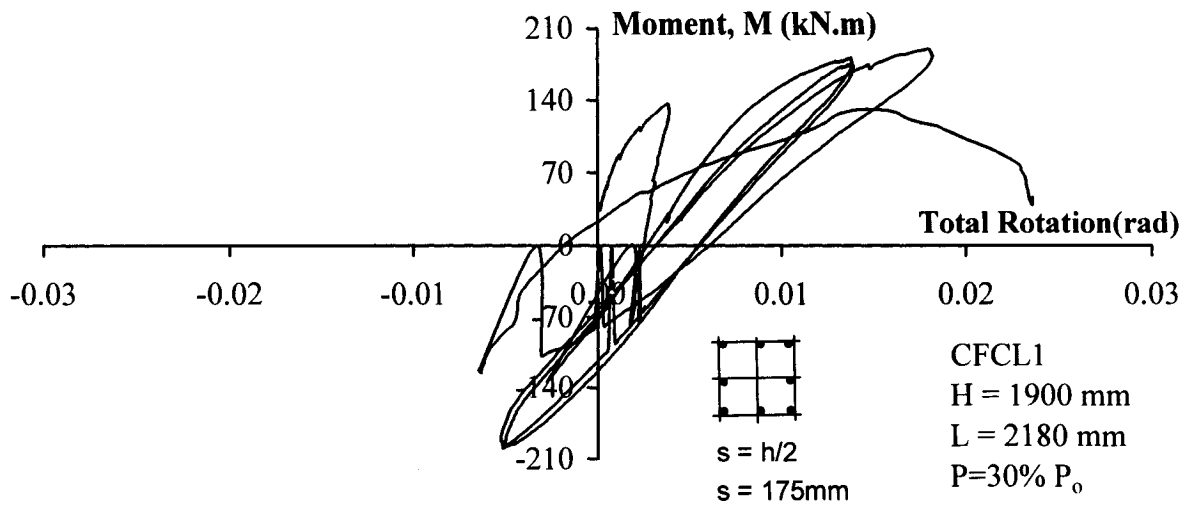
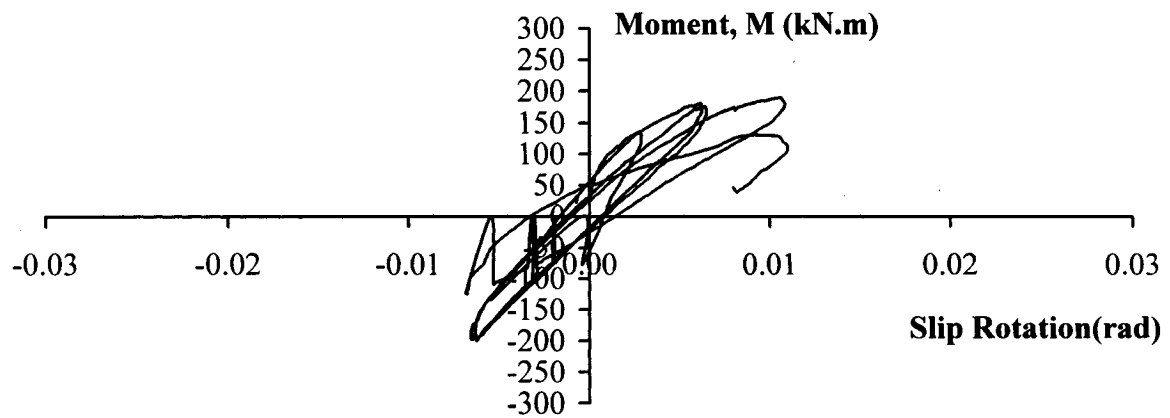


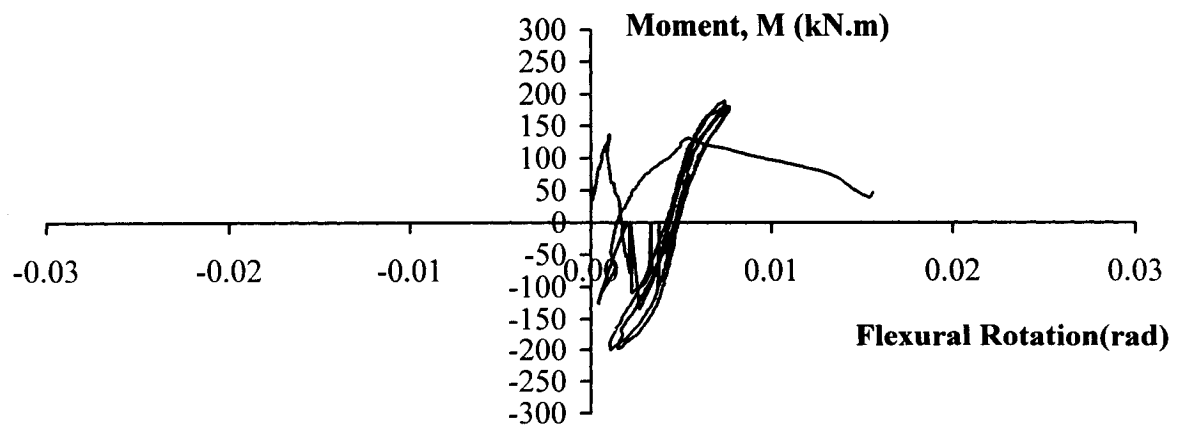
Figure 4.29: Grid Strain Readings in Column CFCL1



(a) Moment-Total Rotation Relationship

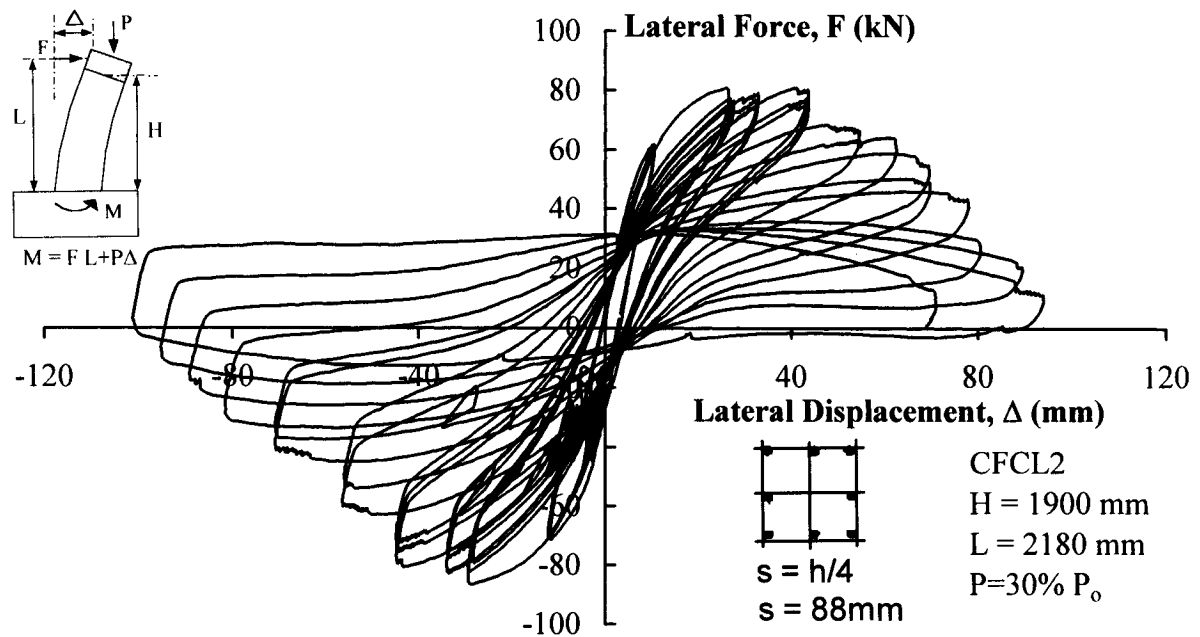


(b) Moment-Slip Rotation Relationship

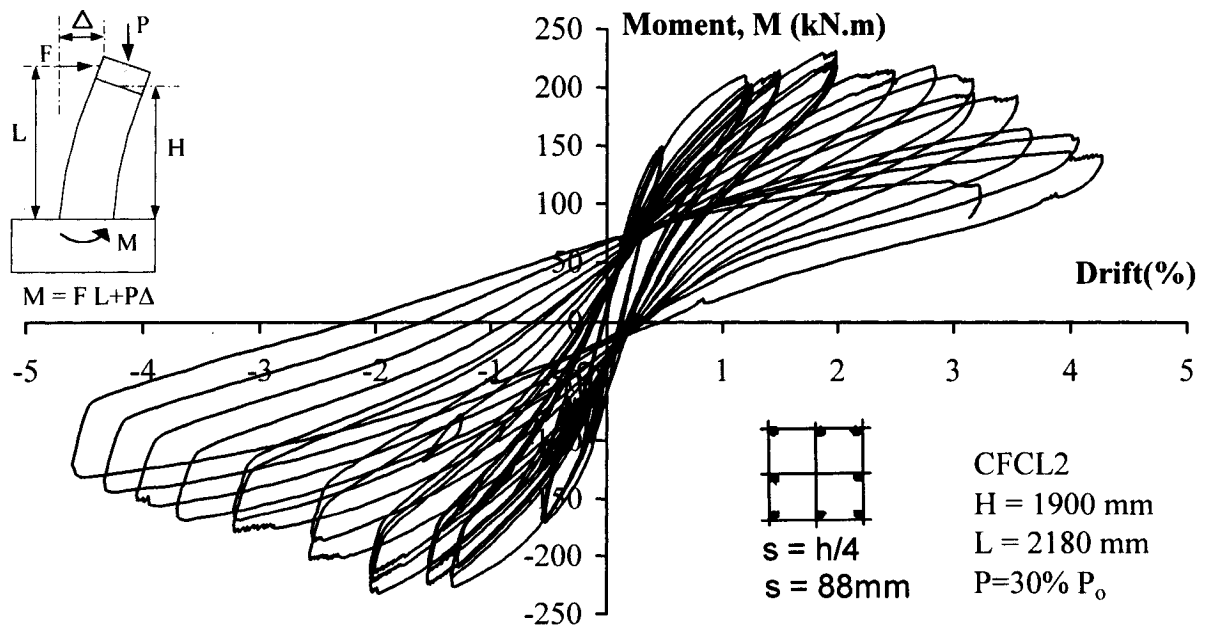


(c) Moment-Flexural Rotation Relationship

Figure 4.30: Hysteretic Moment-Rotation Relationships for Column CFCL1



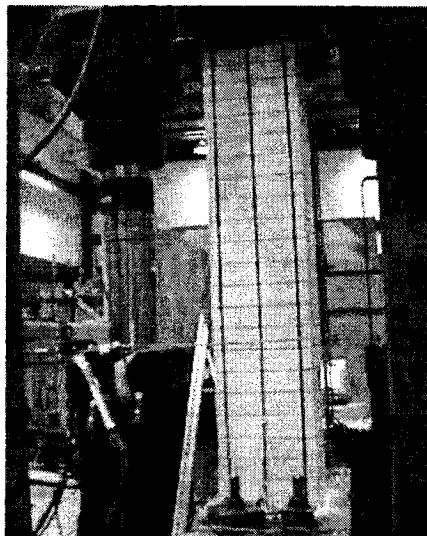
(a) Lateral Force-Lateral Displacement Relationship



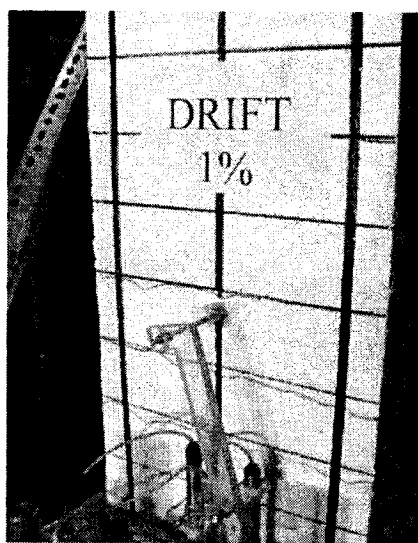
(b) Moment-Drift Relationship

Figure 4.31: Hysteretic Behavior for Column CFCL2

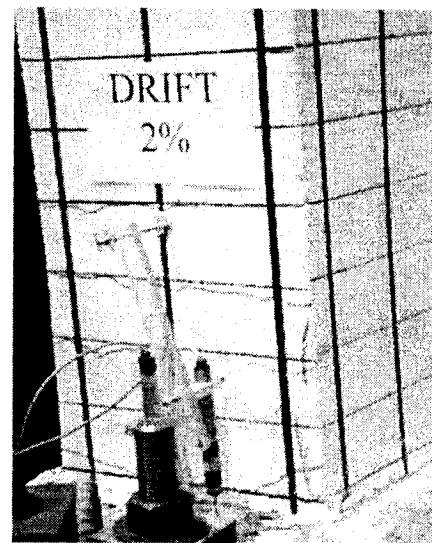




(a) Before Loading



(b) At 1% Drift



(c) At 2% Drift



(d) At 3% Drift



(e) At 4% Drift



(f) Column Failure

Figure 4.32 : Observed Damage in Column CFCL2 at Selected Stages of Loading

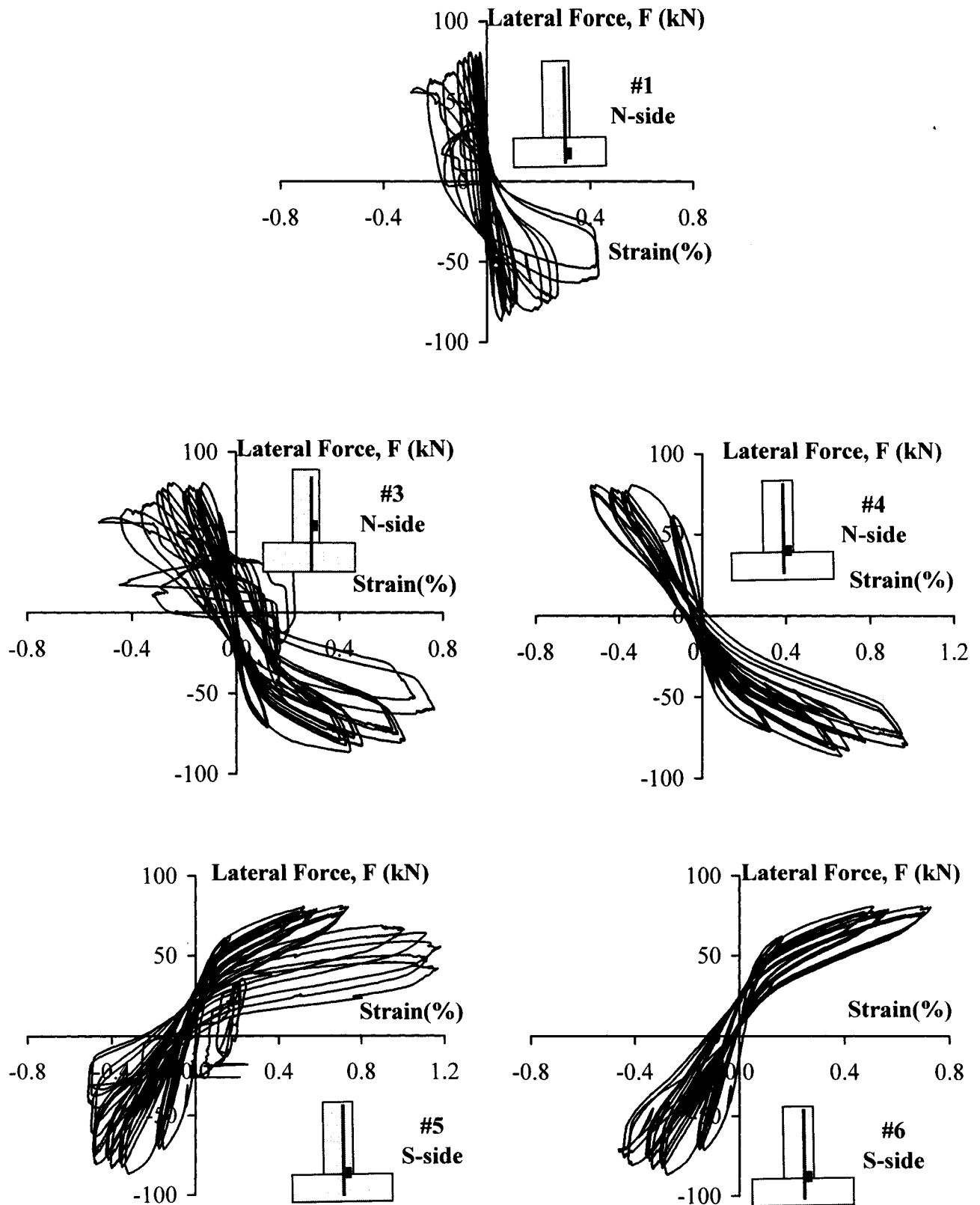


Figure 4.33: Bar Strain Readings in Column CFCL2

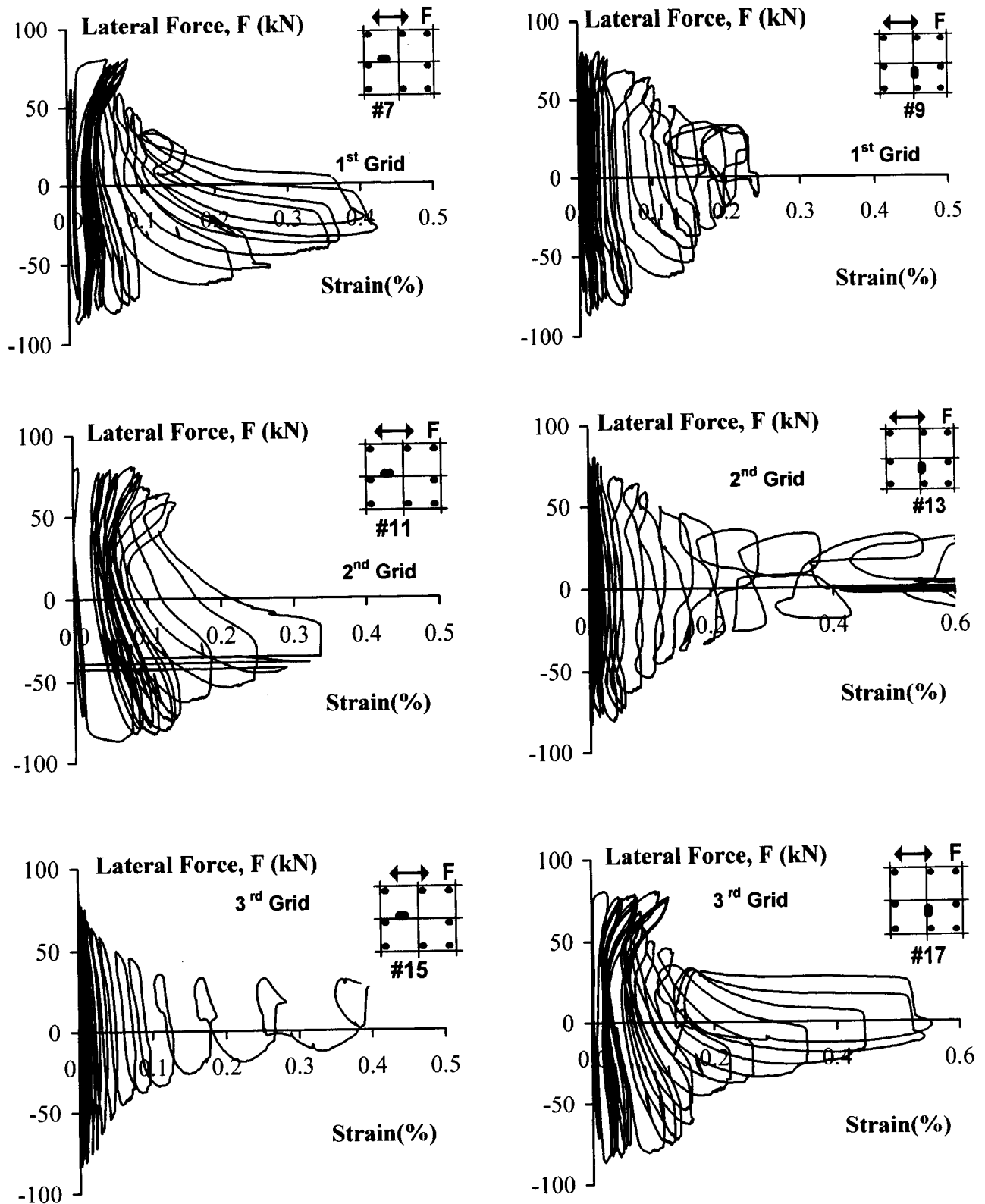
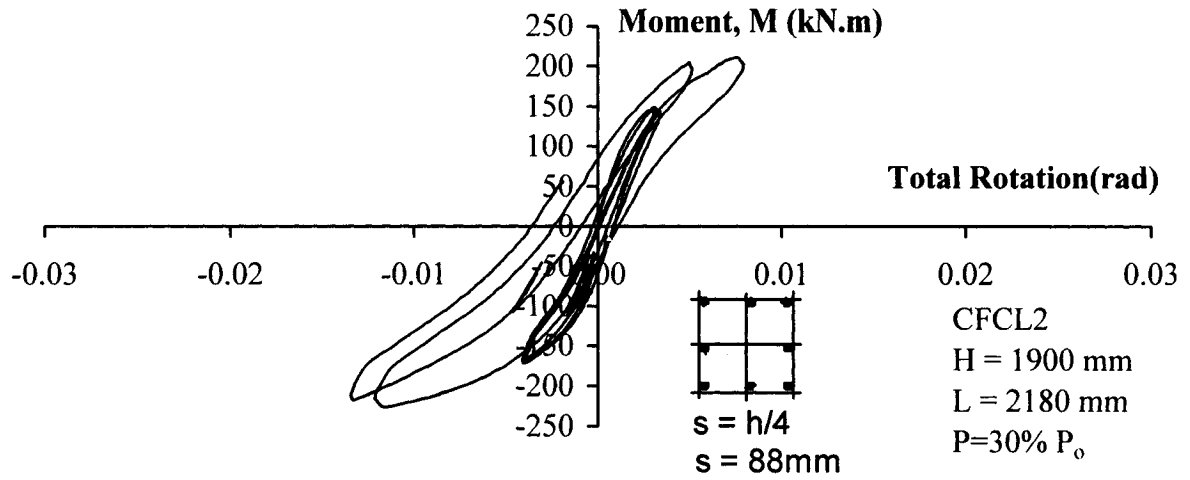
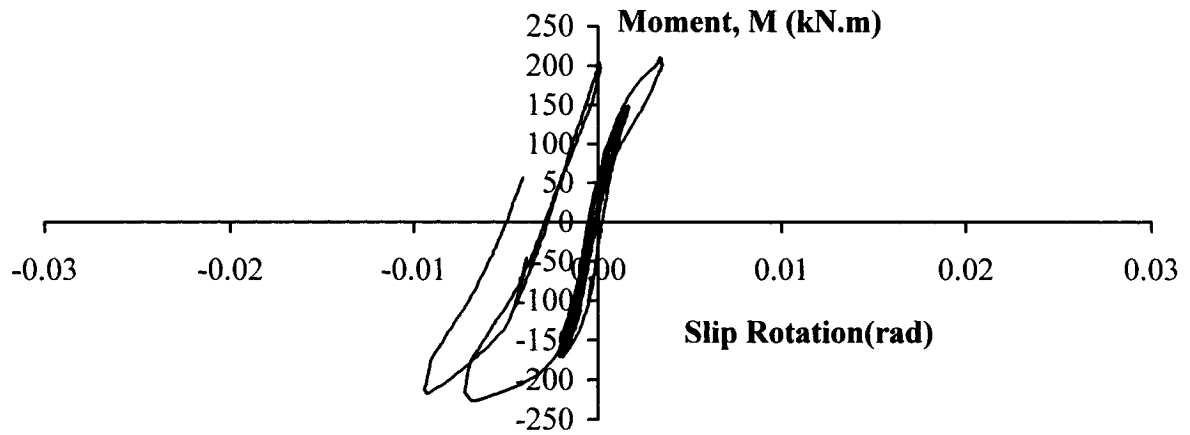


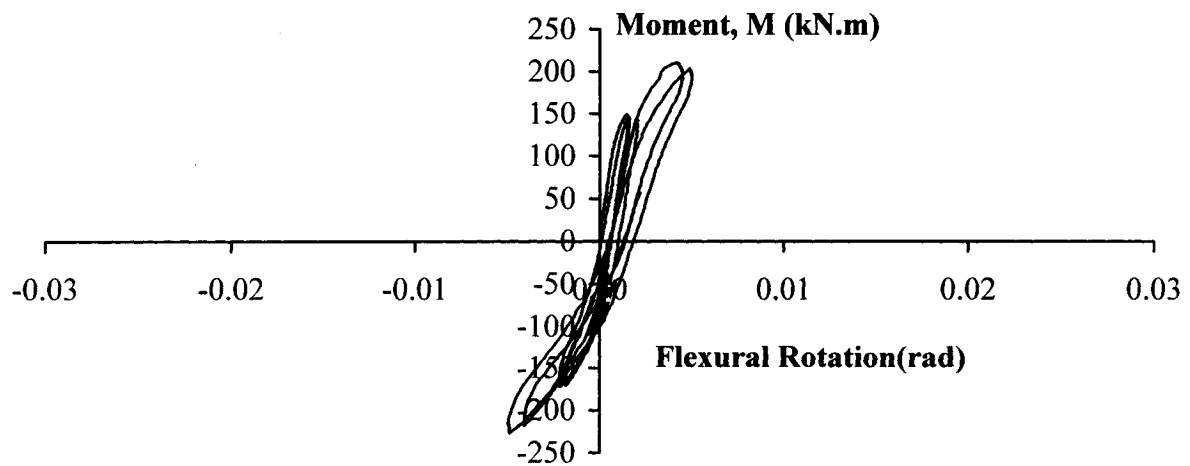
Figure 4.34: Grid Strain Readings in Column CFCL2



(a) Moment-Total Rotation Relationship

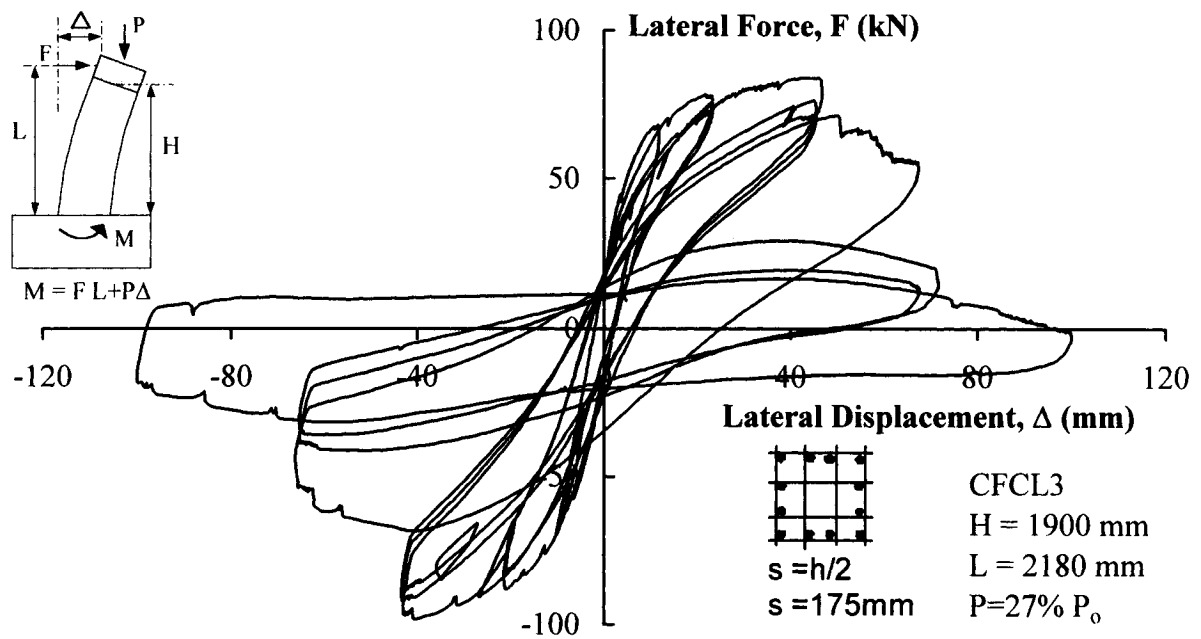


(b) Moment-Slip Rotation Relationship

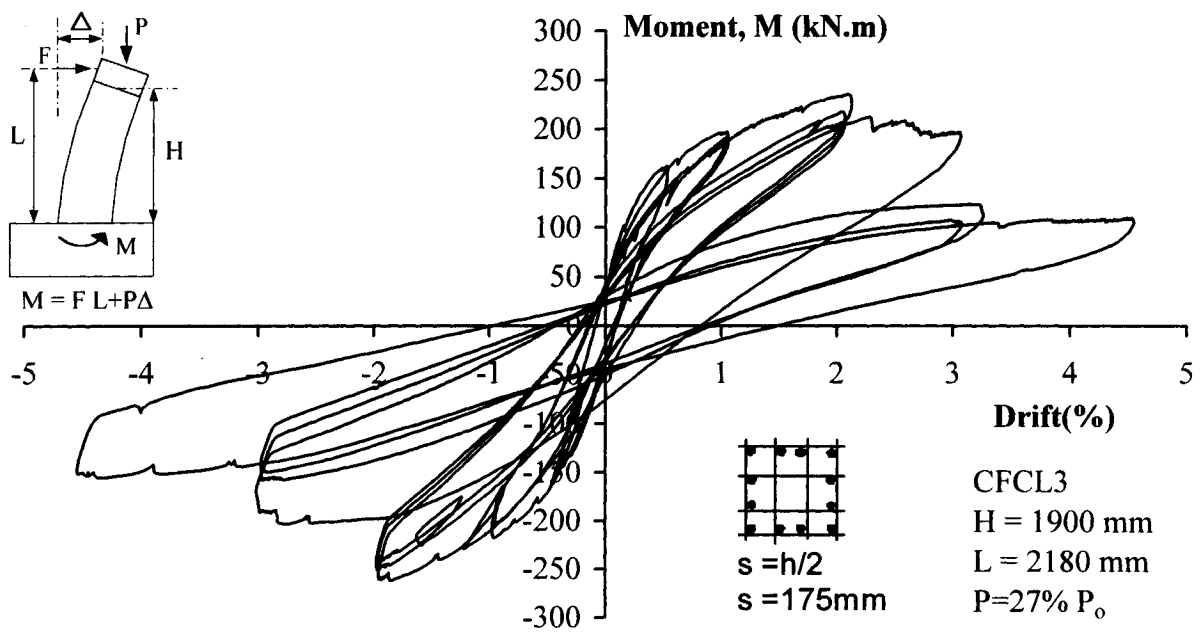


(c) Moment-Flexural Rotation Relationship

Figure 4.35: Hysteretic Moment-Rotation Relationships for Column CFCL2

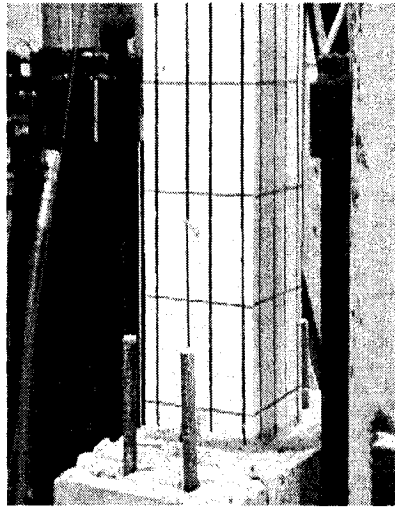


(a) Lateral Force-Lateral Displacement Relationship



(b) Moment-Drift Relationship

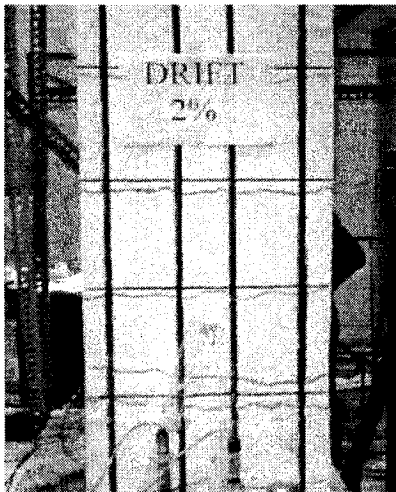
Figure 4.36: Hysteretic Behavior for Column CFCL3



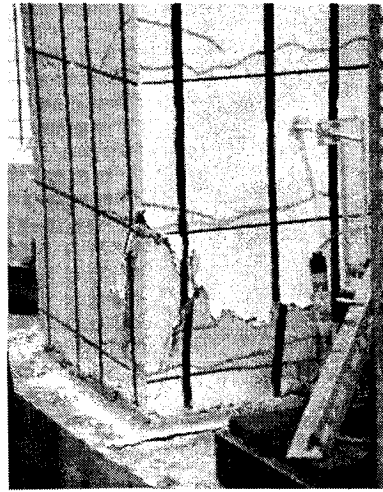
(a) Before Loading



(b) At 1% Drift



(c) At 2% Drift



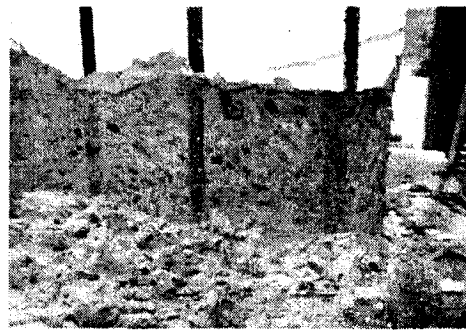
(d) At 3% Drift



(e) At 4% Drift



(f) Column Failure



(g) Buckling of Bars in Compression

Figure 4.37: Observed Damage in Column CFCL3 at Selected Stages of Loading

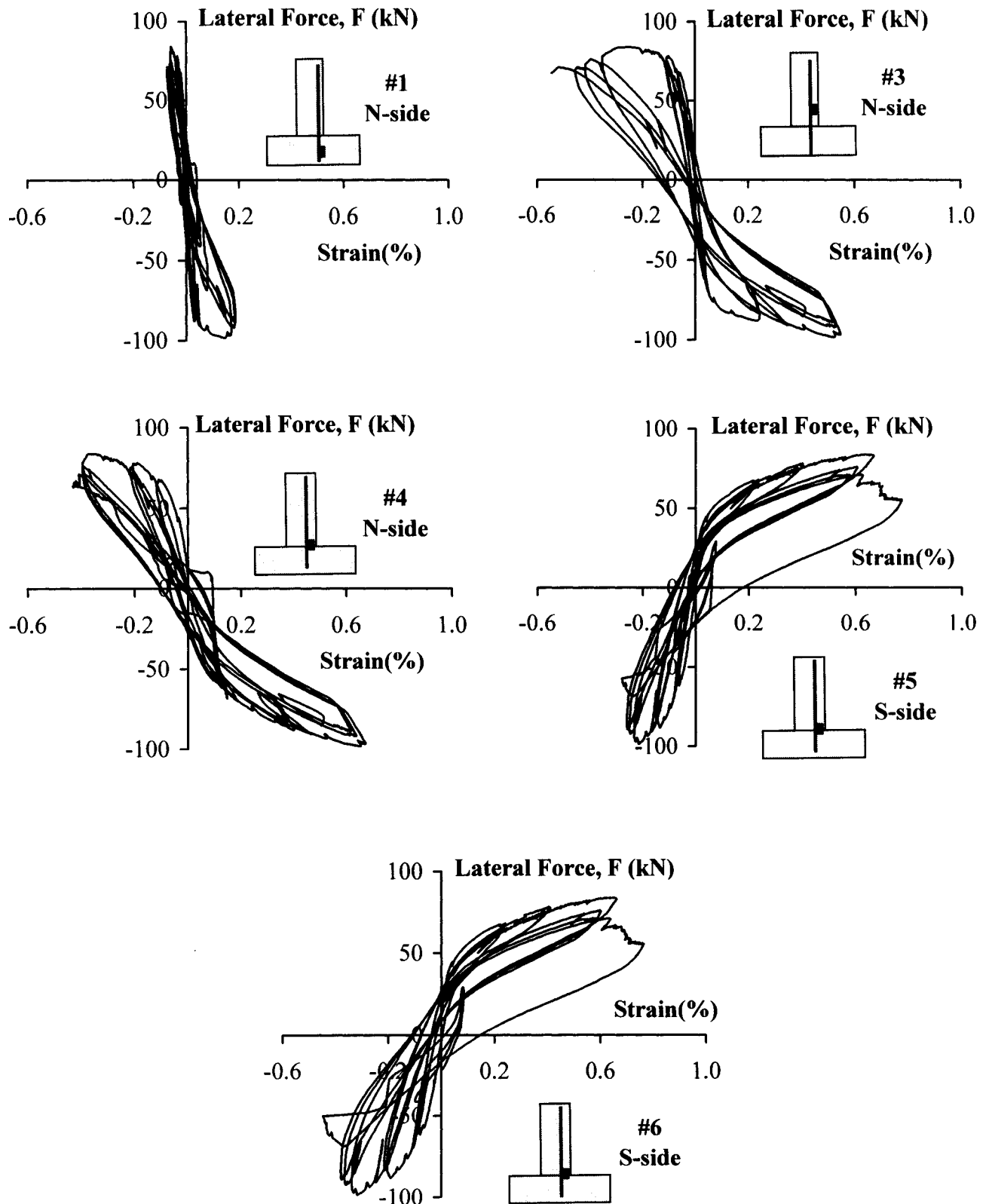


Figure 4.38: Bar Strain Readings in Column CFCL3

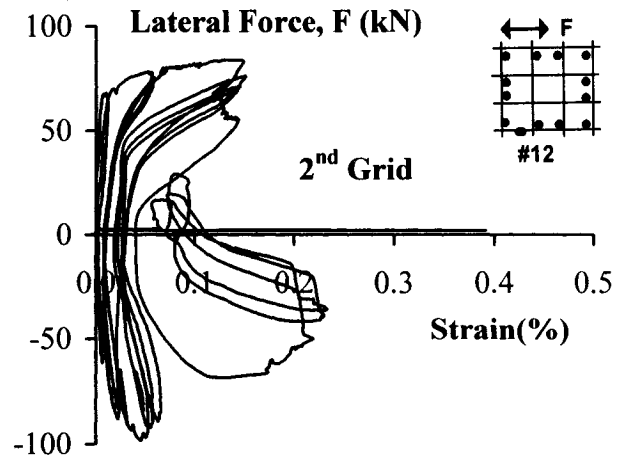
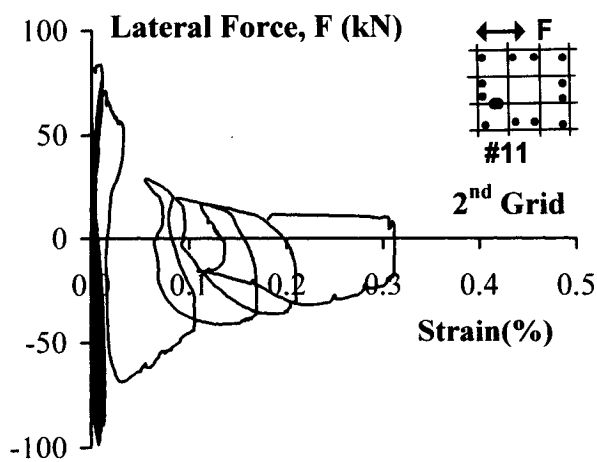
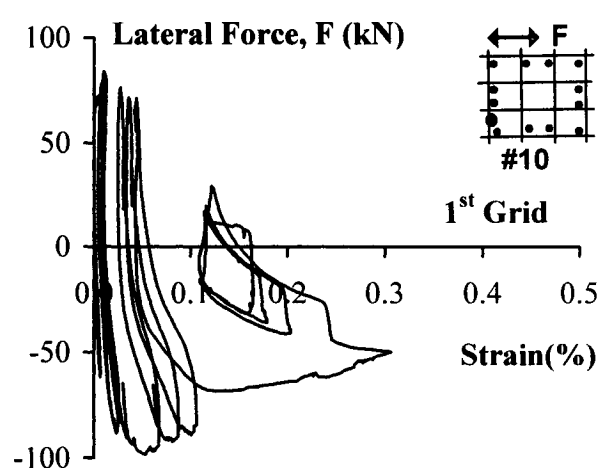
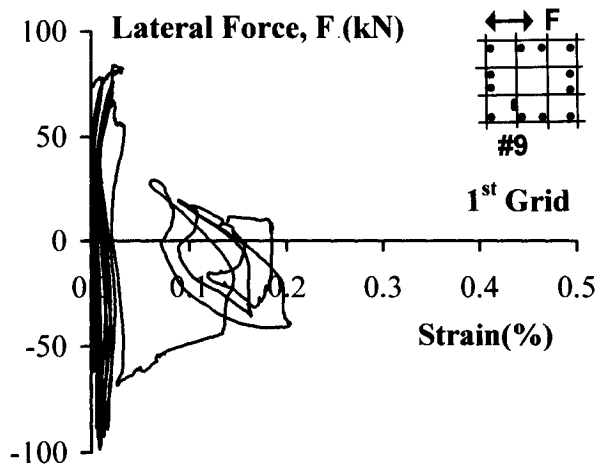
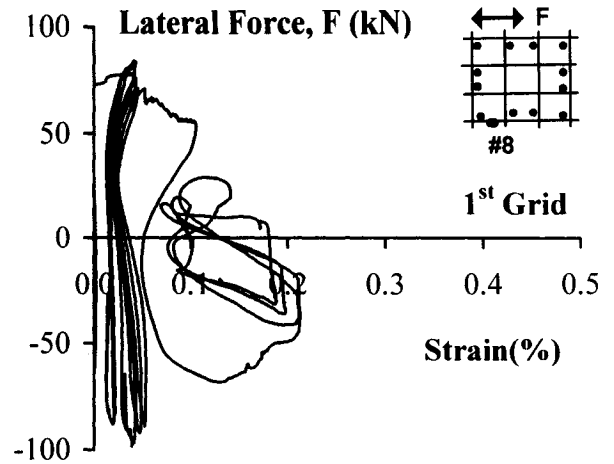
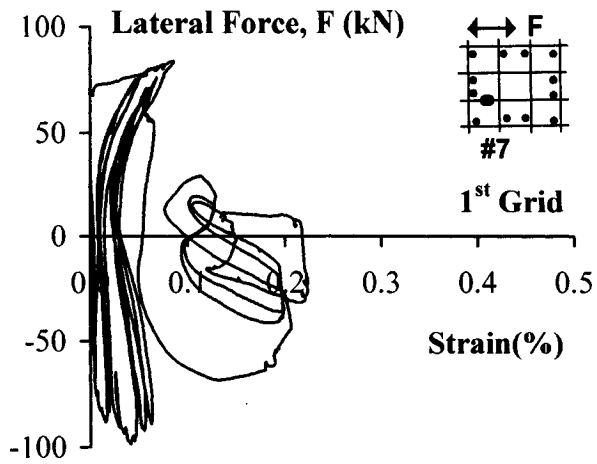
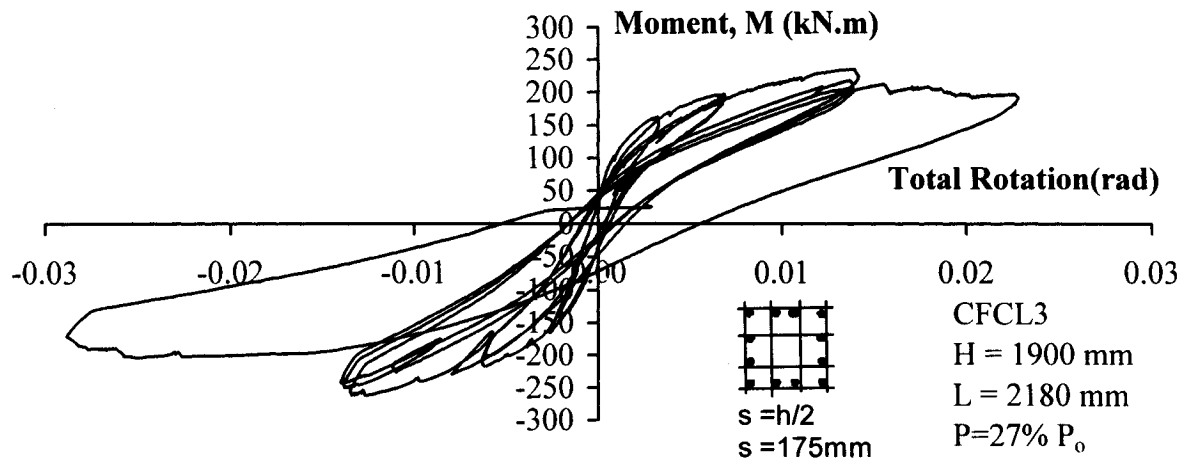
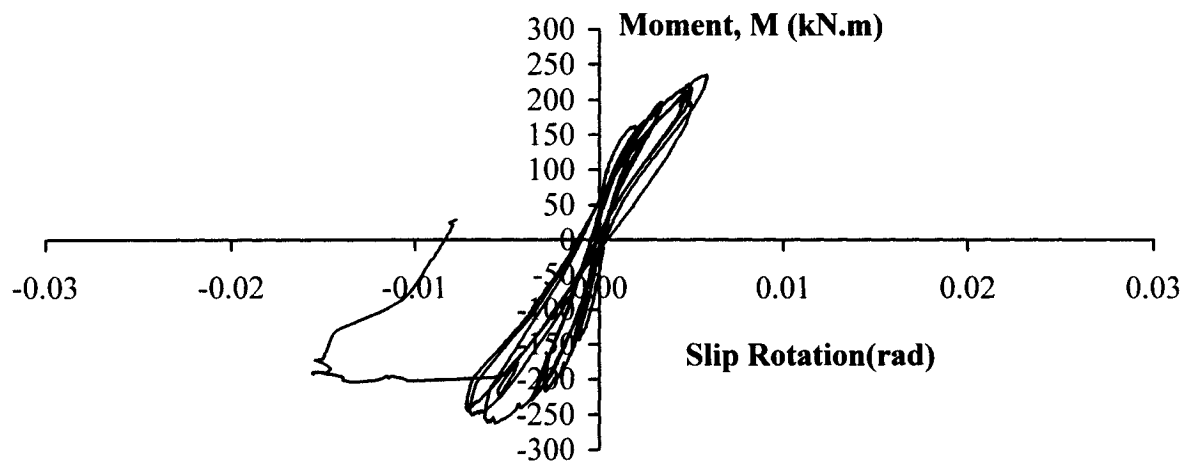


Figure 4.39: Grid Strain Readings in Column CFCL3

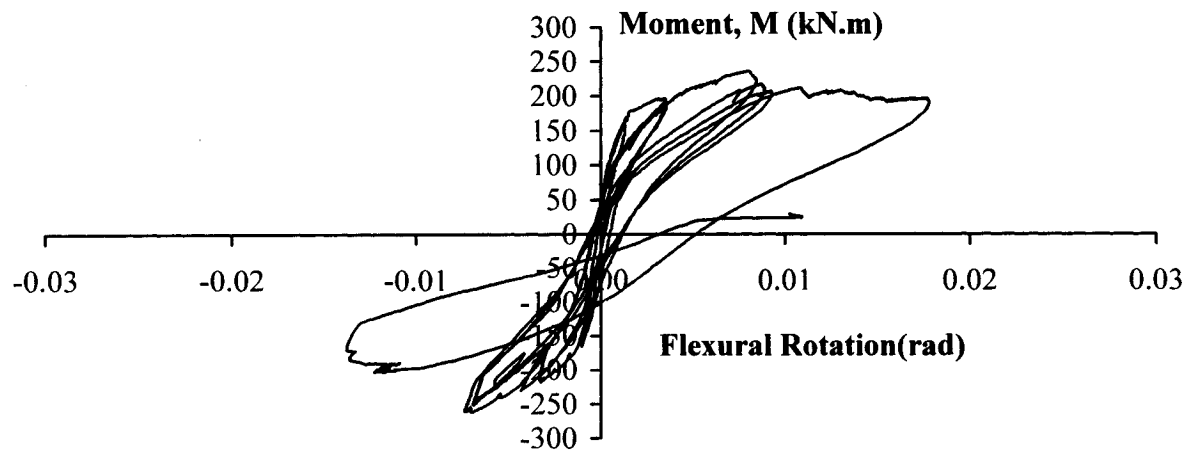




(a) Moment-Total Rotation Relationship

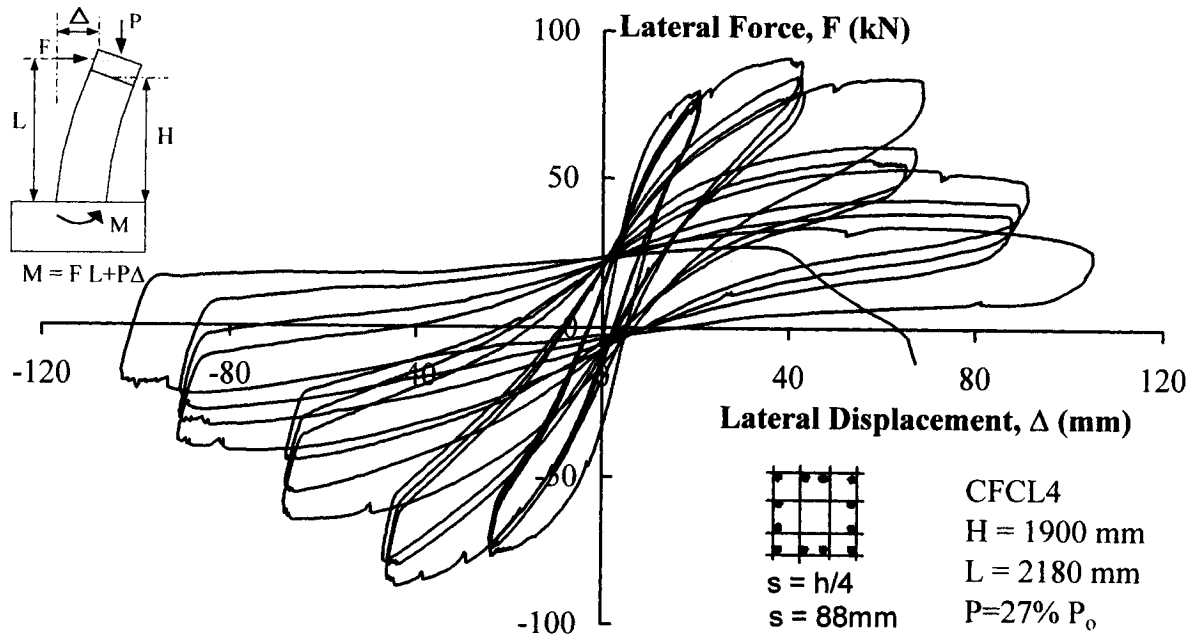


(b) Moment-Slip Rotation Relationship

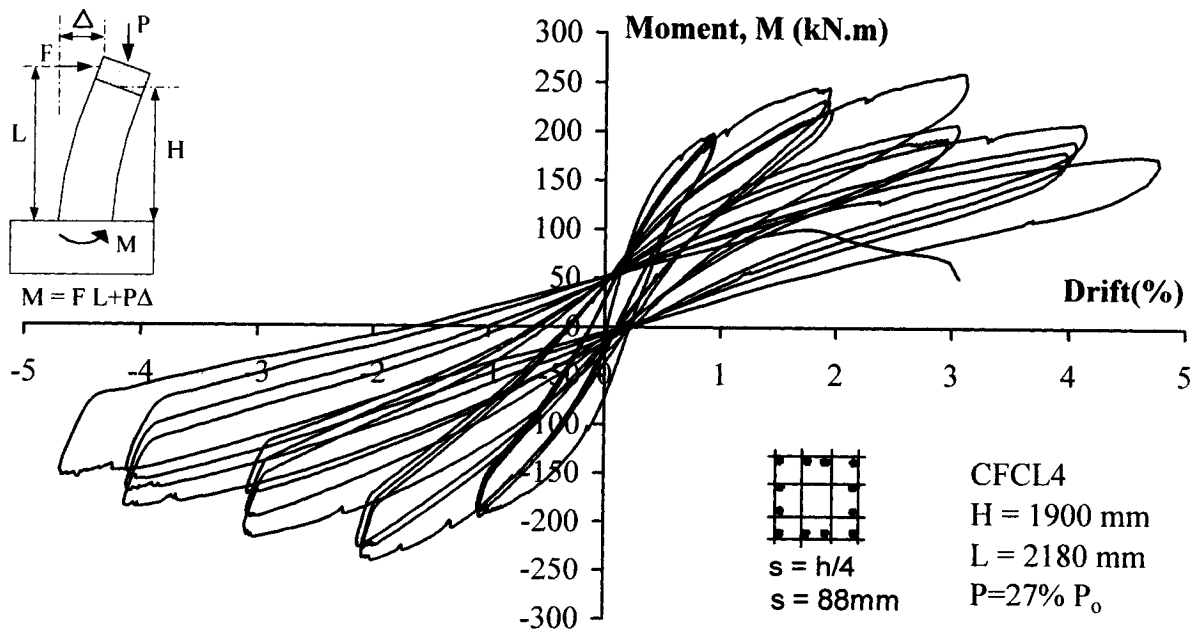


(c) Moment-Flexural Rotation Relationship

Figure 4.40: Hysteretic Moment-Rotation Relationships for Column CFCL3

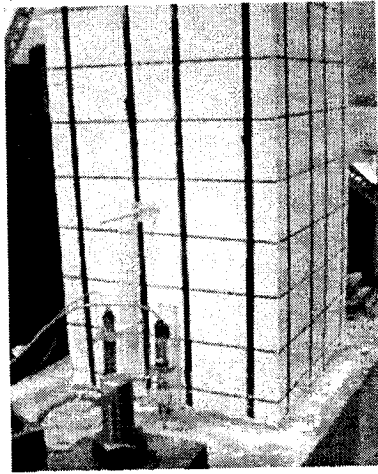


(a) Lateral Force-Lateral Displacement Relationship



(b) Moment-Drift Relationship

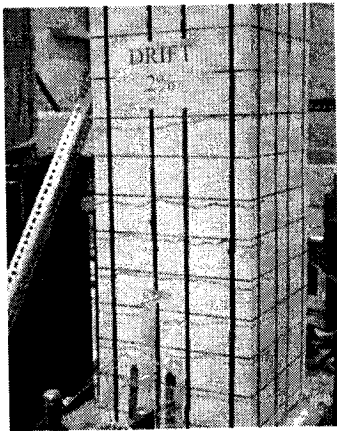
Figure 4.41: Hysteretic Behavior for Column CFCL4



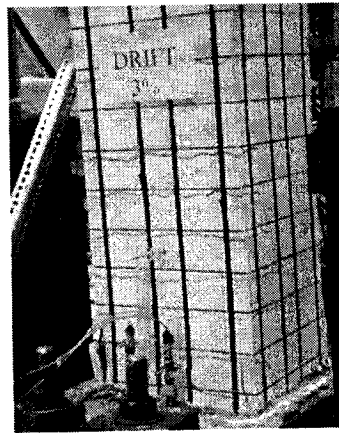
(a) Before Loading



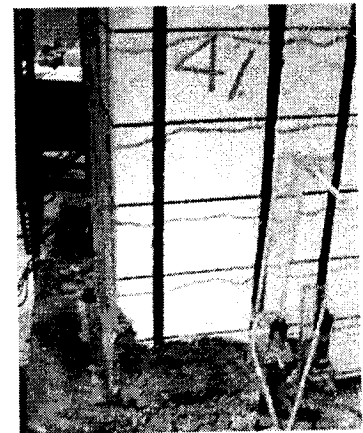
(b) At 1% Drift



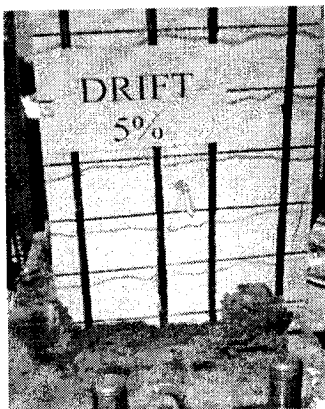
(c) At 2% Drift



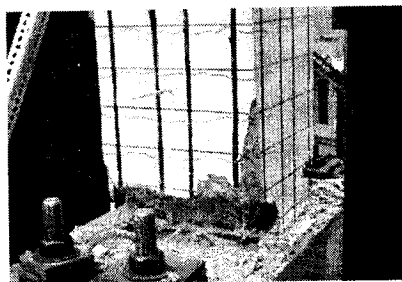
(d) At 3% Drift



(e) At 4% Drift



(f) At 5% Drift



(g) Column Failure



(h) Buckling of Bars  
in Compression

Figure 4.42: Observed Damage in Column CFCL4 at Selected Stages of Loading

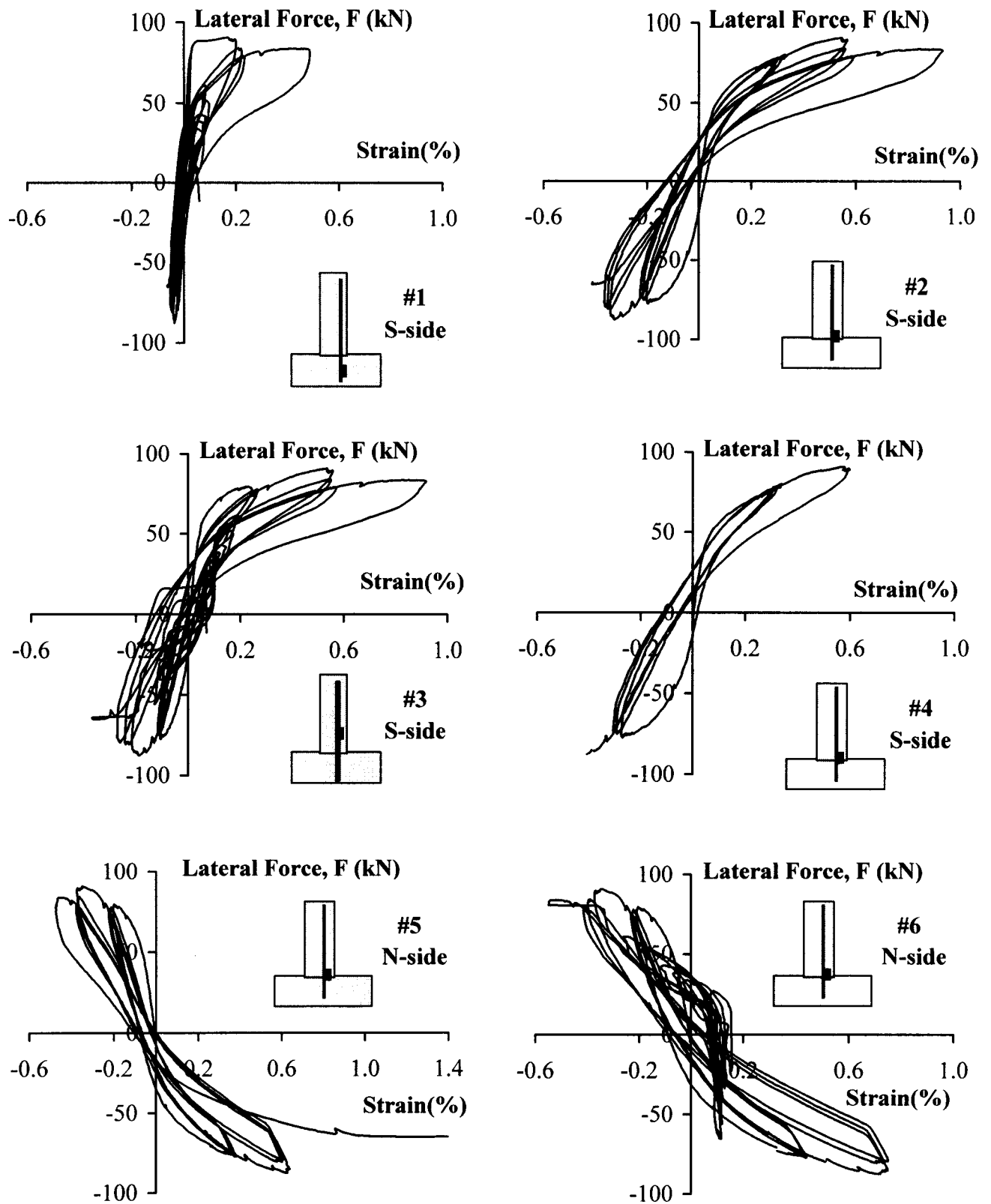


Figure 4.43: Bar Strain Readings in Column CFCL4

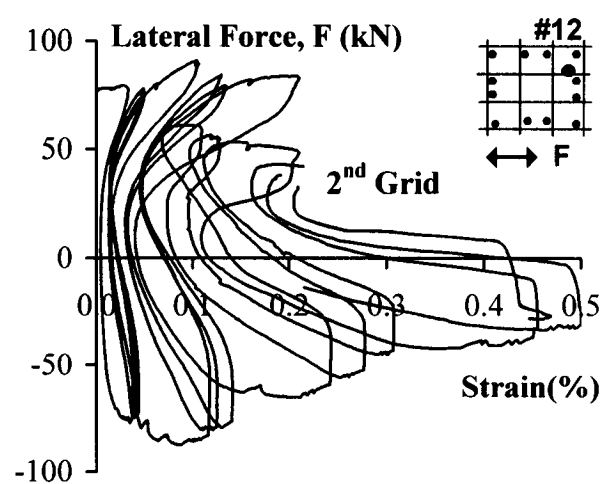
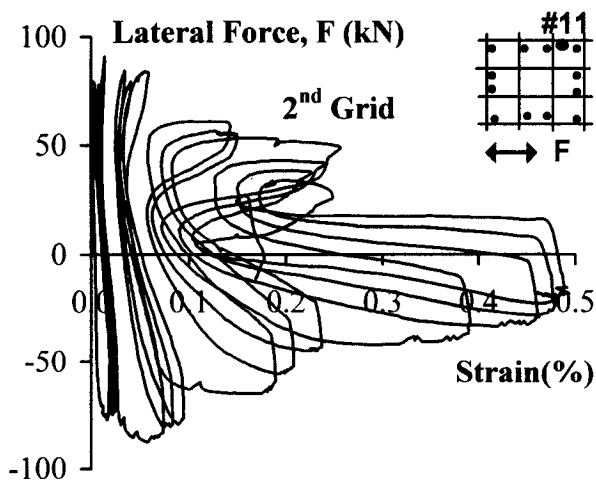
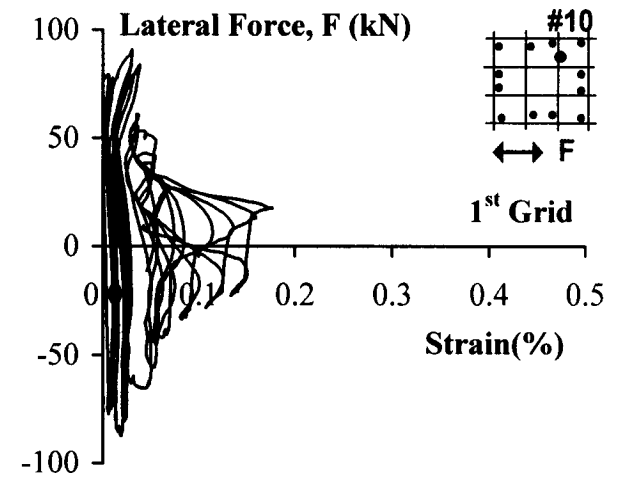
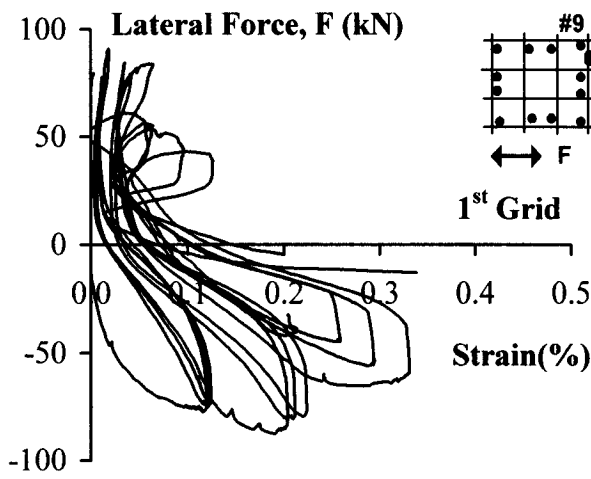
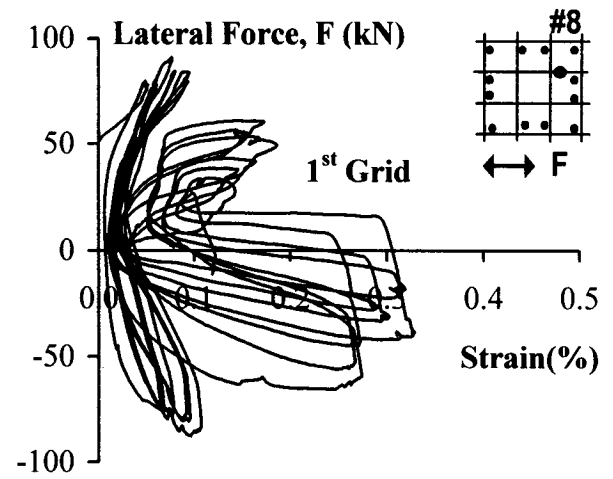
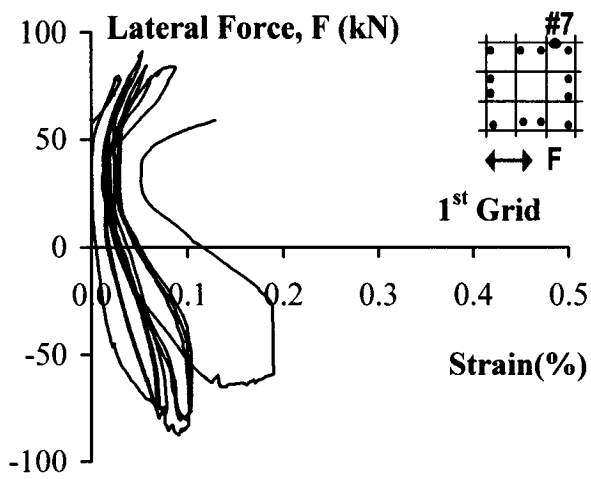


Figure 4.44: Grid Strain Readings in Column CFCL4

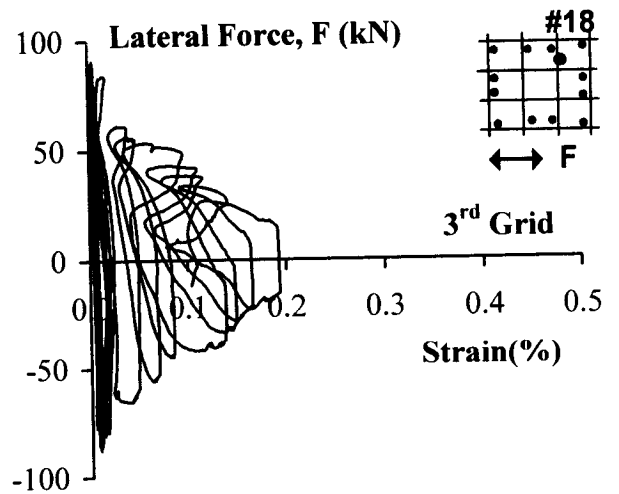
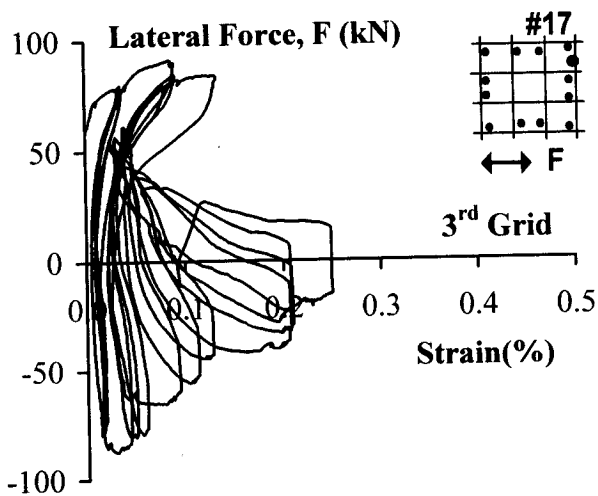
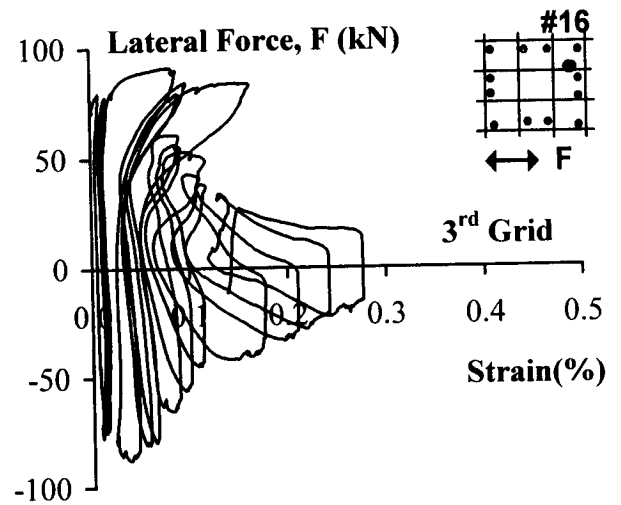
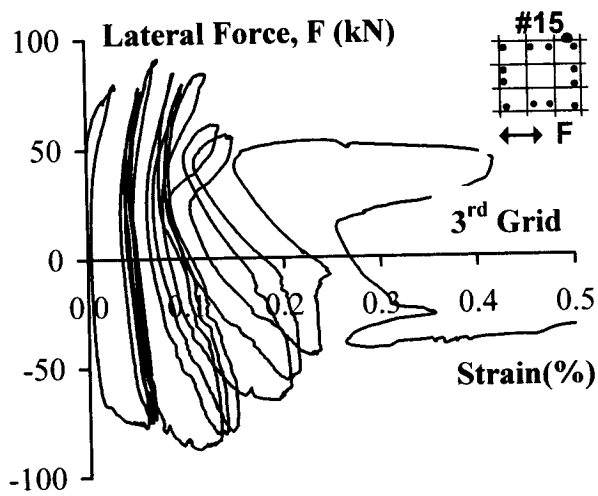
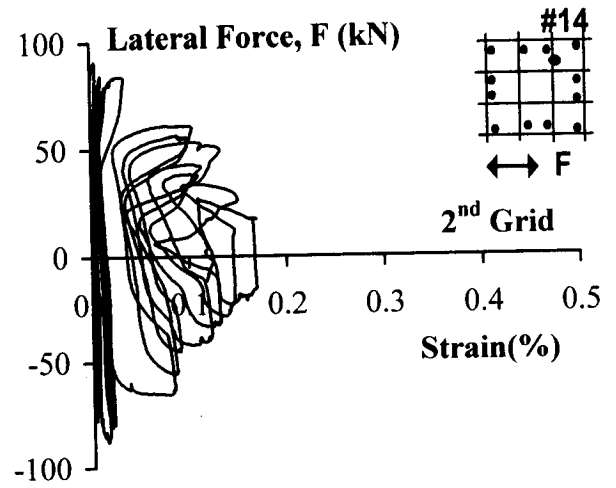
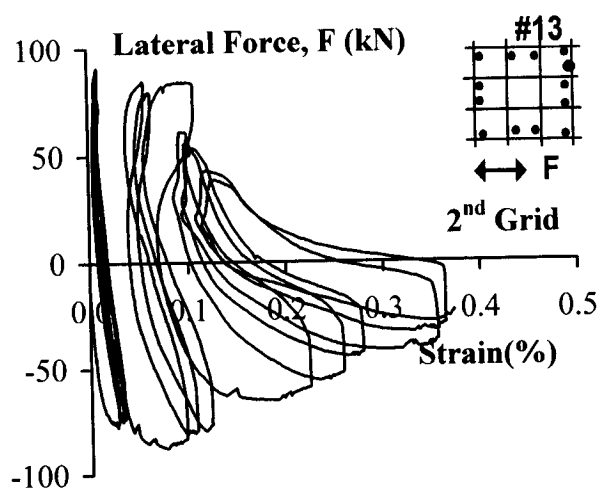
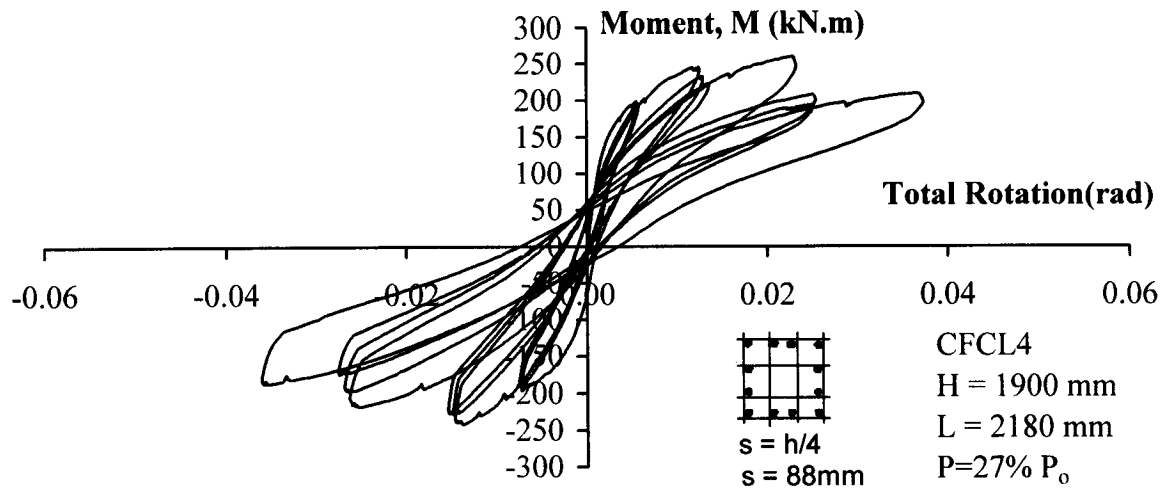
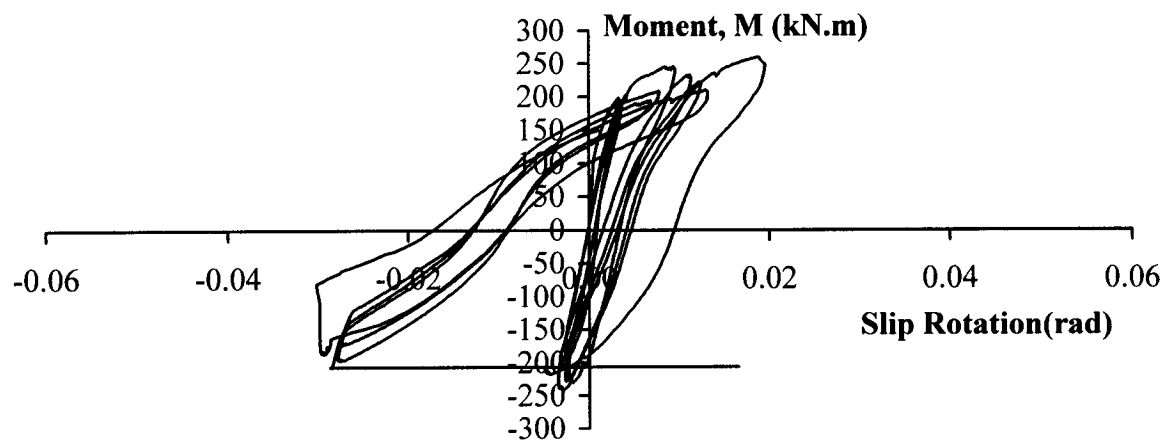


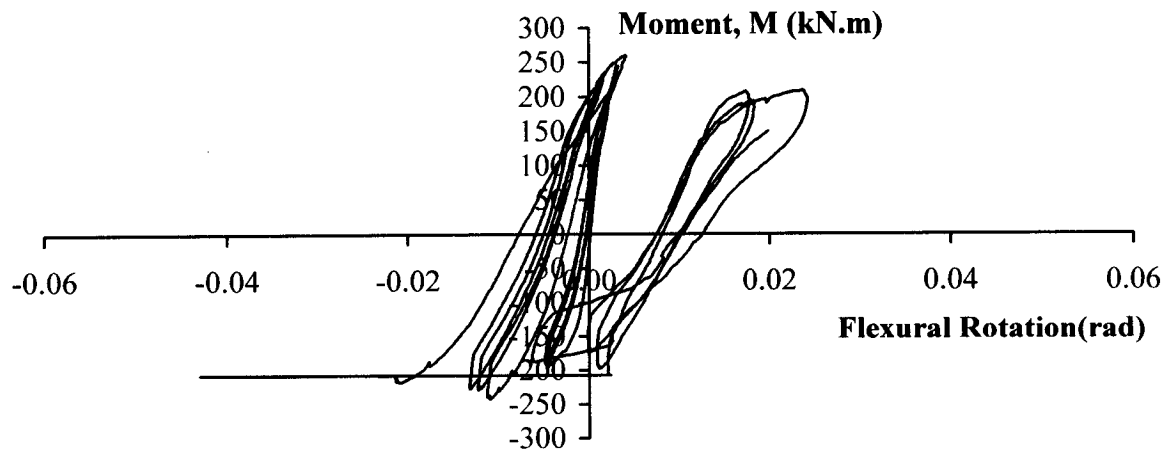
Figure 4.44 (Cont.) : Grid Strain Readings in Column CFCL4



(a) Moment-Total Rotation Relationship

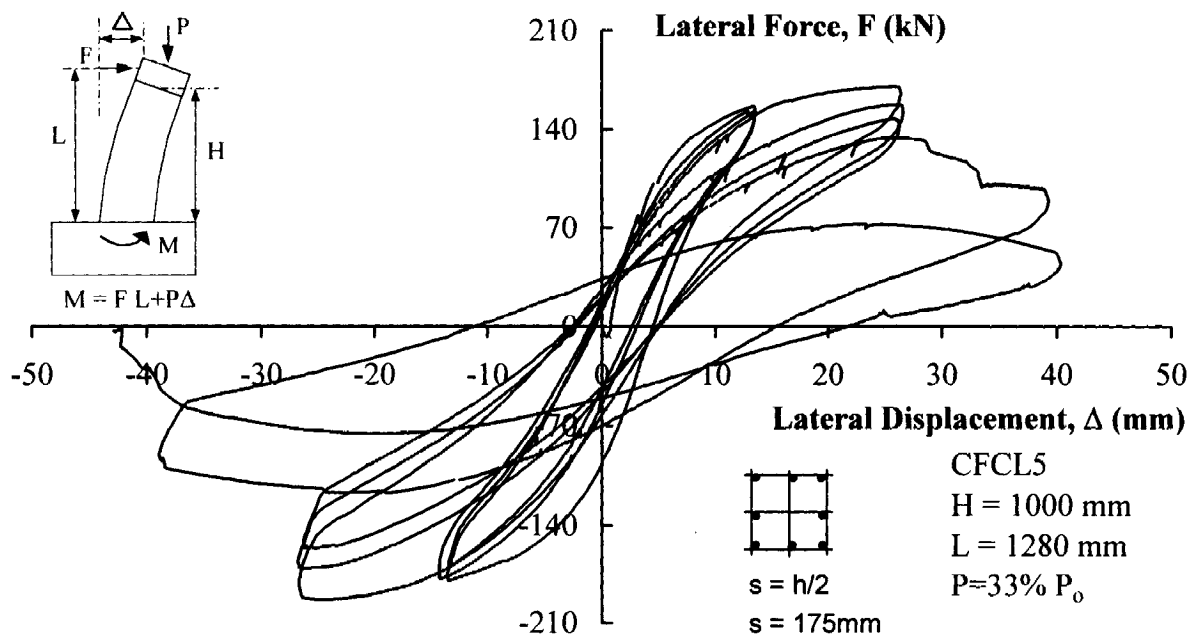


(b) Moment-Slip Rotation Relationship

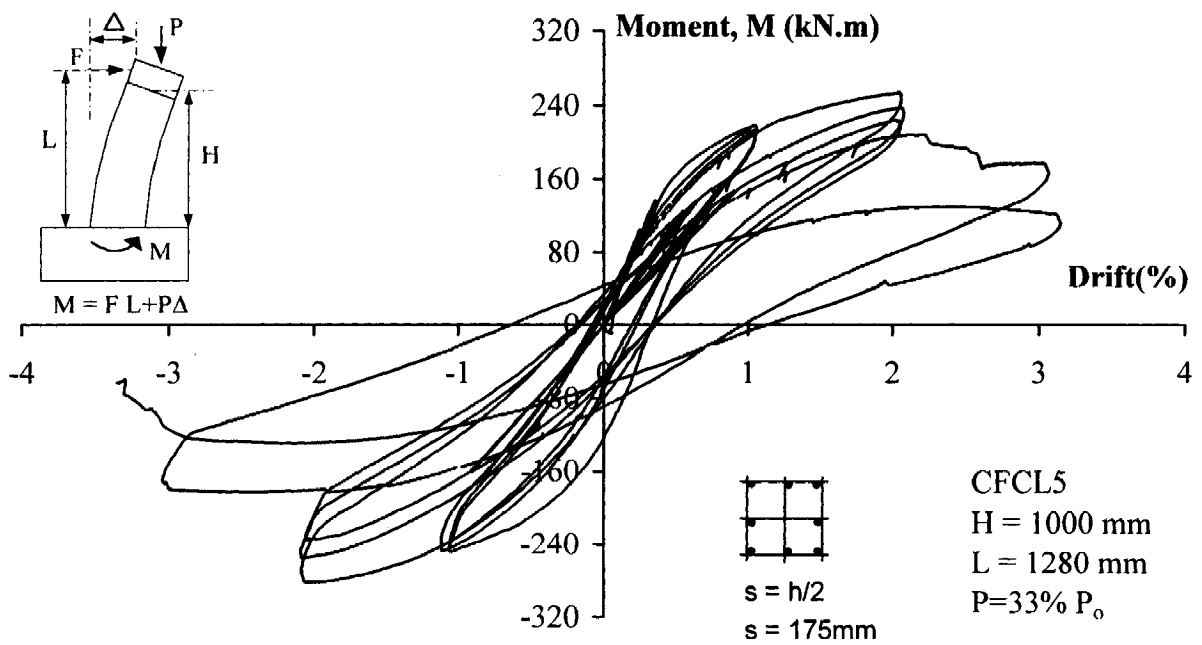


(c) Moment-Flexural Rotation Relationship

Figure 4.45: Hysteretic Moment-Rotation Relationships for Column CFCL4



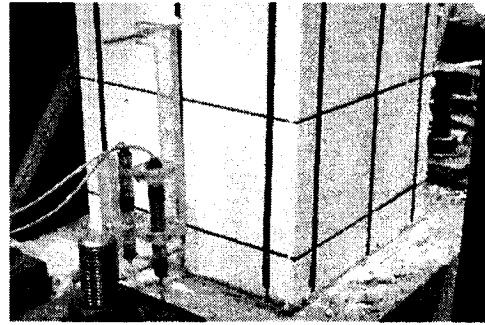
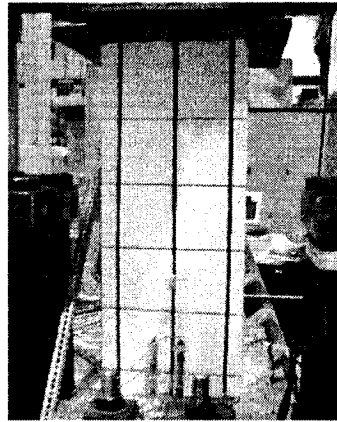
(a) Lateral Force-Lateral Displacement Relationship



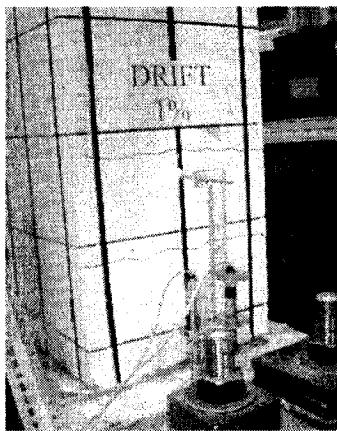
(b) Moment-Drift Relationship

Figure 4.46: Hysteretic Behavior for Column CFCL5





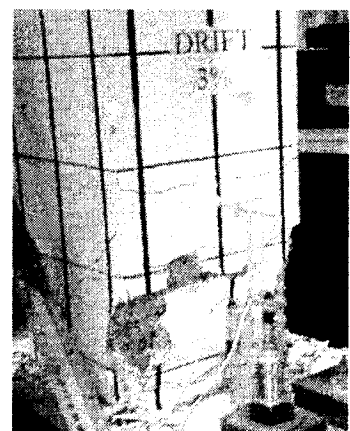
(a) Before Loading



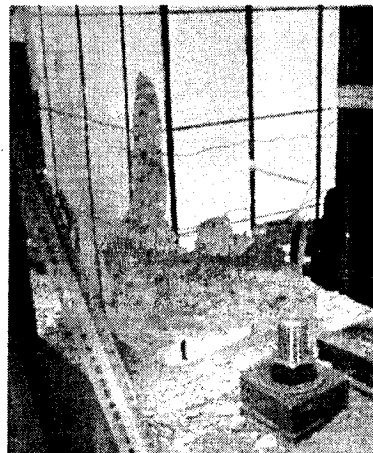
(b) At 1% Drift



(c) At 2% Drift



(d) At 3% Drift



(e) Column Failure



(f) Concrete Crushing

Figure 4.47: Observed Damage in Column CFCL5 at Selected Stages of Loading

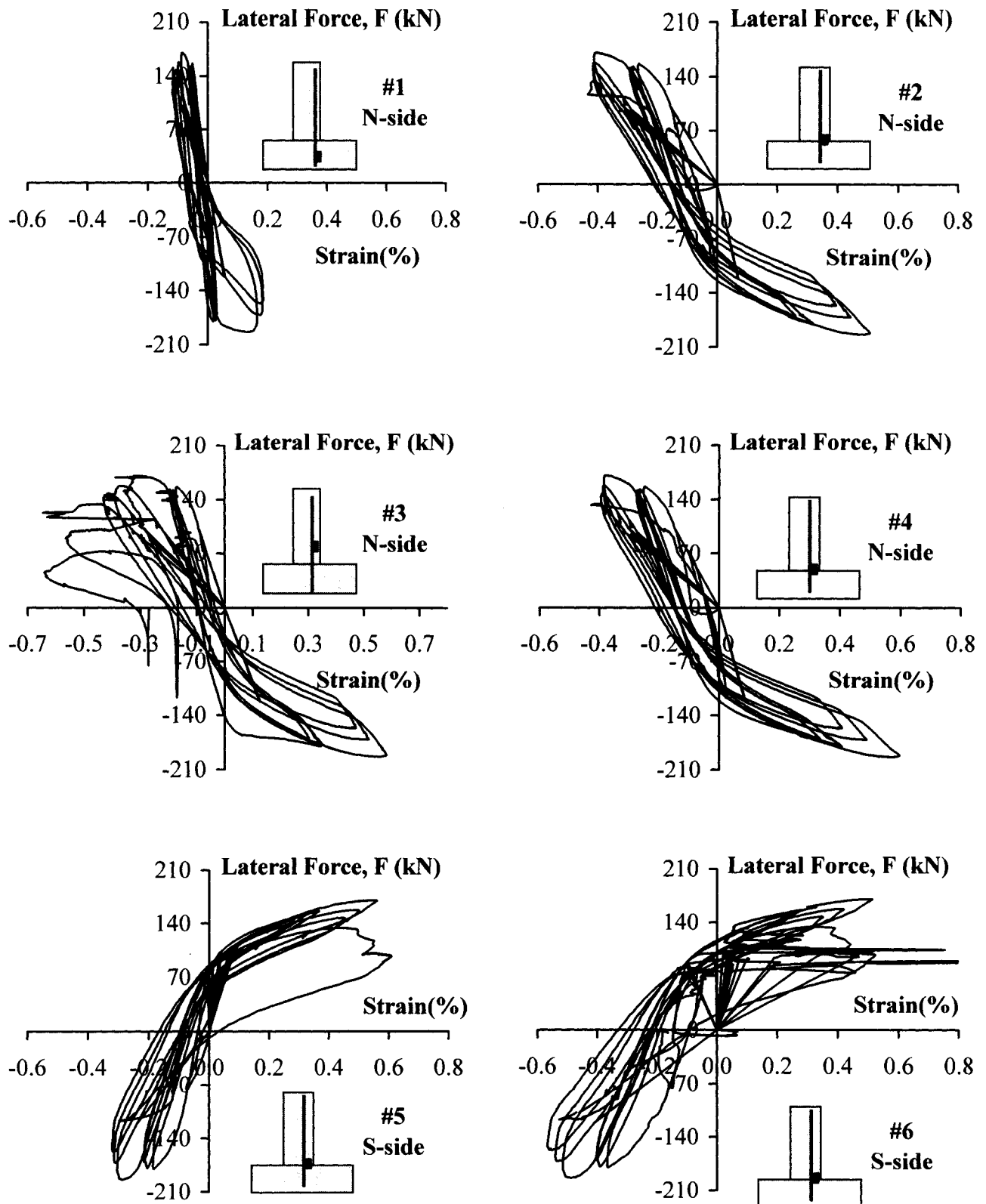


Figure 4.48: Bar Strain Readings in Column CFCL5

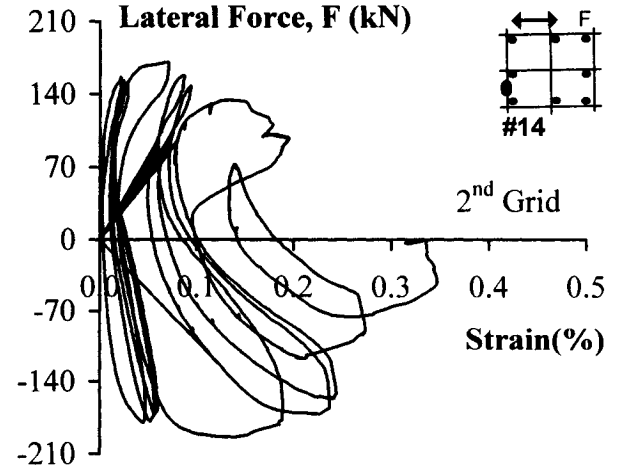
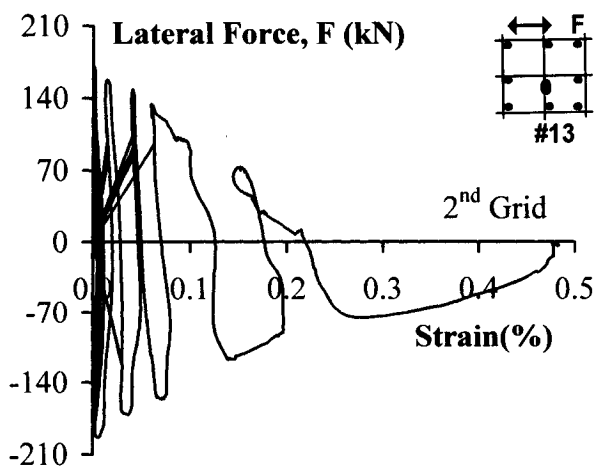
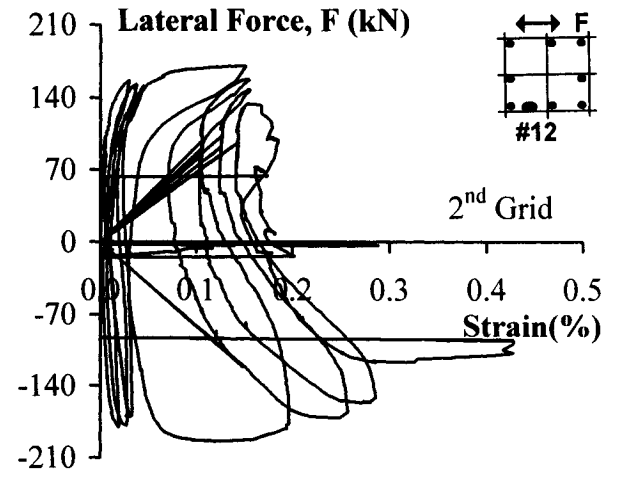
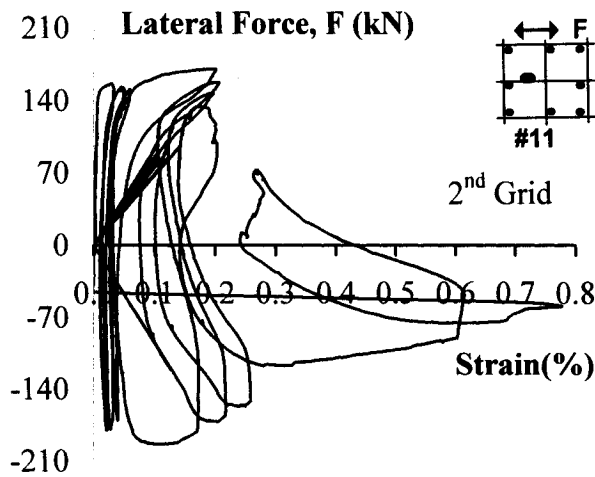
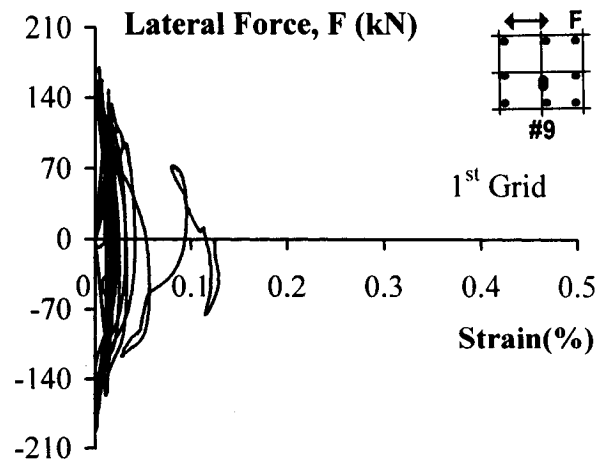
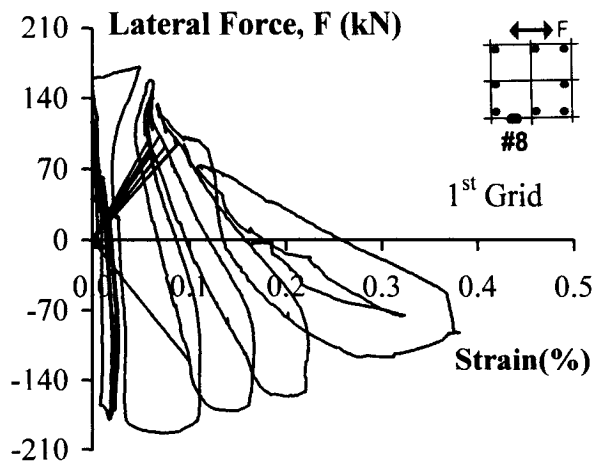
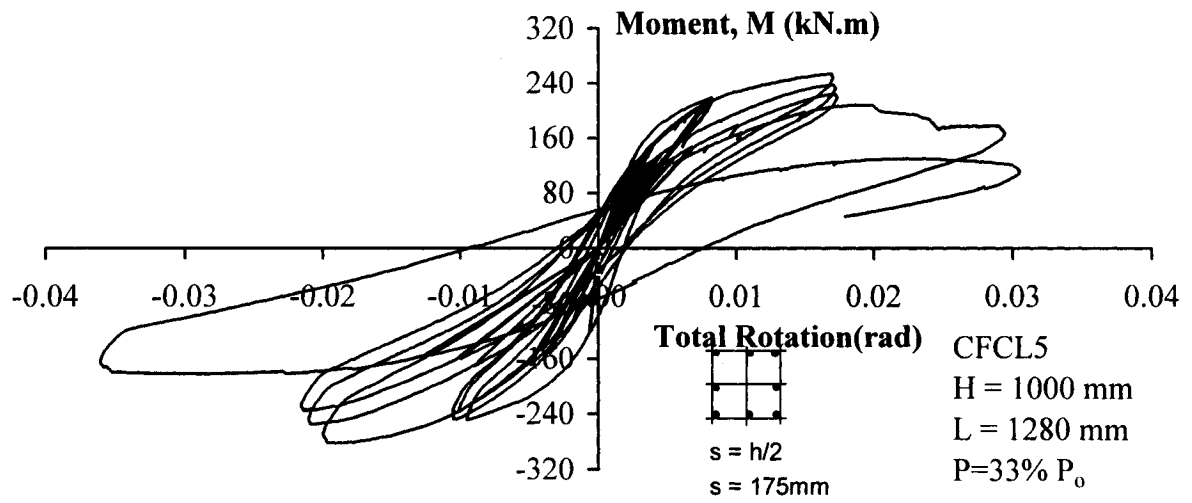
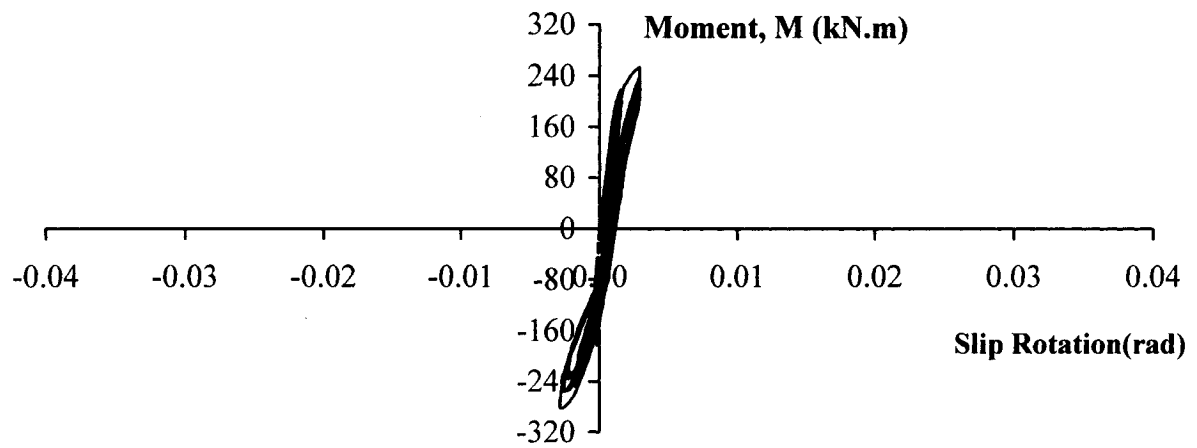


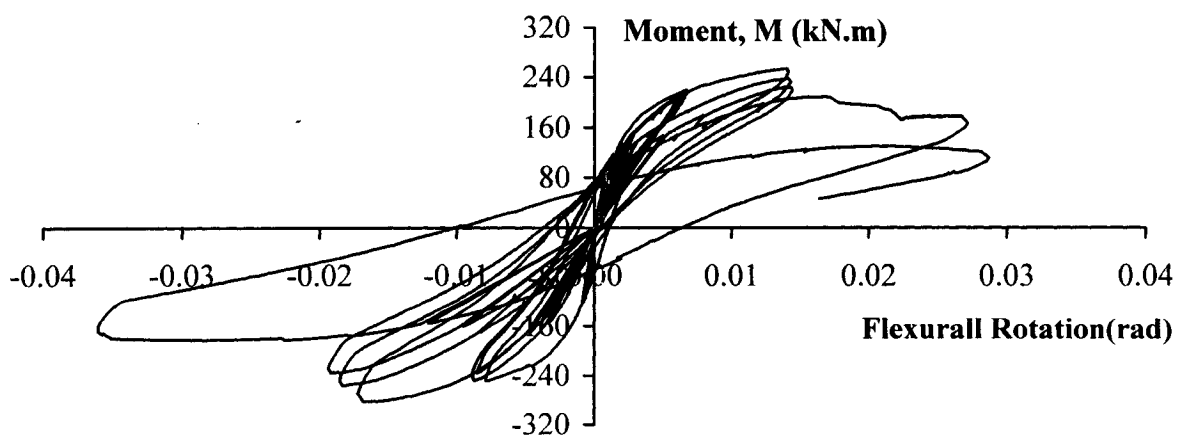
Figure 4.49: Grid Strain Readings in Column CFCL5



(a) Moment-Total Rotation Relationship

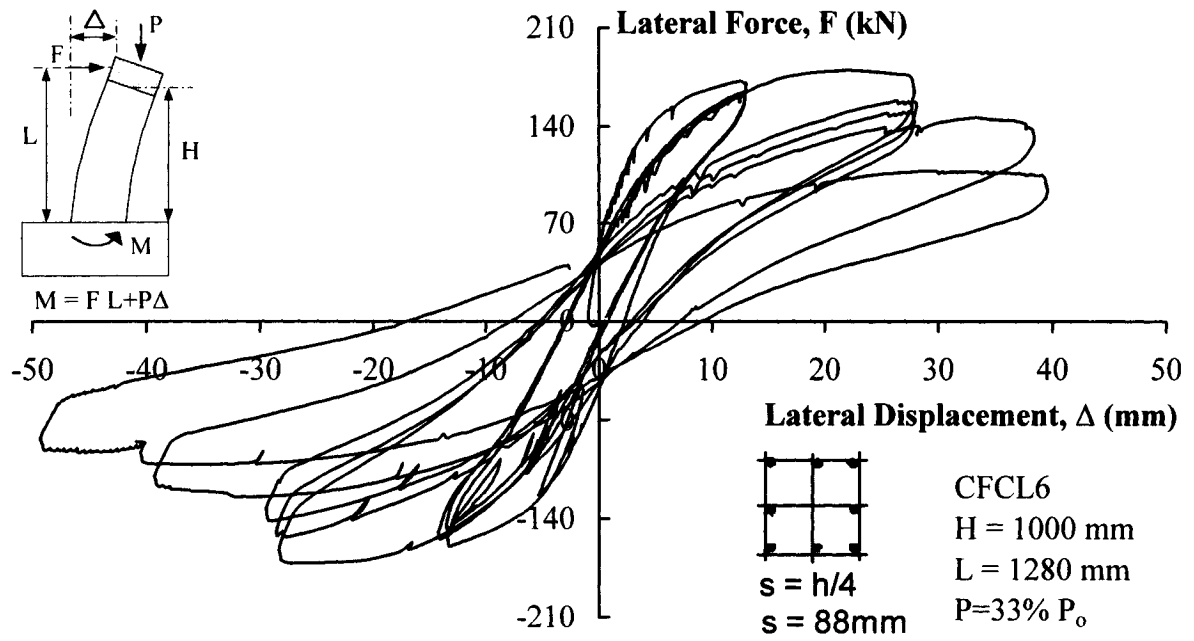


(b) Moment-Slip Rotation Relationship

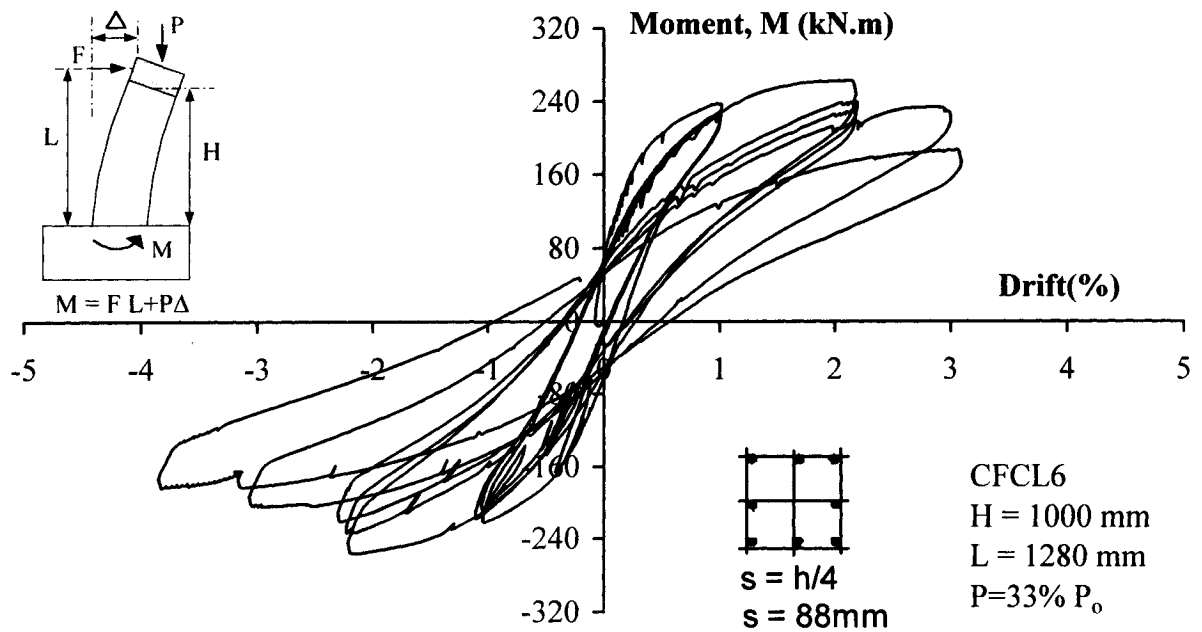


(c) Moment-Flexural Rotation Relationship

Figure 4.50: Hysteretic Moment-Rotation Relationships for Column CFCL5

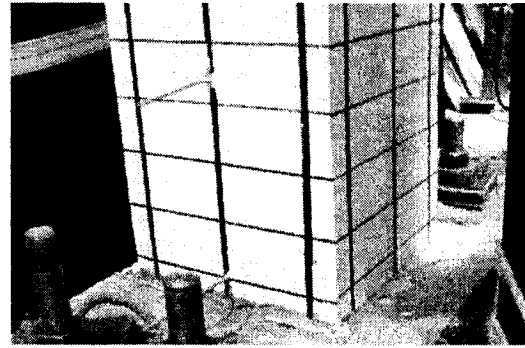


(a) Lateral Force-Lateral Displacement Relationship

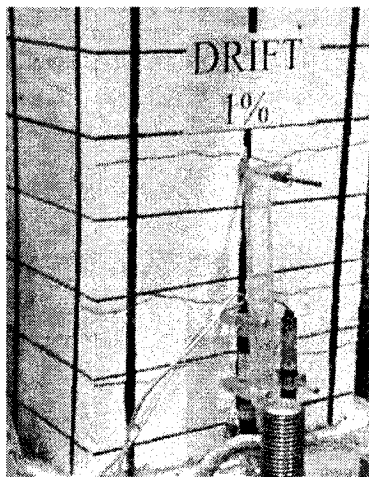


(b) Moment-Drift Relationship

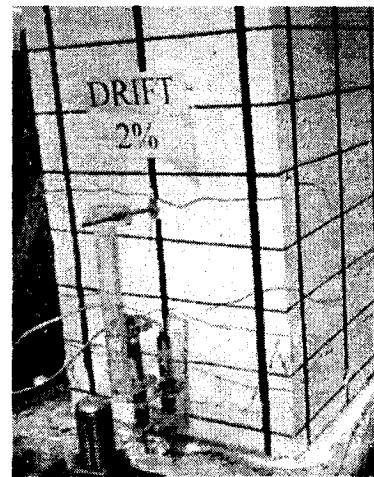
Figure 4.51: Hysteretic Behavior for Column CFCL6



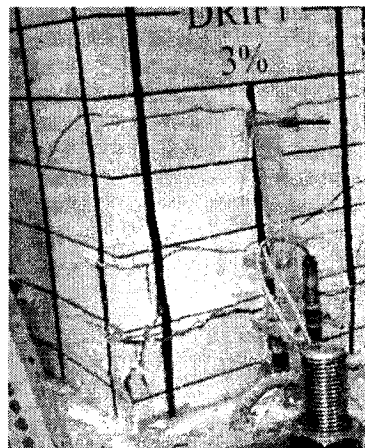
(a) Before Loading



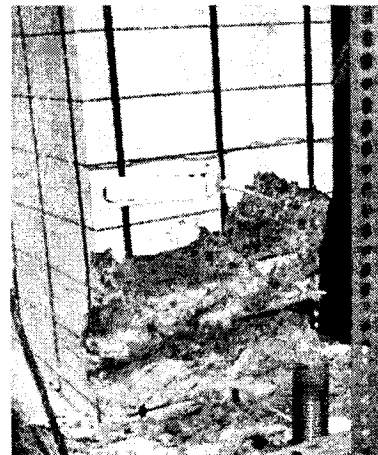
(b) At 1% Drift



(c) At 2% Drift



(d) At 3% Drift



(e) Column Failure

Figure 4.52: Observed Damage in Column CFCL6 at Selected Stages of Loading

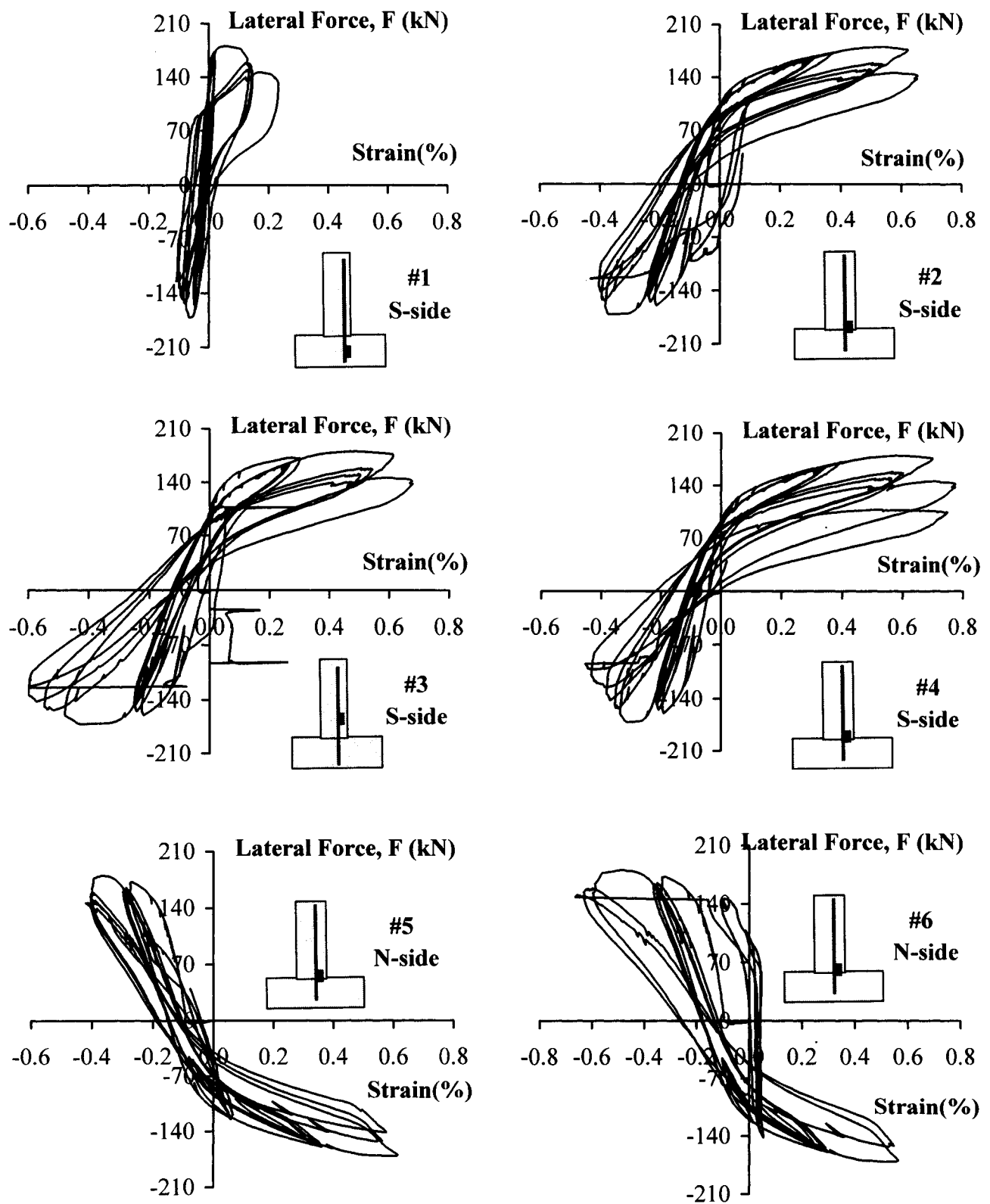


Figure 4.53: Bar Strain Readings in Column CFCL6

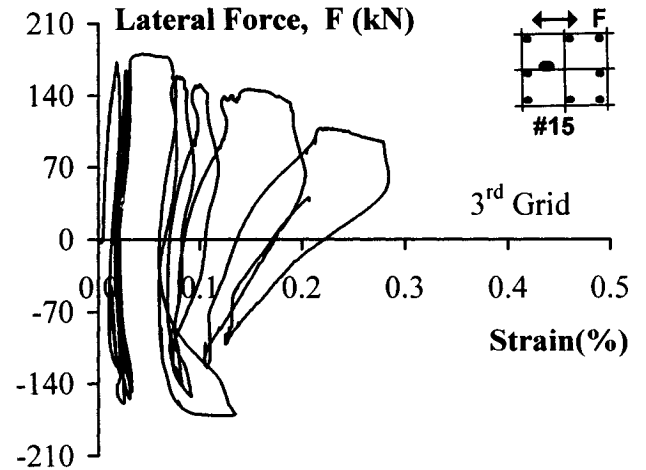
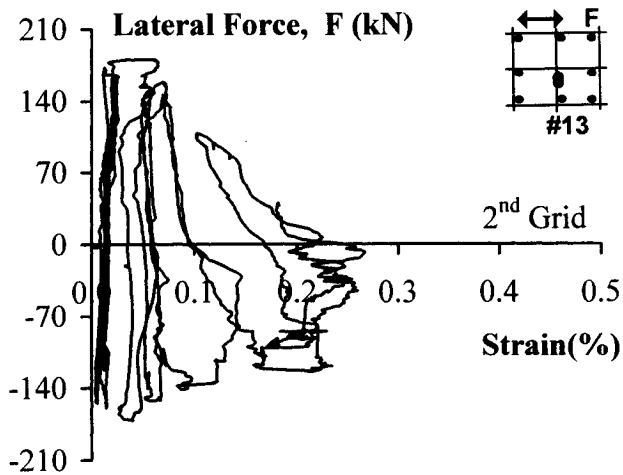
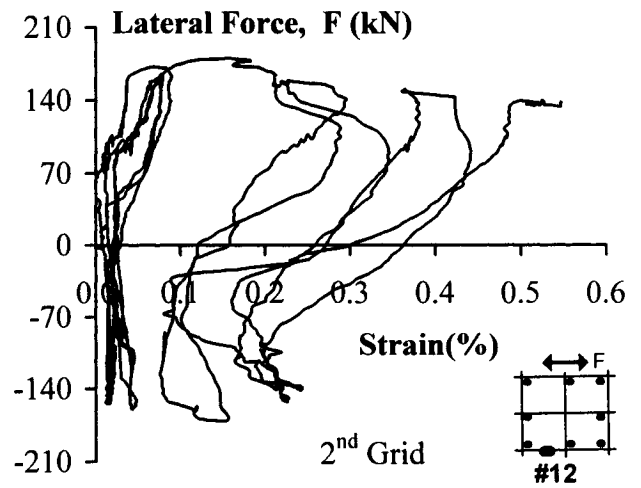
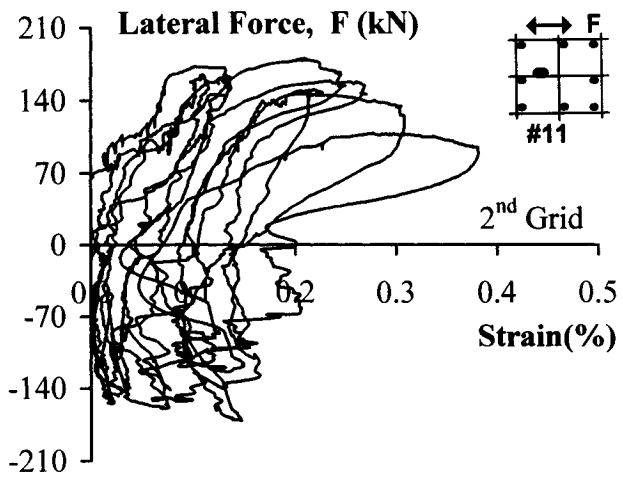
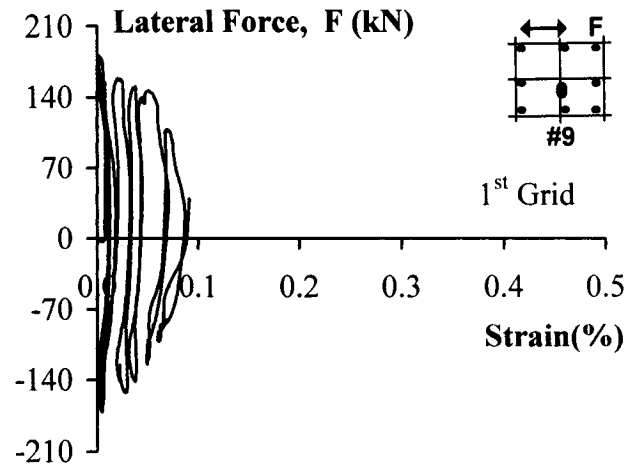
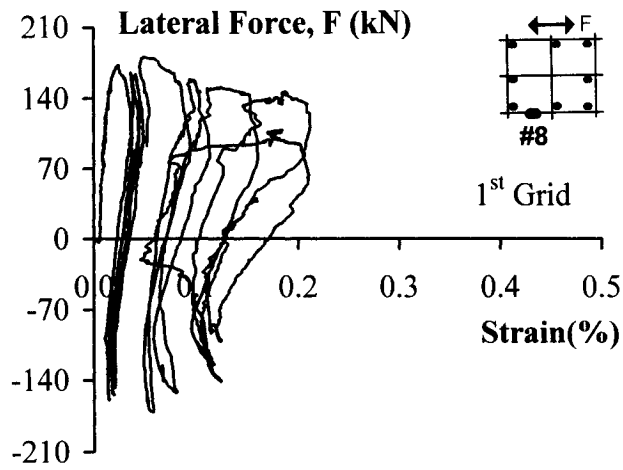


Figure 4.54: Grid Strain Readings in Column CFCL6



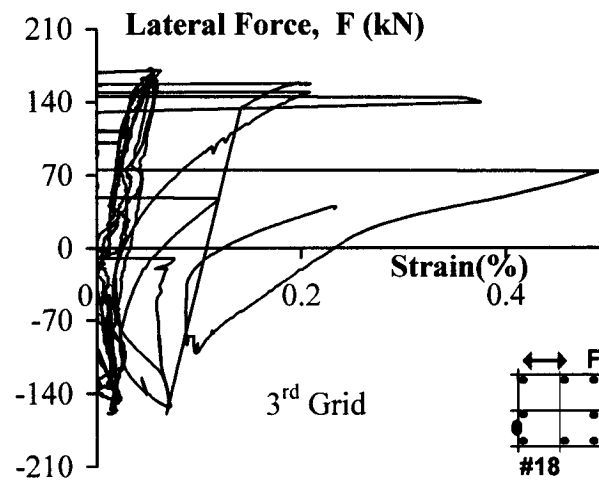
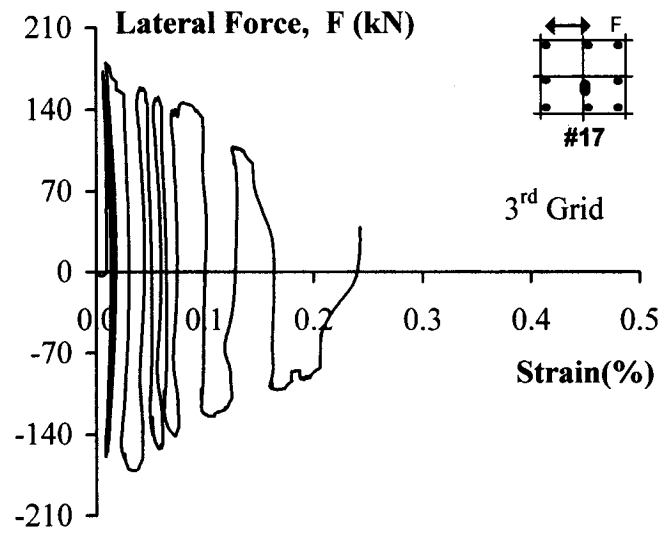
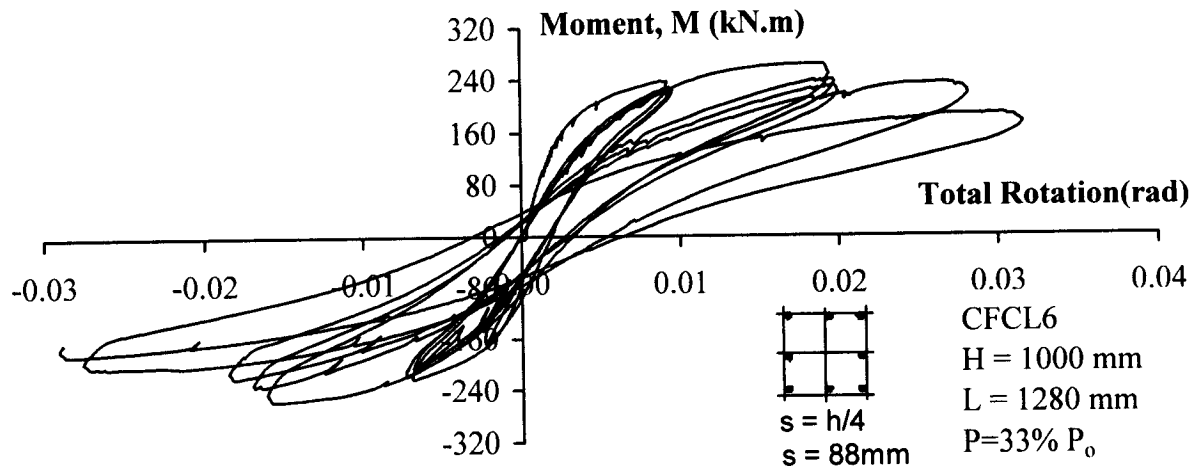
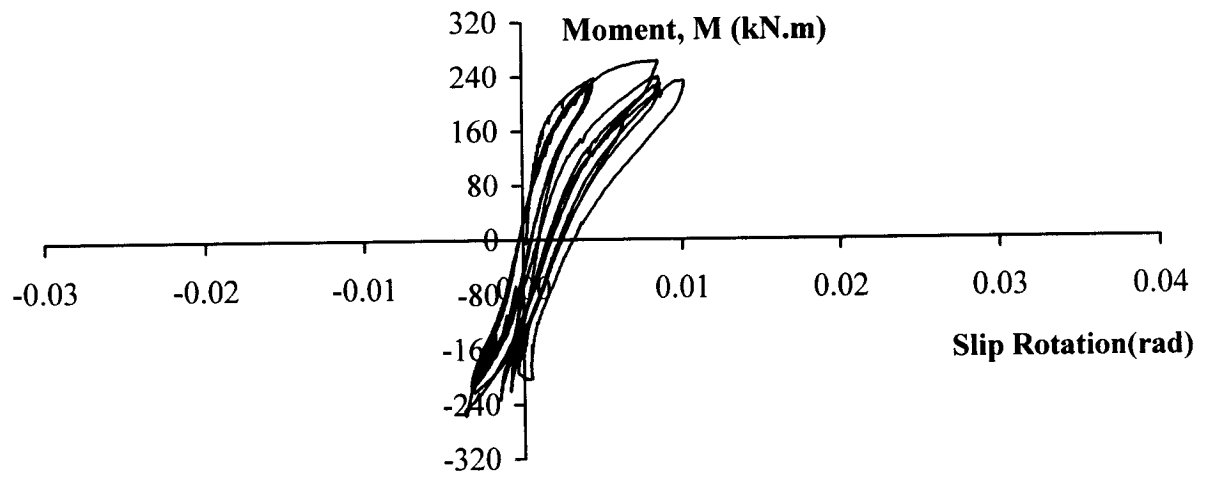


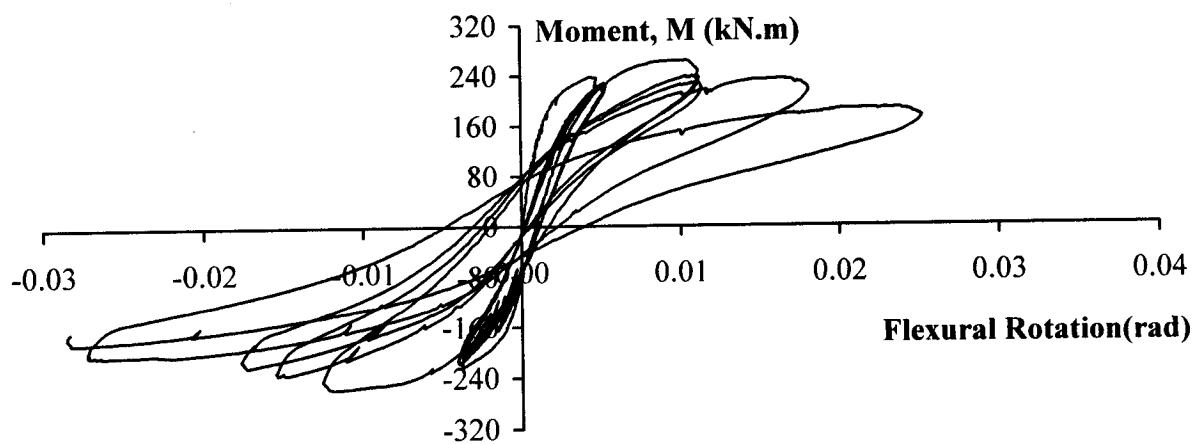
Figure 4.54 (Cont.): Grid Strain Readings in Column CFCL6



(a) Moment-Total Rotation Relationship

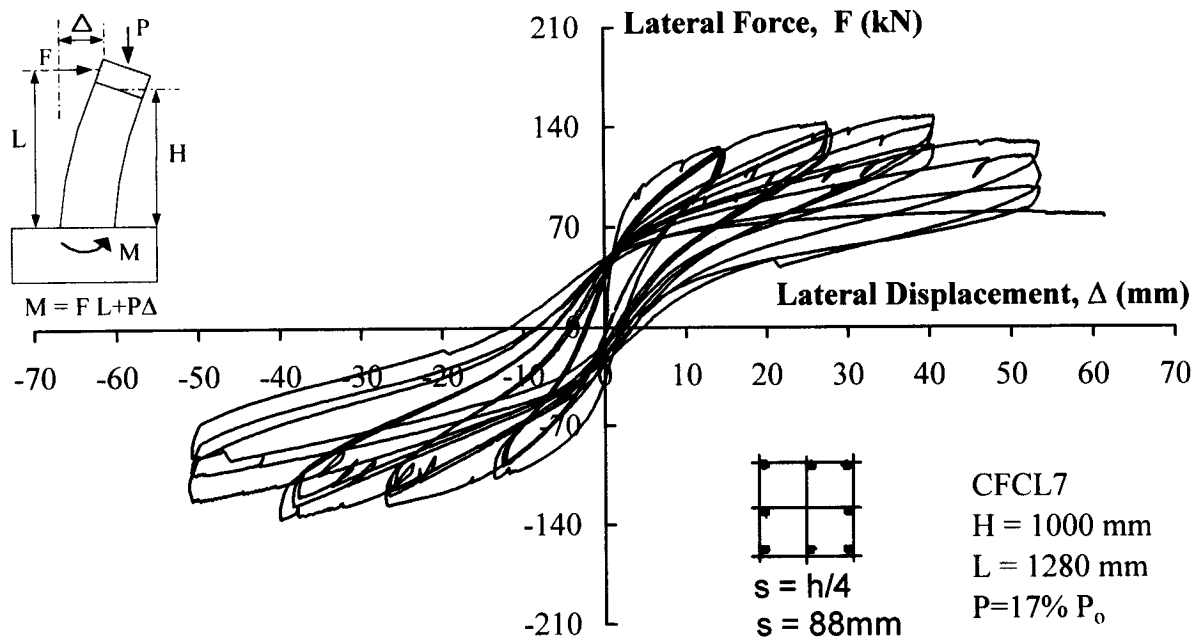


(b) Moment-Slip Rotation Relationship

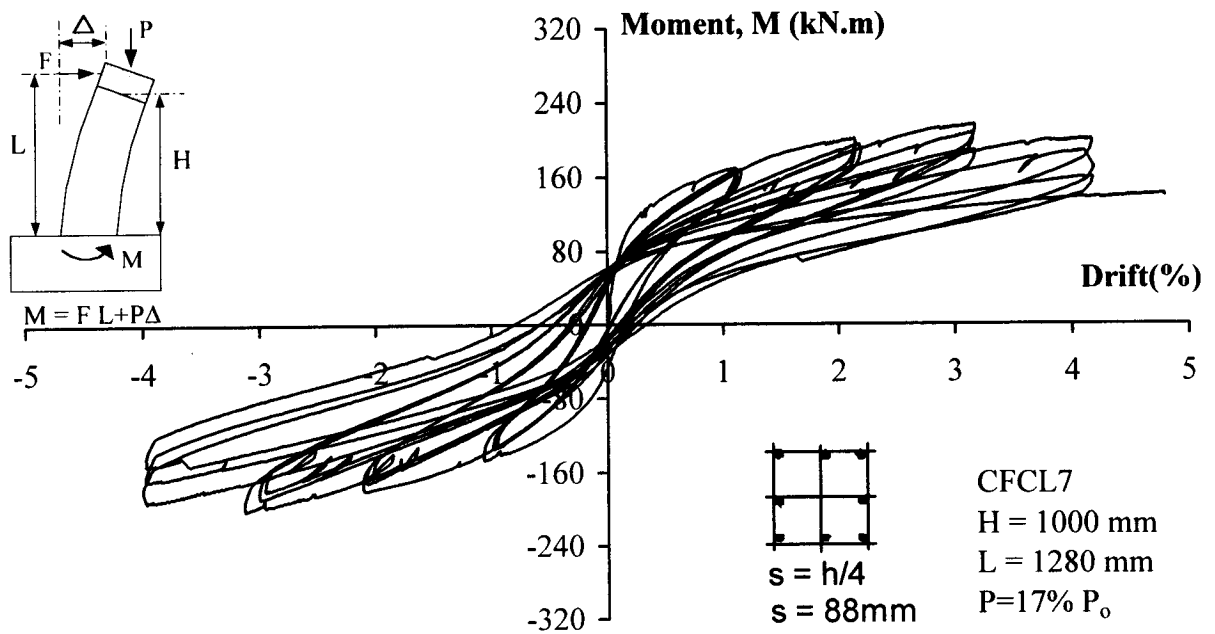


(c) Moment-Flexural Rotation Relationship

Figure 4.55: Hysteretic Moment-Rotation Relationships for Column CFCL6

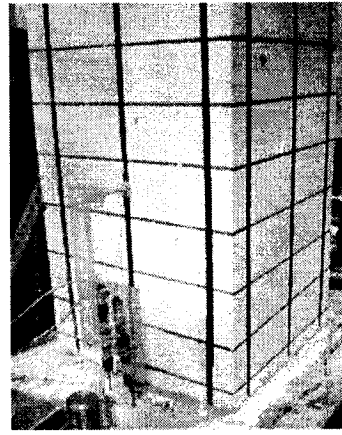
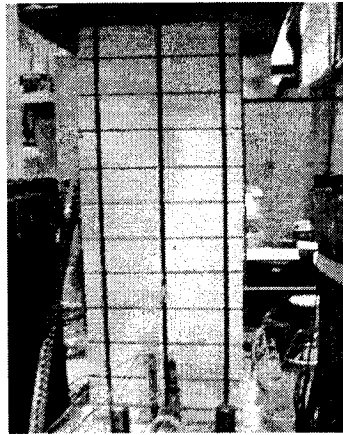


(a) Lateral Force-Lateral Displacement Relationship



(b) Moment-Drift Relationship

Figure 4.56: Hysteretic Behavior for Column CFCL7



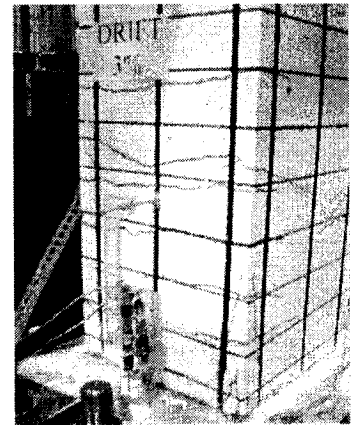
(a) Before Loading



(b) At 1% Drift



(c) At 2% Drift



(d) At 3% Drift



(e) At 4% Drift



(f) Column Failure

Figure 4.57: Observed Damage in Column CFCL7 at Selected Stages of Loading

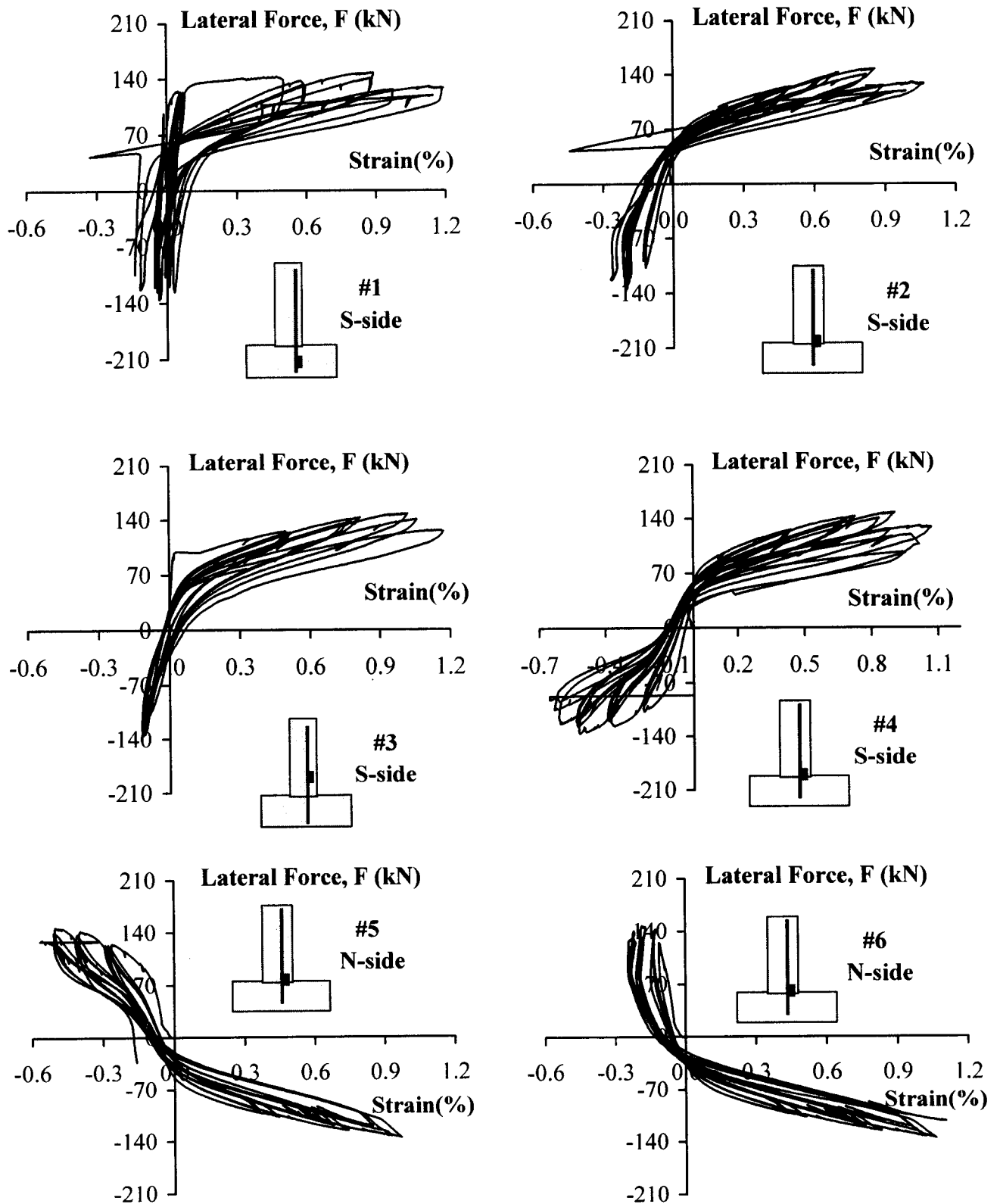


Figure 4.58: Bar Strain Readings in Column CFCL7

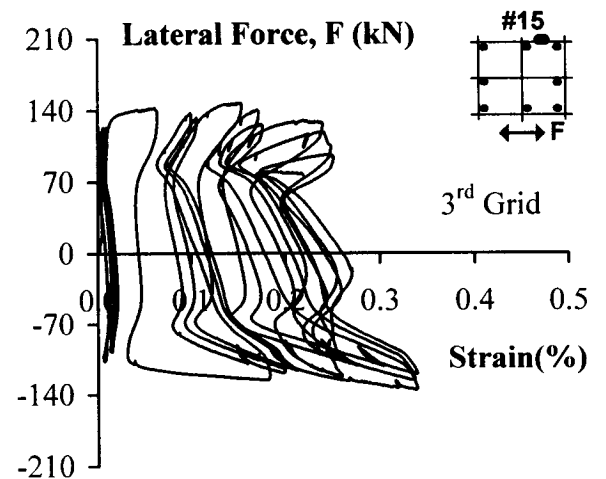
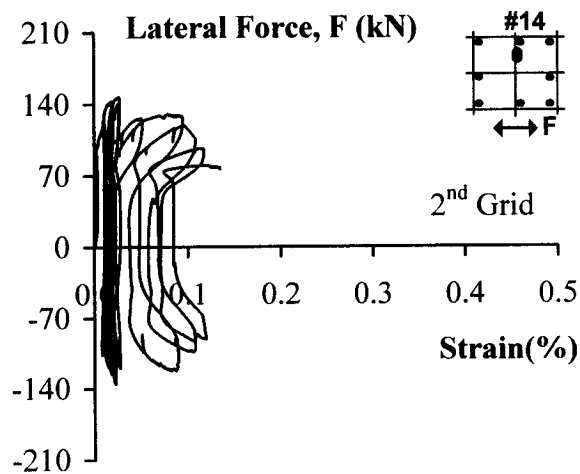
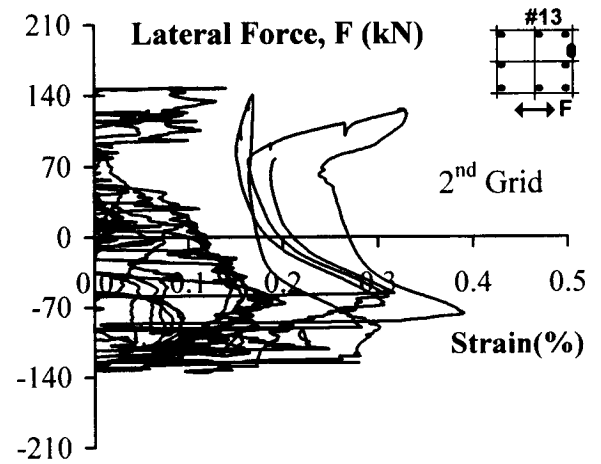
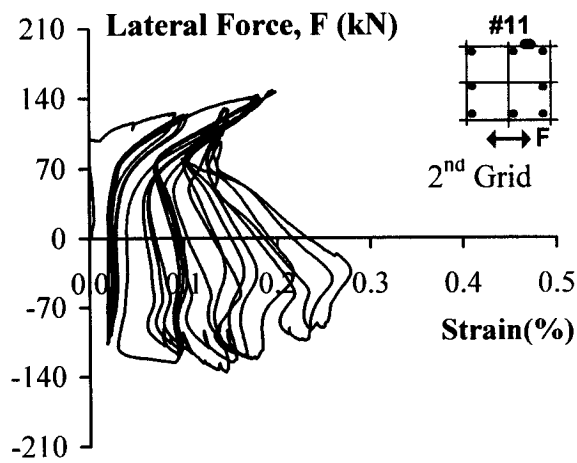
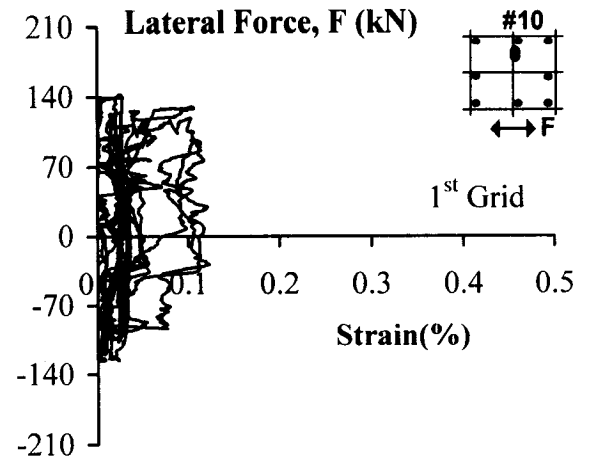
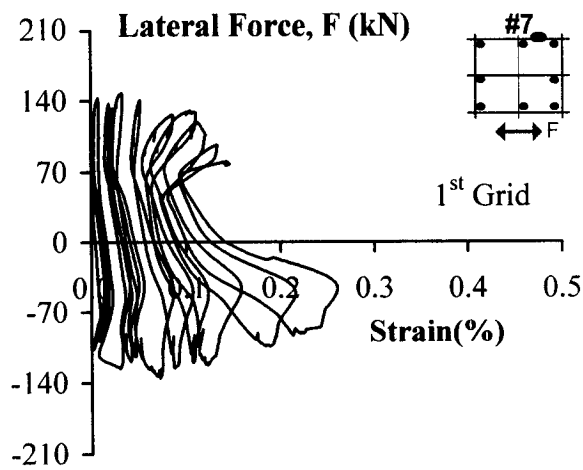


Figure 4.59: Grid Strain Readings in Column CFCL7

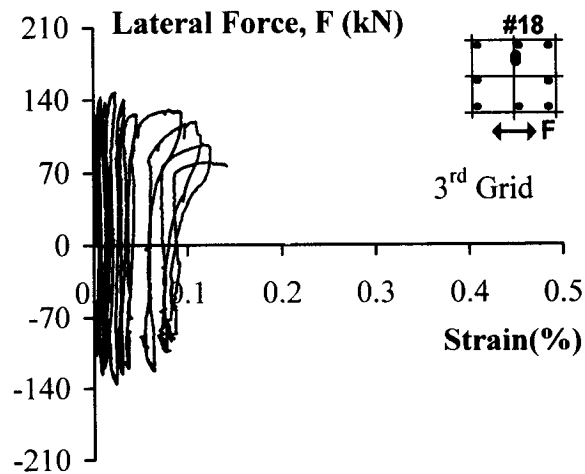
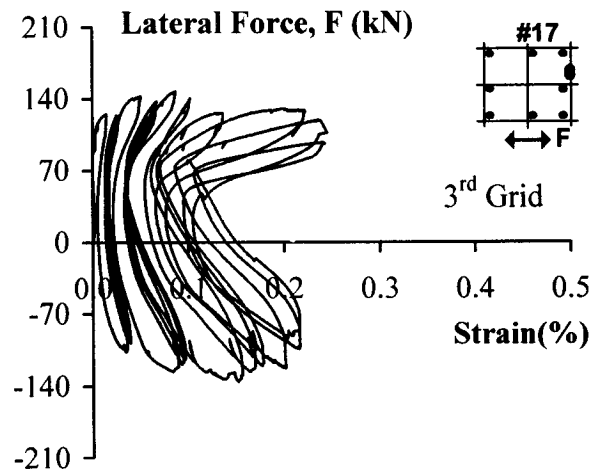
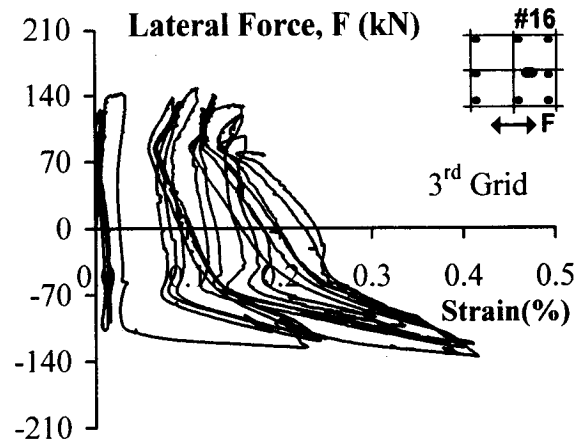
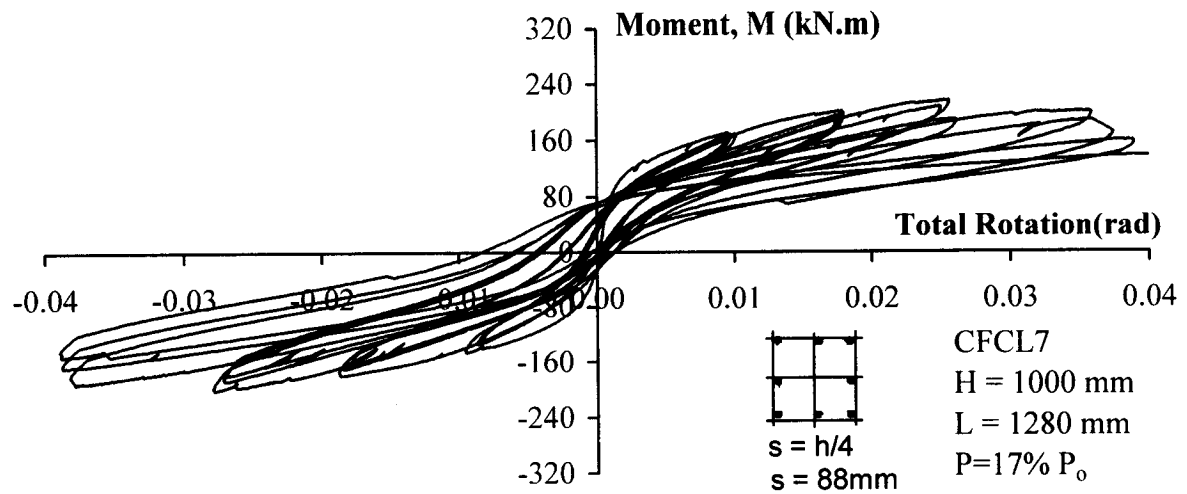
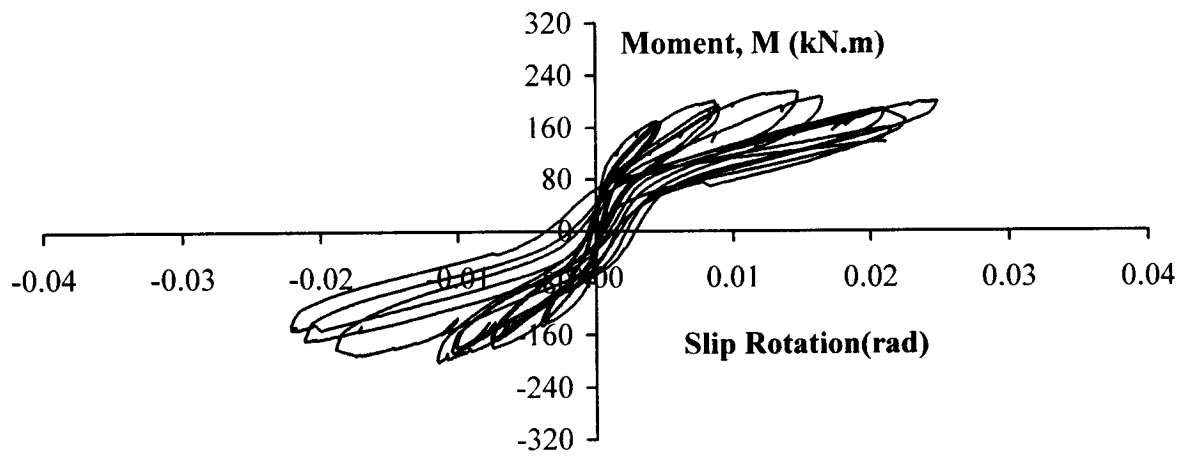


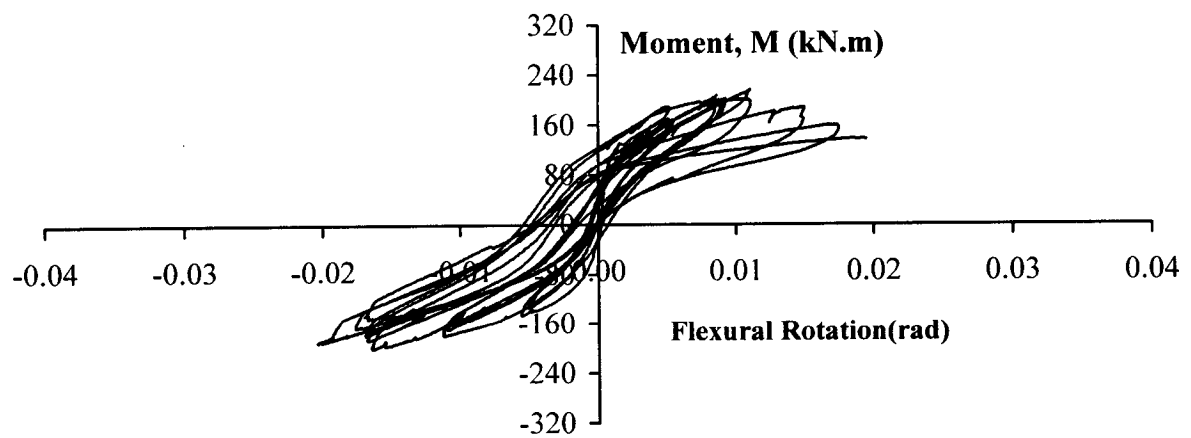
Figure 4.59 (Cont.): Grid Strain Readings in Column CFCL7



(a) Moment-Total Rotation Relationship



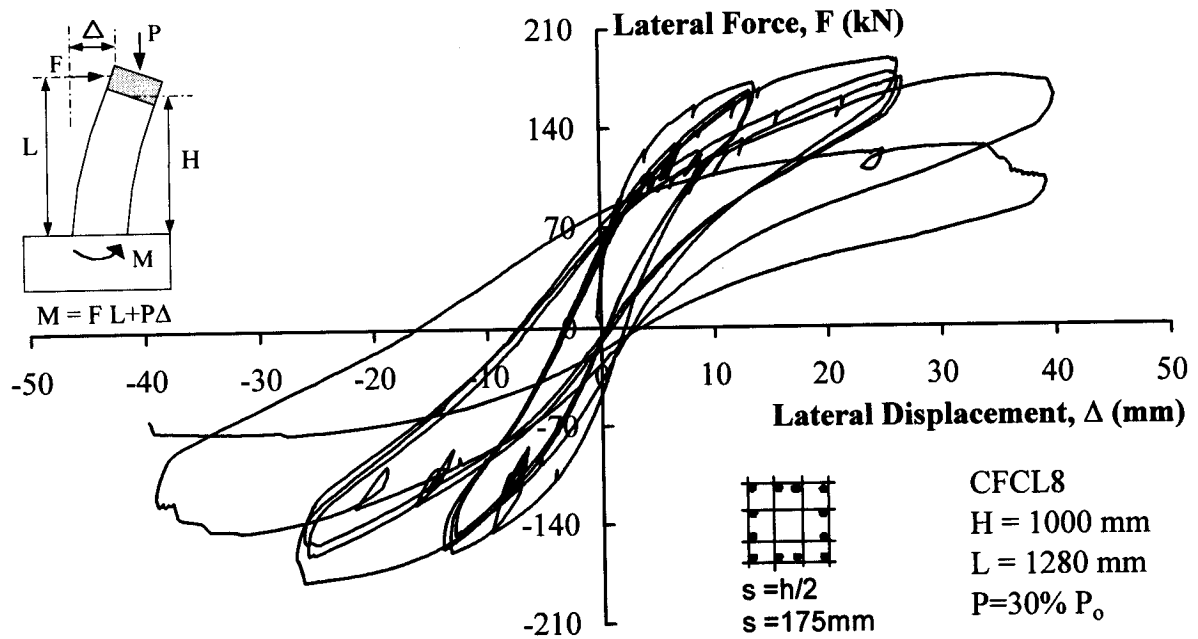
(b) Moment-Slip Rotation Relationship



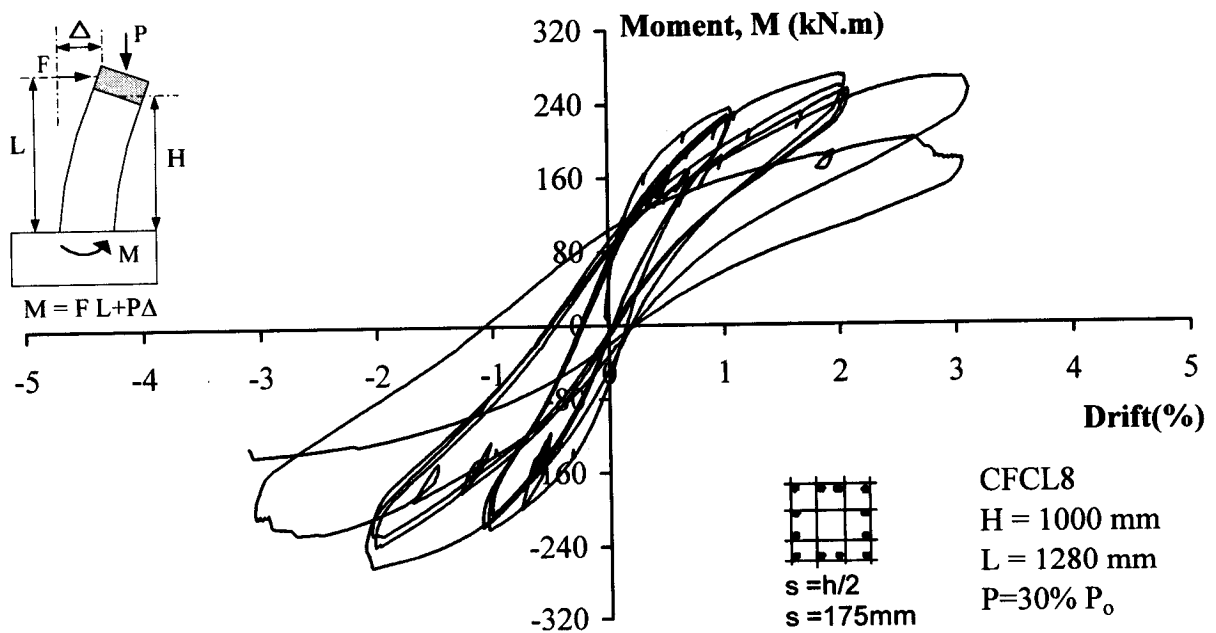
(c) Moment-Flexural Rotation Relationship

Figure 4.60: Hysteretic Moment-Rotation Relationships for Column CFCL7



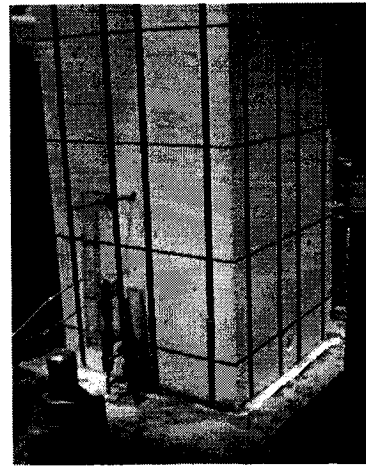
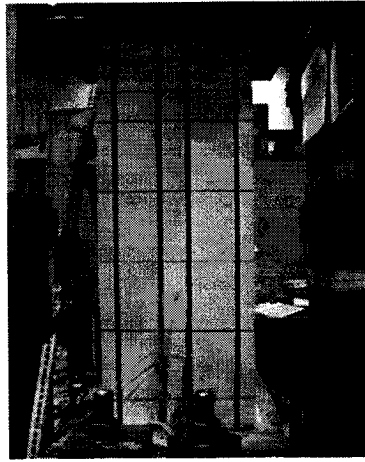


(a) Lateral Force-Lateral Displacement Relationship

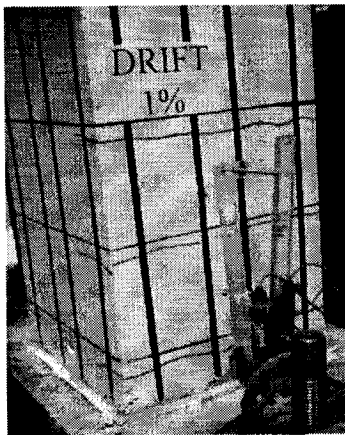


(b) Moment-Drift Relationship

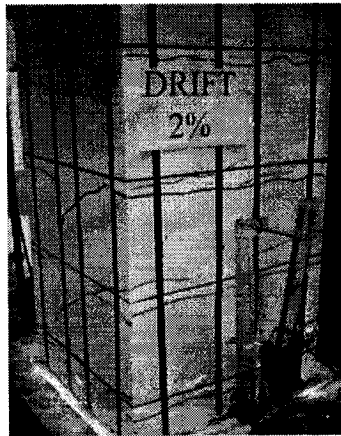
Figure 4.61: Hysteretic Behavior for Column CFCL8



(a) Before Loading



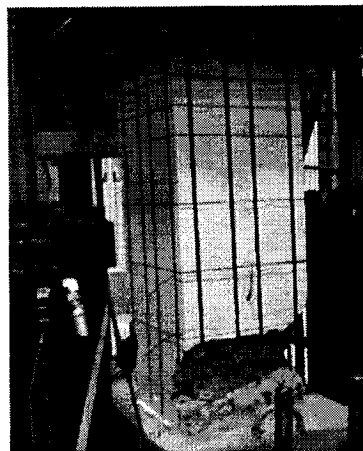
(b) At 1% Drift



(c) At 2% Drift



(d) At 3% Drift



(e) Crushing of Concrete



(f) Buckling of Bars in Compression

Figure 4.62: Observed Damage in Column CFCL8 at Selected Stages of Loading

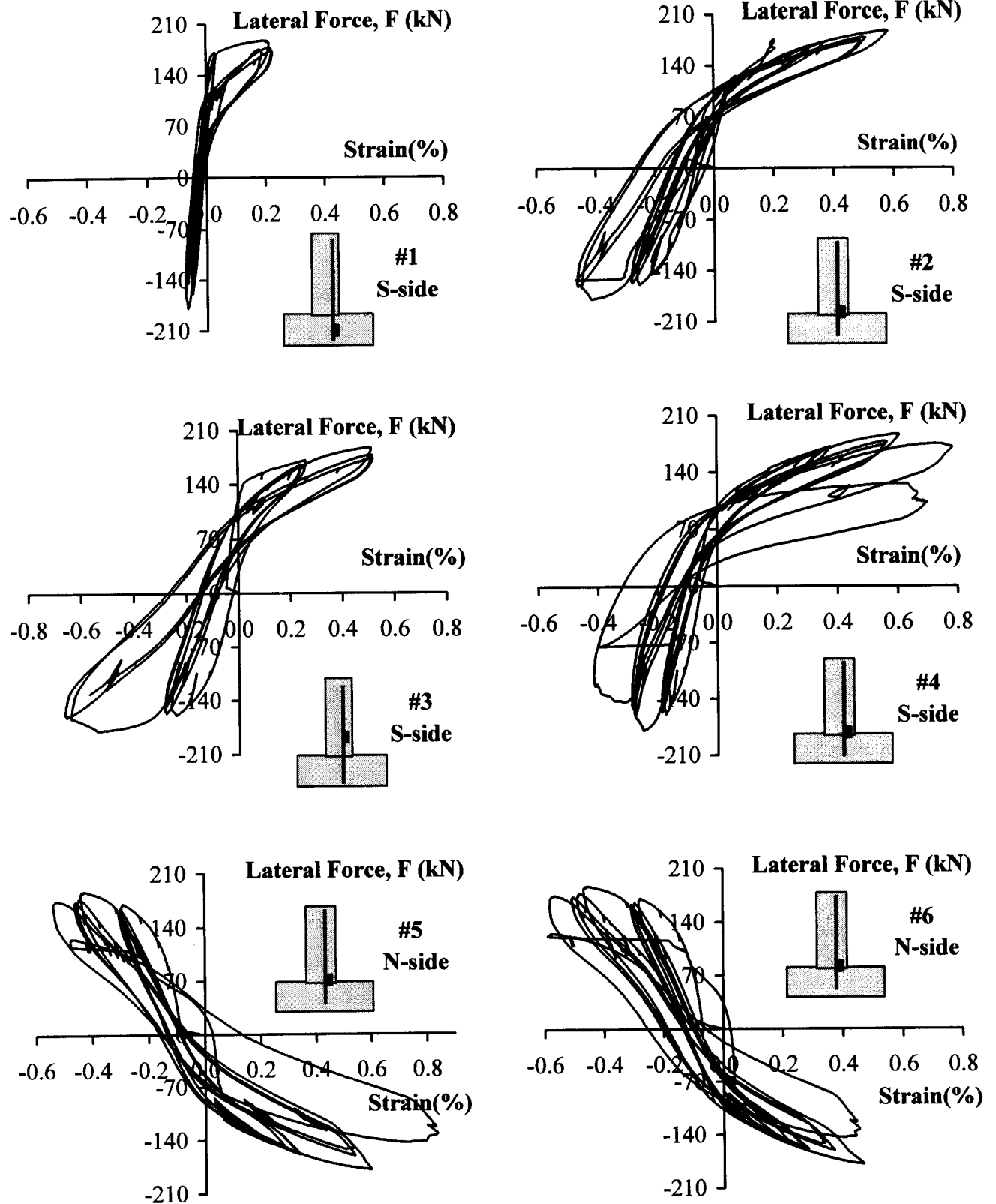


Figure 4.63: Bar Strain Readings in Column CFCL8

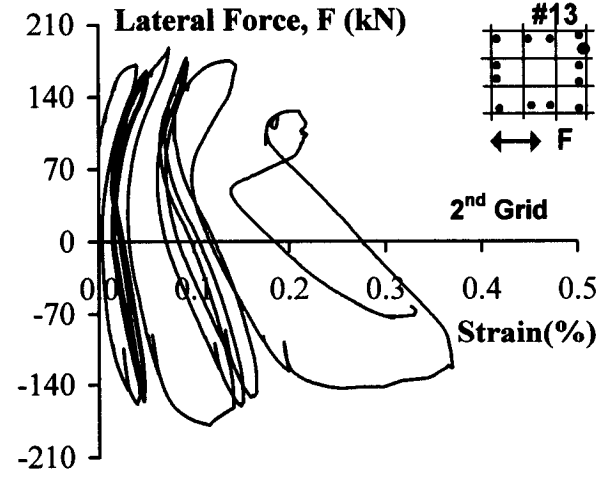
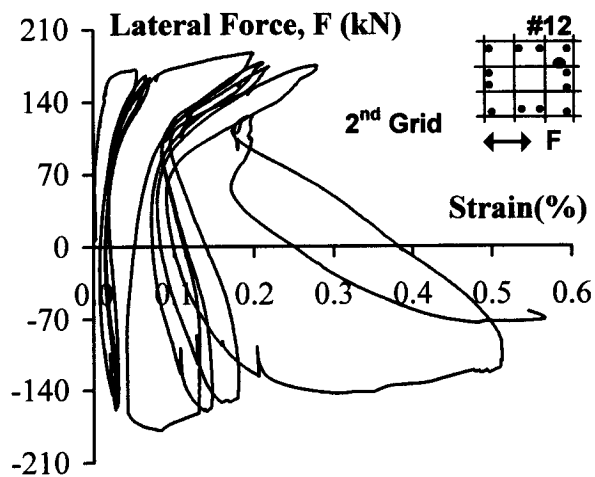
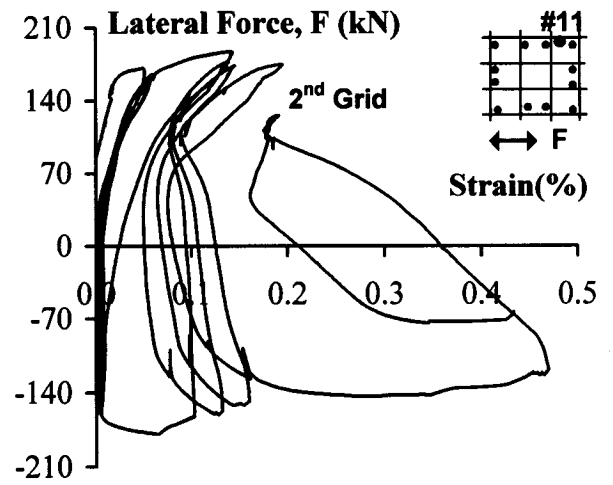
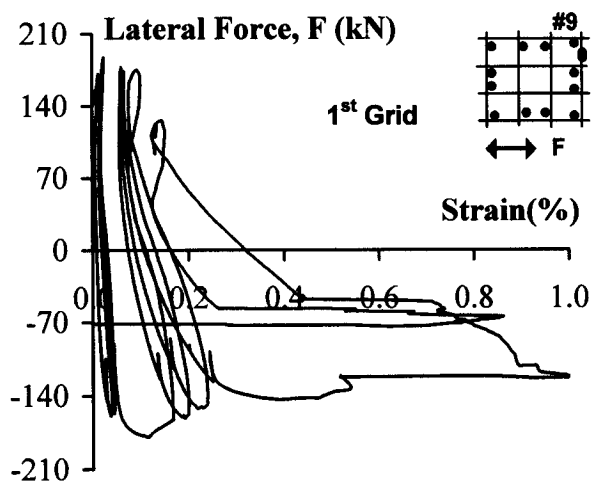
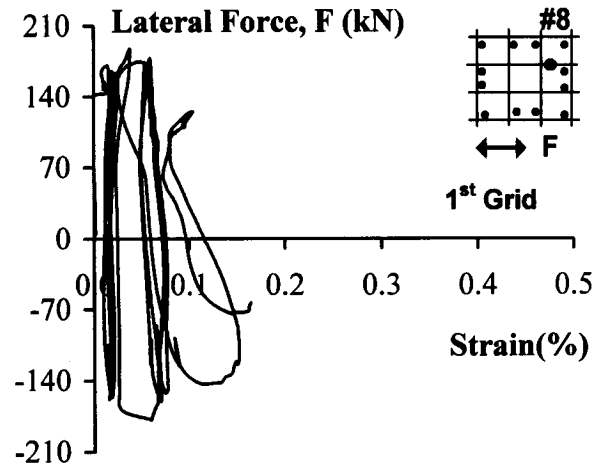
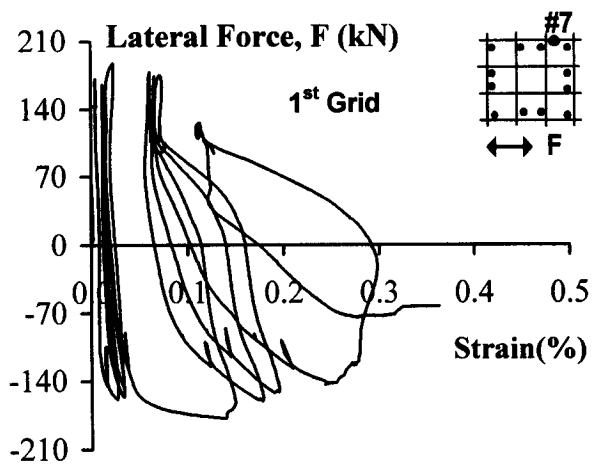
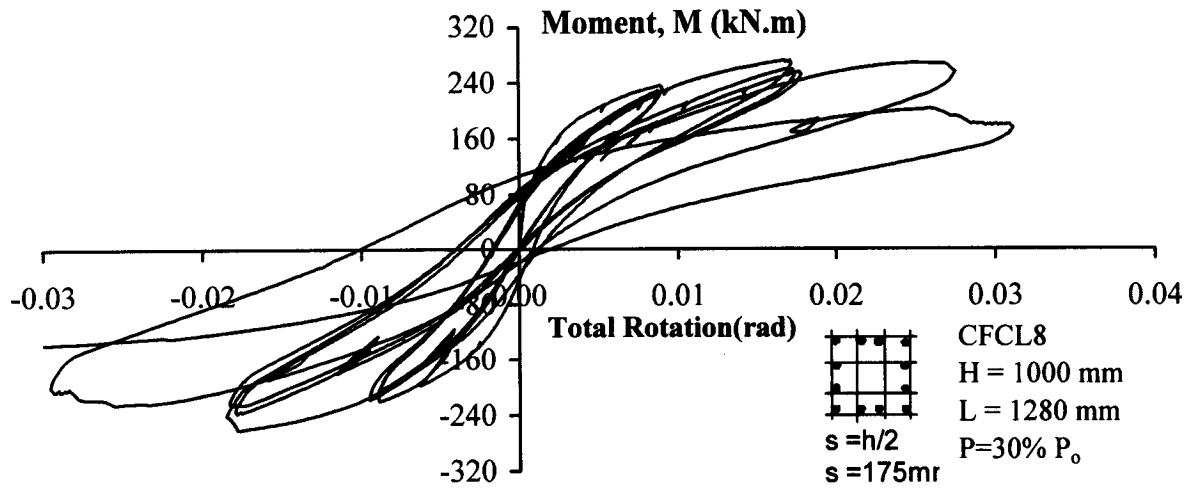
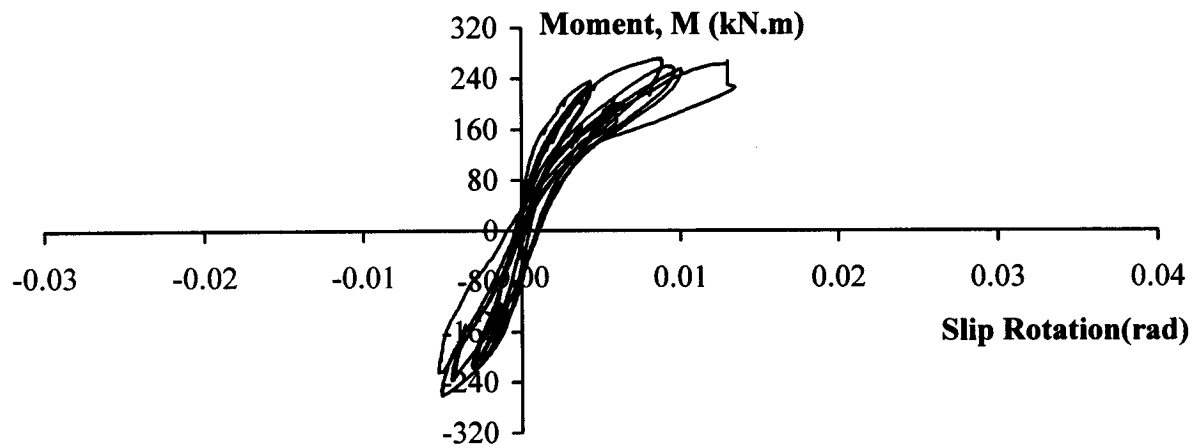


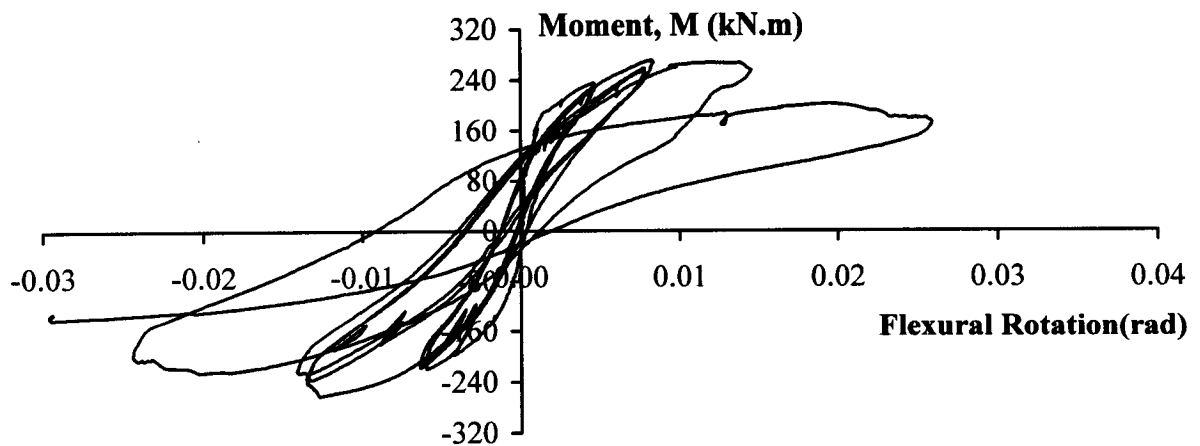
Figure 4.64: Grid Strain Readings in Column CFCL8



(a) Moment-Total Rotation Relationship

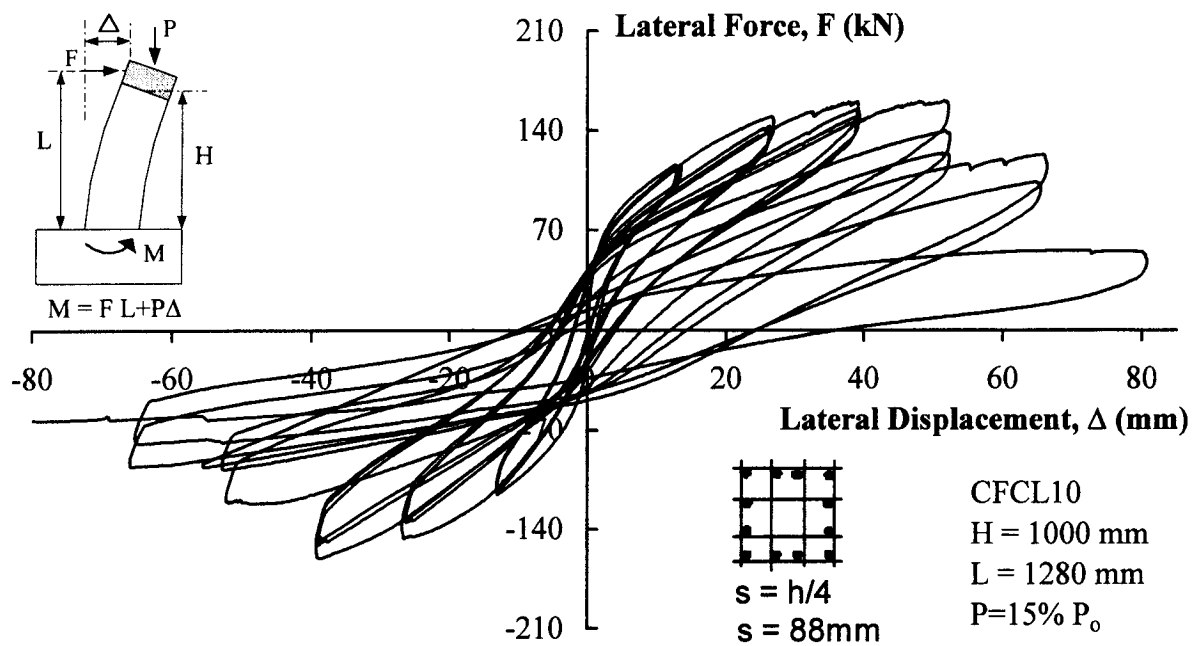


(b) Moment-Slip Rotation Relationship

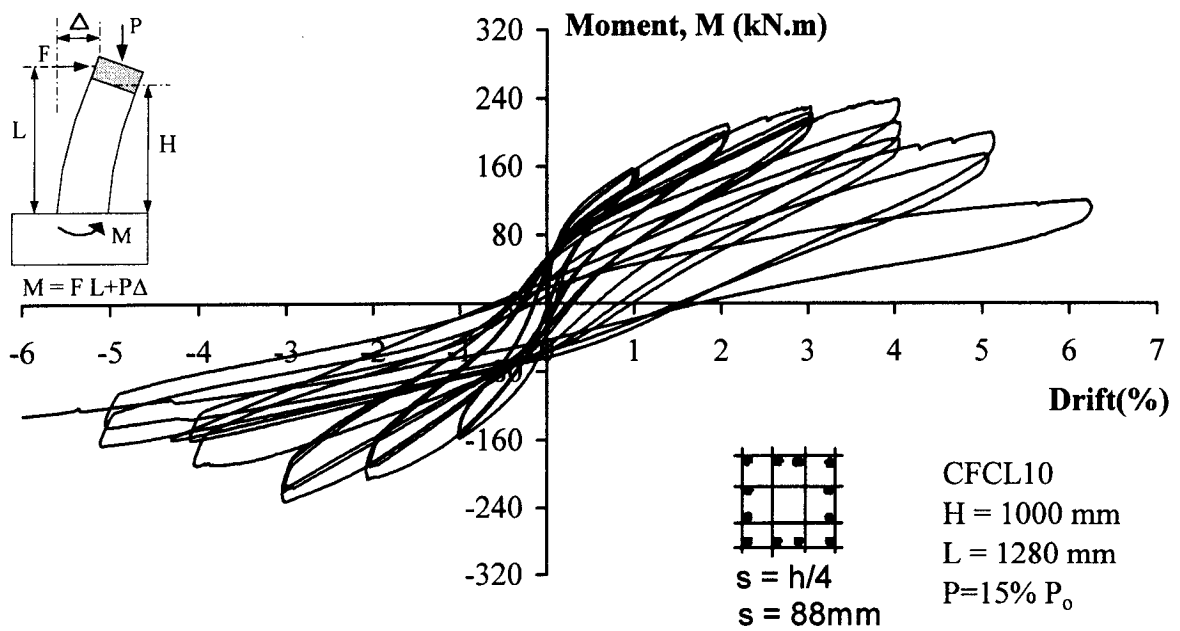


(c) Moment-Flexural Rotation Relationship

Figure 4.65: Hysteretic Moment-Rotation Relationships for Column CFCL8

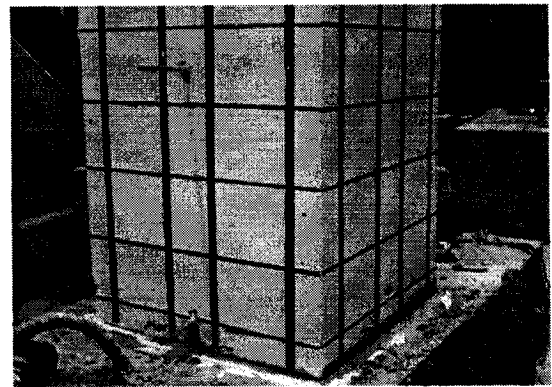
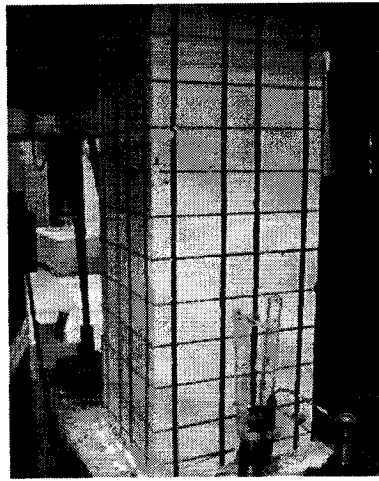


(a) Lateral Force-Lateral Displacement Relationship

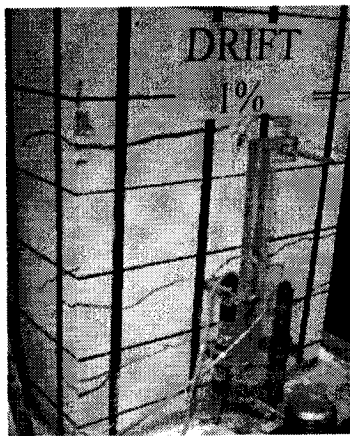


(b) Moment-Drift Relationship

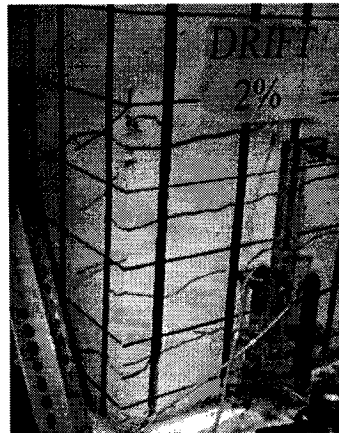
Figure 4.71: Hysteretic Behavior for Column CFCL10



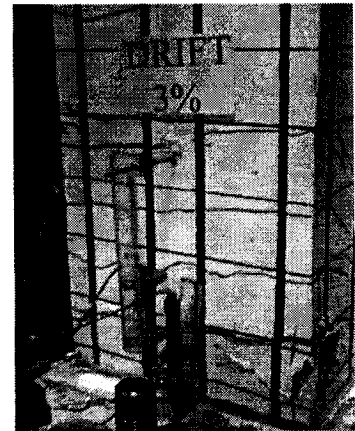
(a) Before Loading



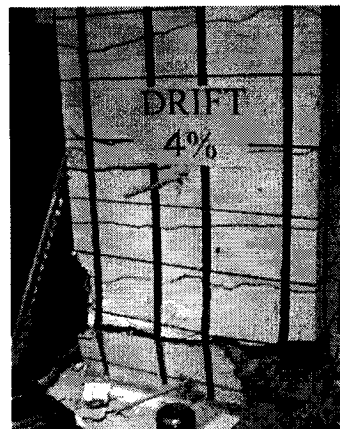
(b) At 1% Drift



(c) At 2% Drift



(d) At 3% Drift



(e) At 4% Drift



(f) Column Failure

Figure 4.67: Observed Damage in Column CFCL9 at Selected Stages of Loading

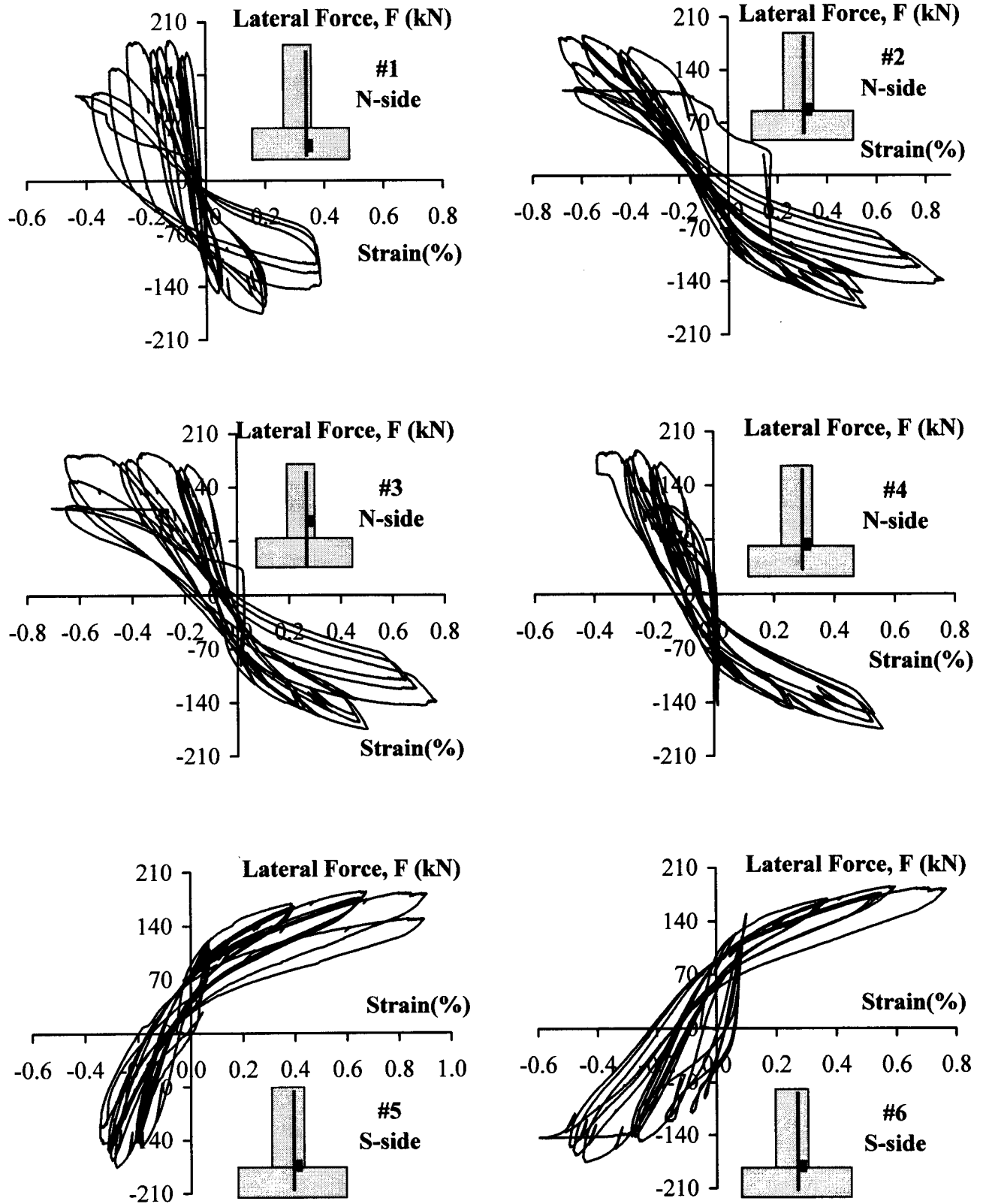


Figure 4.68: Bar Strain Readings in Column CFCL9



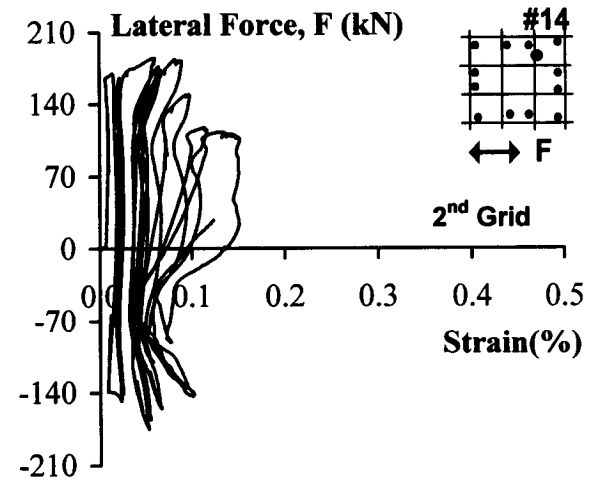
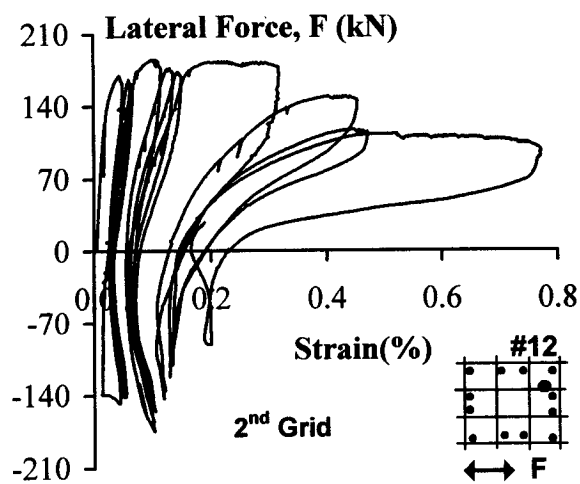
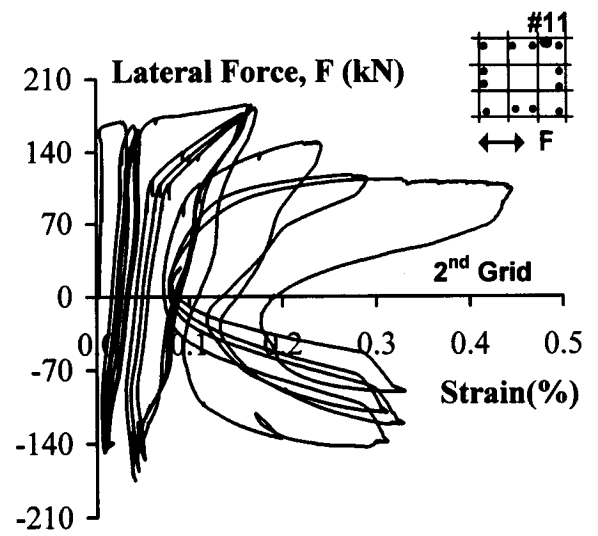
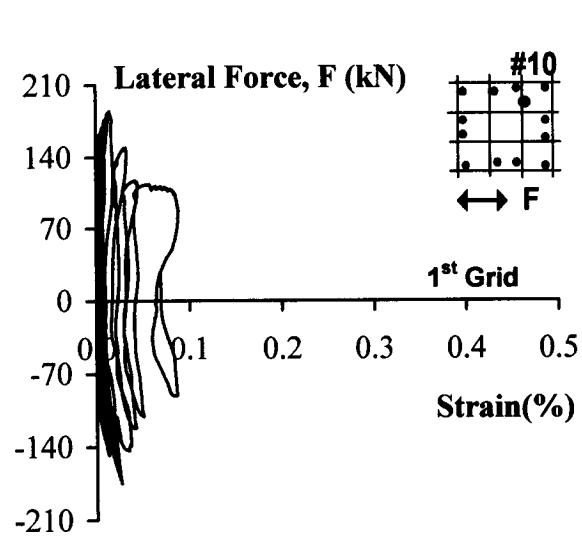
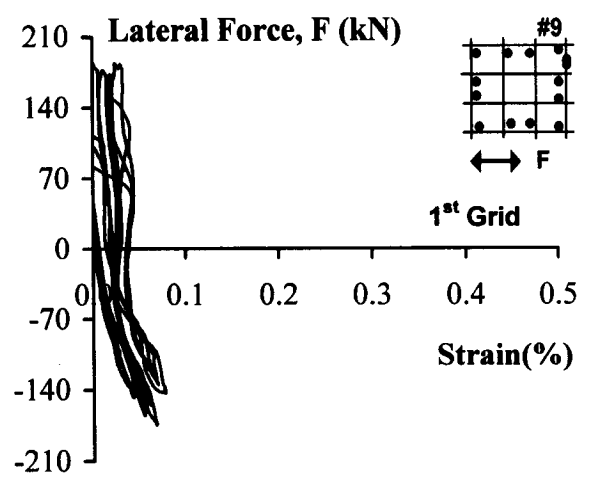
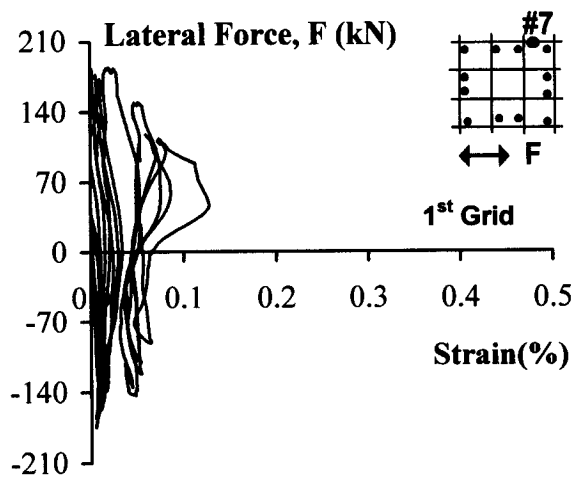


Figure 4.69: Grid Strain Readings in Column CFCL9

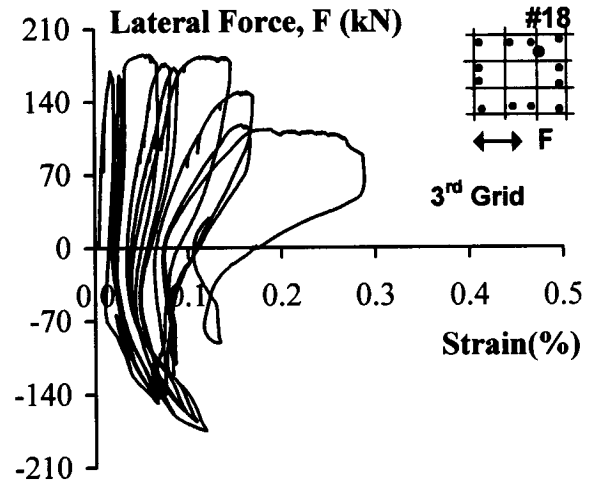
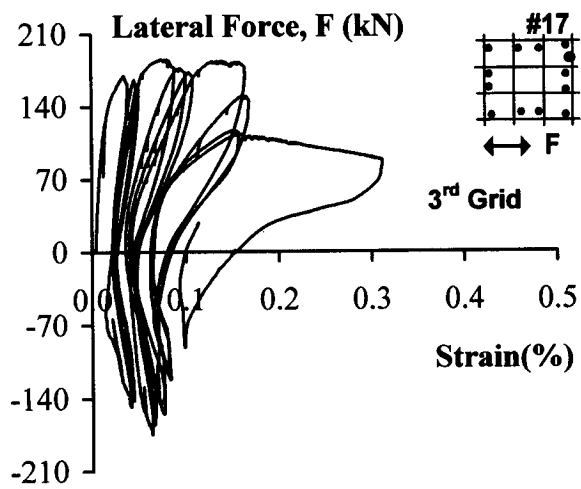
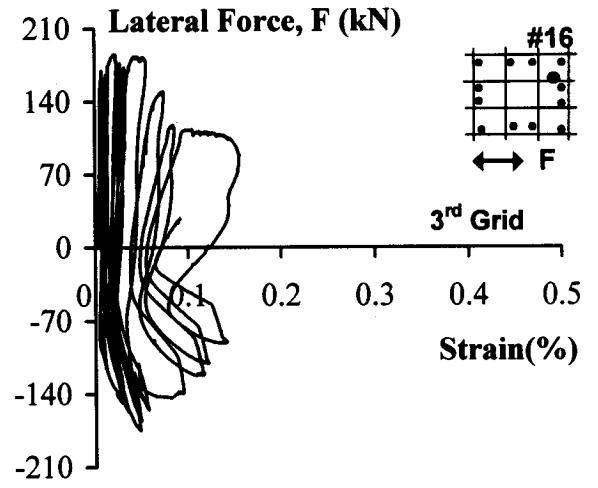
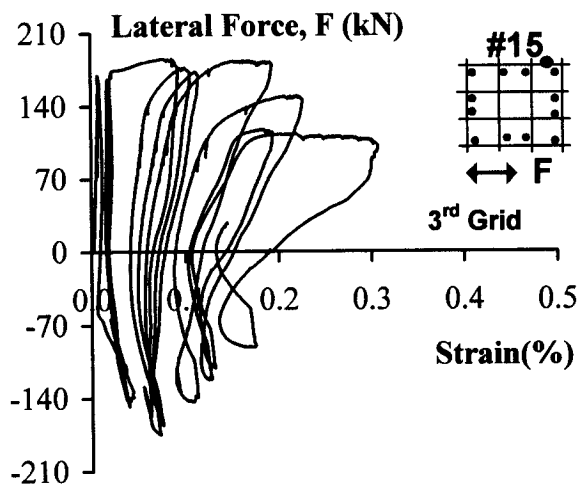
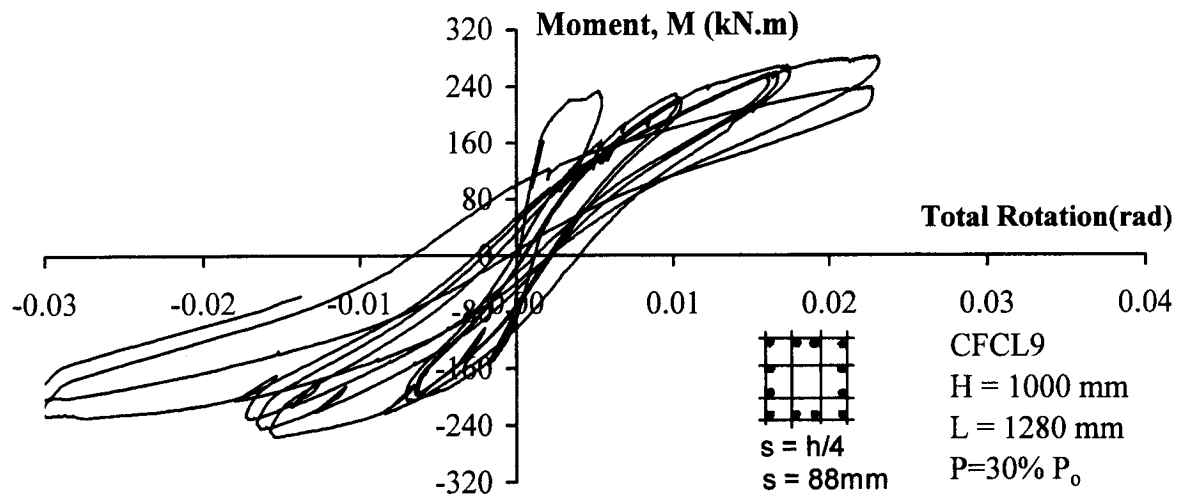
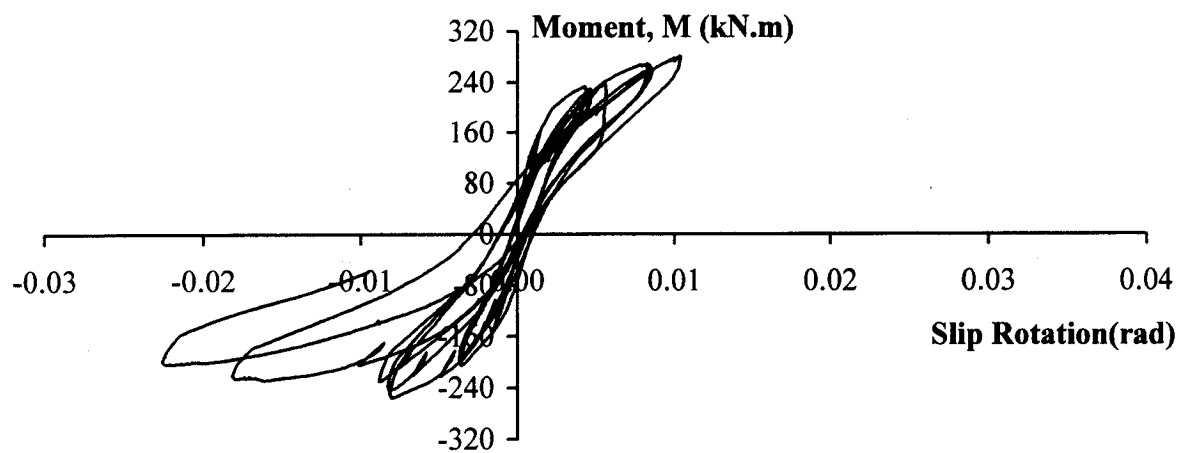


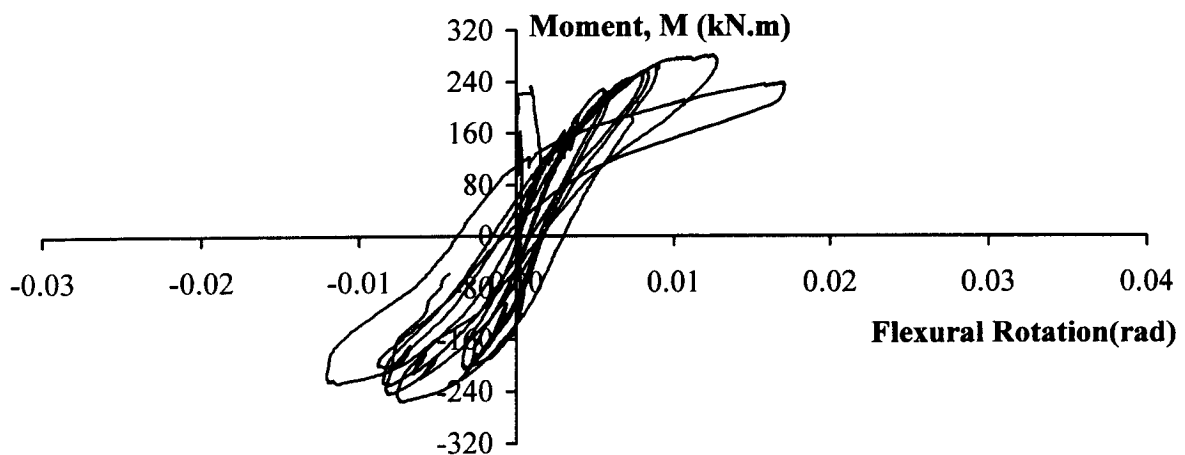
Figure 4.69 (Cont.): Grid Strain Readings in Column CFCL9



(a) Moment-Total Rotation Relationship

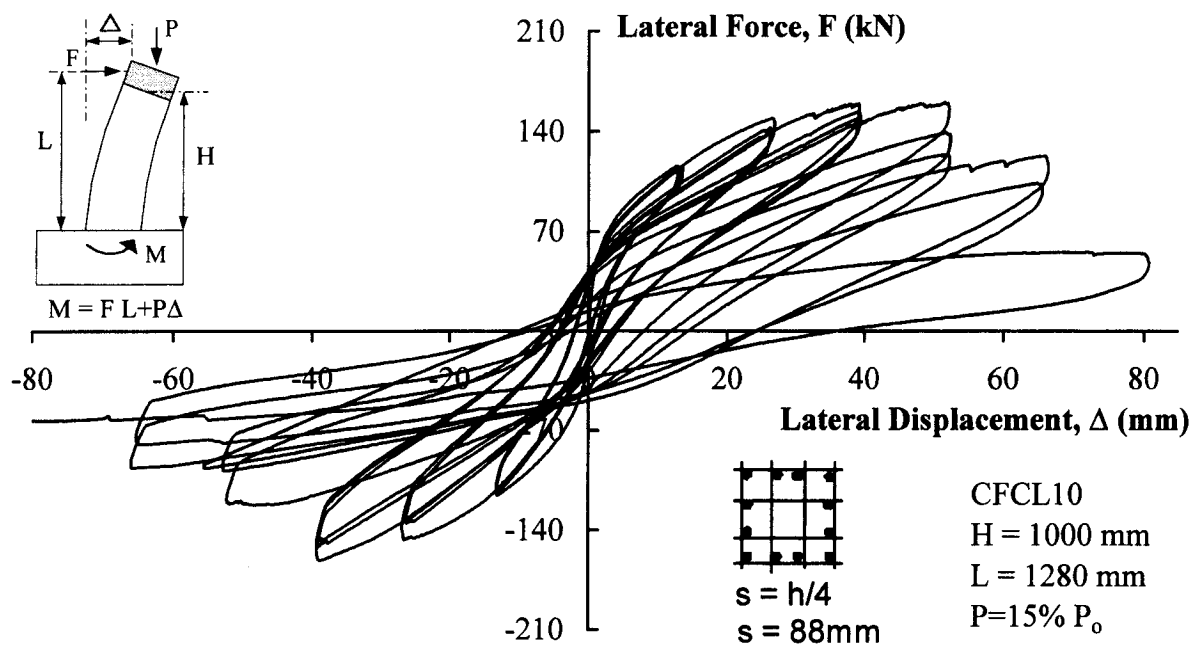


(b) Moment-Slip Rotation Relationship

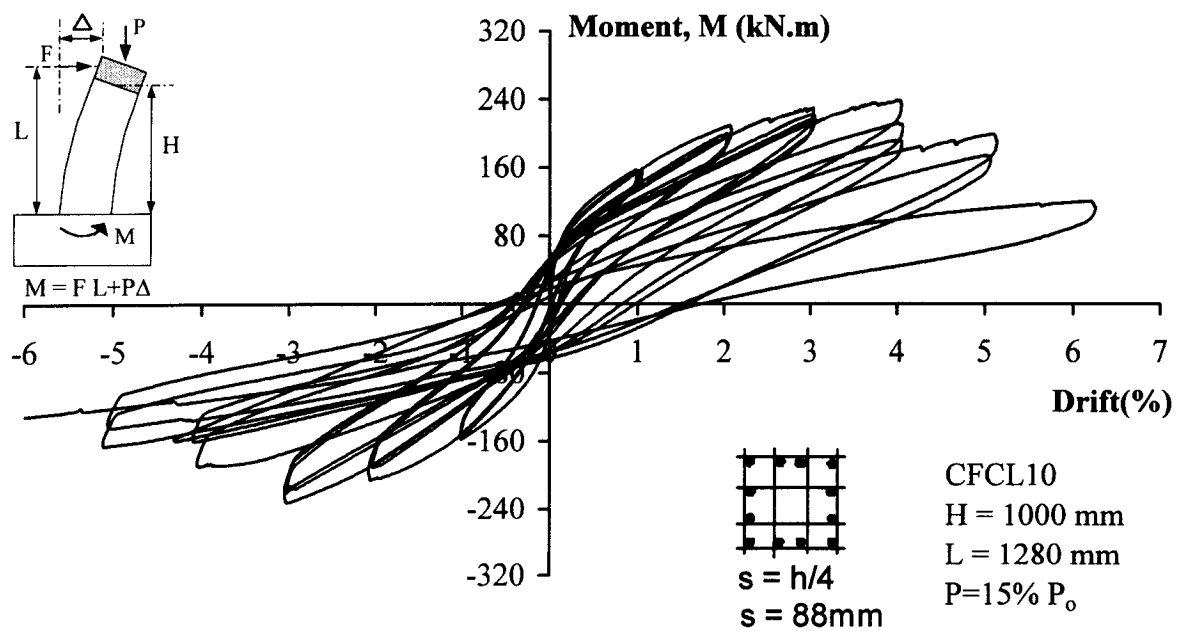


(c) Moment-Flexural Rotation Relationship

Figure 4.70: Hysteretic Moment-Rotation Relationships for Column CFCL9

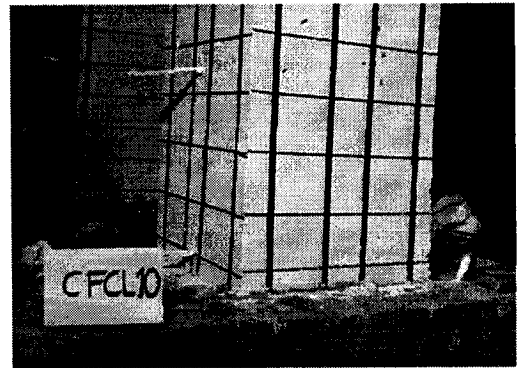
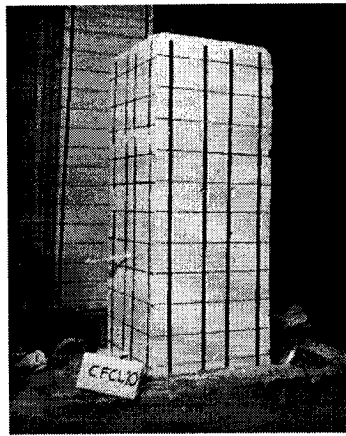


(a) Lateral Force-Lateral Displacement Relationship

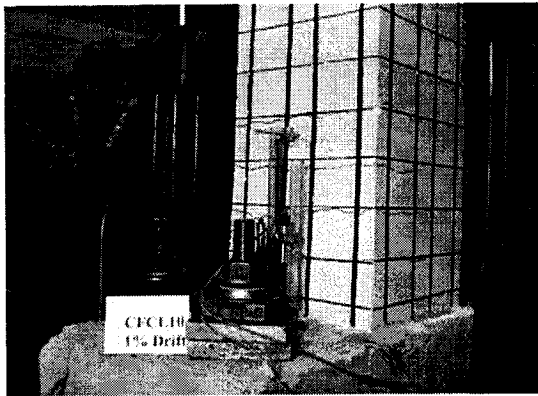


(b) Moment-Drift Relationship

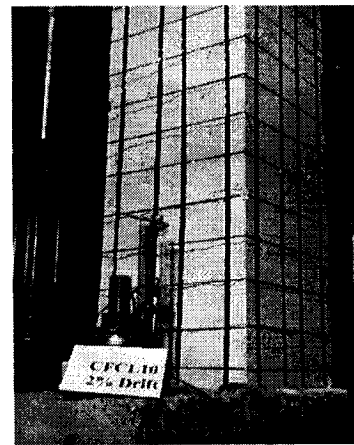
Figure 4.71: Hysteretic Behavior for Column CFCL10



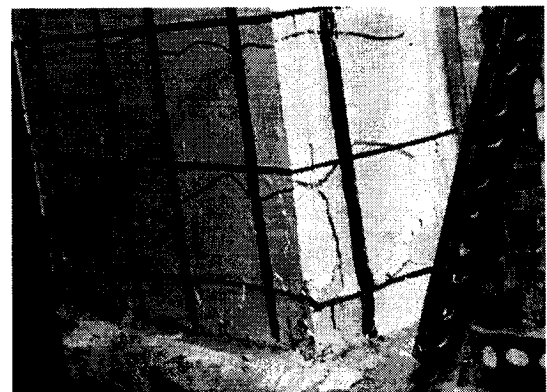
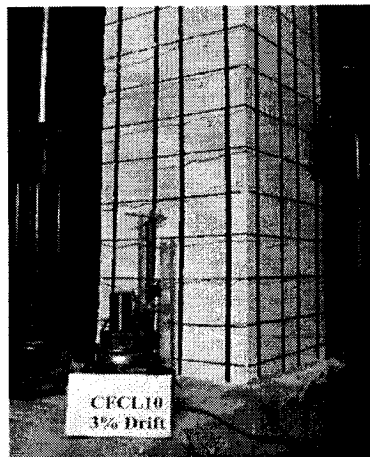
(a) Before Loading



(b) At 1% Drift



(c) At 2% Drift



(d) At 3% Drift

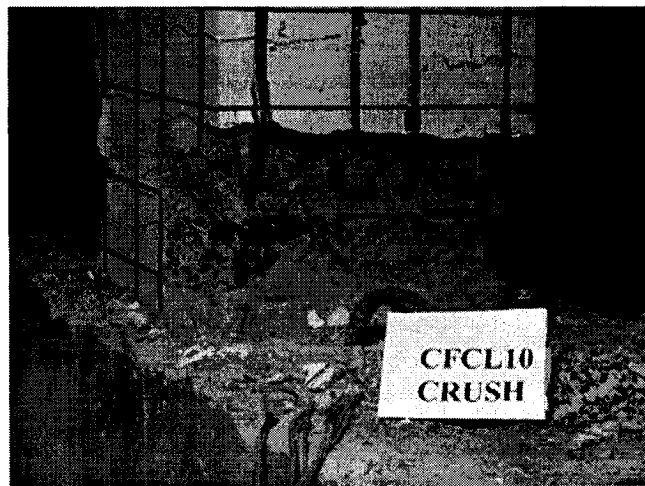
Figure 4.72: Observed Damage in Column CFCL10 at Selected Stages of Loading



(e) At 4% Drift



(f) At 5% Drift



(g) Column Failure



(h) Bar Buckling at Both Sides of the Column

Figure 4.72(Cont.): Observed Damage in Column CFCL10 at Selected Stages of Loading

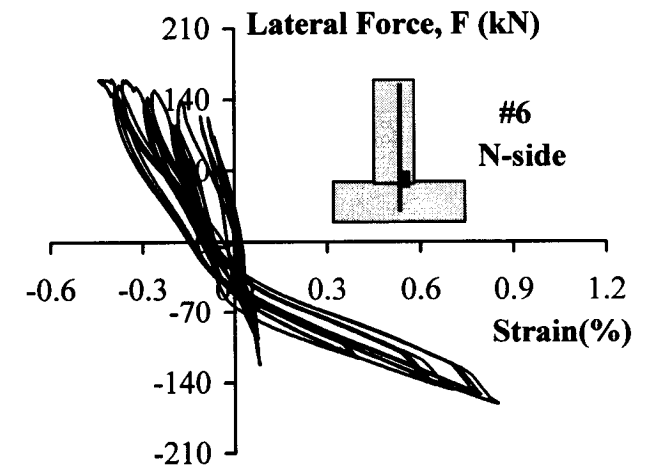
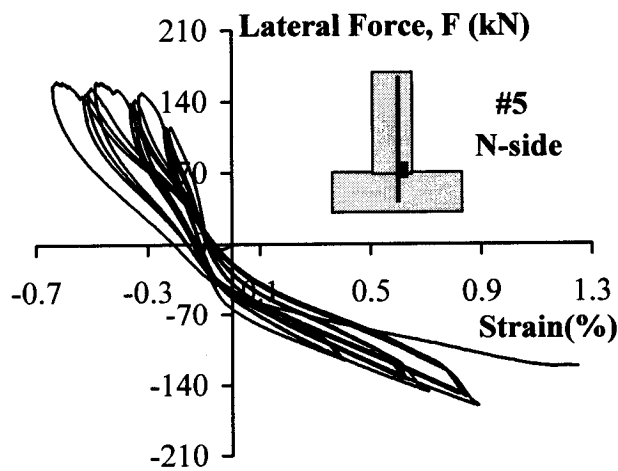
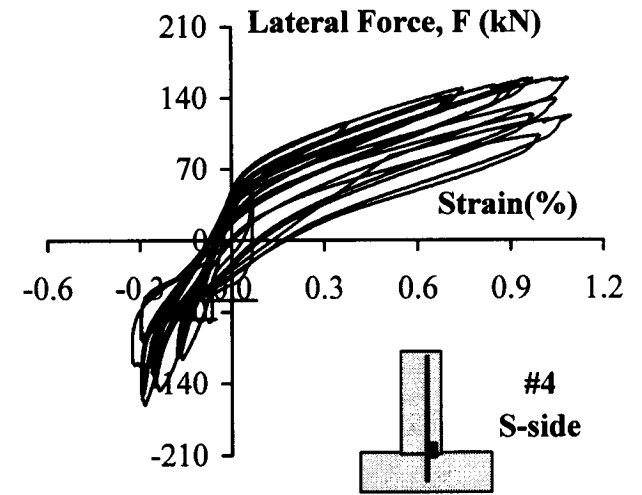
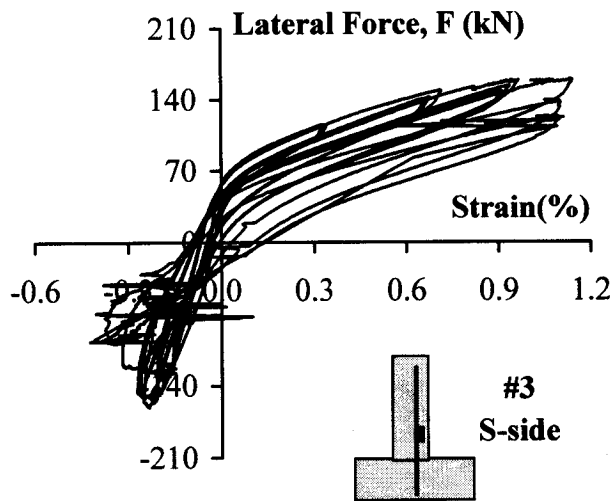
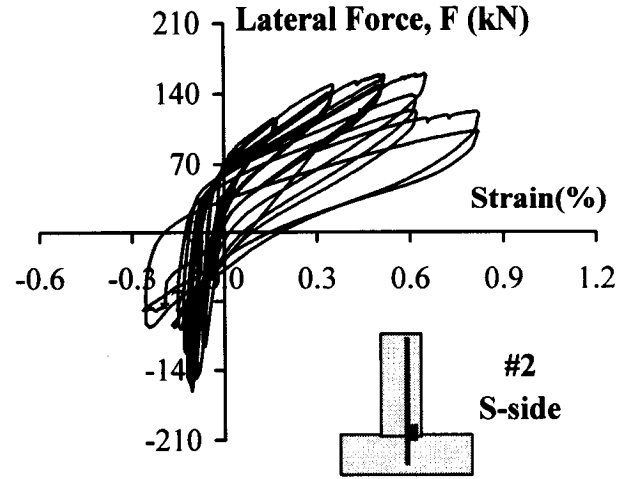
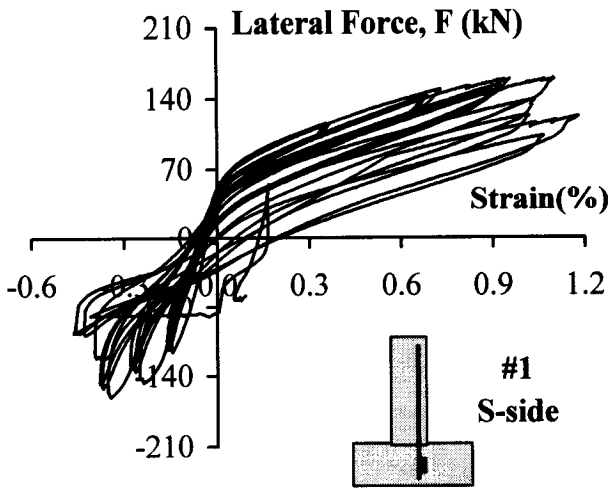


Figure 4.73: Bar Strain Readings in Column CFCL10

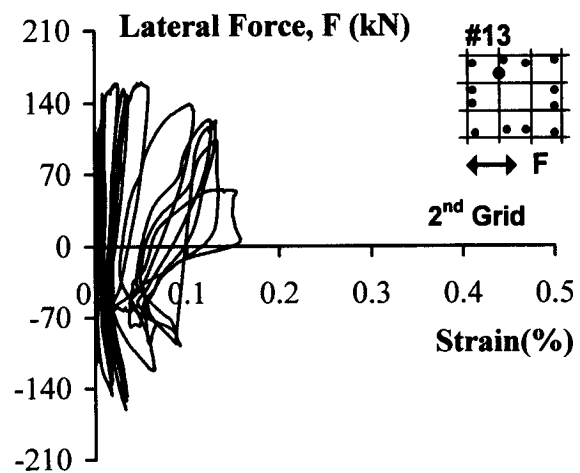
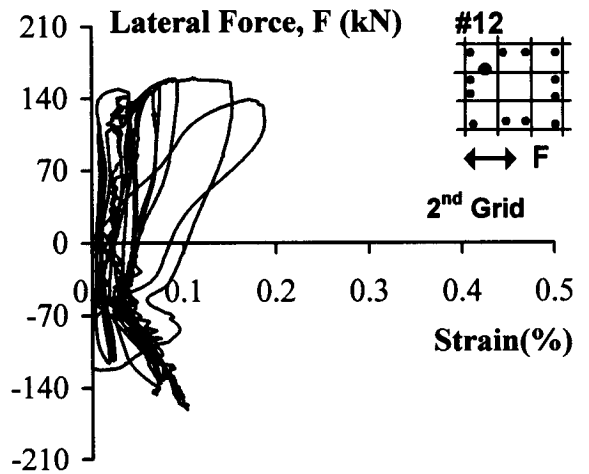
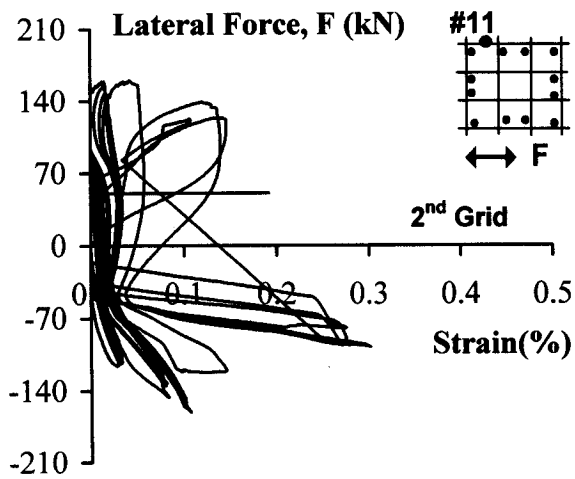
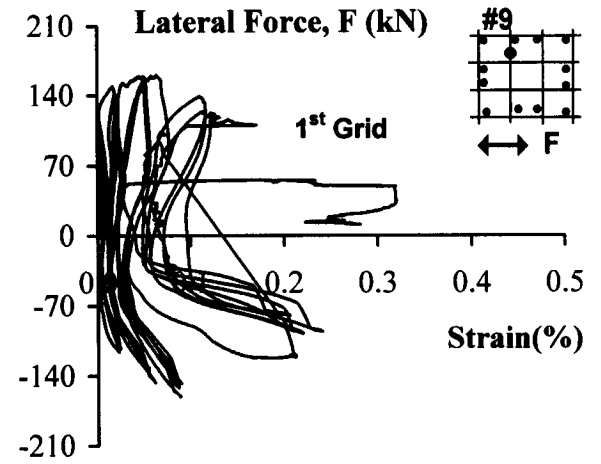
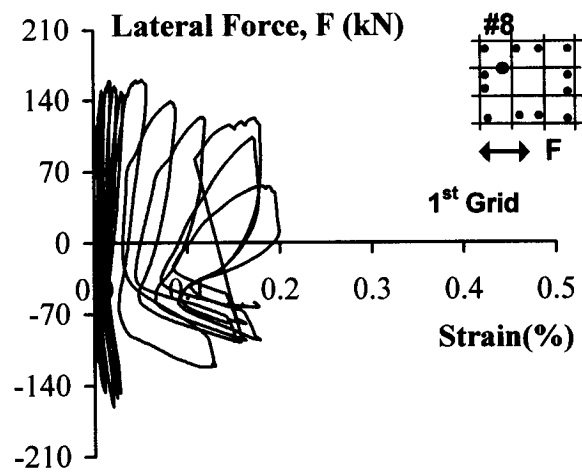


Figure 4.74: Grid Strain Readings in Column CFCL10



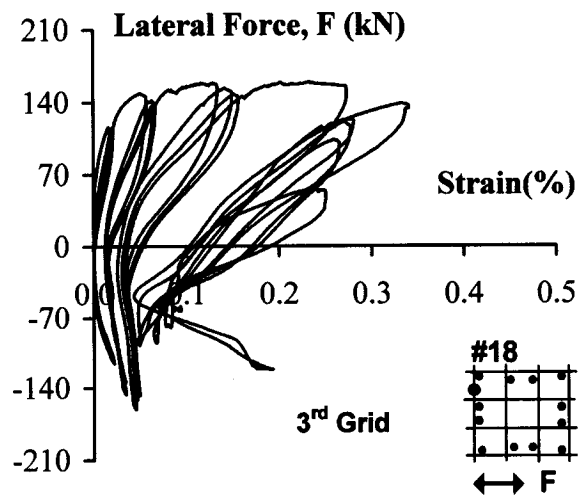
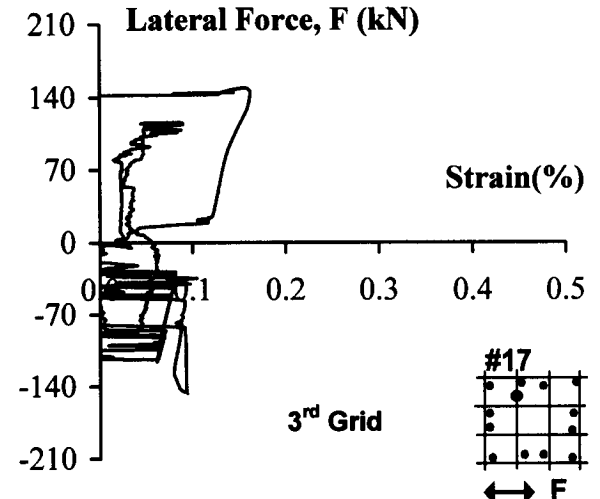
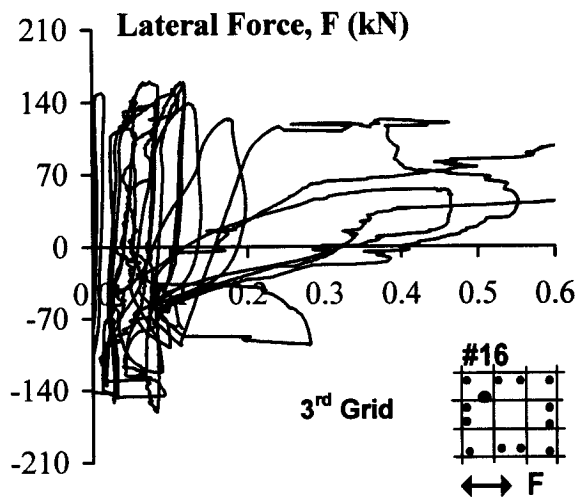
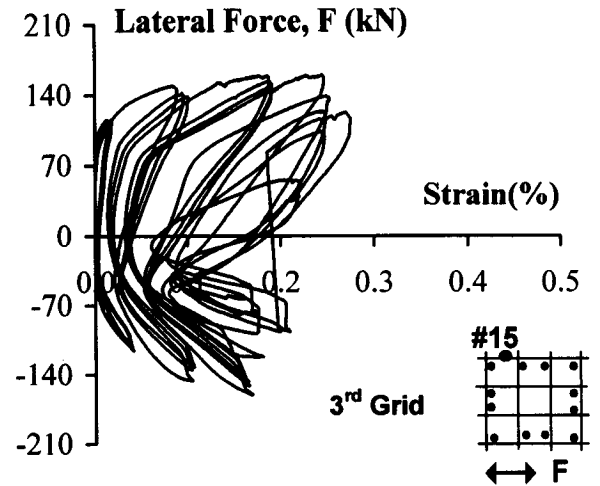
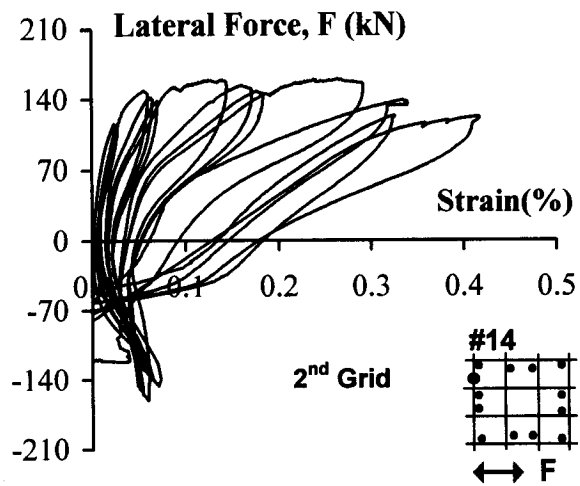
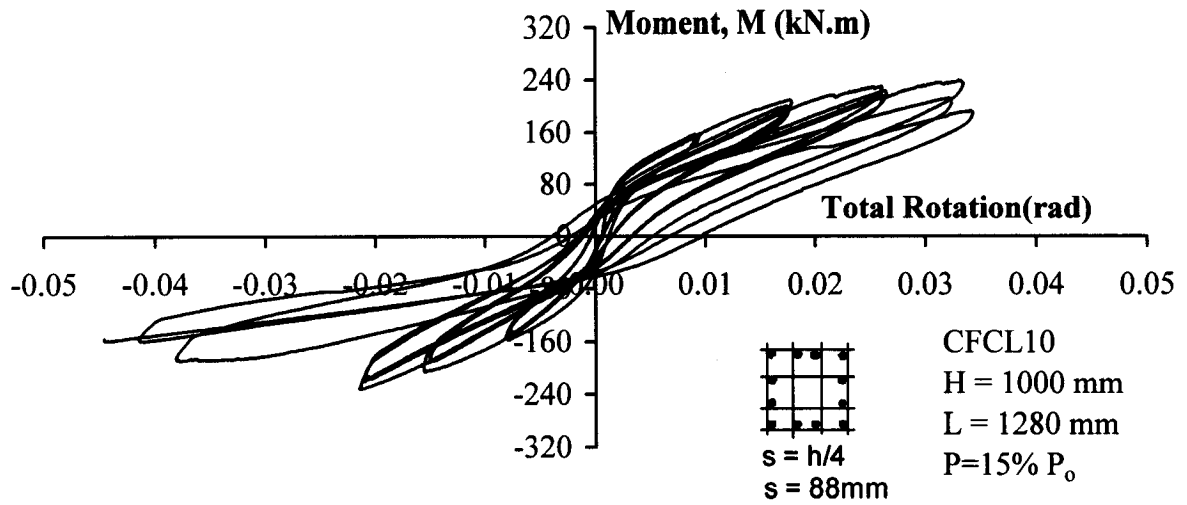
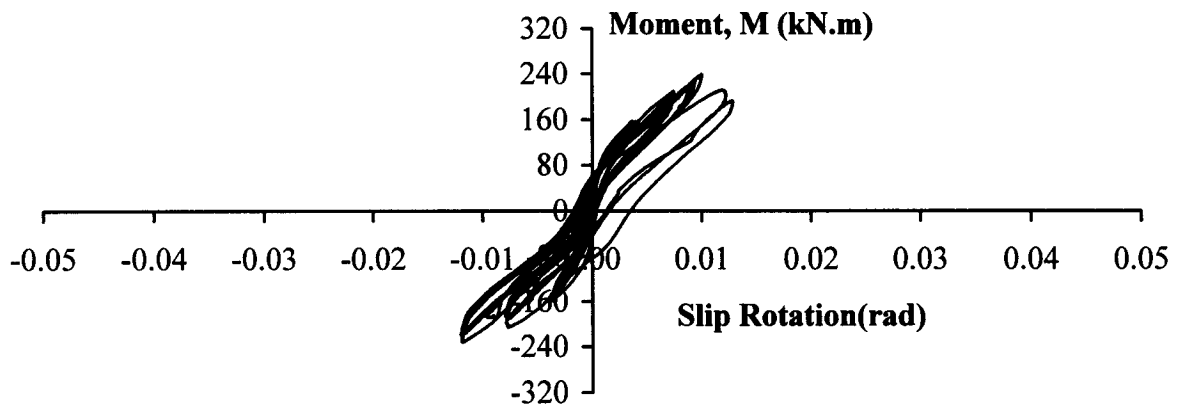


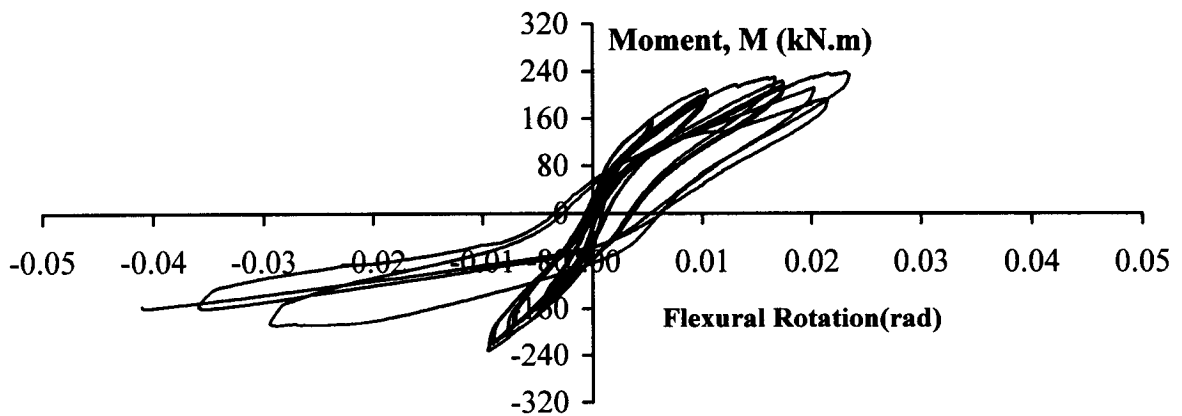
Figure 4.74 (Cont.) : Grid Strain Readings in Column CFCL10



(a) Moment-Total Rotation Relationship

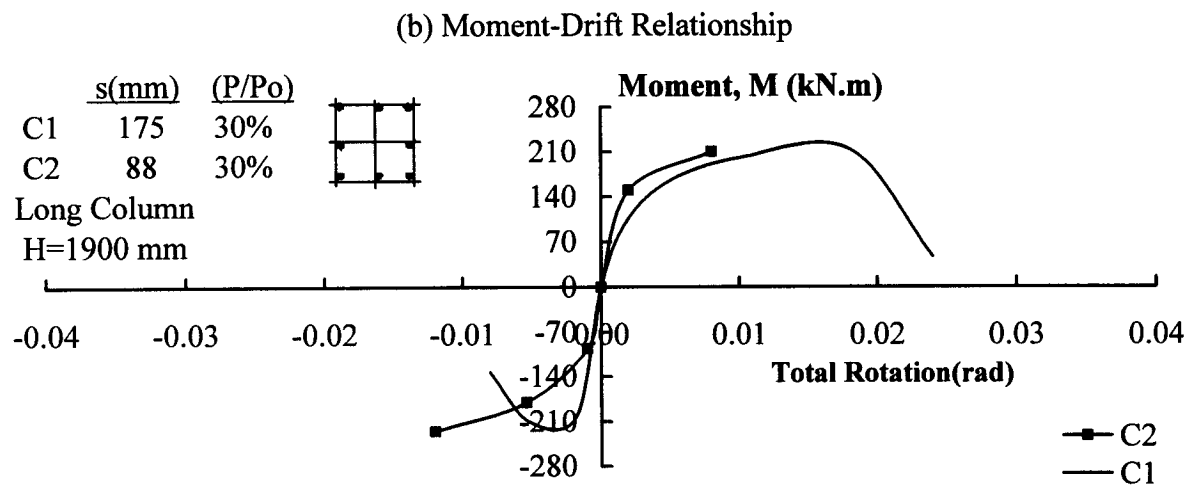
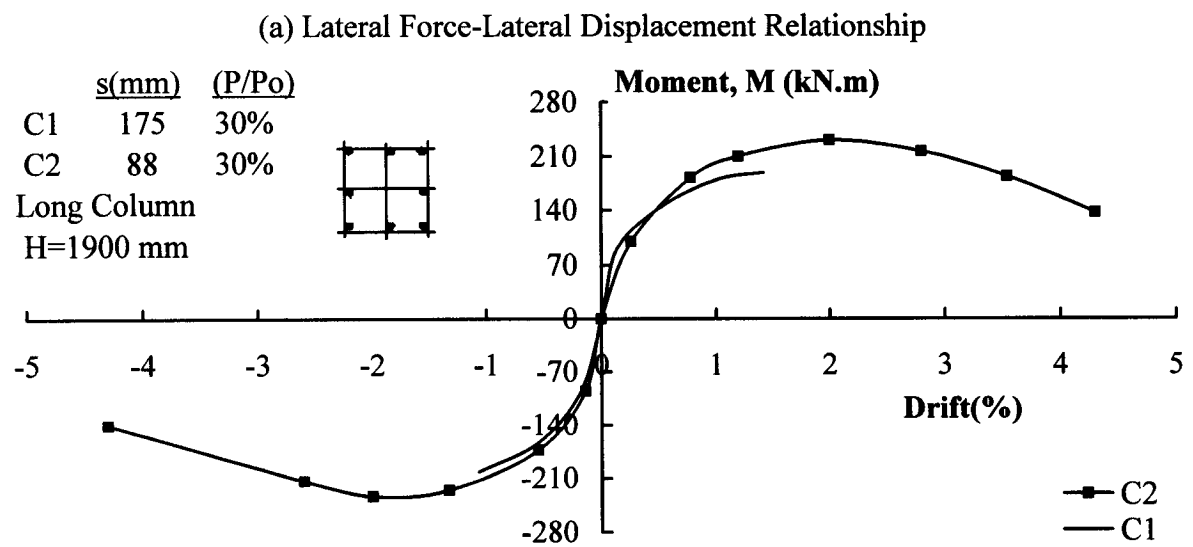
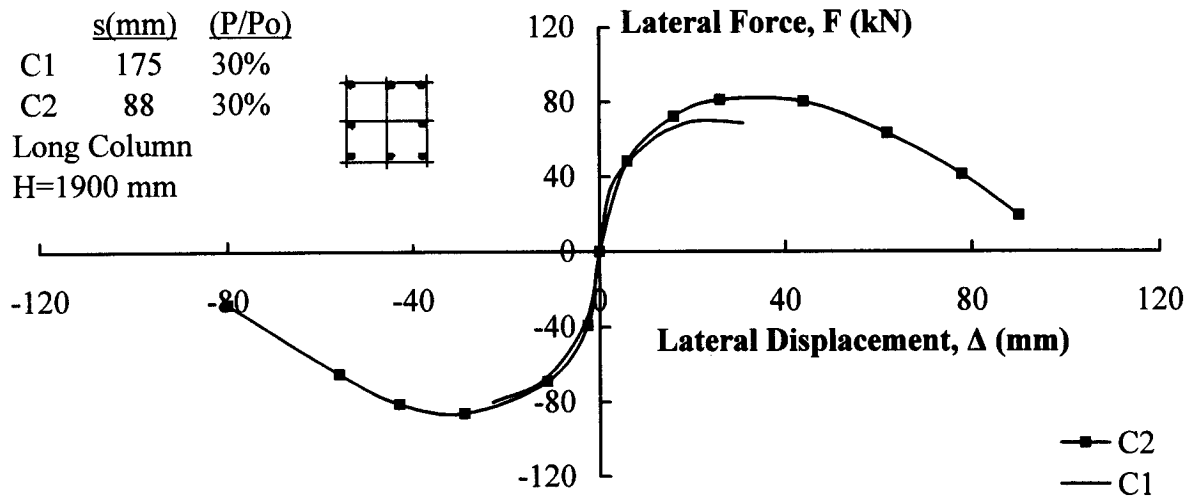


(b) Moment-Slip Rotation Relationship



(c) Moment-Flexural Rotation Relationship

Figure 4.75: Hysteretic Moment-Rotation Relationships for Column CFCL10



(c) Moment-Total Rotation Relationship

Figure 4.76: Comparison of Long Columns for the Effect of Spacing of Grids

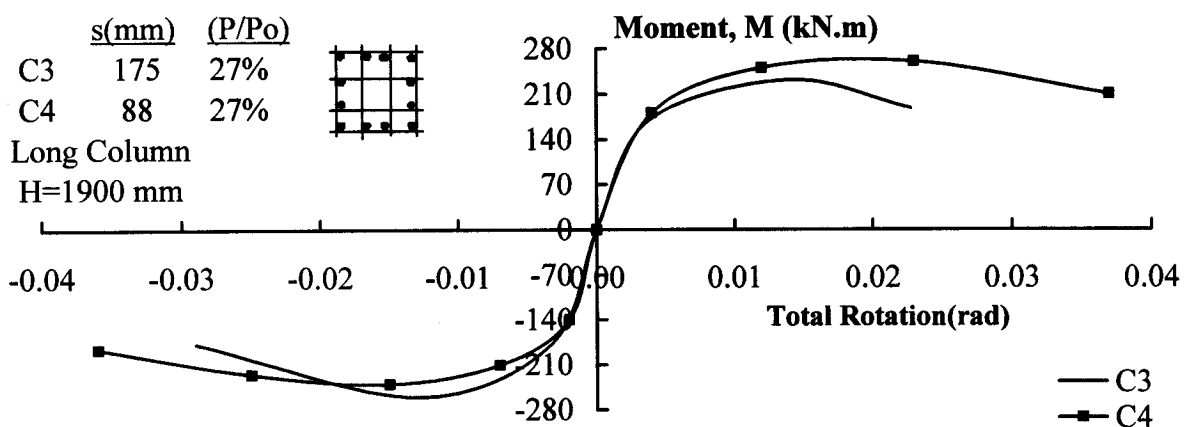
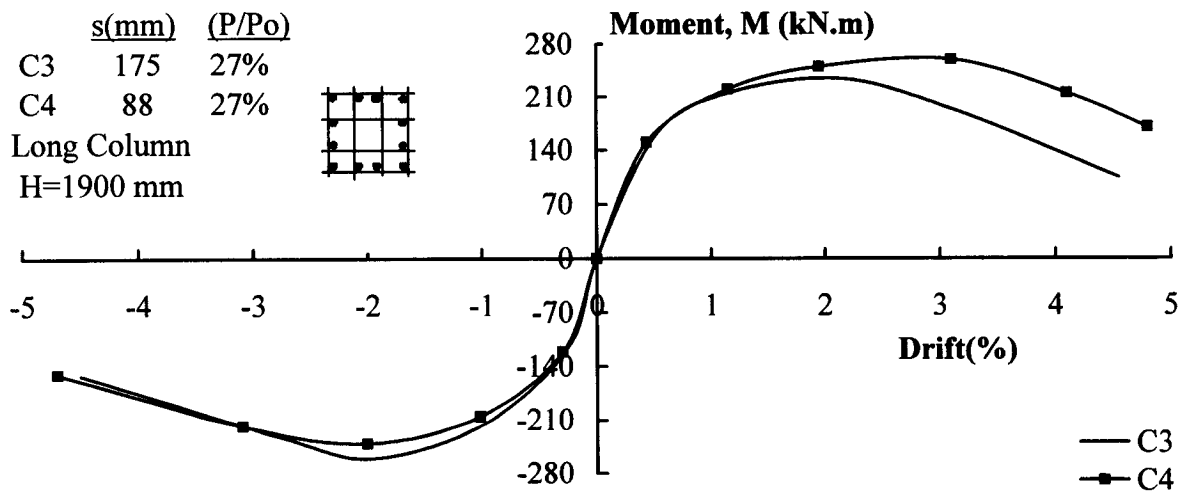
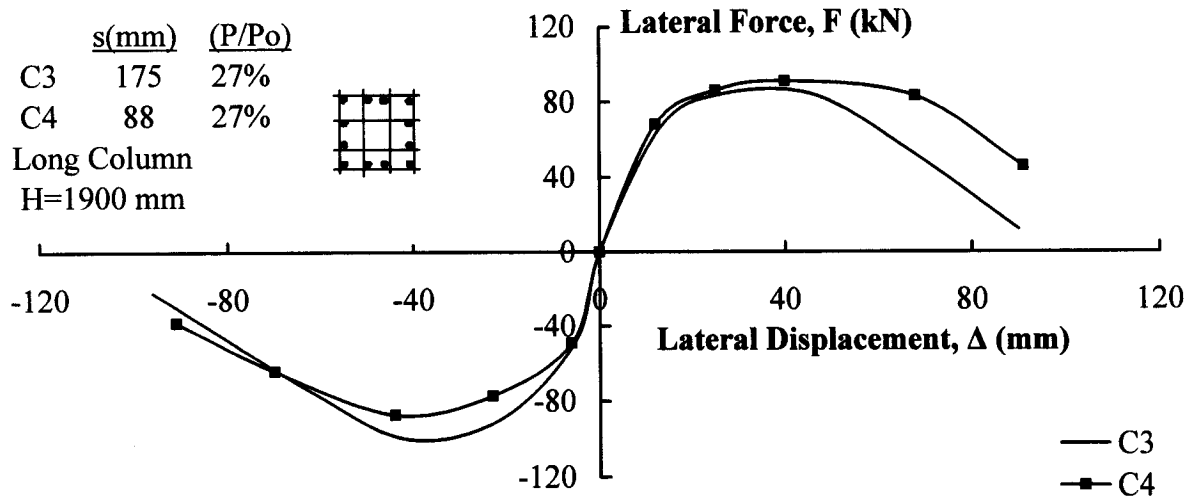
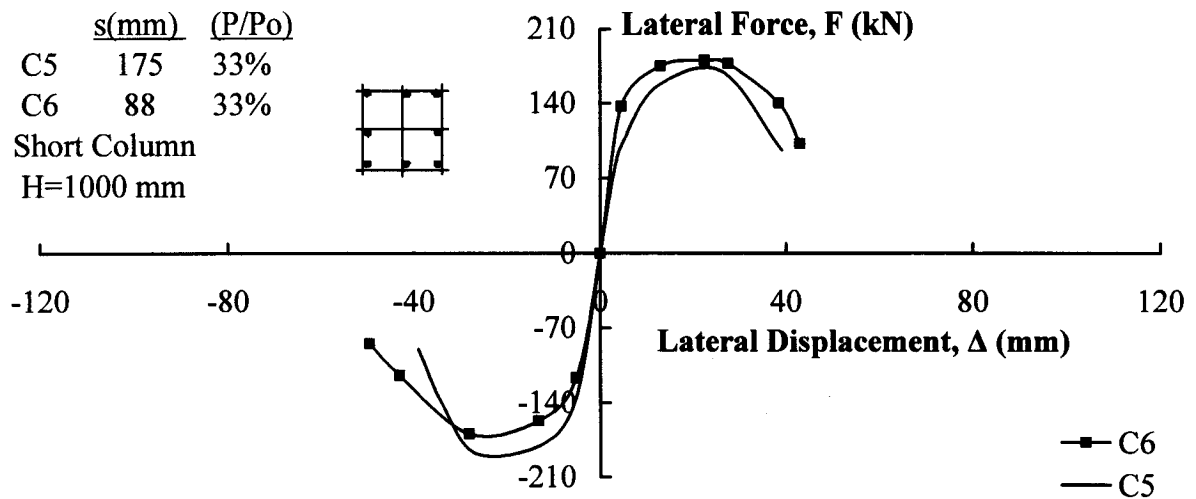
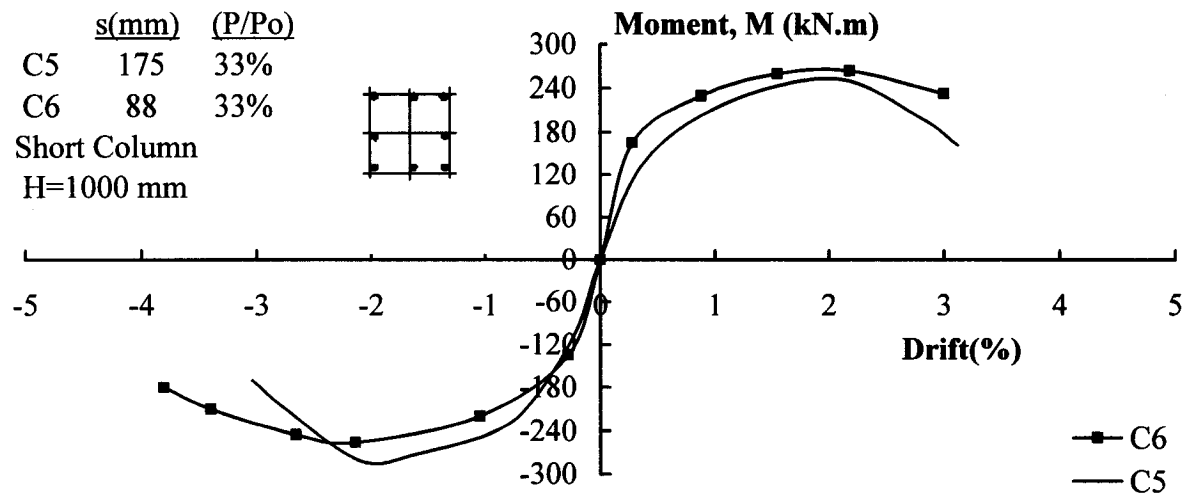


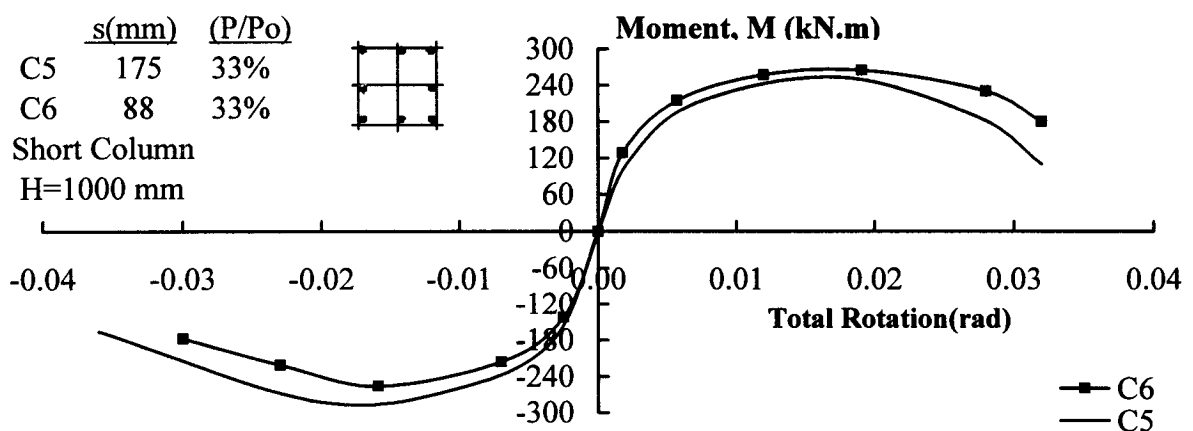
Figure 4.76(Cont): Comparison of Long Columns for the Effect of Spacing of Grids



(a) Lateral Force-Lateral Displacement Relationship

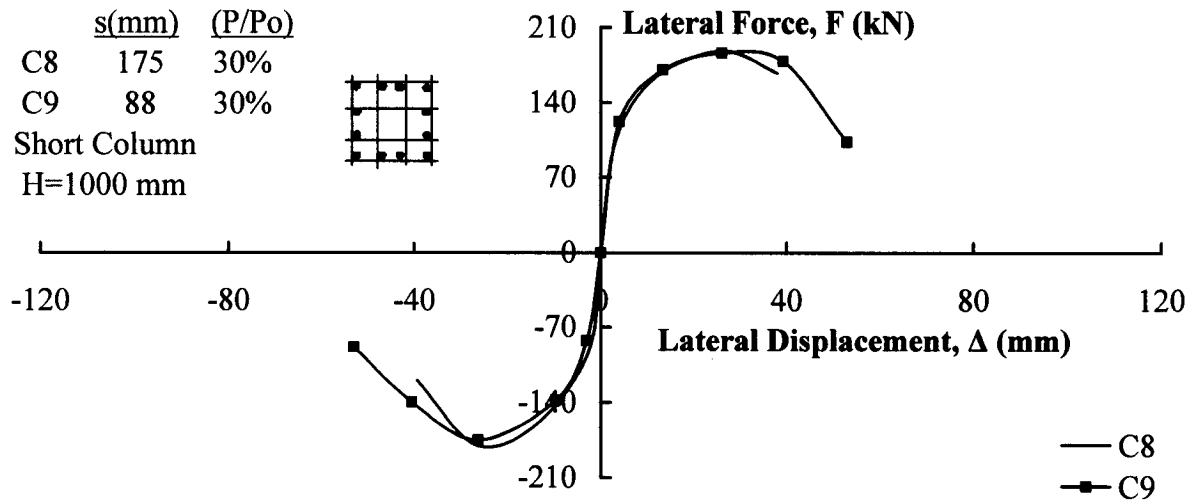


(b) Moment-Drift Relationship

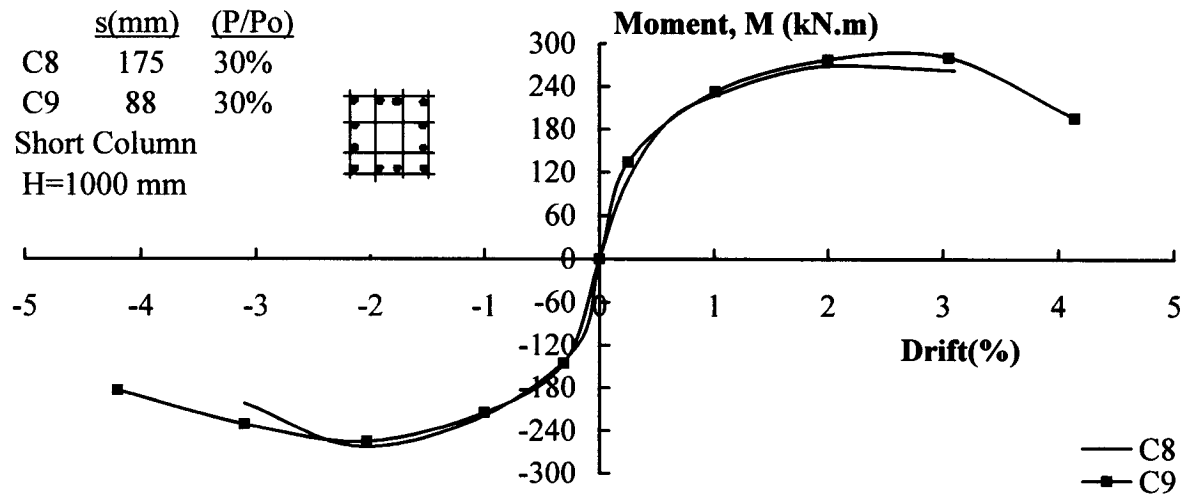


(c) Moment-Total Rotation Relationship

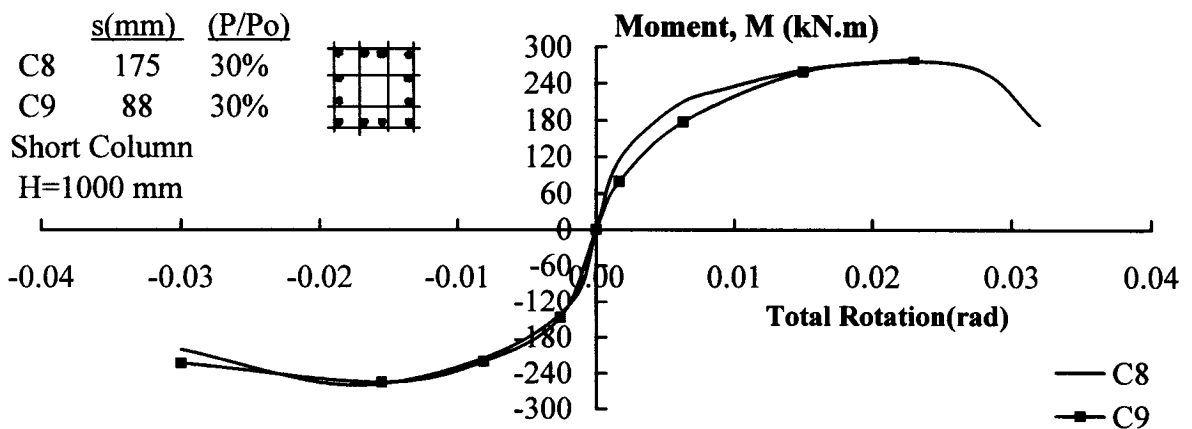
Figure 4.77: Comparison of Short Columns for the Effect of Spacing of Grids



(d) Lateral Force-Lateral Displacement Relationship

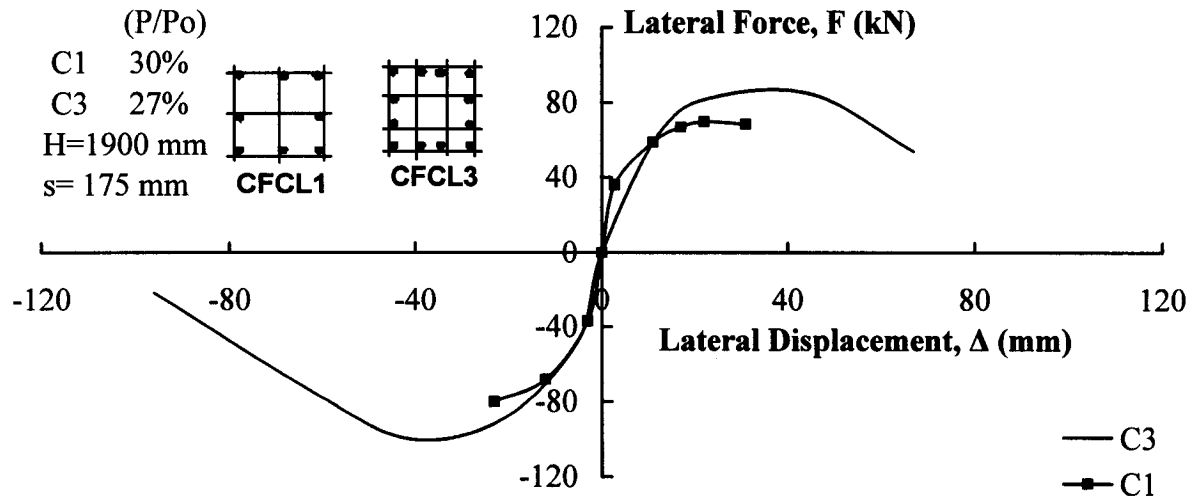


(e) Moment-Drift Relationship

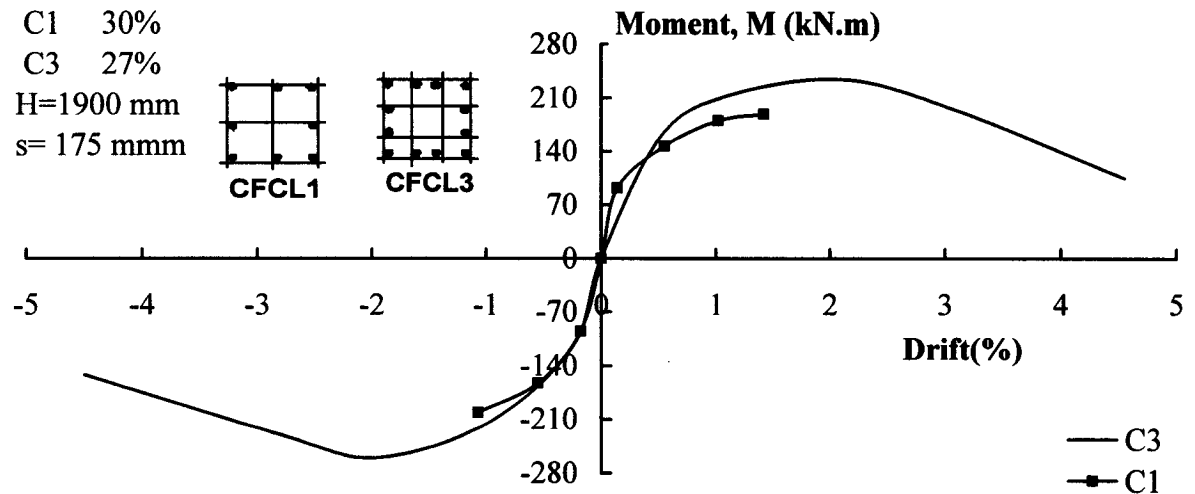


(f) Moment-Total Rotation Relationship

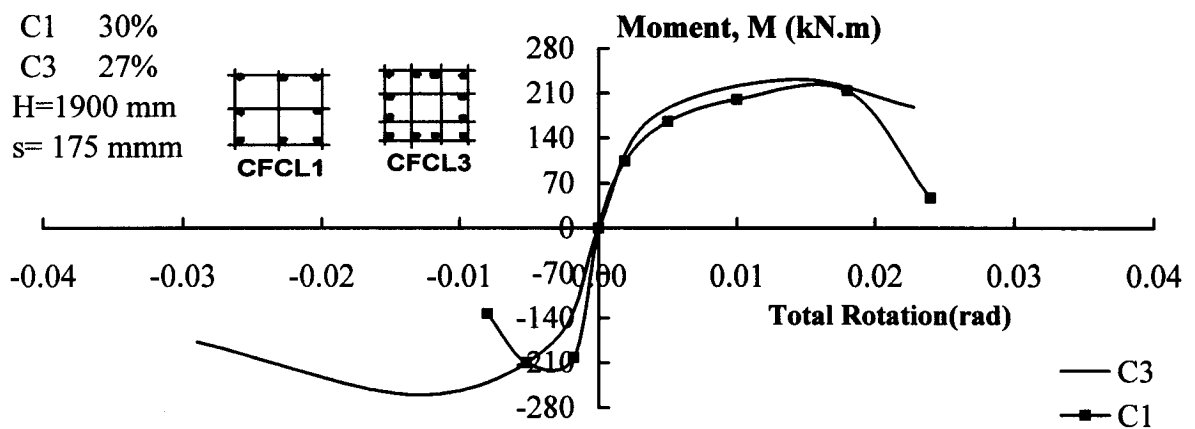
Figure 4.77(Con): Comparison of Short Columns for the Effect of Spacing of Grids



(a) Lateral Force-Lateral Displacement Relationship

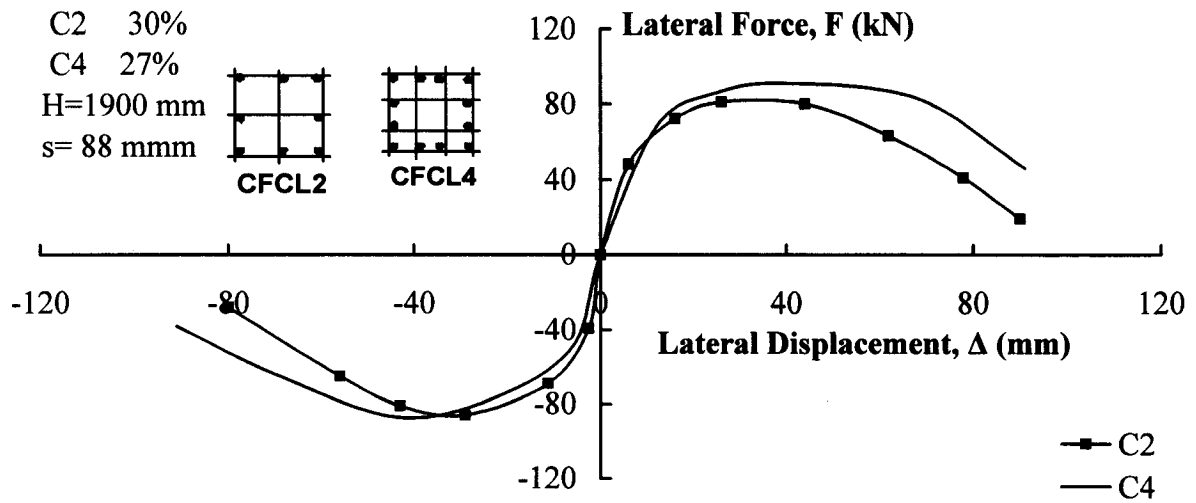


(b) Moment-Drift Relationship

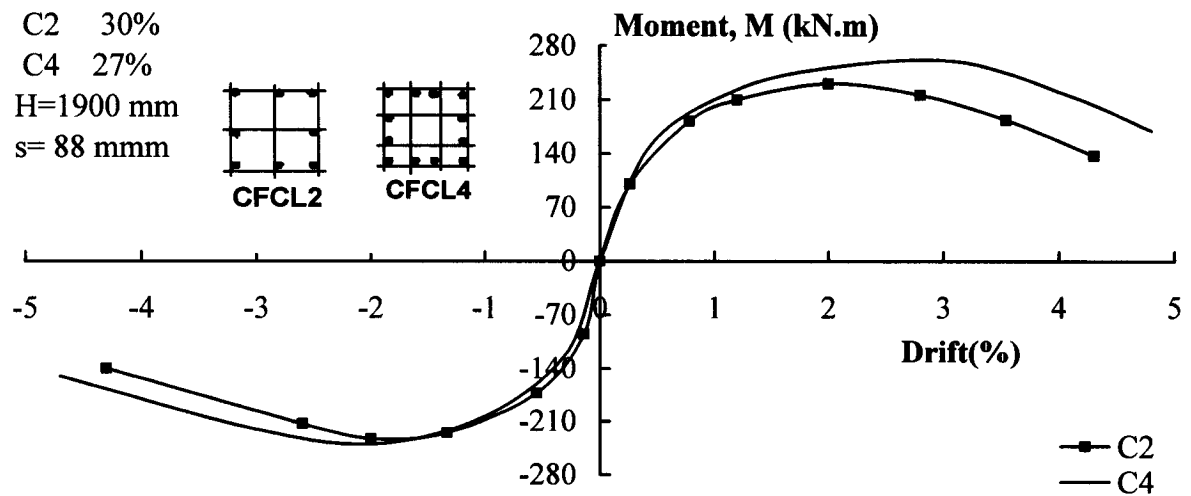


(c) Moment-Total Rotation Relationship

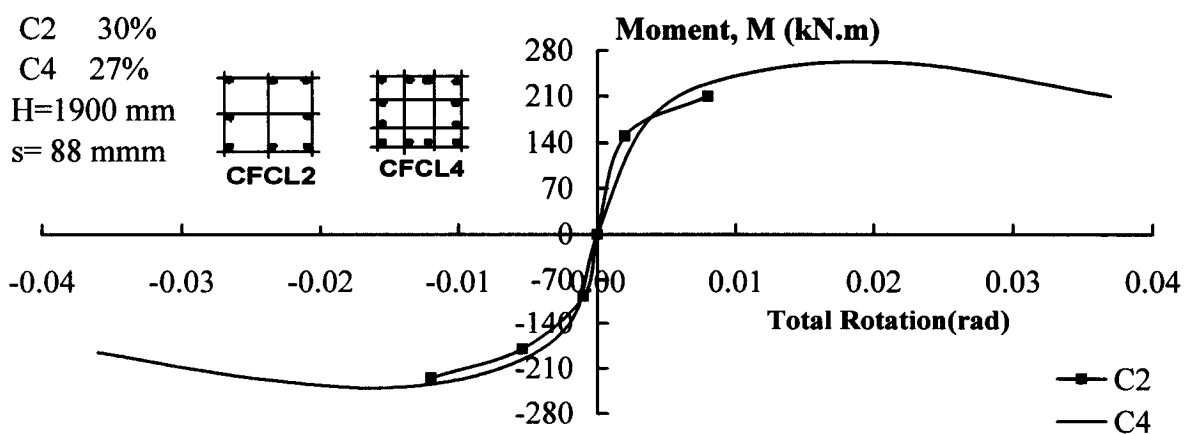
Figure 4.78: Comparison of Long Columns for the Effect of Arrangement of FRP Bars



(d) Lateral Force-Lateral Displacement Relationship



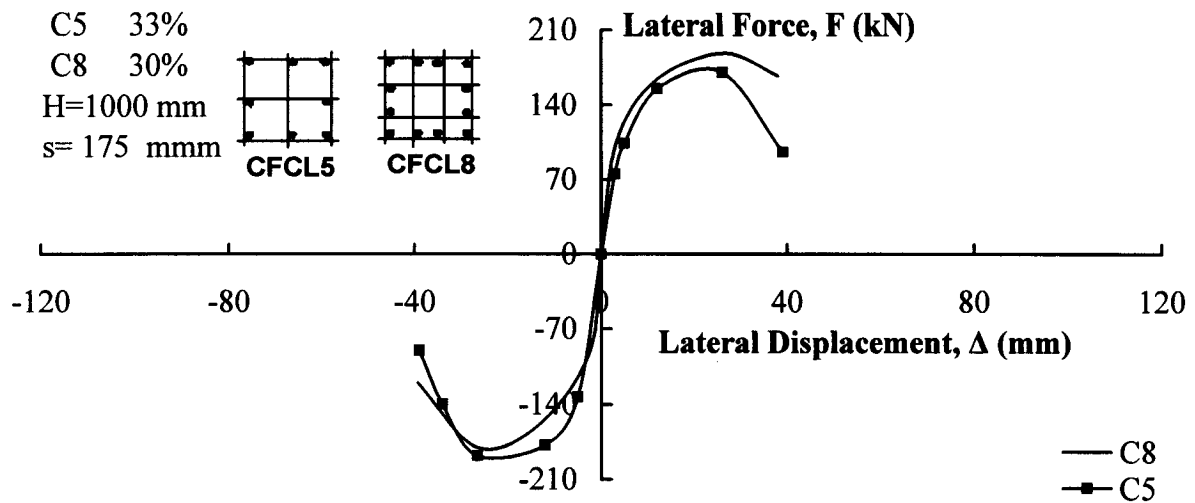
(e) Moment-Drift Relationship



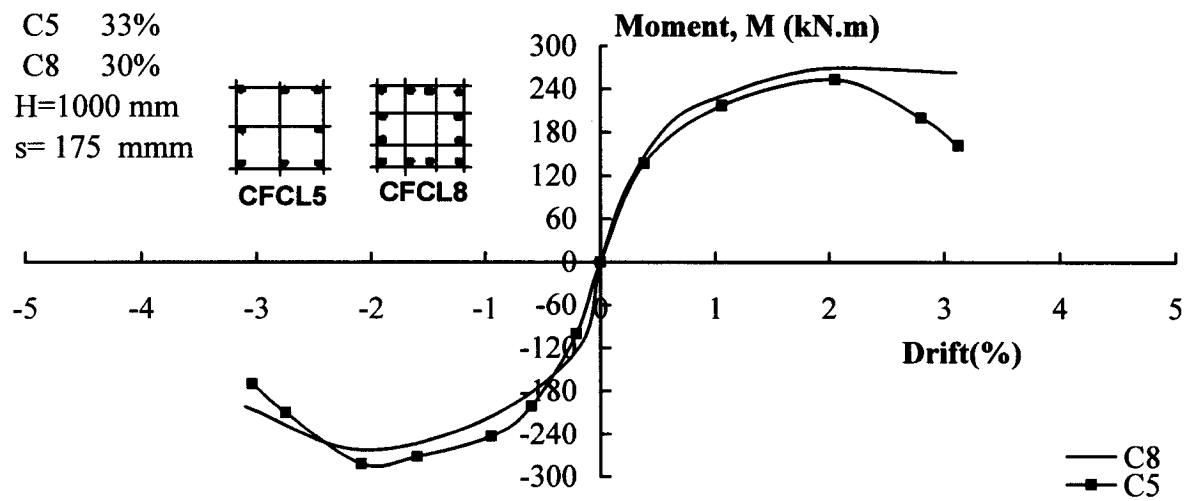
(f) Moment-Total Rotation Relationship

Figure 4.78(C.): Comparison of Long Columns for the Effect of Arrangement of FRP Bars

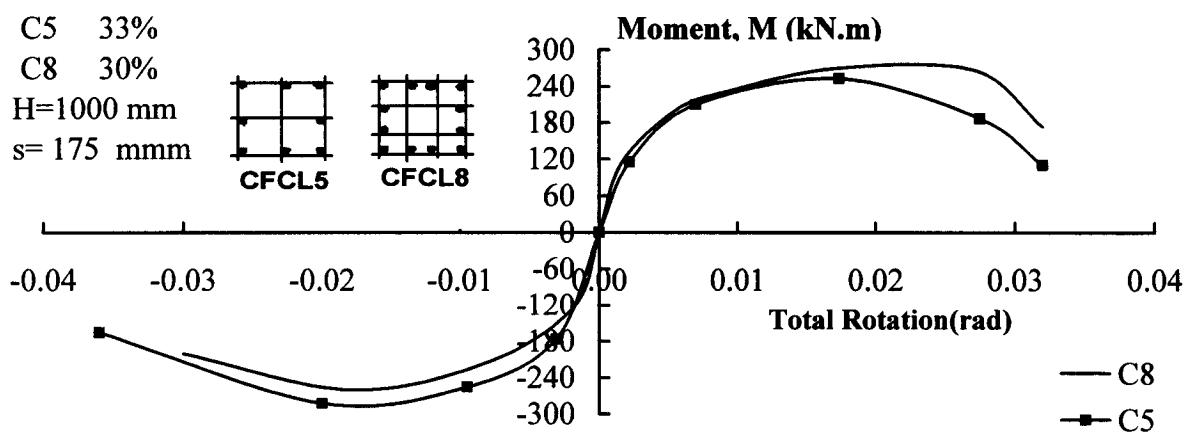




(a) Lateral Force-Lateral Displacement Relationship

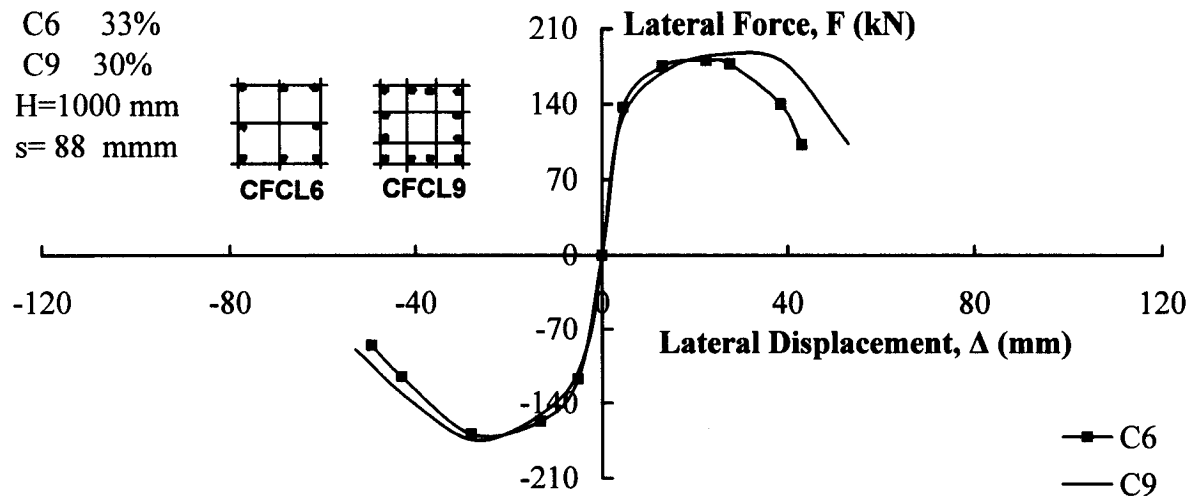


(b) Moment-Drift Relationship

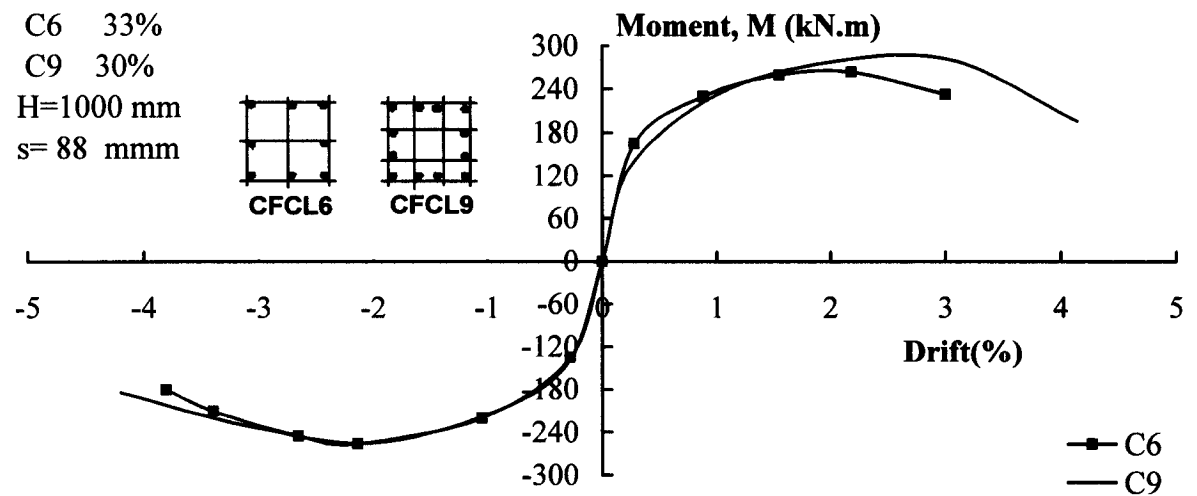


(c) Moment-Total Rotation Relationship

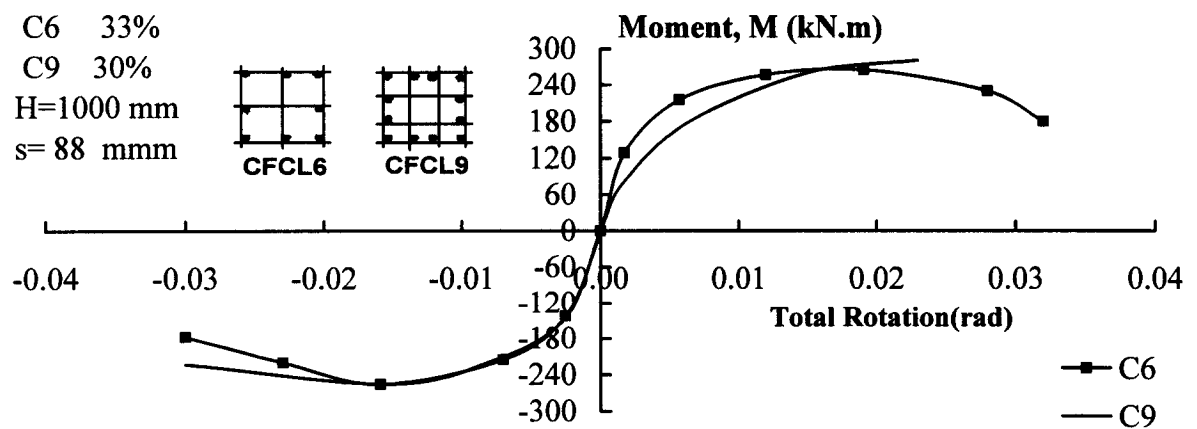
Figure 4.79: Comparison of Short Columns for the Effect of Arrangement of FRP Bars



(d) Lateral Force-Lateral Displacement Relationship

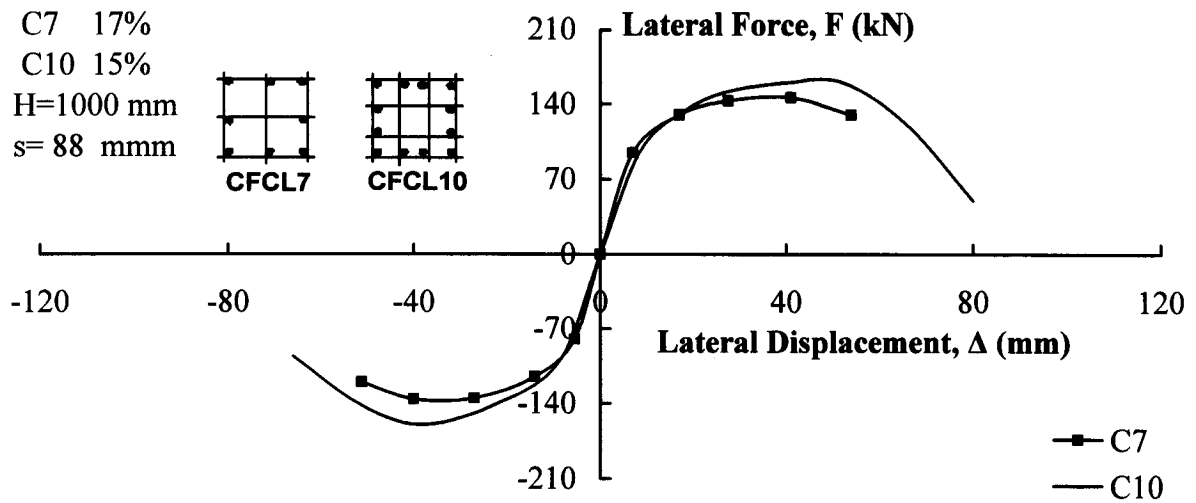


(e) Moment-Drift Relationship

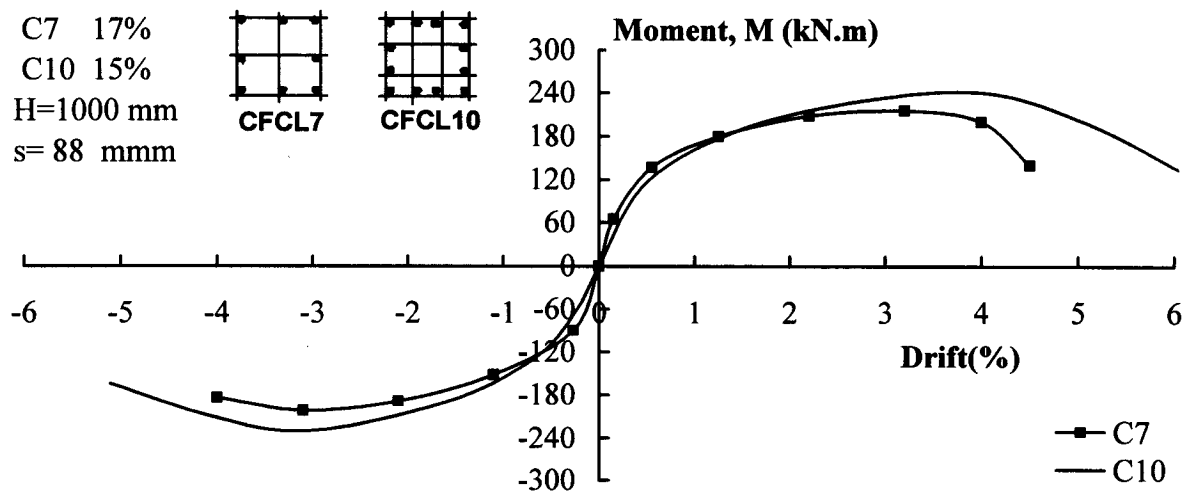


(f) Moment-Total Rotation Relationship

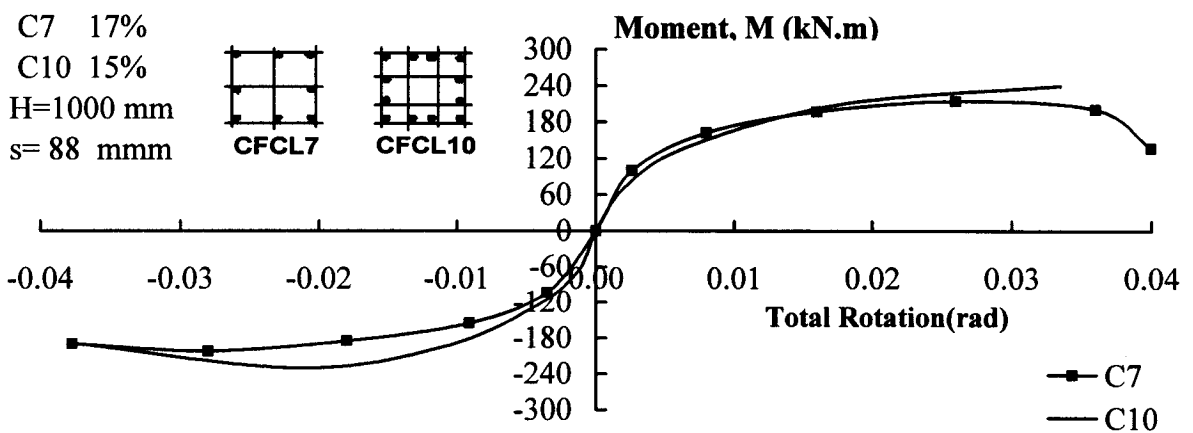
Figure 4.79 (Cont.): Comparison of Short Columns for the Effect of Arrangement of FRP Bars



(g) Lateral Force-Lateral Displacement Relationship



(h) Moment-Drift Relationship



(i) Moment-Total Rotation Relationship

Figure 4.79 (Cont.): Comparison of Short Columns for the Effect of Arrangement of FRP Bars

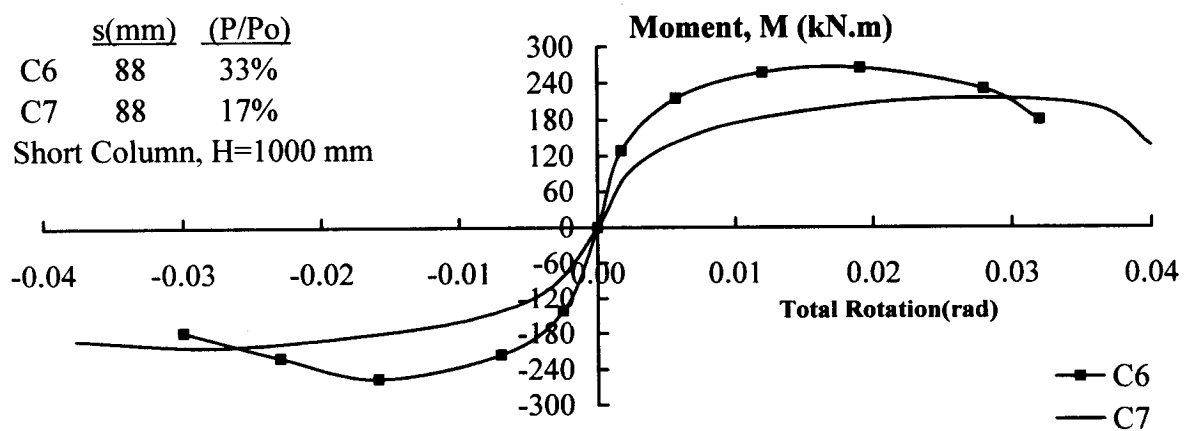
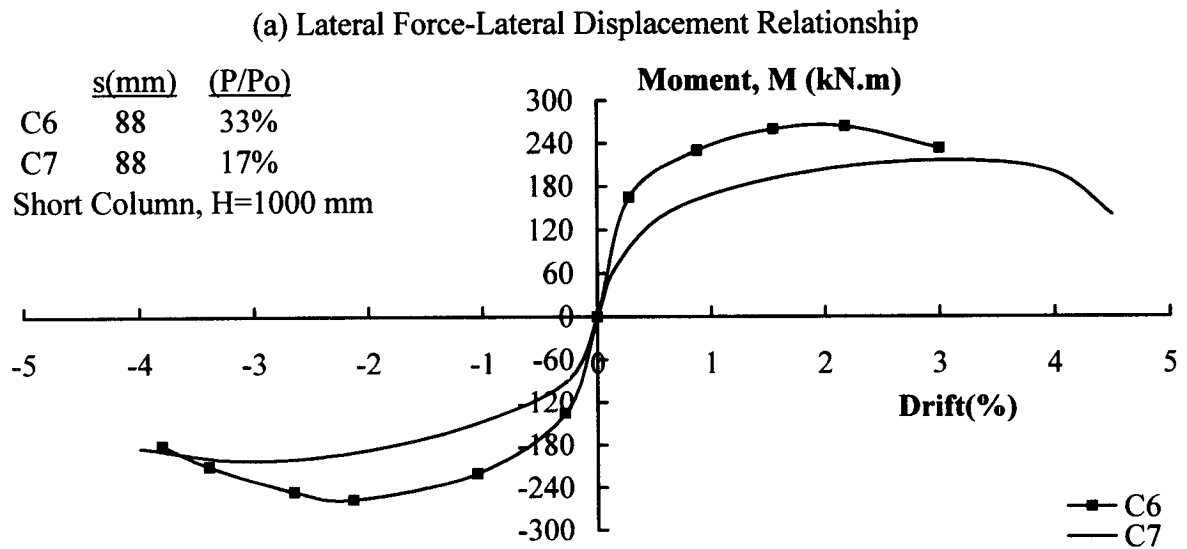
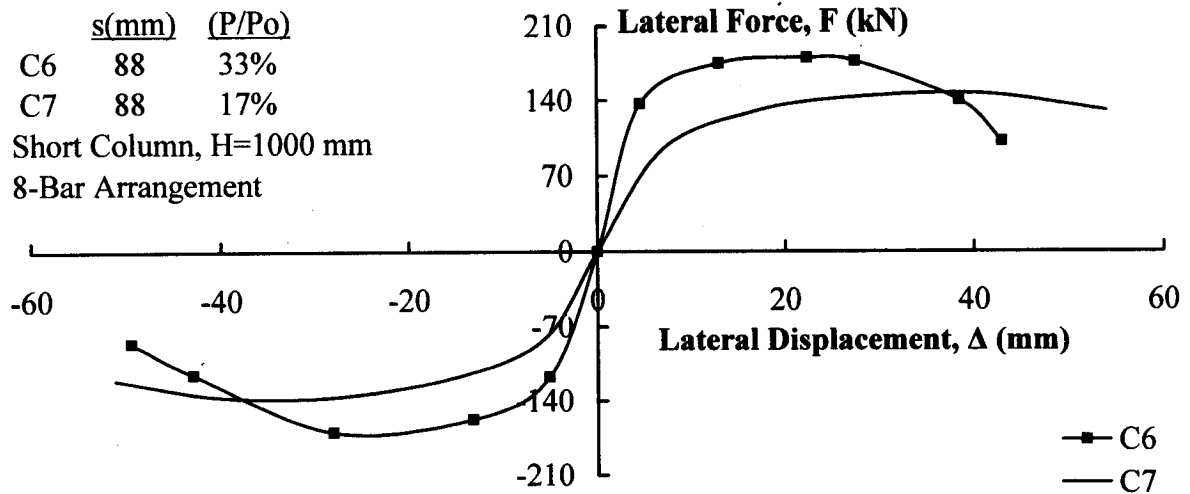


Figure 4.80: Comparison of Short Columns for the Effect of Axial Load Level

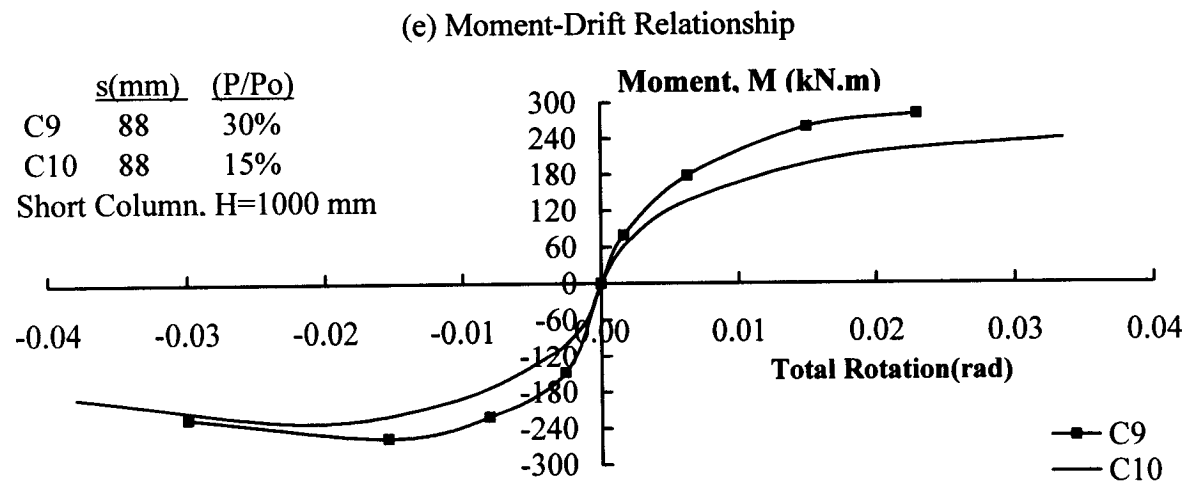
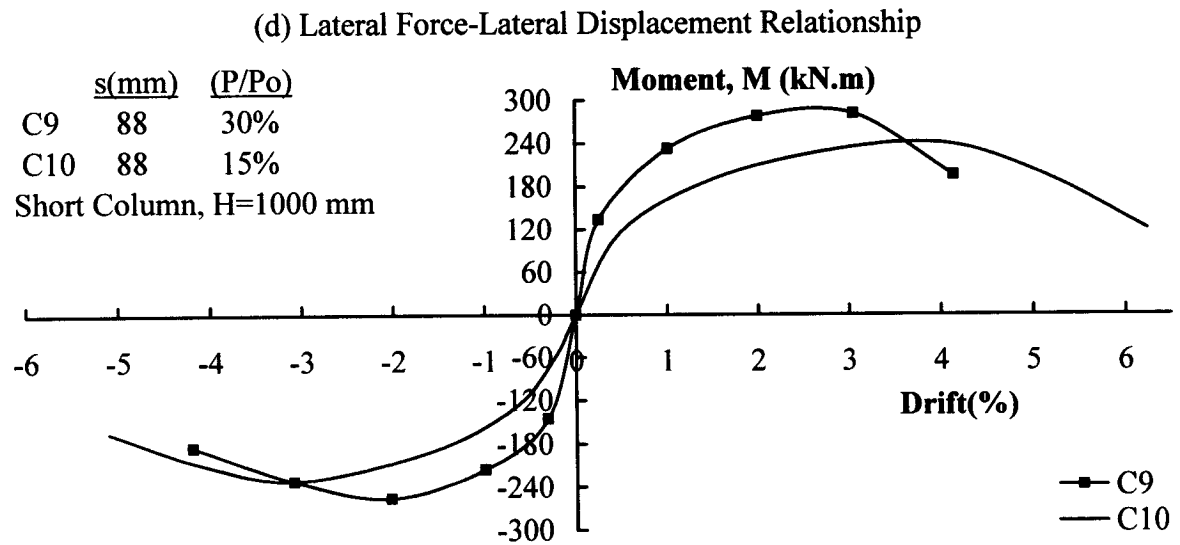
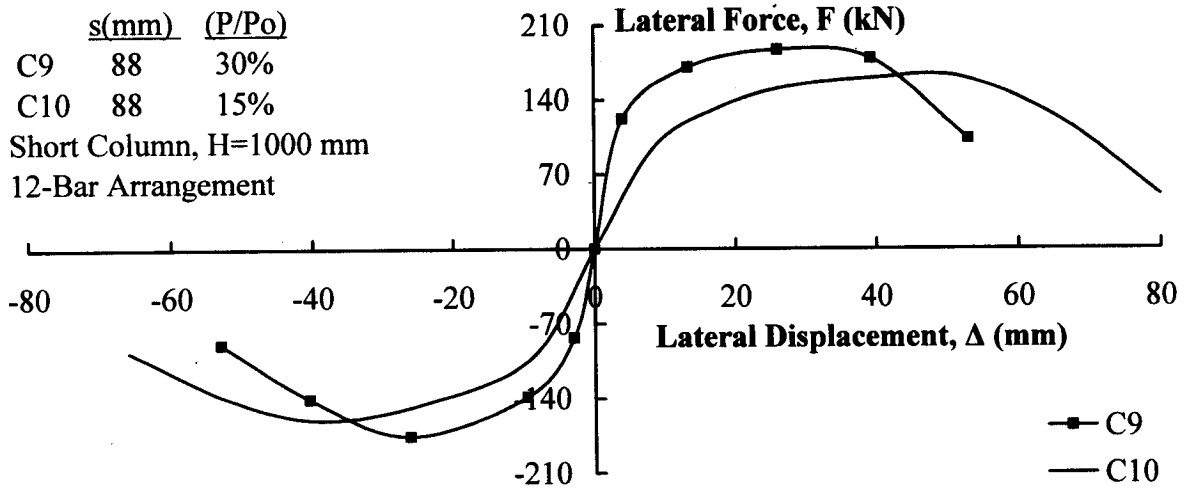
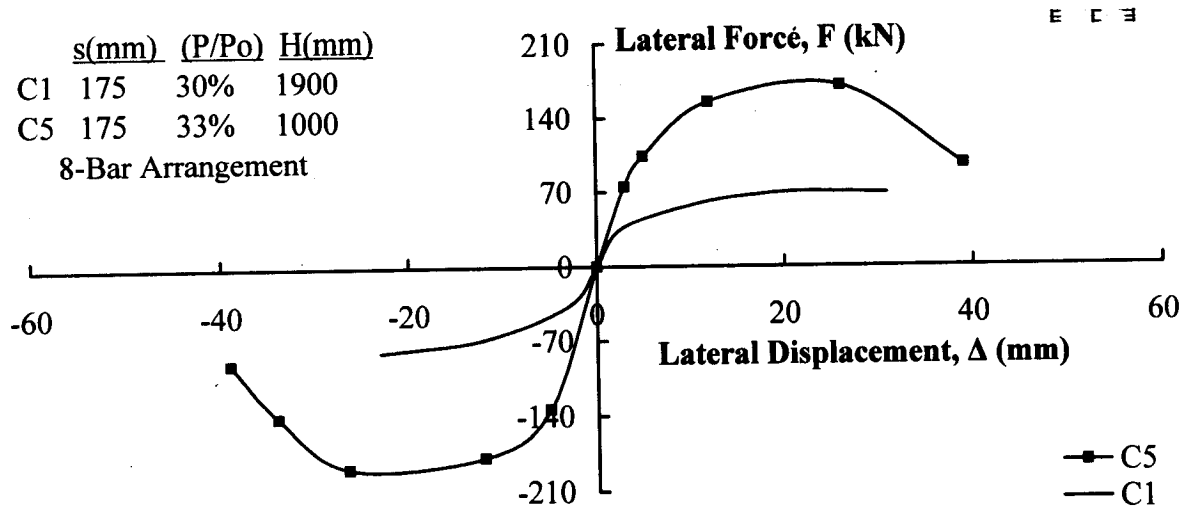
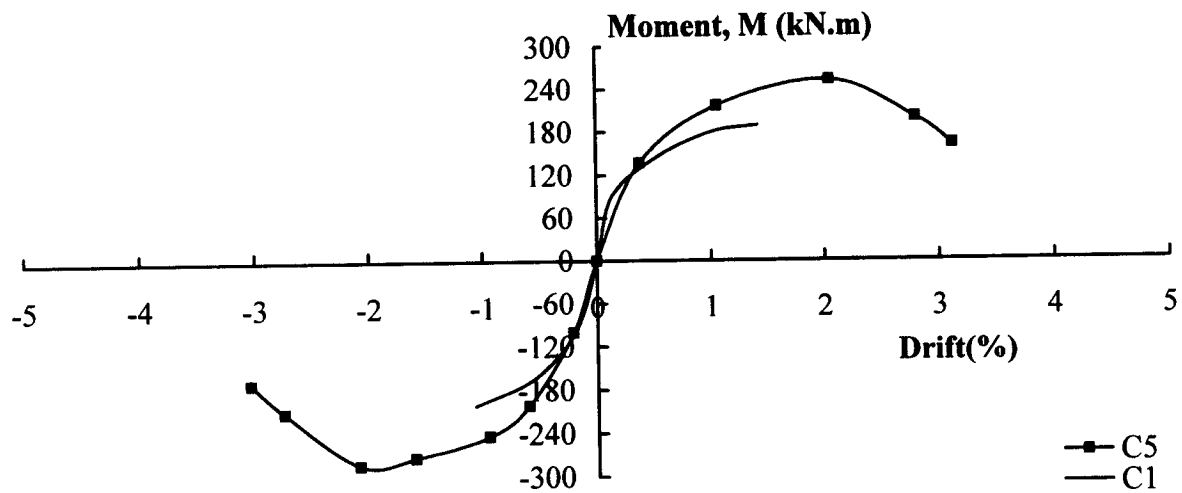


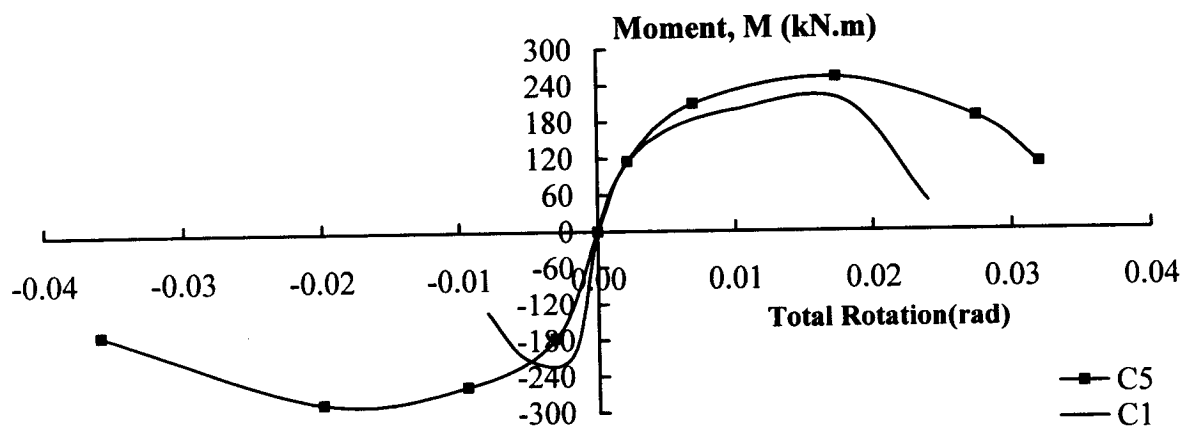
Figure 4.80 (Cont): Comparison of Short Columns for the Effect of Axial Load Level



(a) Lateral Force-Lateral Displacement Relationship

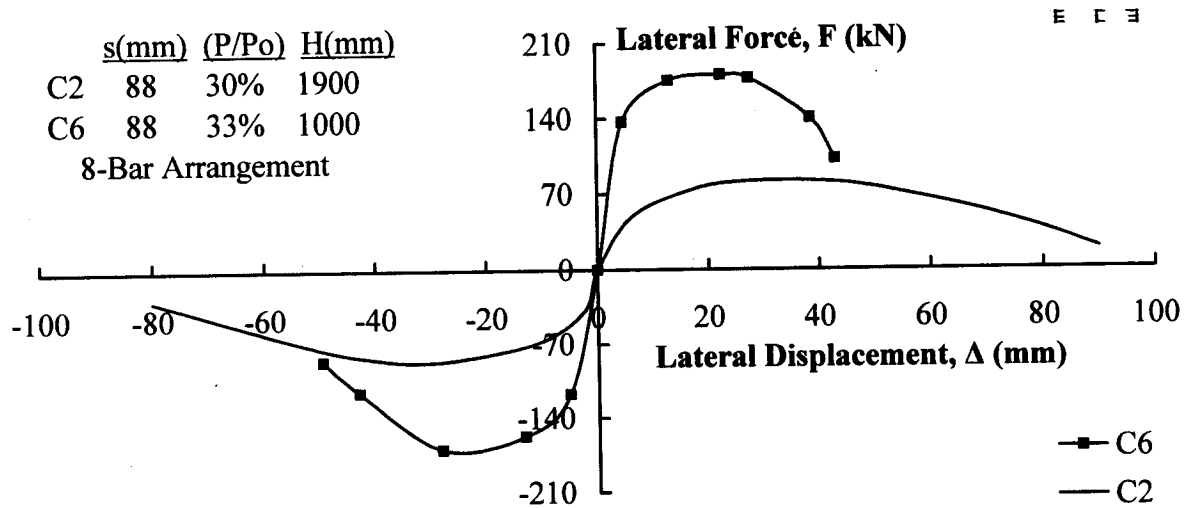


(b) Moment-Drift Relationship

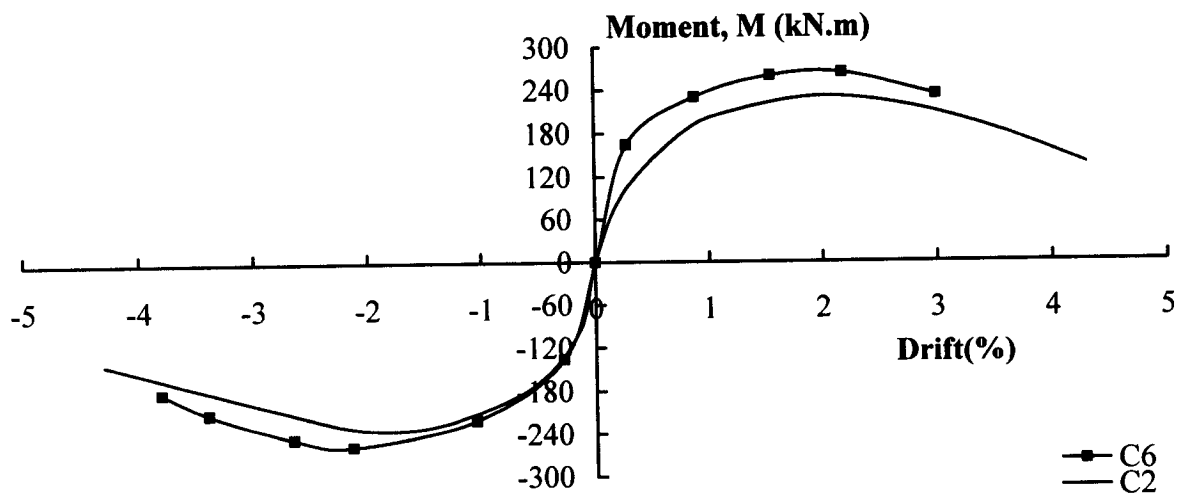


(c) Moment-Total Rotation Relationship

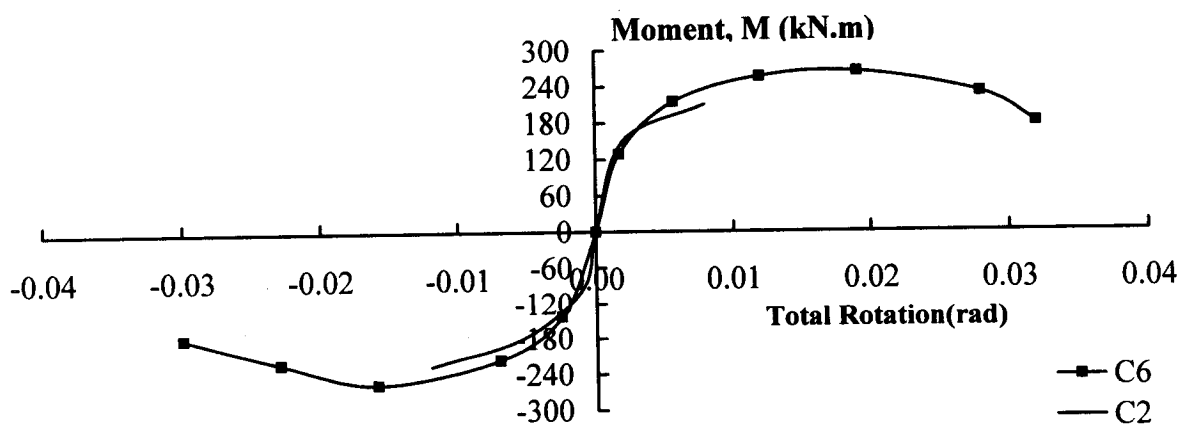
Figure 4.81: Comparison of Columns for the Effect of Column Shear Span



(d) Lateral Force-Lateral Displacement Relationship

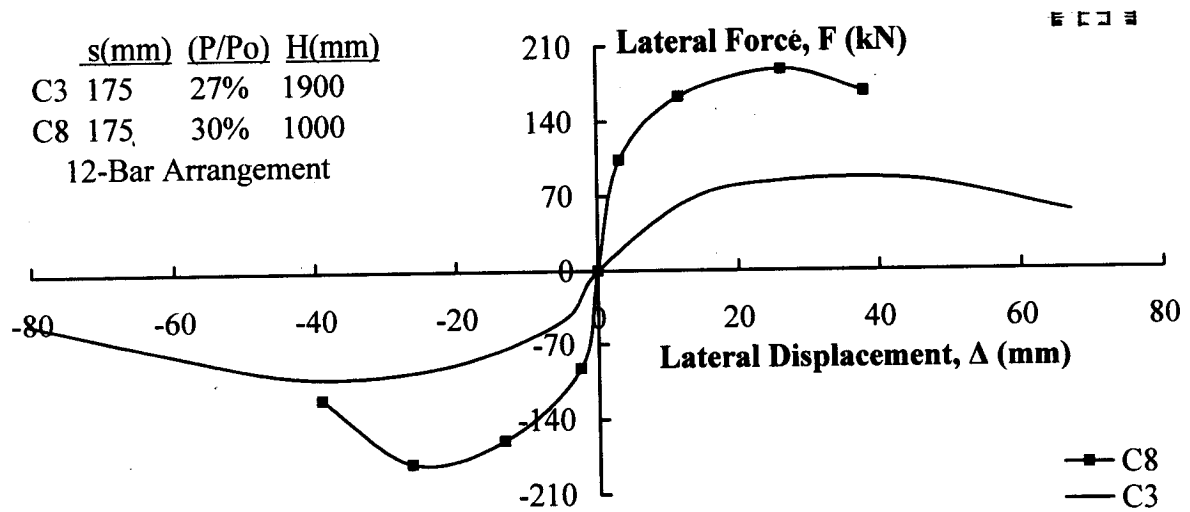


(e) Moment-Drift Relationship

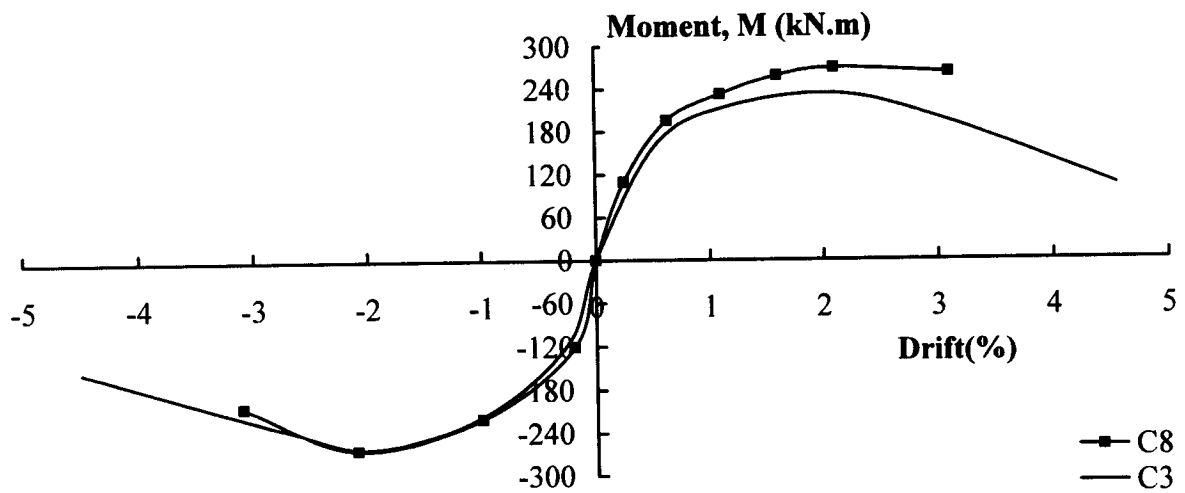


(f) Moment-Total Rotation Relationship

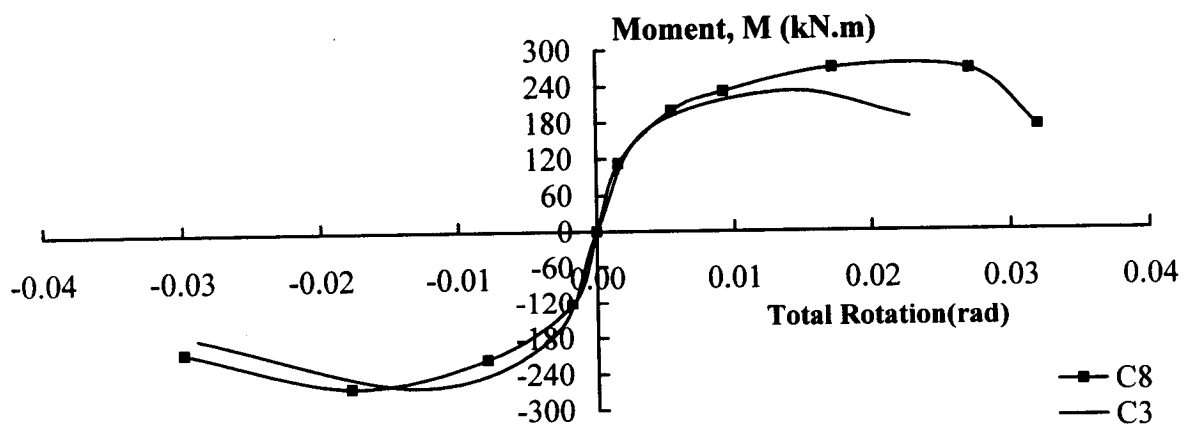
Figure 4.81 (Cont.): Comparison of Columns for the Effect of Column Shear Span



(g) Lateral Force-Lateral Displacement Relationship



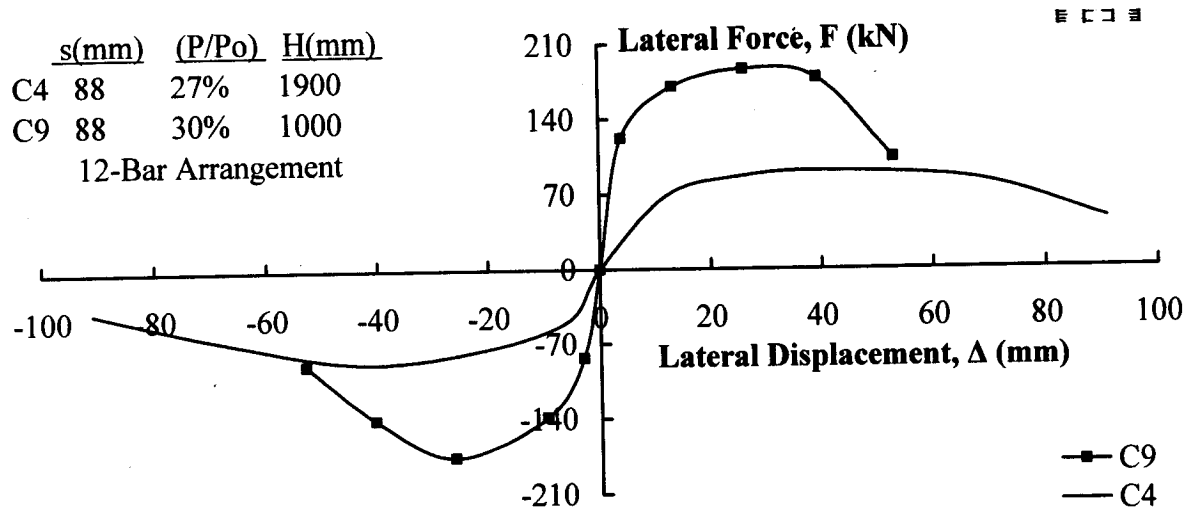
(h) Moment-Drift Relationship



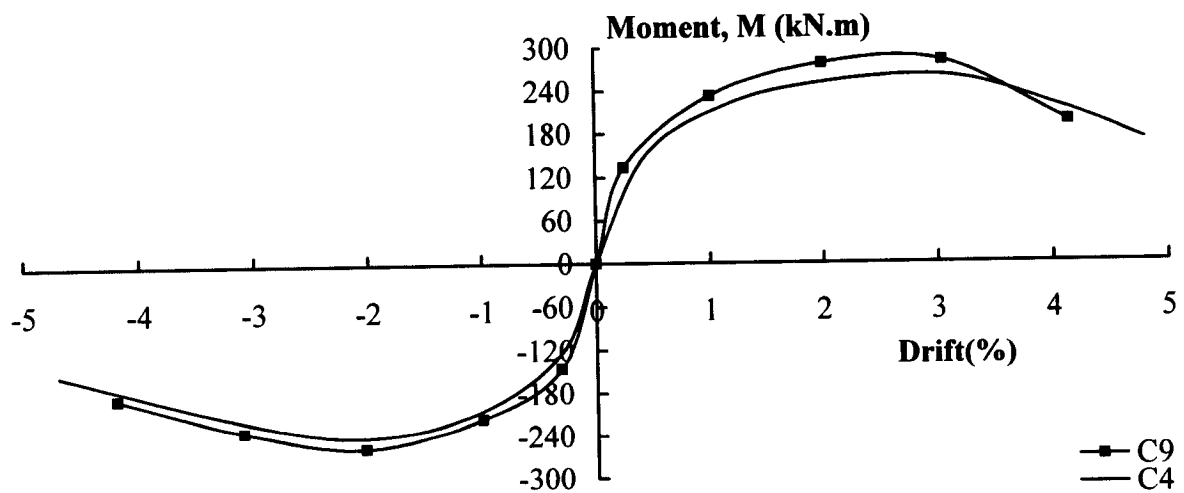
(i) Moment-Total Rotation Relationship

Figure 4.81(Cont.): Comparison of Columns for the Effect of Column Shear Span

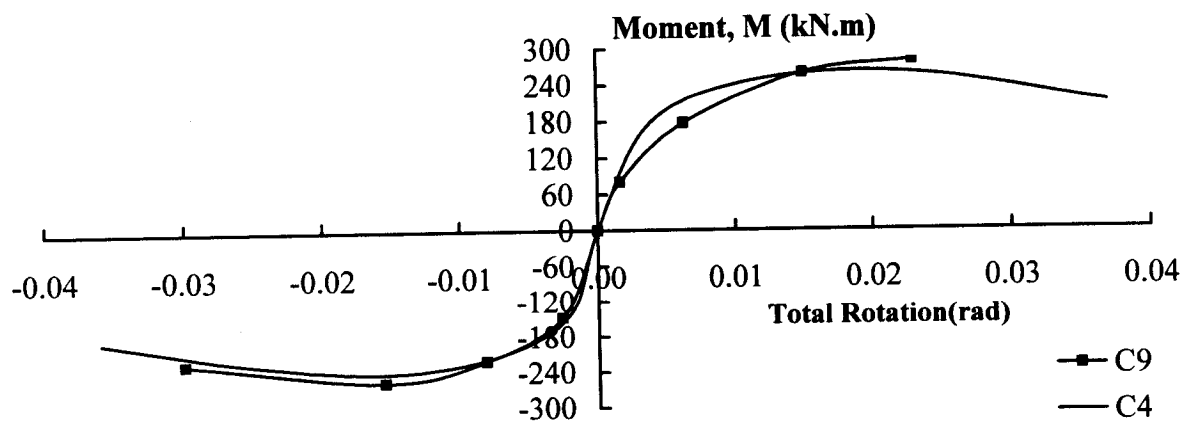




(j) Lateral Force-Lateral Displacement Relationship



(k) Moment-Drift Relationship



(l) Moment-Total Rotation Relationship

Figure 4.81(Cont.): Comparison of Columns for the Effect of Column Shear Span

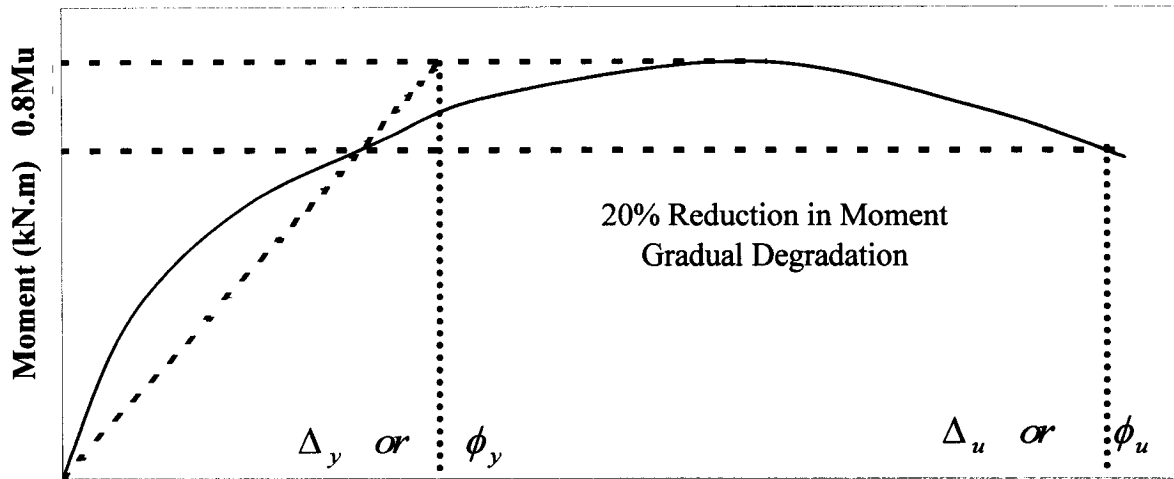


Figure 4.82: Definition of Ductility

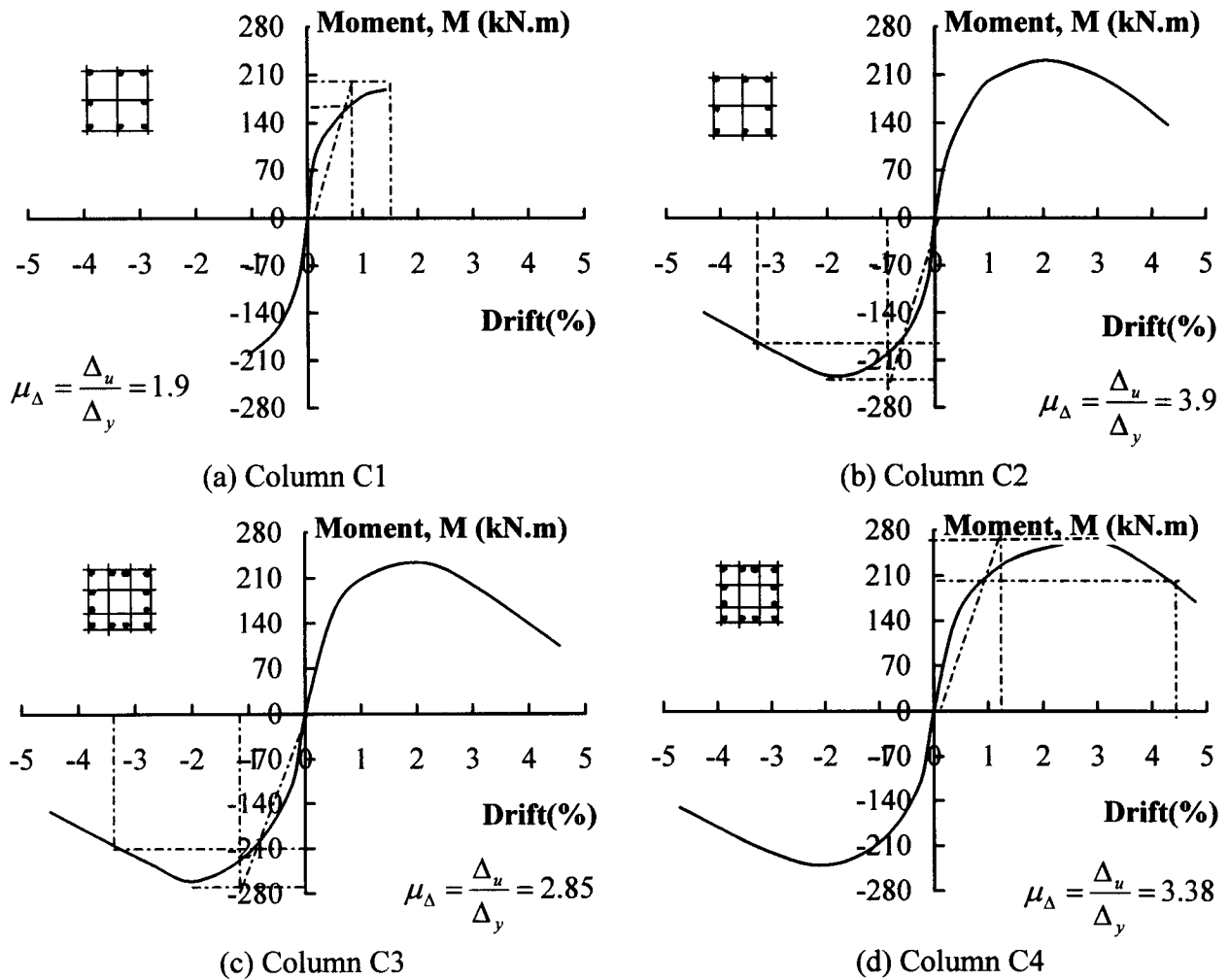
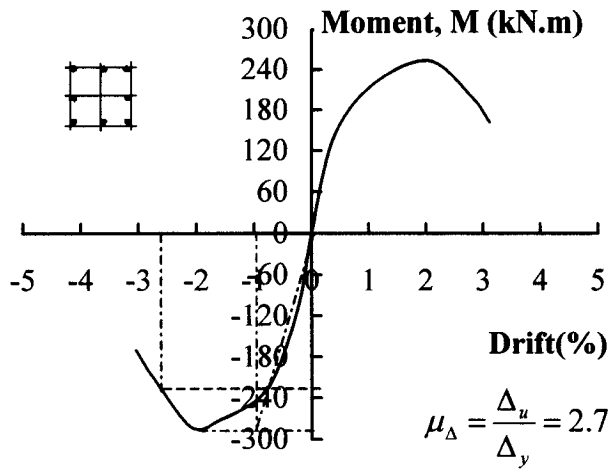
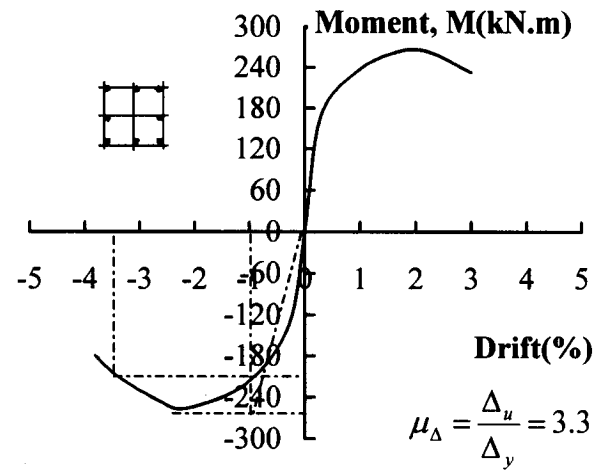


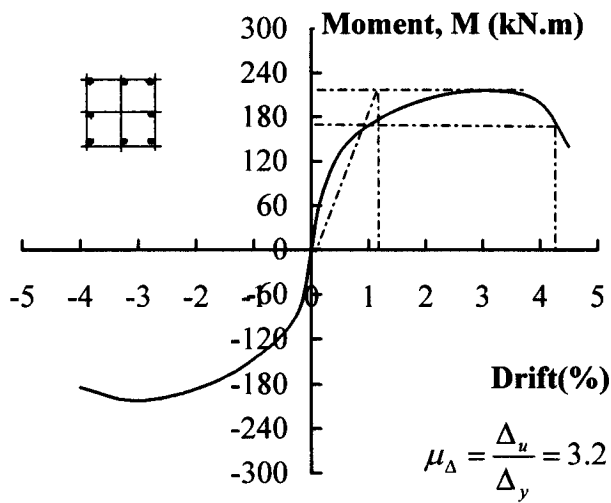
Figure 4.83: Experimental Displacement Ductility for Long Columns



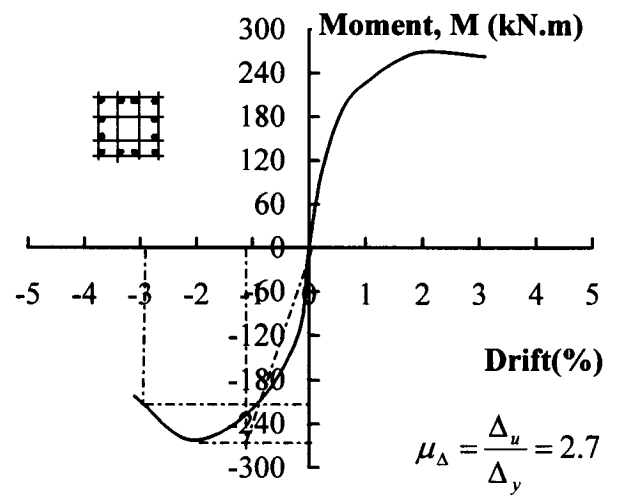
(a) Column C5



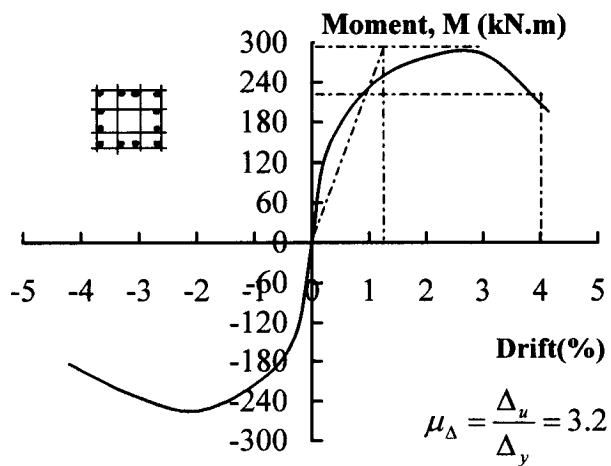
(b) Column C6



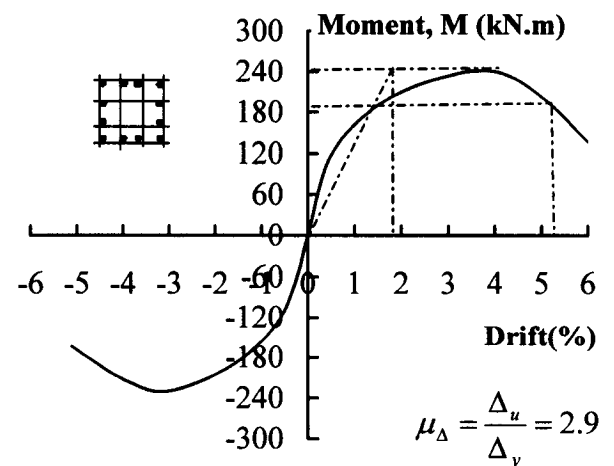
(c) Column C7



(d) Column C8

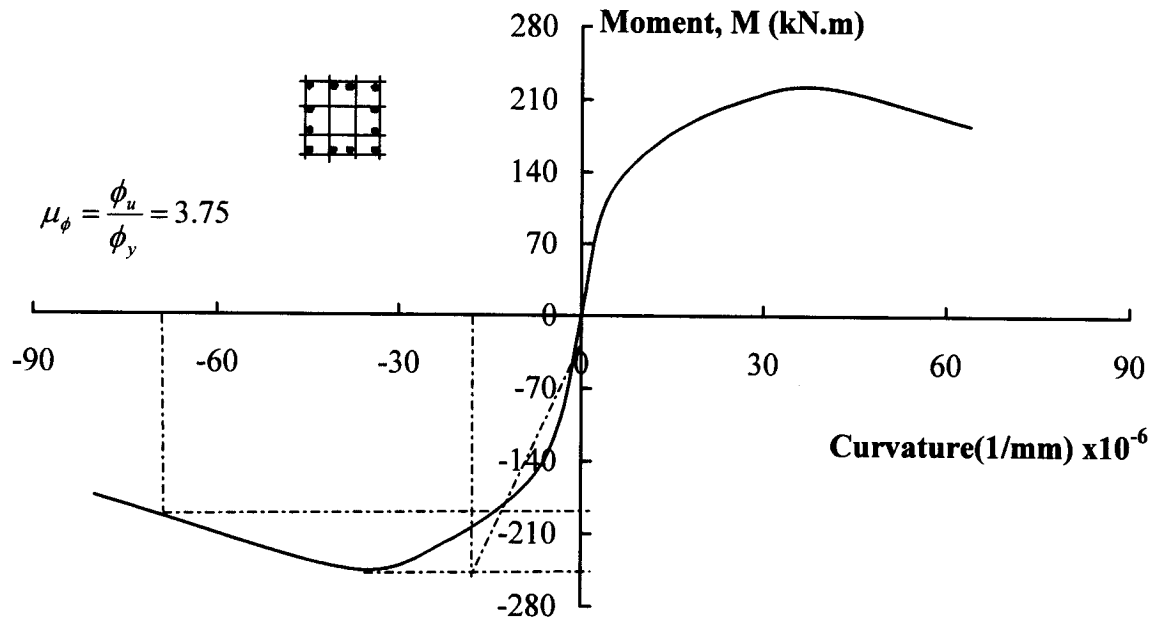


(e) Column C9

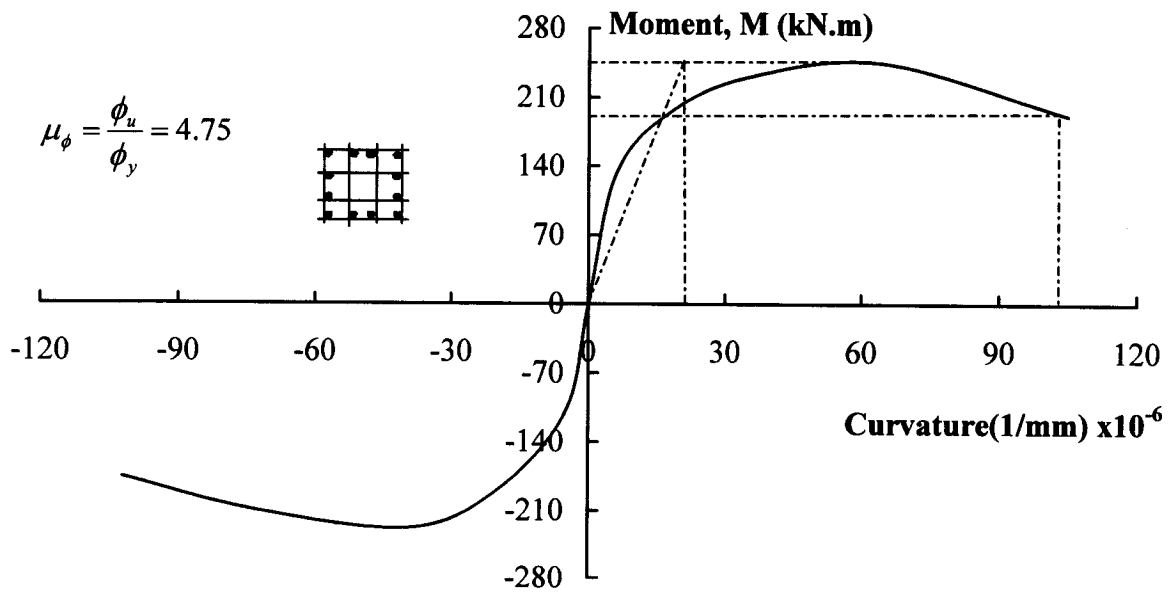


(f) Column C10

Figure 4.84: Experimental Displacement Ductility for Short Columns

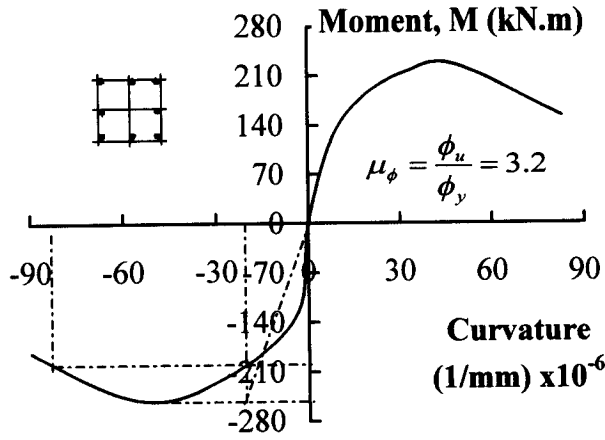


(a) Column C3

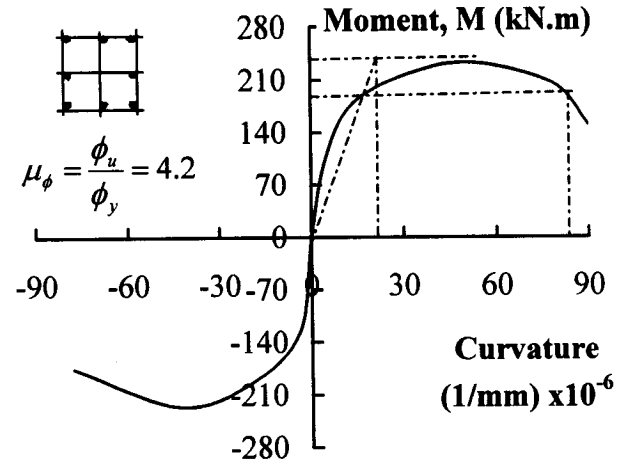


(b) Column C4

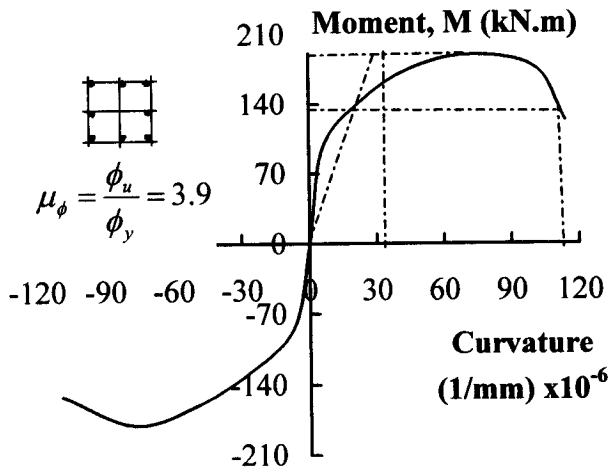
Figure 4.85: Experimental Curvature Ductility for Long Columns



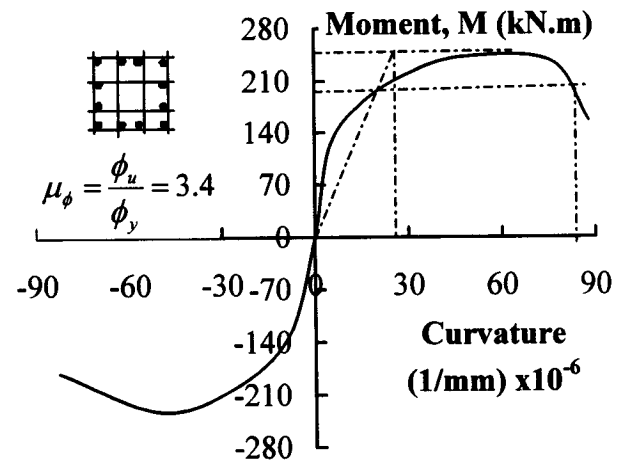
(a) Column C5



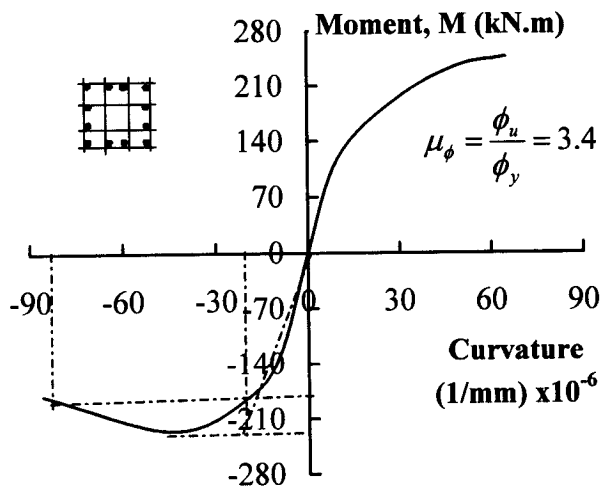
(b) Column C6



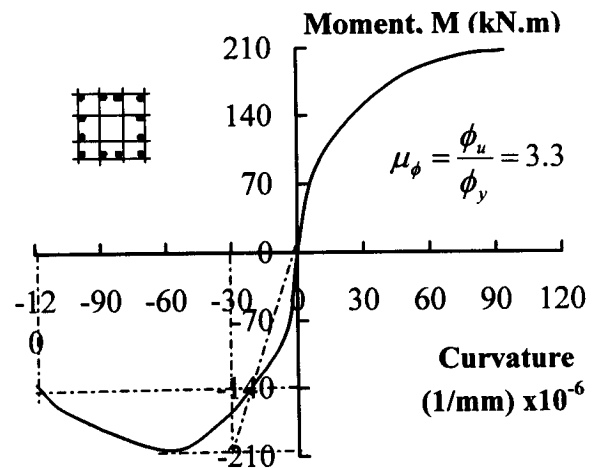
(c) Column C7



(d) Column C8



(e) Column C9



(f) Column C10

Figure 4.86: Experimental Curvature Ductility for Short Columns

## ***Chapter 5***

# **CFRP Reinforced Concrete Beams under Reversed Cyclic and Monotonic Loading**

### **5.1 General**

Full-size cantilever beams, representing a portion of a continuous beam between a fixed end and the point of inflection were designed, built and tested as Phase IV specimens. The beams were tested in the vertical position using the same test setup as that for the columns in Phase III. They were tested either under monotonically increasing lateral loading or simulated seismic loading consisting of deformation reversals. This Chapter provides the details of the experimental program in Phase IV.

### **5.2 Test Specimens**

A total of six full-scale cantilever beams, 305 mm wide and 405 mm deep, were tested. The beams had two different lengths; 1000 mm, and 1900 or 1980 mm, corresponding to shear spans of 870 mm and 1780 or 1870 mm, respectively, measured to the point of application of lateral force. The longer shear span intended to promote flexural behaviour while the shorter span would promote shear failure. Four specimens were tested under reversed cyclic loading and two other companion specimens were tested under monotonic lateral loading. Geometric details of the specimens are shown in Figure 5.1. Design strength of concrete used was 30 MPa. The actual average cylinder strength obtained at the time of testing was 40 MPa.

Only one longitudinal reinforcement ratio was used in all beams. A total of 6 - 9.5 mm diameter Pultrall CFRP bars were used as top (negative) beam reinforcement and 4 same size and type bars were used as bottom (positive) reinforcement. These resulted in 0.39% top and 0.26% bottom reinforcement ratios. The longitudinal bars were extended 470 mm into the footing, which had a total depth of 520 mm. The clear concrete cover in all beams was 15 mm, measured from the face of the beam to the outer surface of the grid reinforcement that was used as transverse stirrups. The grids were placed either at 180 mm spacing (corresponding to  $d/2$ ) or 90 mm spacing (corresponding to  $d/4$ ). Figure 5.2 shows the elevation views of reinforcement in the beams. Table 5.1 provides a summary of beam properties and test parameters.

### **5.3 Material Properties**

Normal Portland Cement concrete batches were ordered from a local ready-mix supplier to cast the test specimens in the Structures Laboratory of the University of Ottawa. Standard 150 mm diameter concrete cylinders were cast from each batch to establish the 28-day strength of concrete, as well as the strength during the period of testing. In all cases the 28-day strength of concrete was higher than the specified design strength of 30 MPa. The actual concrete strength during the period of beam tests was determined by standard cylinders to be 40 MPa. Maximum aggregate size used in all batches was 10 mm and the slump of concrete was 100 mm. The mix proportions of a cubic meter of concrete are specified in Table 2.2 and the material characteristics are indicated in Table 2.3, and are identical to that for Phases I and II specimens of this investigation. Standard molds were used to cast 150 mm by 300 mm cylinders to perform compression tests as discussed in Section 2.3.1. The results are summarized in Table 5.2.

Pultrall CFRP bars, with a nominal diameter of 9.5 mm, were used as longitudinal reinforcement. These bars were the same as those used in Phase III specimens, and are illustrated in Fig. 4.4. The properties of Pultrall bars are presented in Section 4.3.2.

The transverse reinforcement used consisted of NEFMAC CFRP grids. The grids used were the same as those used in the columns of Phase III. The grids had a rectangular configuration

with 250 x 350 mm out-to-out dimensions. Two types of grids were employed; i) 6 x 8 mm rectangular cross-section forming two equal-size rectangular openings, and ii) 8 x 10 mm rectangular cross-section forming two equal-size rectangular openings. Figure 5.3 illustrates the CFRP cages used in Phase IV beams.

#### **5.4 Preparation of Specimens, Instrumentation and Test Set up**

The beams were prepared along with attached reinforced concrete elements, representing the framing columns. These elements were used to fix the beams to the laboratory strong floor to provide full fixity, as in the case of the earlier column tests (conducted in Phase III). Therefore, they were referred to as “footings,” and had the same properties as the footings used in the column tests. The beams were cast vertically along with the column specimens. Figures 5.4 through 5.5 illustrate the preparation of beam specimens, including formwork preparation, reinforcement cages and concrete casting. The instrumentation consisted of LVDTs for displacement and rotation measurements, and electric resistant strain gauges for reinforcement strain measurements. The locations of LVDT's are illustrated in Figure 5.6. The strain gauges were placed on longitudinal and transverse reinforcement in much the same manner as they were done for Phase III columns.

A 1000 kN capacity servo controlled MTS actuator was used to apply the lateral load. No axial load was applied to the beams. At first, the load setup used for Phase III columns was used to load the first beam. However, due to the lack of axial load, the beam suffered from damage to the top anchor bolts used to secure the loading beam. The bolts experienced net tension during loading, which showed signs of pulling out and associated damage to the top concrete, outside the test region. Therefore, the test was stopped and the beam was repaired. It was subsequently decided to apply the lateral load directly on the beams, rather than applying it through the top loading beam assembly. A steel box section was attached to a steel plate by means of dwydag prestressing bars and used to apply the load. The assembly was first secured near the tip of beams, as illustrated in Figure 5.6. Photographs of the loading assembly and the test setup are shown in Figure 5.6. The beams were tested following the loading history shown in Figure 4.25, also used for testing the Phase III columns.



## 5.5 Test Results

Observed behaviour of beams during testing is presented and discussed in this section. Photographs taken at selected stages of loading illustrate the performance of each beam. The hysteretic behaviour is presented both in terms of force-displacement and moment-drift relationships. The moments plotted in these relationships are computed from recorded test data as the net lateral force times the shear span.

Hysteretic moment-total rotation, moment-slip rotation, and moment-flexural rotation relationships are also presented and discussed in this chapter. These readings were taken within the potential hinge region, as explained in Section 4.5. The hinging region was defined as the beam segment between the beam-column (footing) interface and the section 405 mm away from the interface (equal to the longer cross-sectional dimension). The first set of readings gave total rotation of assumed hinging region relative to the column. This set of readings consisted of rotations mostly due to flexure and also due to anchorage slip. The rotations due to the anchorage slip were measured and formed the second set of readings. These readings were taken as the rotation of a beam section near the beam-column interface, relative to the column. Ideally, these readings should have reflected the rotation of the critical beam section at the interface. However, the LVDTs used required some gauge length to be positioned on the beam. Hence, they were mounted on a section approximately 25 mm above the interface. Therefore, these readings also included flexural rotations of the closest 25 mm segment of the beam to the column. Hence they are not accurate representations of anchorage slip. Furthermore, these readings were taken within the most disturbed region of beams because of concrete crushing. Hence, additional errors may have occurred especially near the end of testing when significant damage was observed in the beam hinging region. The difference between the total and anchorage slip rotations gave those caused by flexure.

Even though the anchorage slip caused in steel reinforced columns is closely related to the level of yield penetration that occurs into the footing, the slip rotation may also be important in FRP reinforced beams due to the elastic extension and potential slippage of the bars. The slippage of bars becomes noticeable only if the embedment length is not sufficient to develop

required stresses in reinforcement. It may be significant in beams because of the absence of axial compression.

The variations of reinforcement strains in the longitudinal and transverse reinforcement are presented in the form of hysteretic force-strain relationships. Each beam had four longitudinal bars instrumented with a total of 6 strain gauges to measure the variation of strain in CFRP bars. The strain gauges ceased to function at different stages of loading either due to excessive tension or compression. Hence, the data presented may not represent the behaviour of FRP bars at the end of testing. All four corner bars were instrumented at the critical beam section, at the interface with the attached (column) element. Two additional strain gauges were placed on one of the negative reinforcement bars in the corner to monitor the variation of strains; one gauge was placed at 175 mm away from the column and the other at 175 mm inside the column. The end grids (closer to the columns) were instrumented with strain gauges, as well. Beams with  $d/2$  grid spacing had the first three grids instrumented, while those with  $d/4$  spacing had the first four instrumented.

An important parameter that was researched in beam tests was the effectiveness of transverse grid reinforcement in shear. The amount and spacing of grid reinforcement varied in test beams. These test parameters are expressed relative to those required by CSA S806-02. Therefore, a summary of these requirements is presented below.

CSA S806-02 shear design requirements are based on 45-degree truss analogy with contributions from concrete and FRP reinforcement. Accordingly, the concrete shear resistance,  $V_c$ , is calculated at two different expressions as follows:

- (i) For sections with an effective depth greater than 300 mm and with transverse shear reinforcement less than minimum amount of shear reinforcement.

$$V_c = \left( \frac{130}{1000 + d} \right) \sqrt{f'_c} b_w d \leq 0.08 \sqrt{f'_c} b_w d \quad (5.1)$$

- (ii) For sections having either at least the minimum amount of transverse reinforcement or an effective depth not exceeding 300 mm.

$$0.1 \sqrt{f'_c} b_w d \leq V_c = 0.035 \left( f'_c \rho_w E_F \frac{V_f}{M_f} d \right)^{1/3} b_w d \leq 0.2 \sqrt{f'_c} b_w d \quad (5.2)$$

This amount is equal to 52.7 kN, based on upper limit in Equation (5.1), for Beams CFB1 and CFB4 tested in the current investigation.  $V_c$  is equal 73 kN, Equation (5.2), for the other four beams. In all the beams tested, applied shear forces corresponding to flexural capacities were higher than the computed value of 52.7 kN or 73 kN for concrete resistance, respectively. Therefore, shear reinforcement was required in all beams. The required area of shear reinforcement in CSA S806-02 is specified below.

$$V_F = \frac{A_F \phi_F f_{FH} d}{s} \quad (5.3)$$

where;  $f_{FH}$  is the smaller of  $0.4f_{Fu}$  or  $0.004E_F$ . The maximum spacing of transverse shear reinforcement is limited by CSA A23.3-1994 to 0.7d or 600 mm for members subjected to shear stresses of lower than  $0.1\lambda\phi_c f'_c$ , which is the case for the beams tested in Phase IV. The same spacing limit is specified as 16 times the longitudinal bar diameter, 48 times the minimum cross-sectional dimension of FRP grid, the least dimension of the compression member, or 300 mm for the stability of compression reinforcement. Furthermore, the confinement requirements for seismic design limit the spacing to d/4 or 150 mm, whichever is smaller. The minimum amount of shear reinforcement required is;

$$A_v = \frac{0.3 \sqrt{f'_c} b_w s}{f_{FH}} \quad (5.4)$$

## 5.5.1 Test Results and Observations

### 5.5.1.1 Beam CFB1

Beam CFB1 was a short shear dominant beam, reinforced with 2-cell grids consisting of CFRP legs with an 8 x 10 mm cross-section. The reinforcement area of each grid was 240 mm<sup>2</sup>. The grid spacing was 180 mm which was less than the maximum limit of  $0.7d = 255$  mm for shear, but higher than the maximum limit of  $d/4 = 90$  mm for confinement, though it was closer to the bar stability limit of 16 times the diameter of longitudinal bar, which came out to be 150 mm. The amount of transverse reinforcement (240 mm<sup>2</sup>) was approximately 33% more than that specified by Equation (5.3) as required reinforcement for shear, but 22% below the minimum amount of 310 mm<sup>2</sup> required by the CSA S806-02, Equation (5.4).

The beam was tested under reversed cyclic loading. It showed nearly elastic behaviour up to about 2.2% drift ratio, with some stiffness degradation caused by concrete cracking. The beam developed a wide diagonal tension crack between the first and the second grid from the column face, when it was loaded to 2.5% drift level in the strong direction. At this load stage the compression bars were subjected increased dowel action in addition to compression, and developed stability failure and broke. The reversal of the beam had no flexural resistance left in the weak direction since the tension reinforcement had all but failed during the previous loading in the opposite direction. This is evident in the hysteretic relationship shown in Figure 5.7. The test continued with no load resistance in the weak direction and gradually degrading loops in the long direction, up to about 6% drift ratio.

Observations during testing indicated that flexural cracking started to occur at 0.5 % drift cycles within the lower 450 mm portion of beam. The cracks formed at the first three grid locations, on both sides of the beam. During 1% drift cycles, additional flexural cracks were observed within the 750 mm segment from the column face with significant diagonal shear cracks forming on two side faces. This is shown in Figure 5.8-c. The diagonal cracks cover entire side faces during cycles at 1.5% drift, with increased flexural cracking. One of the diagonal cracks that started just beyond the first grid location became wider during loading to 2% drift ratio and changed its inclination to propagate toward the flexural tension side,

avoiding the next grid. This crack, with no transverse reinforcement between the first and the second grid, could only partially be controlled by the longitudinal reinforcement. Concrete cover suffered from spalling near the beam end in the compression zone. Widening diagonal crack completely diminished concrete contribution to shear during the first load excursion to 2.5% drift ratio, putting significant shear on the compression bars, forcing them to resist dowel forces. The bars then failed in compression due to increased compressive stresses combined with lateral dowel forces and broke at the inclined crack location. This is illustrated in Figure 5.8 (f). Upon reversing the load, the beam was not able to resist any lateral load in the weak direction because of lack of tension elements. However the test was continued with gradually degrading stiffness and strength in the strong direction and zero load resistance in the weak direction. The hysteretic relationship given in Figure 5.7 illustrates this behaviour eloquently. Figure 5.8 illustrates the performance of beam CFB1 at selected stages of loading.

Strain gauge readings indicated that FRP bars experienced a maximum of 1.15% tensile strain and 0.3% compressive strain, whereas the FRP grids experienced a maximum of approximately 0.5% tensile strain. Figures 5.9 and 5.10 show the hysteretic behaviours of strains in bars and grids, respectively.

Moment-rotation hysteretic relationships, recorded during testing, are shown in Figure 5.11. The results indicate that about half of total rotation was caused by anchorage slip, and the remaining by flexure. The relatively high contribution of anchorage slip deformations can be explained by the lack of axial compression in beams, when compared with column behaviour in Phase III tests. Further discussion of these deformation components is made in Chapter 6 where analytical computations are presented.

#### **5.5.1.2 Beam CFB2**

Beam CFB2 was companion to CFB1, except for the grid spacing, which was reduced to 90 mm. This level of spacing is consistent with the spacing requirement for confined concrete, specified in CSA S806 for seismic design. It also meets the maximum spacing requirements

for shear and stability of longitudinal compression bars. The area of transverse reinforcement was 55% in excess of those required for shear.

The beam was tested under reversed cyclic loading. It showed improved behaviour than the companion beam CFB1. The reduction in grid spacing increased the effectiveness of shear resistance while enhancing concrete confinement. The beam was able to develop its flexural strength and showed approximately 35% increase in lateral load resistance as compared with the companion beam CFB1, which failed prematurely due to shear. The hysteresis loops showed near elastic behaviour with some stiffness degradation due to concrete cracking up to 3% drift ratios. Some gradual strength decay was observed beyond this drift level due to the crushing of concrete. However, the compression bars buckled when the drift level was increased to 4% and 4.5% drift levels, which led to the failure of the beam. Figure 5.12 shows the hysteretic relationships recorded during the test.

The observations during testing indicated that flexural cracking started at 0.5% to 1% drift cycles. These cracks generally occurred at grid locations, closer to the column face. In addition, some diagonal cracks were observed at 1% drift ratio. Additional flexural cracks were observed during 2% drift cycles, extending toward the middle of the beam. Some of the positive reinforcement (4-bar side) reached their tensile rupturing point during the third cycle of 2% drift ratio. The strains recorded in these bars were very close to the tensile rupturing strain. The cover concrete on the same side spalled off at 2.5% drift ratio. Degradation of strength began at the first cycle of 3% drift, and continued till 4% drift ratio. At this level of deformation, all the 4 positive bars ruptured in tension causing the beam to lose its flexural resistance in the weak direction. The column was able to resist severely reduced loading in the strong direction when pushed to 5% drift before the end of testing. Figure 5.13 illustrates the performance of beam CFB2 at selected stages of loading.

The strain gauge readings indicated that FRP bars experienced a maximum of 1.4% tensile strain and 0.3% compressive strain, whereas the FRP grids experienced a maximum of 0.5% tensile strain. Hysteretic strain behaviour of bars and grids are presented in Figures 5.14 and 5.15, respectively.

The moment-rotation hysteretic relationships recorded during testing are shown in Figure 5.16. The results indicate that about 62% of total rotation was caused by anchorage slip and 38% by flexure. Further discussion of these deformation components is made in Chapter 6 where analytical computations are presented.

#### **5.5.1.3 Beam CFB3**

Beam CFB3 was identical to beam CFB2, except it was tested under monotonically increasing lateral loading. The results showed that both Beams CFB2 and CFB3 had approximately the same primary (envelope) force-deformation relationship. The beam showed increasing load resistance up to 3% drift, followed by subsequent strength decay beyond this deformation level due to the failure of FRP bars in tension. The strength decay continued gradually up to 6% drift ratio with a clear reduction in load resistance every time there was bar rupture in tension. The monotonic relationships recorded during the test are shown in Figure 5.17.

Experimental observations indicated that initial flexural cracking started at 0.5% lateral drift ratio. At 1% drift ratio, a flexural crack was seen at each grid location, extending up to the steel loading plate on tensile side. Some diagonal cracks were also observed on side faces of the beam at the same deformation level. Once the lateral drift ratio reached 3%, there was a sudden drop of about 40% in load resistance due to the rupture of the first FRP bar in tension. Subsequent bar fractures were recorded at about 3.6%, 4.2%, 4.6%, 5%, and 6% drift ratios for all the remaining 5 bars that the beam had on tension side. The rupturing of longitudinal bars occurred approximately within 100 mm inside the attached column (footing of the specimen), some bar rupturing at exactly at beam column interface while others ruptured beyond the interface, within 100 mm into the adjoining member. This is clearly shown in Figure 5.18(f). It is worth noting that the tension failure in FRP bars occurs as splitting and rupturing of fibers within certain length, as observed in tension coupon tests, contrary to bar failure in compression, which is far more localized and resemble to bars cut at a section. Figure 5.18 illustrates the performance of Beam CFB3 at selected stages of loading.

Strain gauge readings indicated that FRP bars experienced a maximum 1.3% of tensile strain and 0.27% compressive strain, whereas the FRP grids experienced a maximum of 0.42% tensile strain. Monotonic behaviours of recorded strains during testing are presented in Figures 5.19 and 5.20.

Moment-rotation monotonic relationships, recorded during testing, are also shown in Figure 5.21. The results indicate that at peak load resistance, about 35% of total rotation was caused by anchorage slip, and remaining 65% by flexure. Further discussion of these deformation components is made in Chapter 6 where analytical computations are presented.

#### **5.5.1.4 Beam CFB4**

Beam CFB4 was companion to CFB1, except for its longer length of 1900 mm. This length was used to promote flexural behaviour. The hysteretic relationships shown in Figure 5.22 indicate that the beam showed stable hysteresis loops up to 3.5% drift ratio, but developed significant strength decay after the first cycle of 4 % drift ratio. The beam failed during the first cycle at 4.5 % drift ratio due to the failure of FRP bars in tension.

Observations during testing indicated that flexural cracking started at 0.5 % drift cycles. Two flexural cracks occurred within a distance approximately equal to the beam depth, measured from the beam-column interface. Flexural cracking increased and extended up to about 800 mm from the beam-column interface as deformations reached 2% drift ratio. Some diagonals cracks were also observed on side faces, and a hairline crack occurred at the beam-column interface. The width of shear cracks increased during cycles of 3.5% drift ratio. At about 4% drift ratio in the strong direction, the bars started rupturing in tension. All the bars had ruptured when a drift ratio of about 4.5% was reached. This marked the end of testing. Figure 5.23 illustrates the performance of CFB4 at selected stages of loading.

Strain gauge readings indicated that FRP bars experienced a maximum 1.25 % of tensile strain and 0.38% compressive strain, whereas the FRP grids experienced a maximum of 0.44% tensile strain. Hysteretic behaviour of strains for bars and grids are presented in Figures 5.24 and 5.25, respectively.



Moment-rotation hysteretic relationships, recorded during testing, are plotted in Figure 5.26. The results indicate that approximately 40% of total rotation was caused by anchorage slip and the remaining 60% by flexure. Further discussion of these deformation components is made in Chapter 6 where analytical computations are presented.

#### **5.5.1.5 Beam CFB5**

Beam CFB5 was identical to CFB4, except for the spacing of grids, which was reduced to 90 mm. Figure 5.27 shows the hysteretic relationships recorded during the test. Hysteresis loops indicate stable behaviour up to 2% drift ratio, for loading in both directions. The beam showed increased load resistance in the strong direction up to 4% drift ratio. However, it experienced noticeable stiffness decay when loaded beyond 2% drift ratio in the weak direction due to the tension failure of two of the bars. This beam was one of the first beams tested. It was planned to apply the horizontal force the same way as it was done in column tests, i.e., through the top-loading beam. However, due to the absence of axial compression, anchorage failure was observed in the bolts used to attach the steel-loading beam to the beam. This occurred during cycles at approximately 32 mm tip deflection and caused some damage to concrete at the tip of the beam, outside the test region. Therefore, the test was stopped and the concrete was repaired, before it was tested using a different setup. It was decided to apply the lateral load directly on the specimen face through a steel plate, without using any loading beam. The shear span for the beam, measured to the point of application of load was 1875 mm, 4.5% longer than the companion beam CFB4.

Observations during testing indicated that flexural cracking started at 0.5% drift within the 600 mm segment next to the column, forming at grid locations, except for the first grid location. Additional flexural cracks formed within 1100 mm from the beam-column interface when the beam was subjected to 1% drift cycles. Some shear cracks occurred starting from the beam-column interface and propagating towards the third grid, at 1.5% drift cycles. Additional flexural cracking was observed during 2% drift cycles, extending up to the loading plate. More cracks were observed in this beam than the companion Beam CFB4, probably because of initial loading under the old setup. Three long diagonal cracks were observed within the 400 mm segment from the beam-column interface. When the beam was

loaded in the weak direction to 3% drift ratio, two of the four tension bars ruptured as evidenced by loud sound and also accompanying reductions in the stiffness of beam. This was not expected since Beam CFB4, with identical properties and the same flexural capacity was able to sustain similar deformations without a bar fracture. A wide flexural crack was observed on the weak side of beam, another indication of bar failure in tension. Concrete started to spall off at the second cycle of 3% drift ratio. Cracks on the 6-bar side widened near the beam-column interface. These bars started to rupture in tension, one by one, as the deformation increased beyond 4% drift ratio. Finally the beam failed completely at 4.5% drift. Figure 5.28 illustrates the performance of Beam CFB5 at selected stages of loading.

Strain gauge readings indicated that FRP bars experienced a maximum 1.2% of tensile strain and 0.28% compressive strain, whereas the FRP grids experienced a maximum of 0.4% tensile strain. Figures 5.29 and 5.30 show the hysteretic behaviours of recorded strains on bars and grids, respectively.

Moment-rotation hysteretic relationships, recorded during testing, are shown in Fig 5.31. The results indicate that about 13% of total rotation was caused by anchorage slip and the remaining 87% by flexure when the loading was in the strong direction, whereas these percentages changed in the opposite direction to indicate about equal rotations cause by slip and flexure. Further discussion of these deformation components is made in Chapter 6 where analytical computations are presented.

#### **5.5.1.6 Beam CFB6**

Beam CFB6 was identical to beam CFB5, except it was tested under monotonically increasing lateral loading in the strong direction. The results showed that both Beams CFB5 and CFB6 had approximately the same primary (envelope) force-deformation relationship. The beam showed increasing load resistance up to 4% drift, followed by subsequent strength decay beyond this deformation level due to the failure of FRP bars in tension. The strength decay continued gradually up to 6% drift ratio with a clear reduction in load resistance every time there was bar rupture in tension. The monotonic force-deformation relationships are shown in Figure 5.32. Like Beam CFB5, this beam was also initially attempted to be tested

like columns, by applying the lateral load through a steel-loading beam. Similar problems were encountered during testing, and the loading setup was changed subsequently and the test was conducted by applying the load on the face of beam. This resulted in a shear span of 1875 mm, measured to the point of application of lateral force.

Observations during testing indicated that flexural cracks formed at all grid locations within the 1500 mm segment from the beam-column interface when the beam was loaded to 1% drift ratio. Two small diagonal cracks were observed close to the interface at this drift ratio. The crack widths increased at 2% drift ratio. New horizontal cracks formed at 3% drift ratio between the grid locations, within the 400 mm segment from the interface. Once the lateral drift ratio reached 4%, there was a sudden drop in load resistance due to the rupturing of the first two FRP bars in tension. Subsequent bar fractures were recorded at about 5% and 6% drift ratios for all the remaining 4 bars in tension. Figure 5.33 illustrates the performance of Beam CFB6 at selected stages of loading.

Strain gauge readings indicated that FRP bars experienced a maximum of 1.25% tensile strain and 0.15% compressive strain, whereas the FRP grids experienced a maximum of 0.26% tensile strain. Monotonic behaviour of strains for bars and grids are presented in Figures 5.34 and 5.35, respectively.

Moment-rotation relationships under monotonic loading, recorded during testing, are shown in Figure 5.36. The results indicate that about 33% of total rotation was caused by anchorage slip and the remaining 67% by flexure. Further discussion of these deformation components is made in Chapter 6 where analytical computations are presented.

## **5.6 Effects of Test Parameters**

### **5.6.1 Parameters Affecting Shear**

The specimens were designed to investigate the shear behaviour of FRP reinforced beams. Selecting two shear spans and two spacing of transverse shear reinforcement achieved this.

While beams with short shear span would be subjected to higher shear stresses, those with closely spaced transverse grids would have higher shear capacity.

Beams CFB1 and CFB2 had short shear spans, and were subjected to higher shear stresses. The only difference between the two was the spacing of grid reinforcement. Beam CFB1 developed a wide diagonal tension crack due to shear. This occurred between the first and the second grid near the beam-column interface, resulting in diagonal tension failure when it was loaded to 2.5% drift level in the strong direction. At this load stage, the compression bars were subjected to increased dowel action in addition to compression, and experienced stability failure and broke. The reversal of the load in the weak direction had no flexural resistance, and the beam could not sustain any load resistance. The spacing of grid reinforcement had been reduced by 50% in CFB2. This also resulted in the doubling of the amount of shear reinforcement. The beam did not suffer from shear failure, and behaved in the flexure mode despite its short shear span, developing 37% higher shear resistance. This is evident in Figure 5.37 where the envelopes of hysteretic force-deformation relationships are compared. The figure also indicates that flexural rotations measured in CFB2 were 70% and 36% higher than those of CFB1 in the strong and weak directions, respectively. The comparison of long beams with different grid spacing, also presented in Figure 5.37, does not show any significant difference in behaviour. This is expected since these beams behaved predominantly in the flexure mode because of their longer shear spans.

Additional comparisons of beam behaviour are made in Figure 5.38 where the envelope curves of force-deformation relationships for beams with different shear spans are presented. Of significance is the comparison of CFB1 and CFB4, both with wide grid spacing. Figure 5.38 indicates that CFB1, with a short shear span was subjected to higher shear stresses and suffered premature shear failure, whereas CFB4, with the same grid spacing was able to develop its flexural strength without shear failure.

Further discussion of beam shear capacities are given in Chapter 6, where the results are compared with those computed on the basis of CSA Standard S806.

### 5.6.2 Effect of Cyclic Loading

Two of the beams, CFB3 and CFB6 were tested under monotonically increasing lateral loading, while all others were tested under reversed cyclic loading. Each of these two beams had a different shear span, but the same reinforcement, including the same spacing of transverse grids. The effect of cyclic loading could be seen by comparing CFB2 and CFB3 with short shear spans and CFB5 and CFB6 with long shear spans. The envelope curves for these four beams are presented in Figures 5.39. The results indicate that the monotonic force-deformation relationships are similar to the envelopes of hysteretic relationships obtained under cyclic loading, suggesting very little or no effect of cyclic loading on beams.

## 5.7 Conclusion

The following conclusions can be drawn from the beam tests conducted in Phase IV:

- The beams were designed to have 0.39% and 0.26% tension reinforcement ratios in two different directions, respectively. The balanced section reinforcement ratio, based on experimentally measured rupturing strength of 1450 MPa and maximum concrete compressive fiber strain of 0.0035 was 0.39%, exactly equal to the ratio in the strong direction. Therefore, it was expected that the tension reinforcement would rupture prior to significant distress in concrete. Except for CFB1, which was critical in shear, all other beams were critical in flexure and they all failed by rupturing of tension reinforcement at approximately 1.3% strain. All the flexure dominant beams developed their flexural capacities computed on the basis of plane-strain analysis.
- Beam CFB1 had transverse shear reinforcement that was 33% more than that required for shear based on the 45-degree truss analogy (as adopted by CSA S806-02 and CSA A23.3-94), but 22% below the minimum shear reinforcement specified by CSA S806-02. The grid spacing was equal to  $d/2 = 180$  mm. This spacing was too wide to control diagonal cracking and the beam failed due to diagonal tension, which resulted in a failure crack that took place between the grids, implying that the spacing requirement for shear should be more stringent than that specified in CSA A23.3-94.

- The force-deformation envelope for FRP reinforced beams under reversed cyclic loading is approximately the same as that obtained under monotonically increasing lateral force. This implies that a monotonic curve would provide sufficient strength envelope for hysteretic relationships of FRP reinforced beams.
- The hysteretic relationship of flexure dominant FRP reinforced beams indicate stiffness degradation under cyclic loading, due to progressive cracking of concrete and associated softening in the member, developing approximately 3% lateral drift ratio prior to failure. This level of lateral drift may be considered to be sufficient for earthquake resistant construction. However, the beams were not able to develop inelasticity to dissipate seismic induced energy, other than developing some minor energy dissipation mechanism due to the cracking of concrete. The softening of flexural rigidity due to concrete cracking was observed to be more significant than that in steel reinforced members, due to the relatively low elastic modulus of FRP reinforcement.

**Table 5.1. Properties of Phase IV Specimens**  
(Beams under Reversed Cyclic and Monotonic Loading)

| Specimen Label                                                                                                                | $f'_c$<br>(MPa) | Reinf<br>Arrang. | $\rho$<br>(%) | $\rho$<br>(%) | s<br>(mm) | L<br>(mm) |
|-------------------------------------------------------------------------------------------------------------------------------|-----------------|------------------|---------------|---------------|-----------|-----------|
|                                                                                                                               |                 |                  | (strong-side) | (weak-side)   |           |           |
| CFB1                                                                                                                          | 35              | 10-9.5 mm bar    | 0.39          | 0.26          | 175       | 1000      |
| (A <sub>F</sub> =6-9.5 mm at top side & A' <sub>F</sub> =4-9.5 mm at bottom side of beam) ( $\rho=A_F/bd$ , $\rho'=A'_F/bd$ ) |                 |                  |               |               |           |           |
| CFB2                                                                                                                          | 35              | 10-9.5 mm bar    | 0.39          | 0.26          | 88        | 1000      |
| CFB3                                                                                                                          | 35              | 10-9.5 mm bar    | 0.39          | 0.26          | 88        | 1000      |
| CFB4                                                                                                                          | 35              | 10-9.5 mm bar    | 0.39          | 0.26          | 175       | 1900      |
| CFB5                                                                                                                          | 35              | 10-9.5 mm bar    | 0.39          | 0.26          | 88        | 1900      |
| CFB6                                                                                                                          | 35              | 10-9.5 mm bar    | 0.39          | 0.26          | 88        | 1900      |

**Table 5.2: Variation of Concrete Strength with Age in Phase IV Specimens**  
(Beams under Reversed Cyclic and Monotonic Loading)

| <b>Compressive Strength of Concrete in Mpa</b> |        |         |         |         |              |
|------------------------------------------------|--------|---------|---------|---------|--------------|
| Age                                            | 7 days | 14 days | 21 days | 28 days | During Tests |
| Cylinder 1                                     | 26     | ---     | 31.6    | 35      | 42           |
| Cylinder 2                                     | 25     | ---     | 31      | 36      | 38           |
| Cylinder 3                                     | 26.8   | ---     | 32.4    | 35.4    | 41           |
| Average                                        | 25.9   | ---     | 31.7    | 35.55   | 40.3         |

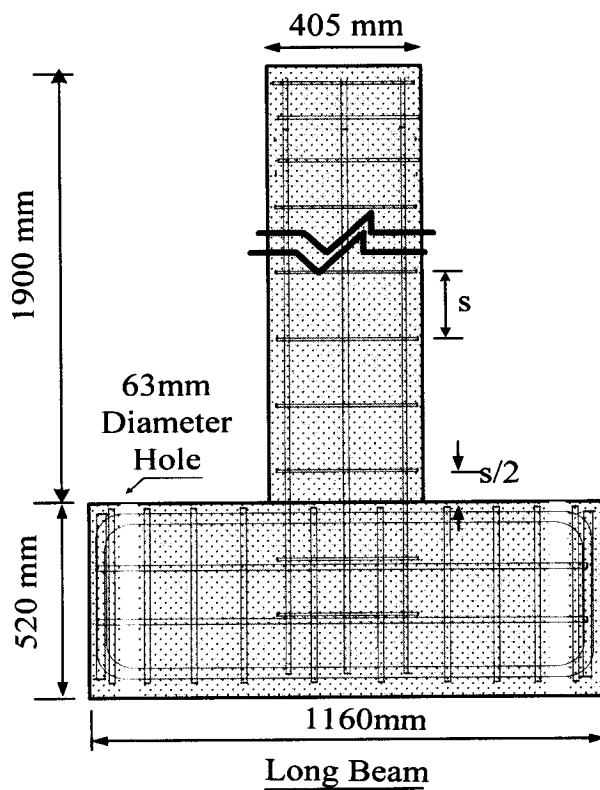
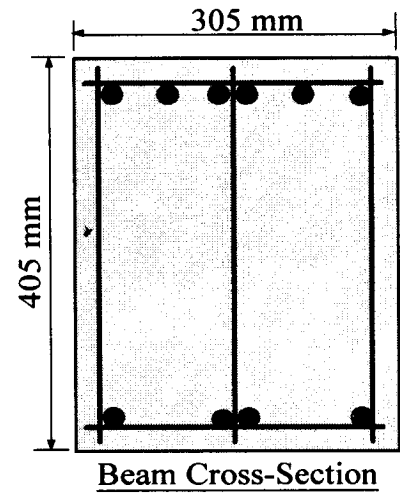
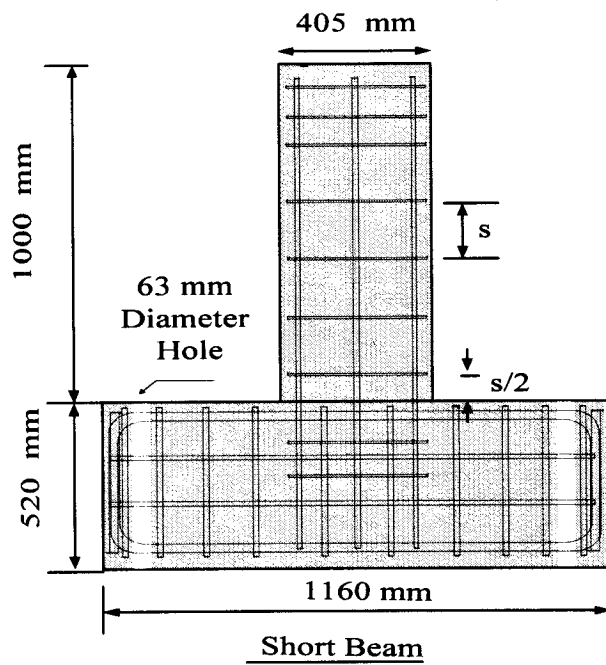
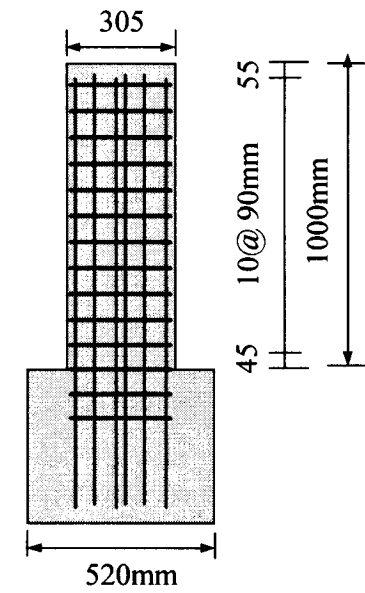
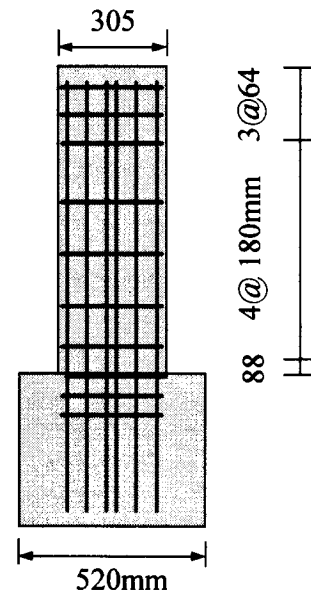


Figure 5.1: Geometry of Phase IV Specimens  
(Beams Subjected to Cyclic or Monotonic Loading)

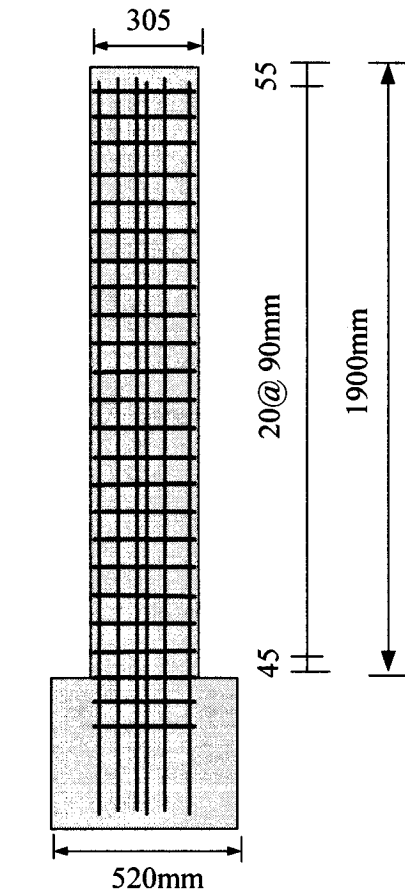




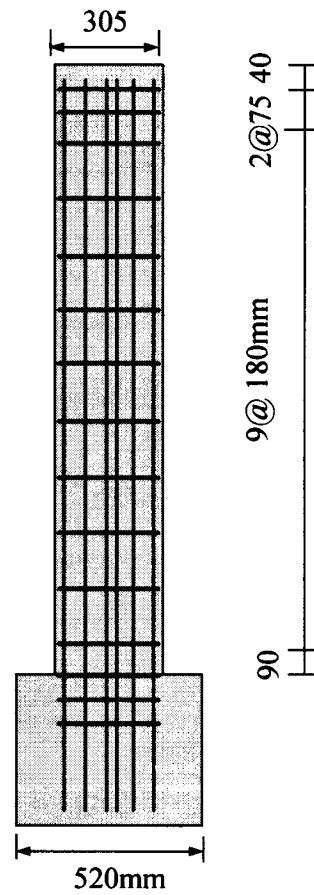
Beams CFB2 & CFB3



Beam CFB1



Beams CFB5 & CFB6



Beam, CFB4

Figure 5.2: Elevation Views of Beam Reinforcement Cages

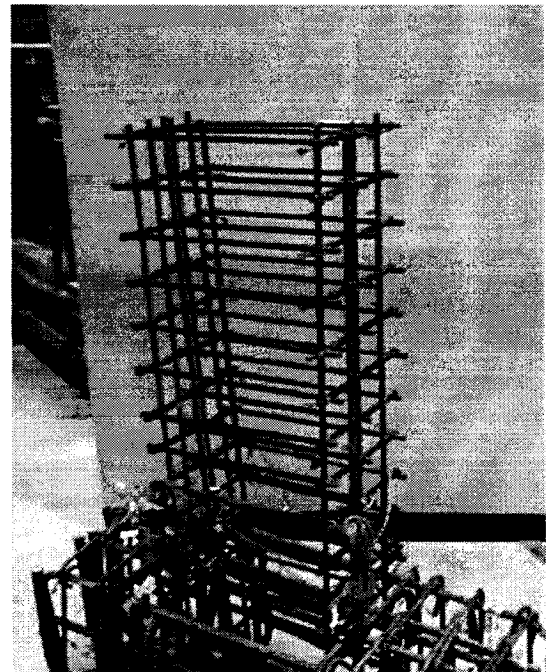
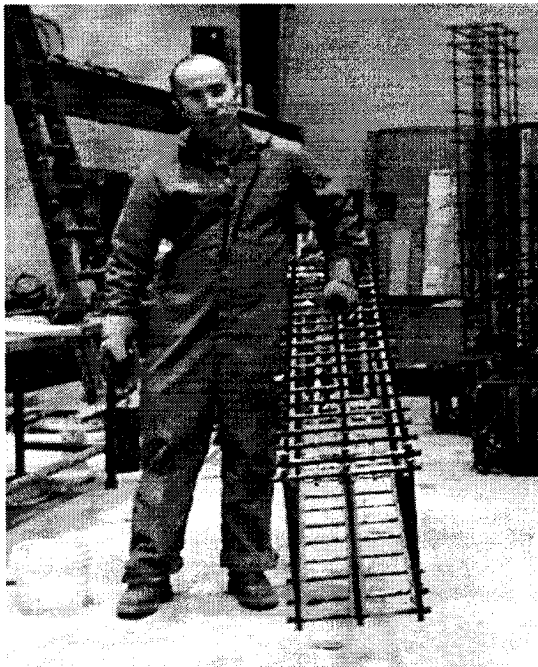
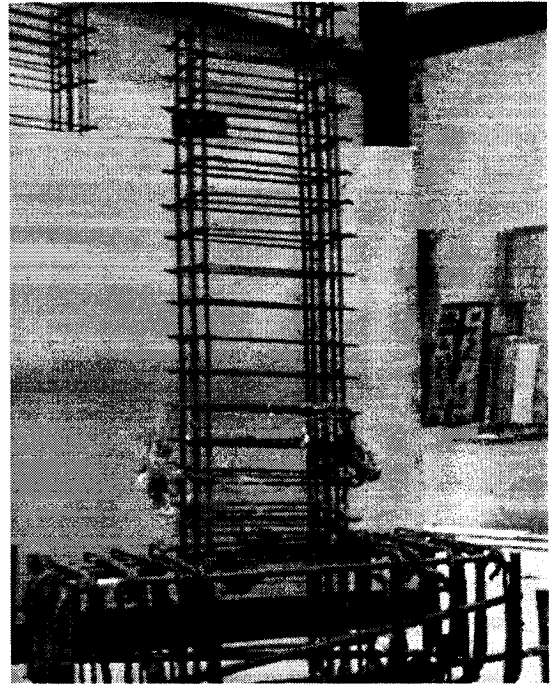
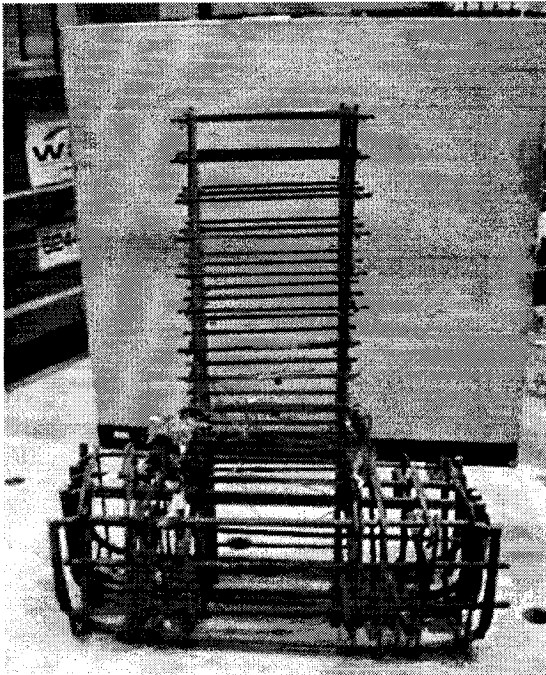


Figure 5.3: CFRP Cages used in Phase IV Beams

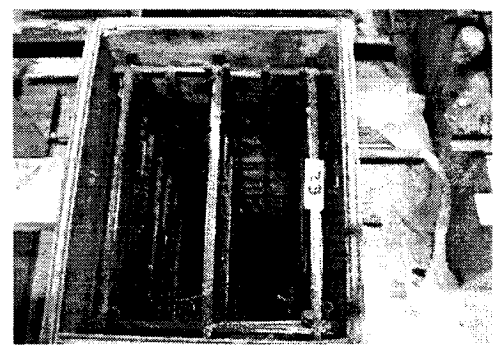
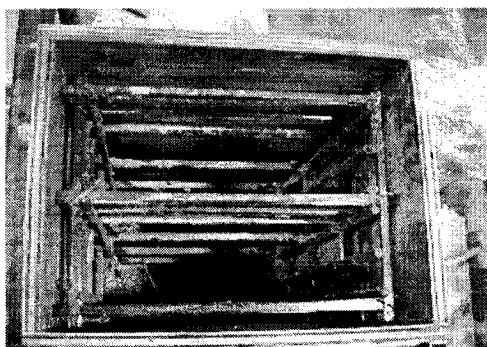
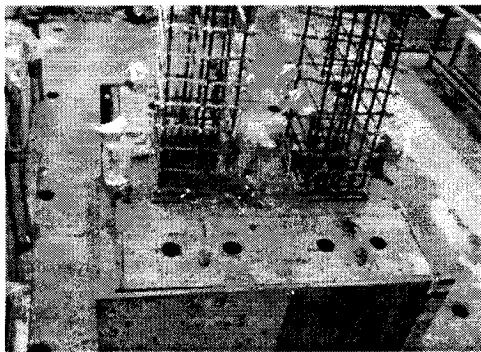
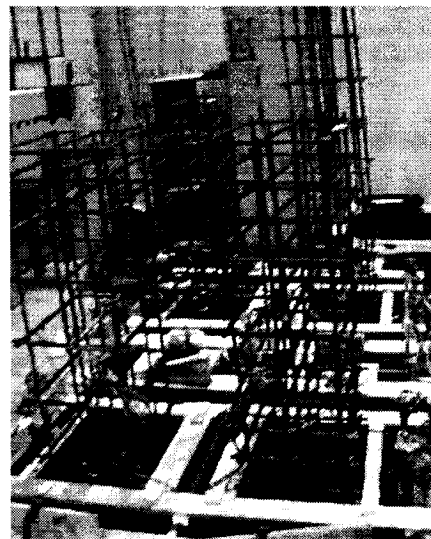
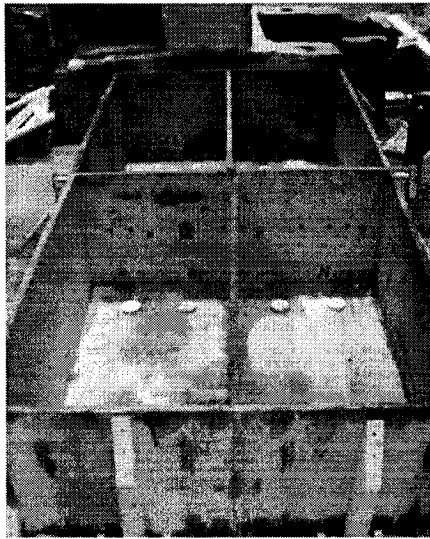
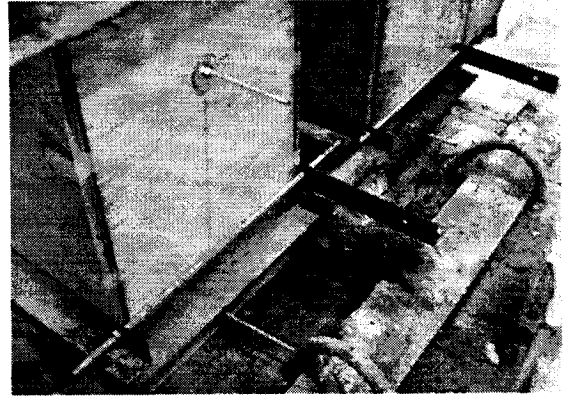
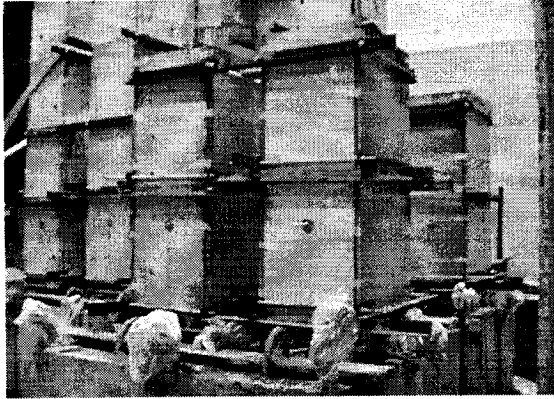
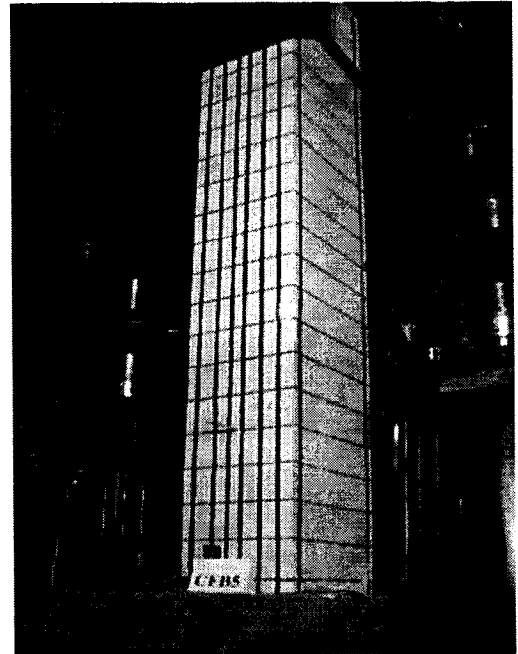
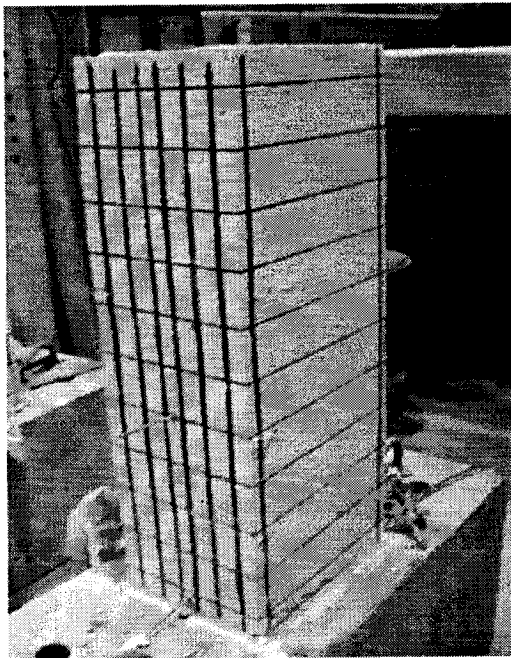


Figure 5.4: Preparation for Concrete Casting



(a) Formwork before Casting Concrete



(b) Preparation of Specimens after Casting Concrete

Figure 5.5: Casting of Beams

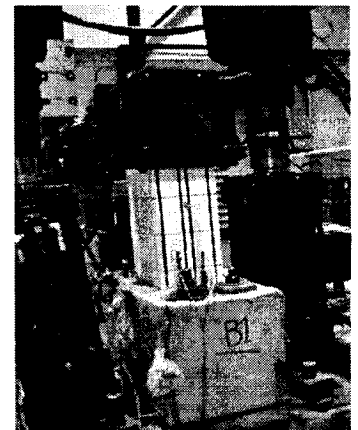
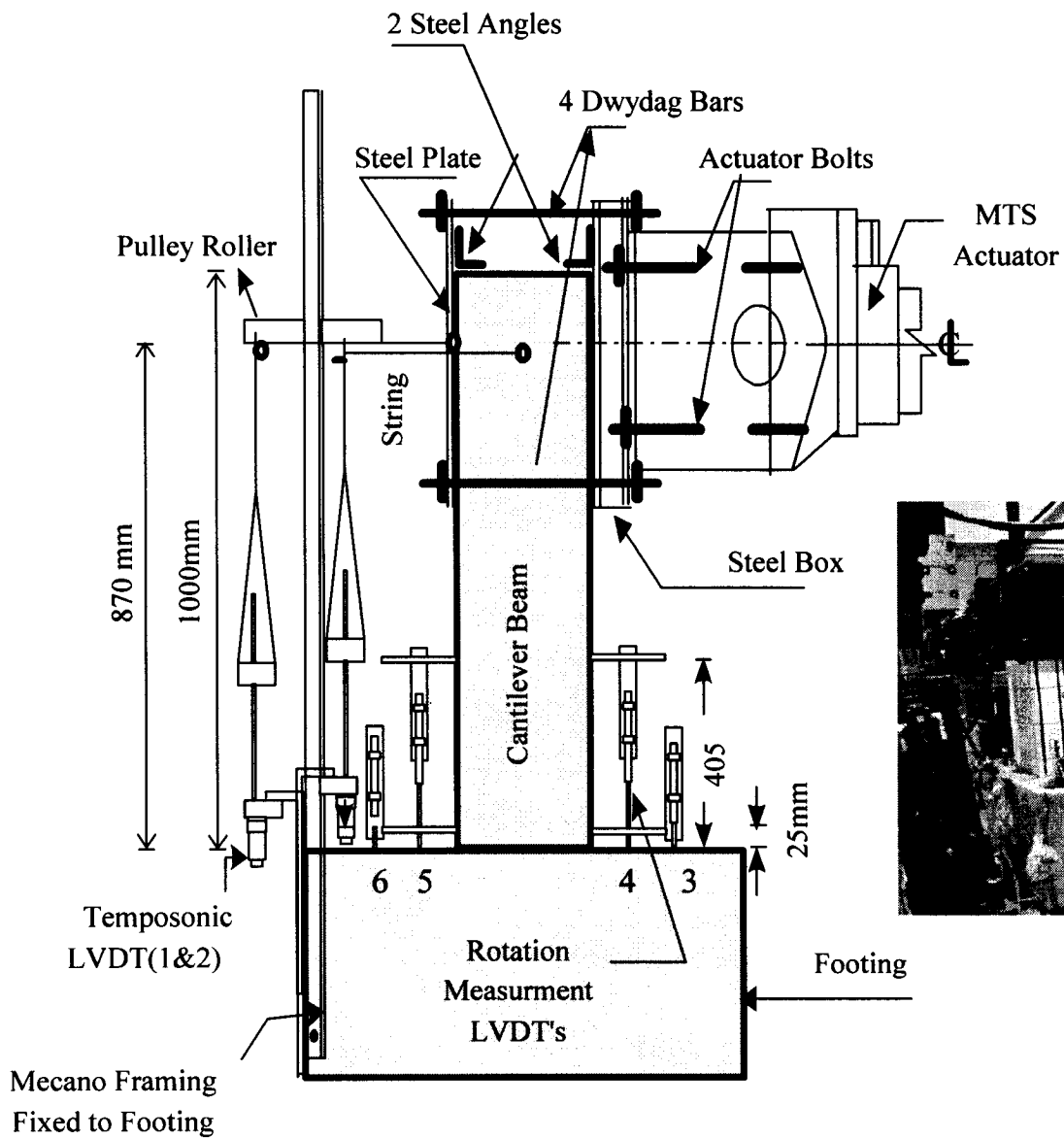
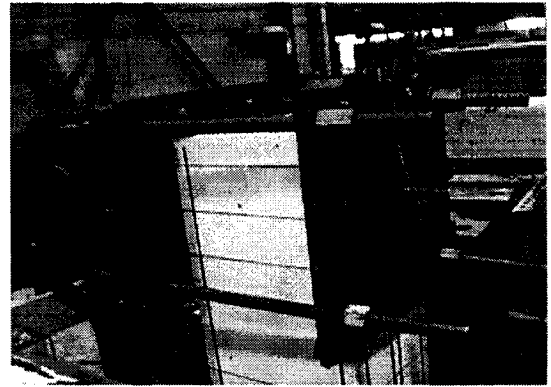
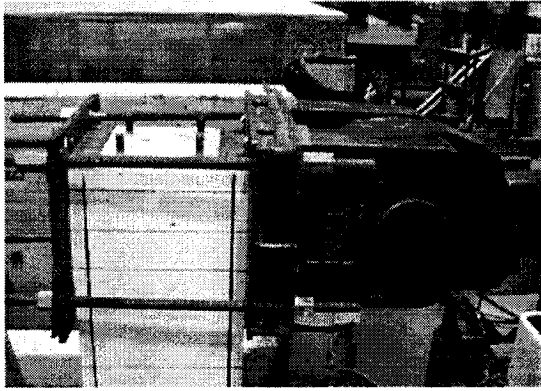
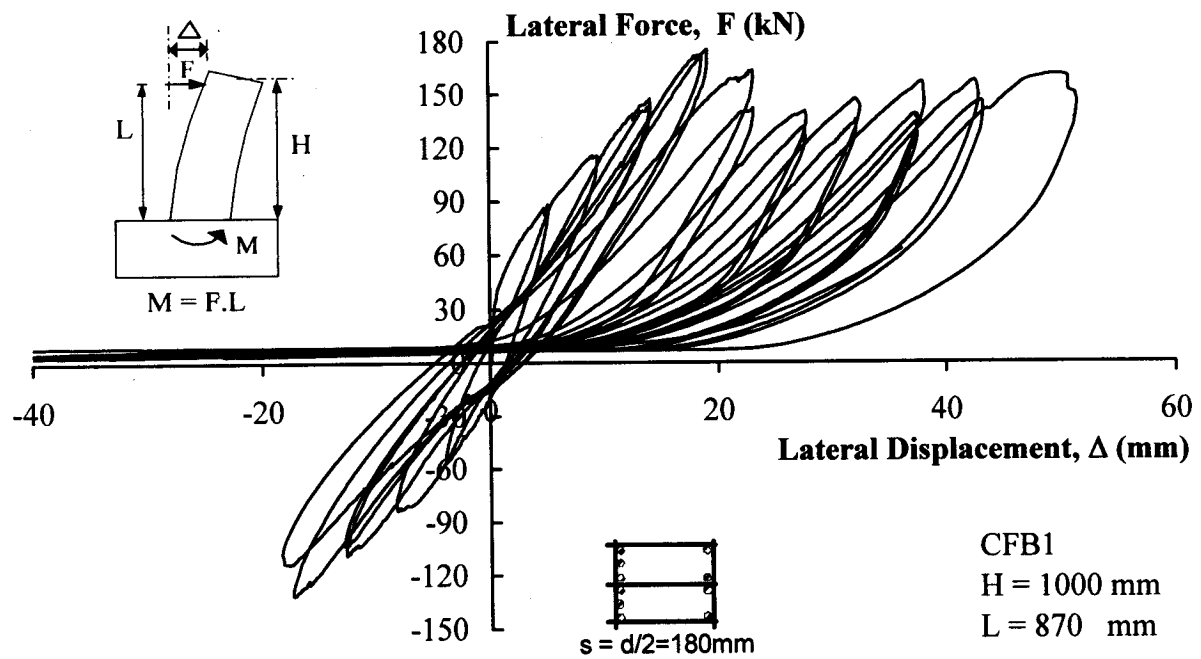
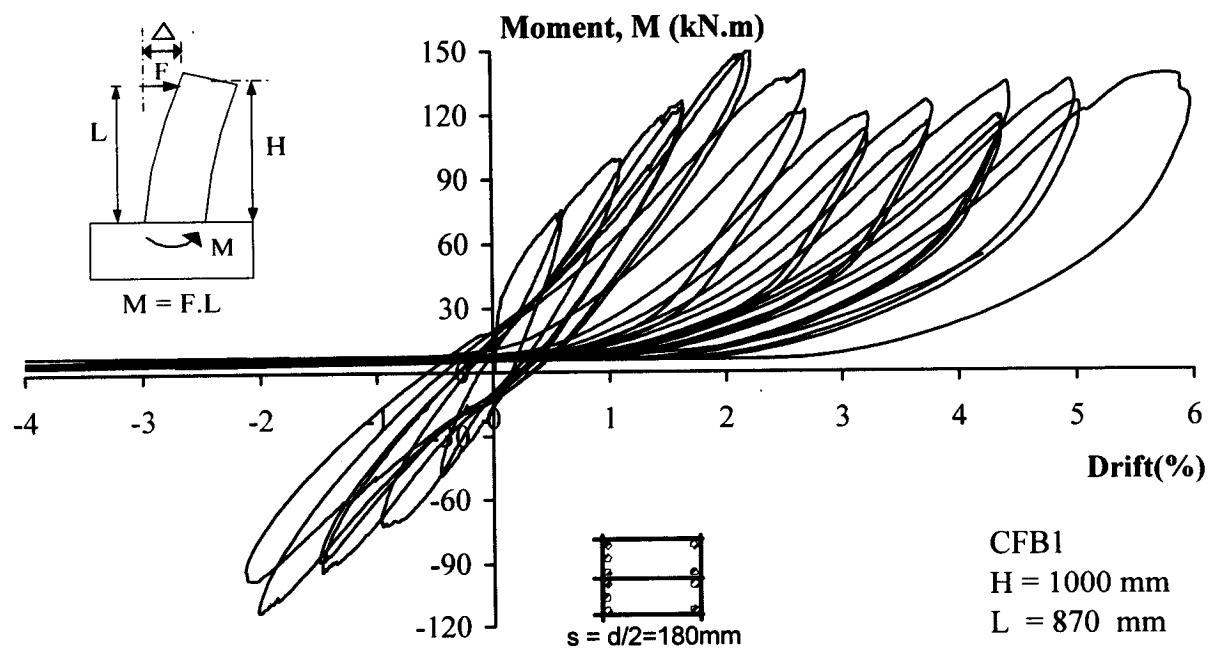


Figure 5.6: Locations of LVDTs and the Test Setup for a Short Beam



(a) Lateral Force-Lateral Displacement Relationship



(b) Moment-Drift Relationship

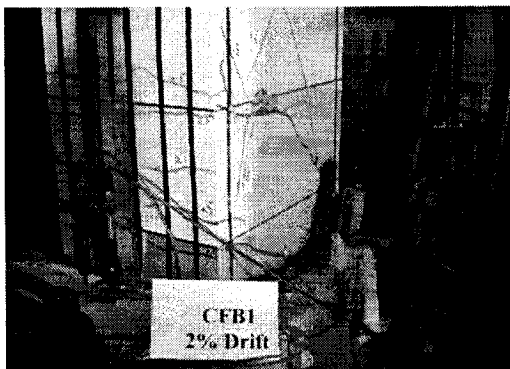
Figure 5.7: Hysteretic Behavior for Beam CFB1



(a) Before Loading



(b) At 1% Drift



(c) At 2% Drift



(d) At 2.5% Drift



Figure 5.8: Observed Damage in Beam CFB1 at Selected Stages of Loading

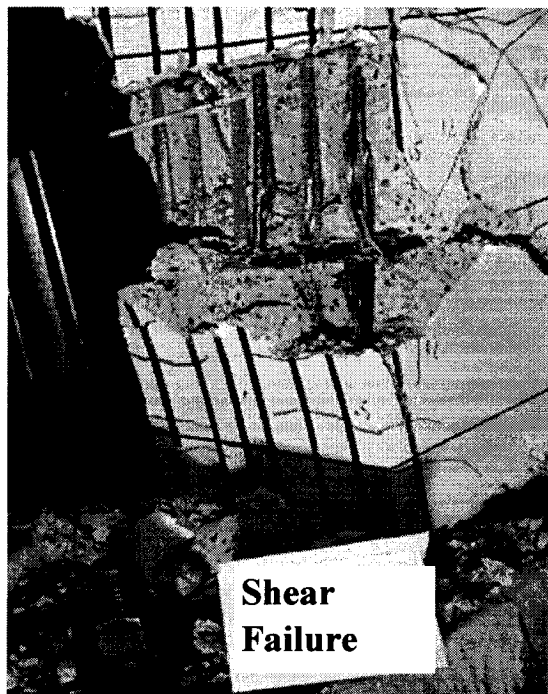




(e) At 3.5% Drift



(f) At 4.5% Drift



(j) Beam Failure



(k) Bars Rupturing

Figure 5.8(Cont.): Observed Damage in Beam CFB1 at Selected Stages of Loading



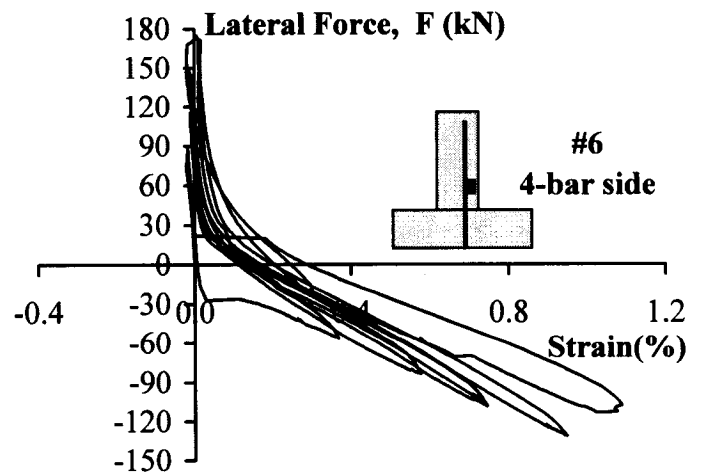
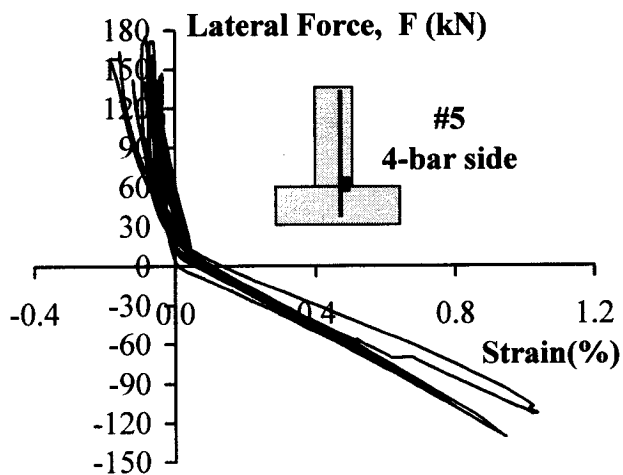
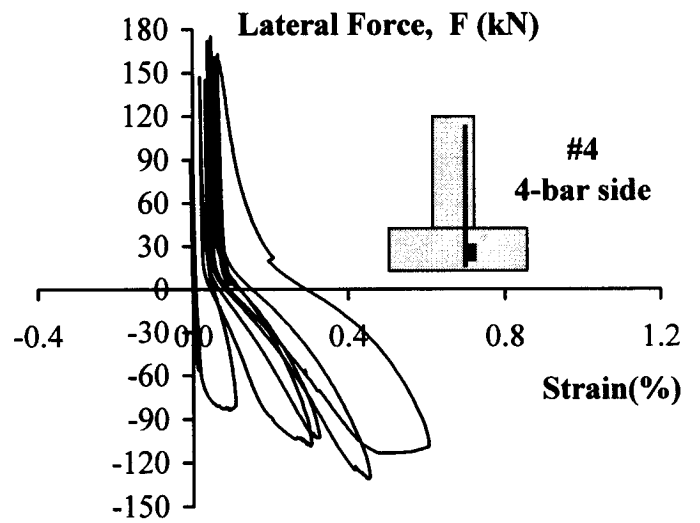
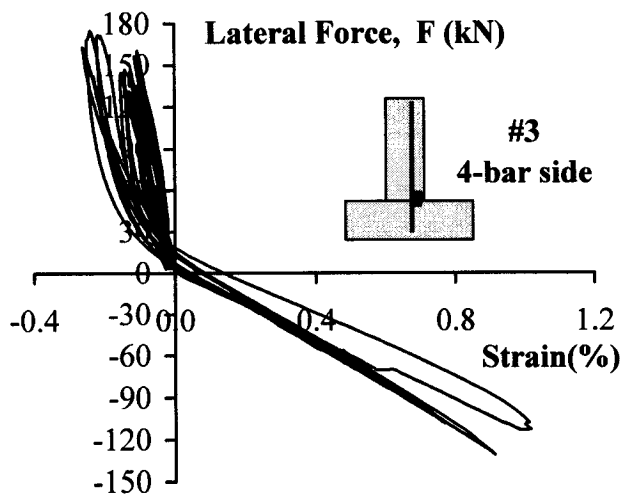
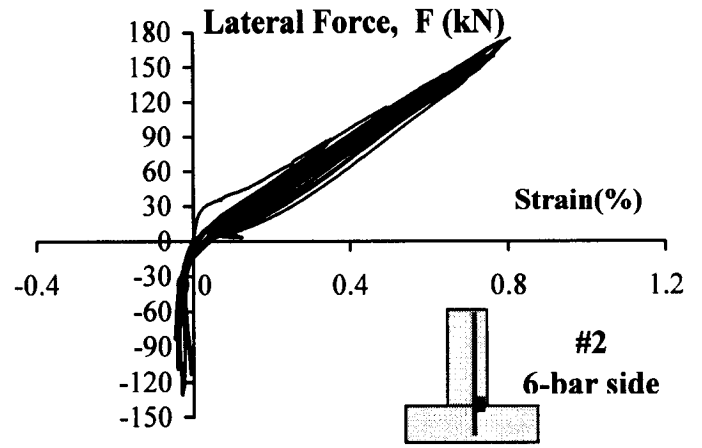
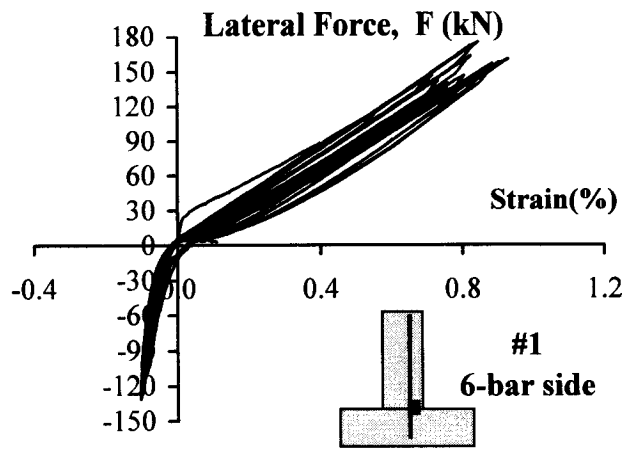


Figure 5.9: Bar Strain Readings in Beam CFB1

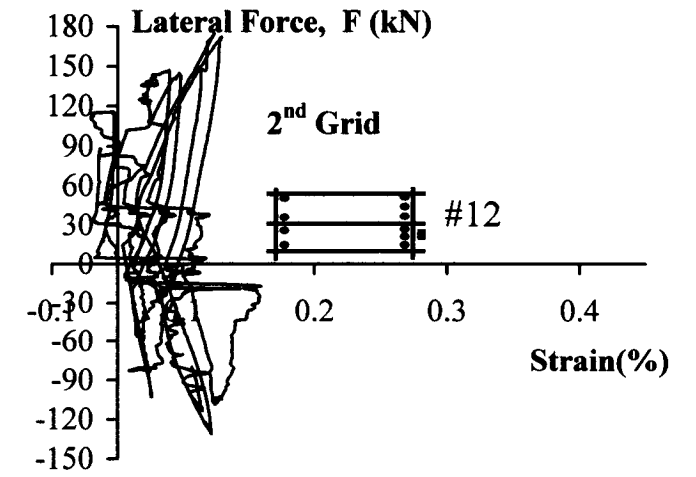
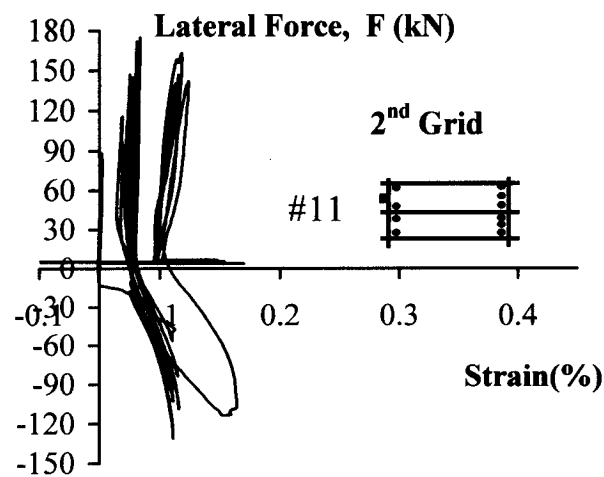
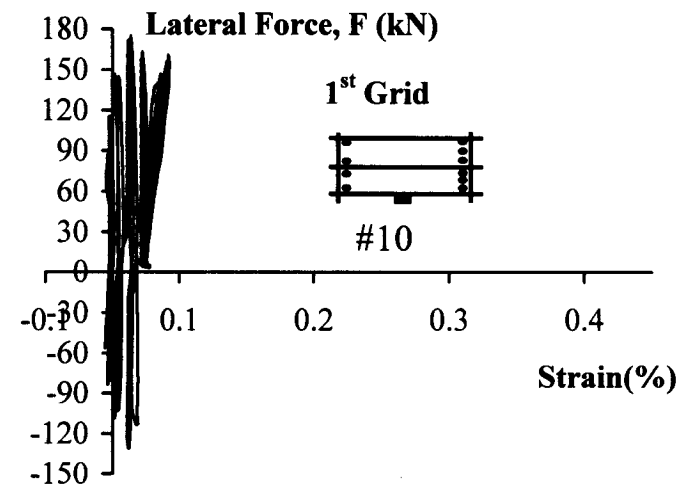
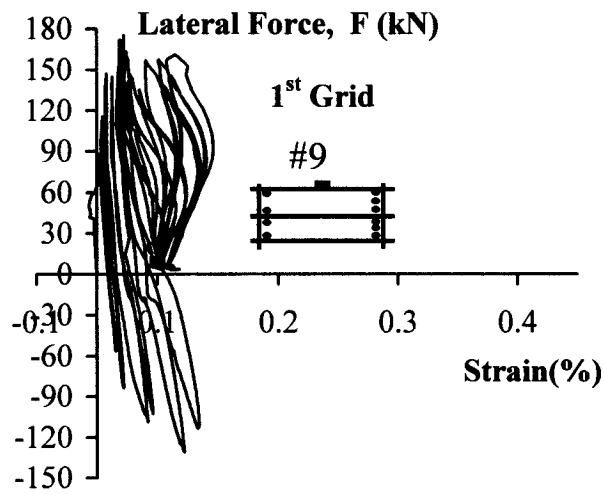
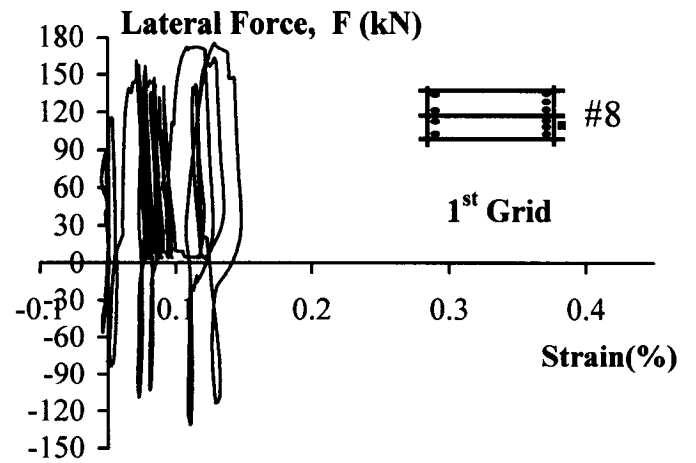
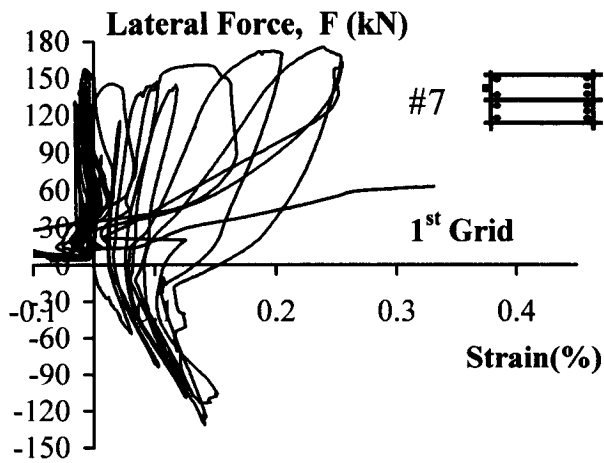


Figure 5.10: Grid Strain Readings in Beam CFB1

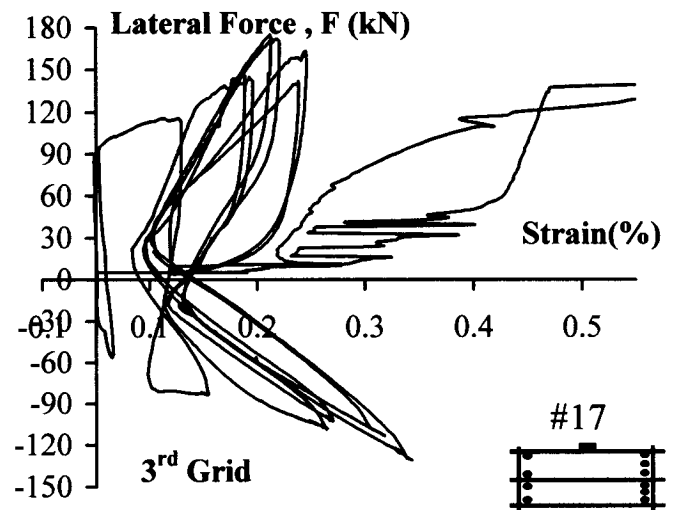
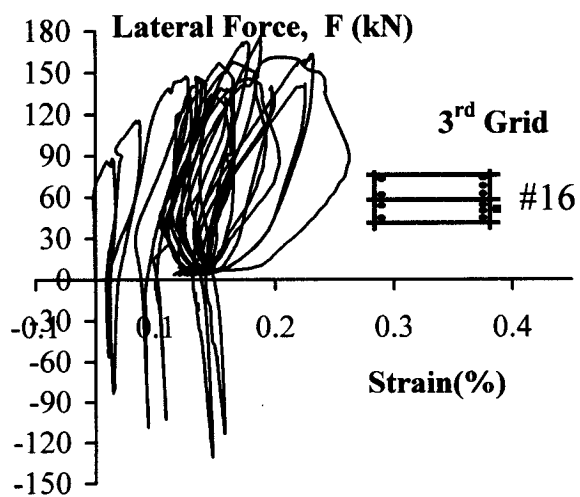
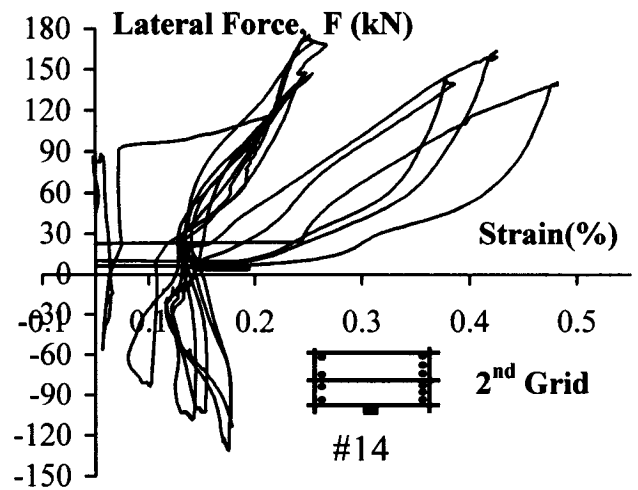
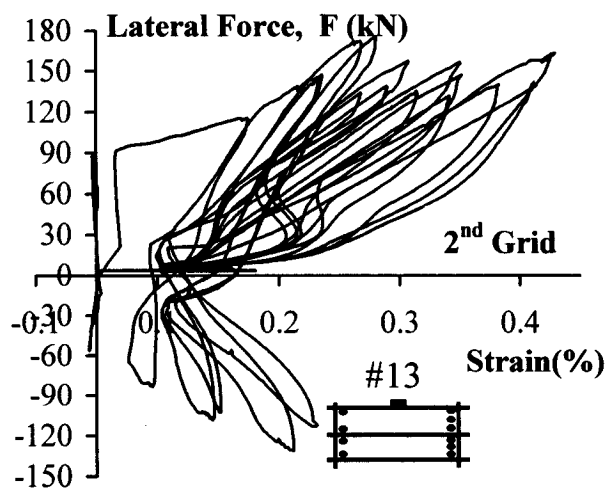
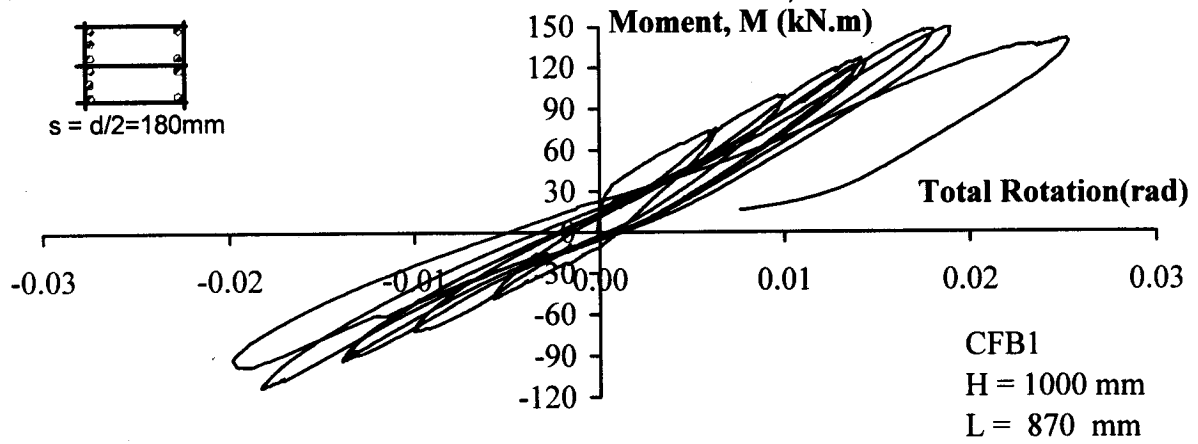
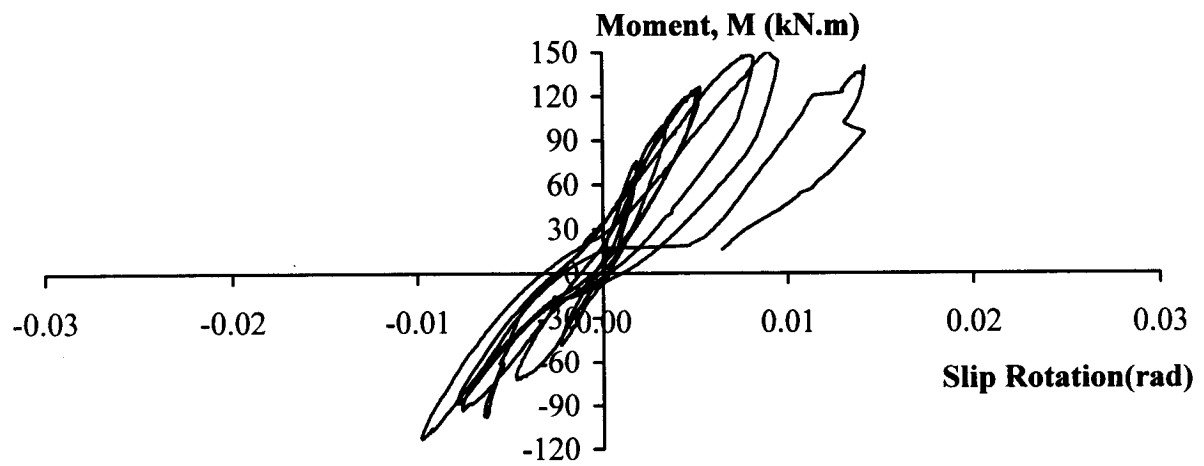


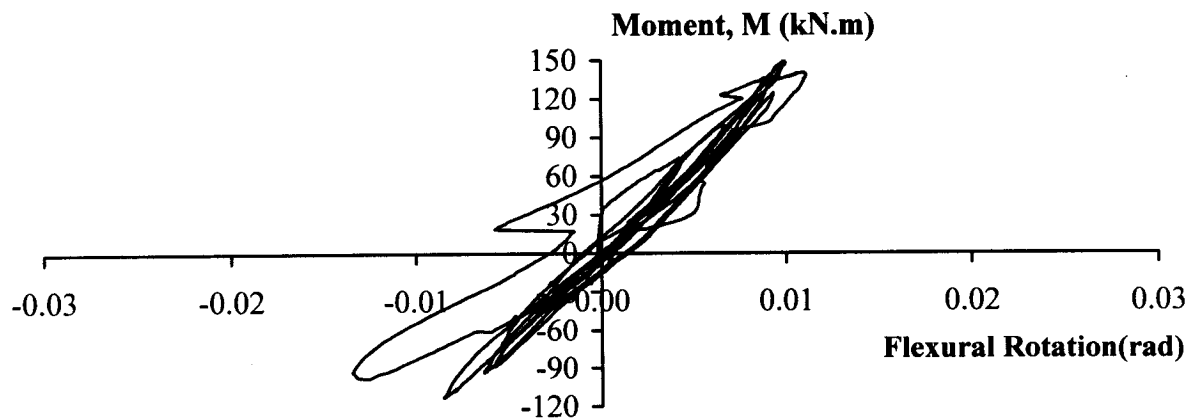
Figure 5.10(Cont.): Grid Strain Readings in Beam CFB1



(a) Moment-Total Rotation Relationship

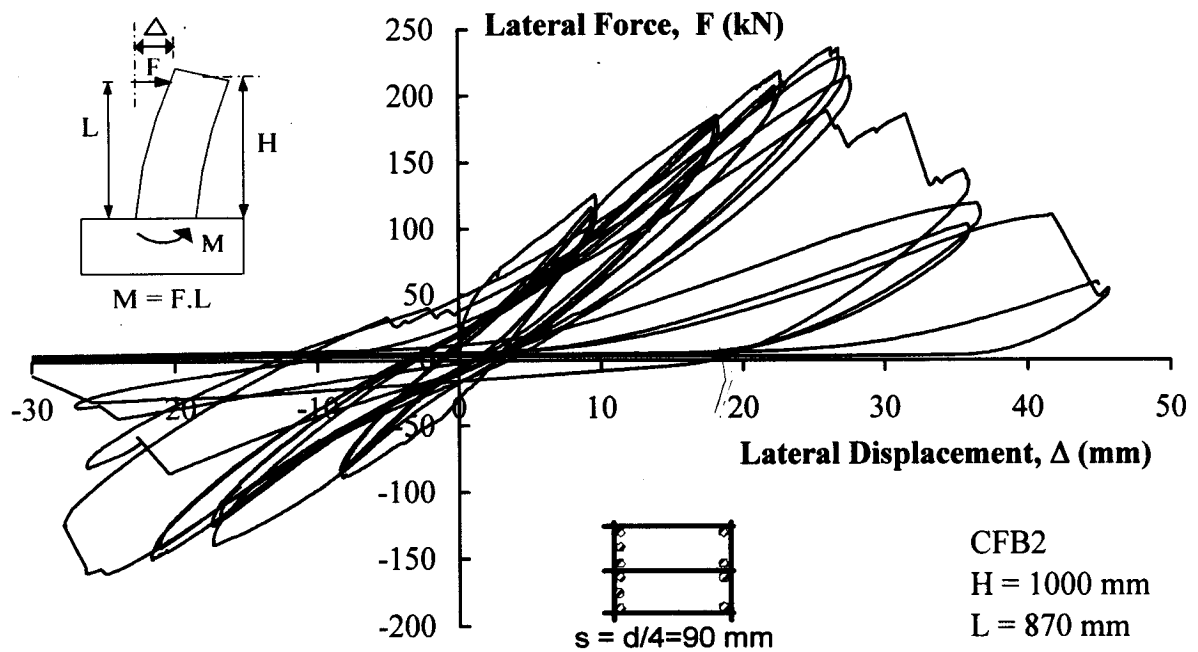


(b) Moment-Slip Rotation Relationship

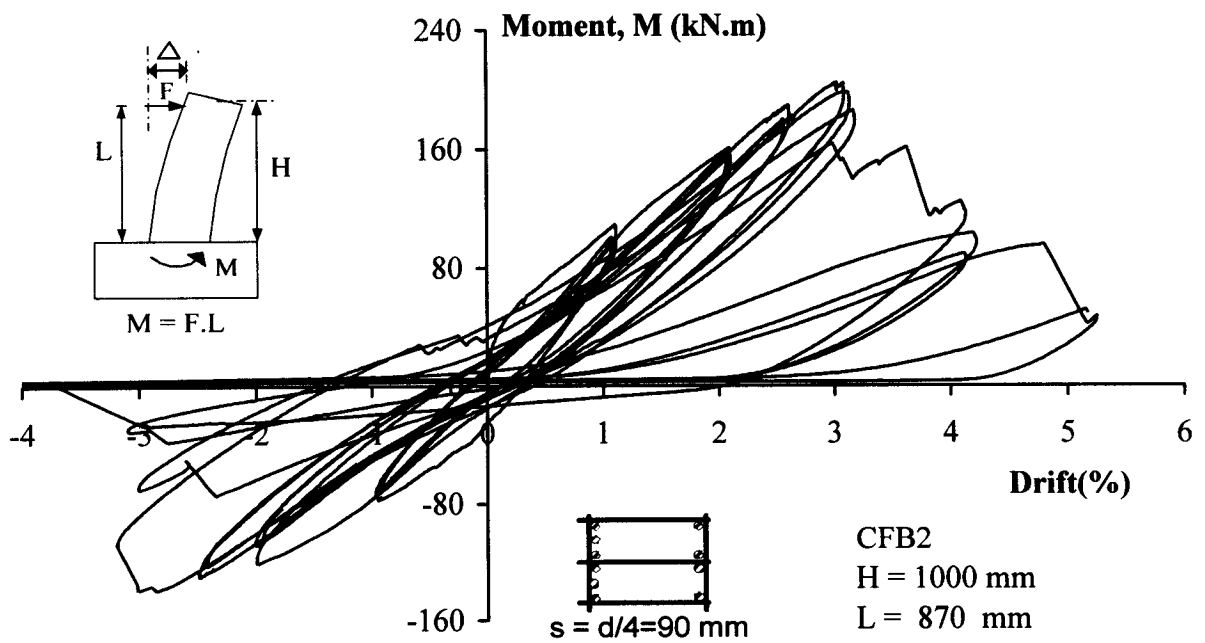


(c) Moment-Flexural Rotation Relationship

Figure 5.11: Hysteretic Moment-Rotation Relationships for Beam CFB1

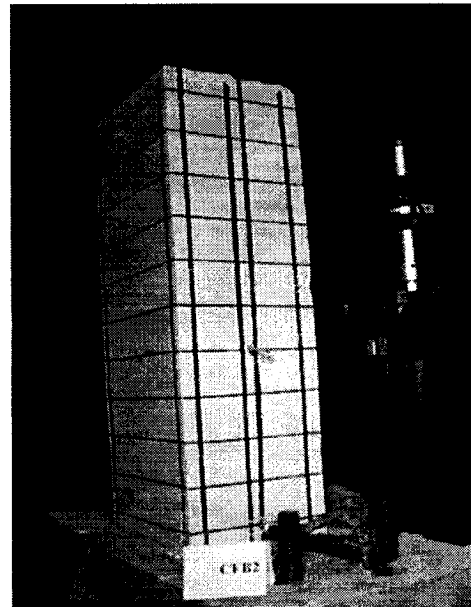
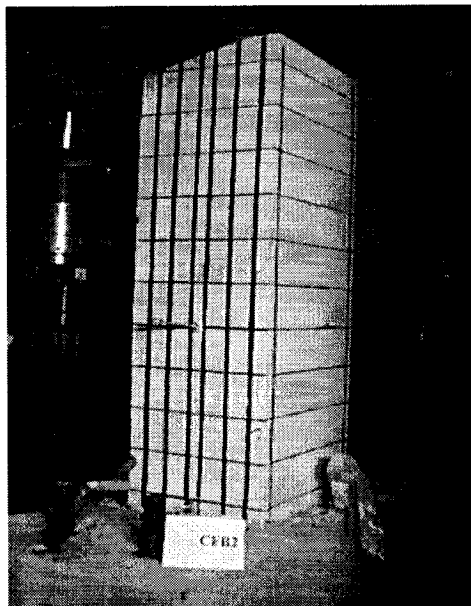


(a) Lateral Force-Lateral Displacement Relationship

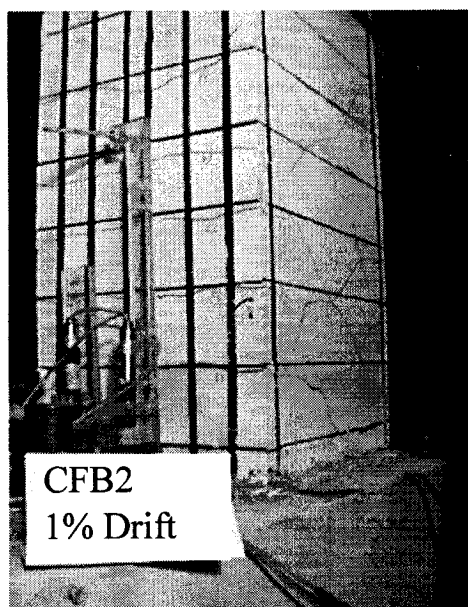


(b) Moment-Drift Relationship

Figure 5.12: Hysteretic Behavior for Beam CFB2



(a) Before Loading



(b) At 1% Drift

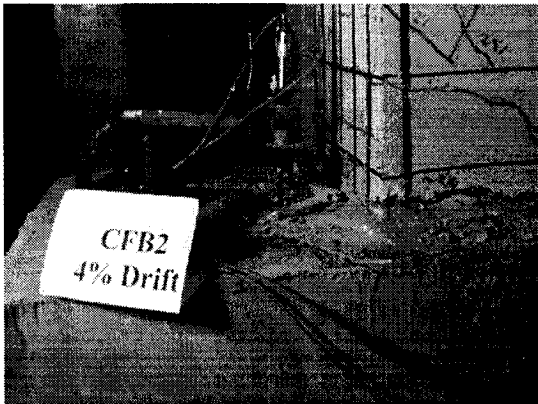


(c) At 2% Drift

Figure 5.13: Observed Damage in Beam CFB2 at Selected Stages of Loading



(d) At 3% Drift



(e) At 4% Drift



(f) At 5% Drift



(g) Rupturing of Bars in Tension

Figure 5.13(Cont.): Observed Damage in Beam CFB2 at Selected Stages of Loading

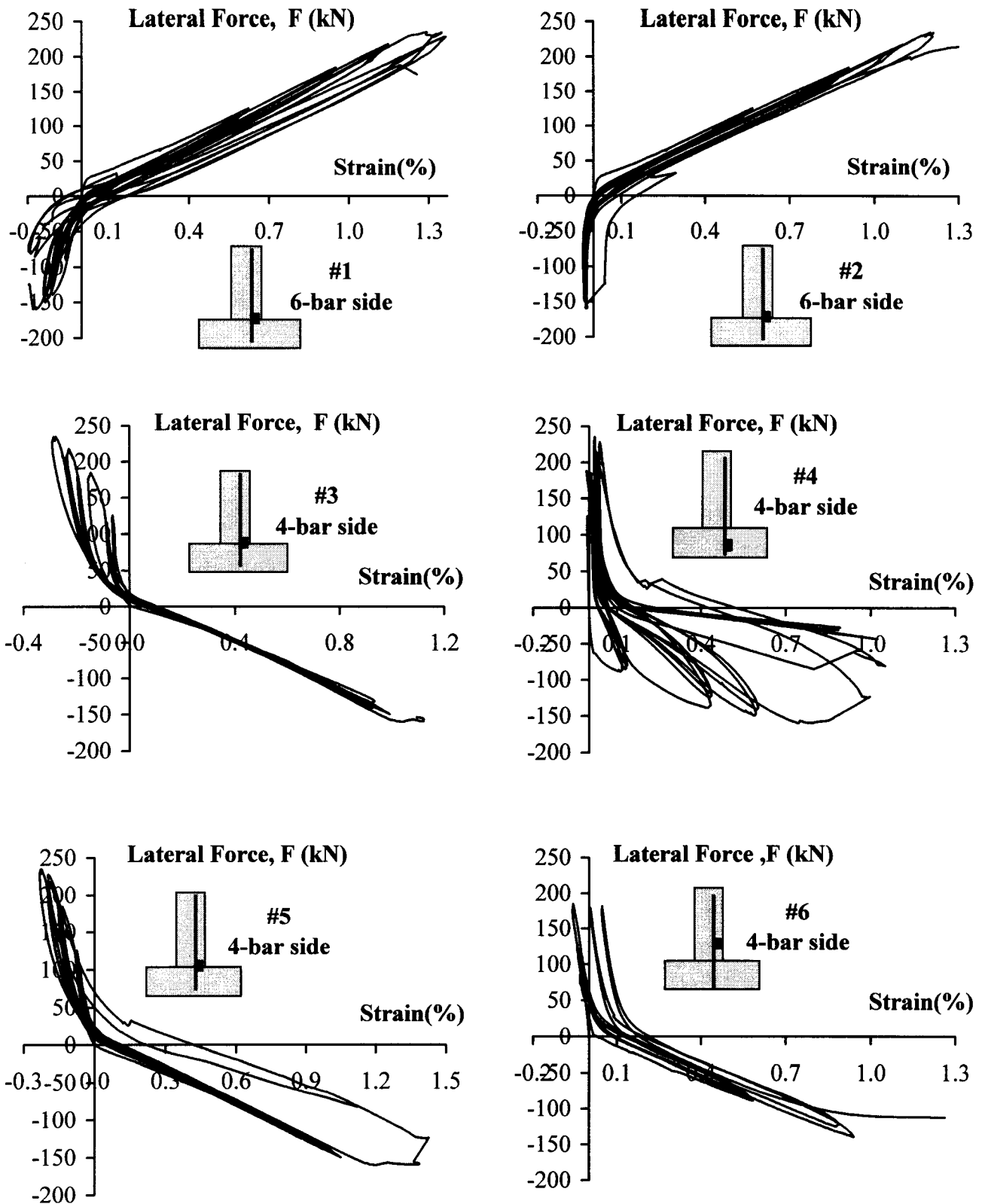


Figure 5.14: Bar Strain Readings in Beam CFB2



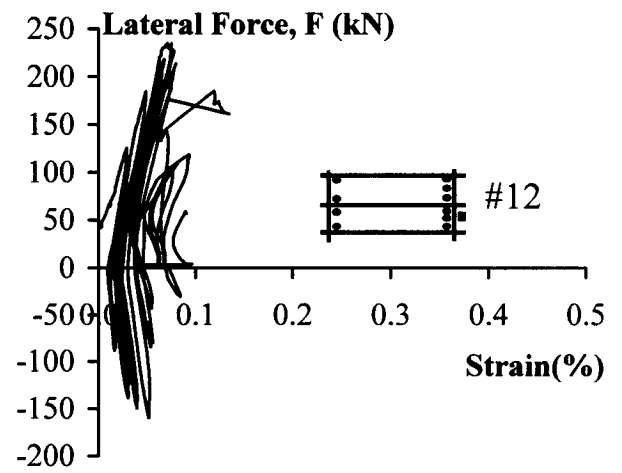
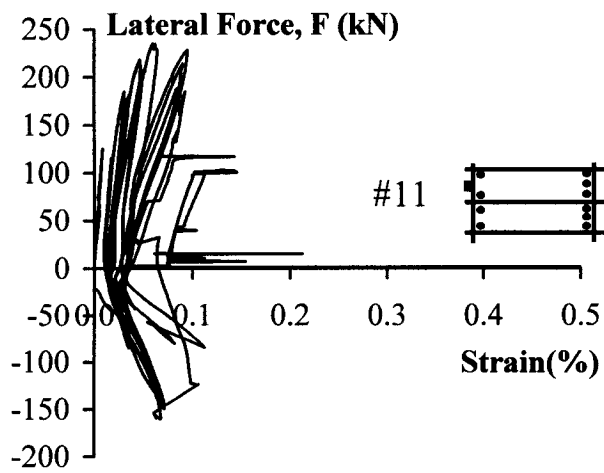
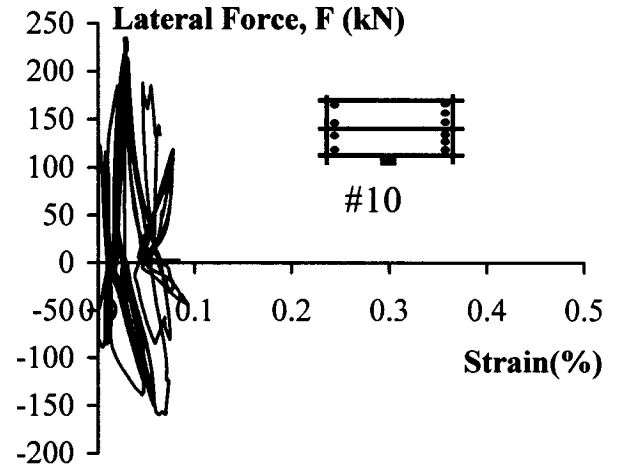
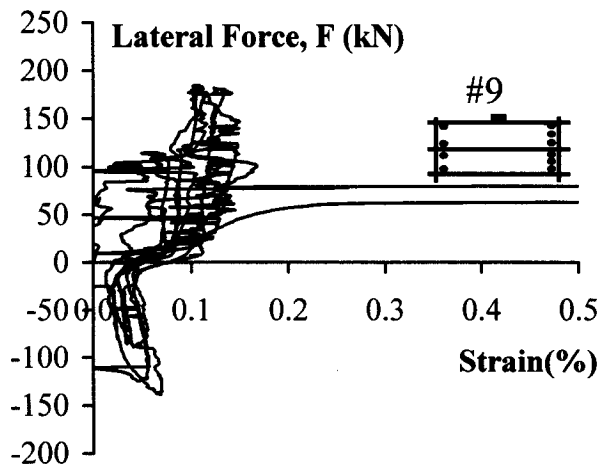
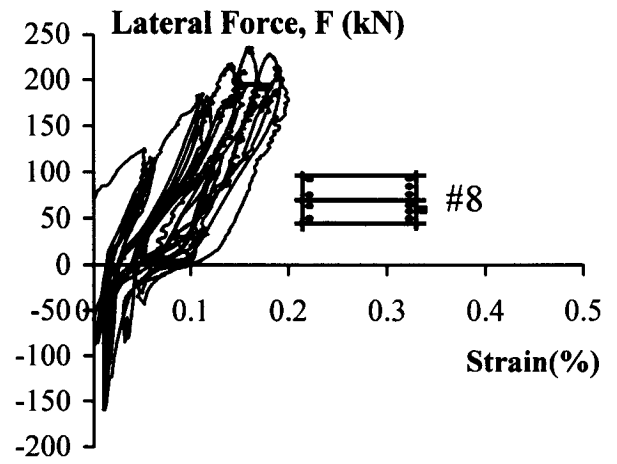
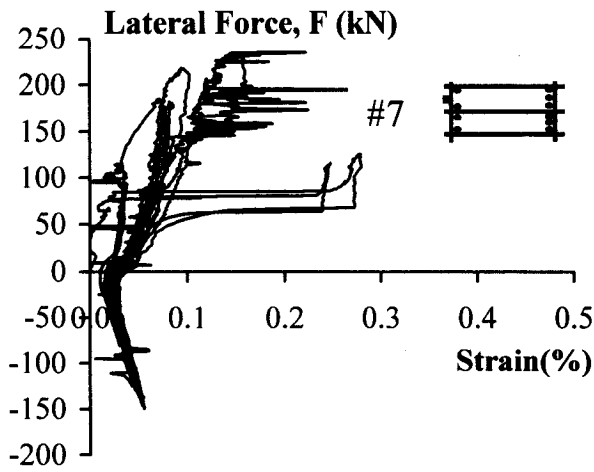


Figure 5.15: Grid Strain Readings in Beam CFB2

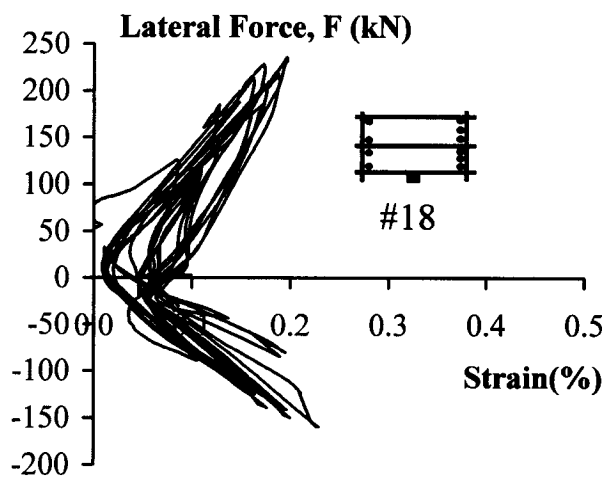
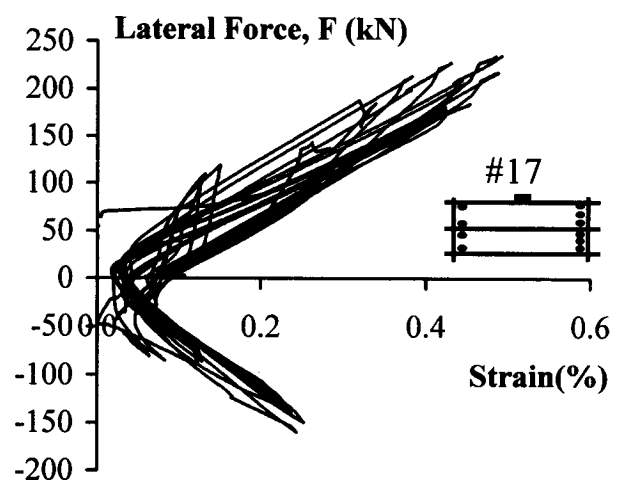
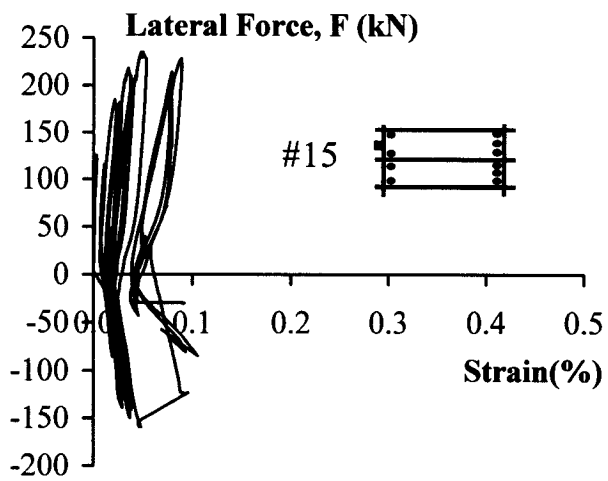
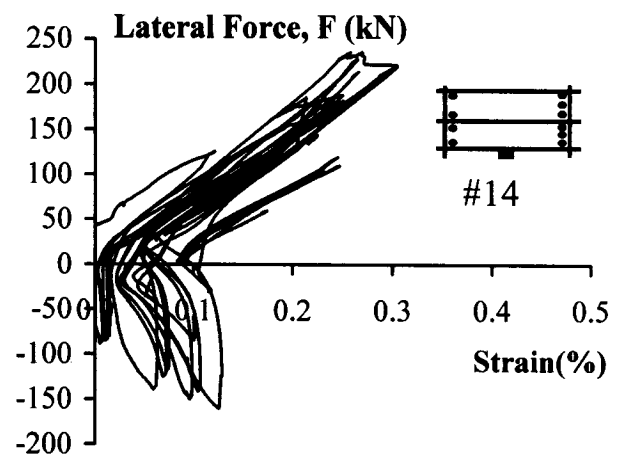
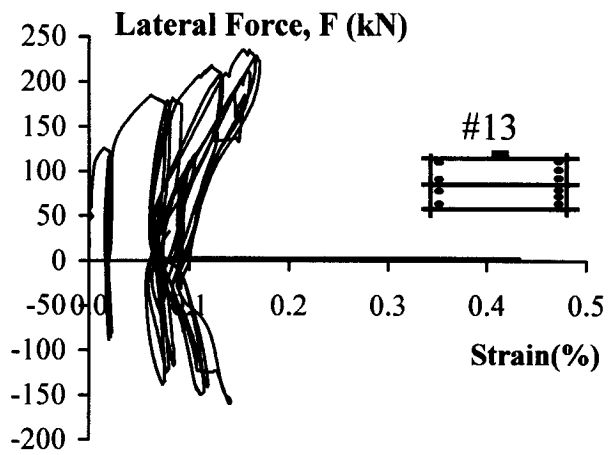
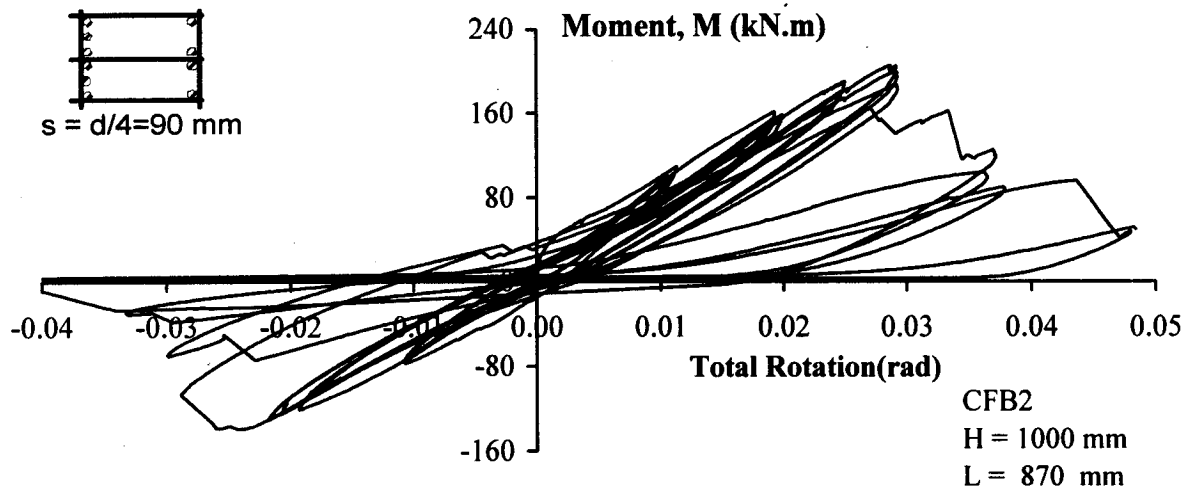
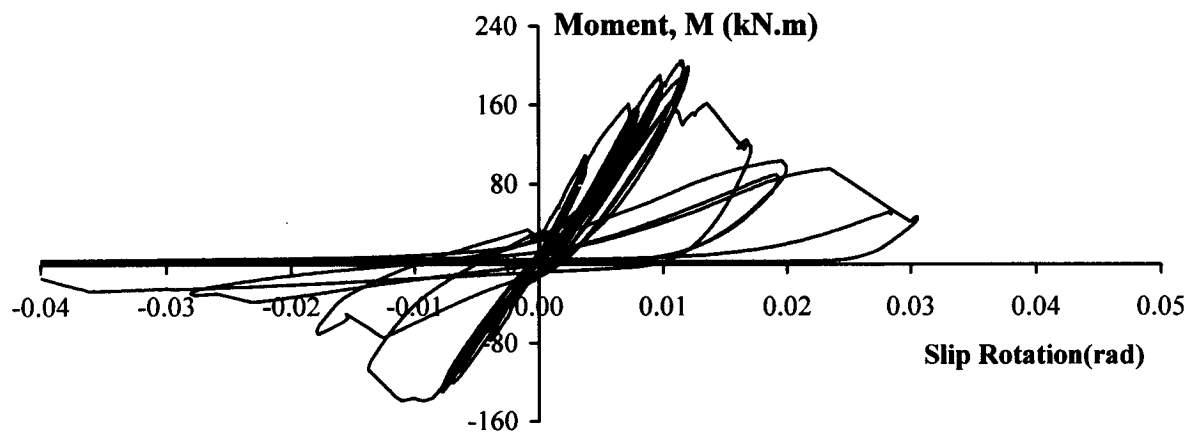


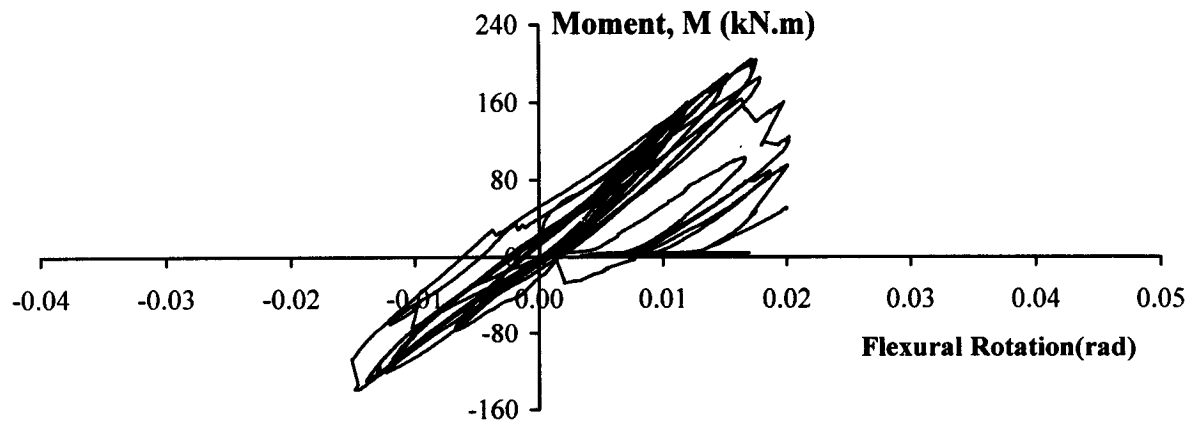
Figure 5.15 (Cont.): Grid Strain Readings in Beam CFB2



(a) Moment-Total Rotation Relationship

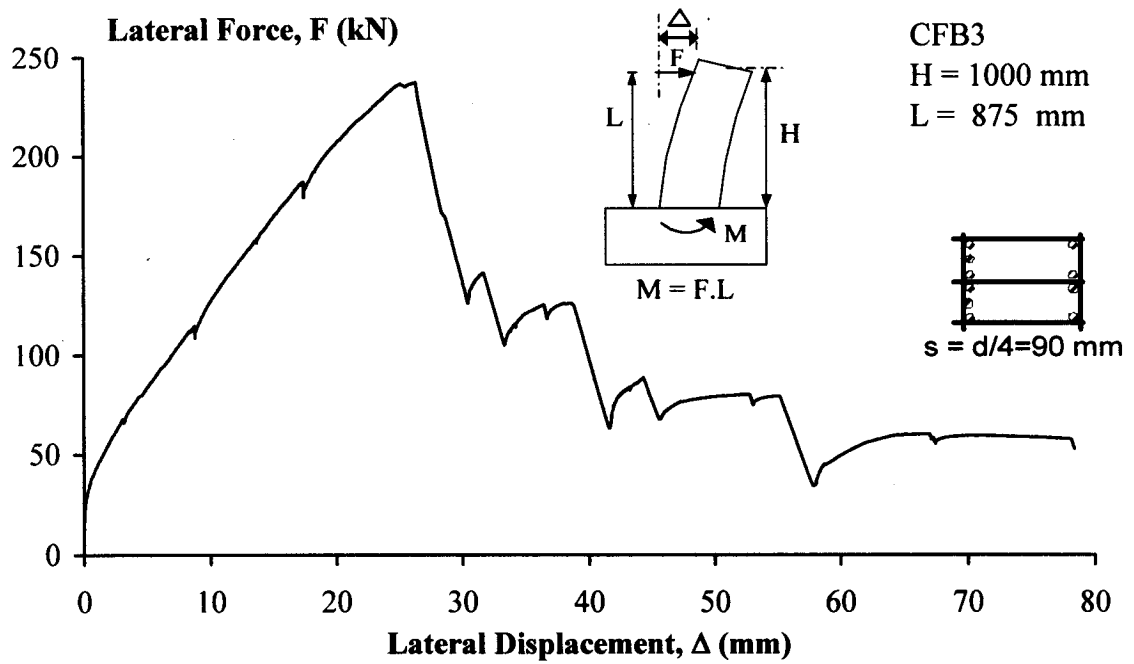


(b) Moment-Slip Rotation Relationship

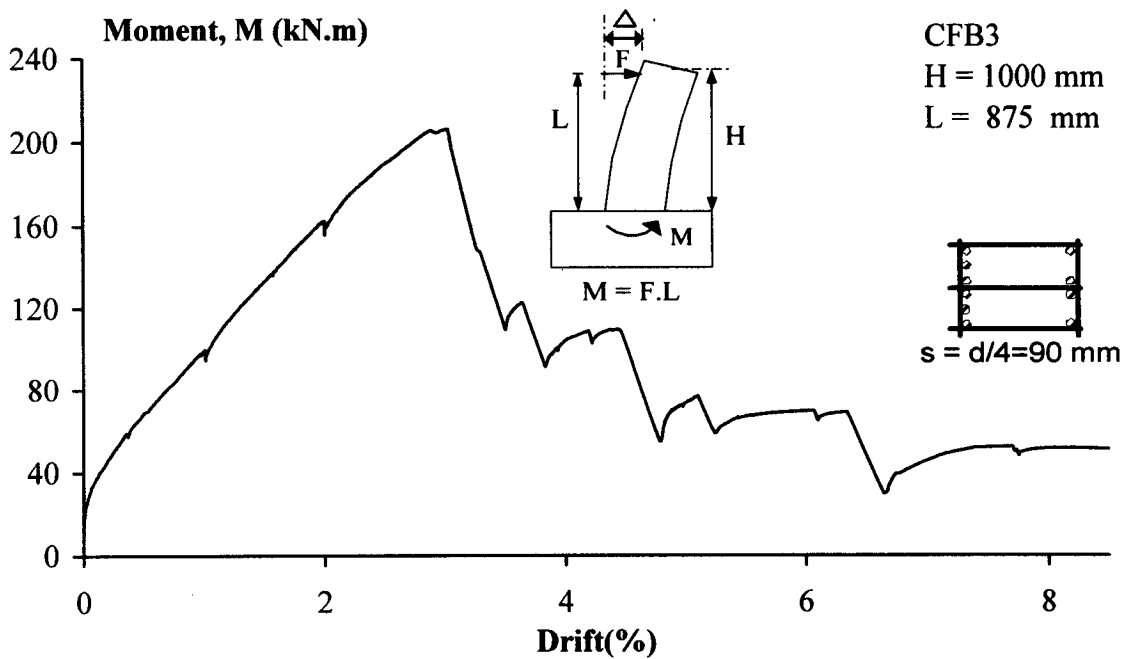


(c) Moment-Flexural Rotation Relationship

Figure 5.16: Hysteretic Moment-Rotation Relationships for Beam CFB2

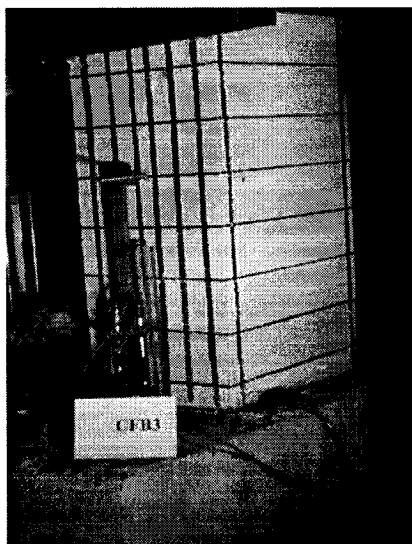


(a) Lateral Force-Lateral Displacement Relationship

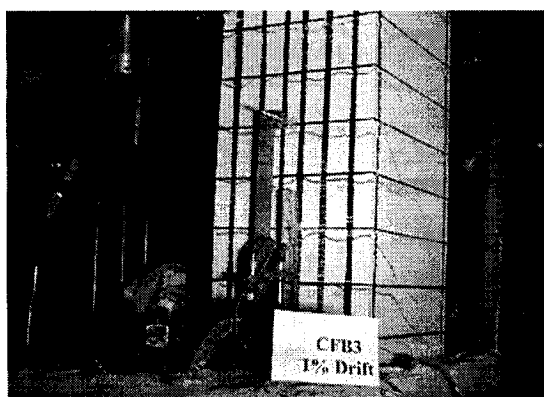


(b) Moment-Drift Relationship

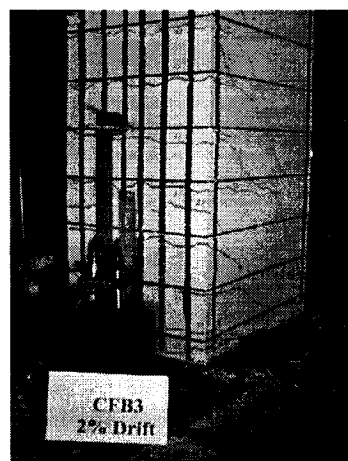
Figure 5.17: Hysteretic Behavior for Beam CFB3



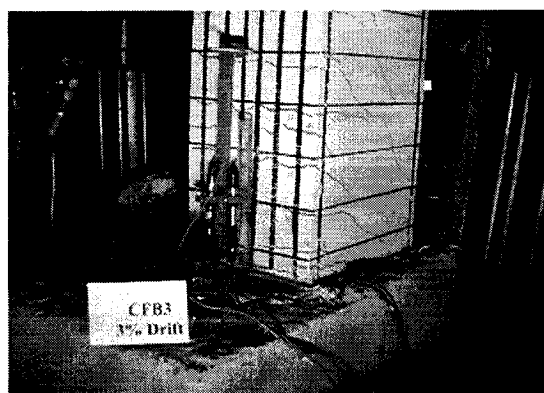
(a) Before Loading



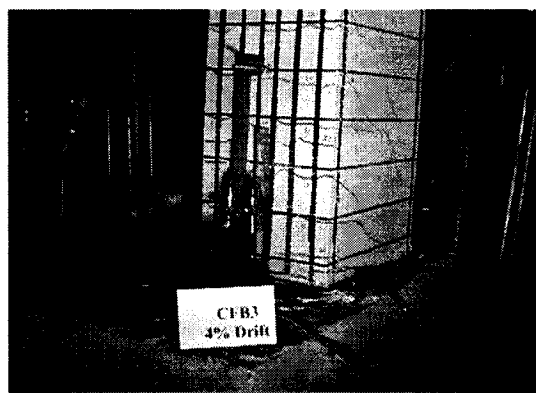
(b) At 1% Drift



(c) At 2% Drift

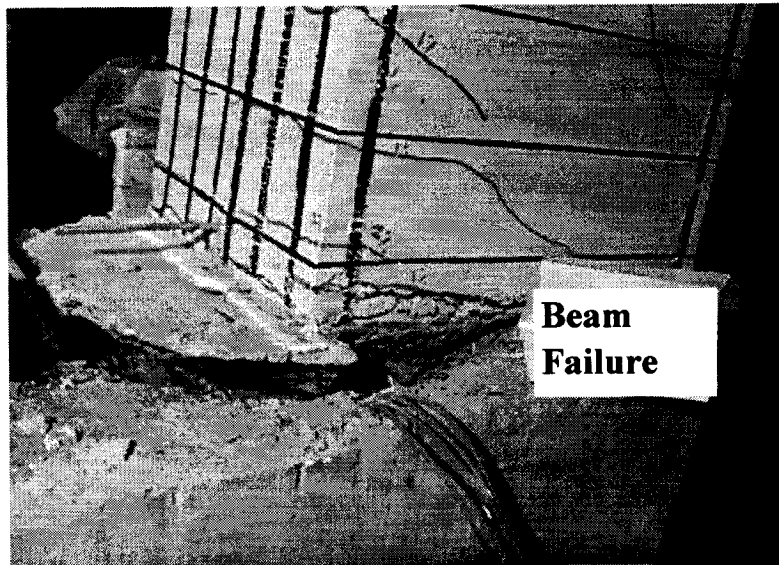


(d) At 3% Drift

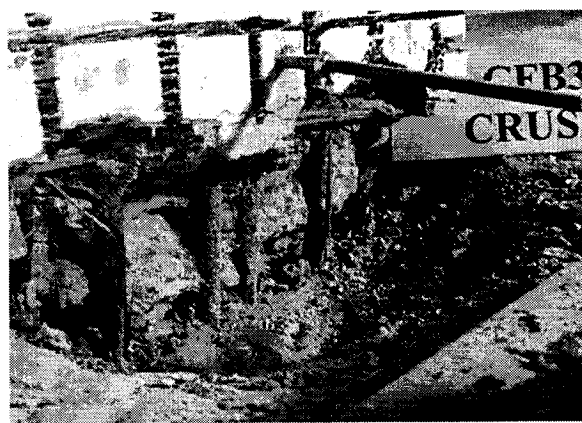
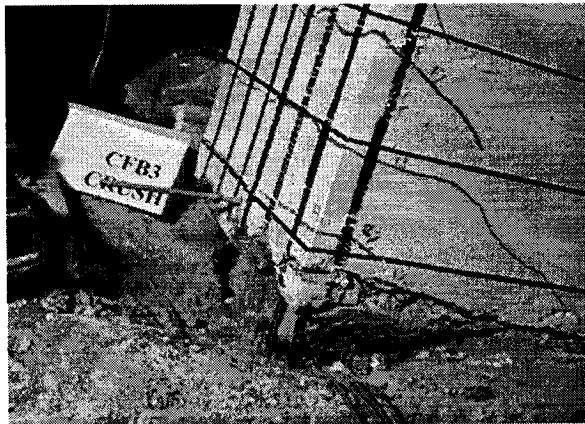


(e) At 4% Drift

Figure 5.18: Observed Damage in Beam CFB3 at Selected Stages of Loading



(f) Beam Failure



(g) FRP Bars Rupturing

Figure 5.18(Cont.): Observed Damage in Beam CFB3 at Selected Stages of Loading

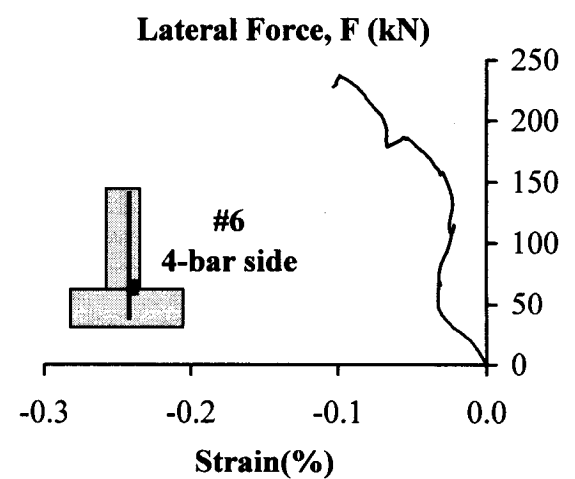
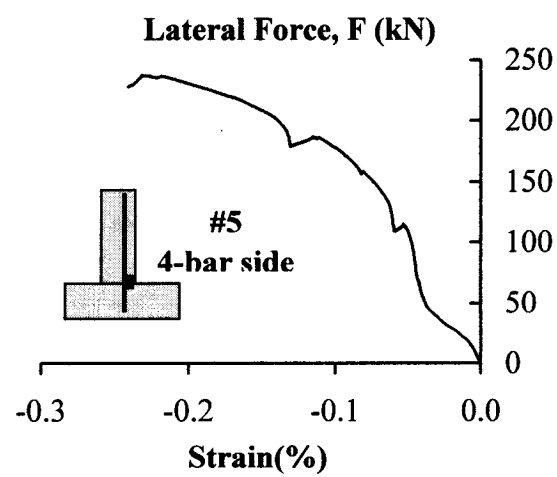
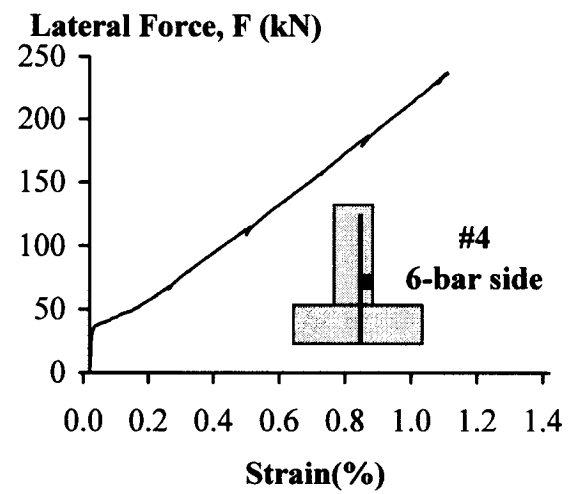
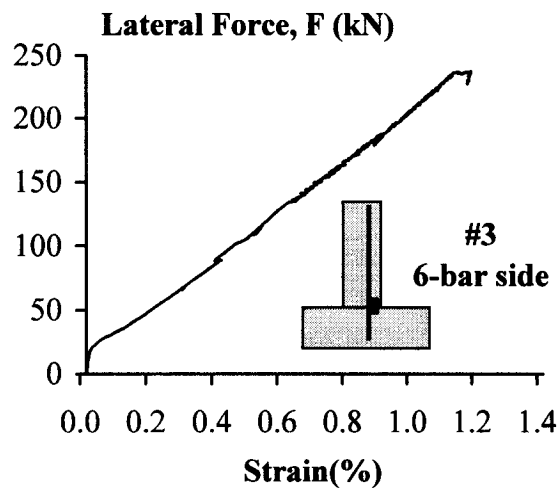
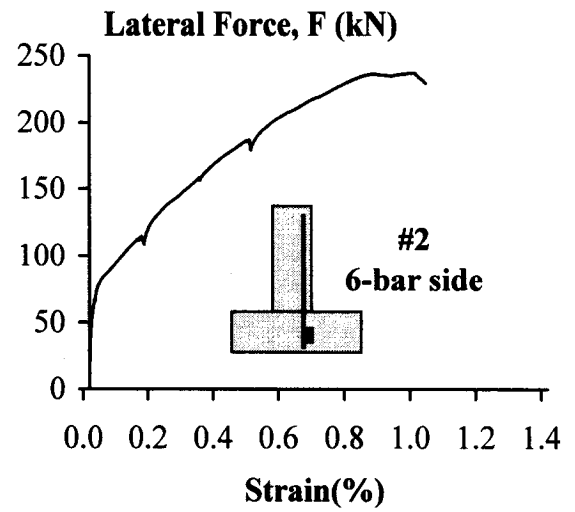
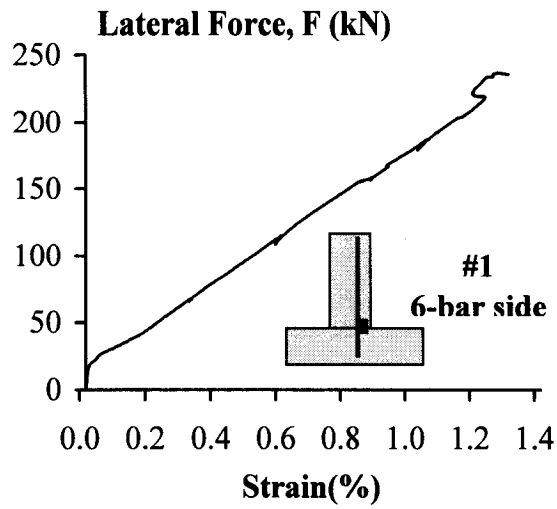


Figure 5.19: Bar Strain Readings in Beam CFB3

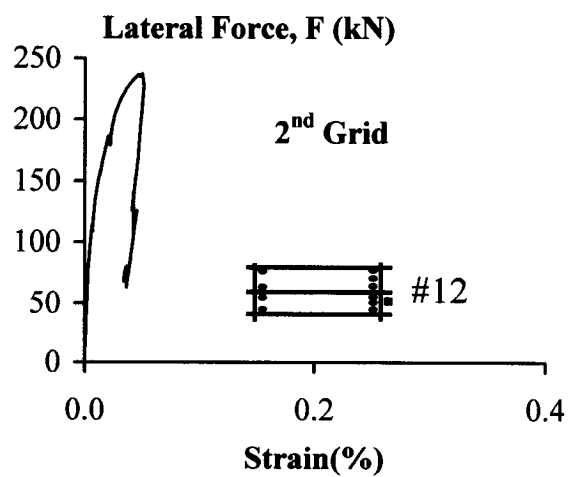
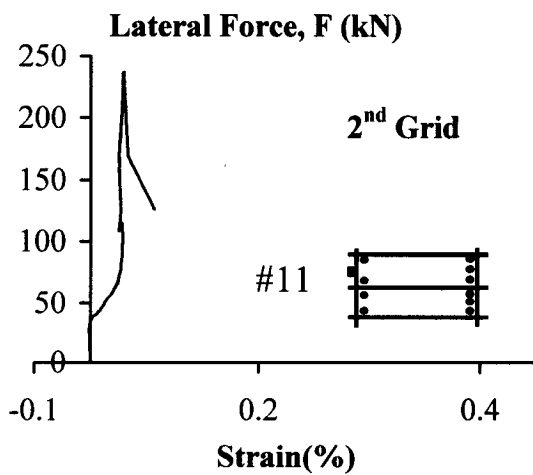
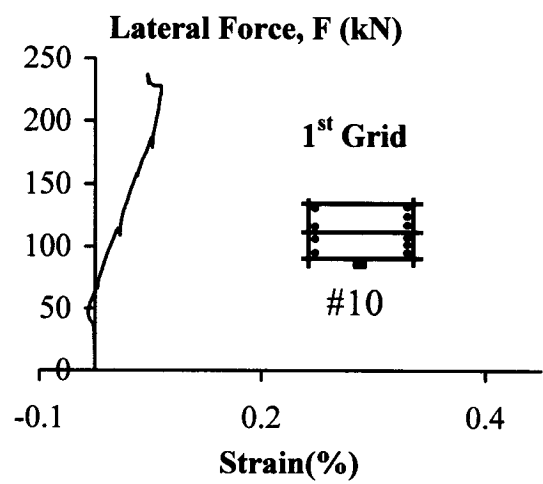
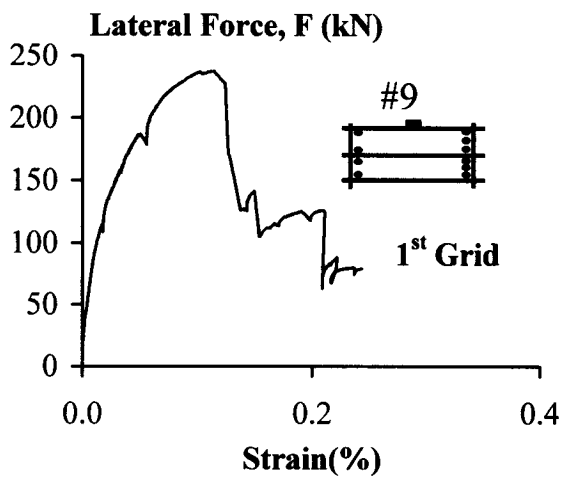
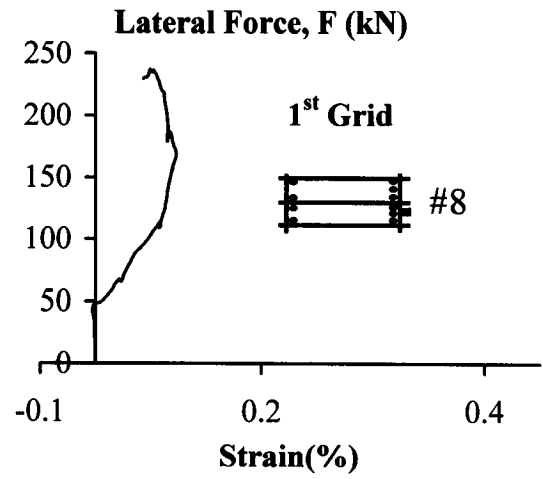
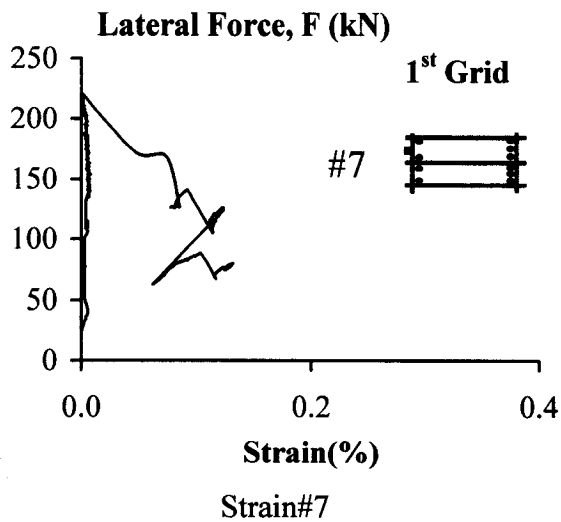


Figure 5.20: Grid Strain Readings in Beam CFB3



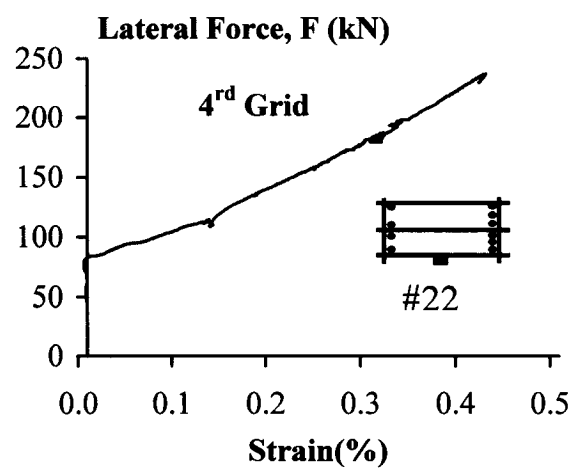
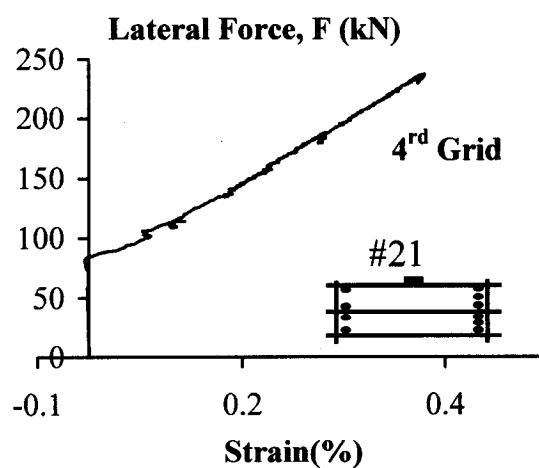
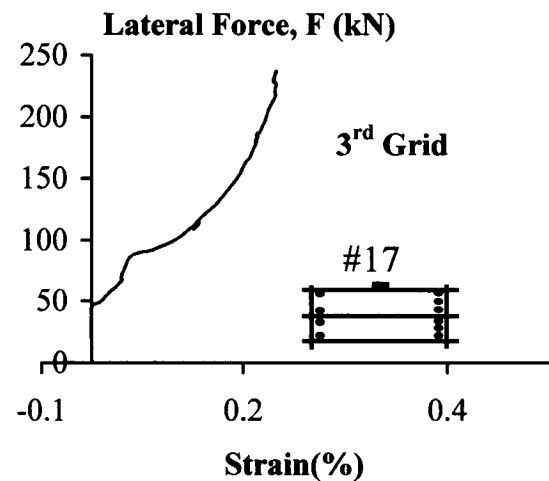
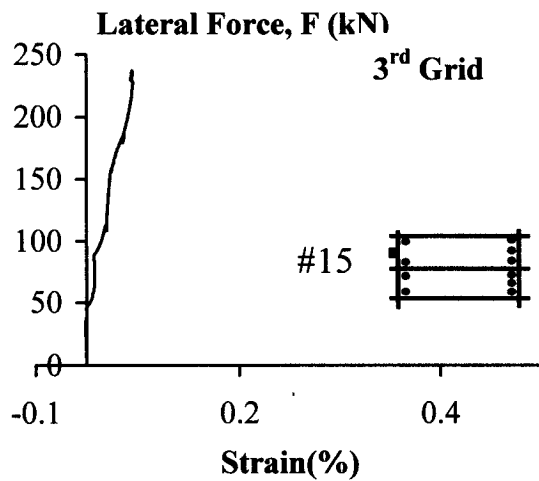
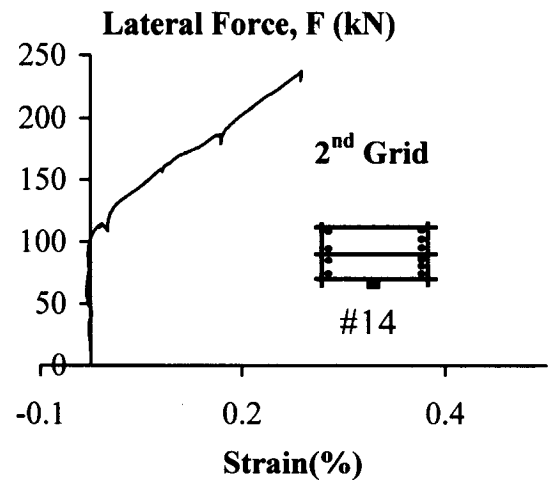
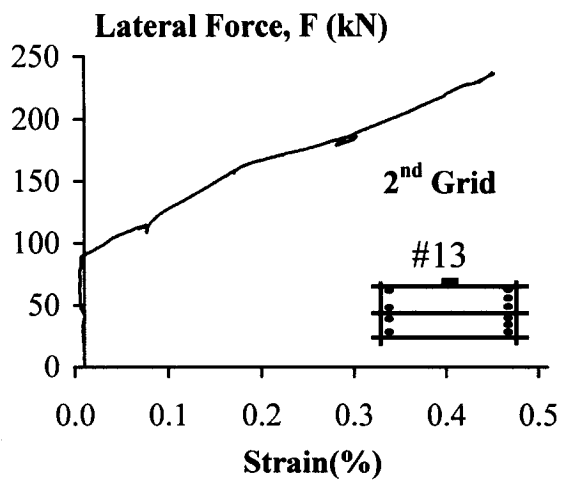
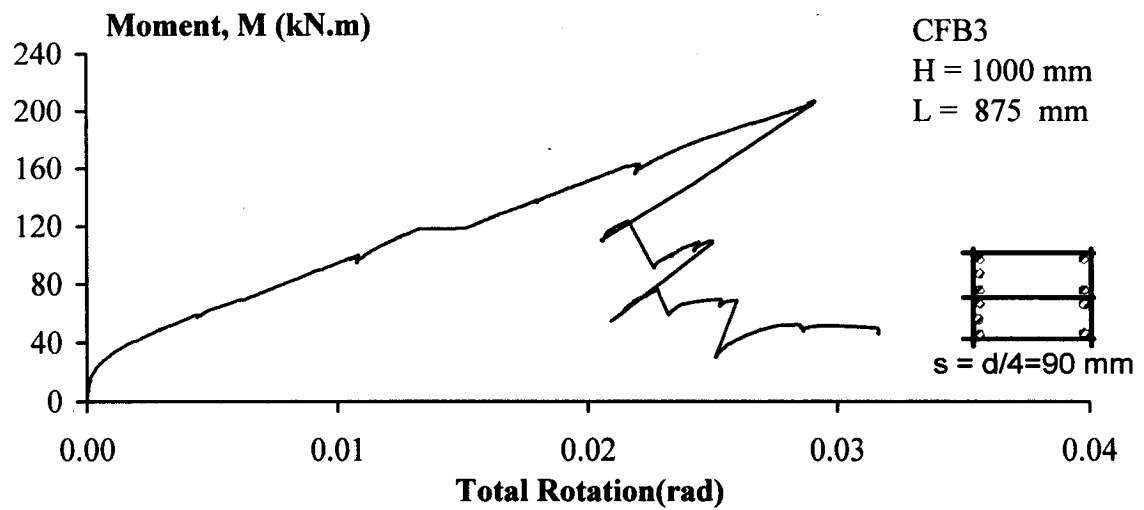
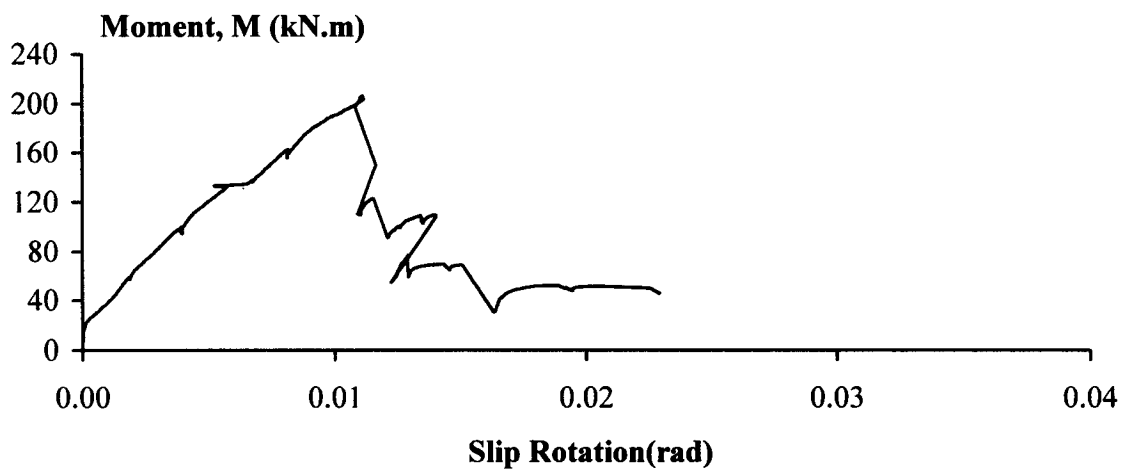


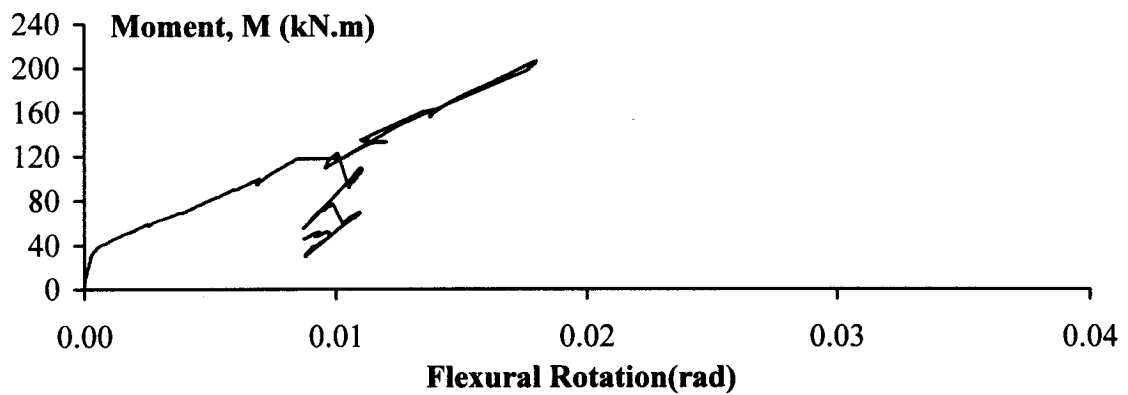
Figure 5.20(Cont.): Grid Strain Readings in Beam CFB3



(a) Moment-Total Rotation Relationship

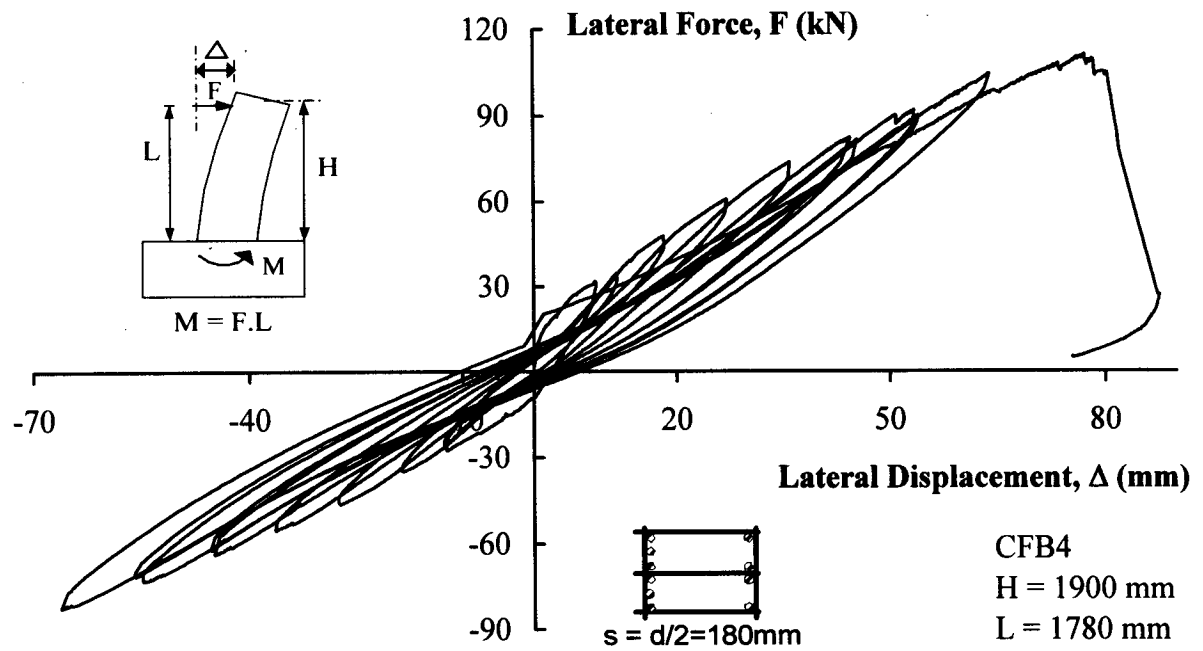


(b) Moment-Slip Rotation Relationship

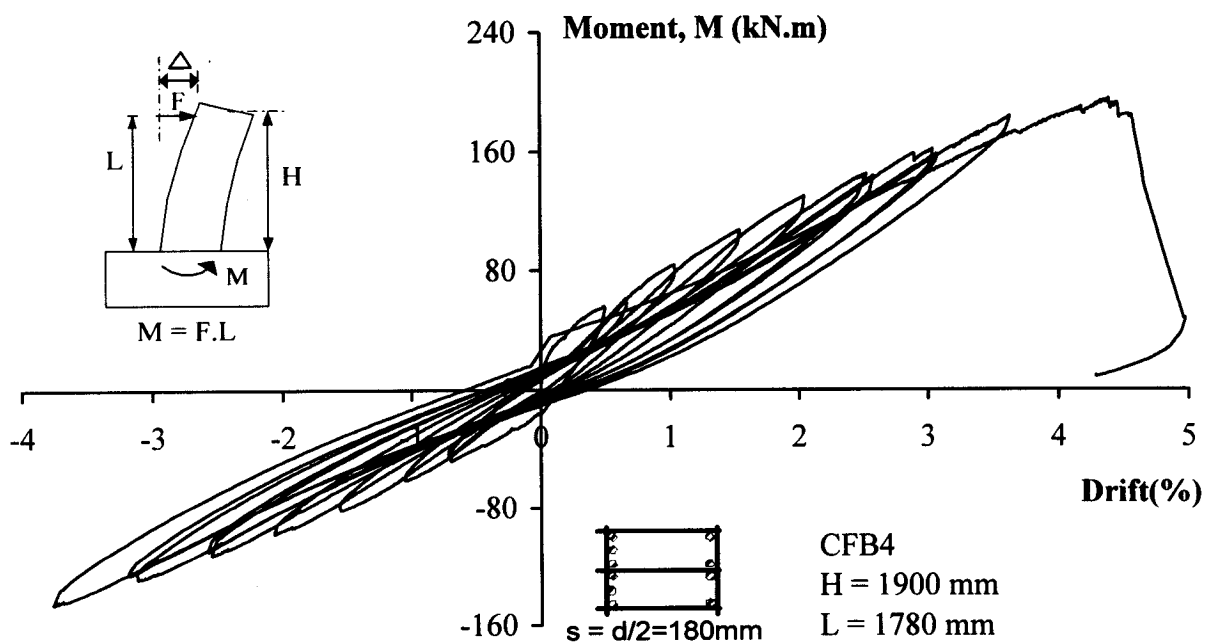


(c) Moment-Flexural Rotation Relationship

Figure 5.21: Hysteretic Moment-Rotation Relationships for Beam CFB3

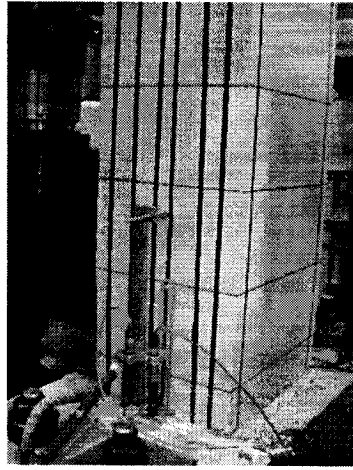


(a) Lateral Force-Lateral Displacement Relationship

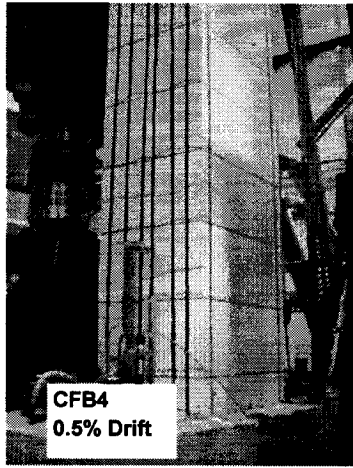


(b) Moment-Drift Relationship

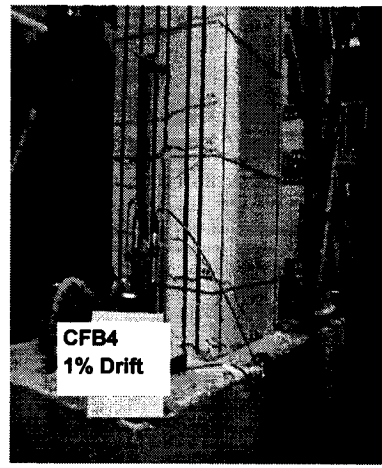
Figure 5.22: Hysteretic Behavior for Beam CFB4



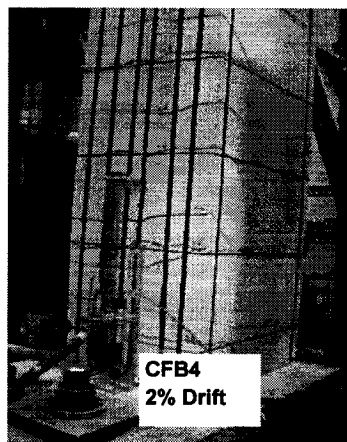
(a) Before Loading



(b) At 0.5% Drift

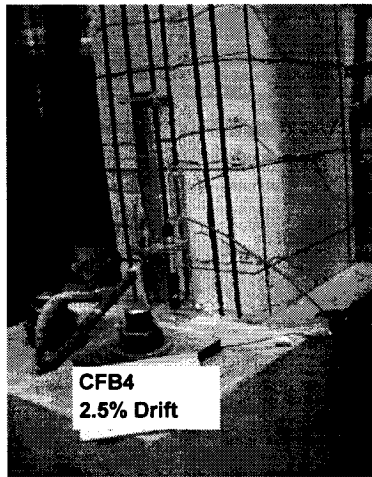


(c) At 1% Drift

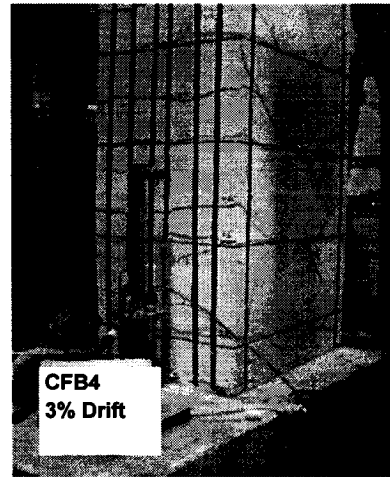


(d) At 2% Drift

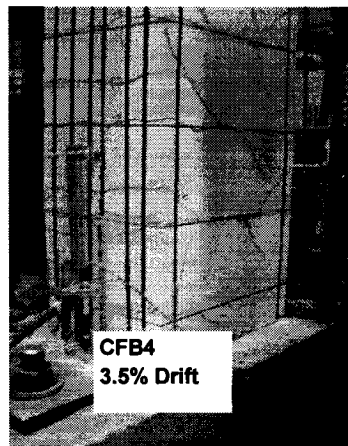
Figure 5.23: Observed Damage in Beam CFB4 at Selected Stages of Loading



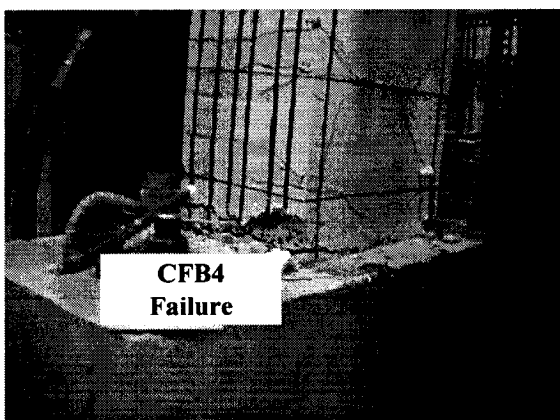
(e) At 2.5% Drift



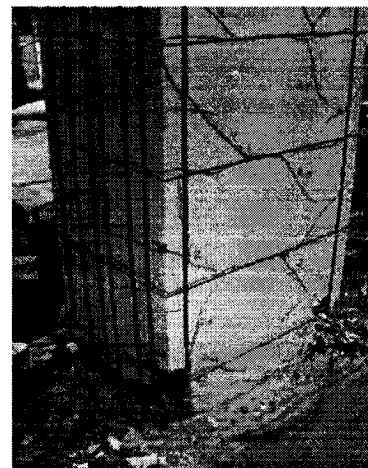
(f) At 3 % Drift



(g) At 3.5% Drift



(h) Beam Failure



(i) FRP Bars Rupturing

Figure 5.23(Cont.): Observed Damage in Beam CFB4 at Selected Stages of Loading

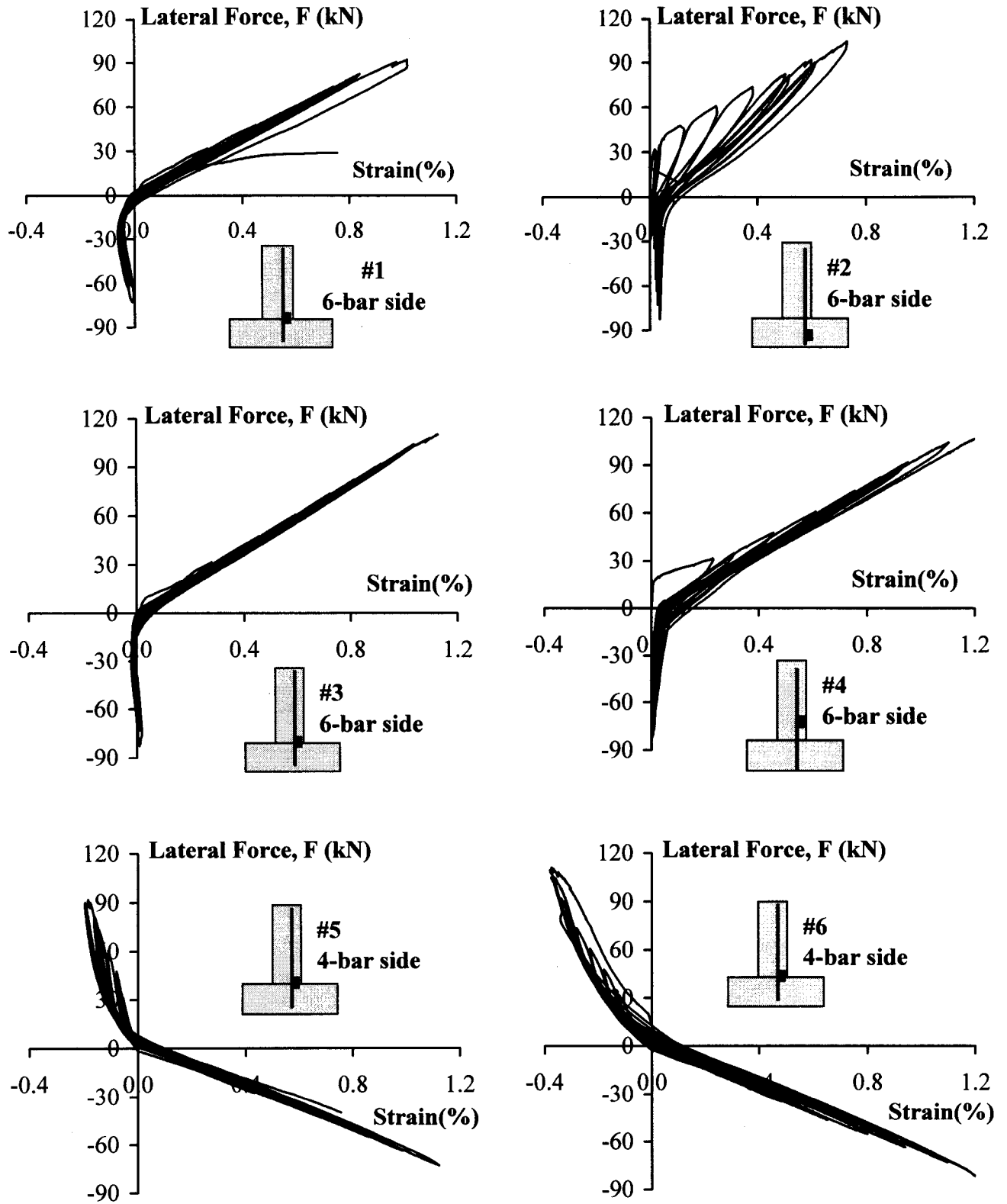


Figure 5.24: Bar Strain Readings in Beam CFB4

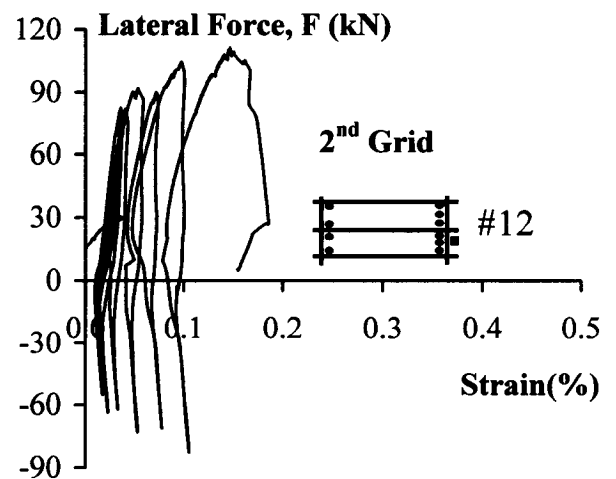
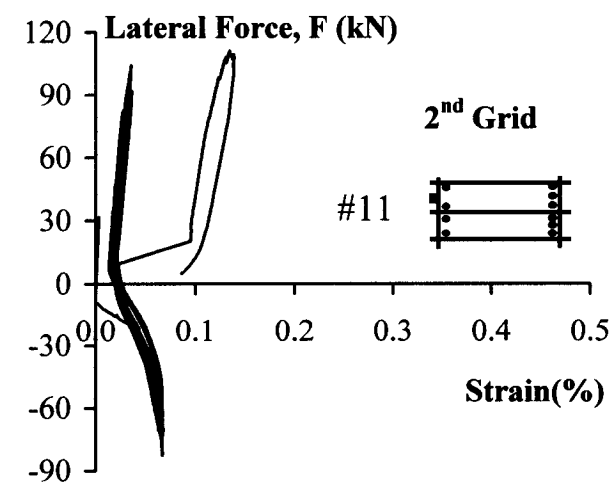
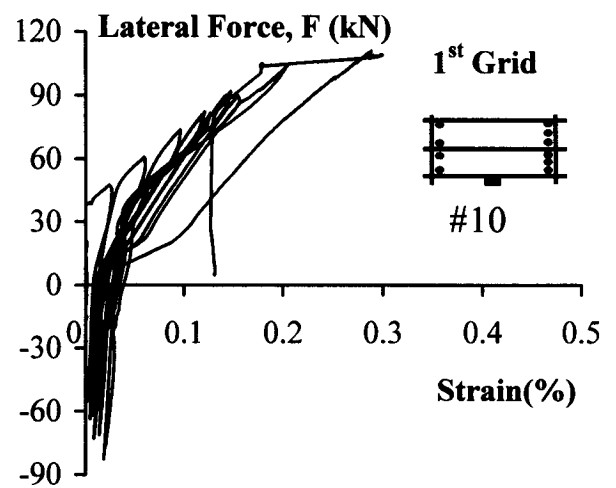
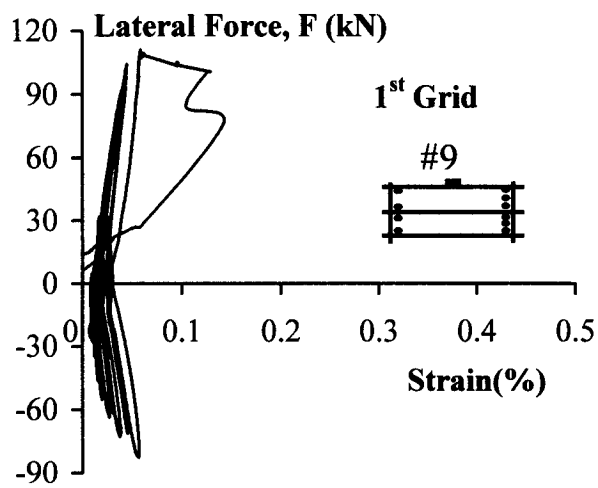
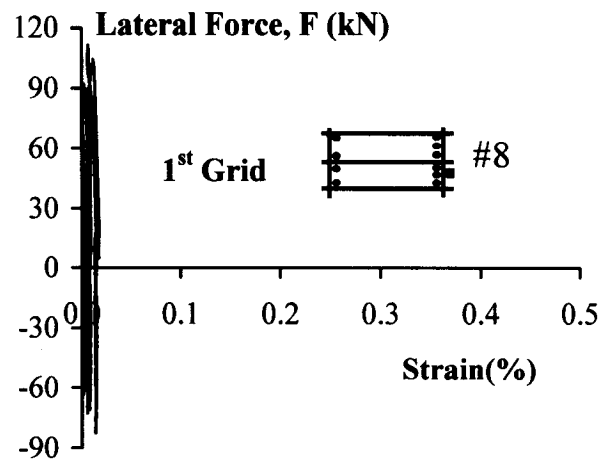
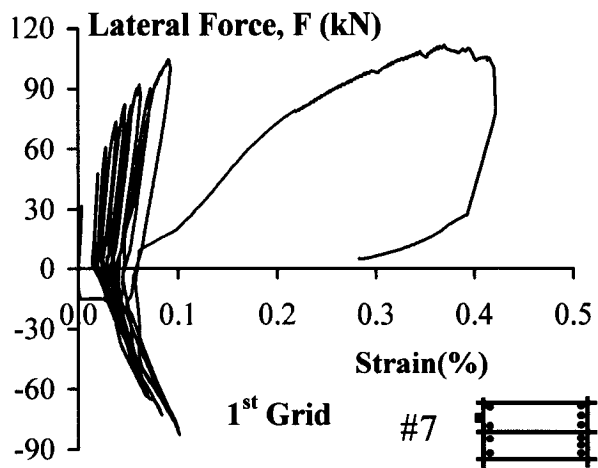


Figure 5.25: Grid Strain Readings in Beam CFB4

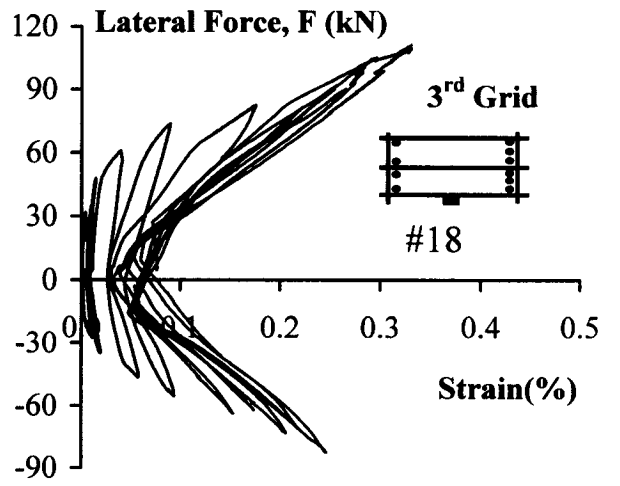
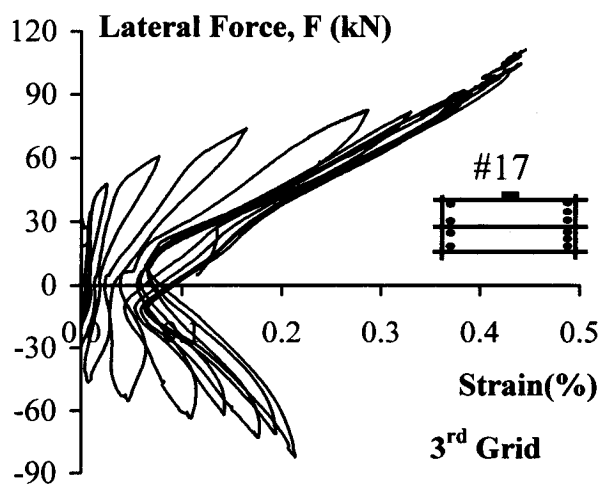
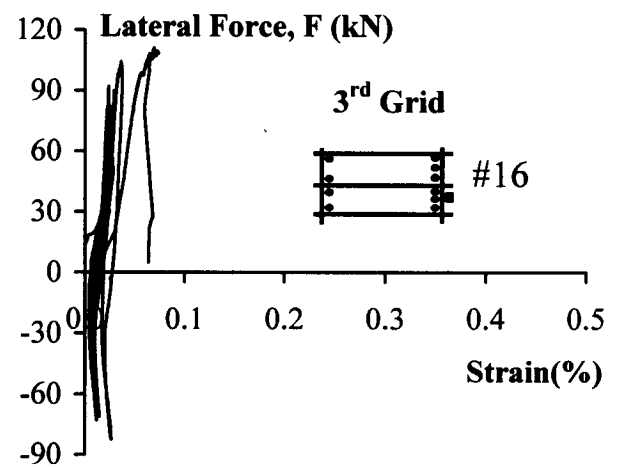
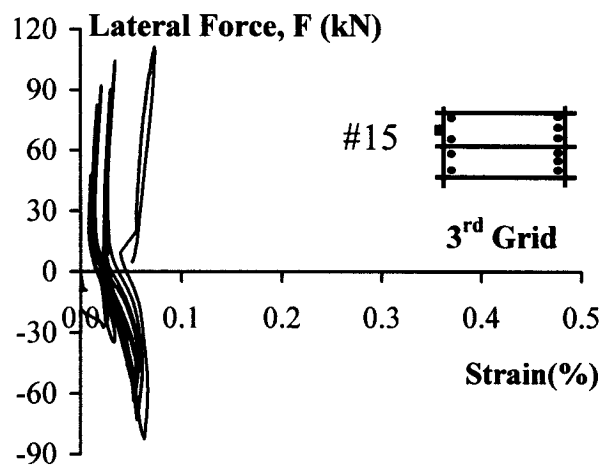
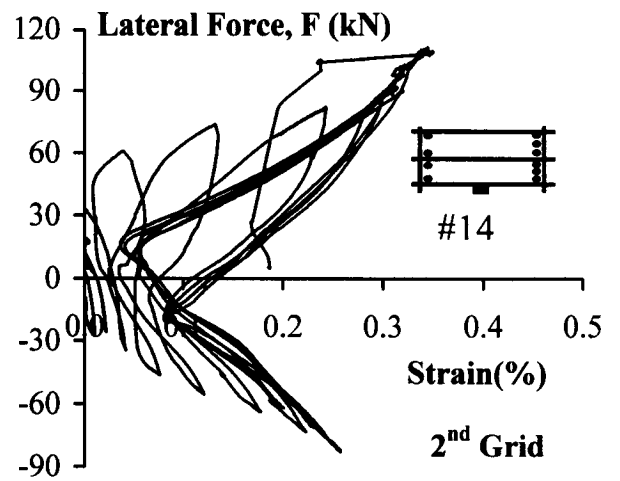
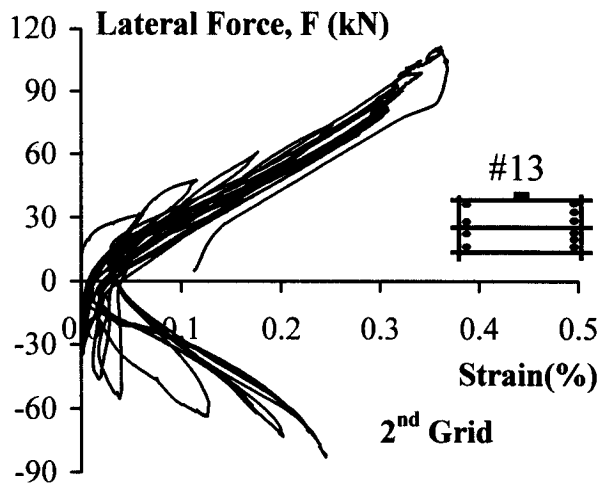
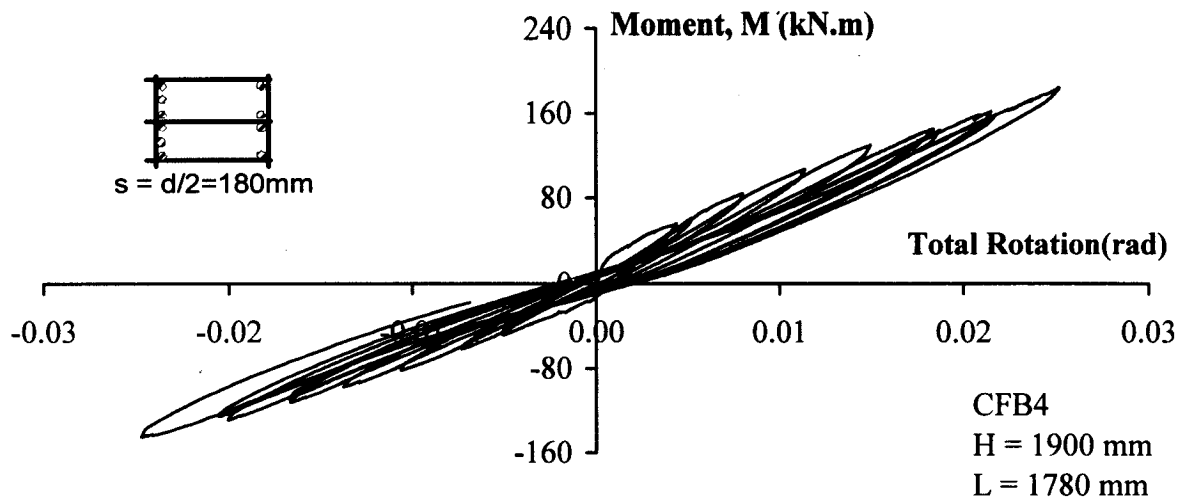
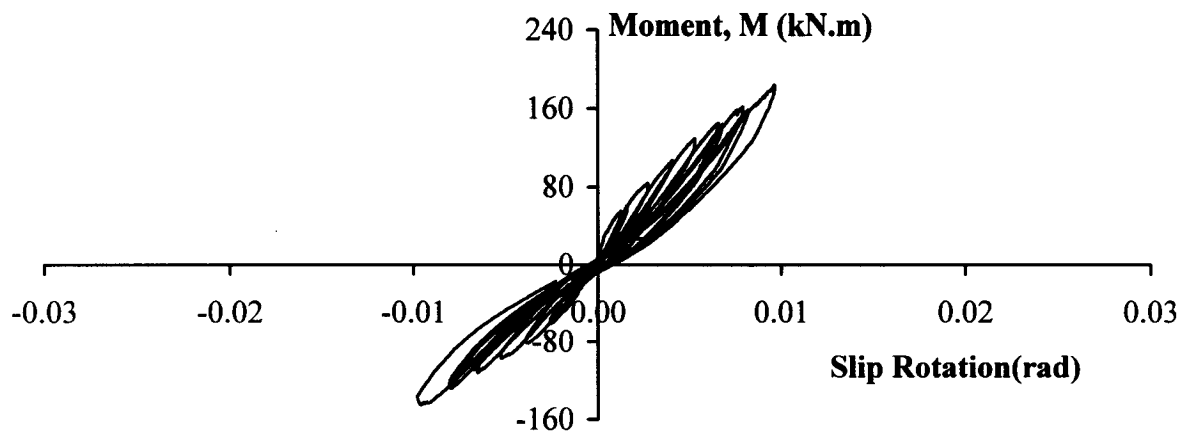


Figure 5.25: Grid Strain Readings in Beam CFB4

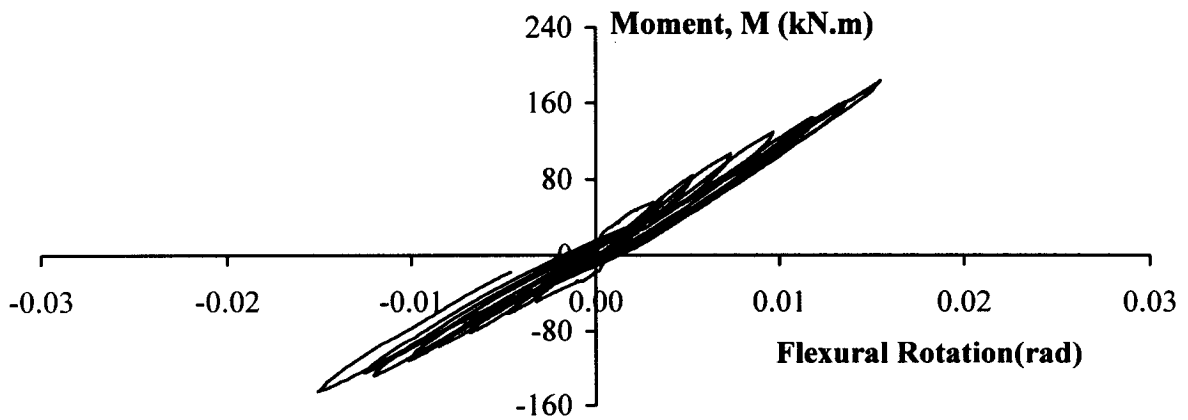




(a) Moment-Total Rotation Relationship

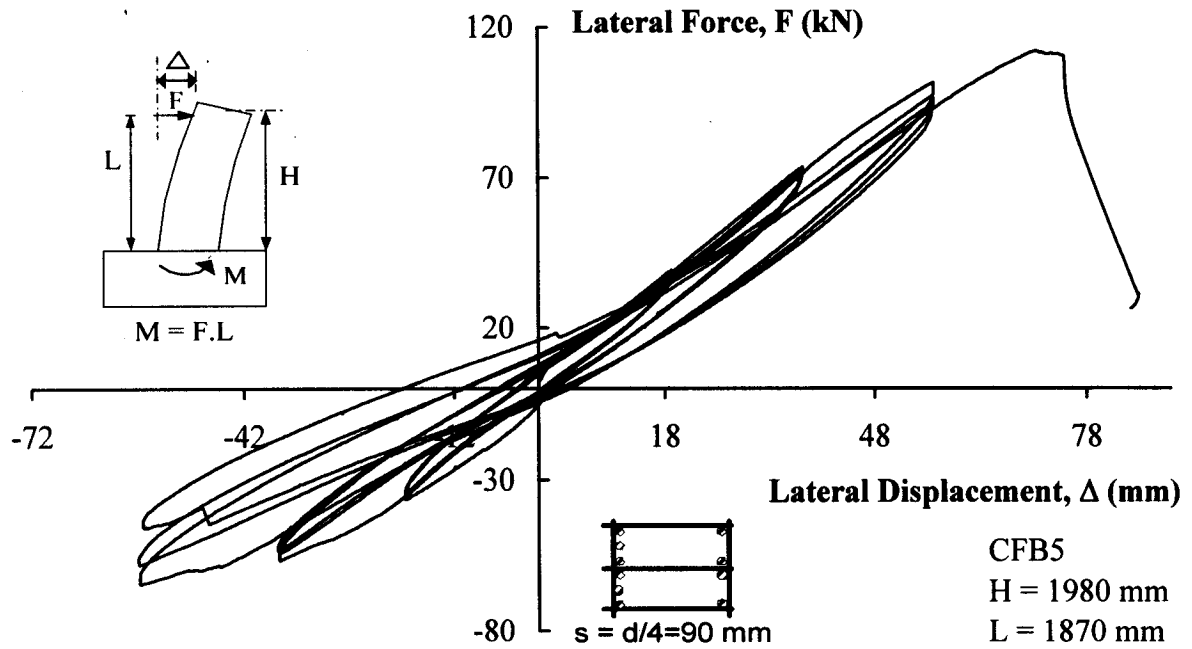


(b) Moment-Slip Rotation Relationship

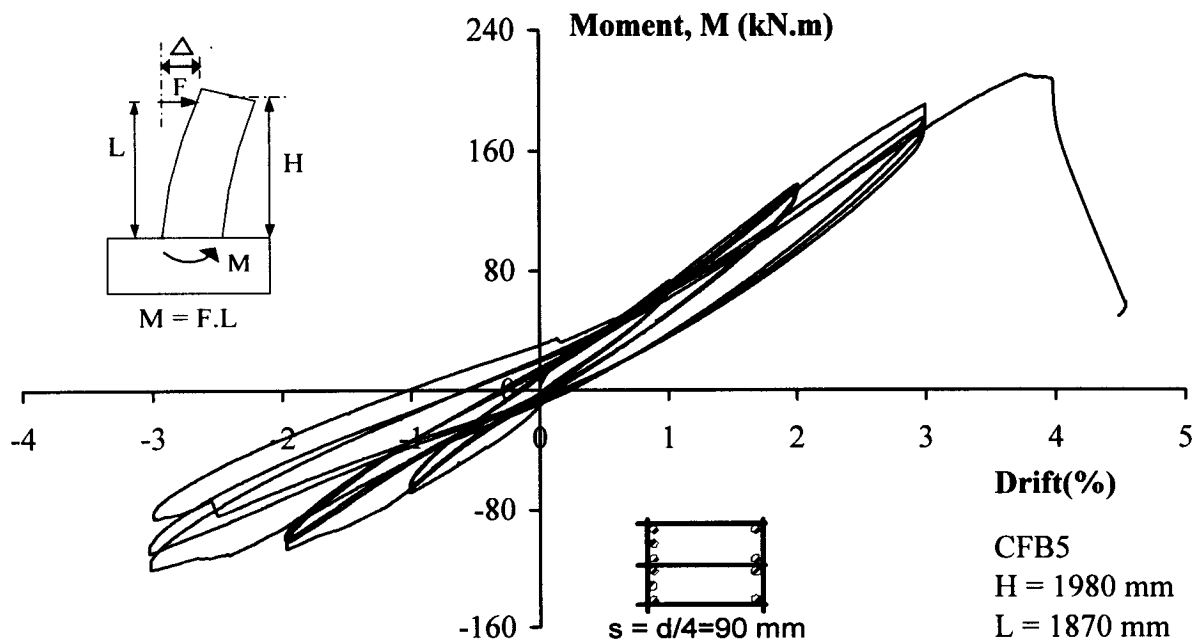


(c) Moment-Flexural Rotation Relationship

Figure 5.26: Hysteretic Moment-Rotation Relationships for Beam CFB4

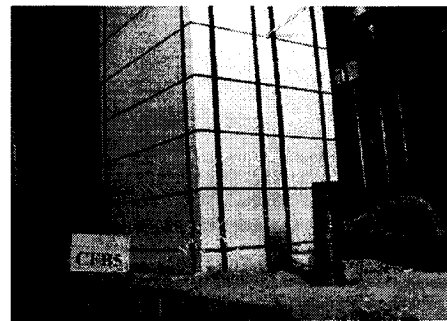
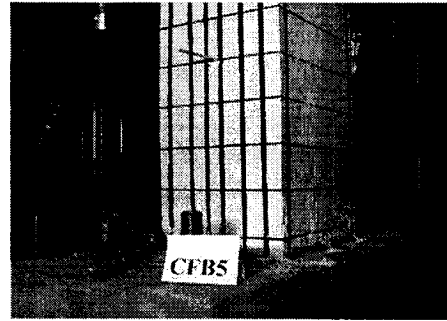
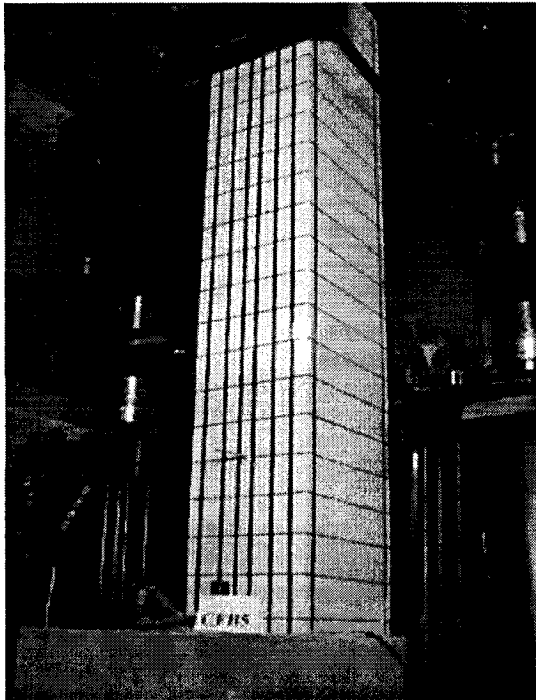


(a) Lateral Force-Lateral Displacement Relationship

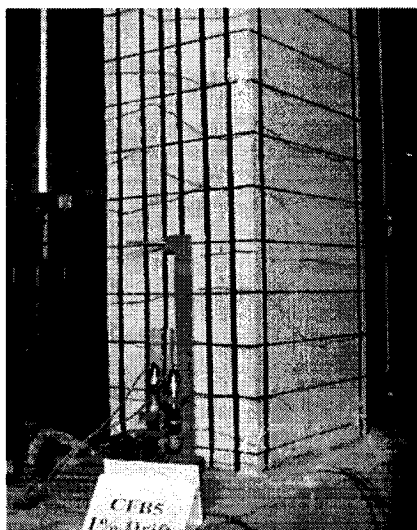


(b) Moment-Drift Relationship

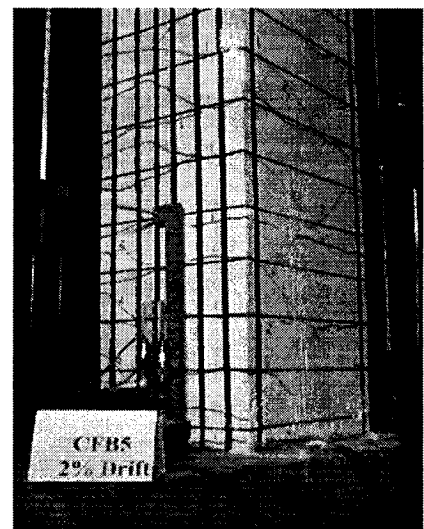
Figure 5.27: Hysteretic Behavior for Beam CFB5



(a) Before Loading

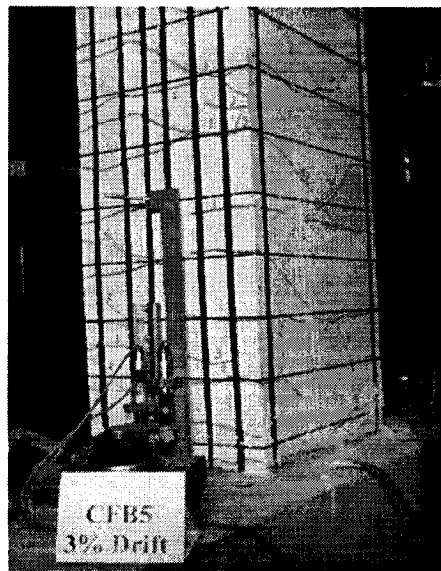


(b) At 1% Drift



(c) At 2% Drift

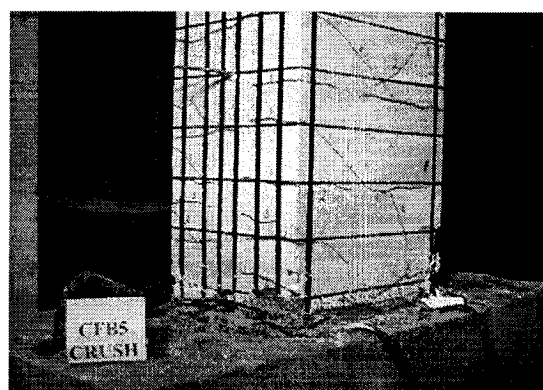
Figure 5.28: Observed Damage in Beam CFB5 at Selected Stages of Loading



(d) At 3% Drift



(e) At 4% Drift



(f) Beam Failure

Figure 5.28(Cont.): Observed Damage in Beam CFB5 at Selected Stages of Loading

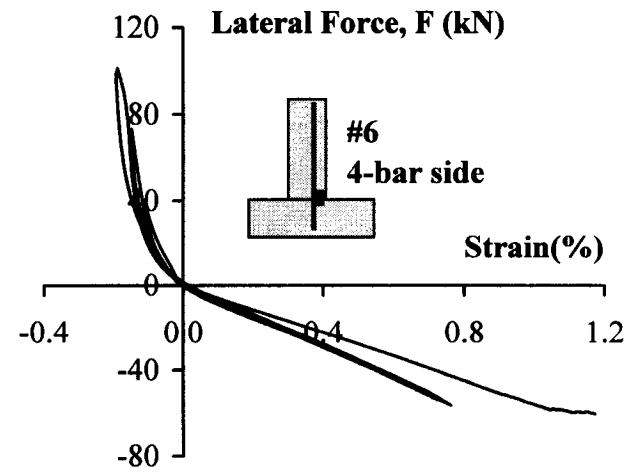
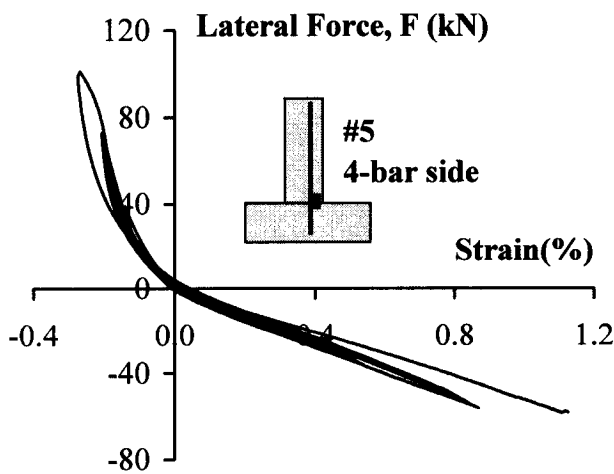
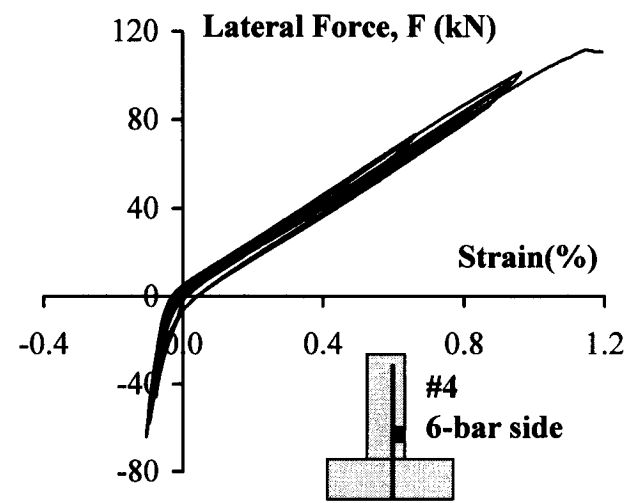
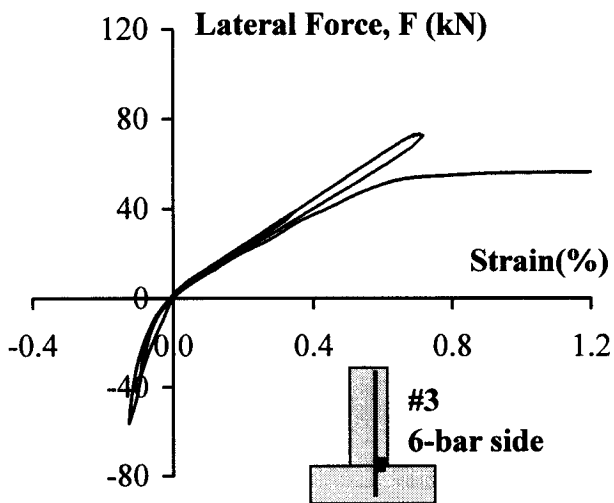
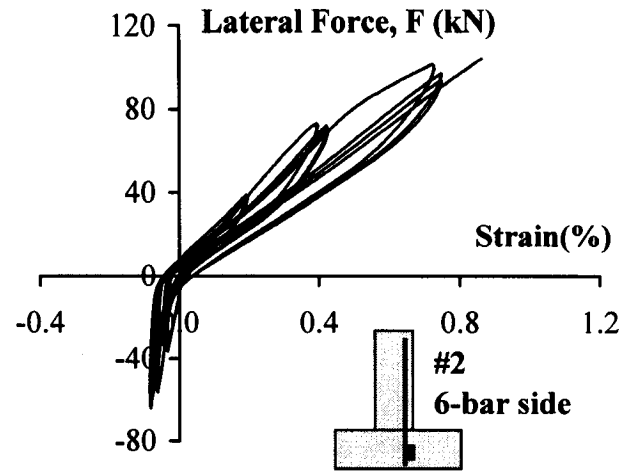
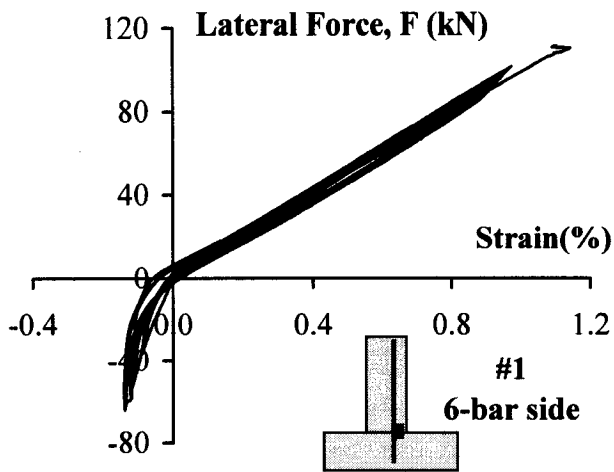


Figure 5.29: Bar Strain Readings in Beam CFB5

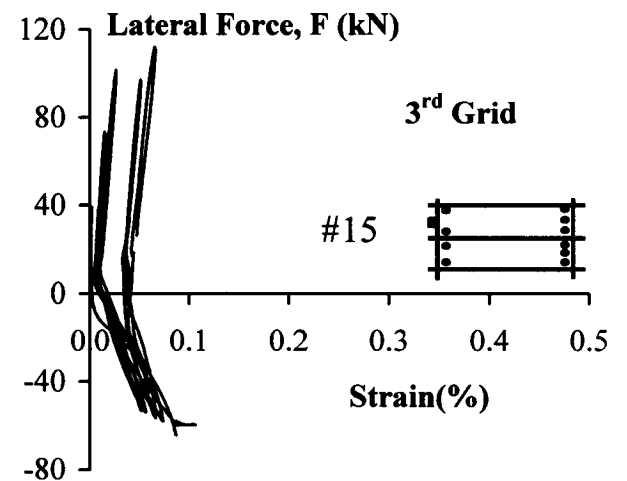
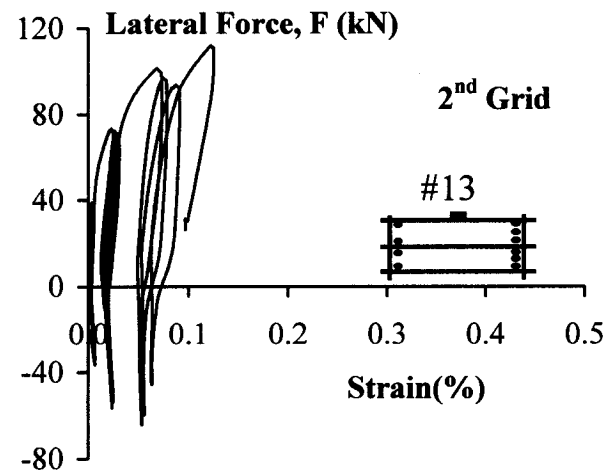
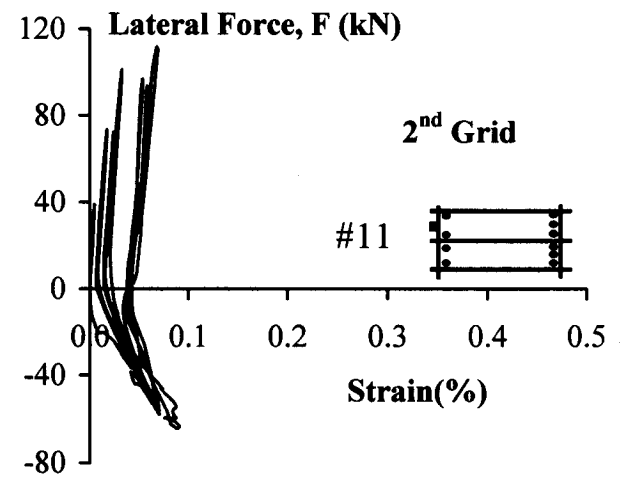
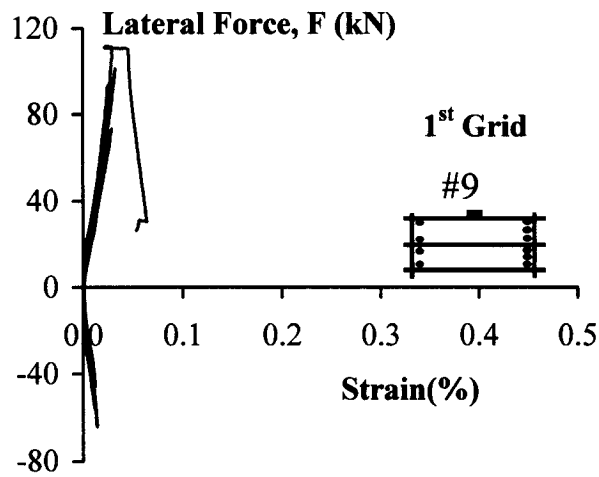
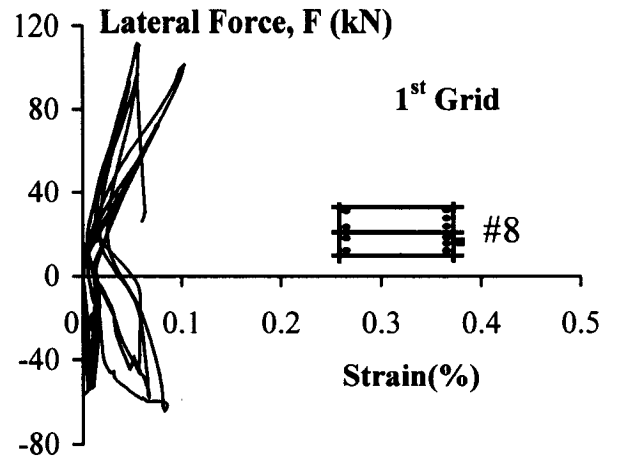
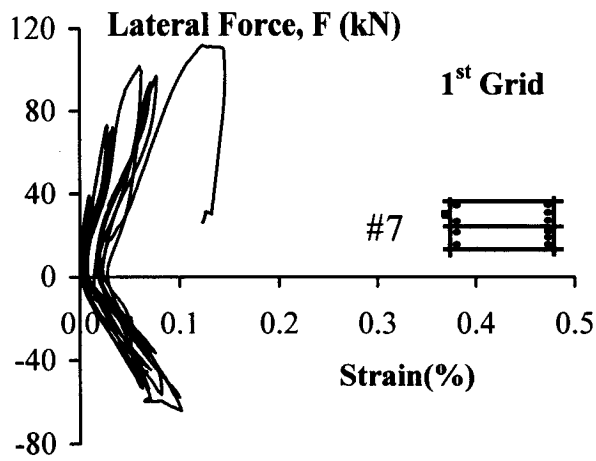


Figure 5.30: Grid Strain Readings in Beam CFB5

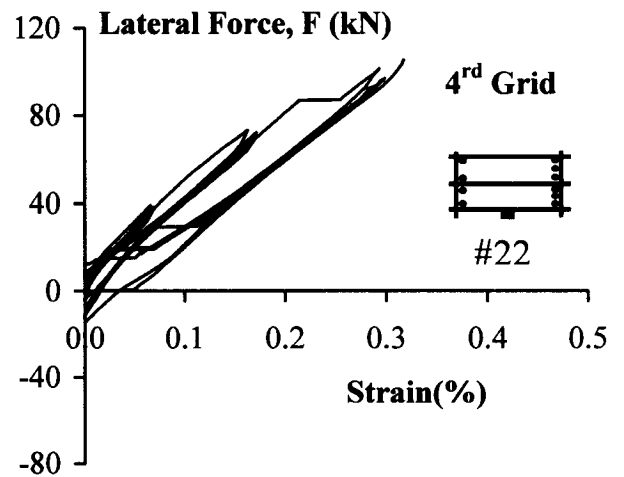
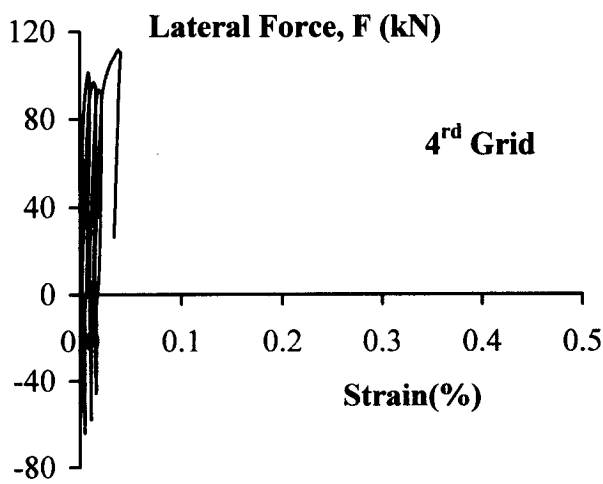
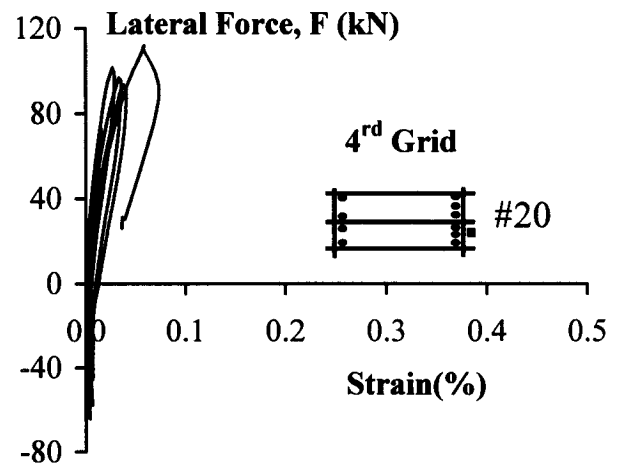
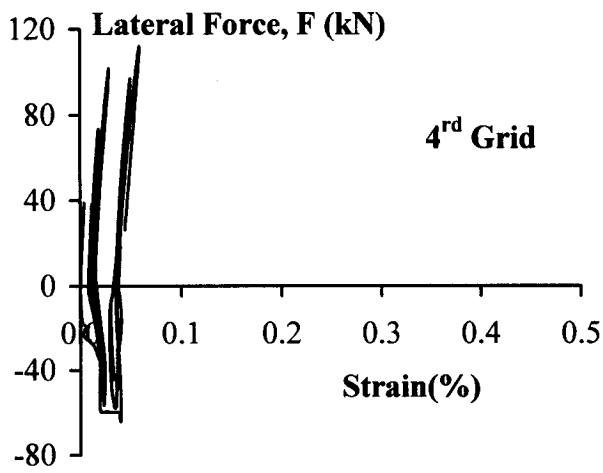
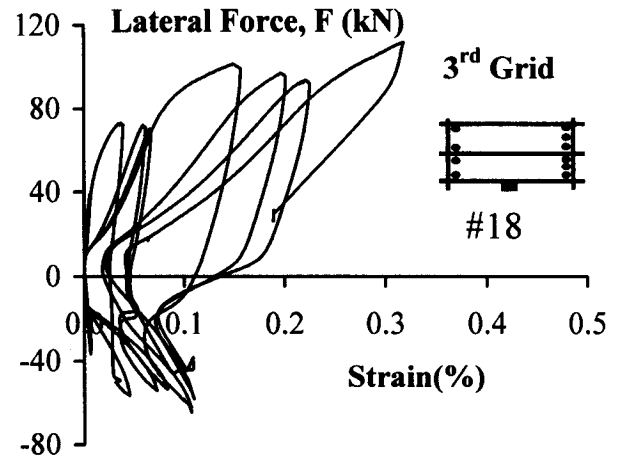
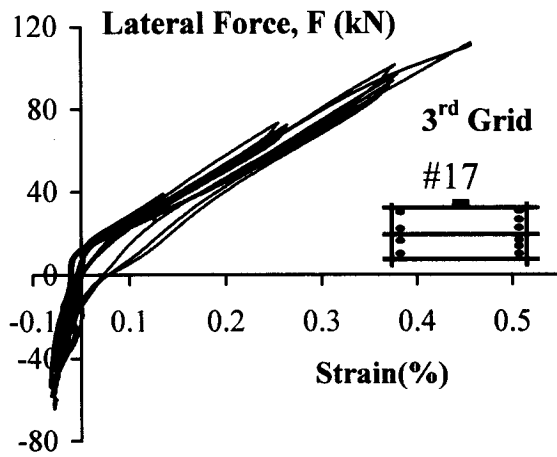
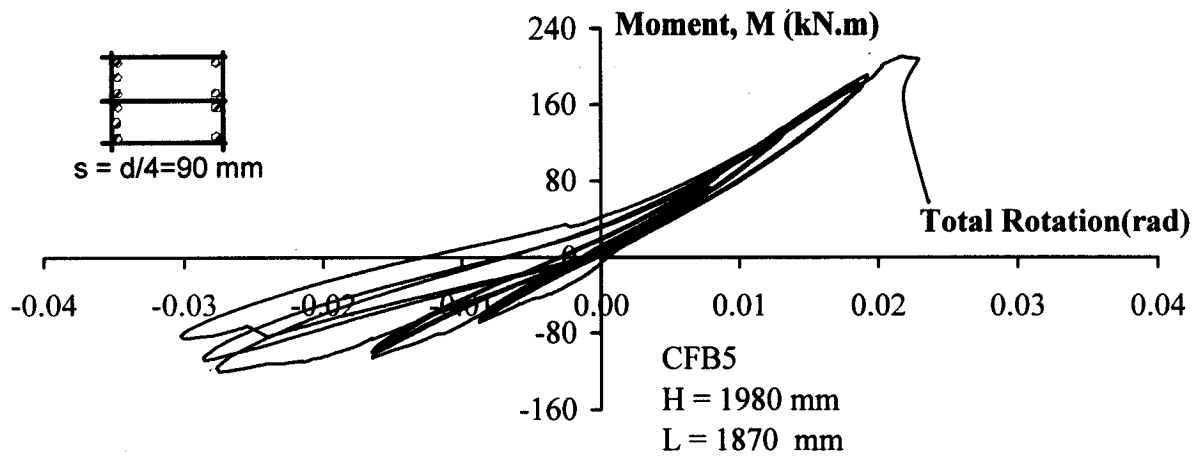
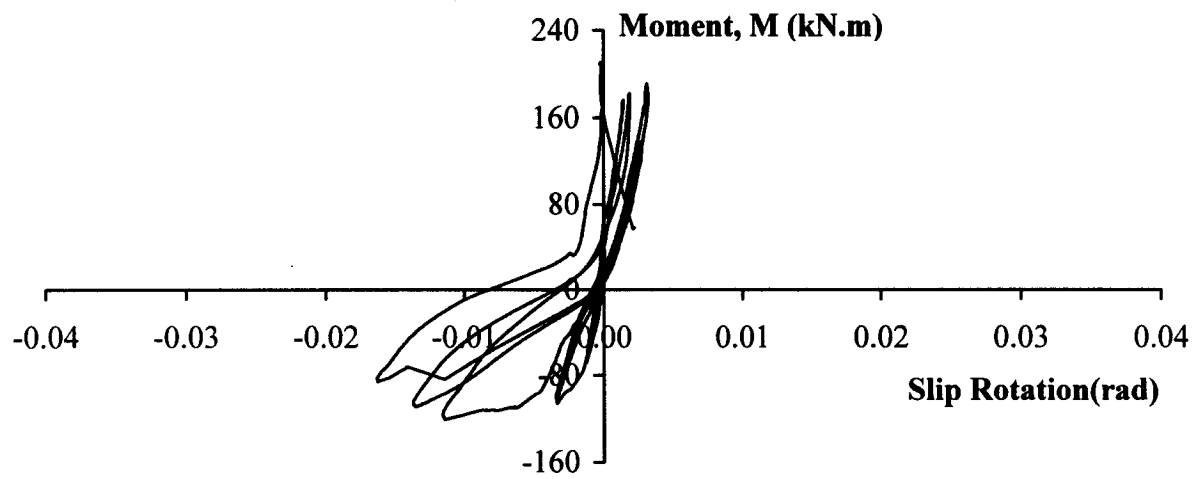


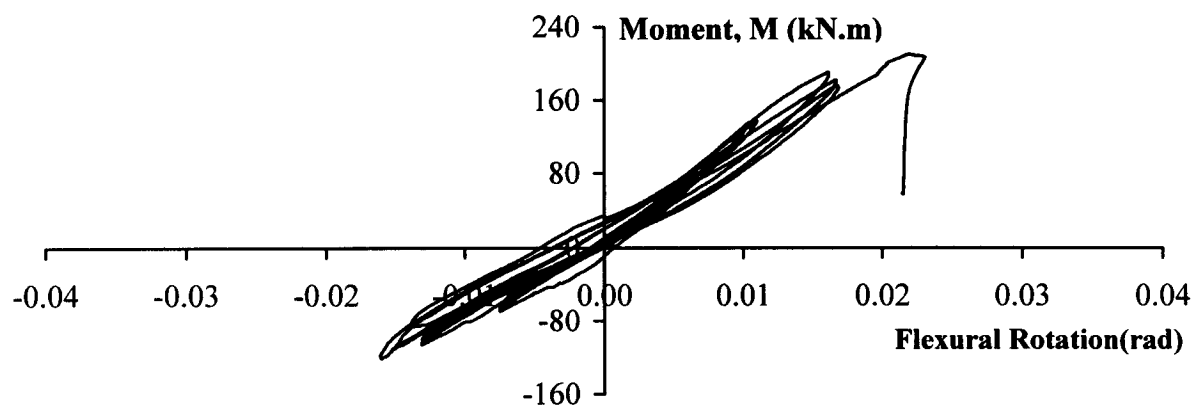
Figure 5.30 (Cont.): Grid Strain Readings in Beam CFB5



(a) Moment-Total Rotation Relationship



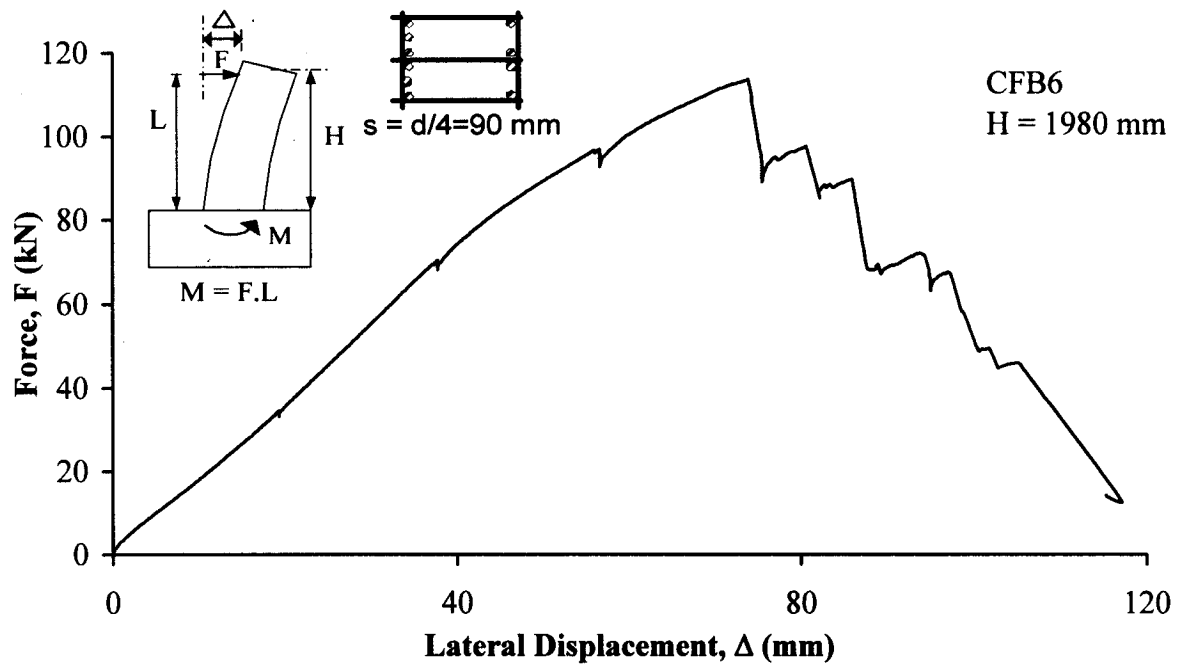
(b) Moment-Slip Rotation Relationship



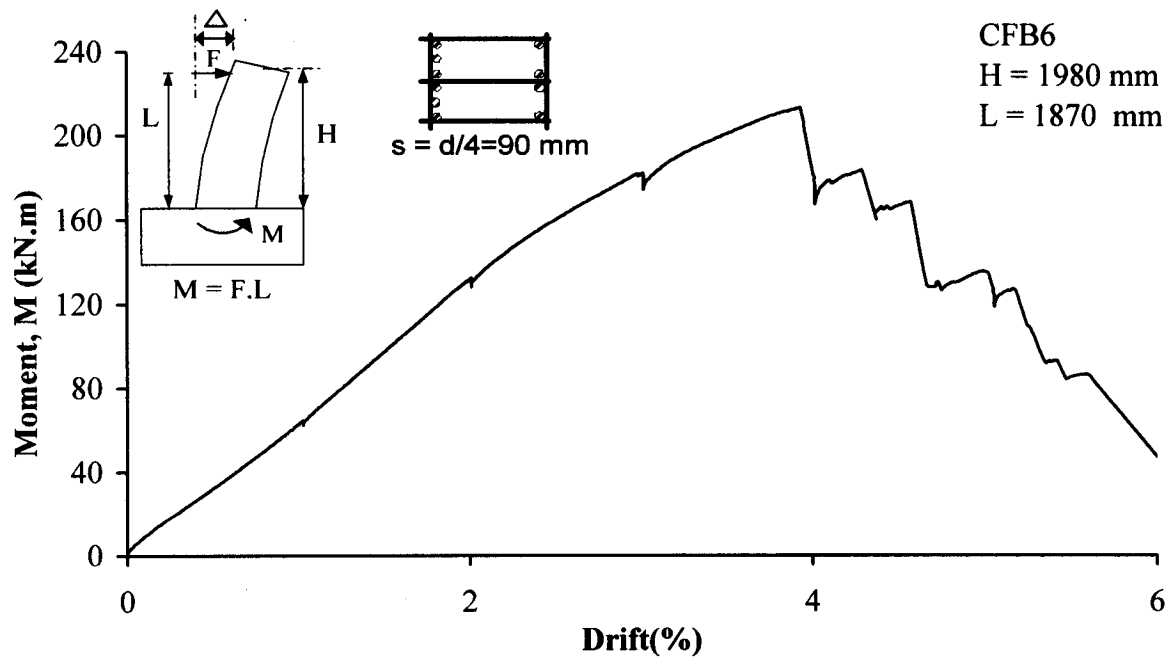
(c) Moment-Flexural Rotation Relationship

Figure 5.31: Hysteretic Moment-Rotation Relationships for Beam CFB5



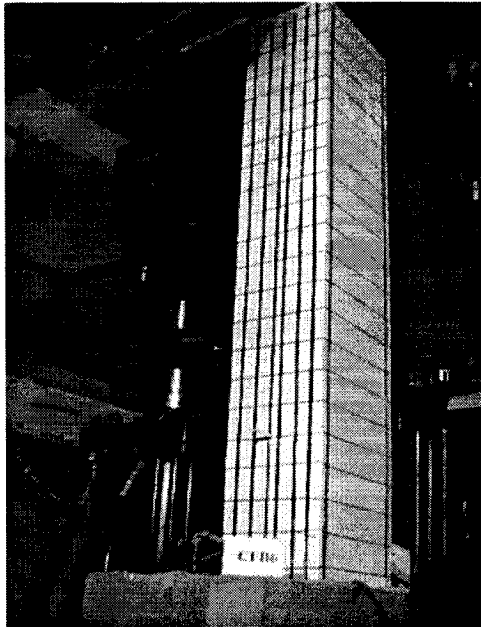


(a) Lateral Force-Lateral Displacement Relationship

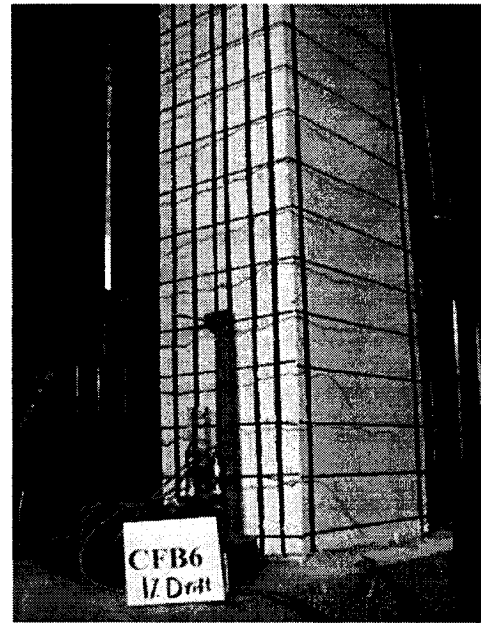


(b) Moment-Drift Relationship

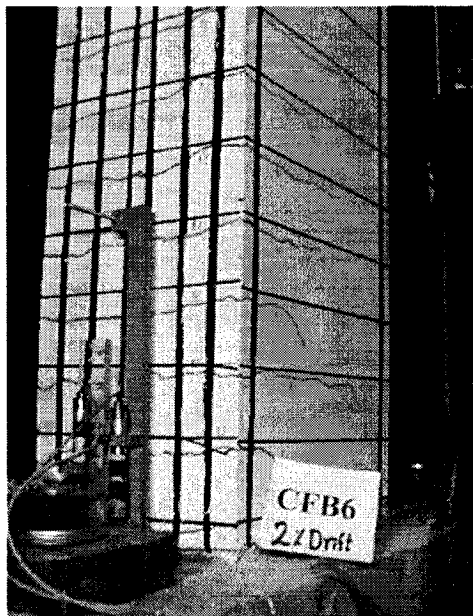
Figure 5.32: Hysteretic Behavior for Beam CFB6



(a) Before Loading



(b) At 1% Drift

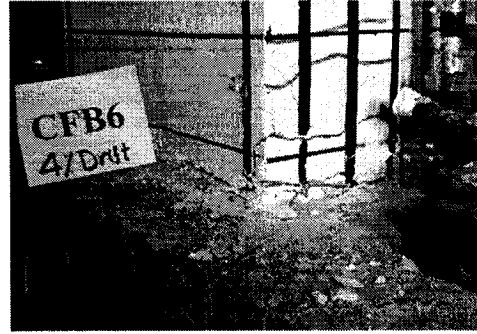


(c) At 2% Drift



(d) At 3% Drift

Figure 5.33: Observed Damage in Beam CFB6 at Selected Stages of Loading



(e) At 4% Drift



(f) At 5% Drift



(g) Beam Failure

Figure 5.33(Cont.): Observed Damage in Beam CFB6 at Selected Stages of Loading

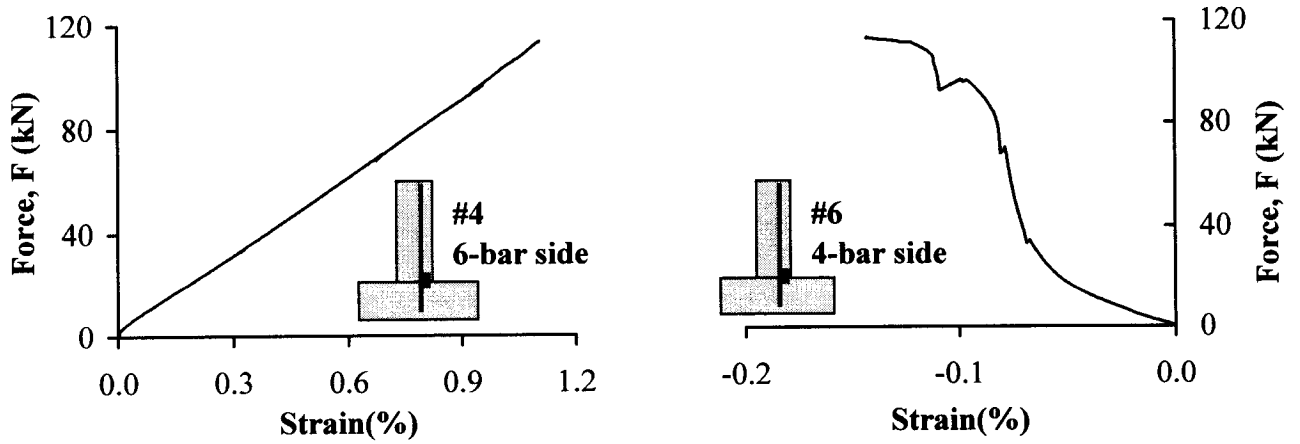
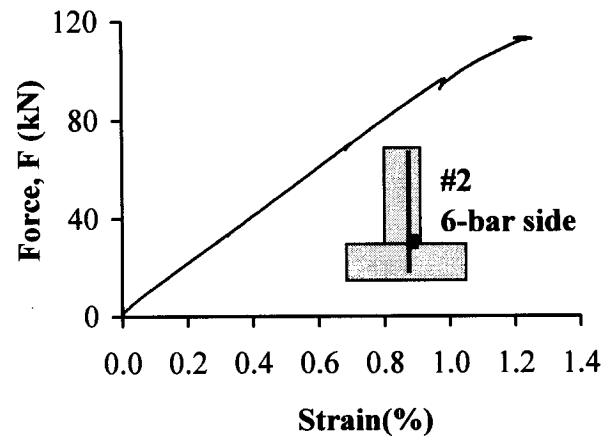
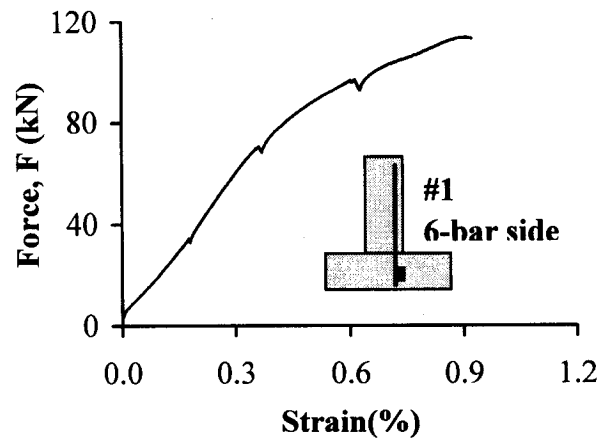


Figure 5.34: Bar Strain Readings in Beam CFB6

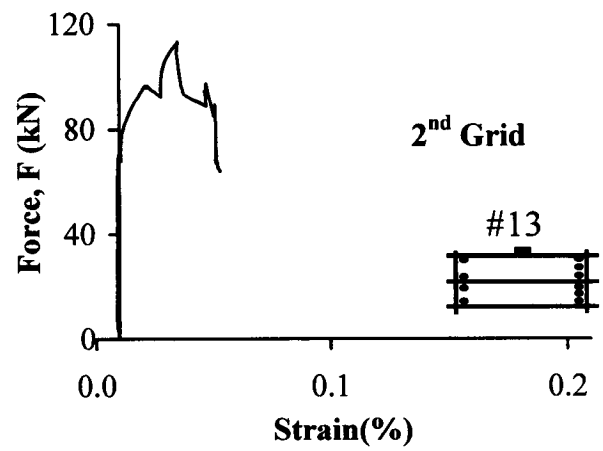
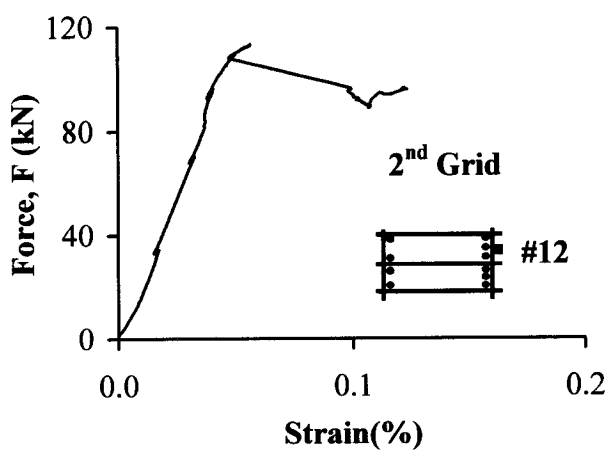
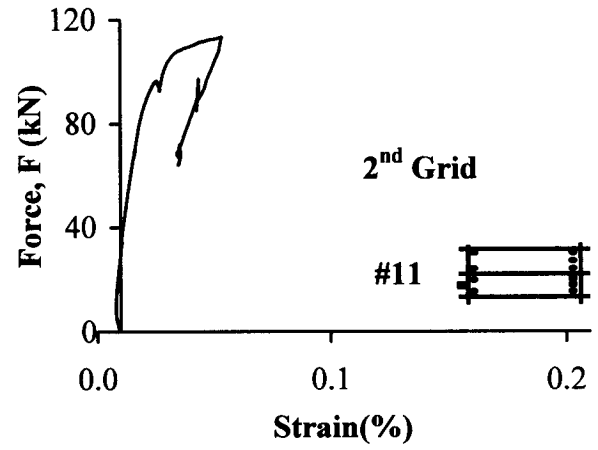
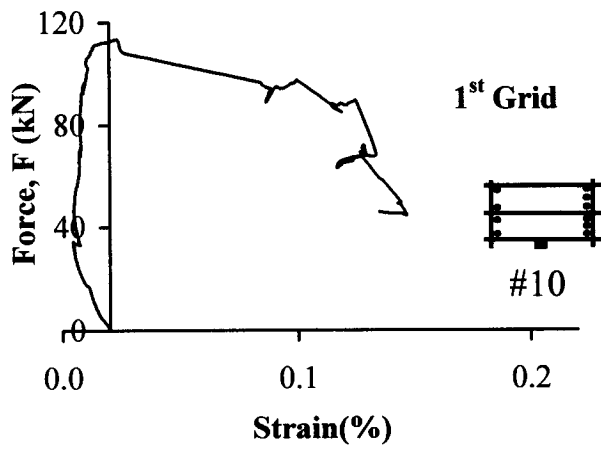
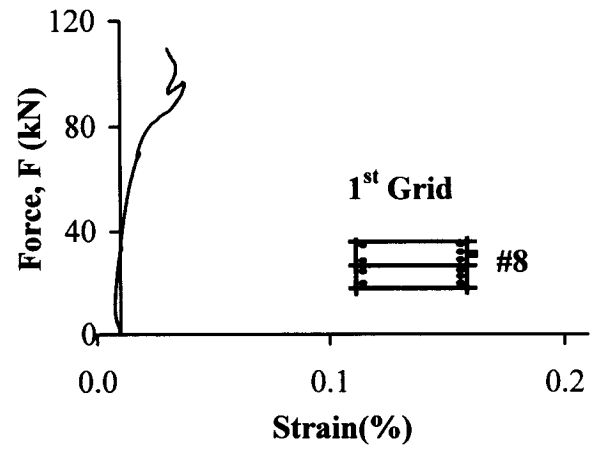
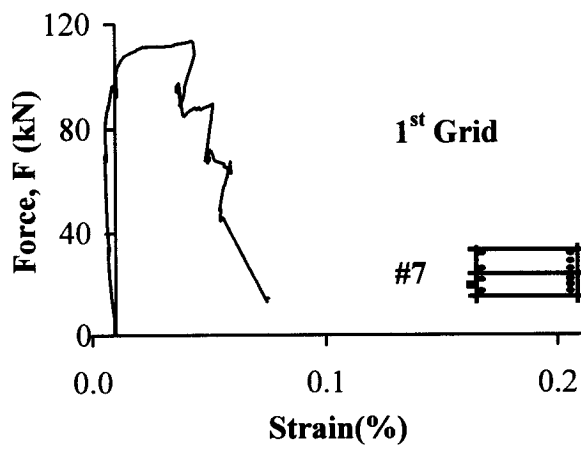


Figure 5.35: Grid Strain Readings in Beam CFB6

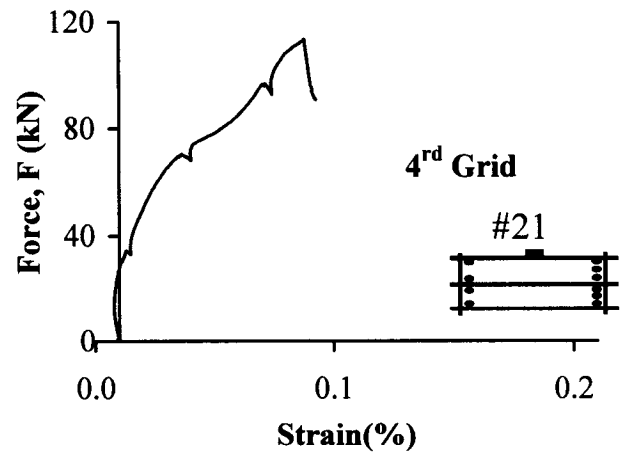
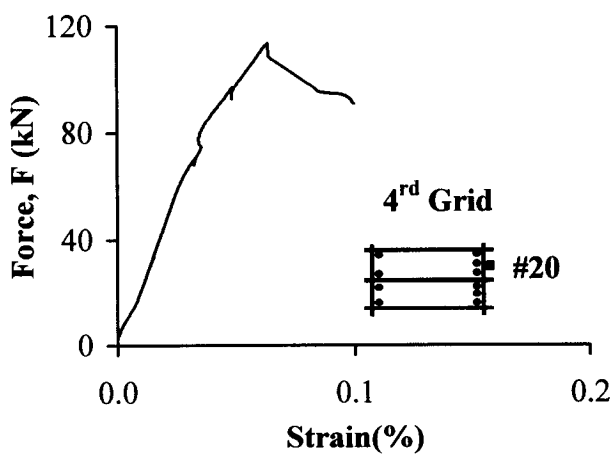
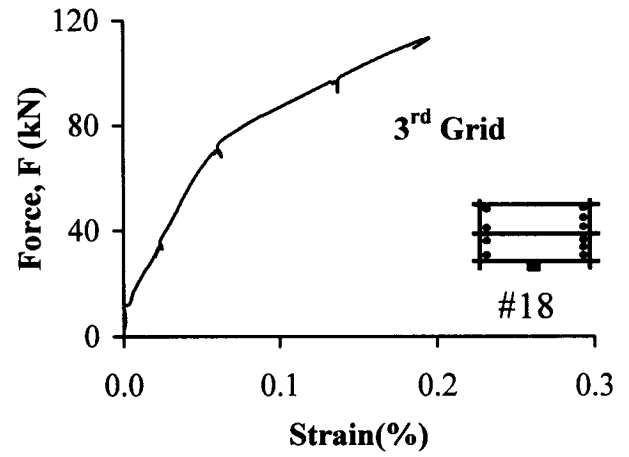
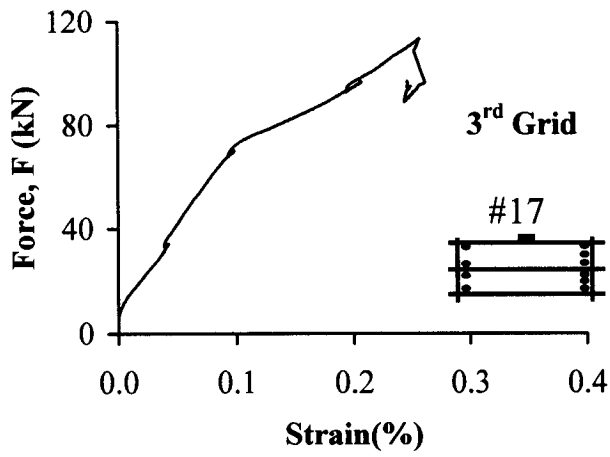
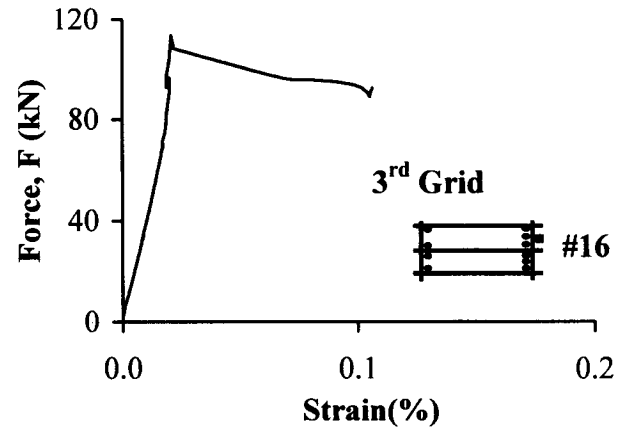
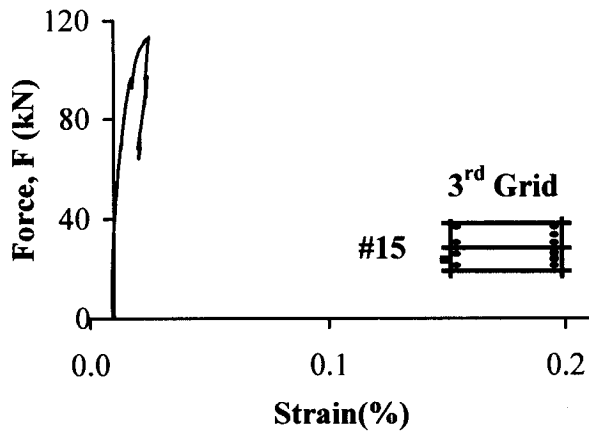
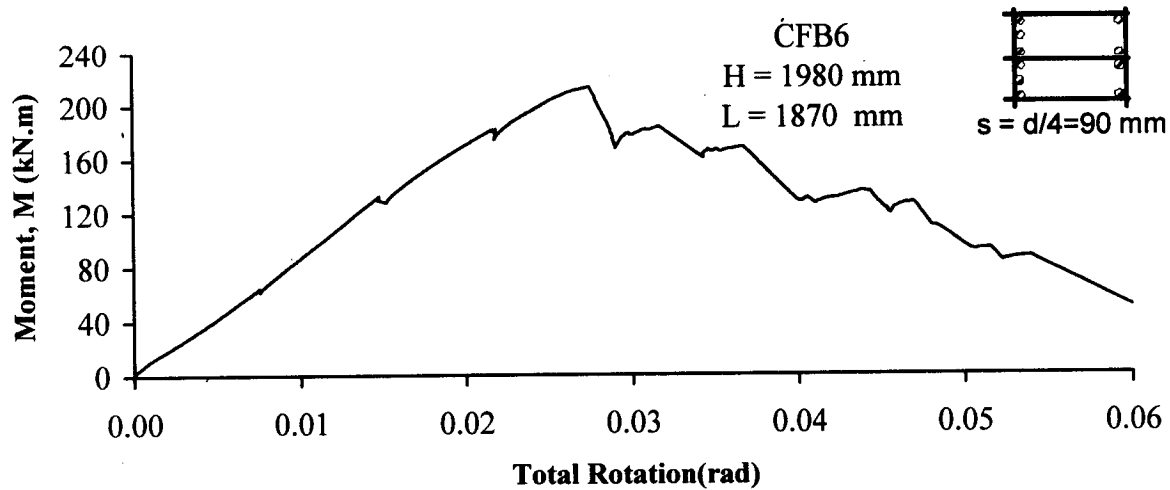
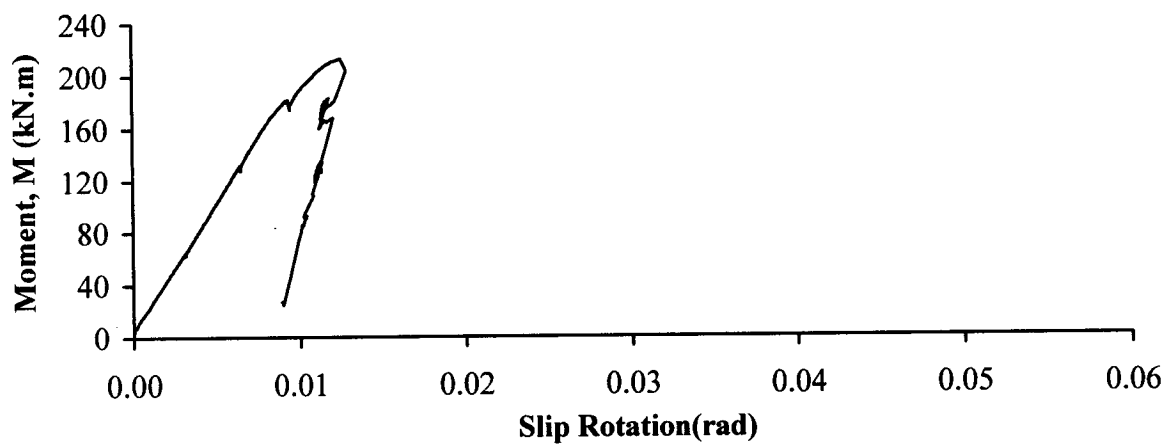


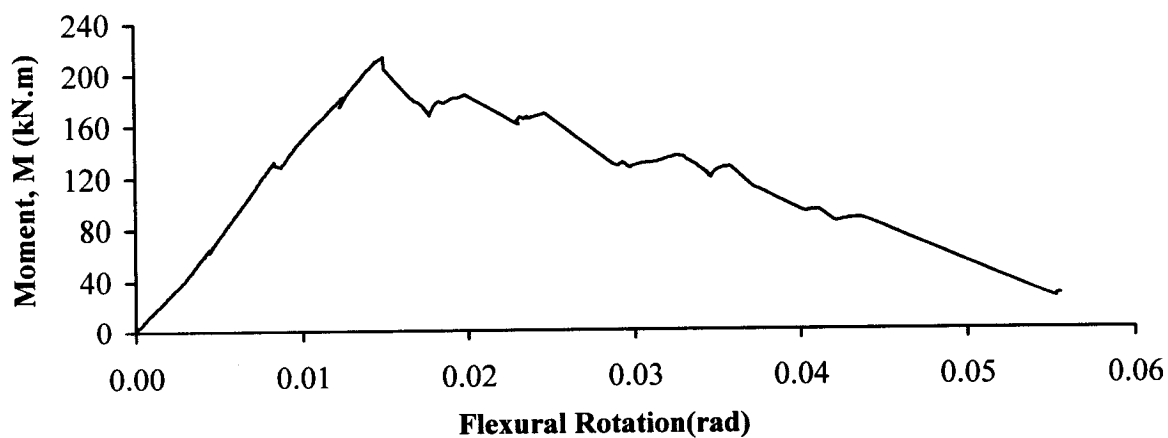
Figure 5.35(Cont.): Grid Strain Readings in Beam CFB6



(a) Moment-Total Rotation Relationship

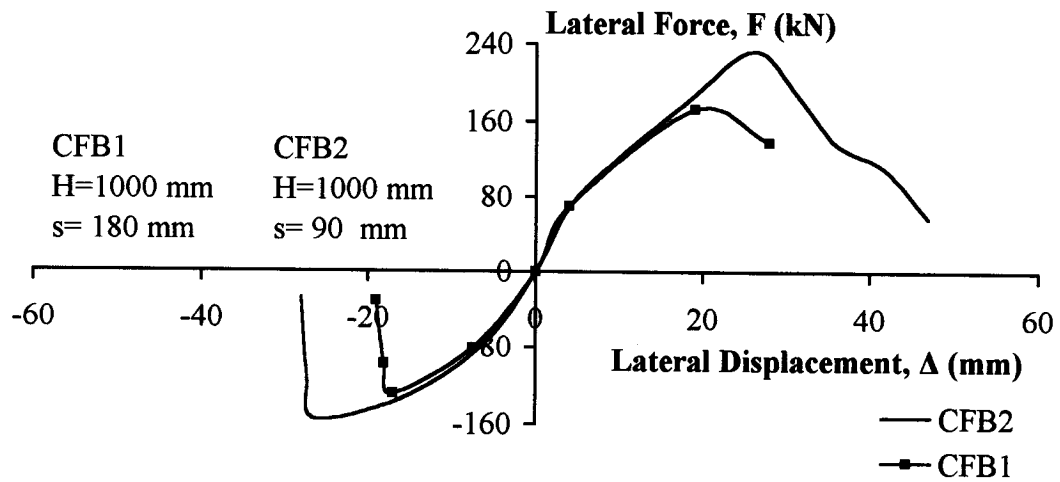


(b) Moment-Slip Rotation Relationship

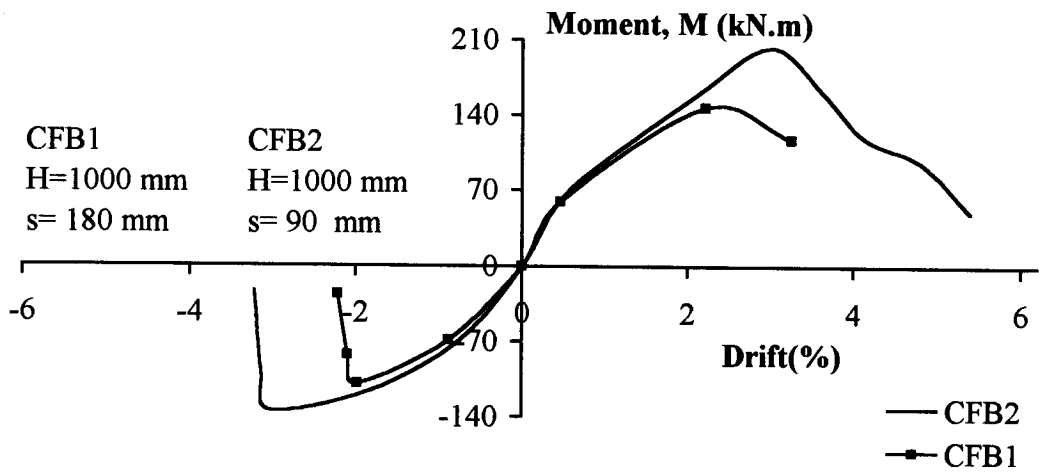


(c) Moment-Flexural Rotation Relationship

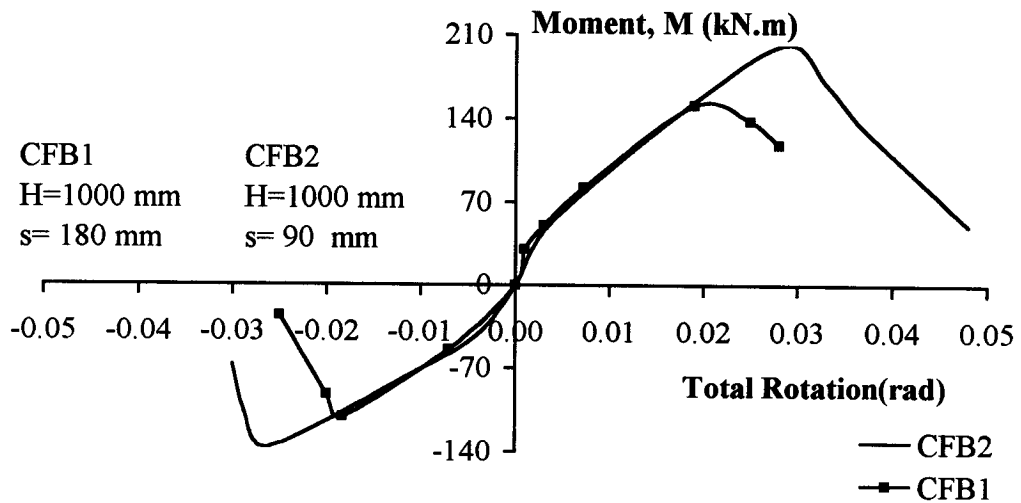
Figure 5.36: Hysteretic Moment-Rotation Relationships for Beam CFB6



(a) Force-Displacement Relationship



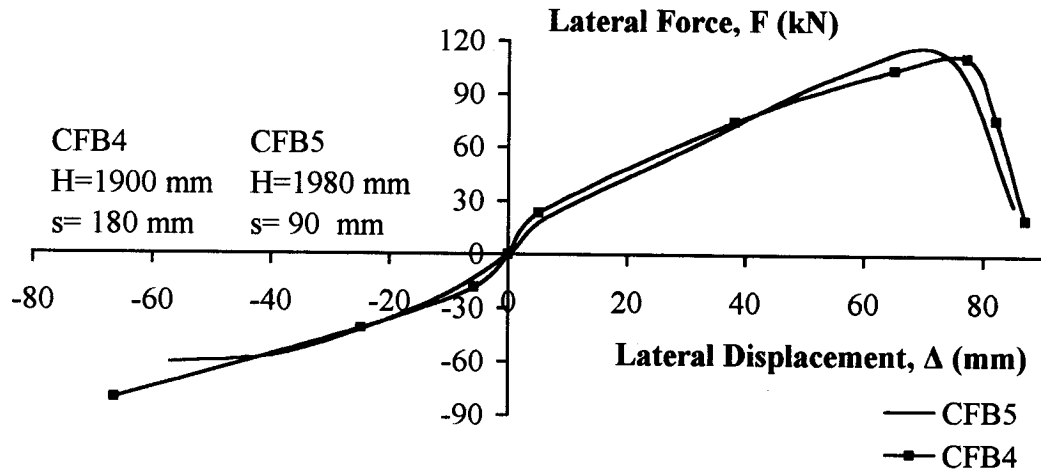
(b) Moment-Drift Relationship



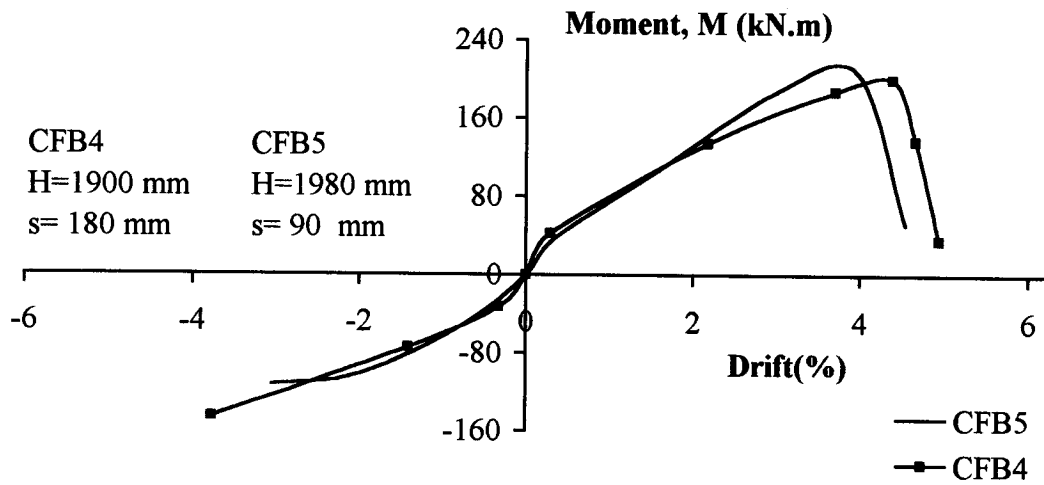
( c ) Moment-Total Rotation Relationship

Figure 5.37: Comparison of Beams for the Effect of Spacing of Grids-CFB1 & CFB2

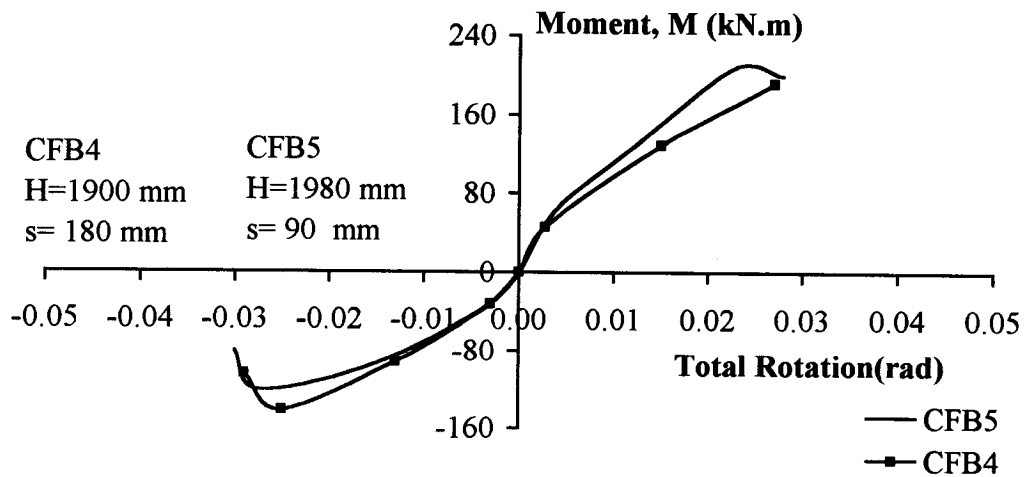




(d) Force-Displacement Relationship

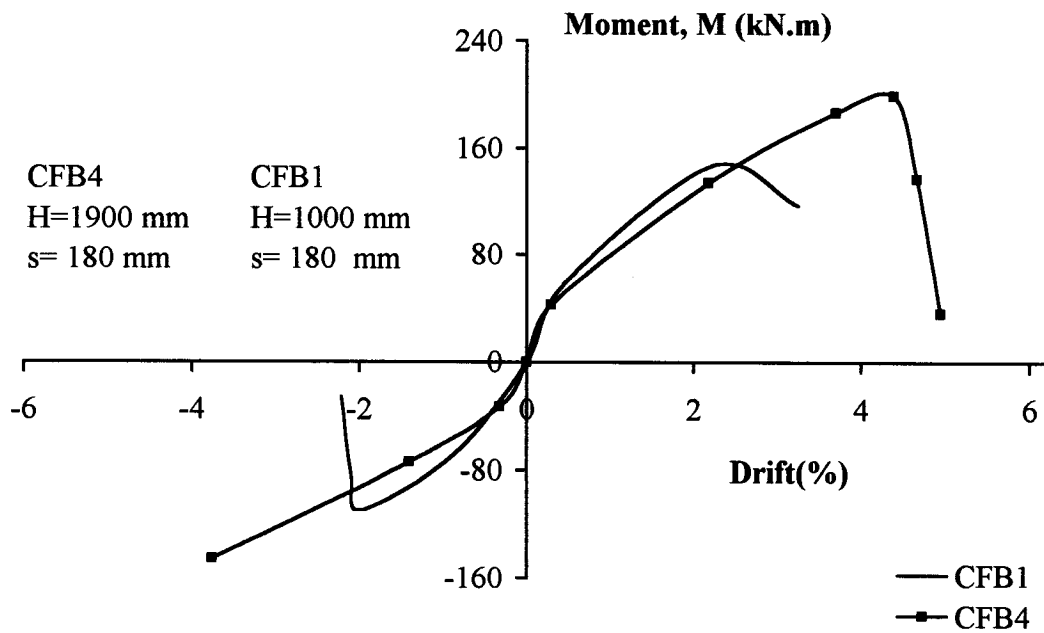


(e) Moment-Drift Relationship

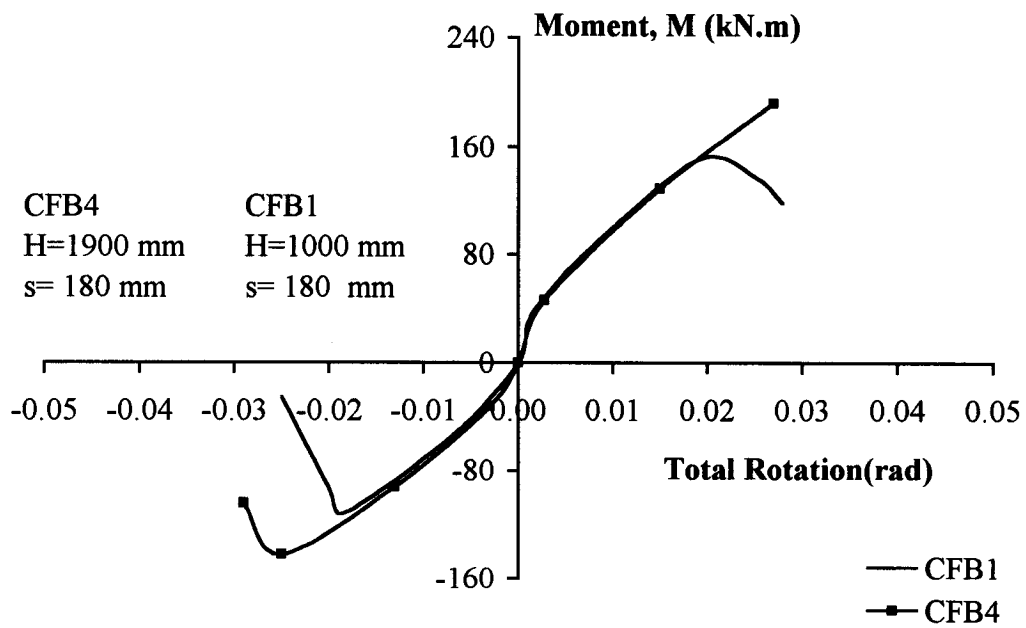


( f ) Moment-Total Rotation Relationship

Figure 5.37(Cont): Comparison of Beams for the Effect of Spacing of Grids-CFB4 & CFB5

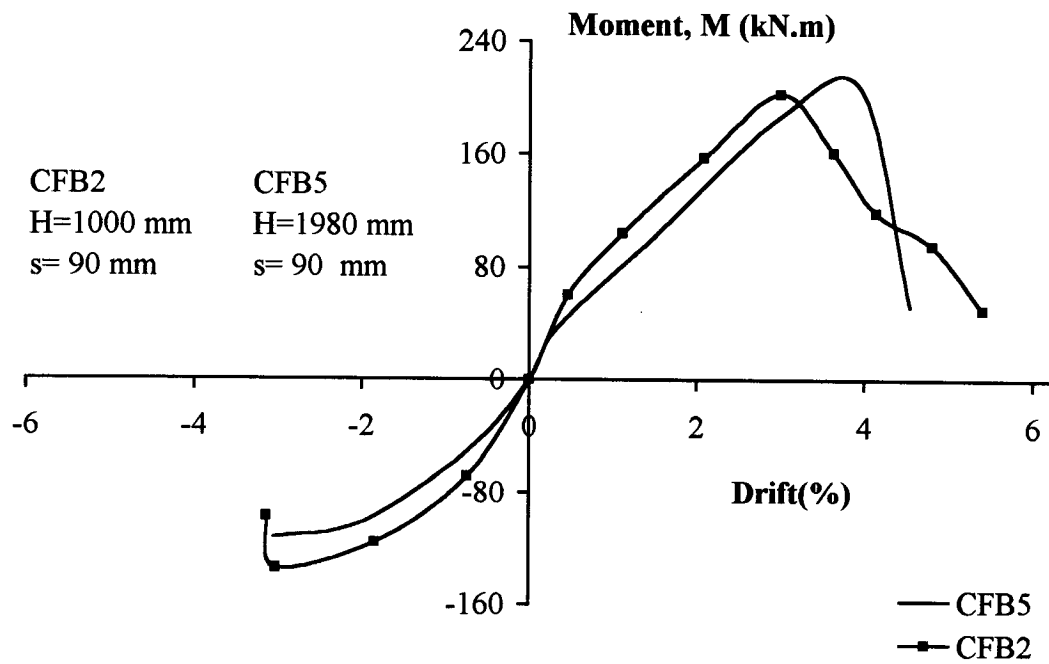


(a) Moment-Drift Relationship

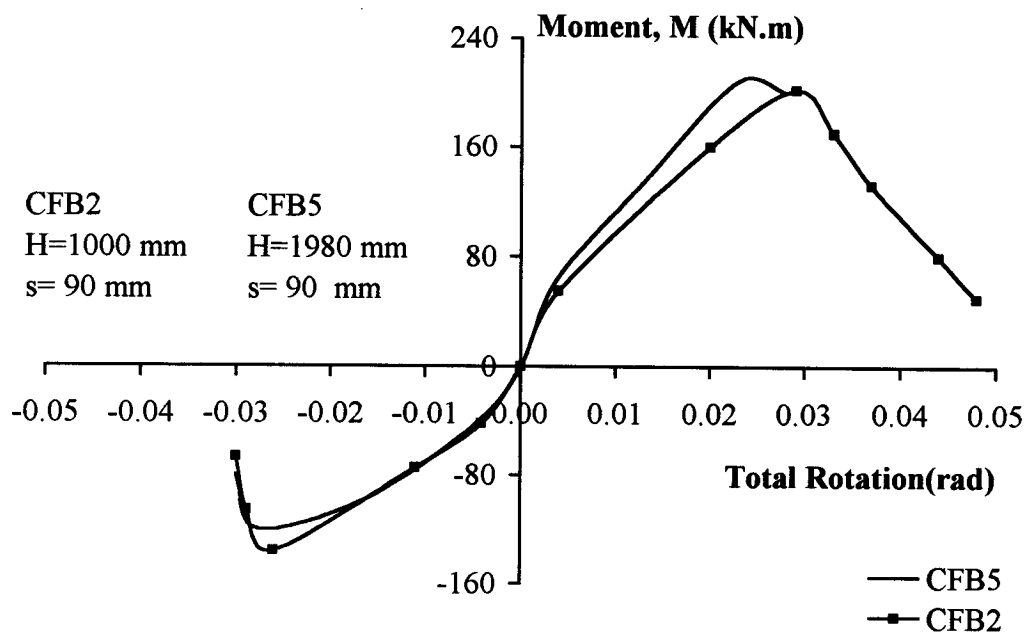


(b) Moment-Total Rotation Relationship

Figure 5.38: Comparison of Beams for the Effect of Shear Spans-CFB1 & CFB4

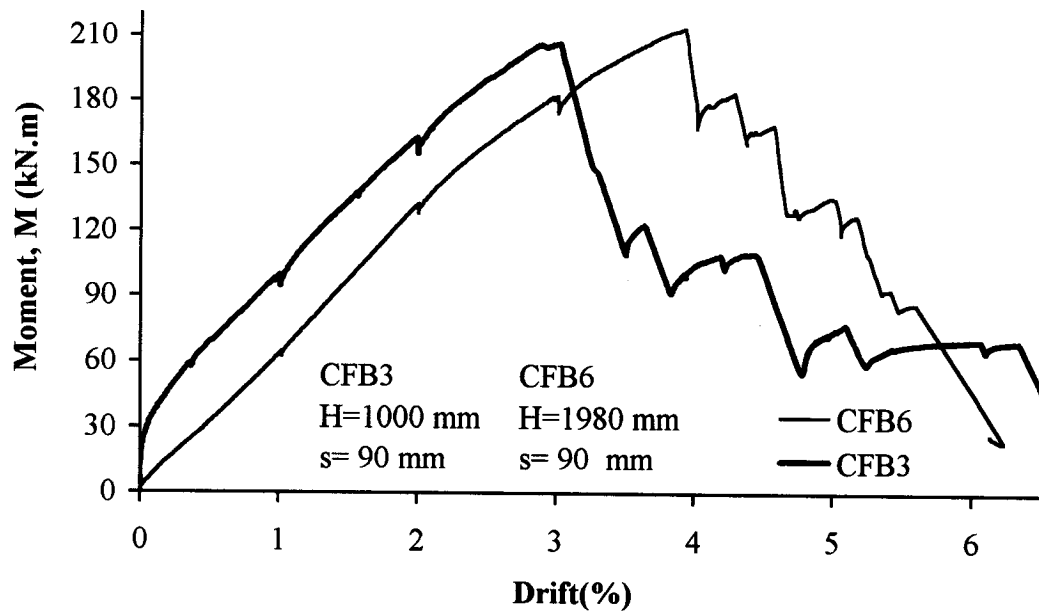


(a) Moment-Drift Relationship

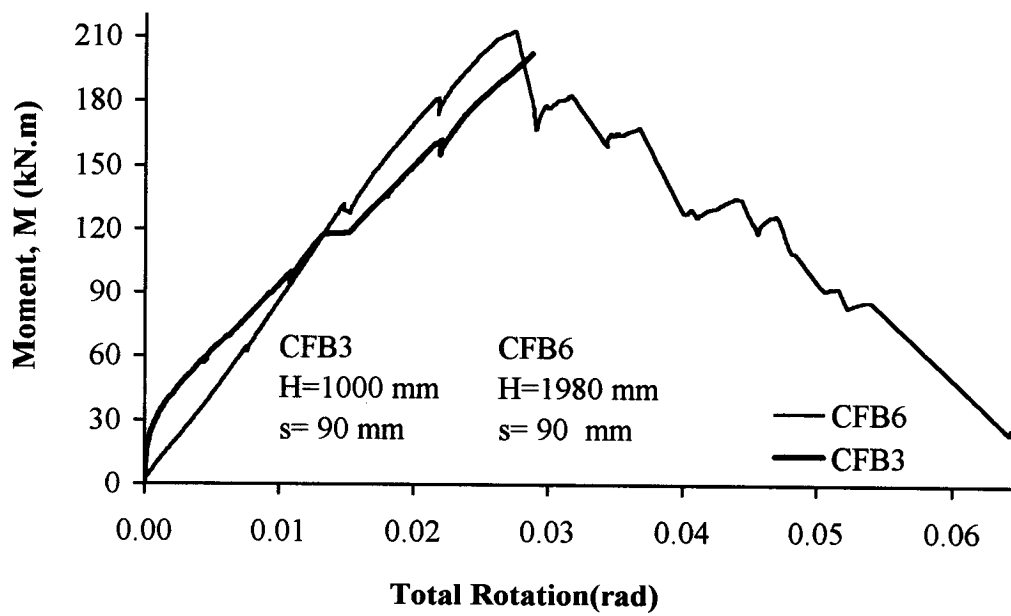


(b) Moment-Total Rotation Relationship

Figure 5.38 (Cont.) : Comparison of Beams for the Effect of Shear Spans-CFB2 & CFB5

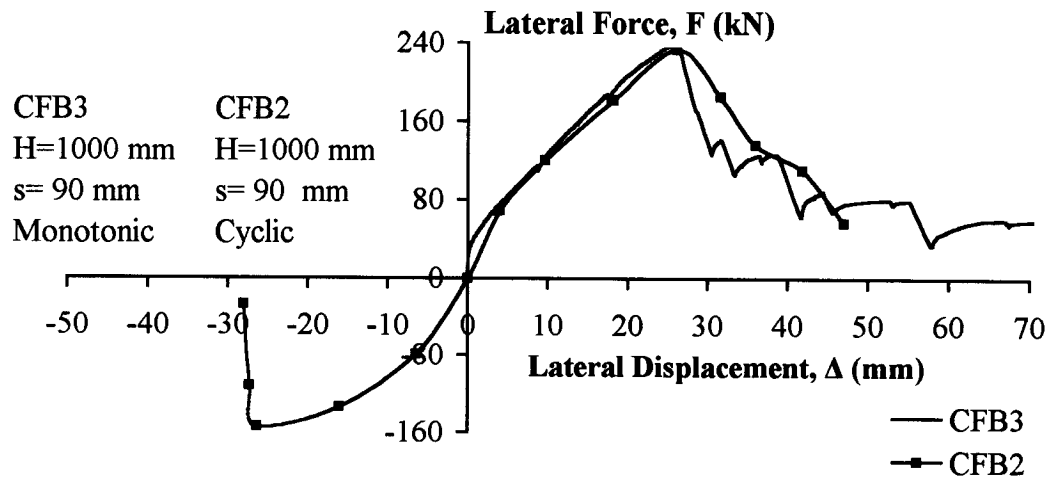


(a) Moment-Drift Relationship

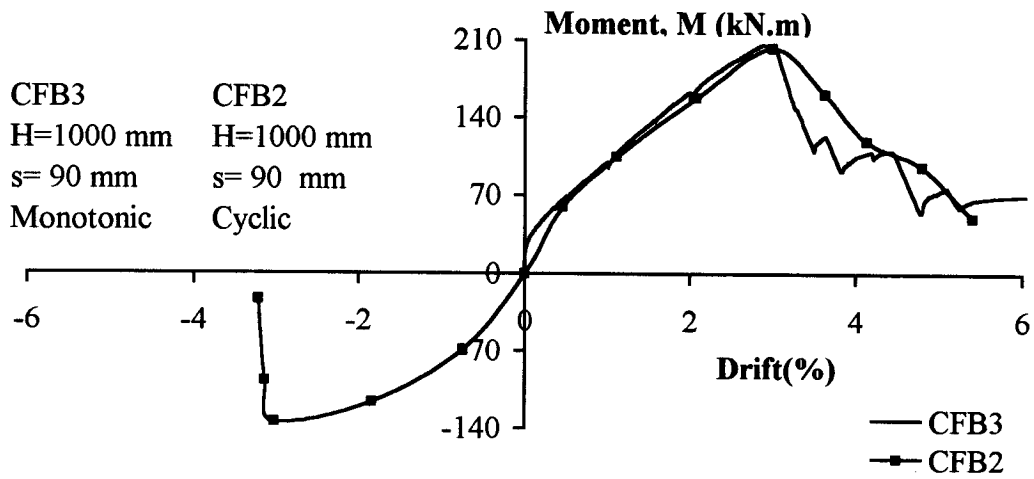


(b) Moment-Total Rotation Relationship

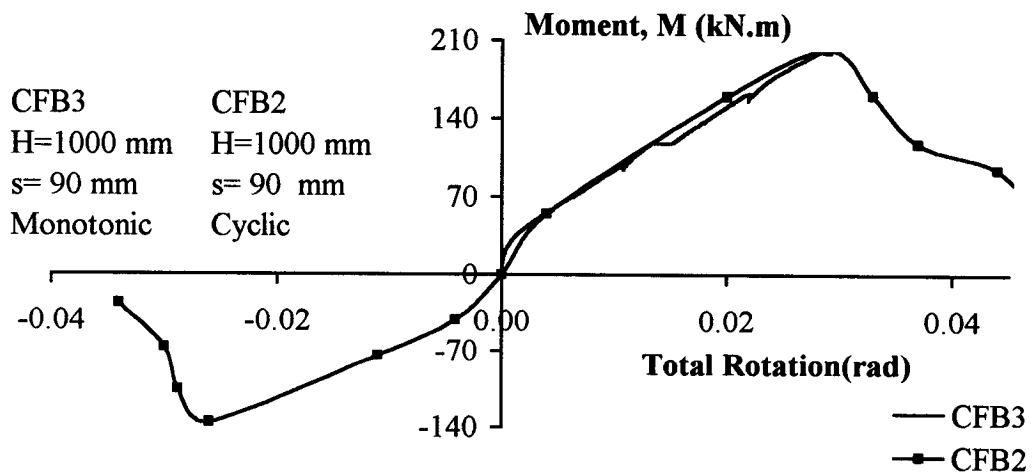
Figure 5.38 (Cont.) : Comparison of Beams for the Effect of Shear Spans-CFB3 & CFB6



(a) Force-Displacement Relationship

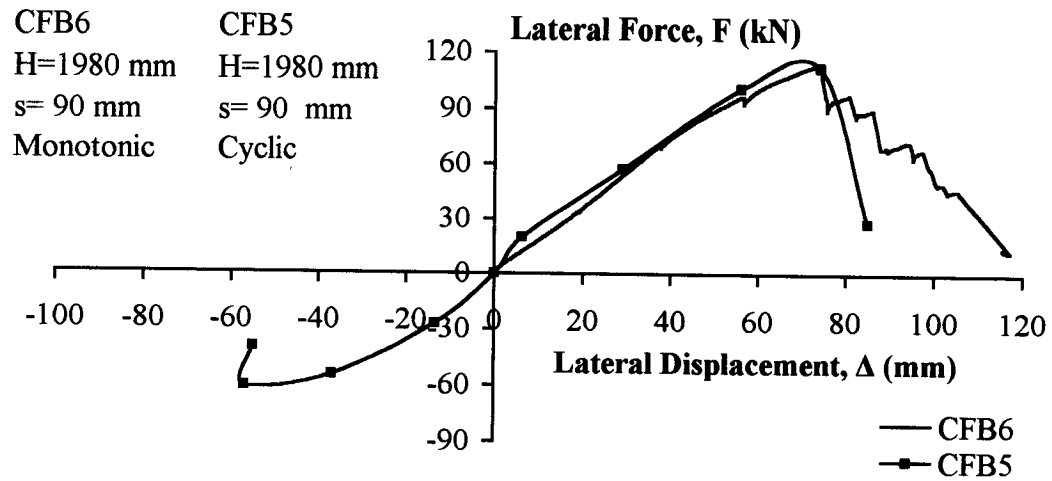


(b) Moment-Drift Relationship

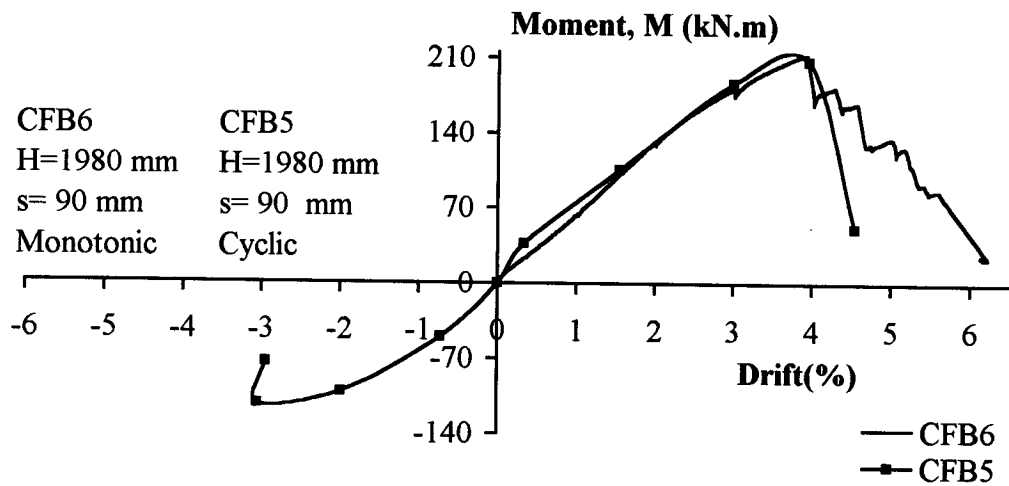


( c ) Moment-Total Rotation Relationship

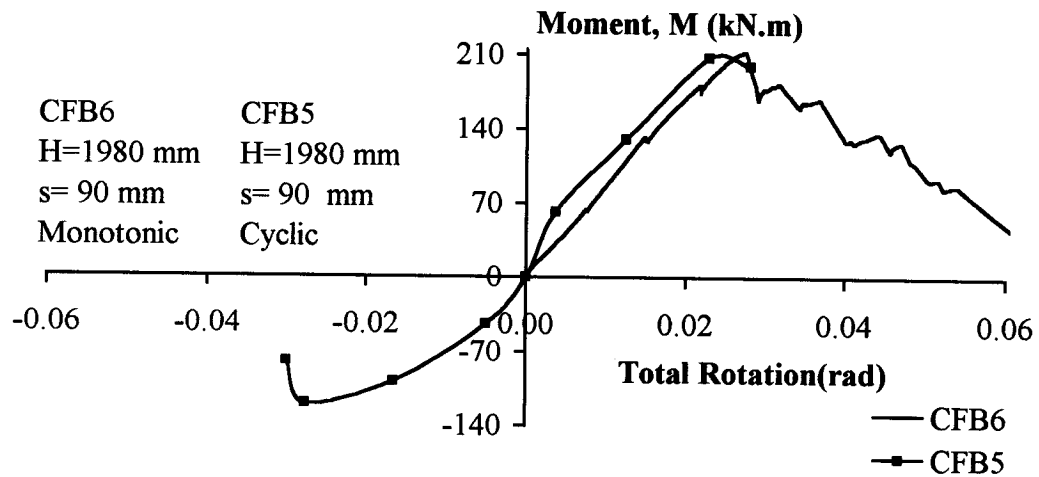
Figure 5.39: Comparison of Beams for the Effect of Cyclic Loading-CFB2 & CFB3



(d) Force-Displacement Relationship



(e) Moment-Drift Relationship



( f ) Moment-Total Rotation Relationship

Figure 5.39(Cont): Comparison of Beams for the Effect of Cyclic Loading-CFB5 & CFB6

## ***Chapter 6***

### **Analytical Research**

#### **6.1 General**

The current research consists of experimental and analytical components. The following topics were researched analytically, based on the experimental data presented in Chapters 2 through 5:

1. Current analytical techniques intended for steel reinforced concrete structures were assessed for their applicability to FRP reinforced concrete structures. Specifically, moment curvature analysis in bending, interaction diagrams for combined axial compression and bending and 45-degree truss analogy for shear were examined.
2. Stress-strain behaviour of FRP reinforcement was established under tension and compression.
3. A hysteretic model was developed for seismic analysis of FRP reinforced concrete structures.
4. The applicability of an existing concrete confinement model was verified for FRP reinforced concrete elements.
5. Recommendations were made for design of FRP reinforced concrete structures under combined axial compression, flexure and shear.

#### **6.2 Plane Section Analysis for Flexure**

Flexural behaviour of a concrete beam or a column with steel reinforcement can be established by plane section analysis. Plane section analysis is based on the following assumptions:

1. Plane sections before bending remain plane after bending. This implies that the strains across the depth of a section are proportional to distances from the neutral axis.
2. Perfect bond is assumed between the steel and concrete. Strain in steel is equal to the strain in surrounding concrete at the same location.
3. Material stress-strain models are available, and stresses in concrete and reinforcement can be obtained from these models for the strain conditions at hand.
4. Concrete below the neutral axis is assumed to be cracked and its contribution to moment resistance is neglected.
5. Force and moment equilibrium are satisfied at the section.

The plane section analysis provides moment and axial force resistances of a section for a given strain profile. Different strain profiles may be considered under increasing flexural stress conditions and the variation of sectional moment resistance can be plotted against sectional curvatures. The resulting diagram provides a moment-curvature relationship, which exhibits all the relevant characteristics of a section in terms of strength, stiffness and deformability. In steel reinforced sections, the failure is governed either by the yielding of steel reinforcement followed by the crushing of concrete or the crushing of concrete prior to the yielding of steel reinforcement, depending on whether the section is under-reinforced or over-reinforced, respectively. Rarely, if the amount of steel reinforcement is very small, the steel yielding is followed by rupturing in tension prior to the crushing of concrete. This form of failure is prevented in practice through minimum steel requirements. Under-reinforced sections are promoted in flexural members due to their inherent ductility. However, the behaviour of an under-reinforced section changes when the member is subjected to significant compression, leading to the crushing of concrete prior to the yielding of steel reinforcement, as often observed in column sections.

Moment curvature analysis is a useful tool for members under constant axial load. Beams, with little or no axial loads, are often evaluated by plotting their moment-curvature relationships, though the same can also be done for columns under a constant axial compression. Flexural analysis of columns under combined bending and axial load can be



conducted to establish their capacity by utilizing the plane section analysis as well. In this case, the strain condition at failure is considered for various levels of axial load, and the corresponding moment capacity is computed. If the analysis is repeated for different levels of axial load, then the variation of moment capacity with axial load can be plotted for the ultimate strain condition. The resulting relationship is known as a “Column Interaction Diagram.”

The applicability of the above assumptions and approach to FRP reinforced concrete is verified in this section. A computer program was used to generate analytical moment-curvature and moment axial force interaction relationships, as applicable, and compared with experimental results. An important step in conducting the analyses was the consideration of realistic material models for constituent materials. The stress-strain relationship for concrete was adopted from earlier research, as discussed in the following section, while the stress-strain relationship for FRP reinforcement was established experimentally.

### **6.3 Analytical Models for Unconfined and Confined Concrete**

FRP reinforcement exhibits linear stress-strain relationship, unlike steel reinforcement, which yields and promotes ductile behaviour in a member. Therefore, one approach to designing FRP reinforced concrete structures is to allow sufficiently high safety margin in design to ensure elastic behaviour of material during service loading and accept brittle failure at ultimate, initiated either by FRP rupturing in tension or concrete crushing in compression. Another approach is to reduce the safety margin and allow the concrete to develop its post peak stresses prior to the tensile failure of FRP. This approach becomes attractive if the compression concrete is confined by reinforcement to have sufficient ductility in the structure. Confinement of concrete by steel reinforcement has been researched extensively in the past. Researchers proposed analytical models to express the stress-strain relationship of concrete confined by steel reinforcement (Richart et al 1928, Balmer 1949, Kent and Park (1974), Sheikh and Uzumeri 1980, Scott et al. 1982, Park and Pauley 1982, Mander et al. 1988, Saatcioglu and Razvi 1992). While these models have different features and somewhat different approaches, they all incorporate the lateral confining action of transverse reinforcement on the basis of the amount, spacing and yield strength of transverse steel

reinforcement. When dealing with FRP reinforcement, which has linear stress-strain characteristics, the applicability of earlier models becomes questionable.

A review of a model for unconfined concrete and three commonly employed confinement models is made in the following sections. This is done to illustrate differences caused by confinement, while also providing information on the specifics of the models used in subsequent analyses. However, all of these models were originally developed for steel reinforced concrete and they have been adapted to FRP reinforcement before they were used.

### 6.3.1 Unconfined Concrete Model

Concrete is a non-linear material showing a stress-strain relationship similar to a second-order parabola. Numerous researchers proposed models for unconfined concrete. The analytical model that was proposed by Hognestad (1951) has been accepted universally as representative of normal-strength unconfined concrete behaviour, which also led to the rectangular stress block used in the ACI 318-02 Building Code and other codes and standards in the world.

The stress-strain relationship of concrete is often defined mathematically on the basis of five properties; (1) compressive strength  $f'_c$ , (2) secant modulus of elasticity  $E_c$ , (3) the shape of ascending branch, (4) strain at peak stress  $\epsilon_0$ , and (5) the shape of descending branch. Sometimes a number of these parameters are combined in one mathematical expression. Concrete compressive strength is established by standard cylinder tests conducted at 28 days. The secant modulus of elasticity,  $E_c$ , is obtained as the slope of the line between zero load and  $0.45 f'_c$  under slowly applied monotonic loading. Several researchers have proposed expressions for the elastic modulus, such as that proposed by Gardner and Lockman (2001). The CSA A23.3 (1994) expression, given below, was used in the analytical part of the current investigation.

$$E_c = 4500\sqrt{f'_c} \quad (6.1)$$

where;  $f'_c$  is specified compressive strength of concrete. The ascending branch of the stress-strain relationship shows nearly linear variation up to approximately 50% of  $f'_c$ , and follows a parabolic variation thereafter. The strain at peak stress, usually denoted by  $\epsilon_0$ , ranges between 0.0015 and 0.0024, depending on the strength of concrete, though most normal strength concretes in the range of 30 MPa to 40 MPa develop 0.002 strain at peak stress. The descending branch starts as the continuation of ascending parabola, but becomes linear as plain concrete unloads suddenly shortly after the peak stress.

The above behaviour of concrete under uniaxial compression was modeled by Hognestad (1951) who suggested the following second-order parabola for the ascending branch.

$$f_c = f'_c \left[ \frac{2\epsilon_c}{\epsilon_0} - \left( \frac{\epsilon_c}{\epsilon_0} \right)^2 \right] \quad (6.2)$$

The descending branch was modelled as a straight line connecting  $\epsilon_0$  strain at peak stress and 0.0038 strain at 85% of the peak stress. This model is shown in Figure 6.1 (a), and was used in the current research program to model unconfined concrete, mostly for the cover concrete.

### 6.3.2 Kent and Park Model

Kent and Park (1974) developed a model for confined concrete, which did not originally incorporate the effect of confinement on strength enhancement but subsequently modified (Park and Paulay, 1982) to eliminate this deficiency. The stress-strain curve consists of a parabolic ascending branch and a linear descending branch with a slope changing with the amount of confinement. The ascending branch consists of a parabola and the descending branch is represented by a straight line, connecting the peak to 20% of the peak at a strain level that depends on the level of confinement. Figures 6.1 (b) shows the modified Kent and Park model.

The ascending branch of the model is described by the following expression:

$$f_{cc} = K f_c'' \left[ \frac{2 \varepsilon_c}{0.002K} - \left( \frac{\varepsilon_c}{0.002K} \right)^2 \right] \quad (6.3)$$

The descending branch is defined by the following expression;

$$f_{cc} = K f_c'' [1 - Z_m(\varepsilon_c - 0.002 K)] \geq 0.2K f_c' \quad (6.4)$$

Where;  $f_c'' = 0.85 f_c'$

Coefficients of  $K$  and  $Z_m$  are given by the following expressions;

$$Z_m = \frac{0.5}{\frac{3 + 0.29 f_c'}{145 f_c' - 1000} + 0.75 \rho_s \sqrt{\frac{h''}{s}} - 0.002K} \quad (6.5)$$

$$K = 1 + \frac{\rho_s f_{yh}}{f_c'} \quad (6.6)$$

### 6.3.3 Sheikh and Uzumeri Model

Sheik and Uzumeri (1982) presented a model for square sections to determine the stress-strain curve of confined concrete. The researchers were the first to show the effects of longitudinal reinforcement arrangement on concrete confinement. The enhanced strength of confined concrete was calculated using the concept of effectively confined core area. The model incorporates the effects of tie spacing and distribution of longitudinal steel around the core perimeter. An empirical equation was developed to compute the peak stress of confined concrete as a function of effectively confined core area. The confinement coefficients were calibrated for square columns confined with hoops and ties and loaded with concentric compression. Subsequently, Sheikh and Yeh (1986, 1992) modified the Sheikh and Uzumeri model for the effects of eccentricity of loading. The model is illustrated in Figure 6.1 (c).

The ascending branch consists of a second-degree parabola. The strength enhancement factor  $K_s$  and the strain at peak stress,  $\varepsilon_{s1}$  can be computed by the following expressions:

$$K_s = 1 + \frac{B^2}{140 P_{oc}} \left[ \left( 1 - \frac{n c^2}{5.5 B^2} \right) \left( 1 - \frac{s}{2 B} \right)^2 \right] \sqrt{\rho_s f'_s} \quad (6.7)$$

Where;

$B$  : Dimension of square core concrete.

$n$  : Number of parabolas that define unconfined concrete in section, reflecting the effect of reinforcement arrangement.

$c$  : Center to center distance between longitudinal reinforcement.

$\rho_s$  : Volumetric ratio of transverse reinforcement.

$f'_s$  : Maximum steel stress at peak concrete strength, almost  $f_y$ .

$s$  : Spacing of ties.

$$P_{oc} = 0.85 f'_c (A_{cc} - A_s) \quad (6.8)$$

Other parameters of the model are defined by the following expressions;

$$\varepsilon_{s1} = (80 \times 10^{-6}) K_s f'_c \quad (6.9)$$

$$\frac{\varepsilon_{s2}}{\varepsilon_0} = 1 + \frac{248}{c} \left[ 1 - 5 \left( \frac{s}{B} \right)^2 \right] \frac{\rho_s f'_s}{\sqrt{f'_c}} \quad (6.10)$$

$$\varepsilon_{85} = 0.225 \rho_s \sqrt{\frac{B}{s}} + \varepsilon_{s2} \quad (6.11)$$

$$f'_{cc} = K_s f'_{co} \quad f'_{co} = 0.85 f'_c \quad (6.12)$$

Where,  $f'_c$  is 28-day cylinder strength.

#### 6.3.4 Saatcioglu and Razvi Model

Saatcioglu and Razvi (1992) developed a model for confined concrete, based on equivalent lateral pressure provided by confinement reinforcement. The model can be used for different concrete sections with rectangular or square hoops with or without cross ties, circular ties or spirals, welded wire fabric or a combination of different types and geometry of lateral reinforcement. It was used to predict the behavior of confined concrete columns under concentric loading as well as under strain gradient and is illustrated in Figure 6.1 (d). The confined concrete strength is expressed as;

$$f'_{cc} = f'_{co} + k_1 f_{le} \quad (6.13)$$

Where,  $k_1$  is a variable that reflects the relationship between lateral pressure and strength enhancement.  $f'_{cc}$  and  $f'_{co}$  are confined and unconfined strengths of concrete in a member, respectively. The equivalent lateral pressure  $f_{le}$  is computed from hoop tension, as shown below.

$$k_1 = 6.7(f_{le})^{-0.17} \quad (6.14)$$

$$f_{le} = k_2 f_l \quad (6.15)$$

$$f_l = \frac{\sum A_s f_{st}}{b_c s} \quad (6.16)$$

Equation (6.17) defines the confinement efficiency coefficient  $k_2$  as a function of the spacing of ties within the cross-sectional plane. This coefficient is less than 1.0 for rectilinear

reinforcement and approaches 1.0 as the spacing of crossties and tie legs supporting longitudinal reinforcement becomes small, increasing the uniformity of confinement pressure which increases the efficiency of confinement. It is equal to 1.0 for closely spaced circular transverse reinforcement, which develops hoop tension, exerting uniform lateral pressure.

$$k_2 = 0.26 \sqrt{\left(\frac{b_c}{s}\right)\left(\frac{b_c}{s_l}\right)\frac{1}{f_l}} \leq 1.0 \quad (6.17)$$

$A_s$  : Cross section area of individual lateral reinforcement/tie in the direction of lateral load.

$b_c$  : Core dimension measured center to center of perimeter tie.

$s$  : Spacing between lateral reinforcement along member length.

$s_l$  : Spacing of transverse legs or crossties within the cross-sectional plane.

$f_{st}$  : Stress in transverse reinforcement at peak concrete stress.

The strain at peak stress of confined concrete,  $\epsilon_1$  is required to define the ascending branch of the stress-strain curve. It is defined on the basis of the level of confinement and expressed as follows.

$$\epsilon_1 = \epsilon_{01}(1 + 5K) \quad (6.18)$$

where;

$$K = \frac{k_1 f_{le}}{f'_{co}} \quad (6.19)$$

The following expression defines the ascending branch of the stress-strain relationship, which is a modified version of the parabola originally suggested by Hognestad (1951) for unconfined concrete.

$$f_{cc} = f'_{cc} \left[ 2\left(\frac{\epsilon}{\epsilon_1}\right) - \left(\frac{\epsilon}{\epsilon_1}\right)^2 \right]^{\left(\frac{1}{1+2K}\right)} \leq f'_{cc} \quad (6.20)$$

The above expression approaches to that of Hognestad's unconfined concrete model as the strength enhancement coefficient  $K$  approaches zero. The descending branch is linear with a slope defined by strain at 85% of strength beyond the peak ( $\varepsilon_{85}$ ).

$$\varepsilon_{85} = 260 \rho \varepsilon_1 + \varepsilon_{085} \quad (6.21)$$

$$\rho = \frac{\sum A_s}{s (b_{cx} + b_{cy})} \quad (6.22)$$

Where,  $\varepsilon_{085}$  is the strain at 85% of strength beyond the peak for unconfined concrete, and may be taken as 0.0038 in the absence of test data, and  $\sum A_s$  is the total area of combined transverse reinforcement in both cross-sectional directions.

## 6.4 Stress-Strain Relationship for FRP Reinforcement

The stress-strain relationship of FRP bars and grids were established by testing tension coupon tests as discussed in Chapters 2 and 4 when the properties of Leadline and Pultrall bars were discussed. The relationships in three different stress conditions are of importance for the analytical research reported in this Chapter. These include stress-strain relationships in i) tension, ii) compression, and iii) tension compression cycles. In all cases strength and elastic modulus are required for the purpose of material modeling.

### 6.4.1 Stress-Strain Relationship in Tension

Stress-strain relationship of FRP reinforcement in direct tension can be obtained by standard coupon tests. This is reported in Chapter 2 for LeadLine bars and in Chapter 4 for Pultrall bars and both show elastic behavior up to tensile fracture, with no yielding and ability to develop plastic deformations. Accordingly, the tensile strength for Leadline and Pultral bars are 2500 MPa and 1470 MPa, respectively. The fracturing strains are 1.6% and 1.25% for



LeadLine and Pultrall bars, respectively, with corresponding elastic modulus of 147,000 MPa and 120,000 MPa. These values were used with linearly varying stress-strain relationships to model the longitudinal reinforcement in tension.

Tensile stress-strain relationships of NEFMAC grids were established by direct coupon tests, reported in Chapter 2. The results indicate that the grid reinforcement has a tensile strength of 1250 MPa with a fracture strain of 1.6%, resulting in an elastic modulus of 78,000 MPa. However, the grids were manufactured by overlapping carbon fibers at grid joints bonded with epoxy, which was used as the resin for the composite material. The joint strength limited the grids from attaining their material strength in tension when they were used as transverse reinforcement. The maximum tensile strain recorded on grids during column and beam tests varied between 0.6% and 0.7% without any sign of failure in FRP material or in grid joints. In earlier research by Grira and Saatcioglu (1999), who used the same NEFMAC FRP grids as transverse column reinforcement with steel longitudinal bars, it was reported that some grids failed at approximately 0.6% strain in trying to prevent compression bars from buckling. A closer examination of the strain gauge data recorded during the current investigation revealed that the grids consistently reached 0.2% to 0.25% strain as the members attained their initial peak or near-peak resistances, and increased up to 0.3% to 0.35% during peak resistance and reached up to 0.6% to 0.7 % at the end of column tests after experiencing significant strength decay. Therefore, the stress in grids that led to the development of confinement pressure on concrete was variable. It is convenient to select a constant value of tensile stress in transverse reinforcement for applying the confinement models discussed earlier, all of which are based on a constant value for stress in transverse reinforcement. In this investigation, an average strain value of 0.4% strain and corresponding stress value of 320 MPa was adopted in applying the confinement models. A sensitivity study was also conducted with different grid stresses to assess the significance of this parameter on concrete confinement. It was concluded that, for the members tested in this investigation, the overall impact on member behaviour was small and the use of 320 MPa FRP stress produced reasonably good agreement with experimental results.

### 6.4.2 Stress-Strain Relationship in Compression

The stress-strain relationship of FRP bars in compression is difficult to establish by tests since the stability of bars must be maintained during testing. Therefore, samples having lengths equal to 3 to 5 times the bar diameter were tested under direct compression until failure. The stress-strain relationships in compression are reported in Chapters 2 and 4 for LeadLine and Pultrall bars, respectively. The failure stress in compression varied between 240 MPa and 310 MPa for Pultrall bars. LeadLine bars showed approximately 275 MPa as the compressive failure stress. These corresponded to 16% to 21% of the tensile strength of Pultrall bars and 11% of LeadLine bars. The corresponding compressive failure strain varied between 1% and 1.3% for Pultrall bars and was approximately 1.8% for LeadLine bars. The failure in direct compression tests was caused either by delamination of fibers and crushing of resin or by splitting of bars longitudinally. The former type of failure was observed when the compression bar sample was 3 to 5 times the bar diameter, and the latter failure type was observed when the sample length was twice the bar diameter. However, the failure type did not appear to affect the compressive strength and failure strain. The behaviour of FRP material in compression is representative of that of the entire composite material with negligibly small contributions from the resin and sensitivity to the alignment of fibers. It was reported by Broutman (1974) that the orientation of fibers in FRP material played an important role on the compressive strength of material. Figure 6.2 illustrates the relationship between the compressive strength and angle of misalignment of fibers, indicating that a 5-degree misalignment would lead to 25% reduction in strength. In FRP reinforcing bars manufactured for use in concrete, where the main intent is to resist tensile stresses, it is normal to expect some misalignment of fibers.

Though material failure was observed at high compressive strains, comparable to the strains at tension failure, the actual FRP bar behaviour in concrete member was different. As discussed in Chapters 2 to 4 the bars suffered from stability failure in compression and failed at lower compressive strains, depending on the confinement of concrete and the lateral straining of transverse grids. It was observed that Pultrall bars with 9.5 mm diameter failed at about 0.39 to 0.44% compressive strain when the grid spacing was  $h/2 = 175$  mm. The range changed to 0.49% to 0.54% when the grid spacing was reduced to  $h/4 = 88$  mm. The lower

values of these ranges were observed in columns confined with 4-cell grids (8 longitudinal bars) and the higher values were observed in columns with 9-cell grids (12 longitudinal bars). Similarly, LeadLine bars with a smaller diameter of 8.0 mm showed stability failure at 0.33% and 0.36% compressive strain in 4-bar columns when the grid spacing was  $h/2 = 115$  mm and  $h/4 = 58$  mm, respectively. In an 8-bar column with a grid spacing of  $h/2 = 115$  mm the failure strain was 0.4%. Conservative compressive strain values for bar buckling may be taken as 0.4% and 0.5% for 9.5 mm Pultrall bars with  $s/d_b$  ratios of 18.5 and 9.2, respectively. Similarly, compressive strain limits of 0.3% and 0.35% may be used for 8.0 mm diameter LeadLine bars with  $s/d_b$  ratios of 14.4 and 7.2, respectively. Therefore, the following expression is suggested to calculate the buckling strain of FRP bars:

$$\epsilon_{c,frp} = 86 \times 10^{-5} \left( 10 + 0.19 n_1 - \frac{46}{d_b} - 0.095 \left( \frac{s}{d_b} \right) \right) \quad (6.23)$$

Where;  $n_1$ ,  $d_b$ , and  $s$  are the number of bars in the arrangement, nominal diameter of FRP bar, and grid spacing, respectively.

The average modulus of elasticity in compression was 23,000 MPa and 15,000 MPa for Pultrall and LeadLine bars, respectively. These values correspond to 20% and 10% of elastic modulus in tension for Pultrall and LeadLine bars, respectively. The experimentally recorded values of elastic modulus were used in member analysis, with linearly varying stress-strain relationships, up to failure.

#### 6.4.3 Stress-Strain Relationship in Tension – Compression Cycles

The stress-strain relationship for longitudinal bars in tension – compression cycles were established through the strain data recorded in column and beam specimens. The strain readings recorded at the same moment level after different number of cycles was approximately the same, showing a few percentage differences up to the peak resistance of members. This difference was attributed to the progressive cracking of concrete in member

and experimental scatter. This was true for both tension and compression strains. The strains showed up to approximately 15% increase with number of cycles beyond the peak load, which was attributed to the progressive crushing of compression concrete and increased cracking of tension concrete. Table 6.1 shows strains recorded at different cycles of load, confirming this observation. Therefore, the stress-strain relationship of FRP bars in concrete subjected to tension-compression cycles was assumed to be the same as that obtained under tension and compression coupon tests separately under monotonically increasing loading. Figure 6.3 illustrates the stress-strain relationship of FRP bars.

## 6.5 Moment-Curvature Relationships

Moment-curvature relationships were computed by plane-section analyses, conducted as described in Section 6.2, and were plotted for comparison with experimentally obtained relationships. Computer software "RC Section" (Goudreault 1998) was employed for this purpose. Phase I columns were subjected to monotonically increasing axial compression. Therefore, the moment-curvature analysis was conducted for different levels of constant axial load, and the results were combined in one curve to reflect the effect of changing axial compression. Phase III columns were subjected to constant axial load and the analysis was conducted for this level of constant compression. Hognestad's concrete model (1955) was used for cover concrete and Saatcioglu and Razvi confinement model (1992) was used for core concrete. Concrete strength in member was assumed to be the same as that obtained by standard cylinder tests. Previous research indicated that, while the difference between the two strength values could be as high as  $0.15f_c$  for concentrically tested columns, this difference approached zero for columns and beams under strain gradient. The effects of using different confinement models were also investigated as discussed later in this section. The analysis continued until the buckling of compression bars occurred at experimentally recorded buckling strains.

Experimental curvatures were obtained based on the LVDT readings. Phase I columns were tested under eccentric loading with maximum moment region at column mid-height, and were instrumented with LVDTs placed in this region. The LVDT gauge length was 300 mm

and hence measured *average curvature* over this length. Consequently, the moments used in the plots were also the *average moment* within the 300 mm gauge length at column mid-height. Similarly, Phase III and IV specimens were instrumented with LVDTs within the potential plastic hinge region. The columns of Phase III had a gauge length of 355 mm (which was equal to the column dimension). Two sets of measurements were taken; i) total rotation of the section 355 mm above the footing, and ii) the rotation of the column base section at 40 mm above the footing, which would provide column rotation caused by anchorage slip (elastic extension of FRP bars within the footing). The 40 mm distance was needed to be able to attach the LVDTs, which inevitably resulted in LVDTs picking up flexural rotations within this short length. Therefore, the anchorage slip readings were slightly higher than what they should be. This point was further investigated by computing anchorage slip from recorded strain data, as discussed in Section 6.6. The difference between measured total rotation and anchorage slip gave flexural rotation within 355 mm of column segment. The flexural rotation divided by the segment length provided experimental *average curvature* to be plotted against *average moment* within the 355 mm segment for comparison with the analytical moment-curvature relationship. The comparison for each column is illustrated in Figures 6.4 and 6.5. The figures also include a second set of relationships for experimental moment-curvature relationships, this time obtained from the strain gauges located on longitudinal FRP bars at column-footing interface. These relationships show curvatures and corresponding moments at exactly the column-footing interface.

The comparisons show very good agreement in nominal moment capacities between analytical and experimental values, confirming that plane section analysis is applicable to FRP reinforced concrete members. Comparisons of complete moment-curvature curves show some small differences between analytical and experimental relationships which are attributed to experimental scatter and inaccuracies in large-scale testing and analytical modeling. However, the observed differences are not believed to be any more significant than those that would be obtained when a similar comparison is made for steel reinforced concrete elements.

Comparisons between analytical and experimental moment-curvature relationships for beams tested in Phase IV were made in much the same manner as those for columns. The same

material models used for columns were also used for the beams, with zero axial load. The experimental curves were obtained from the LVDT readings with appropriate reductions in total rotations to account for anchorage slip. The gauge length for beam LVDTs was equal to 405 mm (equal to the beam dimension in the direction of loading). Average curvature and corresponding average moment within this gauge length were plotted in Figure 6.6 to be compared with analytically computed relationships. Also included in these figures are the experimental moment curvature relationships obtained from FRP strain gauge data at beam-column interface.

The results indicate approximately linear moment-curvature relationship for both analytical and experimental values because of the failure mode of beams. All beams failed by rupturing of FRP bars in tension prior to the crushing of concrete. Maximum compressive fiber strain recorded was approximately 0.003, which was only slightly above the strain at unconfined concrete strength. The agreement between analytical and experimental values is reasonably good, once again confirming the applicability of plane section analysis to FRP reinforced concrete beams.

The influence of different confinement models was investigated before the analytical moment-curvature relationships were established. Columns CFCL2 and CFCL4, as well as Beam CFB5 were analyzed using the three confinement models described in Sect.6.3. The results are plotted in Figure 6.7 and show that the relationships obtained by Saatcioglu and Razvi and Sheikh and Uzumeri models produced similar results while those obtained from the use of modified Kent and Park model was close, but somewhat different.

## **6.6 Moment-Anchorage Slip Relationships and Development of Longitudinal Bars**

Reinforced concrete members in flexure develop two deformation components when the critical section is located at the intersection of two members. This condition occurs in most members subjected to lateral loading. Straining of reinforcement at the intersection of two members propagate into the adjoining member, causing extension of the reinforcement

within the adjoining member, changing the fixity condition at member end. This effect is not accounted for ordinary flexural analysis. For example, the flexural analysis of a beam is conducted by considering the variation of moments and corresponding curvatures along the beam length between the columns, based on the elastic member stiffnesses. However, the beam may be relaxed at its supports due to the straining of anchoring reinforcement in the columns. The accumulation of strains in beam reinforcement within the column (within embedment length) results in a crack at beam ends, at the interface of two members. This results in a rigid body rotation that cannot be computed by flexural analysis. A similar phenomenon occurs in columns, and becomes especially critical at column footing interface where high interstorey drifts occur during lateral loading. The deformations caused by the extension of reinforcement in adjoining members are referred to as anchorage slip (and not necessarily bar slip).

Anchorage slip was investigated in Phase III and IV tests by instrumenting the member end regions by LVDTs, as well as by placing strain gauges on FRP reinforcement within the adjoining member. While two LVDTs placed on opposite faces of members in this region would record the member end rotation due to the anchorage slip directly, the strain gauges would give strain profiles within the adjoining member, from which bar extension and anchorage slip could be computed. The rotations recorded by LVDTs would contain flexural rotations that would occur within the gauge length. The gauge length should ideally be close to zero to measure the crack width at the member interface, spanning between the member end section and the surface of the adjoining member. However, LVDTs require some length to be secured. Therefore, the LVDTs placed on columns and beams would pick up flexural rotations within the gauge length and hence would give slightly higher values than the true deformations caused by anchorage slip. The gauge length was discussed in Chapters 4 and 5, as well as in Section 6.4 to be 40 mm for columns and 25 mm for beams.

Strain readings in longitudinal FRP bars inside the footing (attached columns for beams) are illustrated in Figure 6.8 at different stages of loading. The same figure also shows the progression of development length under increased loading. The area under the strain profile gives the extension of FRP bar within the footing. The extension, divided by the lever arm

within member section (d-c) gives the rotation at member end caused by anchorage slip. This approach of computing anchorage slip rotations is the same as that proposed earlier by Alsiwat and Saatcioglu (1992) for steel reinforced concrete members. Figure 6.9 shows the comparison of moment anchorage slip rotation relationships that were directly obtained by LVDT readings and those computed from strain data. The results look similar, with LVDT reading showing little higher values because of the inclusion of flexural rotations within the gauge length, as explained above.

Bond between FRP bars and concrete must be provided to transfer stresses from concrete to bars, and vice versa. Beams and columns are anchored to attached members by means of reinforcement, and this reinforcement must have sufficient development length to develop the bar forces. The minimum length of bar embedded in concrete beyond the critical section, to be able to resist maximum bar force, is defined as the development length. Development length is usually found from equilibrium of forces. The force (tension or compression) in FRP bar is equal to the resisting force found by the bond stress multiplied by the surface area. This equilibrium condition results in the computation of development length. A large number of parameters play important roles on bond, including; surface condition, bar diameter, shape and embedment length. Therefore, the variation and magnitude of bond is a complex phenomenon that is difficult to quantify accurately. It is usually acceptable to use a constant level of bond stress for the development length calculations. Two kinds of failure may be considered to determine the required development length; (i) splitting failure and (ii) pullout failure. Some researchers; Faza and Gangarao (1990) and Ehsani et al. (1996) suggested expressions based of these two failure modes. CSA 806-02 presents two different expressions for development length of FRP bars in tension, as defined below, using the CSA terminology:

(i) Normal Requirement:

$$l_d = 1.15 \frac{k_1 k_2 k_3 k_4 k_5}{d_{cs}} \cdot \frac{f_f}{\sqrt{f'_c}} \cdot A_b \quad (6.24)$$



(ii) Permitted Variation:

$$l_d = 0.5 \frac{k_1 k_2 k_3 k_4 k_5 f_f}{\sqrt{f'_c}} d_b \quad (6.25)$$

$k_1, k_2, k_3, k_4, k_5$  = Modification Factors

$k_1$  = Bar location factor, equal 1.3 for horizontal bars placed more than 300 mm of fresh concrete is cast in the member below the development length, mostly in all specimens tested in this investigation

$k_2$  = Concrete density factor, equal 1.0 for normal density concrete

$k_3$  = Bar size factor, equal 0.8 for 9.5 mm diameter bar with  $A_b \leq 300$  mm

$k_4$  = Bar fibre factor, equal 1.0 for CFRP bars

$k_5$  = Bar surface profile factor, equal 1.0 for sand-coated surfaces

$A_b$  = Area of individual bar

$d_{cs}$  = the smaller of

(a) the distance from the closest concrete surface to the centre of the bar being developed.

(b) two-thirds of the centre-to-centre spacing of the bars being developed.

But  $d_{cs}$  shall not be taken greater than  $2.5 d_b$ .

$d_b$  = nominal diameter of bar

$f_f$  = design stress in FRP tension reinforcement, which is 75% of tensile strength

$f'_c$  = specified compressive strength of concrete

Equation (6.25) is permitted if the minimum concrete cover and center-to-centre spacing of bars are equal to  $1.5 d_b$  and  $1.8 d_b$ , respectively. Small concrete cover promotes splitting in concrete, necessitating a longer development length computed by Equation (6.24). Concrete cover greater than  $2 d_b$  with widely spaced longitudinal bars, such as the case in the

specimens of the current investigation, becomes critical in pull-out failure. These bars require relatively shorter development length.

ISIS network for centres of excellence also recommended design expressions for the basic development length, as recommended by “ISIS-M04-00 Draft, 2000”. The recommendations were for the two failure modes, as shown below:

i) *For splitting tensile crack:*

$$l_d = 0.028 \frac{A_{frp} f_u}{\sqrt{f'_c}} \quad (6.26)$$

ii) *For Pullout:*

$$l_d = 0.054 d_b f_u \quad (6.27)$$

A modification factor of 1.3 was recommended by Ehsani and Saadatmanesh (1996) for top bars. In addition, a factor of 1.5 was specified to be used with Equation (6.26) for concrete cover less than  $2 d_b$ . The above expressions are based on  $f_u$ , which is the design tensile strength of FRP bars obtained from tension coupon tests as the maximum tensile strength, which may be reduced by 10% for environmental effects.

ACI Committee 440 recommendation (2001) for the computation of basic development length is given below:

$$l_{bf} = \frac{d_b f_{fu}}{18.5} \quad (6.28)$$

Where  $d_b$  and  $f_{fu}$  are the nominal diameter and maximum tensile strength of FRP bar, respectively. Modification factors of 1.3, 1.0, and 1.1 are recommended for top bars, cover

greater than  $2 d_b$ , and normal concrete, respectively. Modification factor of 1.5 is recommended if the concrete cover is less than  $2 d_b$ . Equation (6.28) was derived on the basis of test data conducted on GFRP bars with a lower tensile strength. Therefore no modification factor is presented for CFRP bars and for different bar diameters.

The development length for 9.5 mm Pultrall bars was computed by Equation (6.24) to be 640 mm. Assuming the pull-out mode of failure was the governing mode, because of sufficiently thick concrete cover, Equation (6.27) produced a development length of 870 mm. Equation (6.28) resulted in a value of 870 mm, similar to the value computed by Equation (6.27).

Columns and beams of Phase III and IV of the experimental investigation were instrumented with strain gauges to establish the development length requirements for Pultrall bars. Strain gauges were placed on embedded bars that anchor test specimens to the adjoining members. The gauges were placed at member interface and 170 mm into the adjoining member. The variation of strains is given in Fig. 6.8 at different stages of loading. Figure 6.10 includes all the strain data plotted on one curve, summarizing the experimental data for FRP strains as a function of development length. The experimental data on development length allowed the computation of bond stress  $u_f$  between the FRP bars and concrete, as illustrated below.

$$T = A_{frp} f_{FRP} = u_f \pi d_b \ell_d \quad (6.29)$$

$$u_f = \frac{d_b f_{FRP}}{4 \ell_d} \quad (6.30)$$

The experimental data, shown in Figure 6.11 indicates that a constant bond stress of 5.5 MPa can be used for the Pultrall bars and concrete strength used in the experimental program. Assuming this level of constant bond stress at ultimate bar pull-out condition, the minimum design development length may be derived in three different forms, as indicated below.

$$l_d = \frac{0.035 A_{frp} f_u}{\sqrt{f_c}} \quad (6.31)$$

$$l_d = 0.045 f_u d_b \quad (6.32)$$

$$l_d = \frac{f_u d_b}{22} \quad (6.33)$$

Where,  $l_d$ ,  $f_u$ ,  $d_b$  are the tension development length of FRP bar in mm, tensile strength from coupon tests, and nominal diameter of bar, respectively. ACI design practice for steel reinforcement calls for a safety factor of 1.25 (ACI 318-1995). If the same safety factor is applied to Equations (6.31) to (6.33), the computed development length becomes approximately 25% below the value recommended by ACI Committee 440. The CSA Standard 806-02 incorporated the 25% safety margin in their expressions, because the design tensile strength of bars was limited to 75% of tensile rupturing strength. The difference in development lengths, computed by CSA Equation (6.28) and Equation (6.31) was only 8%, indicating good correlation between the CSA approach and those recorded experimentally.

## 6.7 Moment-FRP Strain Relationships

One of the objectives of the current research program was to investigate the performance of FRP reinforcement in tension, compression and tension-compression reversals. Therefore, the strain gauge data obtained from the bars were analyzed once again in an effort to explain bar behaviour in detail. It is not possible to plot experimental stress-strain relationships using the information obtained from tests of reinforced concrete elements. However, it is useful to see the behaviour of longitudinal reinforcement under increasing bending moment. Figures 6.12, 6.13, and 6.14 illustrate element moment versus FRP strain relationships under both tension and compression. Analytically calculated moment-strain relationships, obtained from plane section analyses, are also included in the same figures. For analytical calculations it was assumed that FRP bars had a linear behaviour in tension and compression, with reduced

elastic modulus in compression, as illustrated in Fig. 6.3. The analyses were conducted until the experimentally observed maximum compressive strains were recorded at bar buckling, or tensile rupturing of FRP bars have occurred, whichever was attained first. The comparisons indicate reasonably good agreement, providing confirmation of the applicability of plane section analysis to FRP reinforced concrete members with the stress-strain relationship of FRP bars as shown in Fig. 6.3.

## **6.8 Moment-Axial Force Interaction Diagrams**

Capacities of reinforced concrete columns are established through axial force-moment interaction diagrams. These diagrams give the failure surface for a column section. The compressive and tensile strengths of FRP bars, as well as their stress-strain relationships are different than those of steel reinforcement. Therefore, the interaction diagrams for FRP reinforced concrete columns are different than those for steel reinforced concrete columns and need to be verified against experimental data before they can be used for design purposes.

In FRP reinforced concrete columns under high levels of axial compression the failure is initiated by the crushing of concrete, as in the case of steel reinforced concrete columns. As the level of axial load decreases, the corresponding moment capacity increases. Steel reinforced columns develop a balanced strain condition when the crushing of concrete and yielding of steel occur simultaneously. FRP reinforcement does not have ability to yield, but does sustain high strains within the elastic range. Drawing analogy between steel reinforced and FRP reinforced columns, one may define a balanced strain condition for FRP reinforced columns as the point at which simultaneous concrete crushing and FRP rupturing occurs. However, FRP reinforced columns show different behaviour when they are reinforced with high percentage of FRP reinforcement, in which case they may fail by the crushing of concrete even under a very low or zero axial compression. Therefore, it is possible to have a column interaction diagram for FRP reinforced columns with concrete crushing governing the entire failure surface with FRP remaining elastic below the rupturing point. If the amount of FRP reinforcement is low, it is possible to have a failure condition that is governed by the rupturing of FRP in tension prior to the crushing of concrete. It is clear that the interaction

diagrams for FRP reinforced column can be different than those for steel reinforced columns. This is shown in Figure 6.15.

Two types of interaction diagrams were plotted for the columns tested in the current investigation. First, the nominal column interaction diagrams were plotted using 0.0035 maximum compressive strain as the compression failure condition and tensile rupturing strain as the tension failure condition, whichever occurred first. Second, interaction diagrams were plotted by considering column confinement. In this case the failure condition was not defined on the basis of an assumed ultimate strain level, but obtained from moment-curvature relationships as the strain condition at which maximum moment resistance was attained for axial force considered. When concrete confinement was considered in heavily loaded columns, the compressive strains were high enough to trigger the stability failure in compression FRP bars followed by the crushing of concrete. When the axial load level was reduced, the failure was initiated either by the crushing of concrete or the rupturing of tension FRP bars, depending on the percentage of reinforcement.

The theoretical moment-axial force interaction diagrams for Phase I and III columns are shown in Figures 6.16 and 6.17, respectively. The balanced axial compression force for Phase III columns was computed to be 450 kN at a maximum compressive concrete strain of 0.0035 and a maximum FRP tensile strain of 0.013. The experimental strength values are also plotted on each interaction diagram. The results indicate that the experimental strength values are consistently above the nominal computed strengths (outside the diagram). They are often very close to the computed values when the confinement of concrete was considered. These comparisons confirm the validity of the plain section analysis once again for flexural members under axial compression. However, the computation of interaction diagrams should be carried out with caution, with the correct limiting strains incorporated for material failure.

## **6.9 Development of an Analytical Model for Hysteretic Moment Displacement Relationship for FRP Reinforced Concrete Members**

Dynamic inelastic response analysis of FRP reinforced concrete members requires realistic conceptual models that can simulate strength stiffness and deformability of members. Currently, there is no analytical model available for the hysteretic behaviour of FRP reinforced concrete members due to lack of research findings. The current research program is one of the few in the world addressing the use of FRP bars in compression members under seismic loading. The experimental phase of research, reported in Chapters 4 and 5 provided sufficient data to develop an analytical model to simulate hysteretic behaviour of flexural members.

FRP reinforced concrete members exhibit significantly different characteristics than those of concrete members reinforced with conventional steel bars, due to differences in stress-strain properties of the two materials. Appropriate modeling of these features is essential for accurate analytical predictions. The proposed model consists of two components; i) primary moment-flexural displacement (chord angle) relationship defining the strength boundary and initial stiffnesses; and ii) hysteretic rules defining the slopes of unloading and reloading branches of moment-displacement relationship under reversed cyclic loading. The primary force-deformation relationship can be computed analytically, starting from material properties and basic concepts of mechanics. The hysteretic rules are obtained from experimental observations and simulate features that are common to members of similar properties.

### **6.9.1 Primary Curve**

Moment-displacement relationship under monotonic loading forms the primary curve. The primary curve provides an envelope of hysteretic moment-displacement relationship and provides a convenient means of defining the strength boundary for the purpose of modeling. This relationship can be generated analytically through incremental analysis, where moment and flexural displacement can be computed for different strain and curvature profiles. It is possible to idealize this relationship by linear line segments for convenience. The primary

curve proposed for this purpose has a tri-linear relationship, within the ascending branch followed by a linear descending segment. This idealization was found to produce good agreement with experimentally recorded envelop curves. Accordingly, the first segment represents the flexural stiffness up to a post-cracking strain of  $\varepsilon = \varepsilon_0 / 2$  at the extreme compression fiber of critical section, which is equal to 0.001 for normal-strength concrete, where  $\varepsilon_0$  is the concrete strain at peak compressive strength of  $f'_c$ . This point was selected for the initial elastic stiffness, instead of the gross-stiffness up to initial cracking moment, since it provided a closer match with the overall curve with least computed points. The second segment represents the cracked stiffness up to the beginning of cover spalling at a maximum compressive fiber strain ( $\varepsilon_0$ ), which can be taken as 0.002 for normal-strength concrete. The moment-curvature relationship shows a marked change in slope when the cover concrete starts spalling off. The computation of the onset of spalling does capture this point on the idealized relationship. The third, and the peak point on the ascending branch corresponds to the failure of FRP reinforcement either due to the buckling in compression or rupturing in tension. These strain conditions are characteristics of material behaviour and can be obtained from the stress-strain relationship of FRP reinforcement.

If the peak resistance is governed by the rupturing of reinforcement in tension, then the post-peak behaviour can be established by conducting column analysis without the ruptured layer of reinforcement. The same is true if the tension reinforcement has previously buckled in compression during loading previous cycle. This results in a sharply reduced strength and increased deflection. If the peak is established by the buckling of compression reinforcement, then the confined concrete may sustain continued inelastic compression and the member may not necessarily experience a sharp decline in load resistance. In this case a post-peak slope can be established by column analysis after discounting the contribution of compression FRP reinforcement.

Figure 6.18 illustrates an FRP reinforced concrete cantilever member, representing a beam or a column, loaded by a lateral and a vertical axial force (if representing a column). Moment and corresponding curvature diagrams are shown for two stages of loading; i) when the extreme compression fiber strain reaches  $\varepsilon_0 = 0.002$  and ii) when the maximum capacity at



failure of longitudinal FRP reinforcement is attained either due to buckling or rupturing. The member displacement for each case can be computed by finding the moment of the area under the curvature diagram about the tip of the member. These result in displacements,  $\Delta_2$ , and  $\Delta_3$ . Figure 6.19(a) illustrates an idealized primary moment-flexural displacement relationship for use with the hysteretic rules outlined below.

The above procedure outlines the construction of moment-flexural displacement primary curve, without the softening effect of the reinforcement anchored in the adjoining member (anchorage slip). The contribution of this deformation component was discussed in Section 6.6, and its effect can be introduced by increasing displacements. The anchorage slip deflection accounted for approximately 25% to 35% of total measured displacement in flexure dominant specimens.

### 6.9.2 Hysteretic Rules

The following rules, defining unloading and reloading stiffness within the primary curve (strength boundary), were obtained from experimental observations, empirically.

1. Initial loading on the primary curve as well as subsequent loadings beyond the previous displacement follows the primary curve.
2. Unloading and reloading between positive and negative  $\Delta_1$  on the primary curve follows the initial stiffness, as shown in Figure 6.19 (b).
3. Unloading from a point between  $\Delta_1$  and  $\Delta_3$  aims at a point on the opposite primary curve with a displacement equal to the average of the two previous maximum displacements in the opposite direction or the opposite  $\Delta_1$ , whichever gives a higher displacement. If the previous displacement in the opposite direction exceeded  $\Delta_1$  only once, then the average of two previous displacements would be computed as the average of the previous maximum displacement and  $\Delta_1$ . Reloading follows the same slope as unloading until the primary curve is reached. This is illustrated in Figure 6.19(c) and (d).

4. Beyond  $\Delta_3$ , the member starts failing with damage to FRP reinforcement either in the form of compression buckling or tension rupturing. Therefore, any loading and reloading in this range of excessive deformations depends on the specifics of the section, primarily on the number and layers of reinforcement. Unloading upon bar failure may be taken as a line aiming towards the origin of moment-displacement relationship (zero point). However, reloading in the opposite direction can not be predicted empirically and the member is best assumed failed at this excessive deformation level.

The comparison of proposed hysteretic model and experimentally obtained moment-flexural displacement relationships are shown for flexure dominant columns CFCL4, CFCL3, and CFCL2 in Figure 6.20. The comparison is made for individual hysteresis loops as well as for the entire hysteretic relationships, and depicts good agreement. Similarly, the comparison of proposed model with hysteretic relationships obtained from FRP reinforced concrete beam tests is made in Figure 6.21. The comparisons indicate good correlations for beams where the failure was governed by bar rupturing, prior to the crushing of concrete. It is worth mentioning that the model was not compared with hysteretic relationships of specimens with short shear spans, since these elements also developed significant inelastic shear deformations, computation of which was not included in the flexural hysteretic model presented in this section.

## **6.10 Shear Resistance of Concrete**

Reinforced concrete members are often modeled using 45-degree truss analogy for the computation of their shear capacity. Current design approach for shear is based on the assumption of a 45-degree diagonal failure plane and contributions provided by concrete and transverse reinforcement. However, the contribution of concrete diminishes with transverse strains and the size of the diagonal crack, as the aggregate interlock mechanism diminishes. Therefore, it is important to bring the stress-strain characteristics of FRP reinforcement into design expressions and re-assess the current approach used for shear resistance of steel reinforced concrete elements.

The CSA S806-02 standard presents the following expressions for nominal shear resistance of concrete for flexural members.

$$V_c = 0.035 \lambda [f'_c \rho_w E_F \frac{V_F}{M_F} d]^{1/3} b_w d \quad (6.34)$$

$$0.1 \lambda \sqrt{f'_c} b_w d \leq V_c \leq 0.2 \lambda \sqrt{f'_c} b_w d \quad (6.35)$$

where;  $\rho_w = \frac{A_{Fv}}{b_w d}$ . These expressions apply to sections having either at least a minimum amount of transverse reinforcement or an effective depth not exceeding 300 mm. The concrete shear resistance of beams with an effective depth greater than 300 mm and with transverse shear reinforcement less than the minimum is presented below:

$$V_c = (\frac{130}{1000 + d}) \sqrt{f'_c} b_w d \leq 0.08 \sqrt{f'_c} b_w d \quad (6.36)$$

Equation (6.34) is applicable to the columns tested in the current investigation. The concrete shear resistance for columns is govern by lower limit of Equation (6.35), and varies between 56 kN and 58 kN depending on the concrete strength and effective depth. The same expressions also apply to the beams of the current investigation when closely spaced grids are used. The shear resistance computed by Equation (6.34) for these beams is 73 kN, within the lower and upper limitations. However, Equation (6.36) applies to concrete shear resistance of Beams CFB1 and CFB4, with  $V_c = 54$  kN, that is govern by upper limit of the expression.

ACI 318-02 Code for steel reinforced concrete buildings has recommendations for concrete shear resistance that may also be applicable to FRP reinforced concrete within the transverse strain limits provided by elastically behaving transverse steel reinforcement. These

expressions are summarized below, and were used to compare with experimentally obtained values.

**i. Simplified Expression for combined flexure and shear**

$$V_c = 0.17\sqrt{f'_c} \ b_w \ d \quad (6.37)$$

**ii. Members subject to axial compression, as well as flexure and shear**

$$V_c = \left(1 + \frac{N_u}{14A_g}\right) (0.17\sqrt{f'_c}) \ b_w \ d \quad (6.38)$$

ACI Committee 440-01 prepared guidelines for design of FRP reinforced concrete structures. Their recommendations include the use of ACI 318 expressions with the following modification.

$$V_{c,f} = \frac{A_f E_f}{A_s E_s} V_c \quad (6.39)$$

The above equation accounts for the axial stiffness of the FRP reinforcement ( $A_f E_f$ ) relative to steel reinforcement ( $A_s E_s$ ).

The contribution of FRP to shear is derived based on 45-degree truss analogy. All codes and standards for concrete structures refer to the following expression for shear resistance provided by transverse reinforcement:

$$V_F = A_F \ f_f \ \frac{d}{s} \quad (6.38)$$

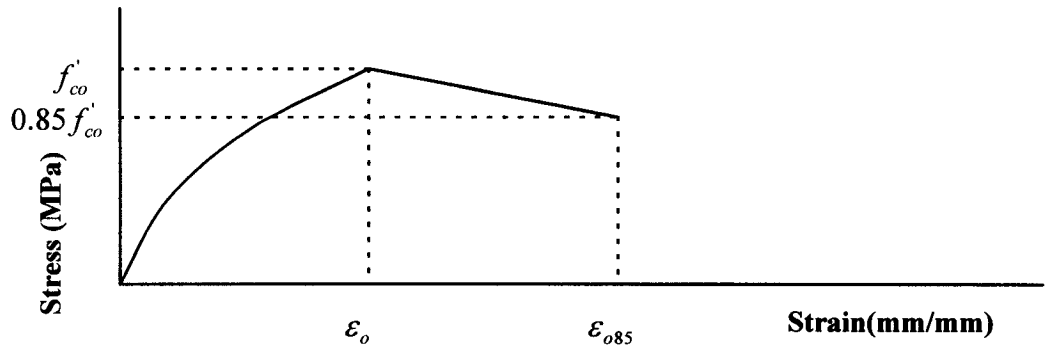
The design tensile strength of FRP grid reinforcement  $f_f$  is limited to  $f_{f \max} = 0.4f_{FU}$  in CSA S806-02.  $A_F$  is the total area of transverse legs of each grid reinforcement. The same expression can be used to compute the experimental contribution of FRP grids to shear in the

specimens tested. In this case the stress in FRP should be computed for each stage of loading from recorded grid strains, in place of  $f_f$ . This can be done by multiplying the maximum recorded strain in two or three grids crossing the main diagonal crack,  $\epsilon_f$ , by the modulus of elasticity of FRP grids. The elastic modulus was established experimentally to be  $E_f = 80000$  MPa. The maximum transverse strain at maximum horizontal load varied between 0.002 and 0.003, though all the specimens except one failed in flexure, but they all exceeded diagonal tension capacity of concrete with some resistance provided by grid reinforcement.

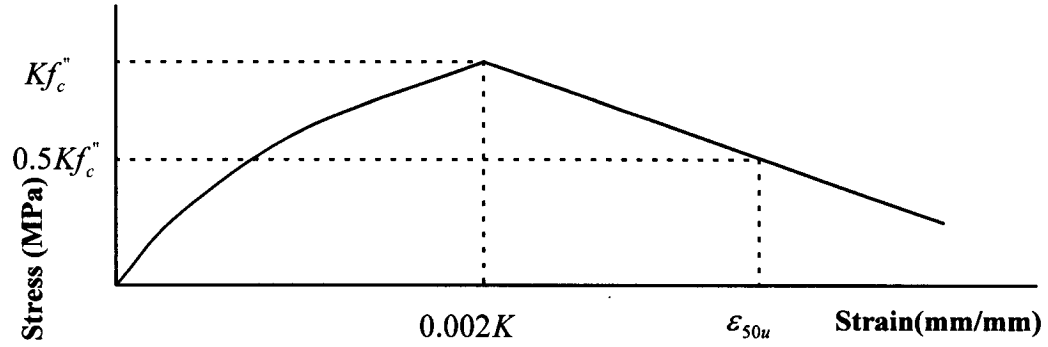
The test data obtained from column and beam tests conducted in the current investigation are compared with the code expressions given above for concrete shear resistance. The experimental values of concrete shear resistance were found by plotting the applied shear force against maximum transverse strains in FRP grids. Figure 6.22 illustrates the variation of maximum recorded transverse tensile strains with applied shear forces on beams and columns. The same figure also provides computed values of  $V_c$  by various codes and standards, as well as the contribution of transverse reinforcement  $V_F$ , as computed from grid strains. The results indicate that the CSA S806 expressions provide a conservative lower bound for all beams and columns, especially for columns under higher axial loads. ACI 318-02 requirements were found to be unsafe for some FRP reinforced concrete elements, while the ACI 440-01 requirements were consistently very conservative.

Table 6.1: FRP Bar Strains Recorded at Different Cycles of Load at Phase III Specimens

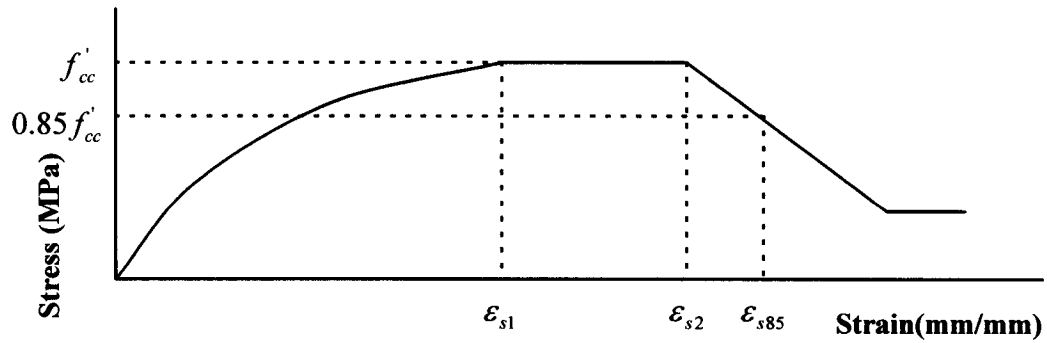
| Label                       | Drift (%) | Experimental Moment (kN.m) | Tensile Strain (%) | Compressive Strain (%) |
|-----------------------------|-----------|----------------------------|--------------------|------------------------|
| CFCL2-1 <sup>st</sup> Cycle | 0.5       | 166                        | 0.32               | 0.155                  |
| CFCL2-2 <sup>nd</sup> Cycle | 0.5       | 166                        | 0.32               | 0.177                  |
| CFCL2-3 <sup>rd</sup> Cycle | 0.5       | 166                        | 0.32               | 0.18                   |
| CFCL2-1 <sup>st</sup> Cycle | 1         | 200                        | 0.51               | 0.27                   |
| CFCL2-2 <sup>nd</sup> Cycle | 1         | 200                        | 0.53               | 0.33                   |
| CFCL2-3 <sup>rd</sup> Cycle | 1         | 200                        | 0.54               | 0.35                   |
| CFCL2-1 <sup>st</sup> Cycle | 1.5       | 210                        | 0.62               | 0.38                   |
| CFCL2-2 <sup>nd</sup> Cycle | 1.5       | 210                        | 0.76               | 0.415                  |
| CFCL2-3 <sup>rd</sup> Cycle | 1.5       | 210                        | 0.765              | 0.42                   |
| CFCL2-1 <sup>st</sup> Cycle | 2         | 220                        | 0.86               | 0.45                   |
| CFCL2-2 <sup>nd</sup> Cycle | 2         | 220                        | 0.92               | 0.47                   |
| CFCL3-1 <sup>st</sup> Cycle | 0.5       | 130                        | 0.089              | 0.062                  |
| CFCL3-2 <sup>nd</sup> Cycle | 0.5       | 130                        | 0.096              | 0.084                  |
| CFCL3-3 <sup>rd</sup> Cycle | 0.5       | 130                        | 0.13               | 0.15                   |
| CFCL3-1 <sup>st</sup> Cycle | 1         | 200                        | 0.33               | 0.16                   |
| CFCL3-2 <sup>nd</sup> Cycle | 1         | 200                        | 0.35               | 0.21                   |
| CFCL3-1 <sup>st</sup> Cycle | 2         | 240                        | 0.53               | 0.28                   |
| CFCL3-2 <sup>nd</sup> Cycle | 2         | 240                        | 0.57               | 0.34                   |
| CFCL3-3 <sup>rd</sup> Cycle | 2         | 240                        | 0.6                | 0.37                   |
| CFCL4-1 <sup>st</sup> Cycle | 1         | 190                        | 0.29               | 0.17                   |
| CFCL4-2 <sup>nd</sup> Cycle | 1         | 190                        | 0.29               | 0.205                  |
| CFCL4-3 <sup>rd</sup> Cycle | 1         | 190                        | 0.29               | 0.22                   |
| CFCL4-1 <sup>st</sup> Cycle | 2         | 220                        | 0.49               | 0.35                   |
| CFCL4-2 <sup>nd</sup> Cycle | 2         | 220                        | 0.48               | 0.35                   |
| CFCL4-3 <sup>rd</sup> Cycle | 2         | 220                        | 0.56               | 0.37                   |



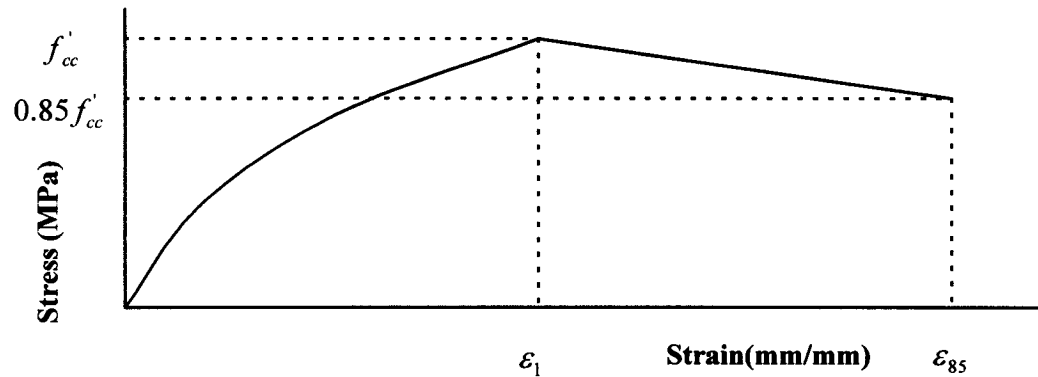
(a) Unconfined Concrete Model



(b) Confined Concrete Model-Kent and Park



(c) Confined Concrete Model-Sheikh and Uzumeri



(d) Confined Concrete Model-Saatcioglu and Razvi

Figure 6.1: The Stress-Strain Relationships for Unconfined and Confined Concretes

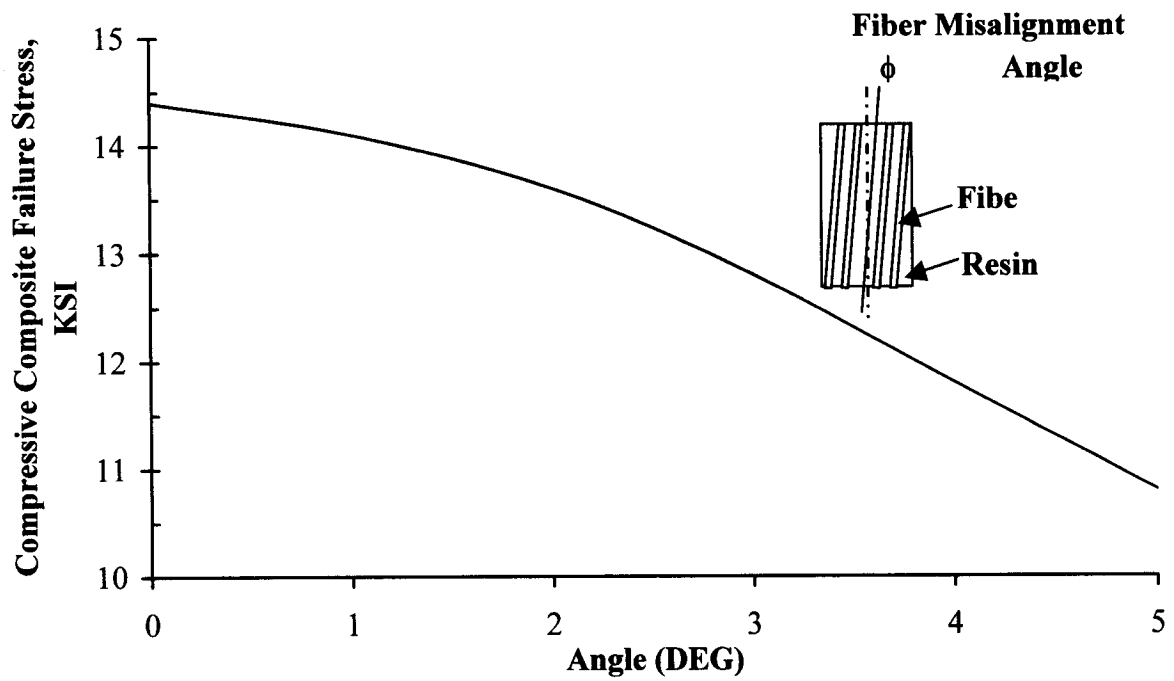


Figure 6.2: Influence of Fiber Misalignment on Compressive Strength of Composites

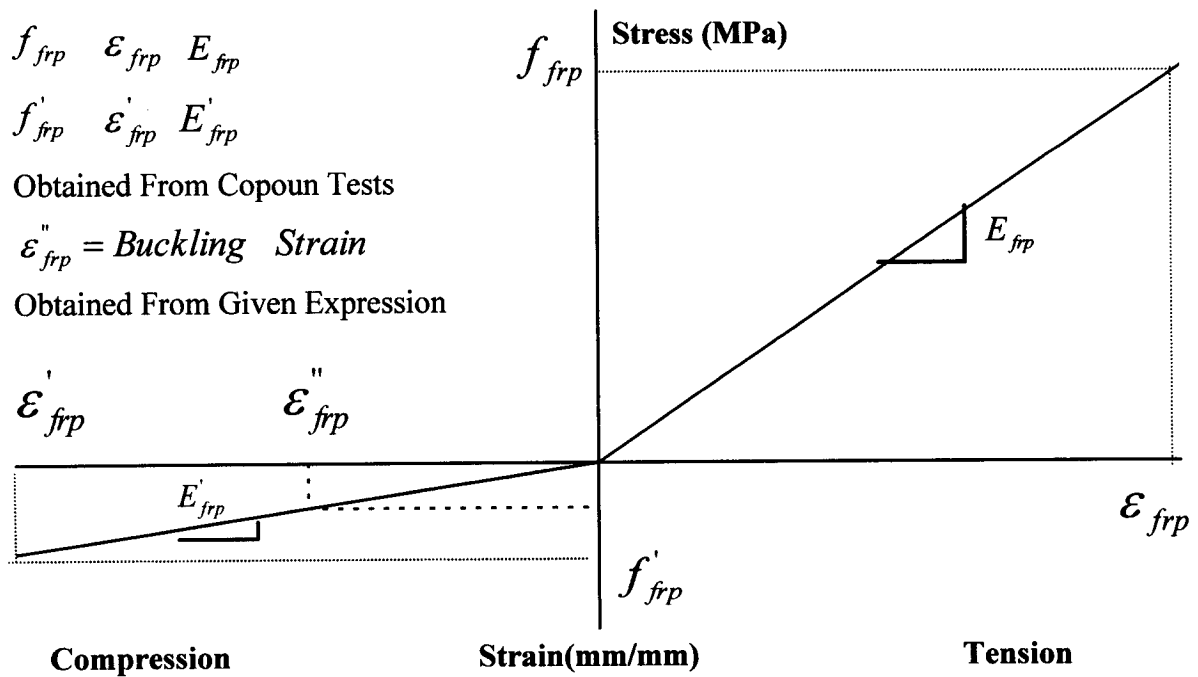
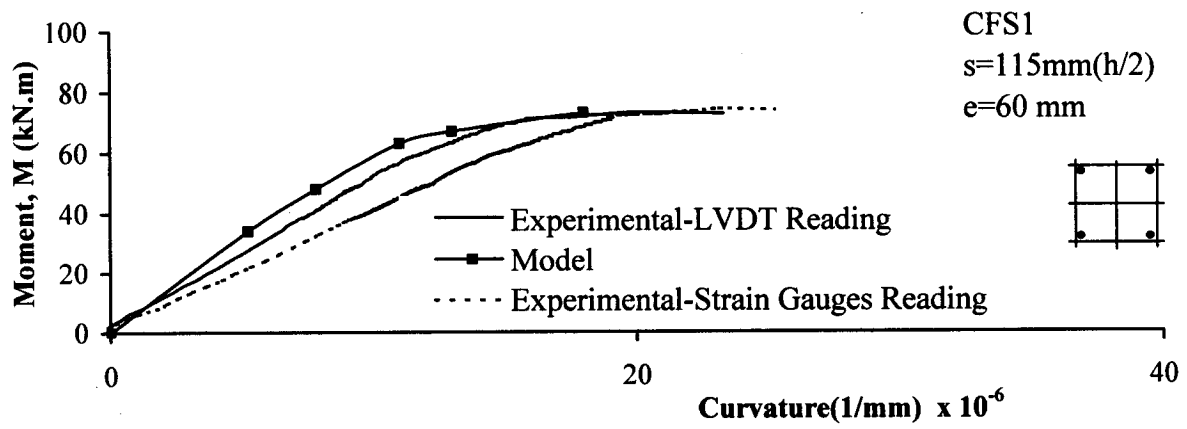
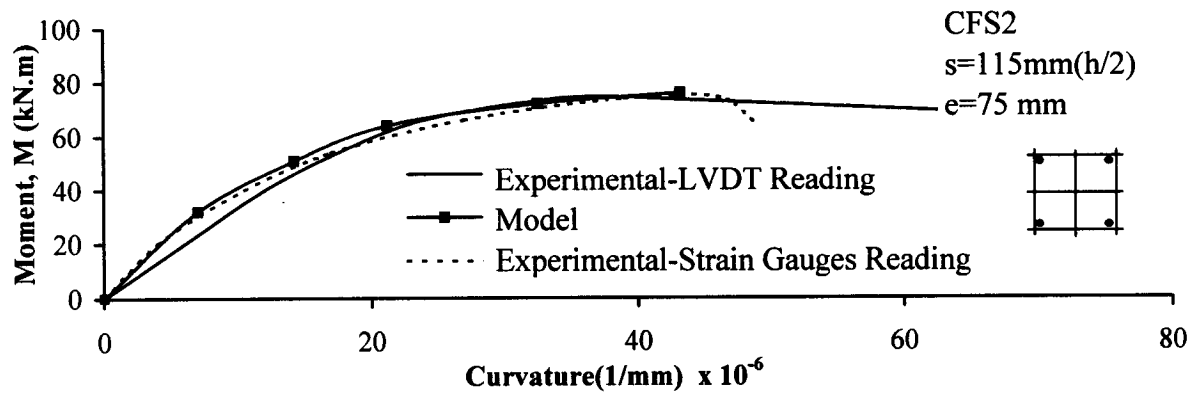


Figure 6.3 The Stress-Strain Relationships for FRP Longitudinal Bars



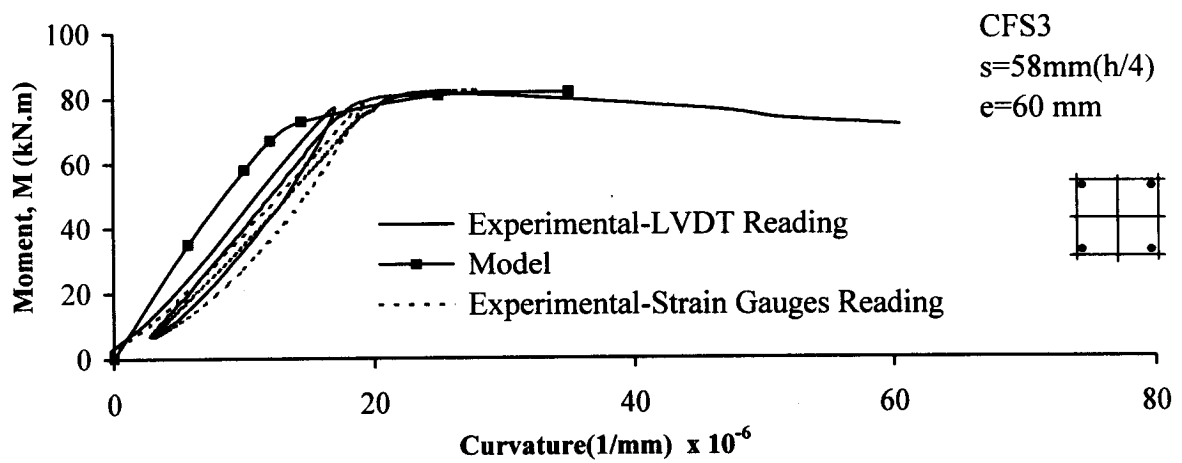


(a) Column CFS1

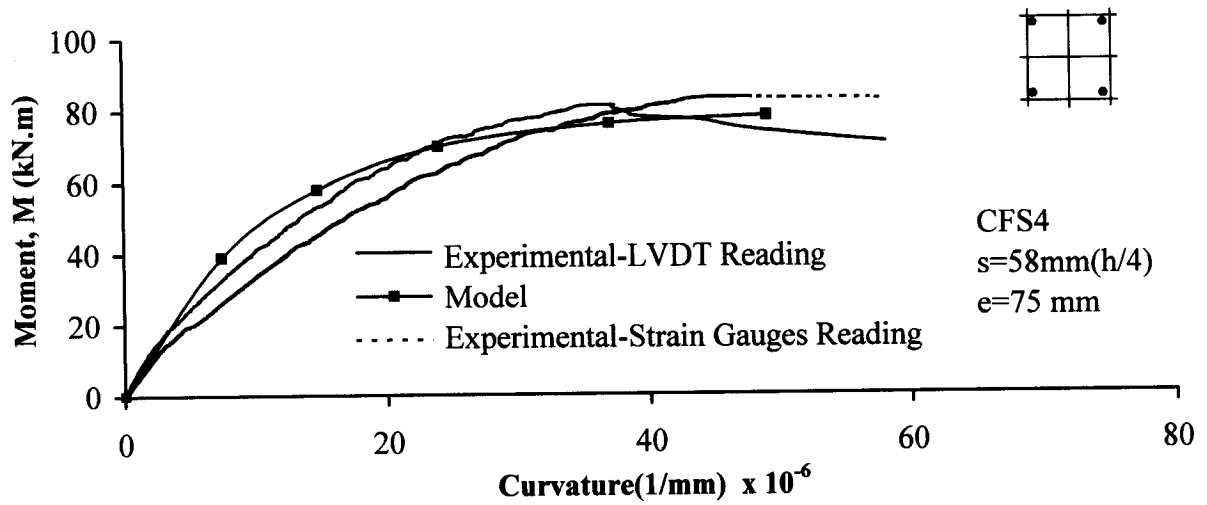


(b) Column CFS2

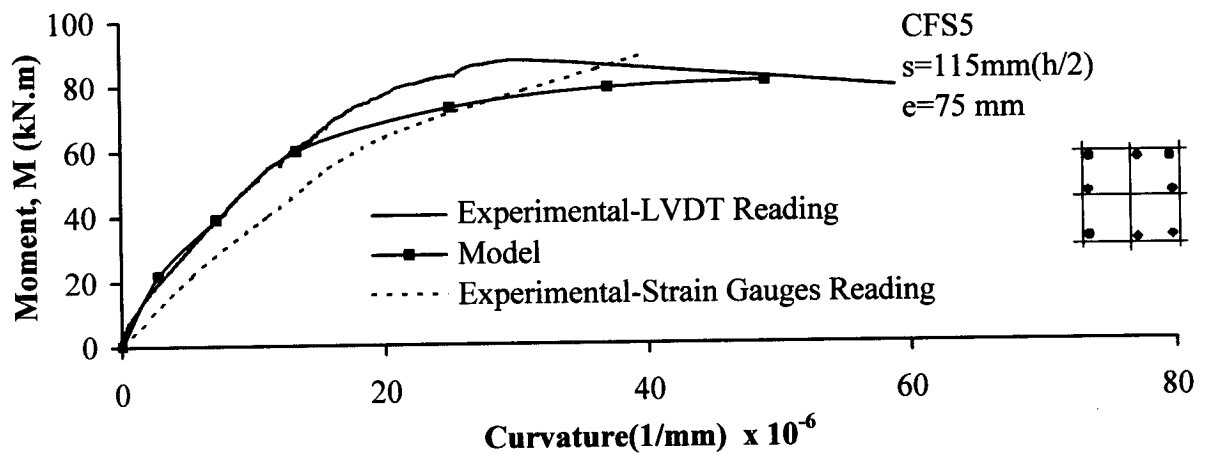
Figure 6.4: Moment-Curvature Diagrams for Phase I Columns (under Eccentric Loading )



(c) Column CFS3

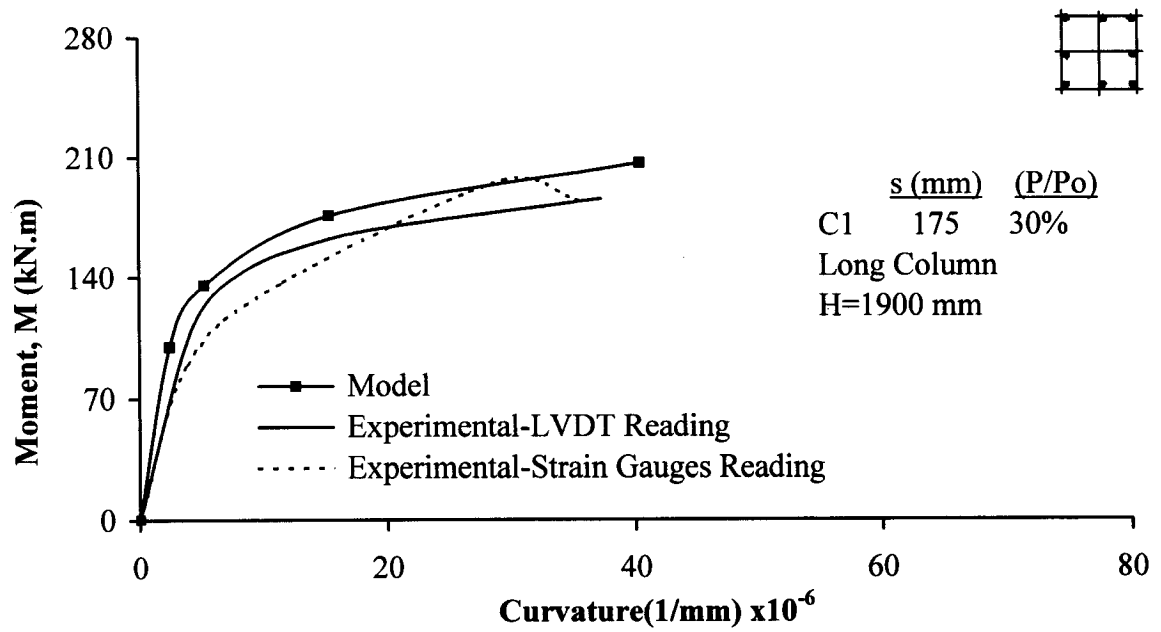


(d) Column CFS4

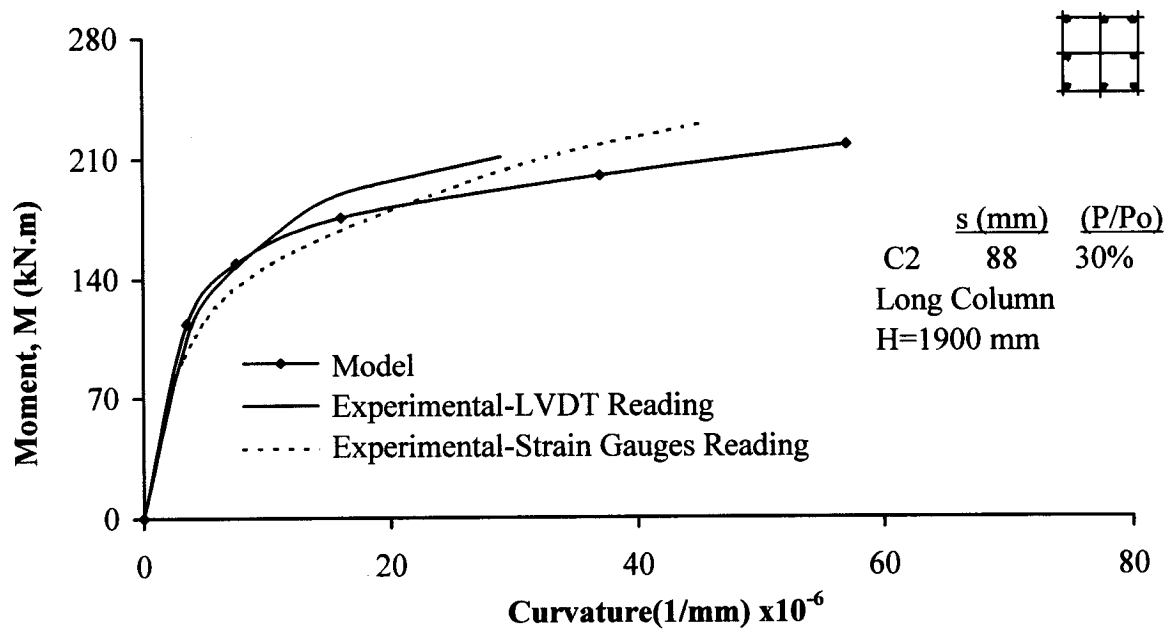


(e) Column CFS5

Figure 6.4 (Cont): Moment-Curvature Diagrams for Phase I Columns

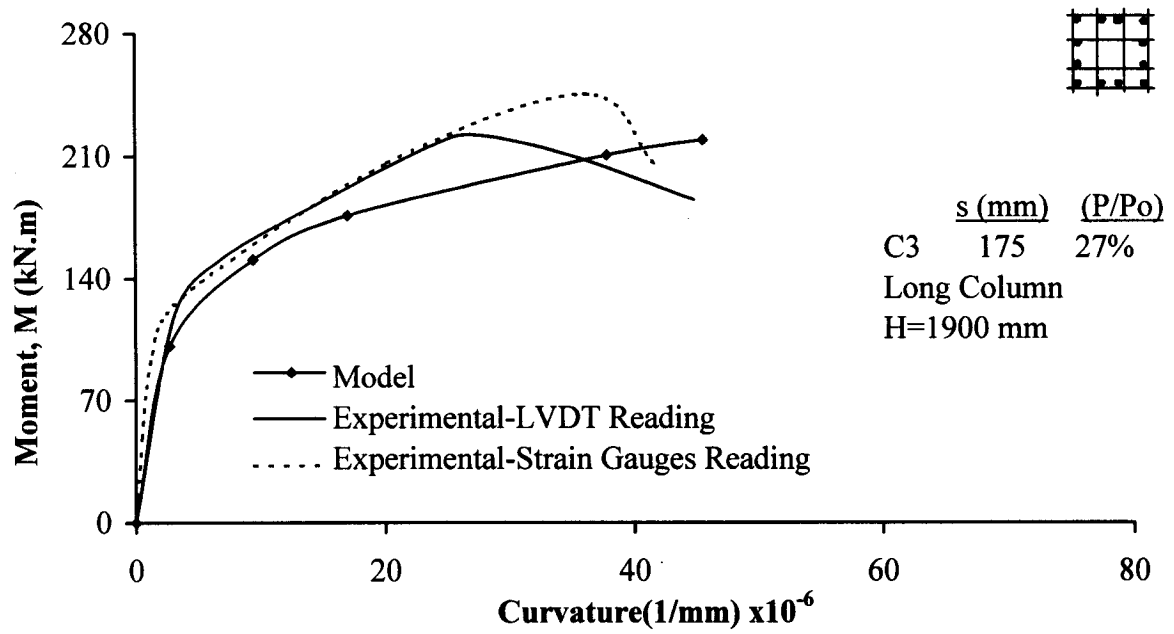


(a) Column CFCL1

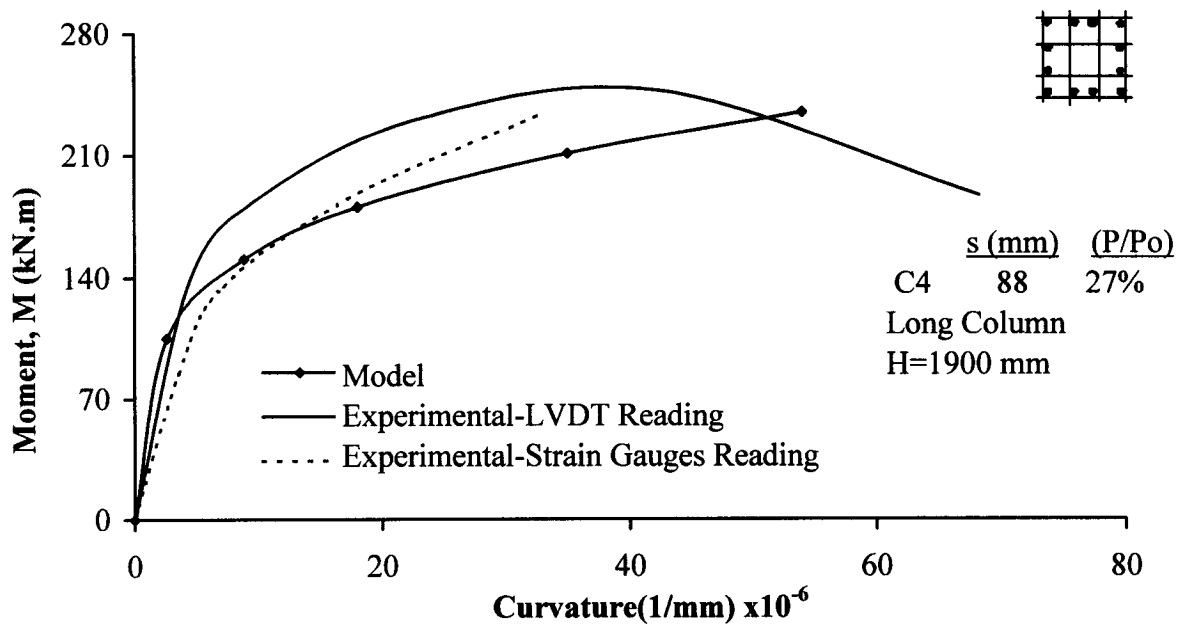


(b) Column CFCL2

Figure 6.5: Moment-Curvature Diagrams for Phase III Columns ( under Cyclic Loading)

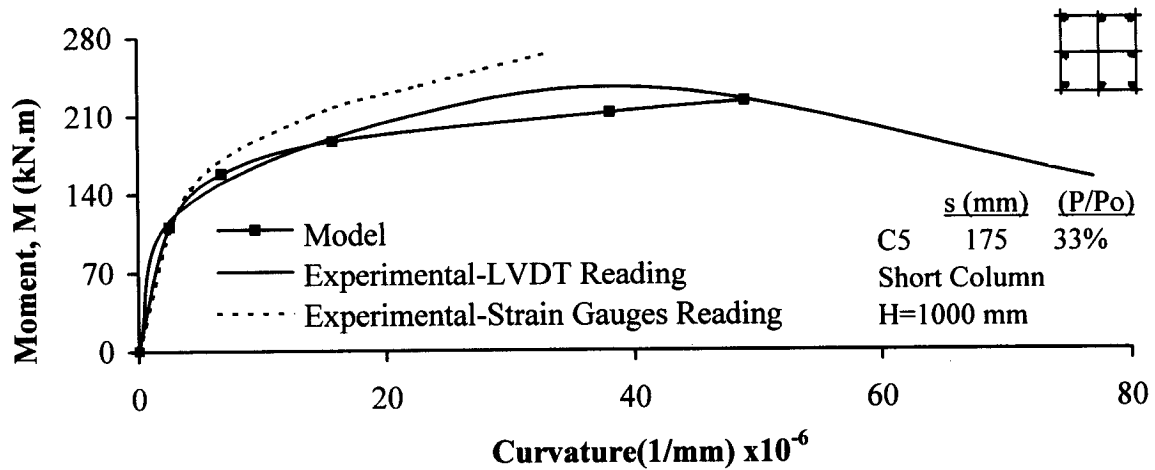


(c) Column CFCL3

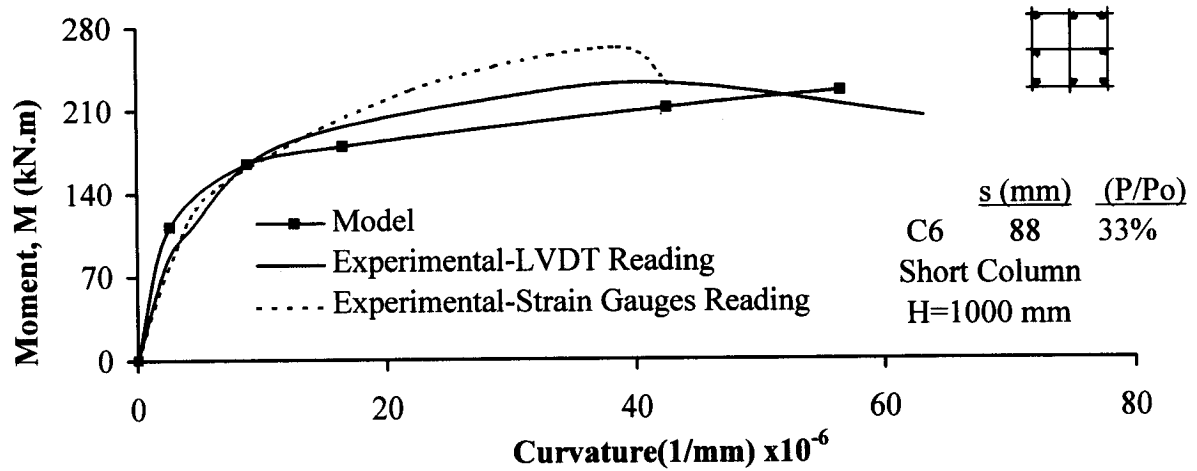


(d) Column CFCL4

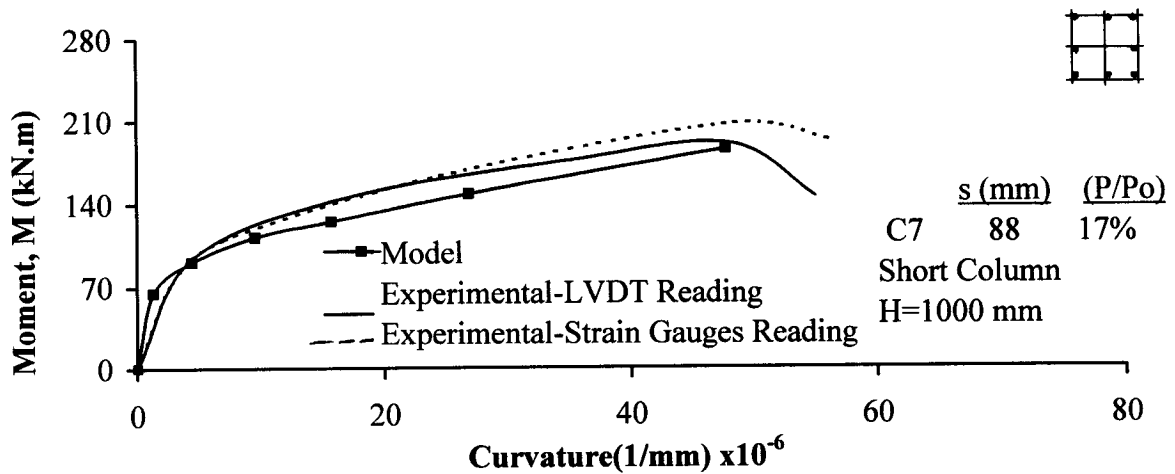
Figure 6.5(Cont): Moment-Curvature Diagrams for Phase III Columns



(e) Column CFCL5

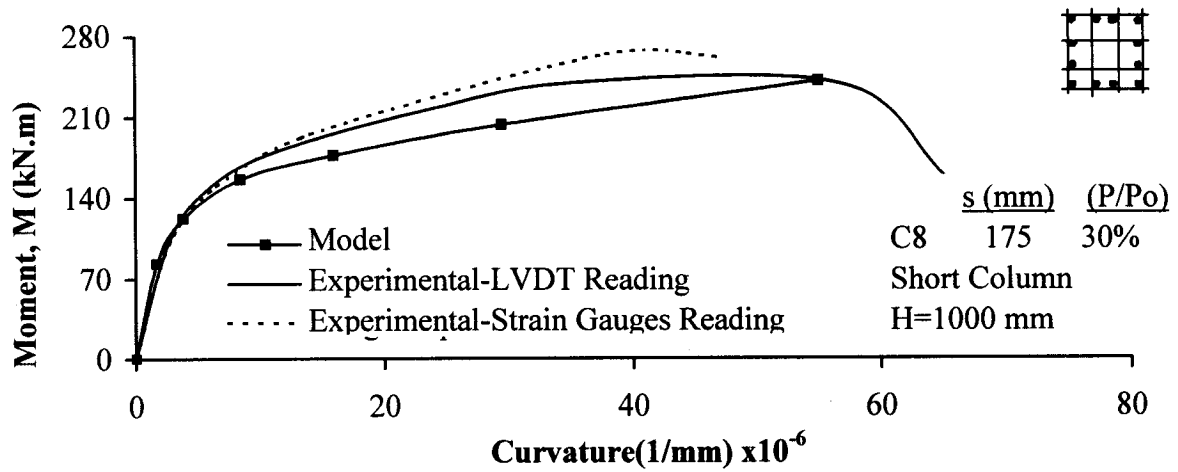


(f) Column CFCL6

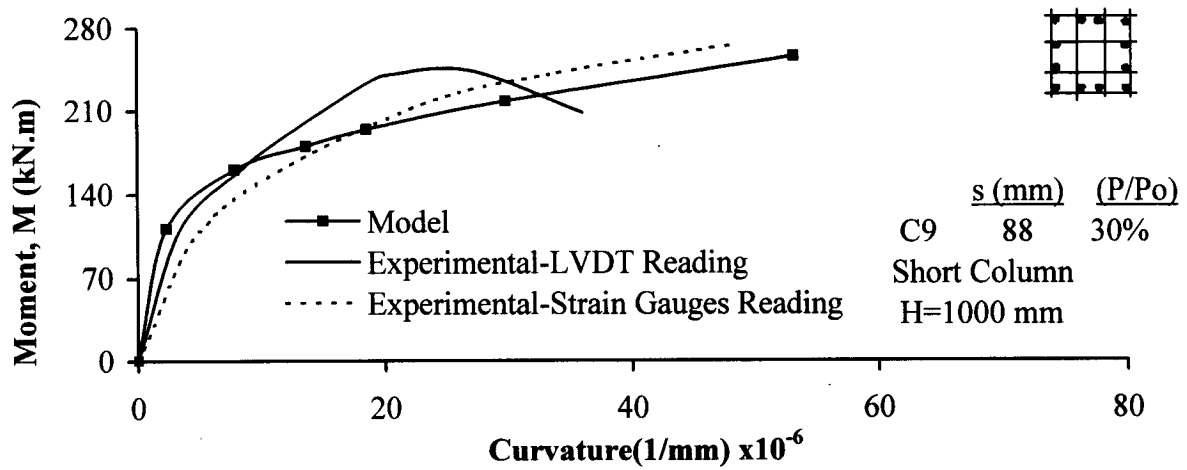


(g) Column CFCL7

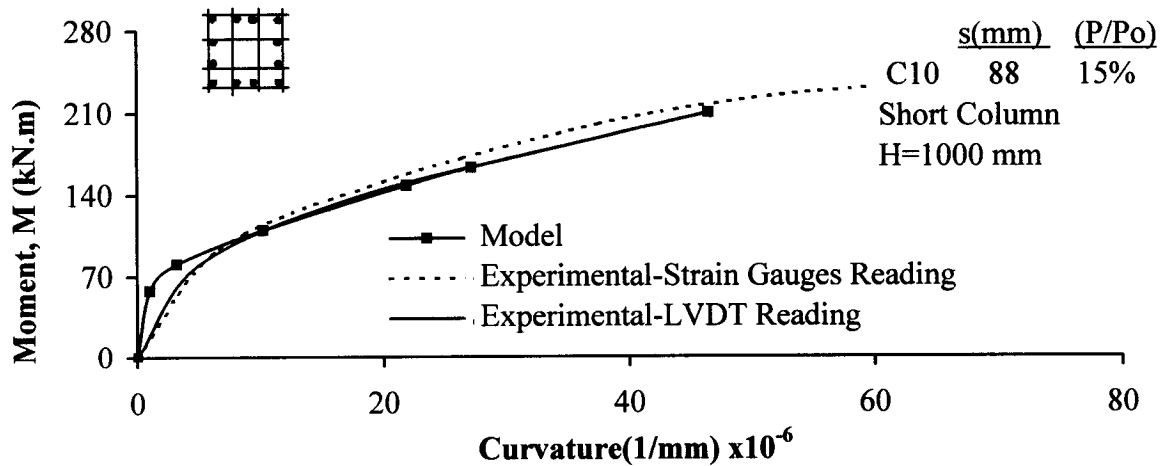
Figure 6.5(Cont): Moment-Curvature Diagrams for Phase III Columns



(h) Column CFCL8

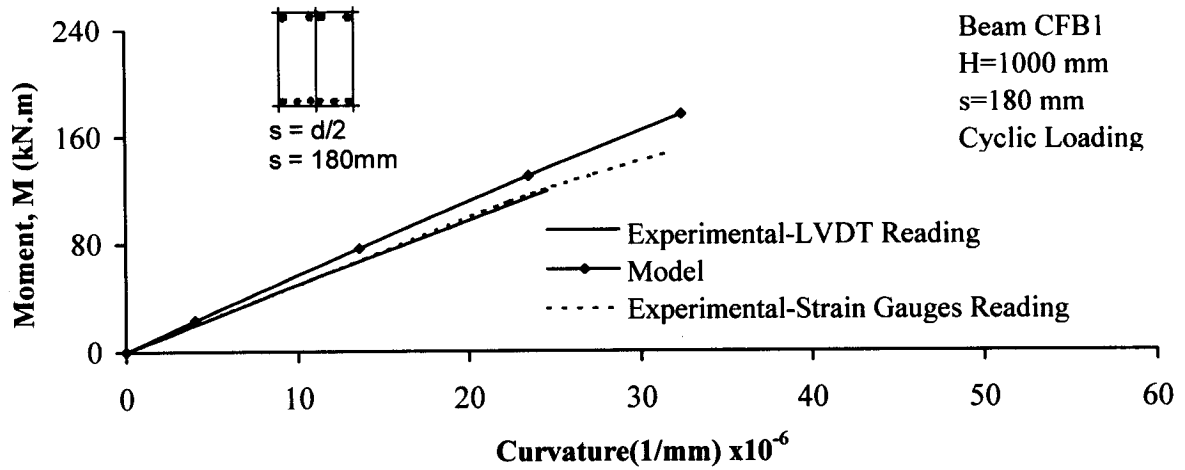


(i) Column CFCL9

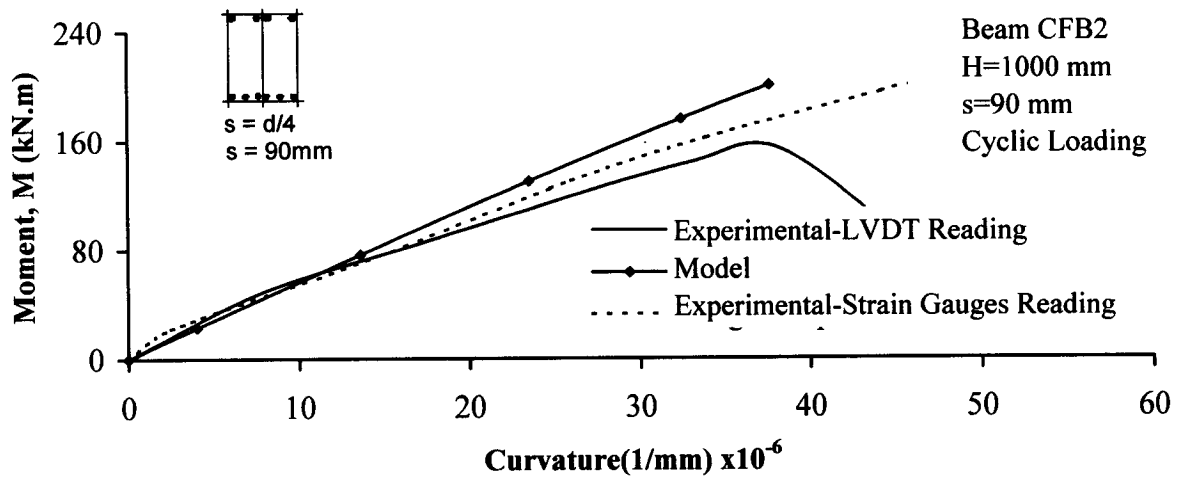


(j) Column CFCL10

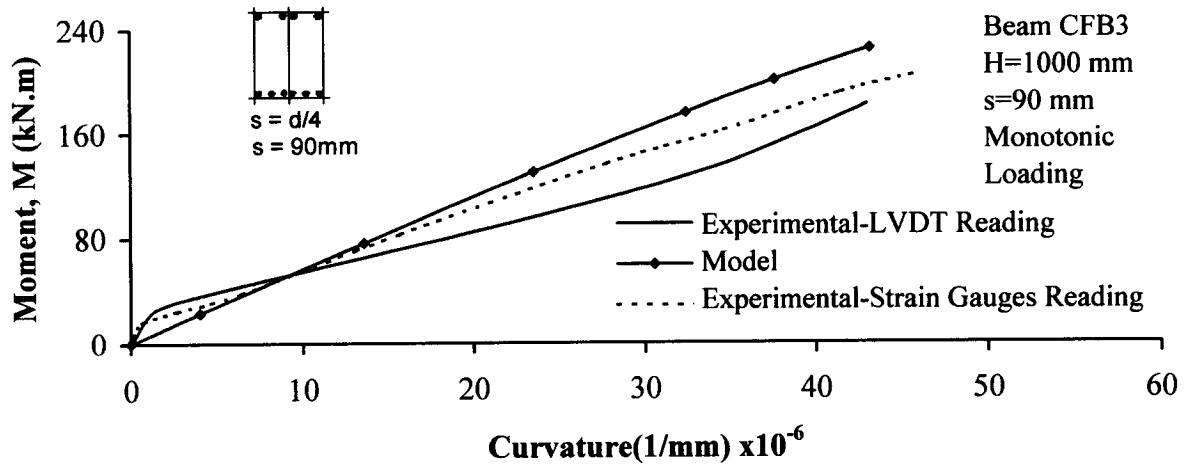
Figure 6.5(Cont): Moment-Curvature Diagrams for Phase III Columns



(a) Beam CFB1

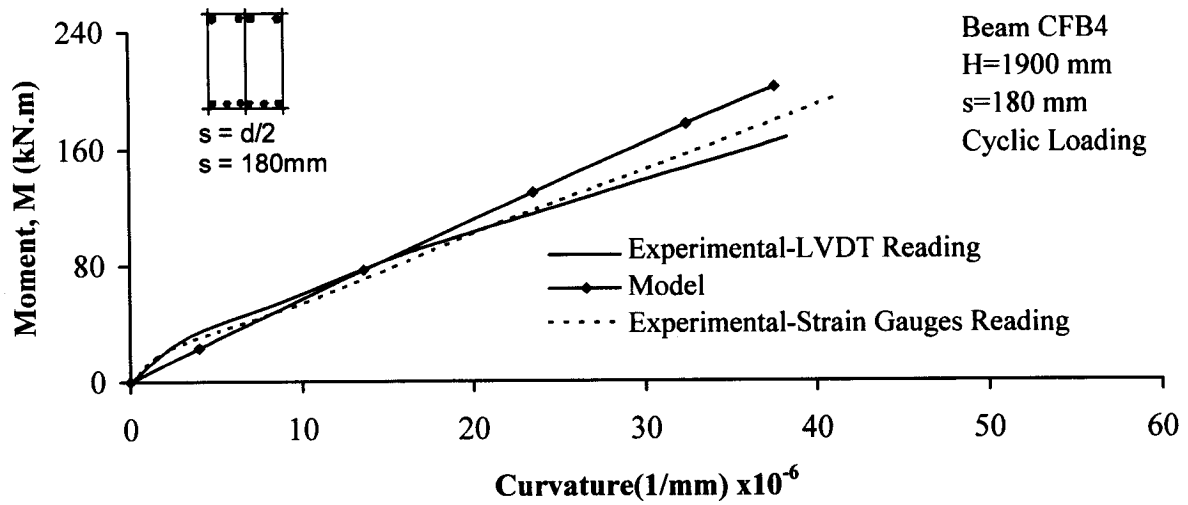


(b) Beam CFB2

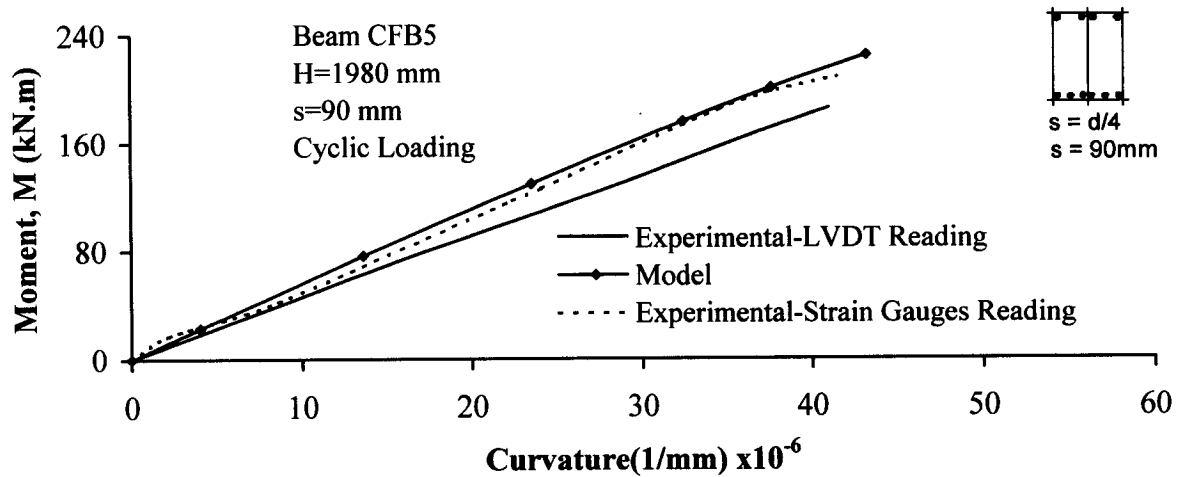


(c) Beam CFB3

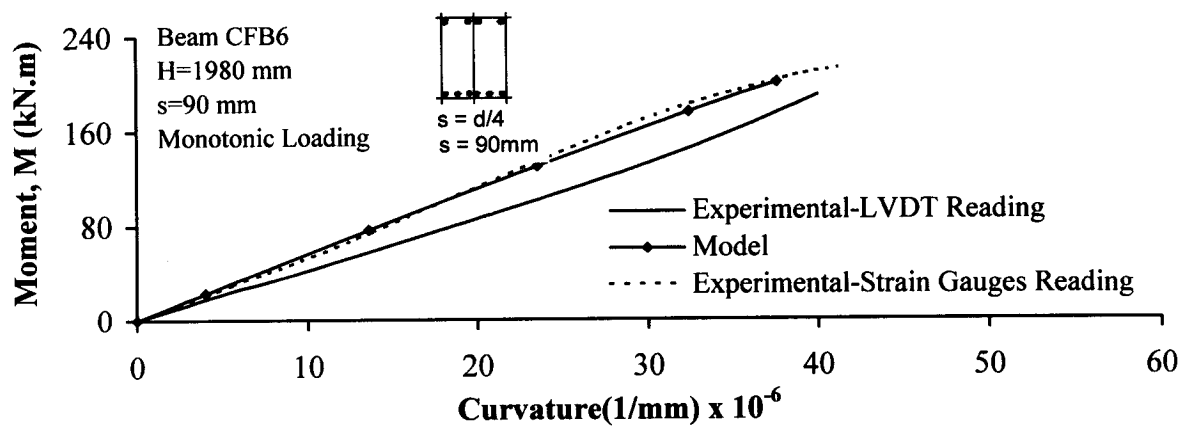
Figure 6.6: Moment-Curvature Diagrams for Phase IV Beams ( under Cyclic and Monotonic)



(a) Beam CFB4



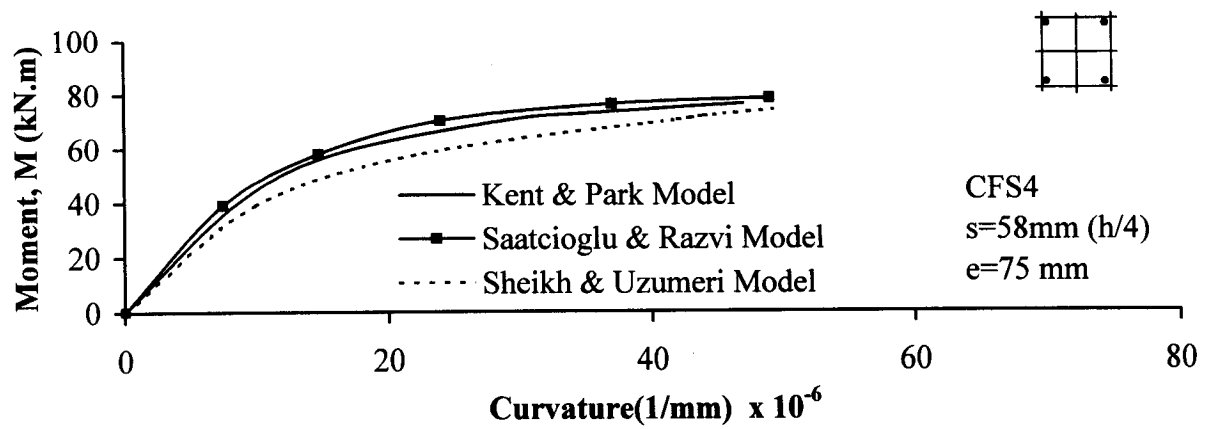
(b) Beam CFB5



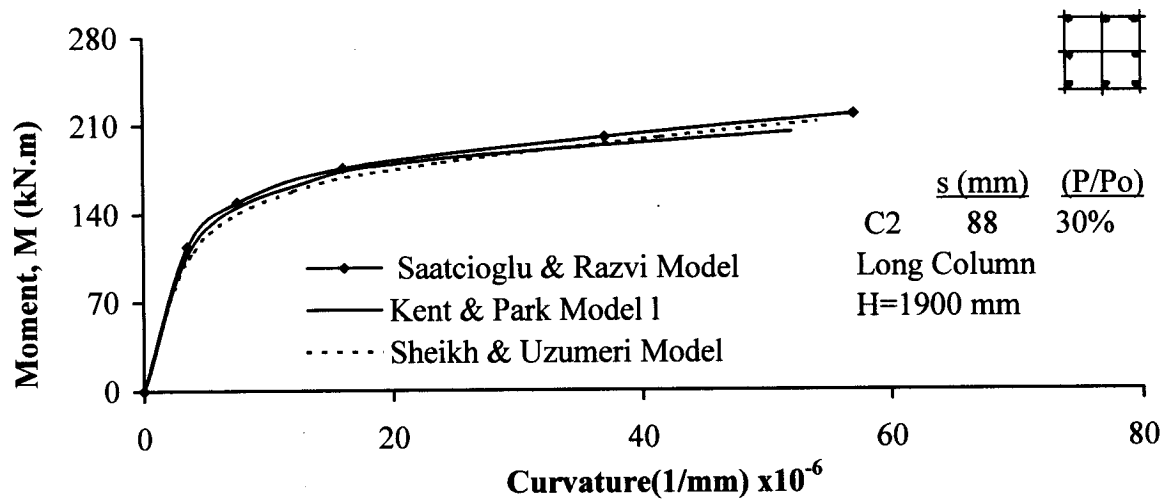
(c) Beam CFB6

Figure 6.6 (Cont): Moment-Curvature Diagrams for Phase IV Beams ( under Cyclic and Monotonic)

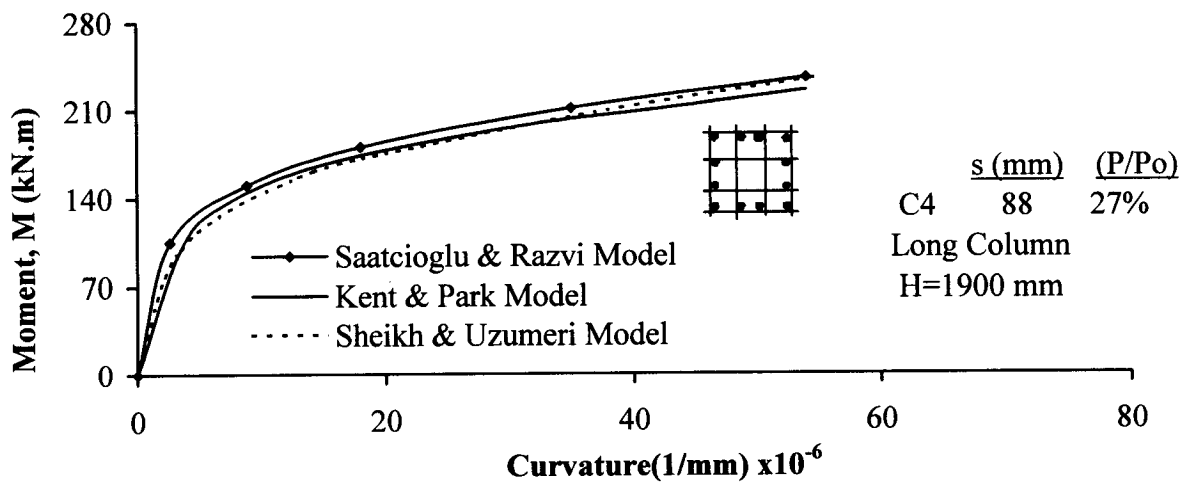




(a) Column CFS4

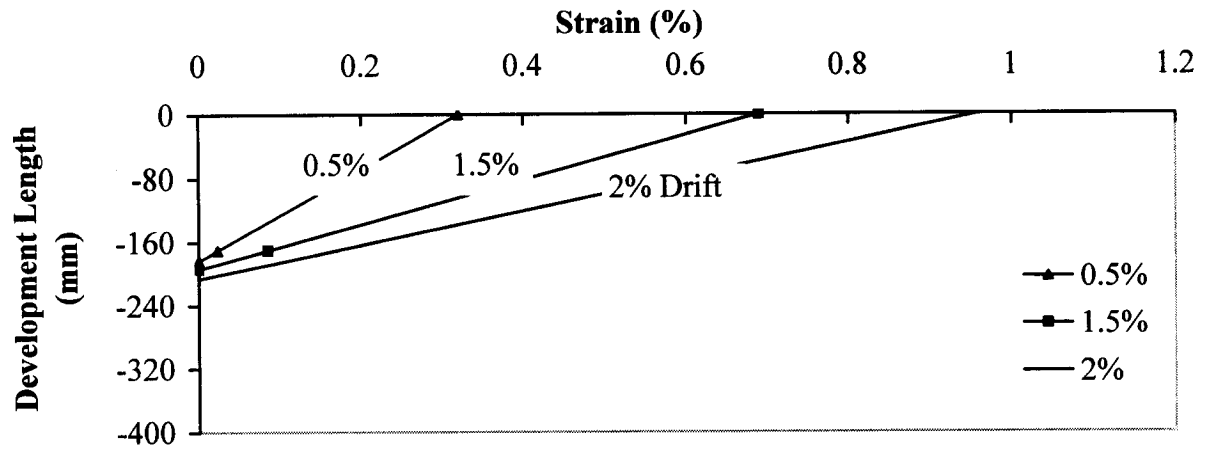


(b) Column CFCL2

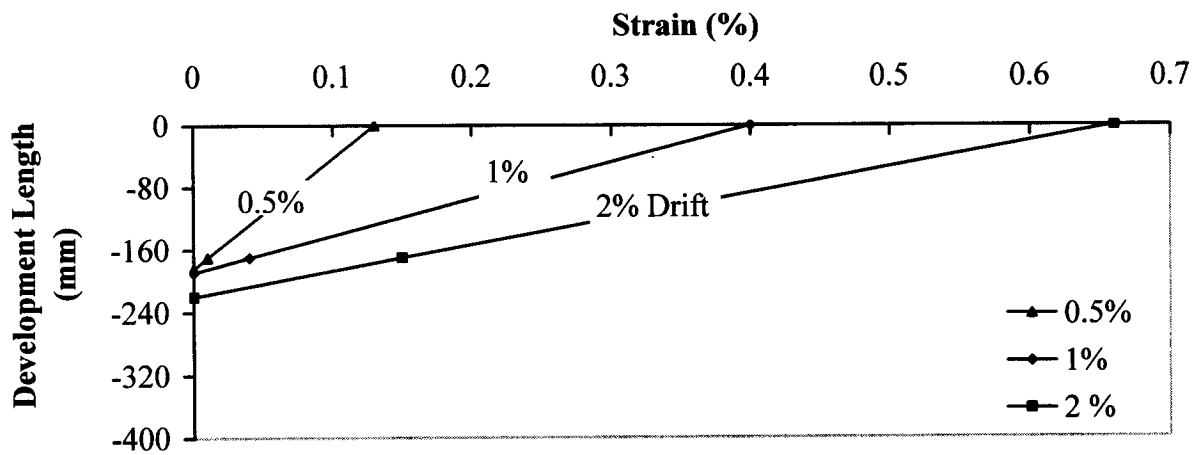


(c) Column CFCL4

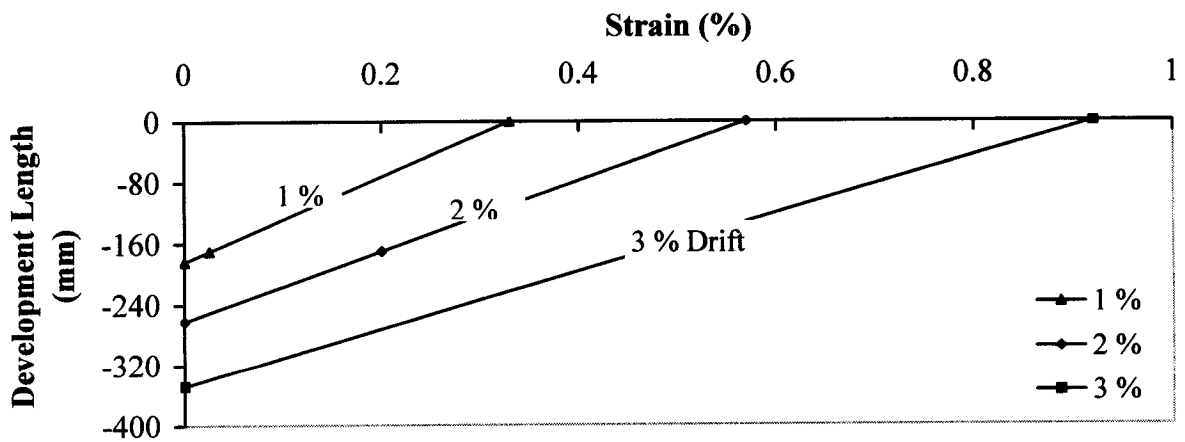
Figure 6.7: Comparison of Different Confinement Models for Moment-Curvature Diagrams



(a) Column CFCL2

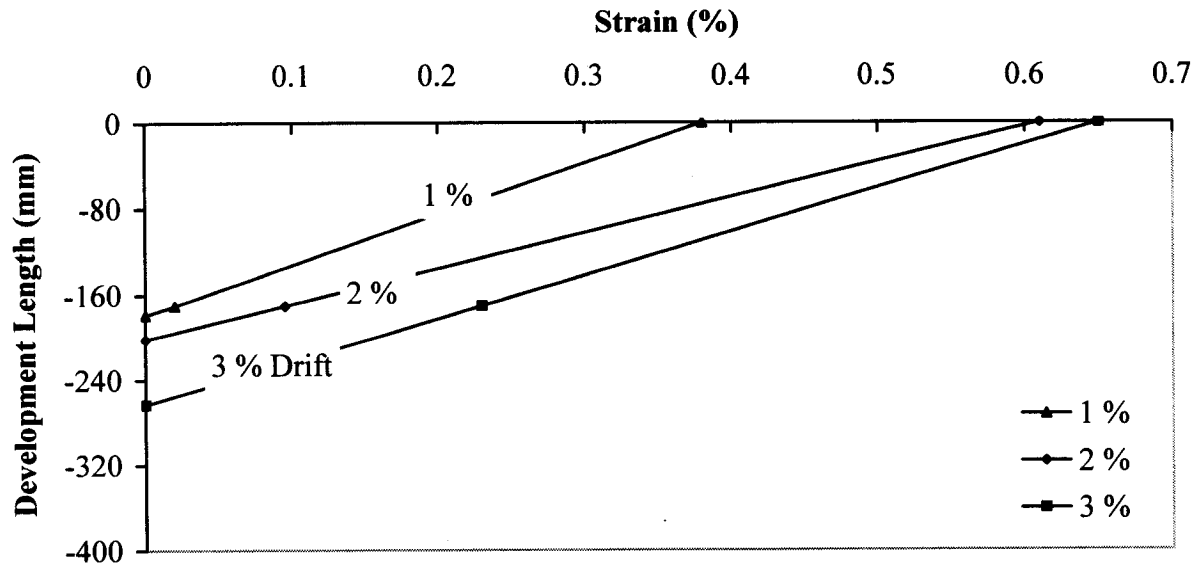


(b) Column CFCL3

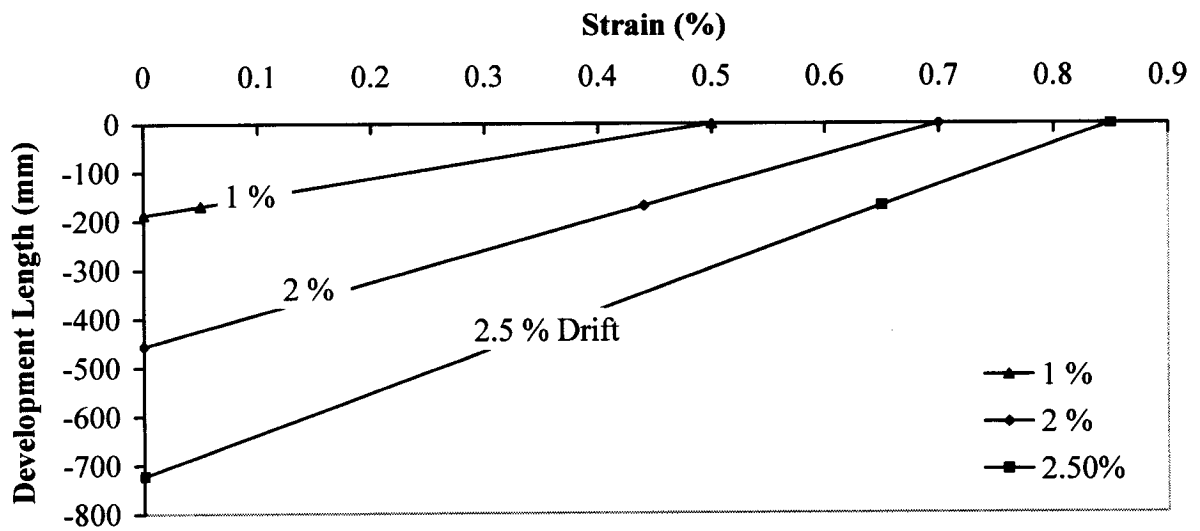


(c) Column CFCL4

Figure 6.8: Strain Readings in Longitudinal FRP Bars Inside the Footing at Different Stages of Loading

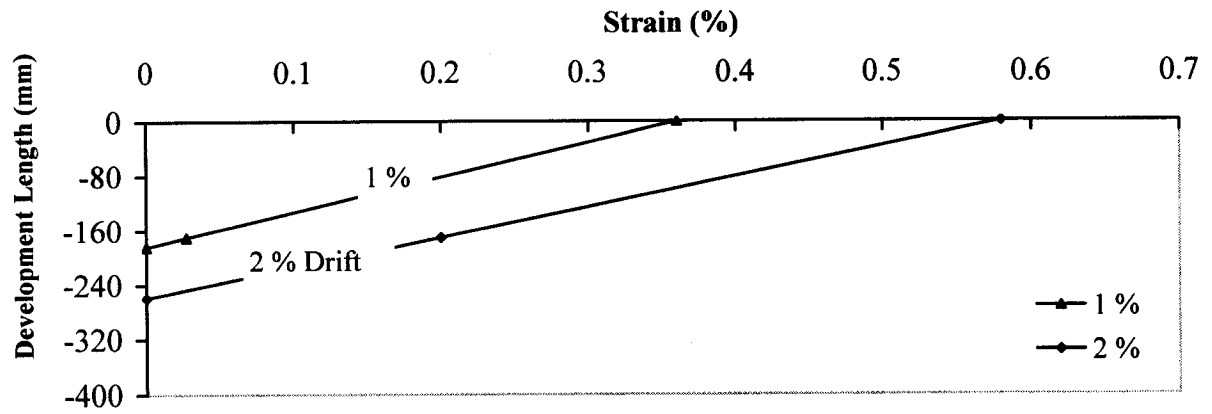


(d) Column CFCL6

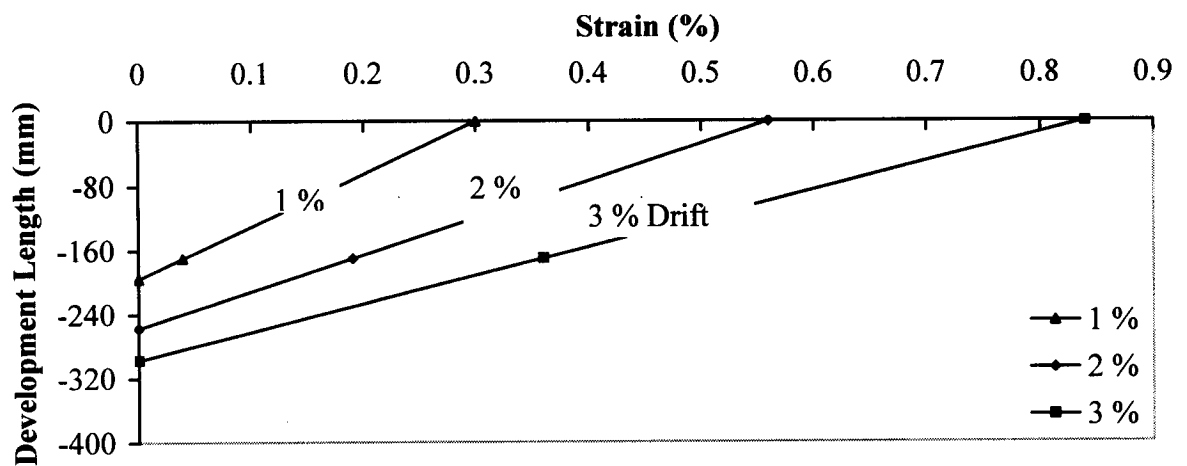


(e) Column CFCL7

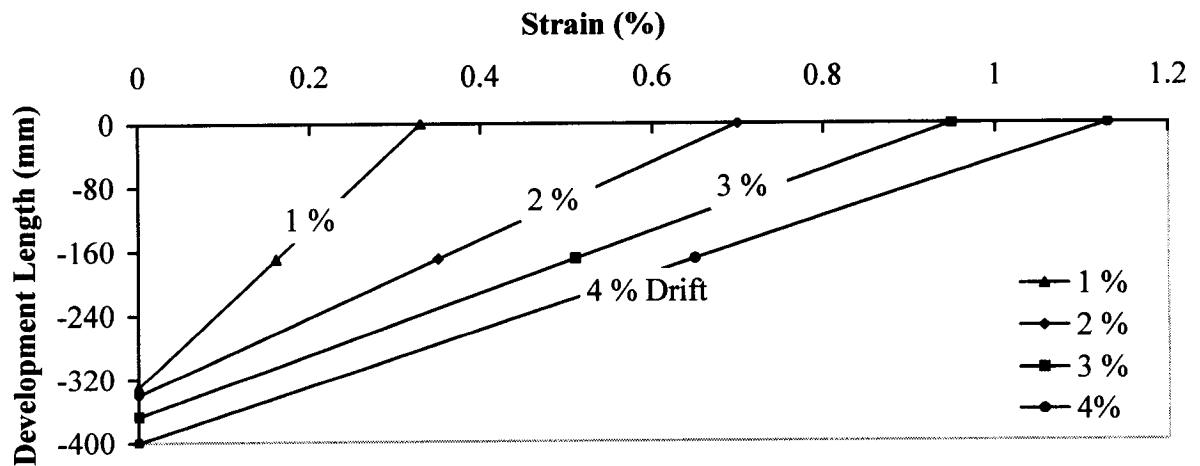
Figure 6.8(Cont.): Strain Readings in Longitudinal FRP Bars Inside the Footing at Different Stages of Loading



(f) Column CFCL8

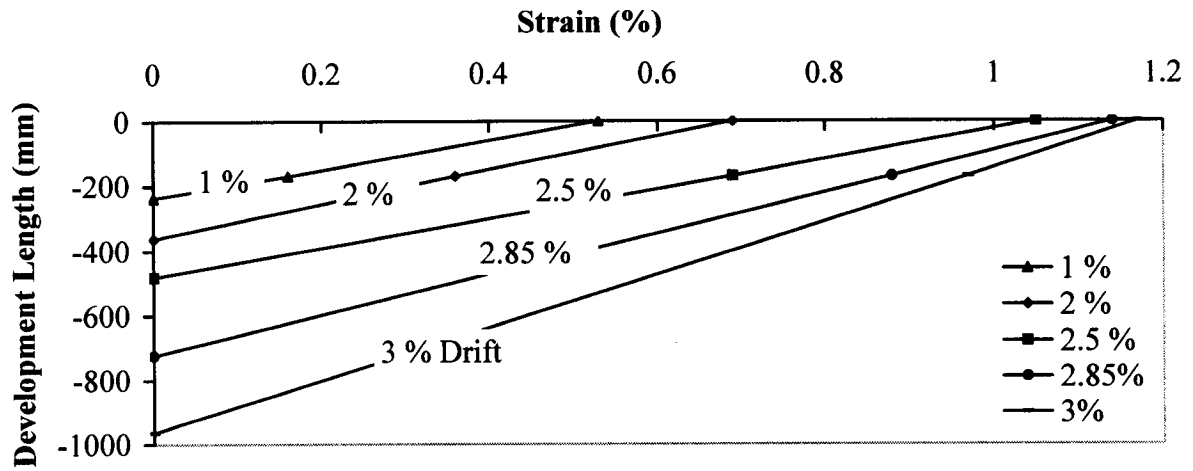


(g) Column CFCL9

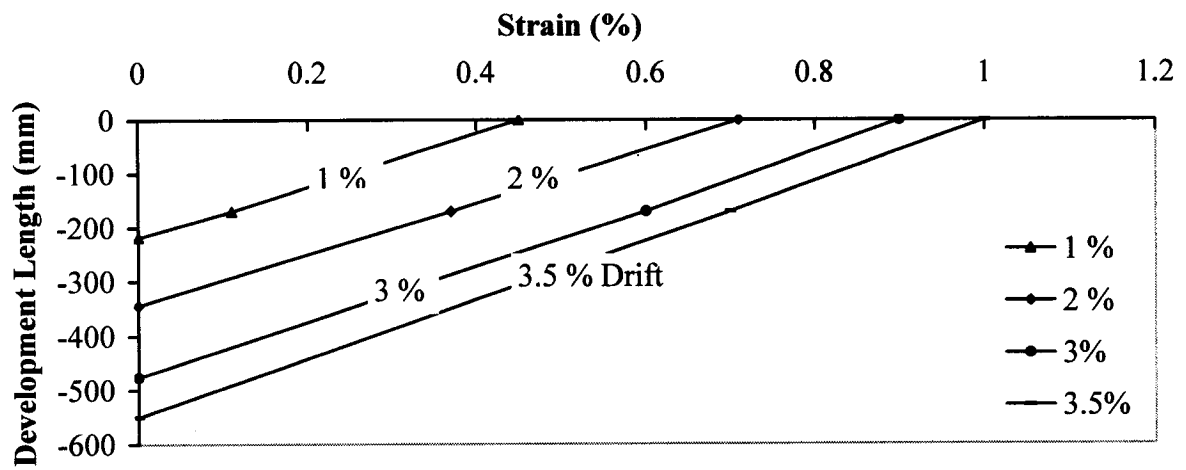


(h) Column CFCL10

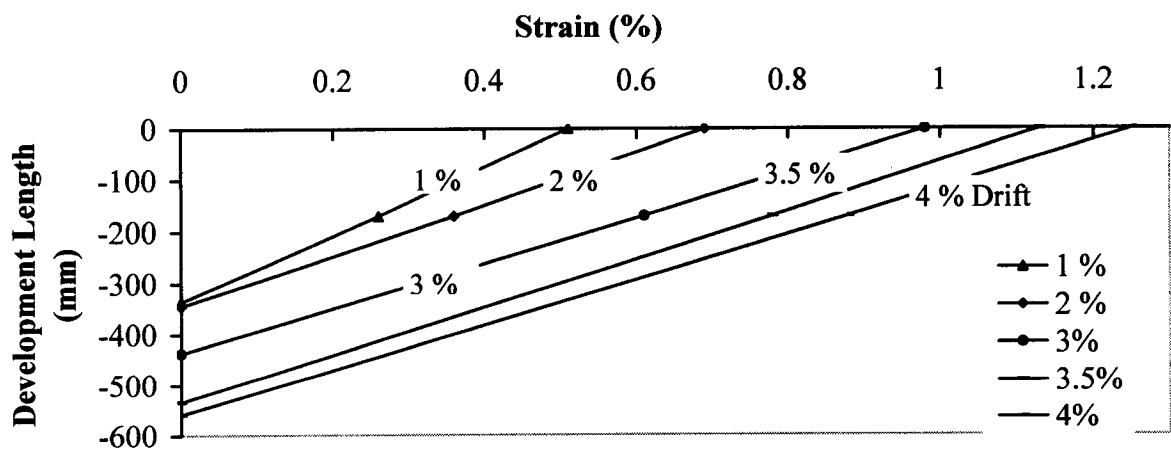
Figure 6.8(Cont.): Strain Readings in Longitudinal FRP Bars Inside the Footing at Different Stages of Loading



(i) Column CFB3

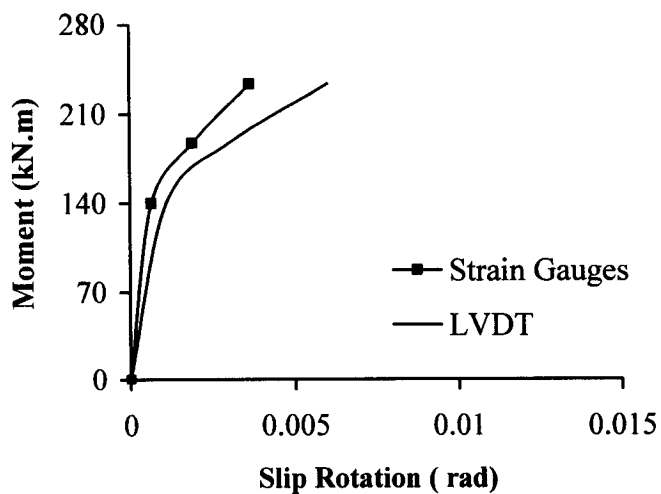


(j) Column CFB4

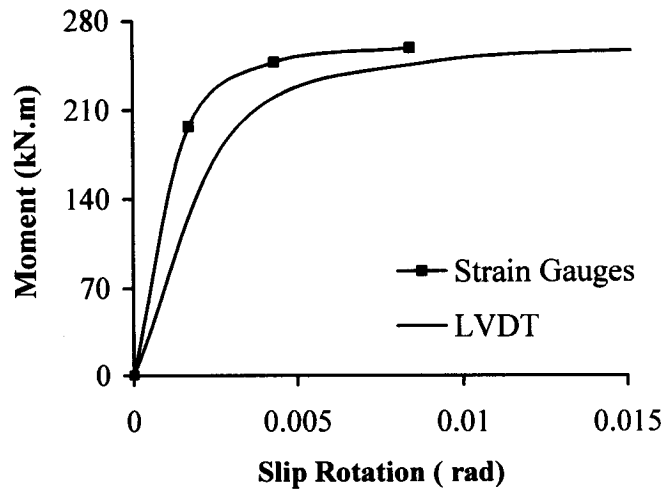


(k) Column CFB6

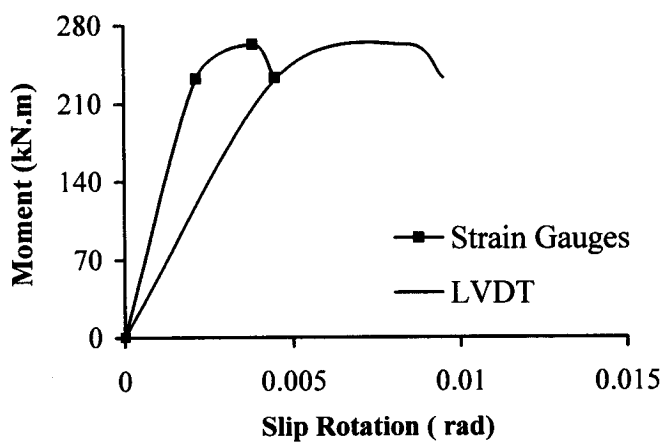
Figure 6.8(Cont.): Strain Readings in Longitudinal FRP Bars Inside the Footing at Different Stages of Loading



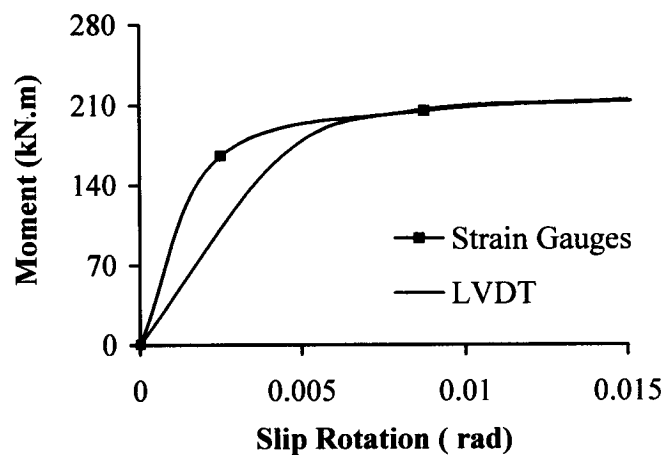
(a) CFCL3



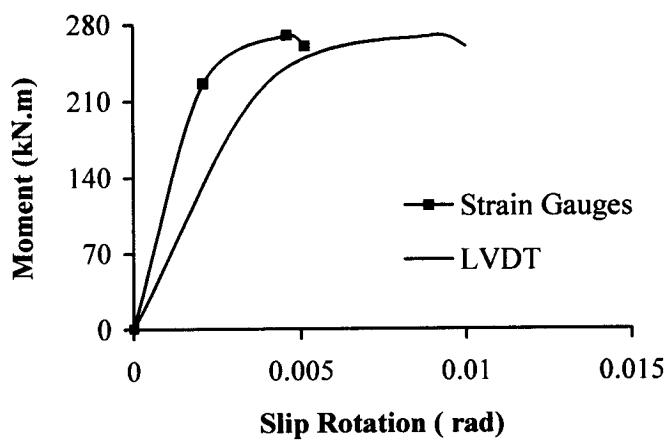
(b) CFCL4



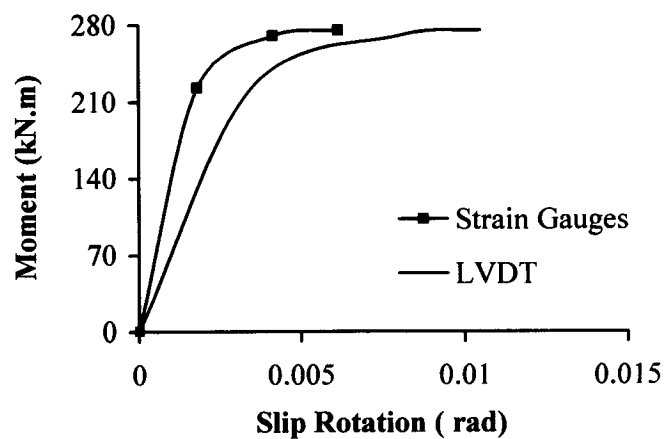
(c) CFCL6



(d) CFCL7

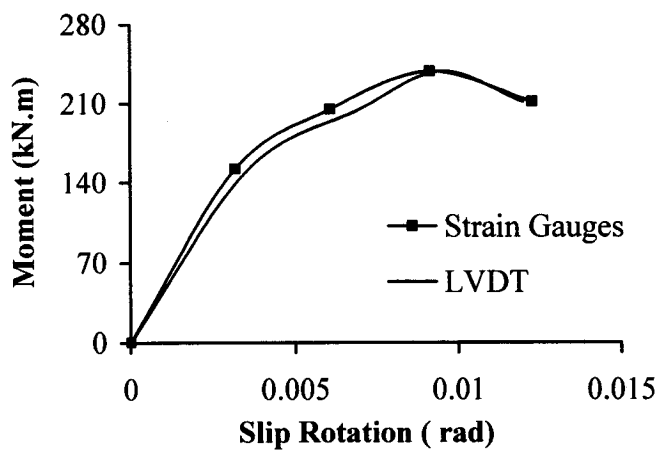


(e) CFCL8

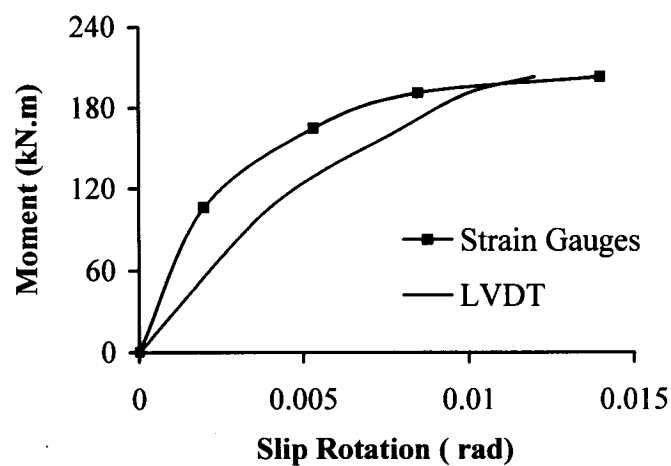


(f) CFCL9

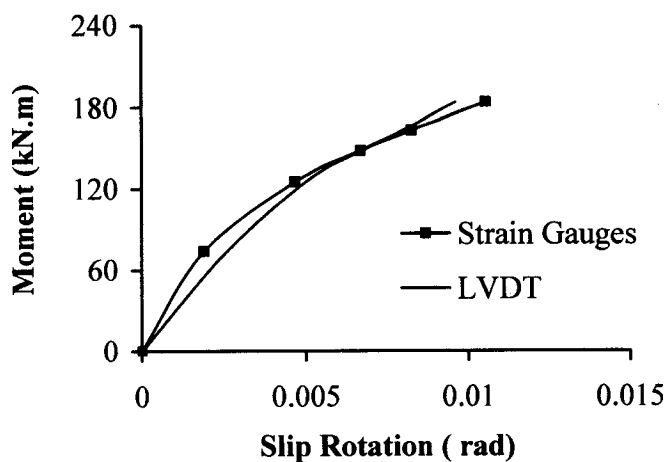
Figure 6.9: Comparison of Moment Anchorage Slip Rotation Relationships Between LVDT and Strain Gauge Readings



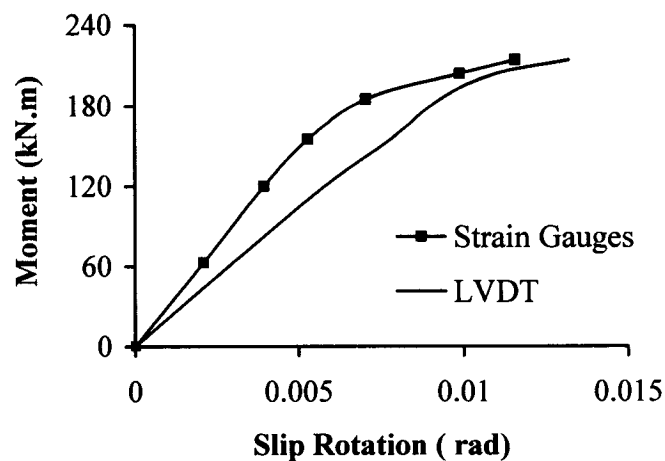
(g) CFCL10



(h) CFB3

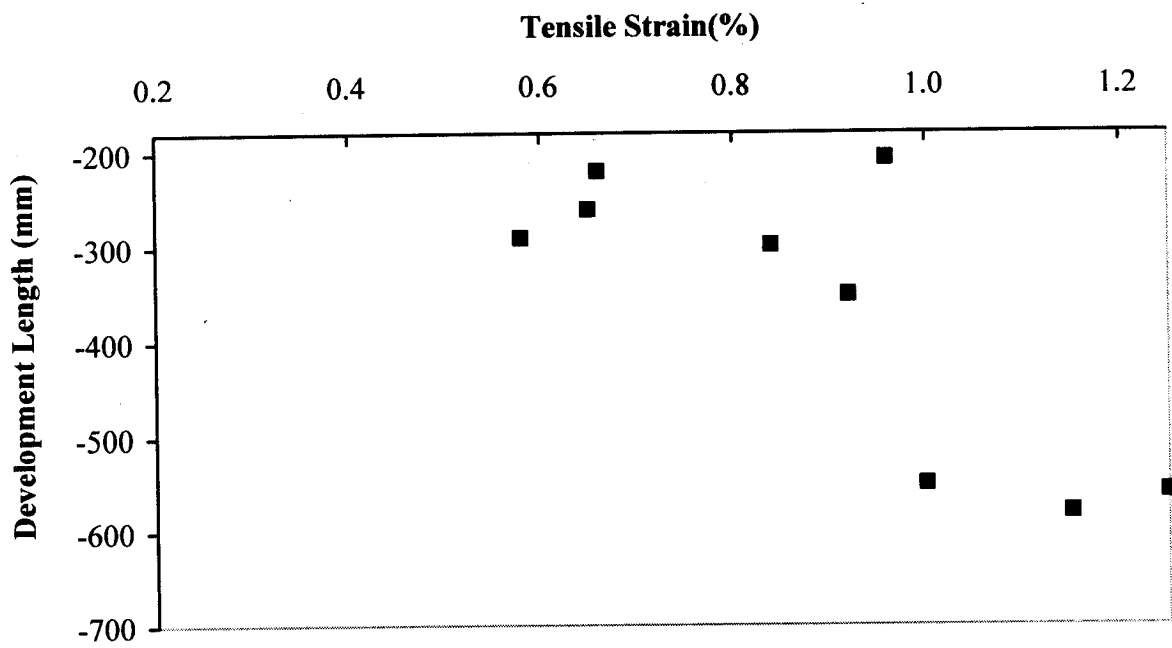


(i) CFB4

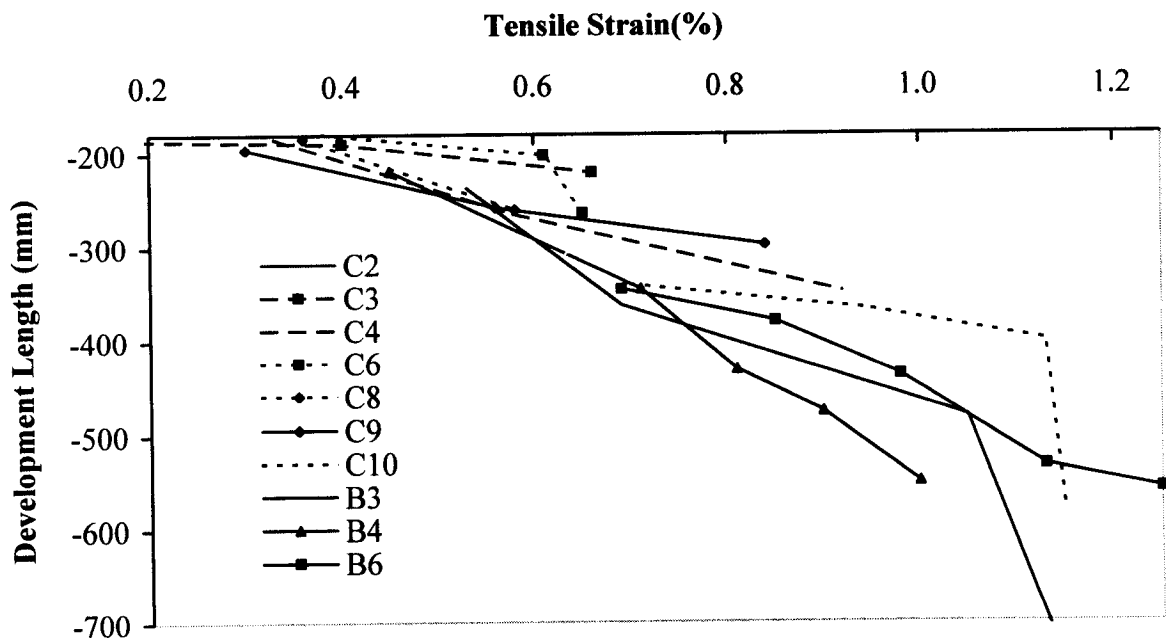


(j) CFB6

Figure 6.9 (Cont.): Comparison of Moment Anchorage Slip Rotation Relationships Between LVDT and Strain Gauge Readings



(a) The Maximum Points



(b) All Points For each Specimen

Figure 6.10: Development Length-Tensile Strain Relationship



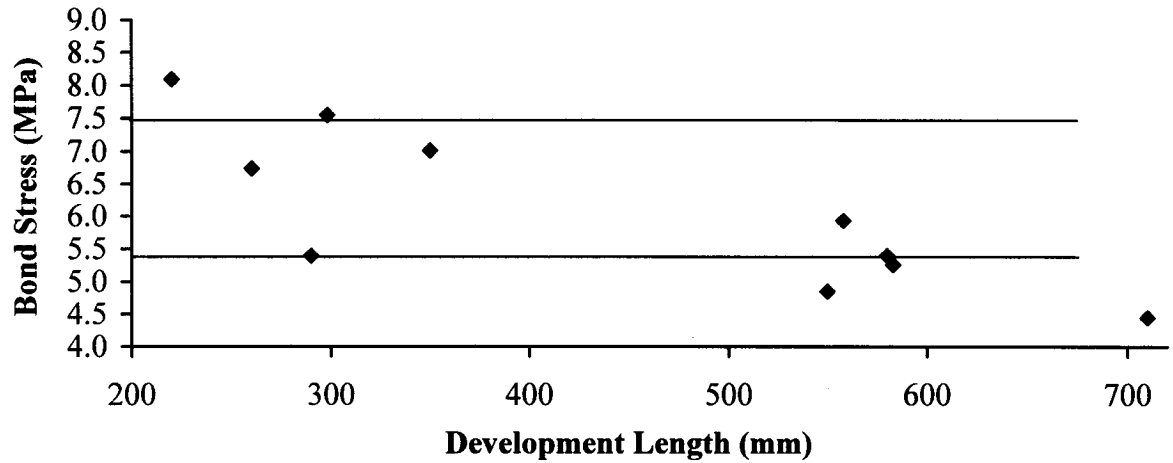
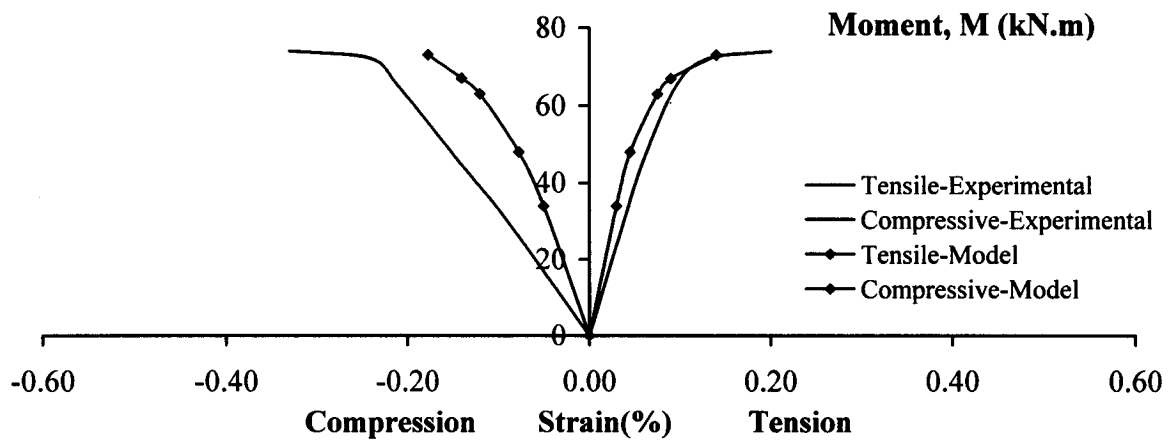
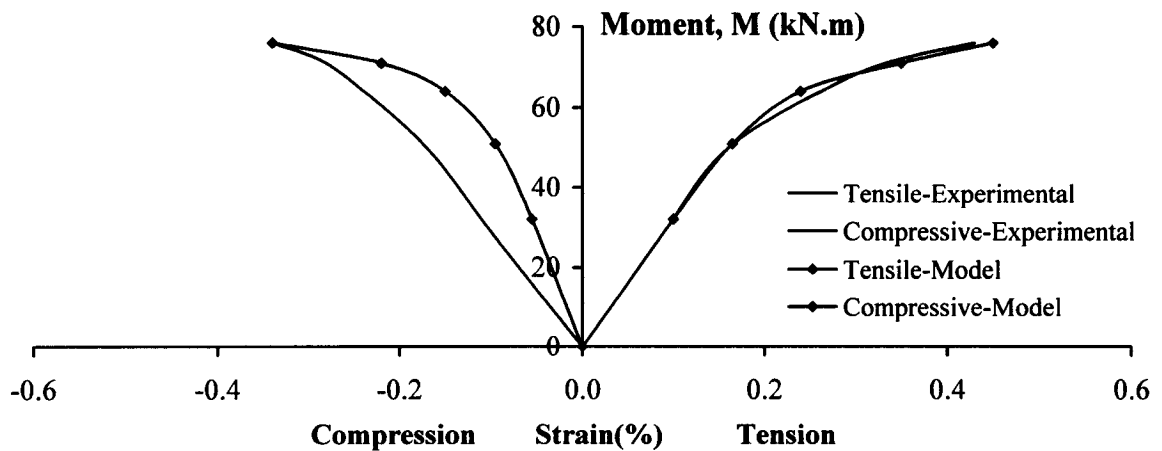


Figure 6.11: Development Length-Bond Stress Relationship

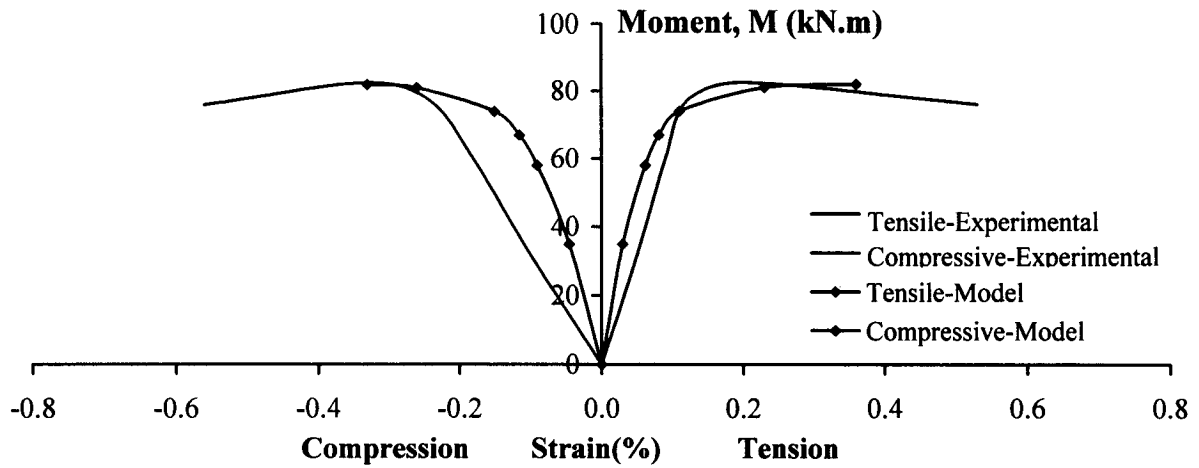


(a) Column CFS1

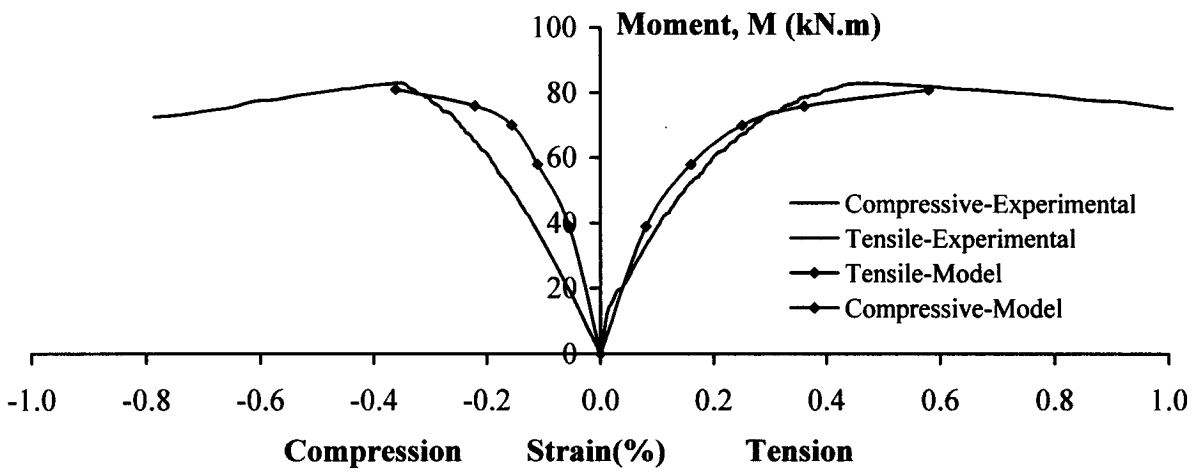


(b) Column CFS2

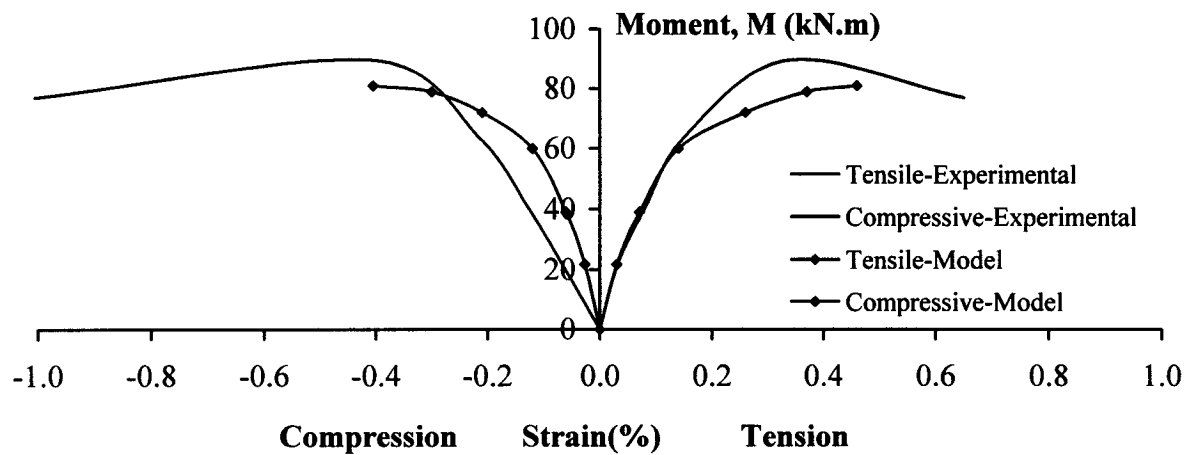
Figure 6.12: Moment-Strain Diagrams for FRP Bars in Phase I Columns (under Eccentric Loading)



(c) Column CFS3

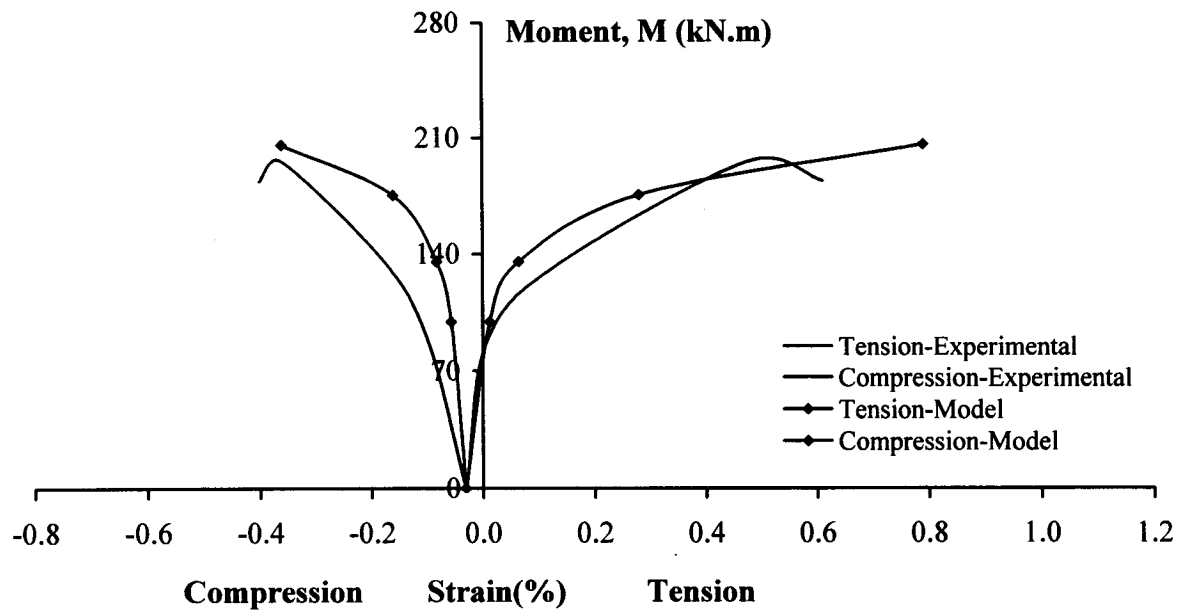


(d) Column CFS4

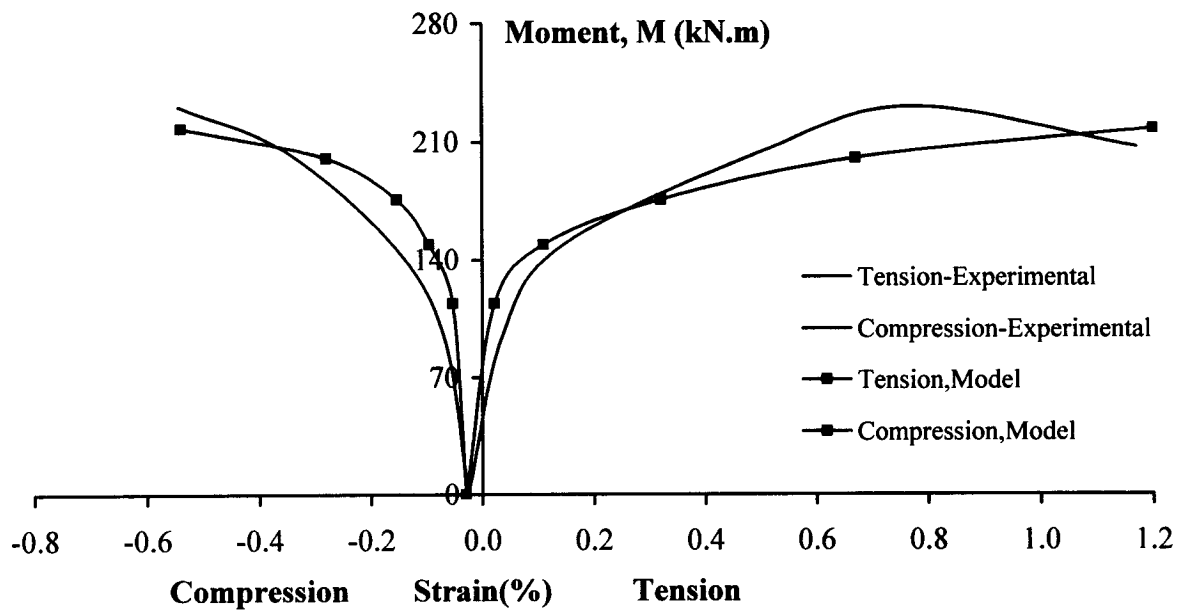


(e) Column CFS5

Figure 6.12 (Cont): Moment-Strain Diagrams for FRP Bars in Phase I Columns

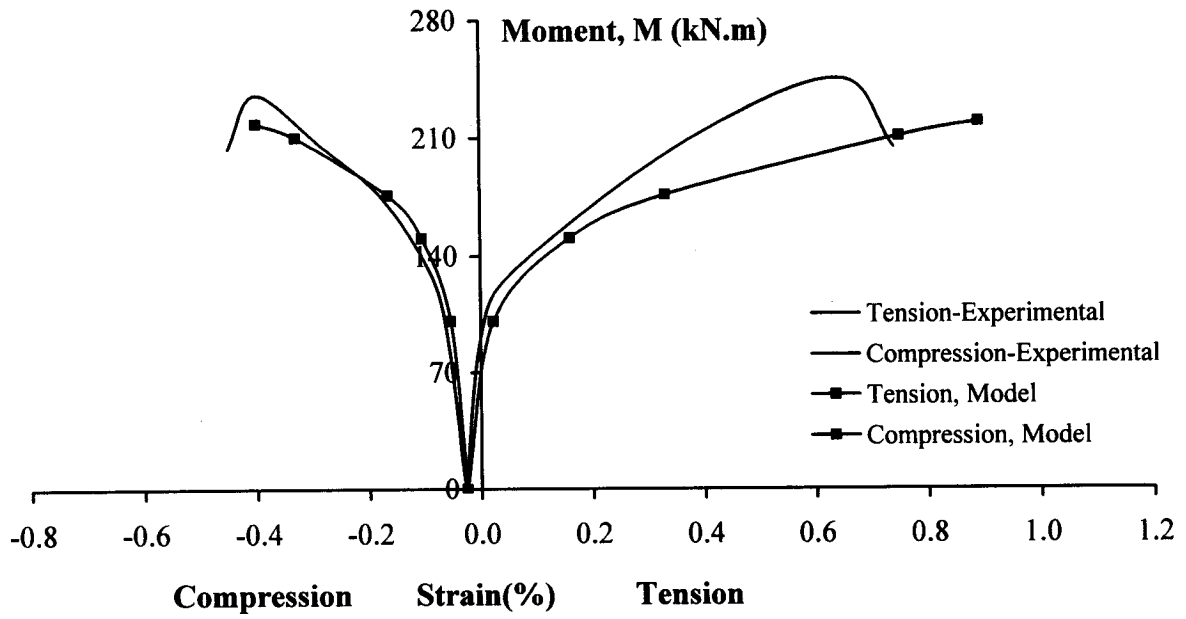


(a) Column CFCL1

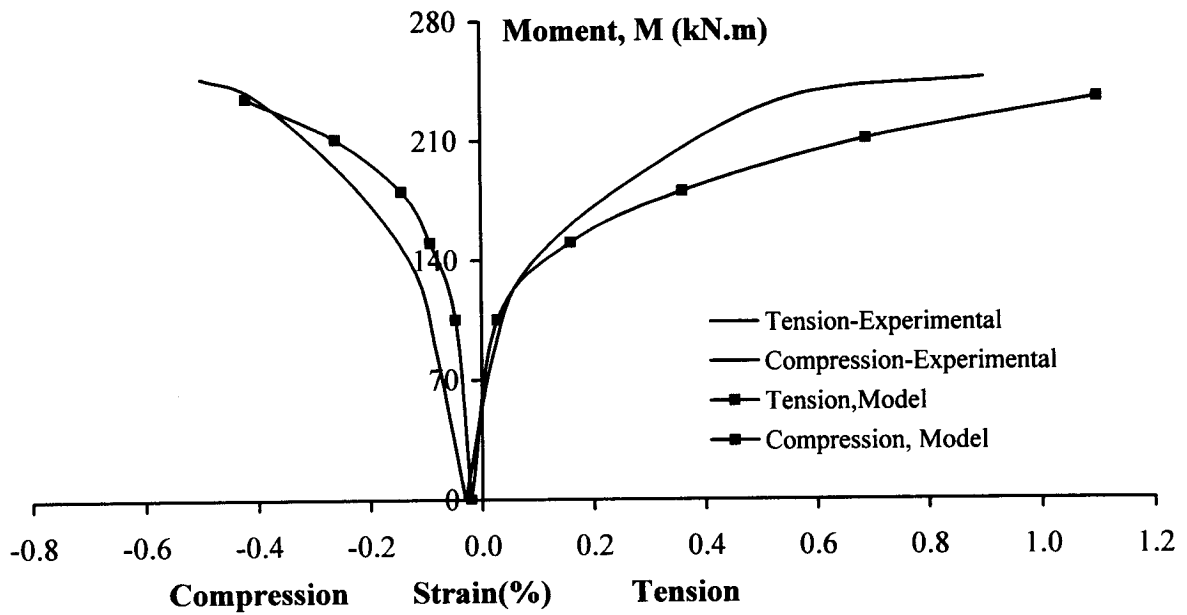


(b) Column CFCL2

Figure 6.13: Moment-Strain Diagrams for FRP Bars in Phase III Columns (under Cyclic Loading)

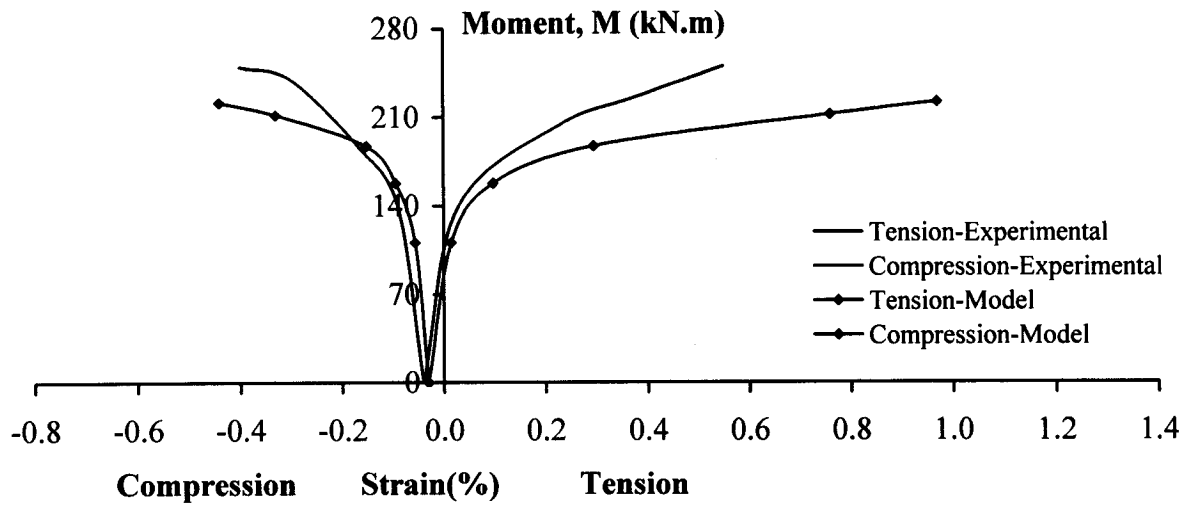


(c) Column CFCL3

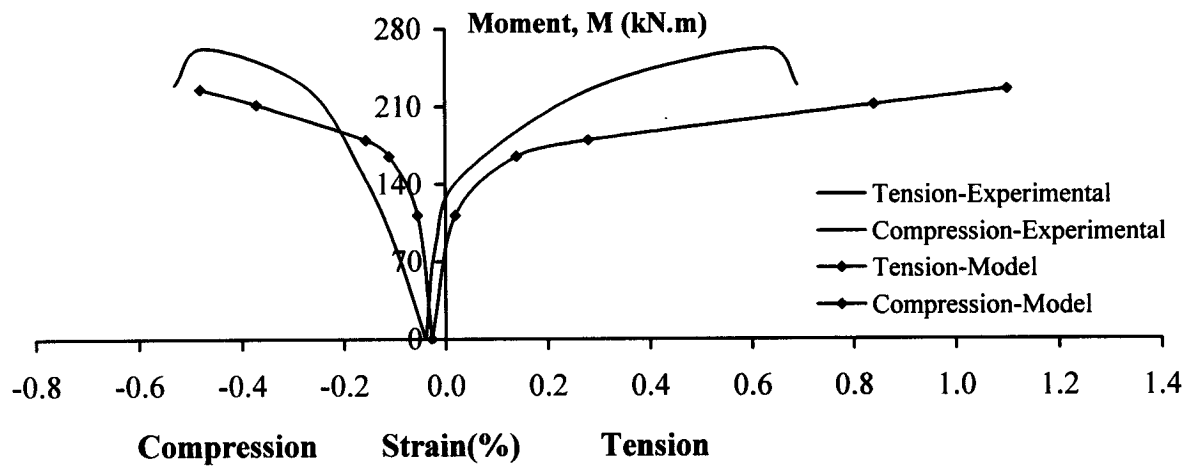


(d) Column CFCL4

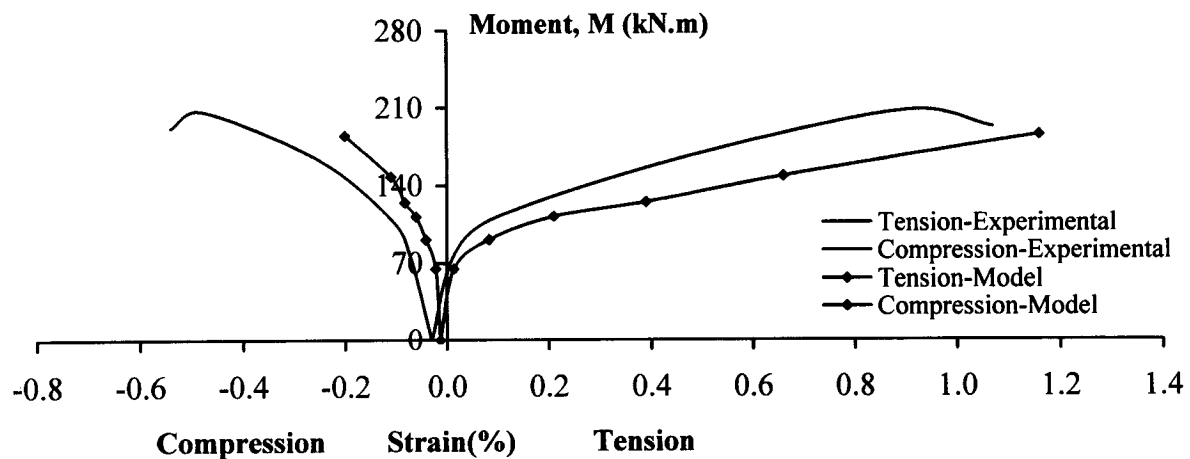
Figure 6.13 (Cont): Moment-Strain Diagrams for FRP Bars in Phase III Columns



(e) Column CFCL5

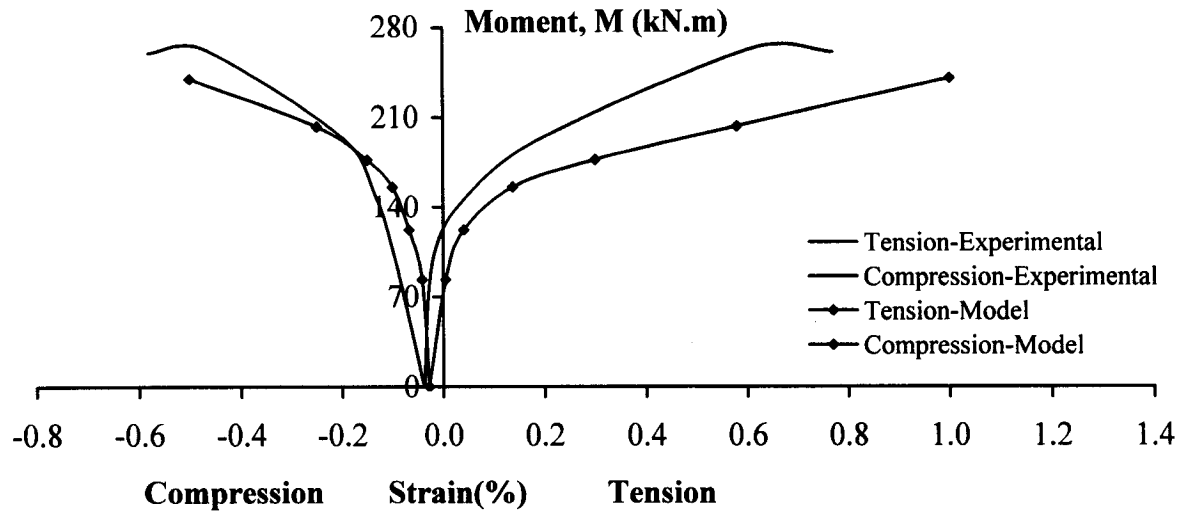


(f) Column CFCL6

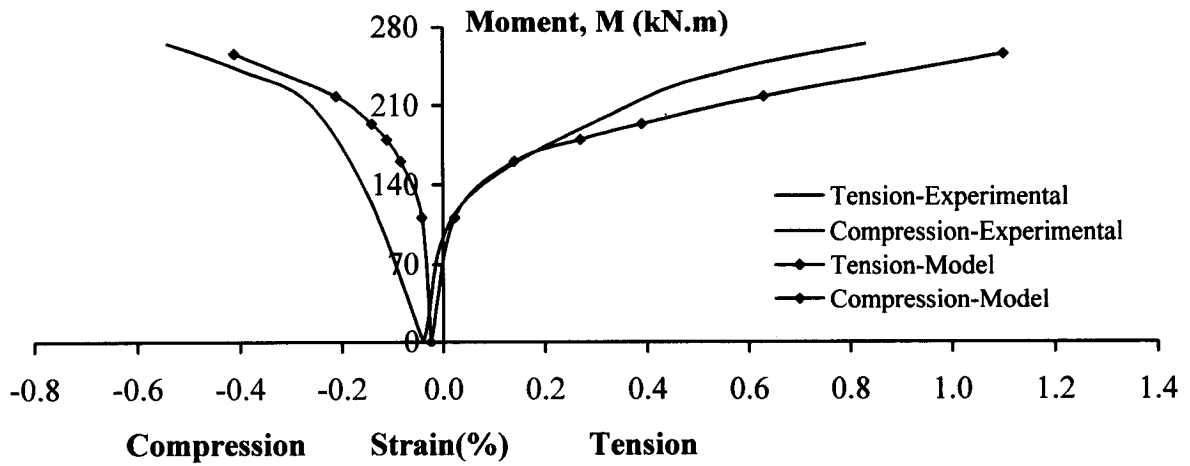


(g) Column CFCL7

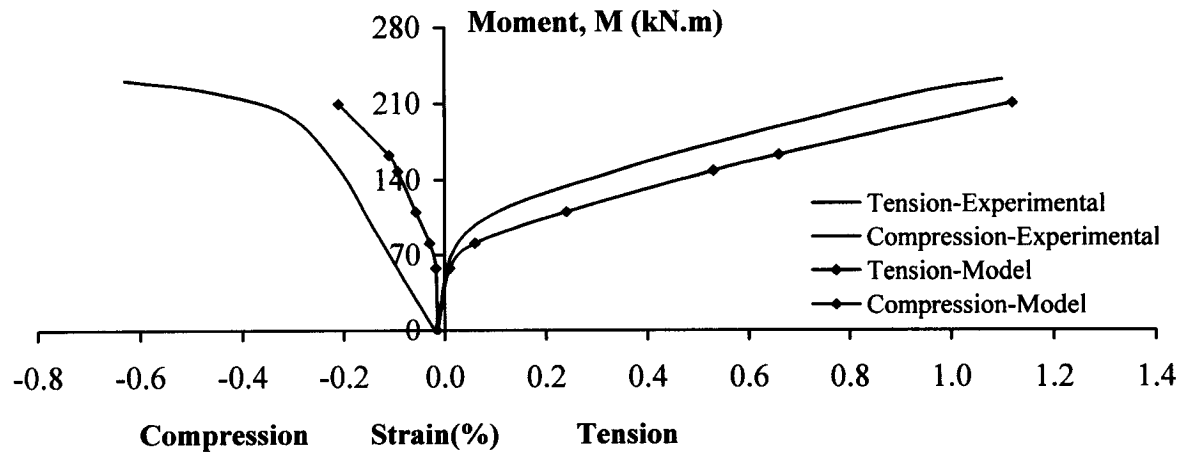
Figure 6.13 (Cont): Moment-Strain Diagrams for FRP Bars in Phase III Columns



(h) Column CFCL8

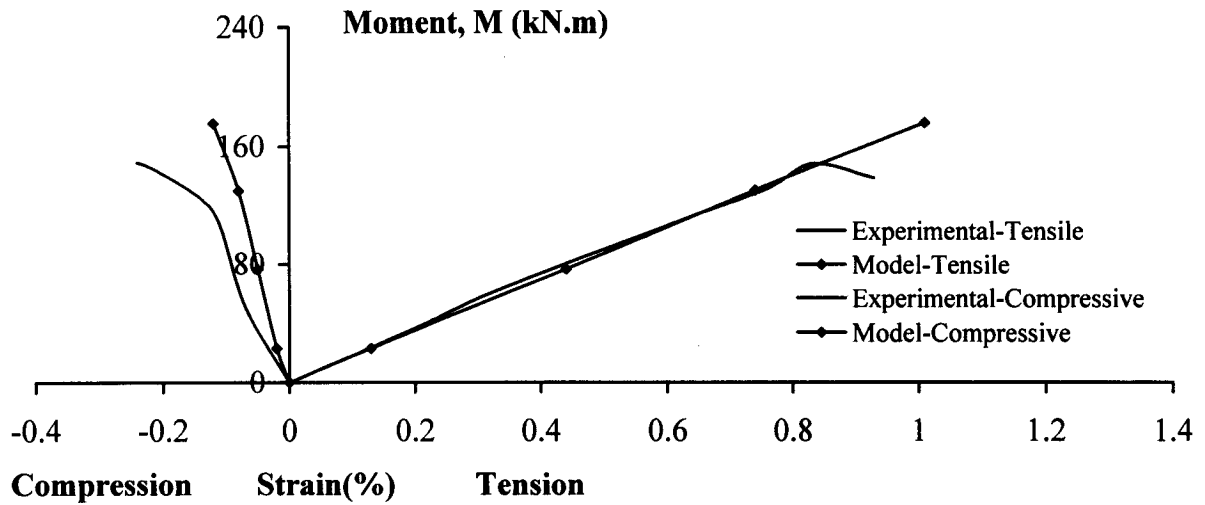


(i) Column CFCL9

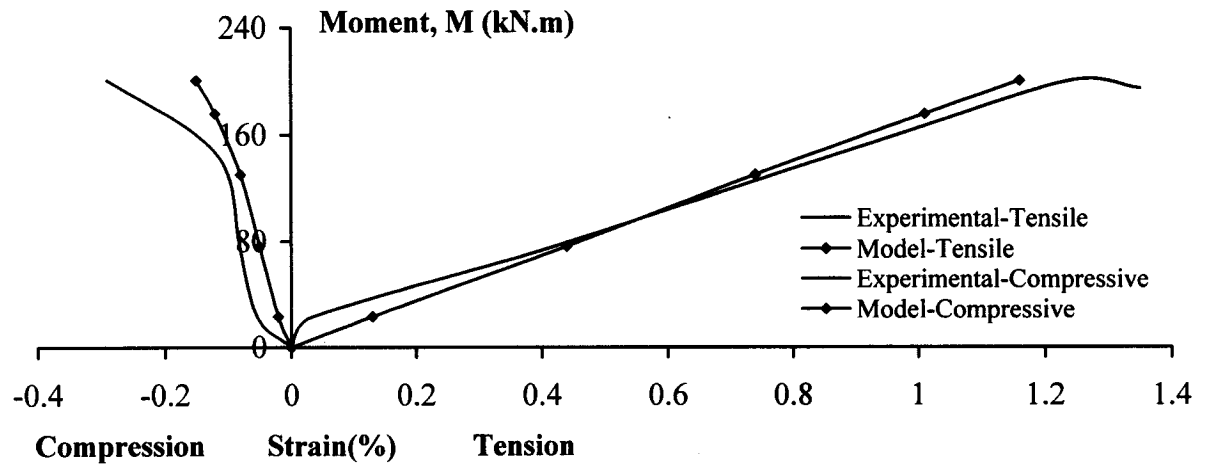


(j) Column CFCL10

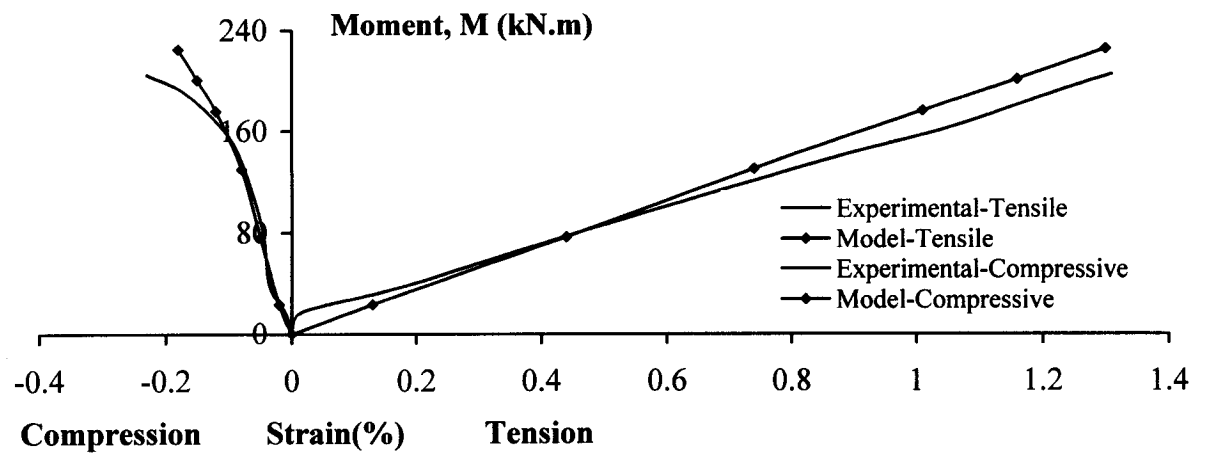
Figure 6.13 (Cont): Moment-Strain Diagrams for FRP Bars in Phase III Columns



(a) Beam CFB1

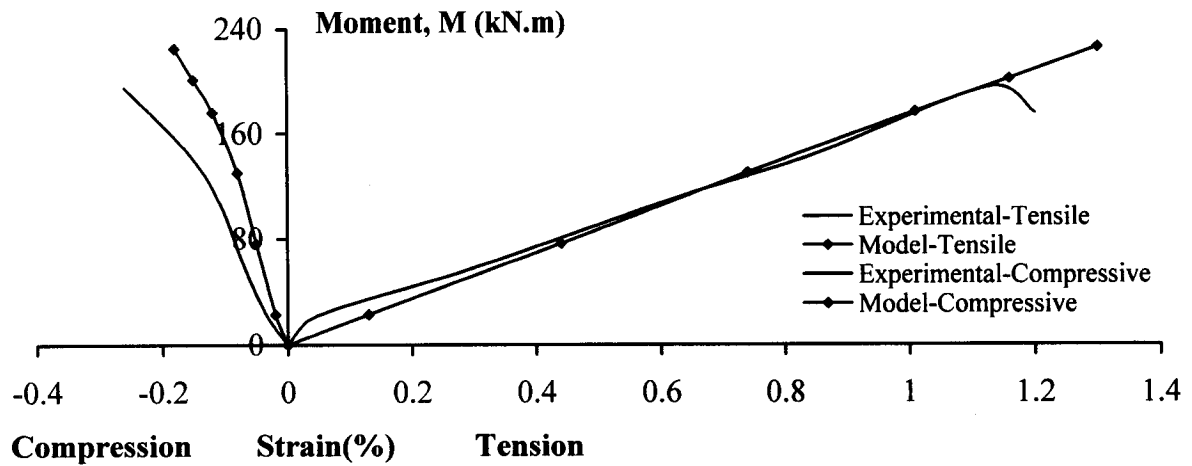


(b) Beam CFB2

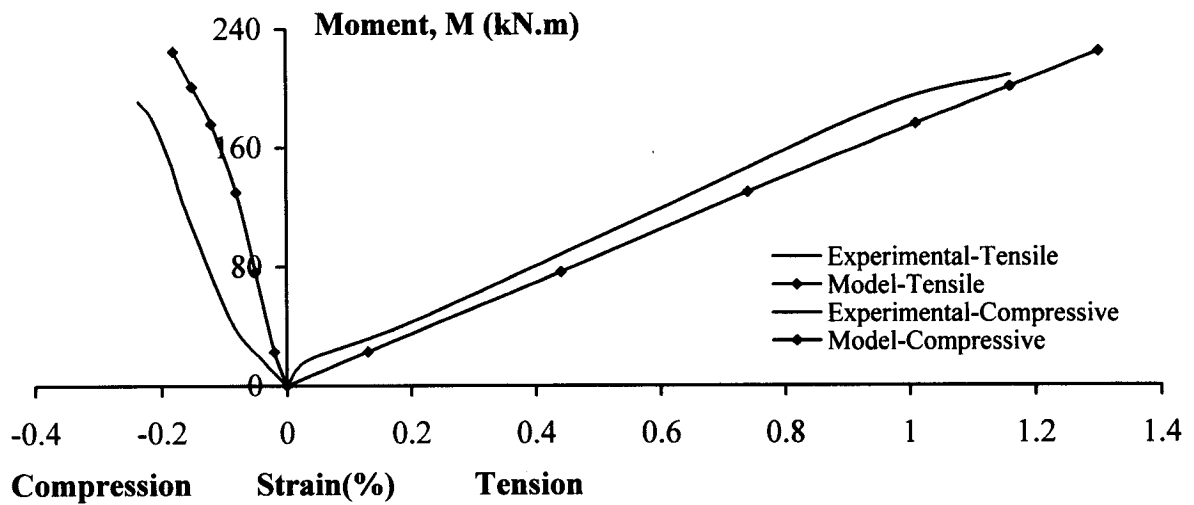


(c) Beam CFB3

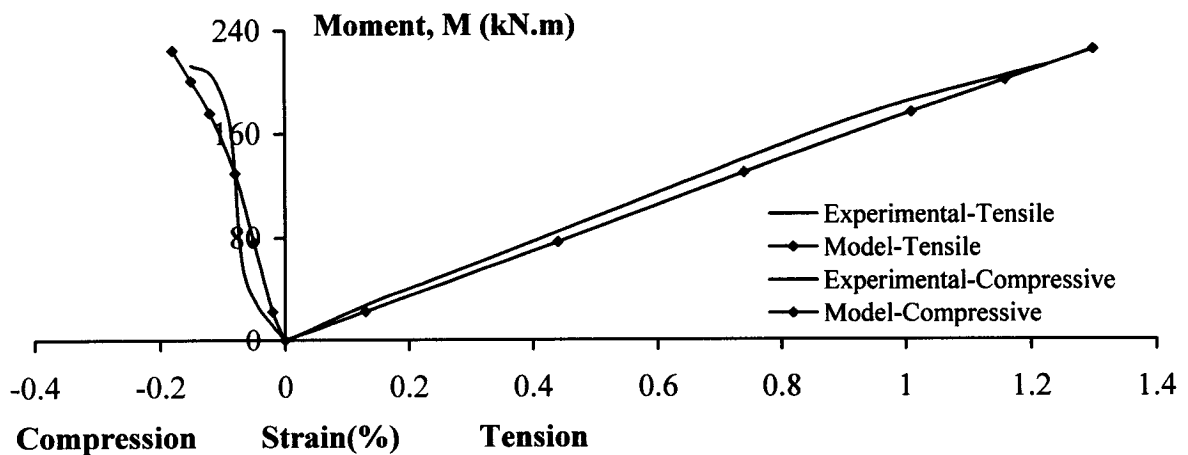
Figure 6.14: Moment-Strain Diagrams for FRP Bars in Phase IV Beams



(d) Beam CFB4



(e) Beam CFB5



(f) Beam CFB6

Figure 6.14 (Cont.): Moment-Strain Diagrams for FRP Bars in Phase IV Beams



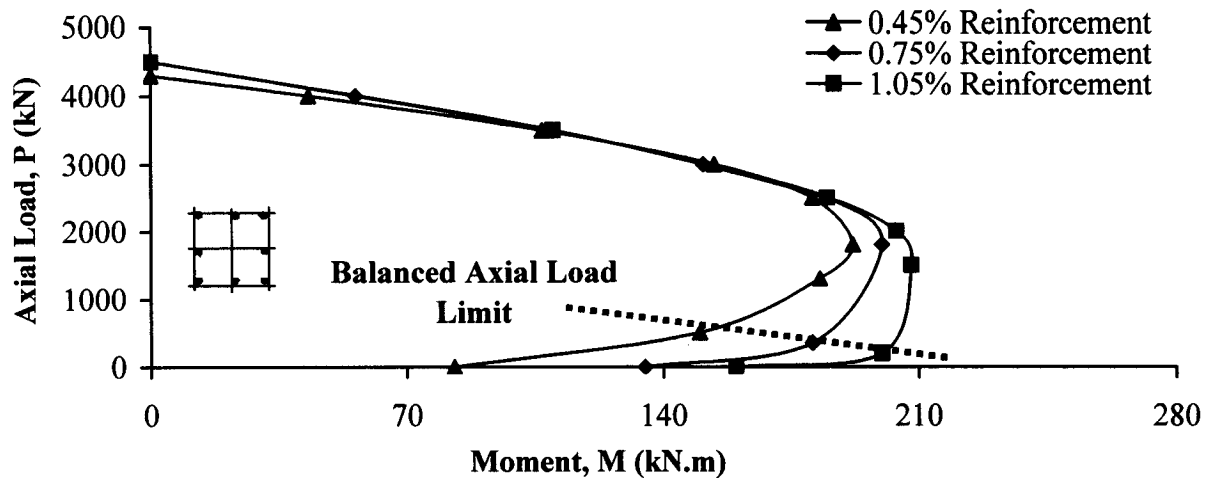
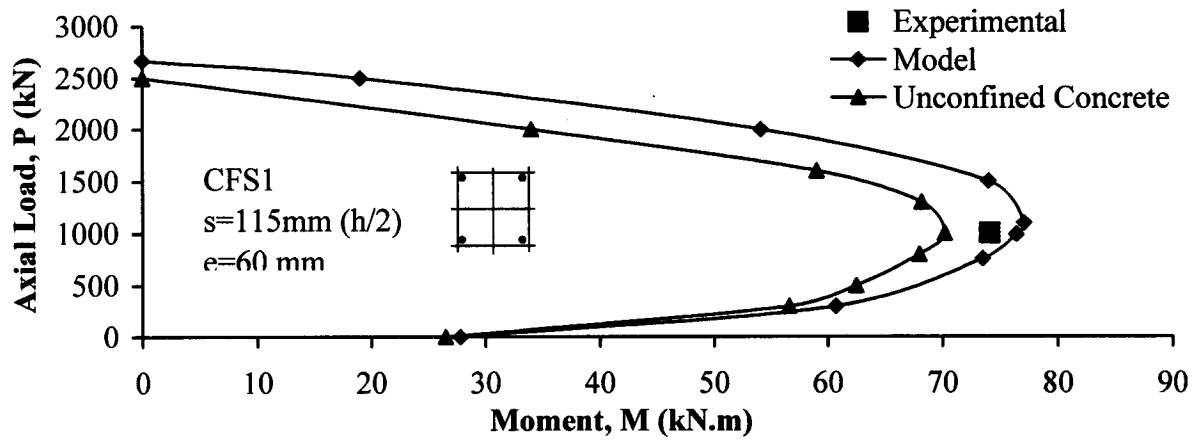
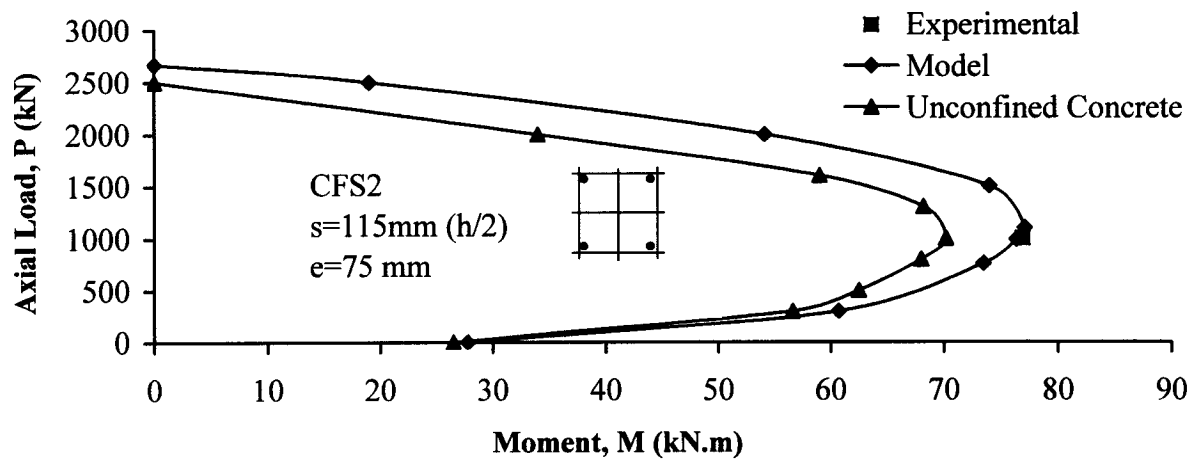


Figure 6.15: Typical Interaction Diagrams for Different Reinforcement Ratios

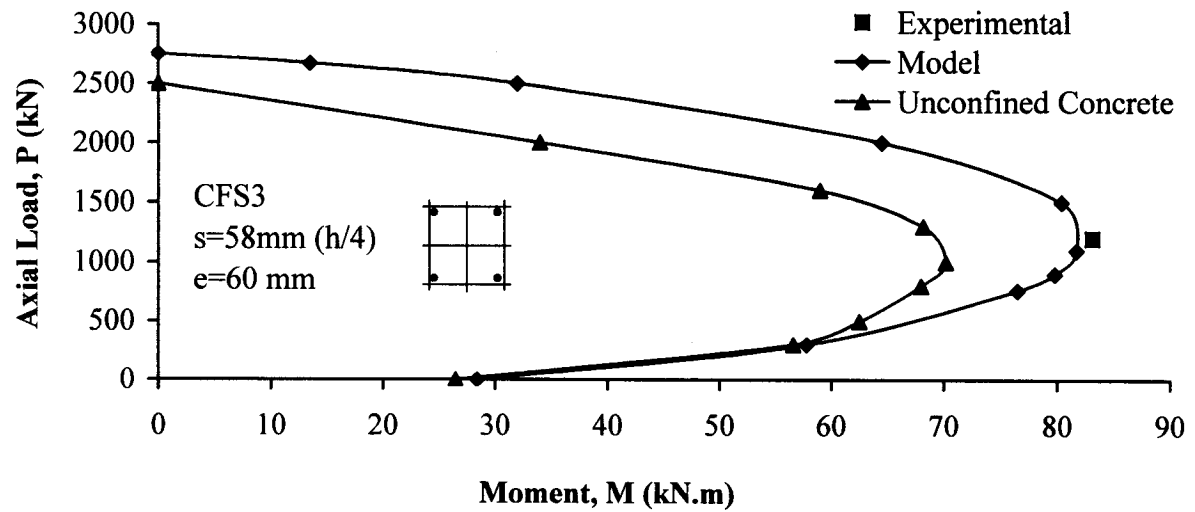


(a) Column CFS1

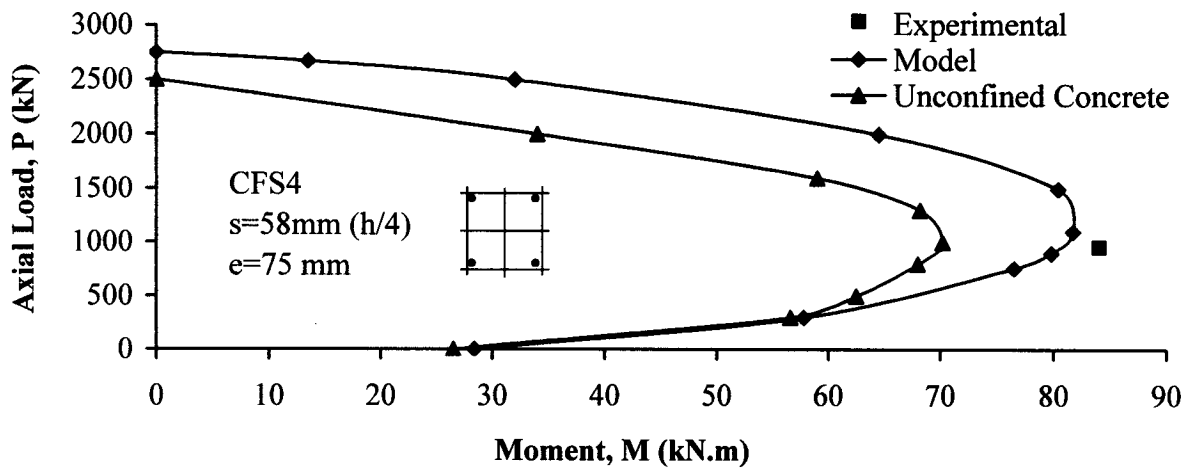


(b) Column CFS2

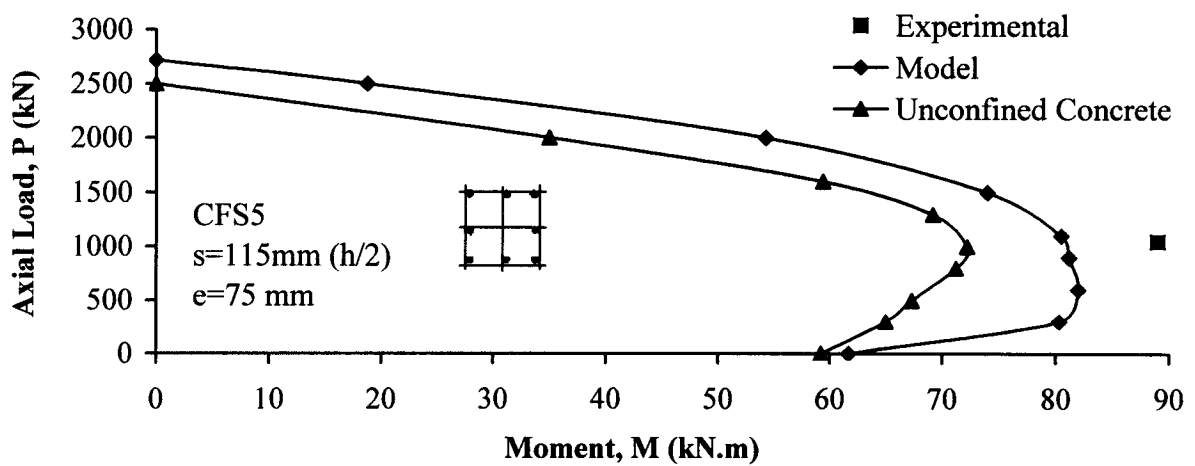
Figure 6.16: Interaction Diagrams for Phase I Columns (under Eccentric Loading )



(c) Column CFS3

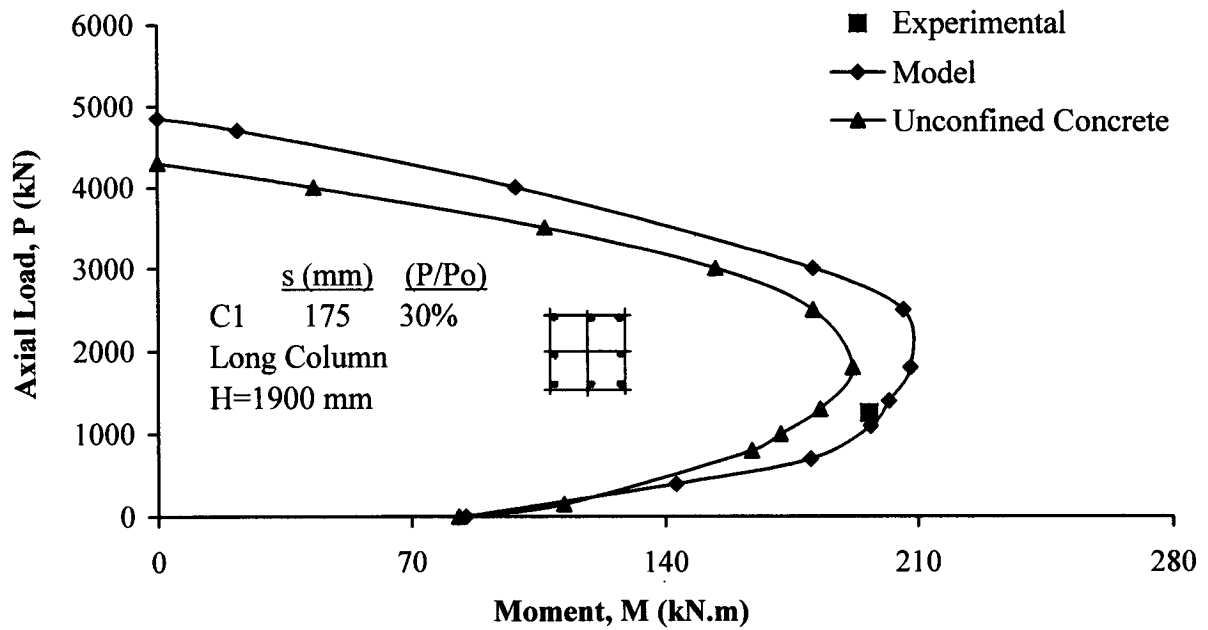


(d) Column CFS4

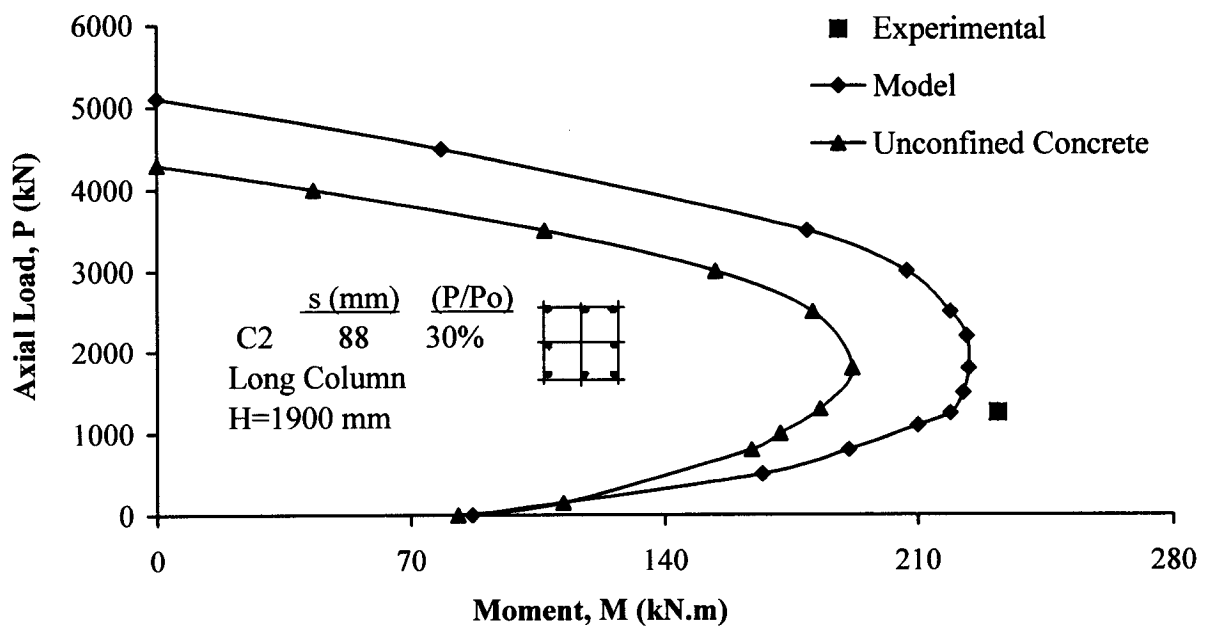


(e) Column CFS5

Figure 6.16 (Cont): Interaction Diagrams for Phase I Columns

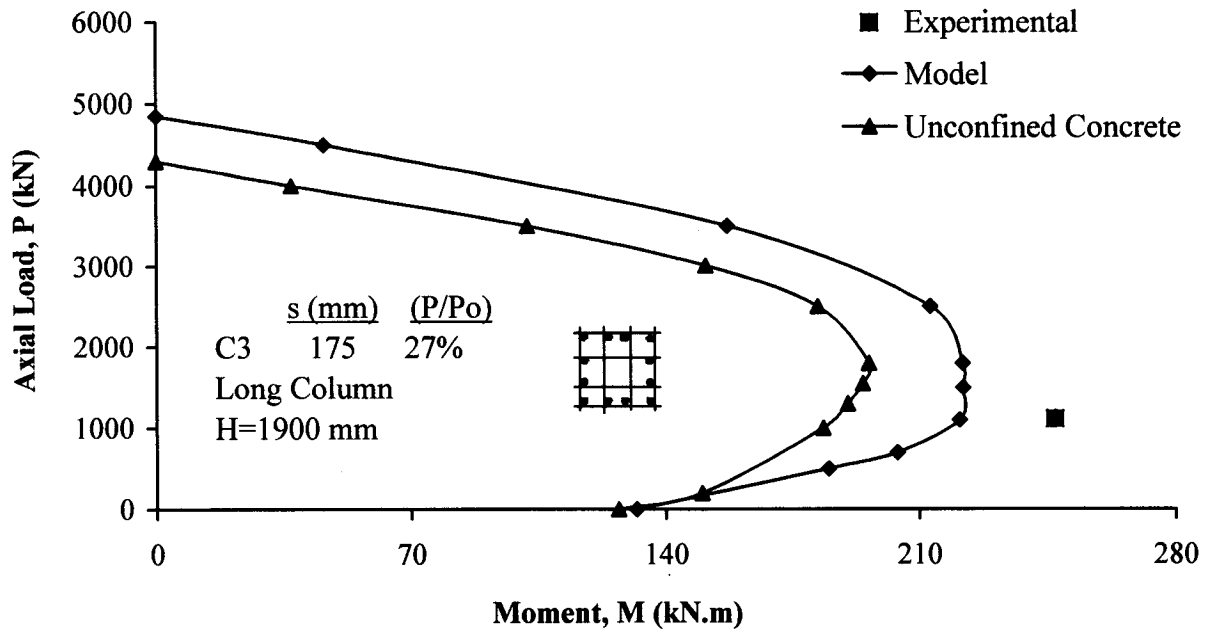


(a) Column CFCL1

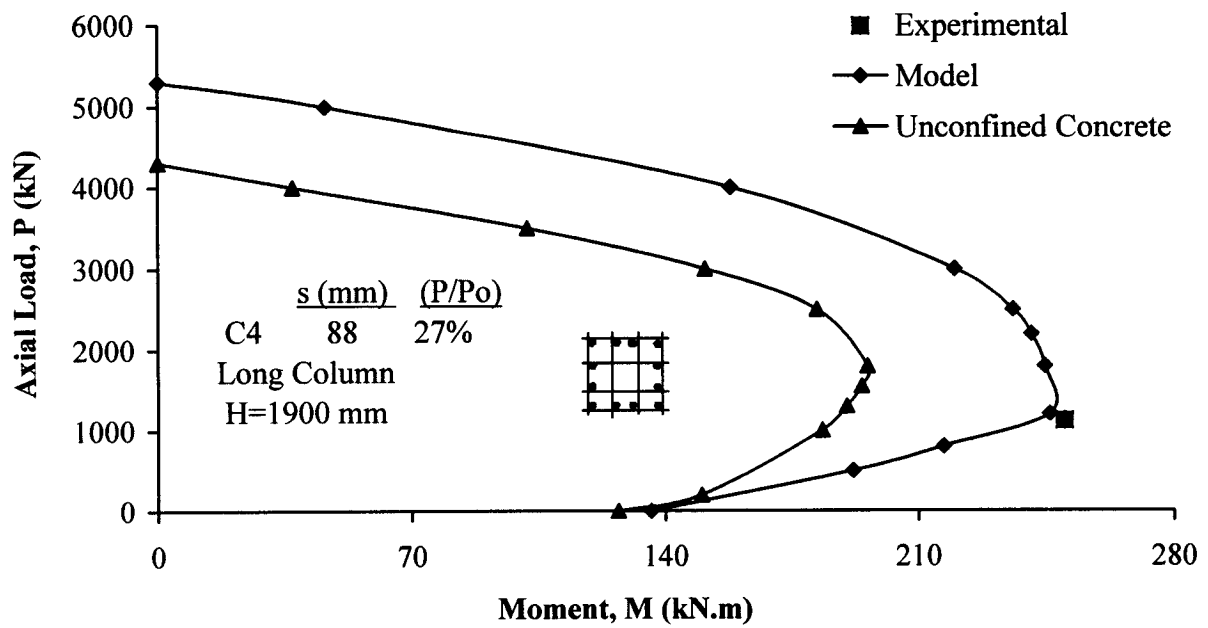


(b) Column CFCL2

Figure 6.17: Interaction Diagrams for Phase III Columns ( under Cyclic Loading)

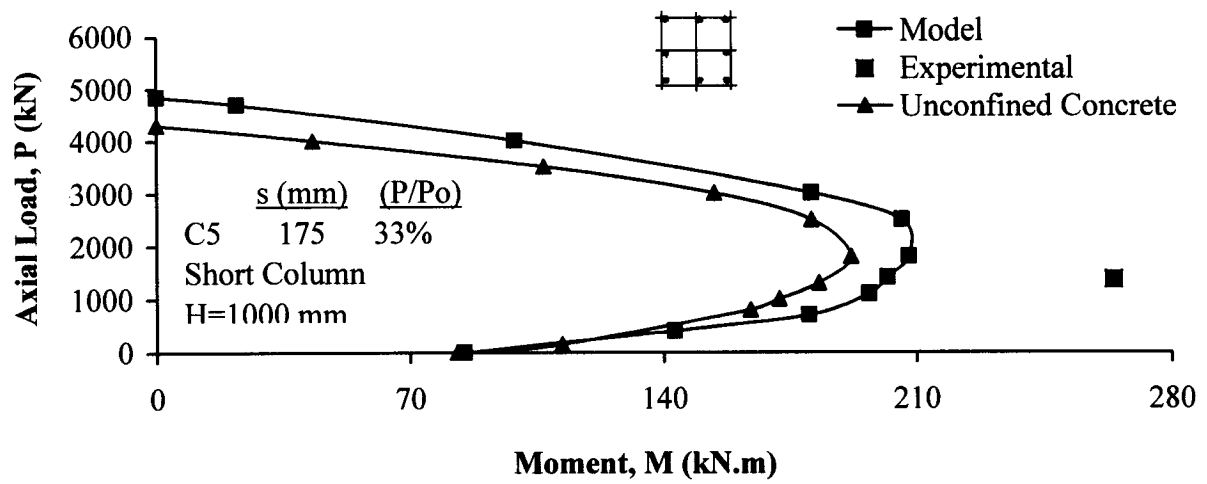


(c) Column CFCL3

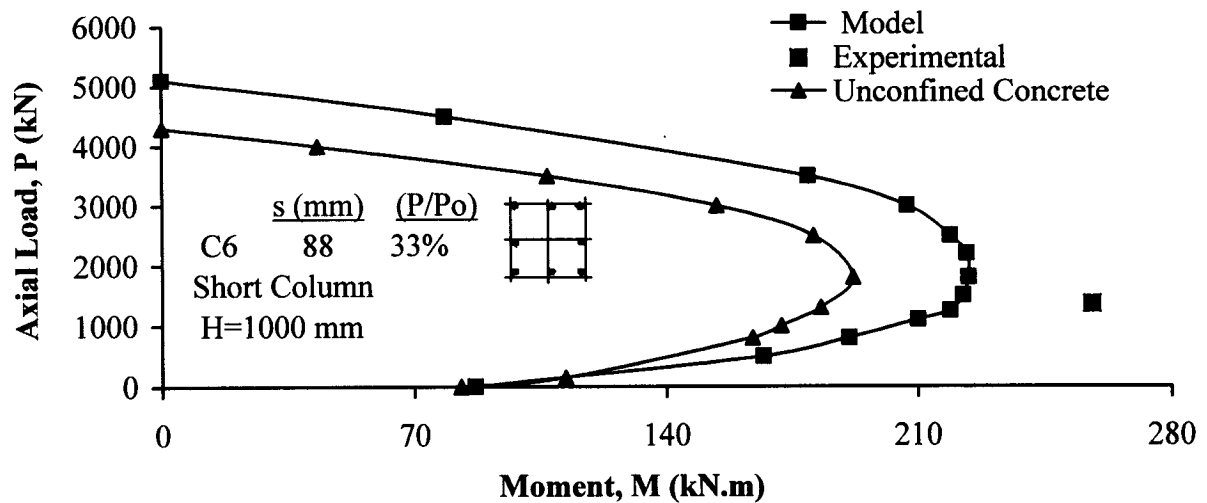


(d) Column CFCL4

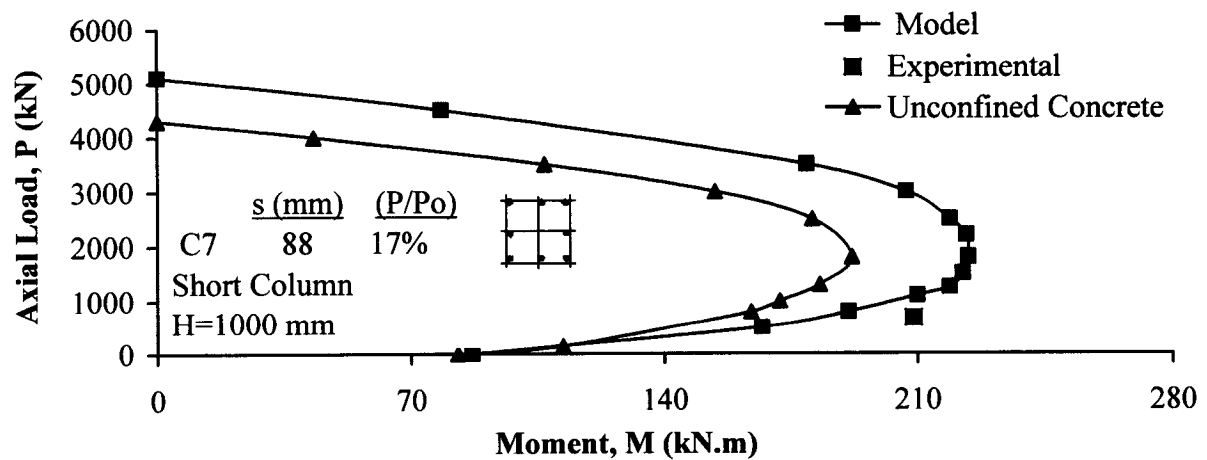
Figure 6.17 (Cont): Interaction Diagrams for Phase III Columns



(e) Column CFCL5

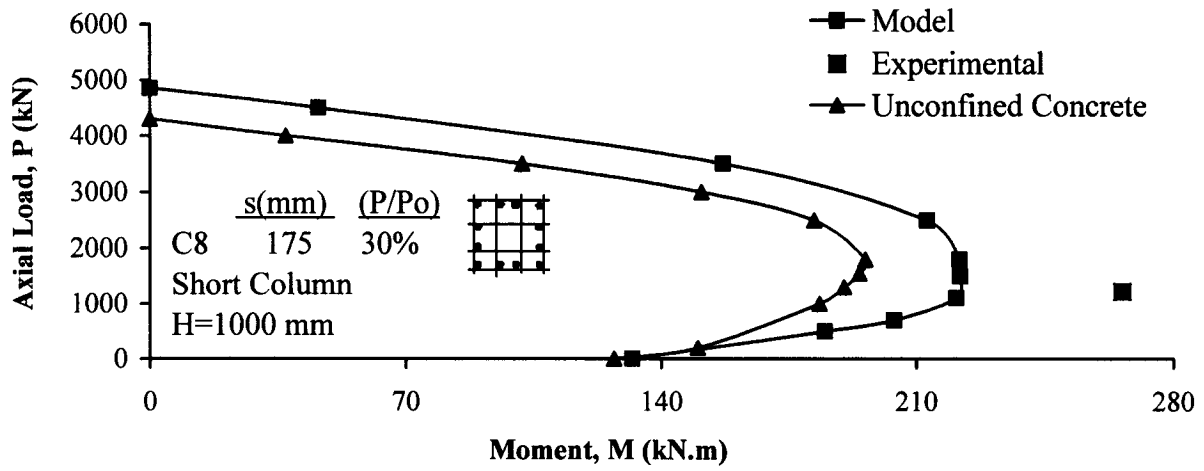


(f) Column CFCL6

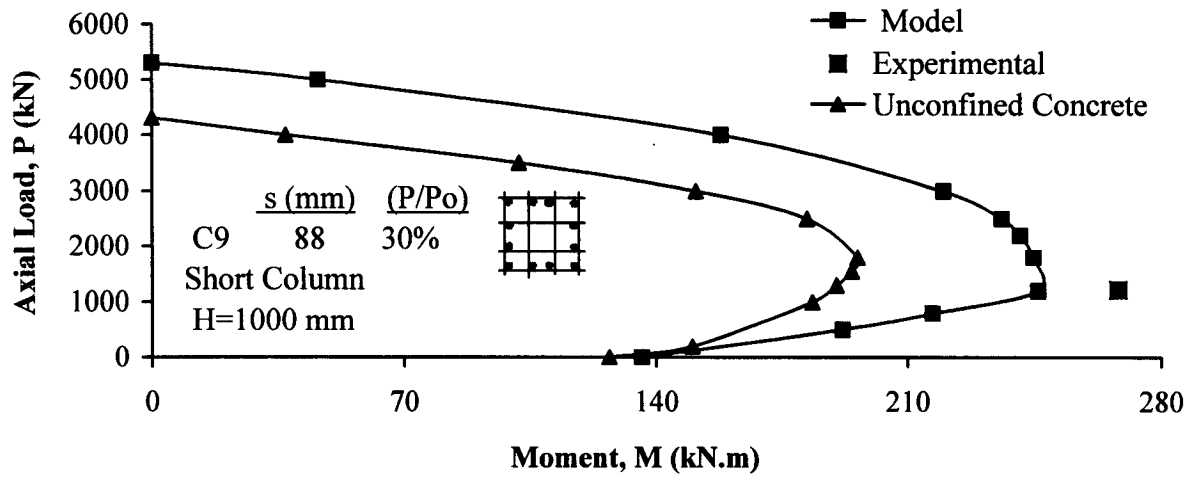


(g) Column CFCL7

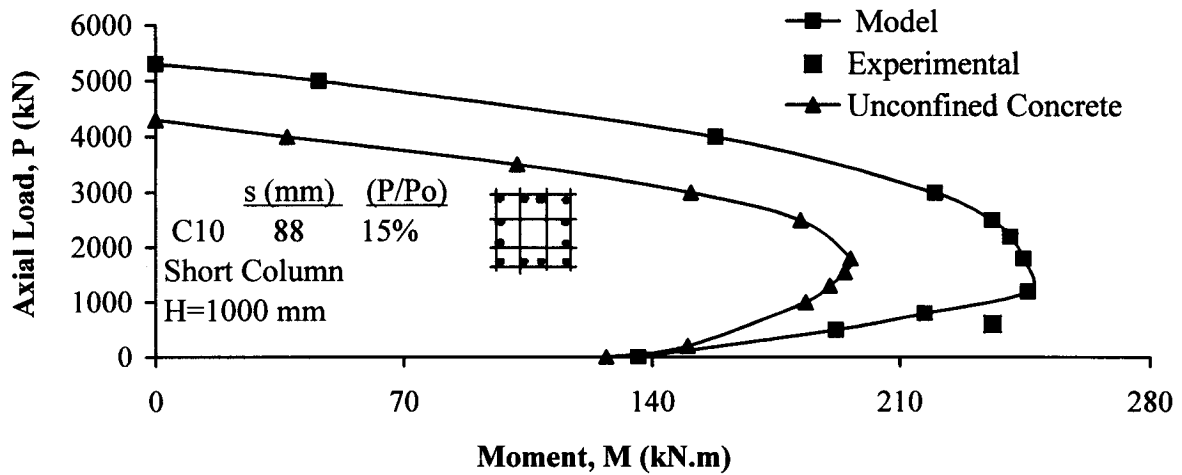
Figure 6.17 (Cont): Interaction Diagrams for Phase III Columns



(h) Column CFCL8



(i) Column CFCL9



(j) Column CFCL10

Figure 6.17 (Cont): Interaction Diagrams for Phase III Columns

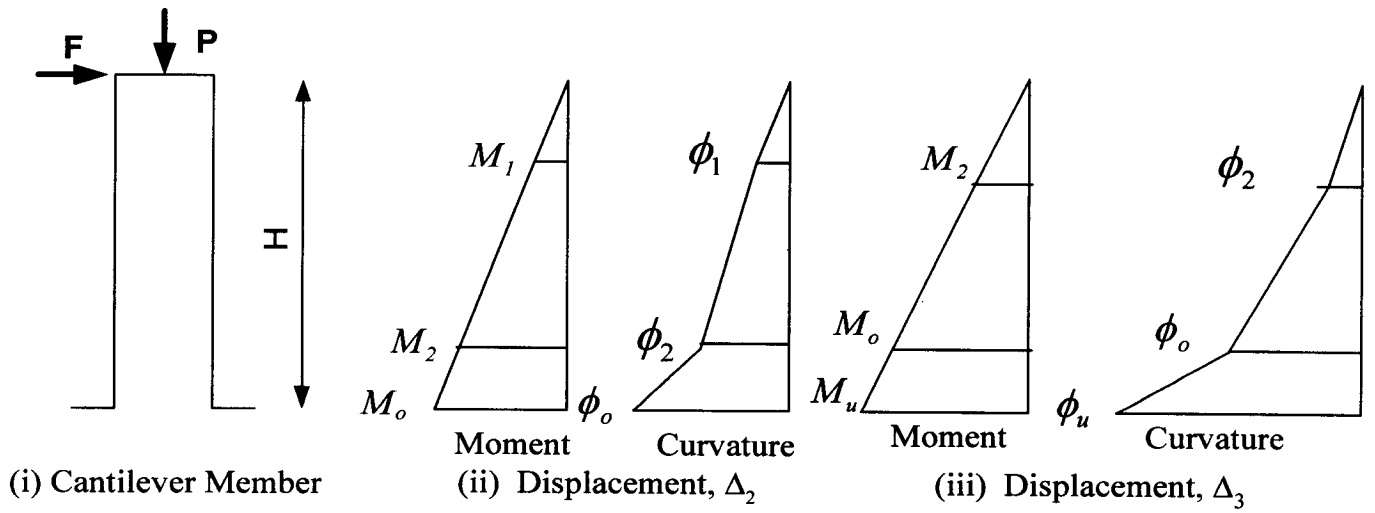
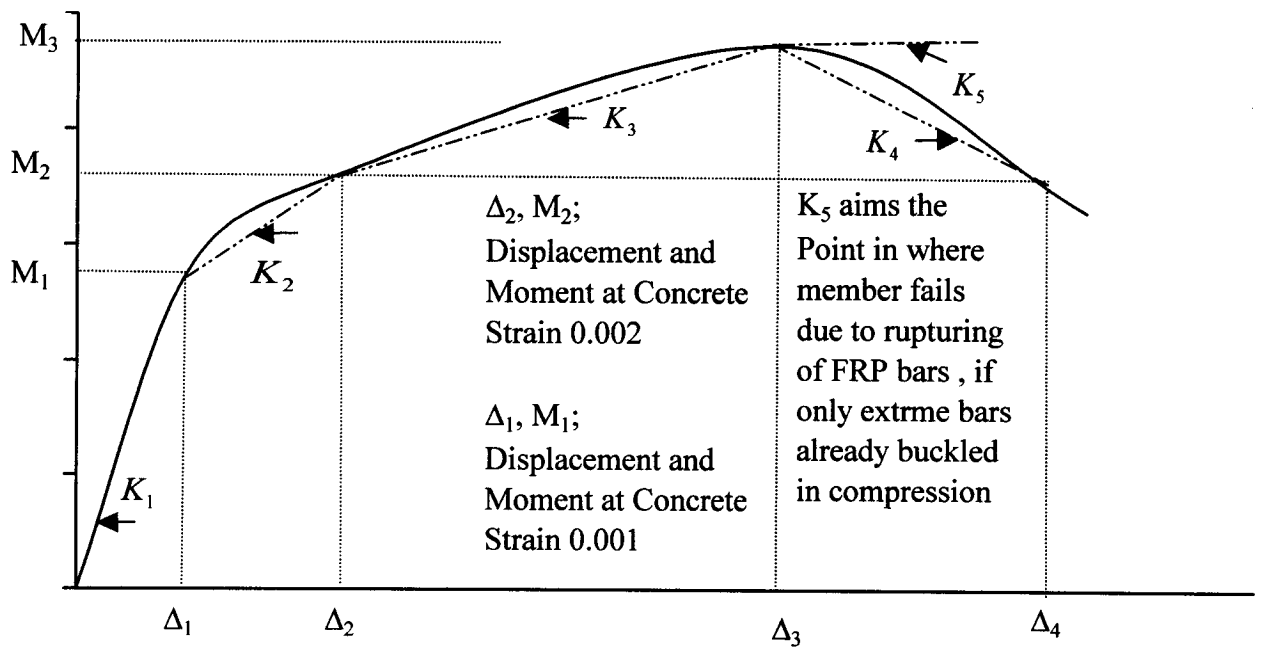
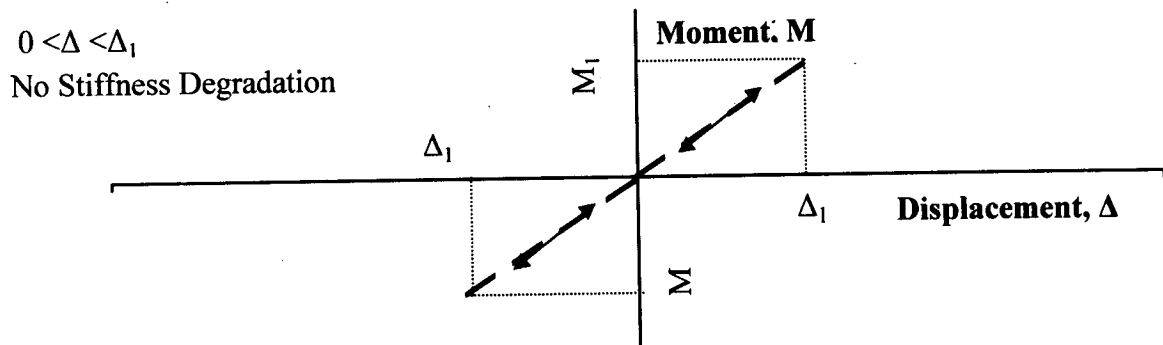


Figure 6.18: Calculation of Displacements of Primary Curve of Proposed M-Displacement Model

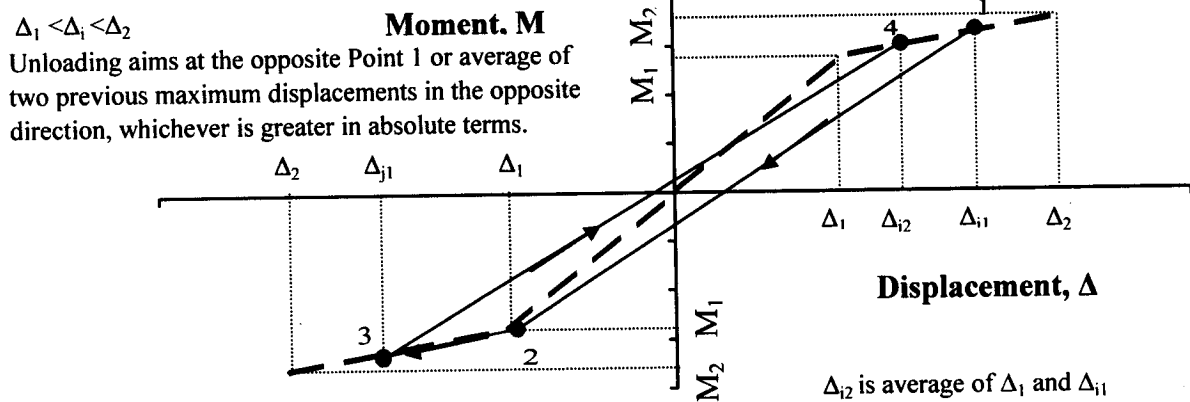


(a) General Primary Curve

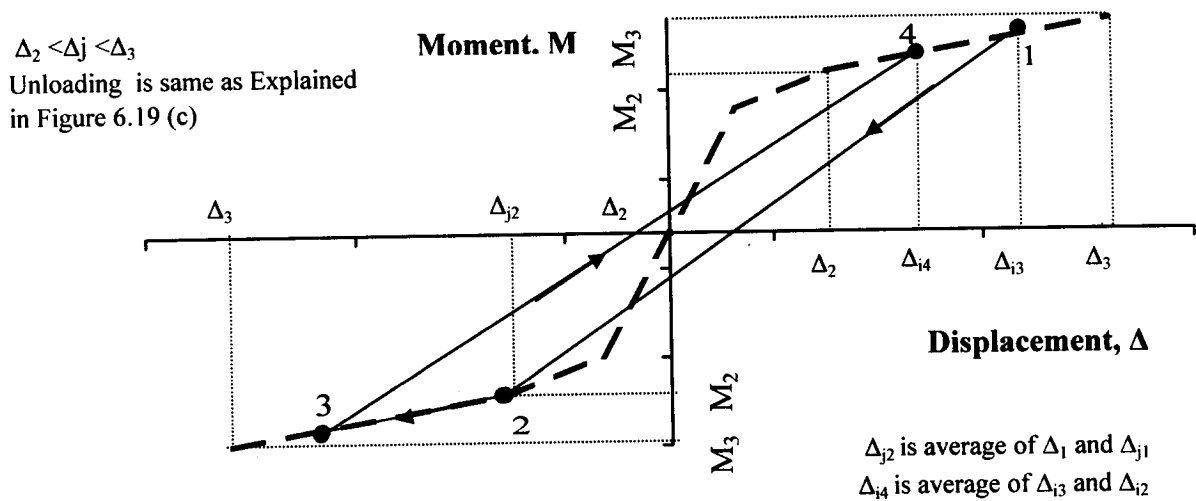
Figure 6.19: Proposed Hysteretic Moment-Flexural Displacement Model  
For FRP Reinforced Concrete Members



( b ) The Behavior of First Segment



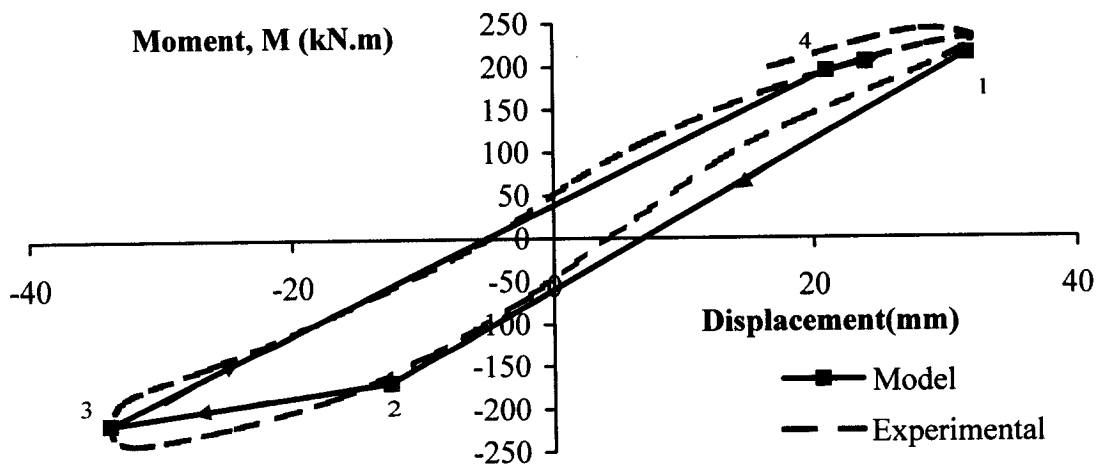
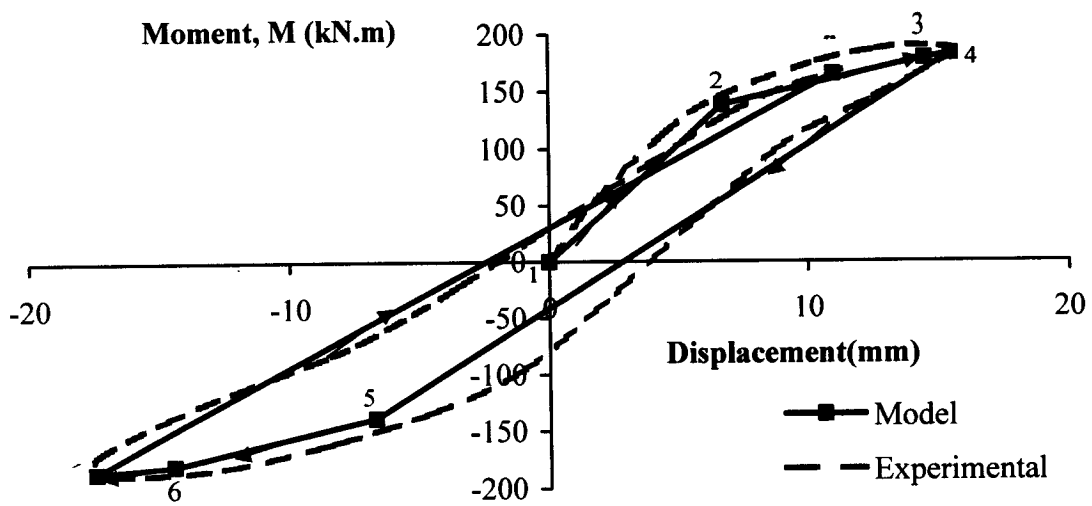
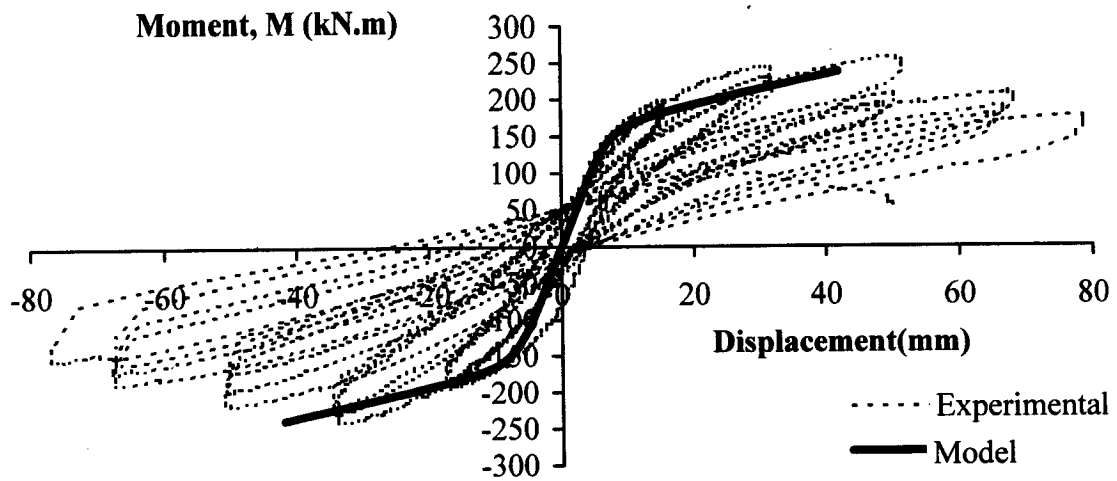
( c ) The Behavior of Second Segment



( d ) The Behavior of Third Segment

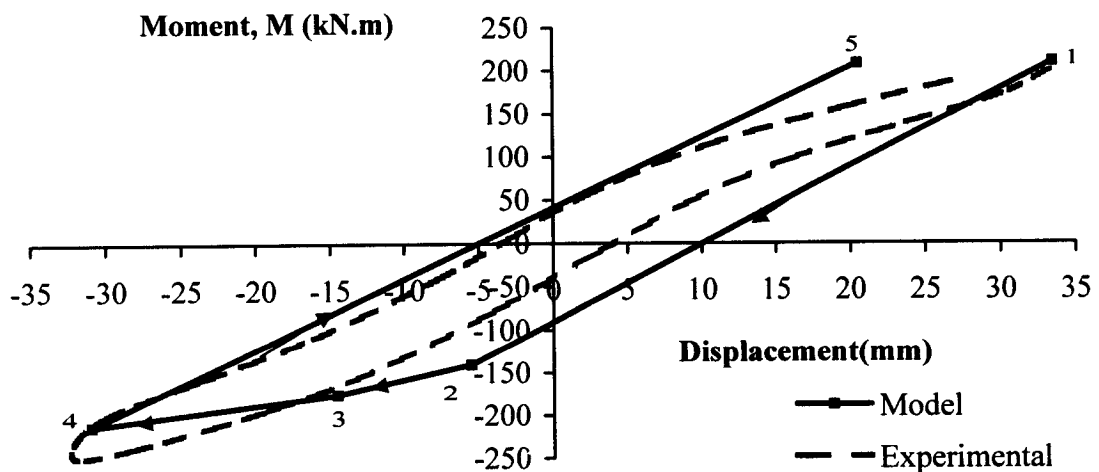
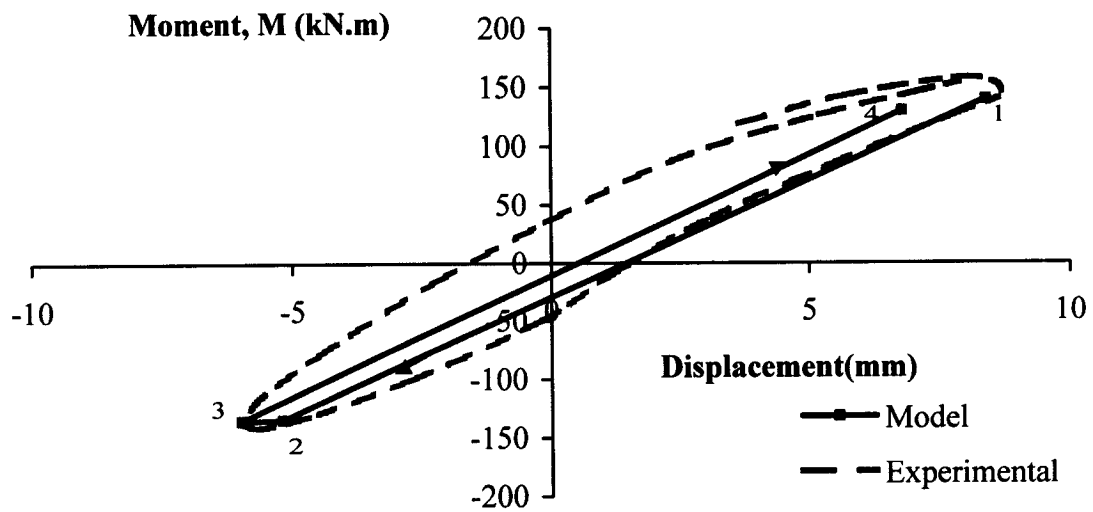
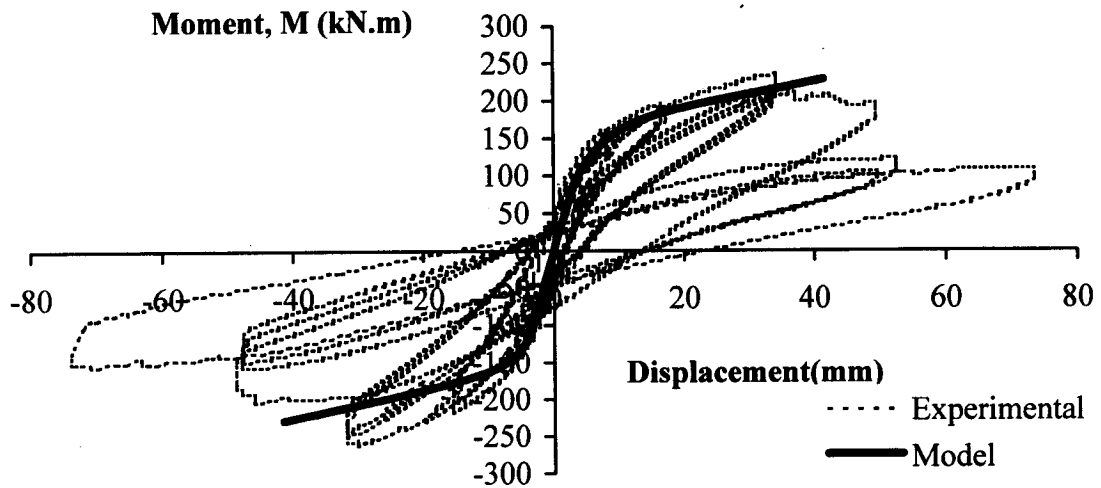
Figure 6.19 (Cont): Proposed Hysteretic Moment-Flexural Displacement Model For FRP Reinforced Concrete Members





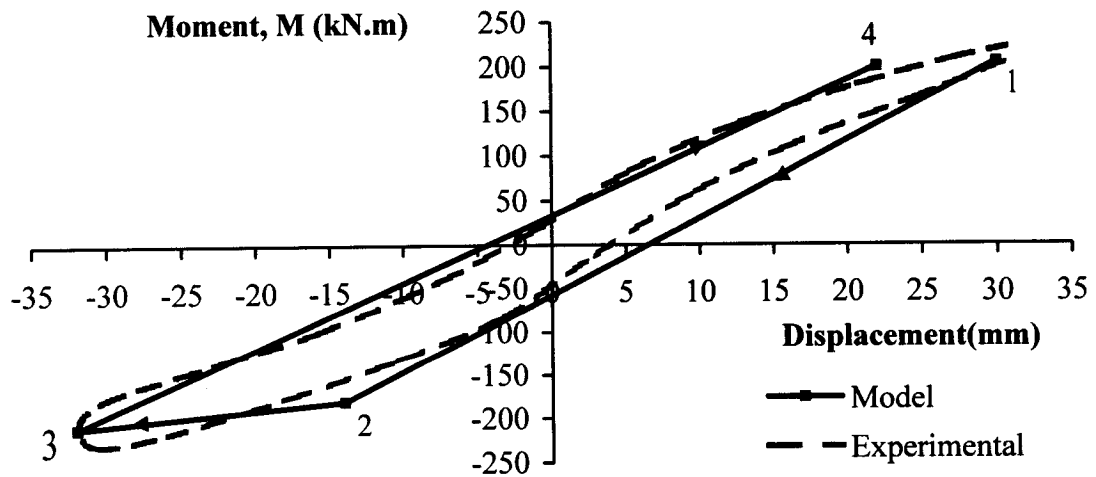
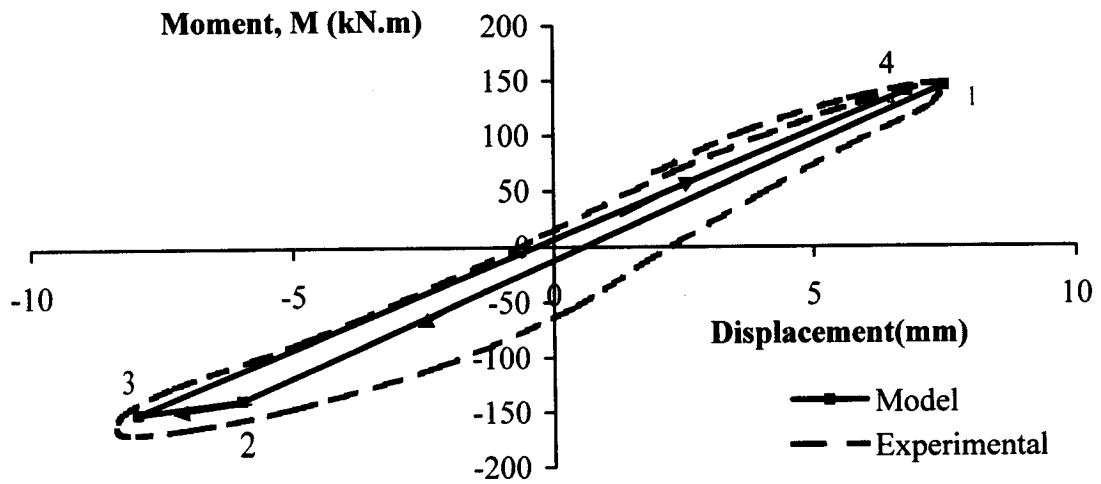
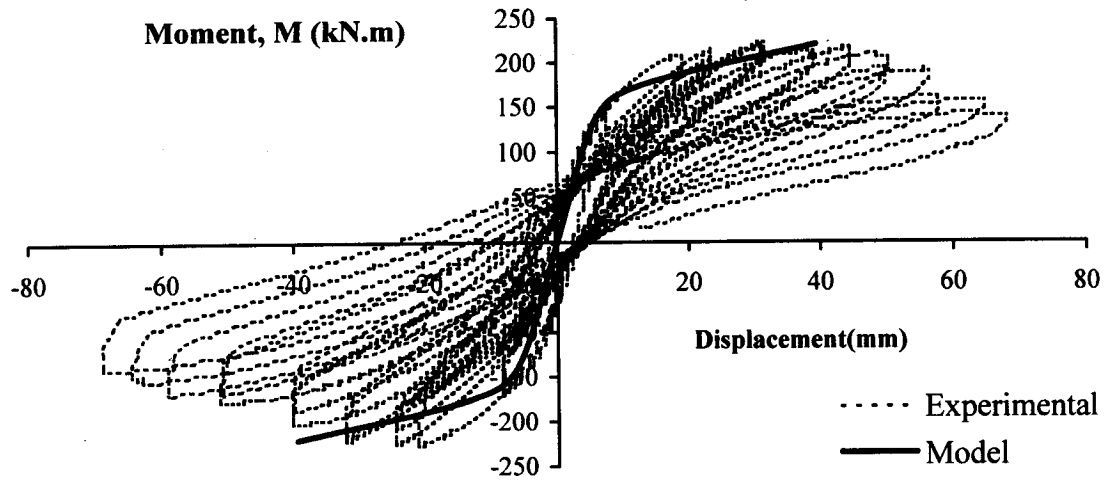
(a) Column CFCL4

Figure 6.20: Comparison of Proposed Hysteretic Model and Experimental Loops of Columns



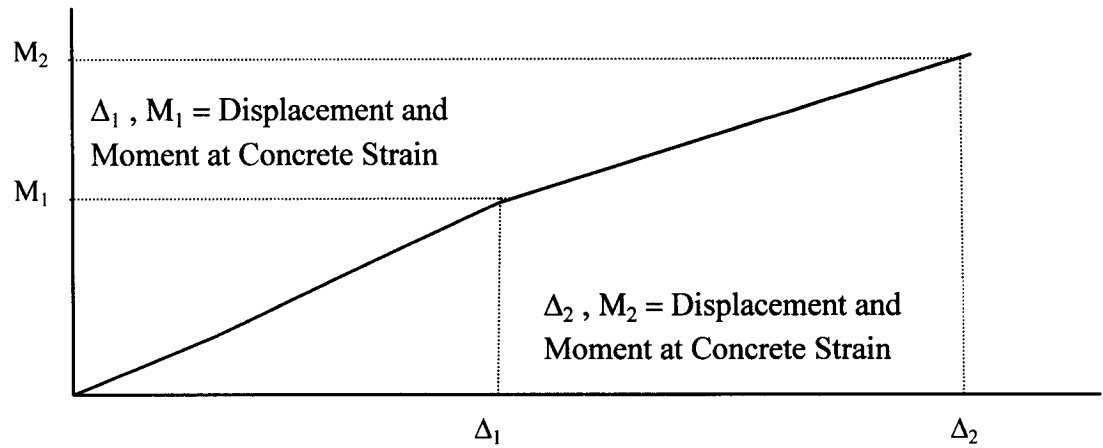
(b) Column CFCL3

Figure 6.20 (Cont.): Comparison of Proposed Hysteretic Model and Experimental Loops of Columns

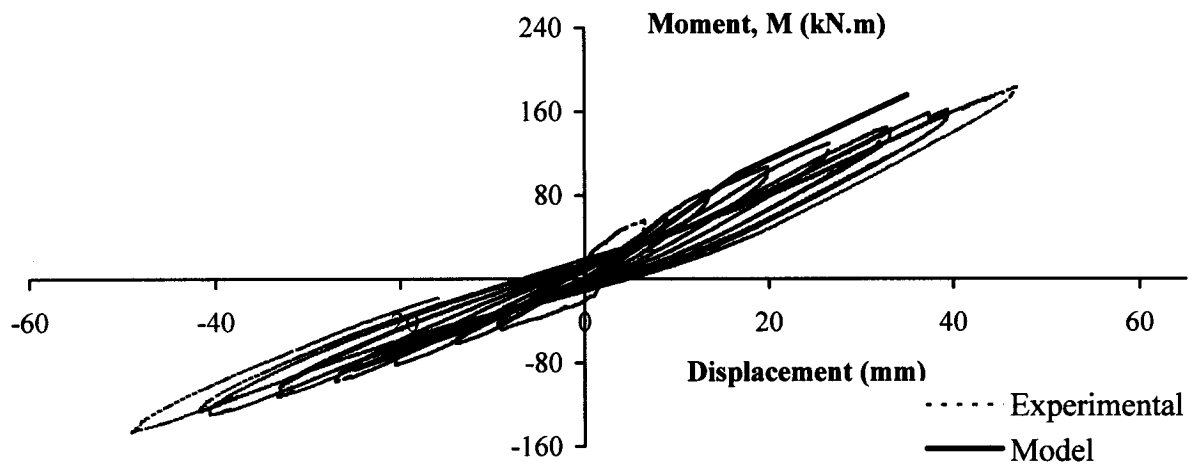


(c) Column CFCL2

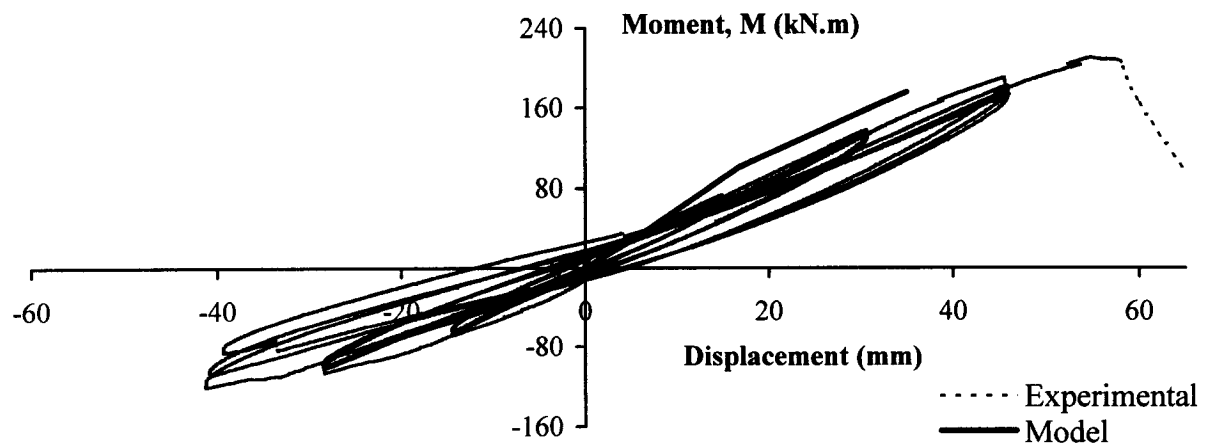
Figure 6.20 (Cont.): Comparison of Proposed Hysteretic Model and Experimental Loops of Columns



(a) General Primary Curve



(b) Beam CFB4



(b) Beam CFB5

Figure 6.21: Comparison of Proposed Hysteretic Model and Experimental Loops of Beams

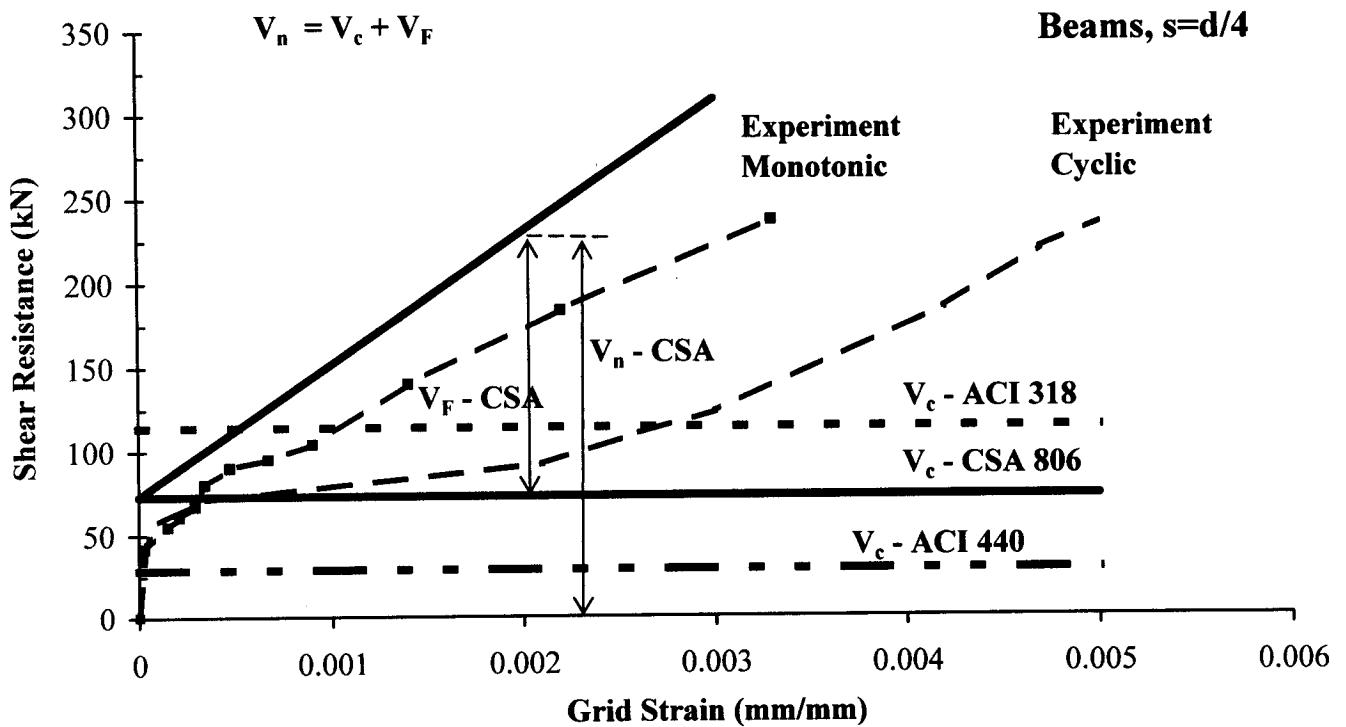
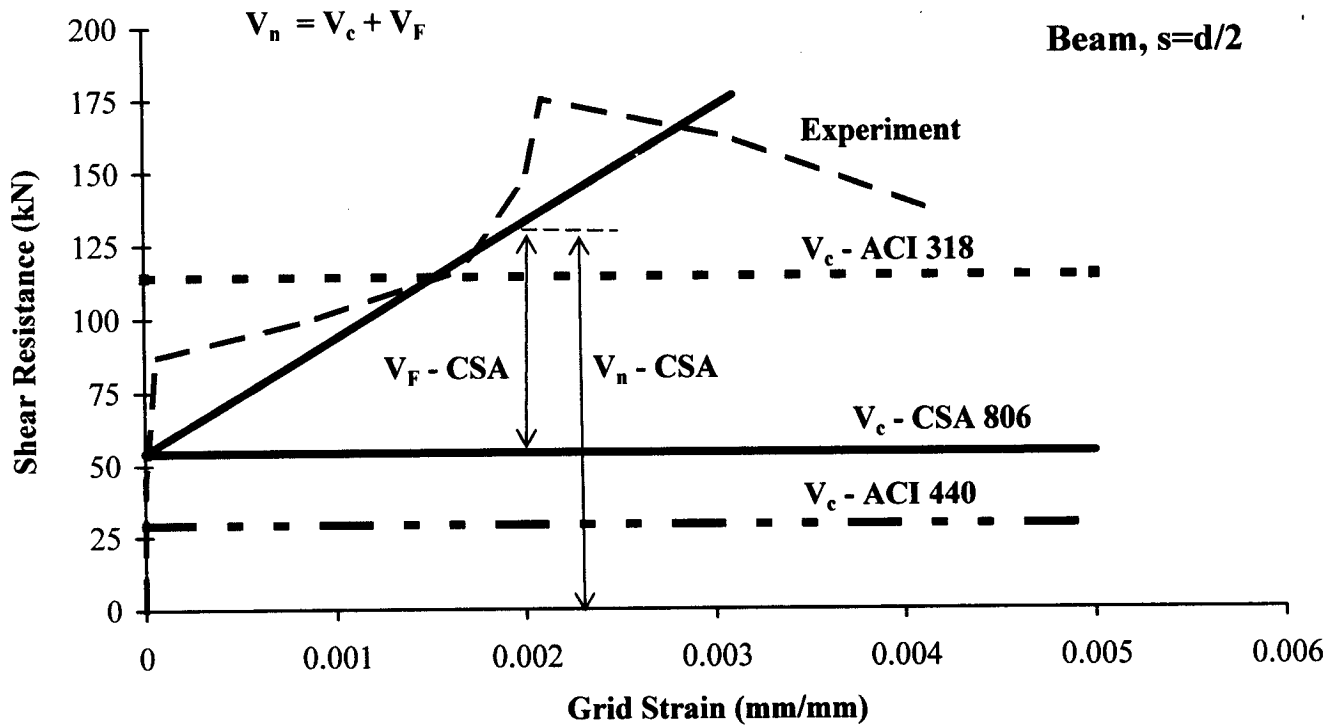


Figure 6.22: Variation of Shear Resistance versus Grid Strain to Show Concrete Shear Resistance

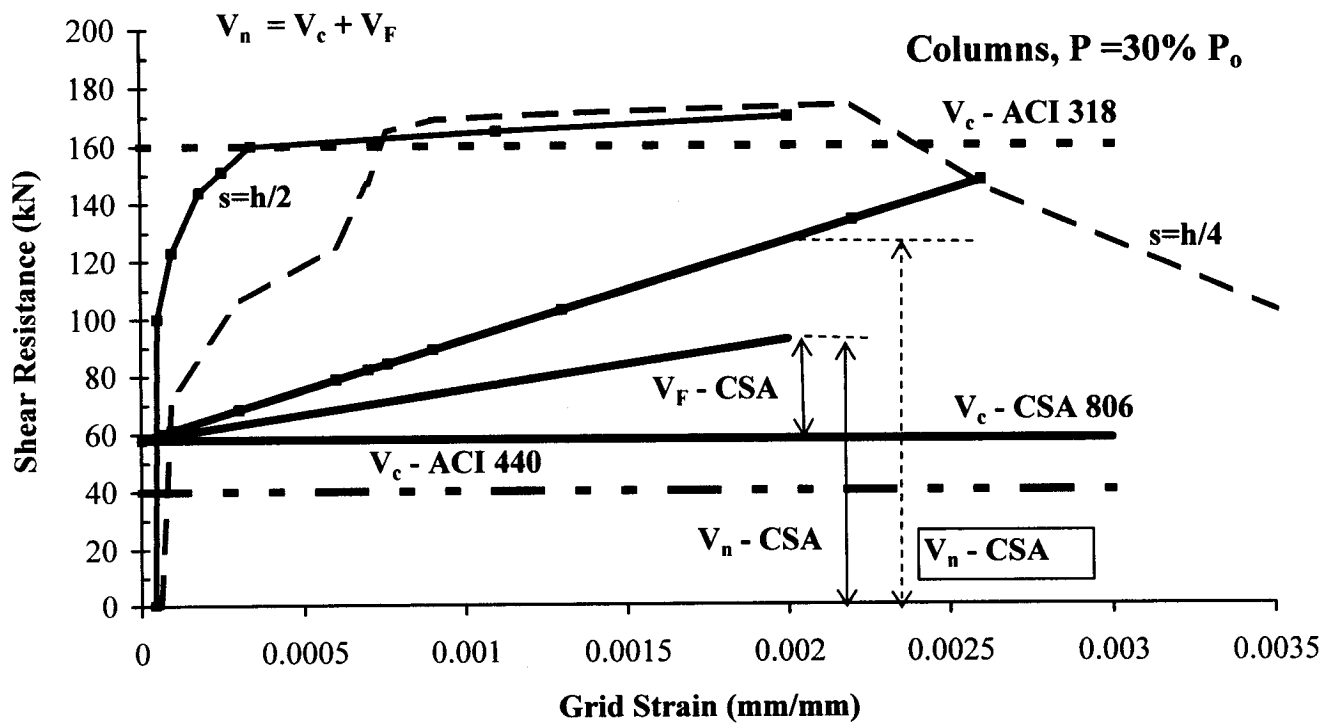
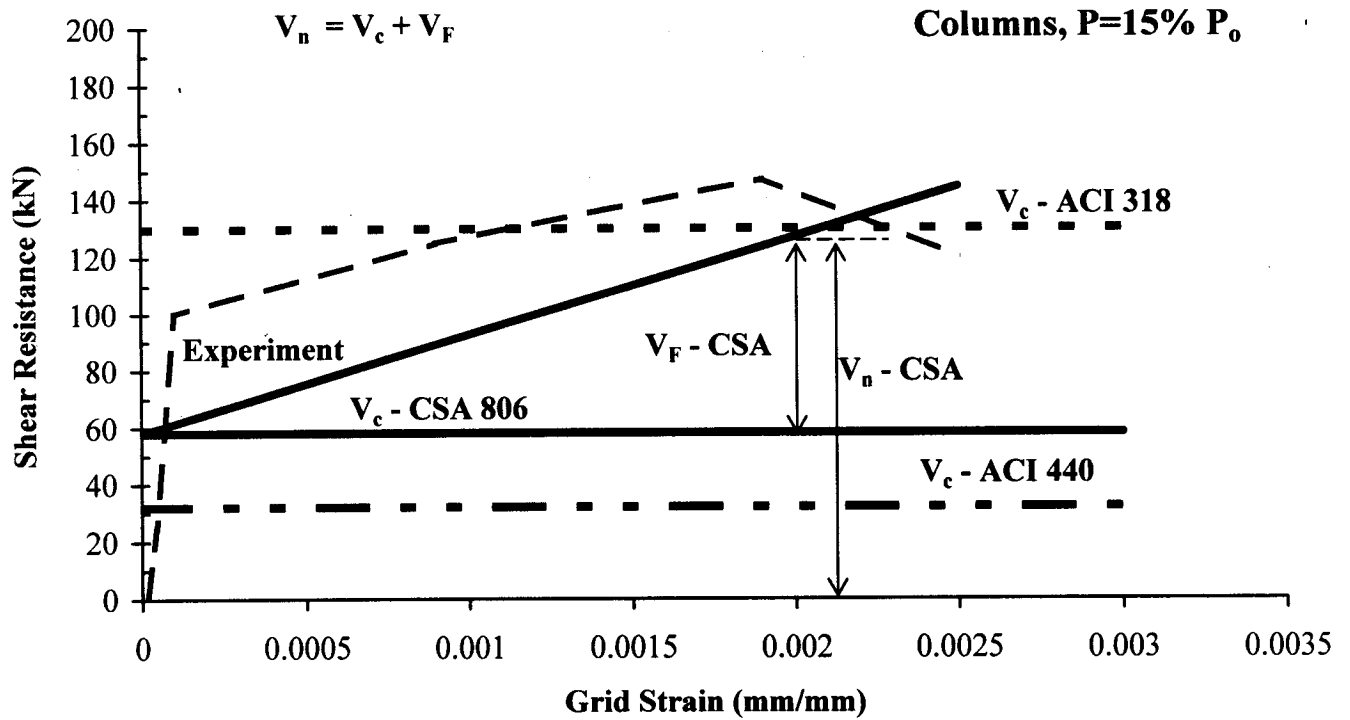


Figure 6.22(Cont): Variation of Shear Resistance versus Grid Strain to Show Concrete Shear Resistance

## ***Chapter 7***

# **Conclusions and Recommendations**

## **7.1 Conclusions**

The following conclusions can be drawn from the experimental and analytical investigation conducted in this research program:

- CFRP reinforced columns under combined axial and flexural stresses develop strengths that can be computed with plane section analysis similarly employed for steel reinforced concrete elements. Columns tested under monotonically increasing eccentric loading as well as columns and beams tested under reversed cyclic loading were able to develop their expected moment capacities.
- Compression dominant FRP reinforced concrete elements can develop their flexural capacities as governed by the crushing of concrete, prior to bar failure in compression. CFRP reinforcement used as column compression reinforcement maintained its integrity and load resistance until after the surrounding concrete crushed and spalled off at peak moment resistance. When the core concrete was well confined, the buckling of compression reinforcement preceded the crushing of confined core. Compressive strains of bars at peak moment resistance of eccentrically loaded columns (LeadLine bars) varied between 0.33% and 0.4%, though a few unreliable strain reading (due to sudden drop of load) showed as high as approximately 0.6% compressive strain after the crushing of concrete. Longitudinal bars (Pultrall bars) suffered buckling under reversed cyclic loading at about 0.39% to 0.54% compressive strains depending on the level of concrete confinement in the core.

- The stability failure of FRP bars in compression was different than that observed in steel reinforcing bars. Unlike steel bars, the FRP bars were not able to deform plastically, developing significant bending before they became unstable. Instead, no sign of bending was observed and the failure of CFRP bars in compression was in the form of breaking the bars into two under bending stresses, as if the bar was cut. The failure section resembled a clean cut by a cutter, rather than delamination or rupturing of fibers. This can be explained by the lack of resistance of FRP bars to bending. No sign of compression crushing of FRP bars was observed inside the concrete, though this was attained in compression coupon tests.
- The stress-strain relationship of the CFRP reinforcement used is perfectly linear, without any inelastic deformability. Tension and compression coupon tests indicated that the bars developed approximately the same level of strain at failure. However, the modulus of elasticity in compression varied between 20% and 30% of the value in tension, with a similar ratio in tensile and compressive strengths.
- The bar strain values at failure indicates full compatibility with surrounding concrete until the lateral restraint provided by concrete is lost due to the spalling of cover concrete and the crushing of core concrete. This implies that the FRP bars were able to provide resistance in compression until after the surrounding concrete crushed. The results of reinforced cylinders indicate that the average concrete strain and the strain measured on CFRP bars show similar behaviour up to the peak load resistance. The bars maintained their integrity until after the failure of cylinders was observed, without any sign of damage or delamination of carbon fibers, implying that CFRP bars can be used as compression reinforcement until the crushing of surrounding concrete.
- Moment-curvature relationships can be established for FRP reinforced concrete sections in much the same manner as for steel-reinforced concrete sections. The analysis, conducted until bar buckling, showed very good agreement with experimentally recorded average moment-curvature relationships, confirming that plane section analysis is applicable to FRP reinforced concrete members with linear stress-strain relationship of FRP bars.



- Two design approaches may be used in designing FRP reinforced concrete structures. One is to allow sufficiently high safety margin in design against tensile rupturing of bars and ensuring elastic behaviour during service loading and accept brittle failure at ultimate initiated either by FRP rupturing in tension or concrete crushing in compression. The other approach is to reduce the safety margin while maintaining elastic behaviour under service loads and allow concrete crushing at ultimate prior to the tensile failure of FRP, provided that the concrete is confined sufficiently to prevent brittle failure. This approach becomes attractive if the compression concrete is confined by sufficient transverse reinforcement to impose ductility to the structure. Three different flexural modes of failure were observed in the specimens tested; (i) crushing of concrete due to compression failure of concrete (ii) stability failure of FRP bars in compression, and (iii) rupturing of tension reinforcement. Columns under eccentric loadings and reinforced cylinders under concentric loading failed due to the crushing of concrete prior to any bar failure. The failure in columns under high axial compression and reversed cyclic loading was initiated by the spalling of concrete cover, followed by the buckling of FRP bars in compression and subsequent crushing of core concrete. Tension rupturing of FRP reinforcement occurred only in two columns where the level of axial compression was low. A few bars showed the tension failure, and the final column failure was attributed to the buckling of compression bars after the crushing of concrete. The type of tension failure was very similar to that observed in coupon tests, where the failure took place within approximately 150 mm segment of the bar, with fibers rupturing randomly at different locations, displaying clear delamination of ruptured fibers from the resin. The level of axial compression in most columns was sufficiently above the balanced load. Beams without axial compression were critical in flexure and they all failed by rupturing of tension reinforcement.
- Tension dominant FRP reinforced concrete members can be analyzed similar to the approaches used for steel-reinforced members. The longitudinal CFRP bars were able to develop high tensile strains required to maintain sectional equilibrium under increased bending as the concrete on the compression side experienced gradual crushing. The beams tested in the current investigation were designed to have 0.39%

and 0.26% tension reinforcement ratios in strong and weak directions, respectively. The balanced section reinforcement ratio, based on experimentally measured rupturing strength of 1450 MPa and maximum concrete compressive fiber strains of 0.0035 was 0.39%, exactly equal to the ratio in the strong direction. Therefore, it was expected that the tension reinforcement would rupture prior to significant distress in concrete. Except for CFB1, which was critical in shear, all other beams were critical in flexure and they all failed by rupturing of tension reinforcement at approximately 1.3% strain. All the flexure dominant beams developed their flexural capacities computed on the basis of plane-section analysis.

- Interaction diagrams for FRP reinforced concrete columns are different than those for steel reinforced concrete columns. FRP reinforced columns may fail by crushing of concrete even under a very low or zero axial compression. Computation of interaction diagrams should be carried out with caution, with the correct limiting strains incorporated for material failure. Comparison of experimental strength values with interaction diagrams indicated that the actual strength values were consistently outside the computed nominal strengths (without the effects of concrete confinement) and often very close to the computed diagrams when the confinement of concrete was considered. These comparisons confirm the validity of the plane section analysis once again for flexural members under axial compression.
- CFRP reinforced concrete elements can develop limited inelastic deformability and ductility. Comparison of ductility ratios of CFRP reinforced columns under eccentric loading with those of steel reinforced columns with 0.25h to 0.30h end eccentricity indicated that CFRP reinforced columns were able to sustain approximately 2/3 the ductility ratios observed in companion steel reinforced columns. FRP reinforced columns subjected to reversed cyclic loading had displacement ductility ratios ranging between 2.7 and 3.9 depending on the level of axial compression, degree of confinement and stability of compression bars as governed by transverse grid spacing.
- FRP reinforced columns develop limited drift capacities. The columns tested in the current investigation sustained lateral drift ratios that were slightly higher than the 2% and 2.5% limits specified for earthquake resistant columns in the National Building Code of Canada (1995, 2005). The hysteretic relationships indicated progressive

stiffness degradation due to concrete cracking, and inelasticity due to gradual concrete crushing. However, the shape of hysteresis loops was distinctly different than those of steel reinforced columns. The unloading branches of hysteresis loops for FRP reinforced concrete members aim towards the origin, due to the elastic behaviour of FRP bars in tension.

- Earthquake resistant design of FRP reinforced concrete structures must be based on elastic member behaviour while taking advantage of relative flexibility of FRP and FRP reinforced concrete and associated elongation in the fundamental period of structures, attracting lower spectral accelerations.
- The force-deformation envelope for FRP reinforced beams under reversed cyclic loading is approximately the same as that obtained under monotonically increasing lateral force. This implies that a monotonic curve would provide sufficient strength envelope for hysteretic relationships of FRP reinforced members.
- The hysteretic model developed as part of the current investigation provides good predictions of observed response under reversed cyclic loading. The proposed model, consisting of primary moment-flexural displacement relationship and hysteretic rules, provides good agreement with experimentally obtained hysteresis loops as well as the overall response.
- CFRP grids were able to fulfill their functions as transverse reinforcement in terms of providing lateral support to compression bars and confining the compression concrete. The examination of the confinement parameters indicated that the expected trends, as observed in steel reinforced columns, were also observed in CFRP reinforced columns, i.e., the deformability increased with reduced tie (grid) spacing, increased number of laterally supported longitudinal bars and increased end eccentricity.
- The ACI 318 requirements for concrete shear resistance provide good agreement with experimental values obtained in the current investigation. Although these provisions were intended for steel reinforced concrete elements, they produced better correlation with test data than any other recommendations currently available, especially when the effect of axial compression was incorporated. The provision provided in CSA

S806 provides a conservative lower bound for columns under axial compression, though it overestimates concrete shear resistance in beams.

- The shear resistance provided by transverse grids can be computed based on the expressions provided for steel reinforced concrete elements and 45-degree truss analogy.
- Deformations caused by the extension of reinforcement in adjoining members, referred to as anchorage slip, forms approximately 25% to 35% of total deformations
- Bond stress between concrete and Pultrall bars used in the current investigation was computed to be 5.5 MPa. Comparison of experimentally observed development lengths for Pultrall bars and those recommended by ACI-440 and CSA S806 indicate that the code expressions were approximately 24% and 8% more conservative, respectively.

## **7.2 Recommendations**

The current investigation consisted of a large number of tests, covering small scale material tests to large scale beams and columns under monotonic and cyclic loading. A large volume of test data was generated. However, because the field of FRP reinforced concrete columns has little previous research conducted in the area, additional tests are recommended. Specifically the following tests are recommended for future experimental research:

- FRP reinforced columns with different percentages of reinforcement.
- FRP reinforced columns under different levels of axial compression, combined with monotonic and cyclic loading.
- FRP reinforced circular and rectangular columns.
- Tests of members with different CFRP bars.
- Columns under concentric loading to establish the buckling length of bars between transverse reinforcement.
- FRP reinforced beams with higher percentage of longitudinal reinforcement and different levels of concrete confinement.

- FRP reinforced beams and columns with short shear spans to promote shear behaviour so that experimentally obtained shear capacities can be used to develop new and more accurate shear design expressions.

In addition, the following analytical research is recommended:

- Development of new empirical design expression for shear resistance of concrete in FRP concrete elements and development length of CFRP bars.
- Verification of column interaction diagrams for FRP reinforced columns with different levels of axial compression and different percentages of reinforcement.
- Incorporation of the hysteretic model developed into a computer program for dynamic inelastic analysis of FRP reinforced concrete buildings.
- Study of inelastic seismic drift demands of FRP reinforced concrete structures under NBCC compatible design response spectra.
- Development of over-strength and ductility factors for FRP reinforced concrete structures for seismic design.

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