

**PERFORMANCE OF FRP STRENGTHENED CONCRETE
COLUMNS UNDER SIMULATED BLAST LOADING**

By
Aws Hasak

A thesis submitted to
the Faculty of Graduate and Postdoctoral Studies
in partial fulfilment of the requirements
for the degree of
Master of Applied Sciences
in Civil Engineering

Department of Civil Engineering
Faculty of Engineering
University of Ottawa

© Aws Hasak, Ottawa, Canada, 2015

Abstract

In this thesis the effectiveness of CFRP laminates as externally applied column jacketing material was investigated experimentally. The research project forms part of a comprehensive research program with a scope of developing FRP protection systems for reinforced concrete columns subjected to extreme loads. The research program is carried out by a multi-disciplinary team including various institutes of the National Research Council Canada and the University of Ottawa. The experimental program of the current project consists of nine half-scale seismically designed reinforced concrete (RC) columns, tested under combined axial compression and simulated lateral blast loading, applied statically. Seven columns were strengthened by four types of CFRP laminates. The columns were first subjected to service axial loads of 45% of the design axial load capacity, followed by uniformly distributed lateral load up to failure. The columns were analyzed under incrementally increasing static loads, well into the inelastic range of deformations. The FRP was modelled as a linear material up to rupturing in tension. The behaviour of CFRP in compression was modelled with reduced strength and modulus of elasticity. It is found that the use CFRP laminates as column jacketing improves strength and deformability of the columns significantly. The use of CFRP laminate with unidirectional fibers and woven $\pm 45^\circ$ laminas results in significantly lower lateral deformation than companion unretrofitted columns. The retrofitted columns also develop well distributed strains within the critical plastic region, promoting a better redistribution of stresses. Adding woven $\pm 45^\circ$ laminas resulted in higher deformability and slightly lower strength than the laminate with only unidirectional fibers. The overall conclusion of the project is that the CFRP jackets, when

designed properly, especially with the addition of ± 45 degree fibres result in significantly improved strength and deformability of concrete columns.

Acknowledgements

It's been a long road, but here I am at the end, but there are so many people to whom thanks I extend!

First and foremost, is a well-deserved thanks to my supervisors, Dr. Husham Almansour and Dr. Murat Saatcioglu, for their continued guidance, encouragement and financial support throughout this research project. I am very proud of what we have achieved together, thank you both.

I would also like to express my sincere appreciation to my colleague graduate student, Mr. Bessam Kadhom, for his assistance in the laboratory and the many educational discussions we had on response of structures to static and dynamic loading.

Special thanks to Dr. Muslim Majeed, Dr. Amina Mohammed (University of Ottawa), Omran Maadani, Jim Margeson and Roger Smith (National Research Council), for their valuable expertise and attentive work during laboratory testing and analysis.

My sincere gratitude and appreciation are directed to the National Research Council Canada and University of Ottawa for the technical and academic support.

Last, but not least, I must also thank my family for the support they provided me through my entire life and in particular, I must acknowledge my wife and best friend, Gufran, without whose love, support, encouragement and editing assistance, I would not have finished this thesis. Thanks for your understandings and for giving me so much freedom to pursue my little research dream that now is finally complete.

Table of Contents

Abstract.....	ii
Acknowledgements	iv
Table of Contents	v
List of Figures	vii
List of Tables	xi
Notations.....	xii
Chapter 1: Introduction.....	1
1.1. General.....	1
1.2. Objectives.....	3
1.3. Scope.....	4
1.4. Thesis outline	4
Chapter 2: Literature Review	6
2.1. General.....	6
2.2. Previous Research on FRP Retrofit of Concrete Elements under Axial Compression and Flexure.....	6
2.2.1. FRP Jacketing of Columns for Concrete Confinement	7
2.2.2. FRP Strengthened beams in flexure	10
2.2.3. Behaviour of Fibre Orientation on Column Behaviour	11
2.3. Previous research on blast performance of FRP retrofitted columns.....	13
2.4. CSA Design Standards and the Canadian Design Approach.....	19
Chapter 3: Experimental Program	21
3.1. Introduction	21
3.2. Material Properties.....	21
3.2.1. Concrete.....	21
3.2.2. Steel.....	23
3.2.3. Carbon Fiber Reinforced polymer (CFRP).....	23
3.3. Column Specimens	33
3.4. Construction of Specimens.....	35
3.4.1. Reinforcement Cages	35
3.4.2. Strain Gauges.....	36
3.4.3. Casting and Curing of Concrete.....	36
3.4.4. Grinding the Surface of Specimens.....	38
3.4.5. CFRP strengthening of the column specimens	38
3.5. Measurements of Axial and Lateral Displacement	40

Chapter 4: Experimental Results.....	41
4.1. Introduction	41
4.2. Column Specimens and Loading Method	41
4.3. Column Group I: Control RC Column-1 (Column S5).....	44
4.4. Column Group I: Control Column-2 (Column S6).....	49
4.5. Column Group II: Column strengthened with CFRP-L1 (Column S1)	53
4.6. Column Group III: Column strengthened with CFRP-L2 (Column S2)	54
4.7. Column Group III: Column strengthened with CFRP-L2 (Column S4)	62
4.8. Column Group IV: Column Strengthened with CFRP-L3 (Column S7)	67
4.9. Column Group IV: Column Strengthened with CFRP-L3 (Column S8)	68
4.10. Column Group V: Column Strengthened with CFRP-L4 (Column S3).....	77
4.11. Column Group V: Column strengthened with CFRP-L4 (Column S9)	78
Chapter 5: Effects of Test Parameters and Analysis of Columns.....	86
5.1. Introduction	86
5.2. Comparisons of Columns within Each Group and Relative to Control Columns.....	87
5.3. Effects of CFRP Laminate Type on Column Performance	93
5.4. Analysis of Columns and Comparison with Experimental Data	94
5.4.1. General	94
5.4.2. Stress-strain models for concrete and reinforcing steel	96
5.4.3. Nonlinear sectional analysis procedure	99
5.4.4. Nonlinear member analysis procedure	100
5.4.5. Comparisons with experimental data	103
Chapter 6: Summary and Conclusions.....	112
6.1. Summary	112
6.2. Conclusions	113
6.3. Future Work	115
References.....	116

List of Figures

Figure 2. 1: External confinement with FRP (Rahai, et al., 2008)	8
Figure 2.2: Test building for field testing and locations of DB 6 and DB 8 columns	14
Figure 2.3: DB 6 and DB 8 columns after field test	14
Figure 2.4: Lateral load system used in full-scale laboratory tests (Crawford et al. 2001)	15
Figure 2.5: Full scale CFRP RC column tested by lateral loading system	15
Figure 2.6: Comparison between DB 6 test and laboratory test.....	16
Figure 2.7: Comparison of lateral response of columns with and without two-layers of CFRP hoops (Crawford et al. 2001)	17
Figure 2.8: Comparison of lateral response of columns with and without six-layers of CFRP hoops (Crawford et al. 2001)	17
Figure 3. 1. Typical stress-strain relationship for concrete.....	22
Figure 3. 2. Stress-strain curve for transverse reinforcement and material properties	24
Figure 3. 3. Stress-strain curve for longitudinal rebar and material properties.....	24
Figure 3. 4. A CFRP laminated sheet with 2 sets of 5 specimens coupons cut in 0° and 90° direction.....	25
Figure 3. 5. CFRP laminated sheet from which five coupons were cut.....	26
Figure 3. 6. A sample coupon	26
Figure 3. 7. MTS Tensile Load Frame and a test coupon with extensometer installed	26
Figure 3. 8. Stress-strain relationships for CFRP-L1 in $X_{Laminate}$ and in $Y_{Laminate}$ directions	27
Figure 3. 9. Stress-strain relationship for CFRP-L2.....	28
Figure 3. 10. Stress-strain relationship for CFRP-L3.....	29
Figure 3. 11. Stress-strain relationship for CFRP-L4.....	30
Figure 3. 12. Stress-strain relationships for different tested CFRP laminates at $X_{Laminate}$ direction (a) full strain range; (b) strain range up to ultimate strains of the four laminates.....	32
Figure 3. 13. Reinforcement details for all column specimens	34
Figure 3. 14. Different fiber orientations relative to column axis	35
Figure 3. 15. Stirrup bending jig, lever arm, and a completed stirrup.....	36
Figure 3. 16. Locations of strain gauges on longitudinal reinforcement	37
Figure 3. 17. Locations of strain gauges on the surface of column	37
Figure 3. 18. Protected strain gauge assembly and taped wires to the rebar.....	37
Figure 3. 19. Using of curbing trowel to smooth the top edges of all column specimens	38
Figure 3. 20. FRP is impregnated in resin and the air is rolled out from the saturated fabric.....	39
Figure 3. 21. Wrapping the stripped ends of a specimen.....	40
Figure 3. 22. Locations of the displacement sensors (LVDTs).....	40
Figure 4. 1. Column Subjected To Axial Load and Uniformly Distributed Lateral Load	43
Figure 4. 2. Effect of the loading stages on the column: (a) axial load cause camber; (b) initial lateral load required to bring the column back to state of zero lateral deformation.	44
Figure 4. 3. Column S5 prior to testing	46
Figure 4. 4. Damage to Column S5 near the roller support	46
Figure 4. 5. Damage to Column S5 near the hinge support	46
Figure 4. 6. Damage on side face of Column S5 at maximum load.....	46
Figure 4. 7. Damage to Column S5 at maximum load	46
Figure 4. 8. Variation of lateral load and axial load with mid-span deflection (Column S5)	47
Figure 4. 9. Deflection profiles at selected stages of loading (Column S5).....	47
Figure 4. 10. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S5).....	48
Figure 4. 11. Strains in stirrups and top concrete surface at mid-span (Column S5)	48
Figure 4. 12. Column S6 prior to testing	50
Figure 4. 13. Damage to Column S6 near the roller support	50

Figure 4. 14. Damage to Column S6 near the roller support	50
Figure 4. 15. Damage to Column S6 near the hinge support	50
Figure 4. 16. Damage on side face of Column S6 at maximum load	50
Figure 4. 17. Damage to Column S6 at maximum load.....	50
Figure 4. 18. Variation of lateral load and axial load with mid-span deflection(Column S6)	51
Figure 4. 19. Deflection profiles at selected stages of loading (Column S6).....	51
Figure 4. 20. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S6).....	52
Figure 4. 21. Strains in stirrups and top concrete surface at mid-span (Column S6)	52
Figure 4. 22. Column S1 prior to testing	55
Figure 4. 23. Damage to Column S1 near the roller support	55
Figure 4. 24. Damage to Column S1 near the hinge support	55
Figure 4. 25. Damage on side face of Column S1 at maximum load	55
Figure 4. 26. Damage to Column S1 at maximum load.....	55
Figure 4. 27. Variation of lateral load and axial load with mid-span deflection(Column S1)	56
Figure 4. 28. Deflection profiles at selected stages of loading (Column S1).....	56
Figure 4. 29. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S1).....	57
Figure 4. 30. Strains in stirrups and top concrete surface at mid-span (Column S1)	57
Figure 4. 31. Column S2 prior to testing	59
Figure 4. 32. Damage to Column S2 near the roller support	59
Figure 4. 33. Damage to Column S2 near the hinge support	59
Figure 4. 34. Damage on side face of Column S2 at maximum load	59
Figure 4. 35. Damage to Column S2 at maximum load.....	59
Figure 4. 36. Variation of lateral load and axial load with mid-span deflection (Column S2)	60
Figure 4. 37. Deflection profiles at selected stages of loading (Column S2).....	60
Figure 4. 38. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S2).....	61
Figure 4. 39. Strains in stirrups and top concrete surface at mid-span (Column S2)	61
Figure 4. 40. Column S4 prior to testing	64
Figure 4. 41. Damage to Column S4 near the roller support	64
Figure 4. 42. Damage to Column S4 near the hinge support	64
Figure 4. 43. Damage on side face of Column S4 at maximum load	64
Figure 4. 44. Damage to Column S4 at maximum load.....	64
Figure 4. 45. Variation of lateral load and axial load with mid-span deflection (Column S4)	65
Figure 4. 46. Deflection profiles at selected stages of loading (Column S4).....	65
Figure 4. 47. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S4).....	66
Figure 4. 48. Strains in stirrups and top concrete surface at mid-span (Column S4)	66
Figure 4. 49. Column S7 prior to testing	70
Figure 4. 50. Damage to Column S7 near the roller support	70
Figure 4. 51. Damage to Column S6 near the hinge support	70
Figure 4. 52. Damage on side face of Column S7 at maximum load	70
Figure 4. 53. Damage to Column S7 at maximum load.....	70
Figure 4. 54. Variation of lateral load and axial load with mid-span deflection (Column S7)	71
Figure 4. 55. Deflection profiles at selected stages of loading (Column S7).....	71
Figure 4. 56. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S7).....	72
Figure 4. 57. Strains in stirrups and top concrete surface at mid-span (Column S7)	72
Figure 4. 58. Column S8 prior to testing	74
Figure 4. 59. Damage to Column S8 near the roller support	74

Figure 4. 60. Damage to Column S8 near the hinge support	74
Figure 4. 61. Damage on side face of Column S8 at maximum load	74
Figure 4. 62. Damage to Column S8 at maximum load.....	74
Figure 4. 63. local buckling of the CFRP with three bumps on the top surface of S8	74
Figure 4. 64. Variation of lateral load and axial load with mid-span deflection (Column S8)	75
Figure 4. 65. Deflection profiles at selected stages of loading (Column S8).....	75
Figure 4. 66. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S8).....	76
Figure 4. 67. Strains in stirrups and top concrete surface at mid-span (Column S8)	76
Figure 4. 68. Column S3 prior to testing	79
Figure 4. 69. Damage to Column S3 near the roller support	79
Figure 4. 70. Damage to Column S3 near the hinge support	79
Figure 4. 71. Damage on side face of Column S3 at maximum load	79
Figure 4. 72. Damage to Column S3 at maximum load.....	79
Figure 4. 73. Variation of lateral load and axial load with mid-span deflection (Column S3)	80
Figure 4. 74. Deflection profiles at selected stages of loading (Column S3).....	80
Figure 4. 75. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S3).....	81
Figure 4. 76. Strains in stirrups and top concrete surface at mid-span (Column S3)	81
Figure 4. 77. Column S9 prior to testing	83
Figure 4. 78. Damage to Column S9 near the roller support	83
Figure 4. 79. Damage to Column S9 near the hinge support	83
Figure 4. 80. Damage on side face of Column S9 at maximum load	83
Figure 4. 81. Damage on back side of Column S9 at maximum load	83
Figure 4. 82. Damage to Column S9 at maximum load.....	83
Figure 4. 83. Variation of lateral load and axial load with mid-span deflection (Column S9)	84
Figure 4. 84. Deflection profiles at selected stages of loading (Column S9).....	84
Figure 4. 85. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S9).....	85
Figure 4. 86. Strains in stirrups and top concrete surface at mid-span (Column S9)	85
Figure 5. 1. Applied load and lateral reactions vs mid-span deflection for Column Group CG-I (control columns S5 and S6).....	91
Figure 5. 2. Applied load and lateral reactions vs mid-span deflection for Column Group CG-II (Column S1)	91
Figure 5. 3. Applied load and lateral reactions vs mid-span deflection for Column Group CG-III (Columns S2 and S4).....	92
Figure 5. 4. Applied load and lateral reactions vs mid-span deflection for Column Group CG-IV (Columns S7 and S8).....	92
Figure 5. 5. Applied load and lateral reactions vs mid-span deflection for Column Group CG-V (Columns S3 and S9).....	93
Figure 5. 6. Comparison of the lateral load versus lateral deflection relationships of selected columns from different column groups	95
Figure 5. 7. Comparison of deflection profiles over column unsupported length for selected columns in different column groups.....	95
Figure 5. 8. Stress-strain relationship for unconfined and confined concrete	101
Figure 5. 9. Tensile stress-average tensile strain relation (after Gilbert and Warner 1978).....	101
Figure 5. 10. Stress-strain relationship for reinforcing steel (after Yalcin and Saatcioglu 2000)	102

Figure 5. 11. External forces on the section and possible strain distributions: (a) cross section;(b) forces on the section;(c) strain distribution due to flexure: case(i);(d)strain distribution due to axial load & flexure: case(ii);(e)strain distribution due to axial load &flexure: case(iii-1); (f)strain distribution due to axial load &flexure: case(iii-2); (g)strain distribution due to axial load &flexure: case(iii-3)	102
Figure 5. 12. Numerical integration of nonlinear stresses over the cross section: (a) cross section divided into strips; (b) strain distribution due to axial load & flexure: ϵ_{ci} concrete strain, ϵ_{sti} steel strain; (c) stress distribution in concrete & steel: f_{cci} concrete stress at compressive level, f_{cti} concrete stress at tensile level, and f_{sti} steel stress; (d) internal forces in concrete & steel: F_c summation of internal forces of concrete(F_{cc}) and steel (F_{sc})in compressive level, F_t summation of internal forces of concrete (F_{ct}) and steel (F_{st})in tension level	103
Figure 5. 13. Three different alternatives for the contribution of the CFRP laminate in the nonlinear sectional analysis.....	103
Figure 5. 14. Comparisons of analytically generated moment-curvature relationships computed using RC-Blast software with experimentally recorded flexural capacities.....	106
Figure 5. 15. Comparisons of analytically generated moment-curvature relationships computed using Mohammed's software with experimentally recorded flexural capacities	107
Figure 5. 16. Comparisons of analytically generated force-displacement relationships computed using RC-Blast software with experimentally recorded data	108
Figure 5. 17. Comparisons of analytically generated force-displacement relationships computed using Mohammed's software with test results	109
Figure 5. 18. Comparison of the experimental with the analytical results of the total lateral UDL versus the deflection for Columns S1 and S5	110
Figure 5. 19. Comparison of the experimental with the analytical results of the total lateral UDL versus the deflection for Columns S2 and S5	110
Figure 5. 20. Comparison of the experimental with the analytical results of the total lateral UDL versus the deflection for Columns S7 and S5	111
Figure 5. 21. Comparison of the experimental with the analytical results of the total lateral UDL versus the deflection for Columns S9 and S5	111

List of Tables

Table 3. 1: Concrete properties.	22
Table 3. 2: Mechanical Properties of Reinforcing Steel.	25
Table 3. 3: Laminate type and properties.	25
Table 3. 4: Column specimens groups according.	34
Table 4. 1: Columns and their CFRP Laminate protection and column group.	41

Notations

<u>Acronyms</u>	<u>Definition</u>
ASTM	- American Society for Testing and Materials / American Standard Test Method
C	- Strain gauge on the compressive longitudinal steel reinforcement
CFRP	- Carbon fibre reinforced polymer
FRP	- Fibre reinforced polymer
GFRP	- Glass fibre reinforced polymer
HSC	- High strength concrete
HSS	- High strength steel
LVDT	- Linear variable differential transformer
MOR	- Modulus of rupture
NRC	- National Research Council
NSC	- Normal strength concrete
NSS	- Normal strength steel
RC	- Reinforced Concrete
SAP	- Super absorbent polymer
T	- Strain gauge on the tensile longitudinal steel reinforcement
UD	- unidirectional
UDL	- lateral uniformly distributed load
W	- Woven

Symbols

ρ	- Longitudinal reinforcement ratio
d	- Effective depth
D	- Diagonal of the cross-section for a rectangle or a square shape
E	- Modulus of elasticity
E_F	- Modulus of elasticity of FRP
f'_{co}	- Strength of unconfined concrete
f'_c	- Peak compressive strength

f_c	- Compressive strength
f_{cc}	- Compressive strength in confined concrete
f_{Fj}	- Stress in FRP jacket
f_{FRP}	- FRP circumferential stress
f_{Fu}	- Ultimate rupturing strength of FRP
f_{sh}	- Hardening stress
f_u	- Ultimate steel stress
f_y	- Yield steel stress
h	- Specimen cross-section height
k	- Stiffness
k_c	- Confinement coefficient
L	- Length of specimen
$\emptyset$	- Resistance factor
$\emptyset_{FRP}$	- Material strength reduction factor for FRP
P	- Axial load
P_0	- Initial axial load
P_{max}	- Peak load
t_j	- FRP jacket thickness
w/c	- Water to cement ratio in concrete
$X_{Laminate}$	- Laminate in X direction
$Y_{Laminate}$	- Laminate in Y direction
δ	- Design lateral drift ratio
ε	- Strain
ε'_c	- Strain at peak compressive strength
ε'_{o1}	- Strain at peak unconfined strength
ε'_{o85}	- 85% of the peak unconfined strength
ε'_{sh}	- Hardening strain
ε'_u	- Peak concrete strain
ε'_y	- Yield concrete strain
ρ_s	- Steel reinforcement ratio

Chapter 1: Introduction

1.1. General

The problem of the exposure of a reinforced concrete column to an extremely high lateral uniformly distributed load while it is under relatively high service axial load is of interest from the fundamental structural mechanics and the protection design points of views. Most of previous research addressed the problem of the combined loading on beam-columns as a result of the axial load eccentricity in continuous structural frames. In such loading cases, the bending moment(s) is always developed from the axial load eccentricity. However, in this study, the axial load has a constant value when the lateral load is applied, then the lateral load is increased successively up to failure.

When a building with reinforced concrete columns is subjected to a far field blast pressure, the columns can be under any level of axial load when a very high and sudden uniformly distributed pressure is applied over the entire columns heights. Assuming the axial load as a constant is based on the large difference between the lateral acceleration of up to 500 g due to the application of the lateral load (blast pressure) and the gravitational acceleration (1 g) in the vertical direction of the columns. Hence, evaluating the structural behavior of the column under two loadings is important to design an efficient protection system and/or to evaluate the strength and residual capacity of the columns, which are considered as the most critical elements to the stability of the structural system.

Fiber reinforced polymer (FRP) is widely used for the strengthening of reinforced concrete columns. The typical application usually focuses on: (i) enhancing the column axial load

capacity while improving column deformability through the confinement of concrete in the entire section (cover and core); (ii) enhancing the shear capacity of columns, specifically those columns that are subjected to seismic loads; and (iii) enhancing the flexural capacity. For the resistance capacity enhancements (i) and (ii), carbon or glass fibers are applied on the column external surface where they are oriented parallel or perpendicular (transverse) to the column axis, though longitudinal fibres also contribute to the mechanism of shear resistance. For the resistance capacity enhancement (iii) the fibers are oriented parallel to the column axis on the column surface where higher tensile stresses are expected to be developed.

When FRP is applied by laying-up of several thin layers (laminas), where continuous fibers in each layer is oriented differently from other layers, a sheet with orthotropic mechanical properties called “laminate” is formed. The inter-laminar shear plays significant role in the FRP mechanical properties (Teng, et al (2002)), structural response, and developing the failure mechanisms. It is also the key element to identify the laminate contribution to the structural system formed from the strengthened RC element and the strengthening laminate plate, laminate shell or laminate tube. When the laminate is of a very simple design, as in the case of only two laminas (layers), one with 0° fiber orientation and the other with 90° fiber orientation and each lamina has a large thickness, then the inter-laminar shear can be ignored, and the two laminas can be considered mechanically independent. On the other hand when, several laminas are formed from continuous uni-directional fibers or woven, braided, or stitched in different directions, then the mechanical properties become more complicated. However, the properties of the laminate can be designed and optimized (or tailored) based on the required properties, cost, and service conditions.

To address the existing lack of understanding regarding blast and impact response of reinforced concrete members and the need for retrofit techniques to address deficiencies in existing structures, a multi-disciplinary team including various institutes of the National Research Council and the University of Ottawa has initiated work towards developing a fibre reinforced polymer composite protection system for RC columns subjected to extreme shocks. This thesis will focus on exploring the best structural performance of strengthened RC column using simple laminate design when the protected reinforced concrete columns are subjected to severe uniformly distributed lateral loads. The Carbon FRP (CFRP) laminate is used as a jacket covering the entire column surface over its entire height using continuous hand and wet lay-up approach.

1.2. Objectives

The primary objective of this research project is to investigate the effectiveness of carbon fibre reinforced polymer (CFRP) laminates with different fibre orientations as column retrofit material for resisting blast pressures. The objective also includes generation of experimental data under equivalent static loads for comparison with similar columns tested under high strain rates in the companion test program, and the verification of analyses techniques commonly used for column analysis in the inelastic range of deformations.

1.3. Scope

The scope of this research program consists of the follows items:

1. Review of previous research into the use of FRP jackets as column retrofit material in R/C structures;
2. Design and build nine half scale reinforced concrete columns as part of the experimental program;
3. Application of four different CFRP laminates on seven columns, keeping two columns as control specimens.
4. Column tests using the two-dimensional column loading system at the National Research Council (NRC) of Canada by subjecting the columns to uniformly-distributed and monotonically increasing lateral loads while the columns are also subjected to initially constant levels of axial compression.
5. Data collection, evaluation and presentation.
6. Assessment of the effects of selected experimental parameters through a comparative study between different columns.
7. Conduct non-linear sectional and member analyses and compare the analyses results with test data.
8. Present the results while also identifying possible future work.

1.4. Thesis outline

This thesis is divided into six chapters. Chapter 1 provides introduction to the research project and outlines the objectives and the scope. Chapter 2 presents review of previous research on behaviour of FRP retrofitted columns; Chapter 3 gives the details of the experimental program and consists of discussions on the construction of specimens, material properties, and

instrumentation. Chapter 4 presents the data recorded during testing. Chapter 5 discusses the experimental results obtained from the tests, while also presenting analytical research and comparisons of experimental and analytical force-deformation relationships. Chapter 6 provides conclusions and suggestions for future research.

Chapter 2: Literature Review

2.1. General

The use of fibre reinforced polymers (FRP) as a structural retrofit material for civil engineering infrastructure has expanded significantly during the last two decades. The previous research on FRP retrofit focused primarily on two aspects; ductility enhancement through concrete confinement and strength enhancement. The majority of previous efforts concentrated on rehabilitation of structural components that suffered from corrosion-related performance problems, strengthening of structurally deficient elements under static loading, and improving response to seismic excitations in the inelastic range of deformations. Seismic research dominated the field as FRP wrapping of reinforced concrete columns with fibres oriented in column transverse direction resulted in substantial improvements in concrete confinement, increasing ductility and energy dissipation capacity of otherwise brittle and substandard columns. The benefits of improved ductility under strong earthquakes have been recognized in the structural design community in Canada and elsewhere in the world. While extensive previous research on the use of FRPs for seismic retrofit applications increased the comfort level in practice for this type of extreme dynamic loading, research on performance of FRP retrofitted concrete columns under blast loading has been limited in scope. This Chapter provides an overview of strength and ductility improvements attained in FRP retrofitted concrete columns, followed by review of previous research on blast performance of FRP retrofitted columns. An overview of the Canadian practice on FRP retrofit applications is also provided with references to the available Canadian Standards Association (CSA) documents.

2.2. Previous Research on FRP Retrofit of Concrete Elements under Axial Compression and Flexure

Previous research on FRP retrofit of surface bonded and/or wrapped FRP sheets on structurally deficient reinforced concrete elements can be examined under two different categories; FRP wrapped columns for concrete confinement with fibres oriented in column transverse direction, and FRP strengthened beams with surface-bonded and/or anchored FRP sheets with fibres oriented in the longitudinal direction. This section provides an overview of previous research in

these two areas with emphasis on seismic performance. An additional subsection is provided to present the existing literature on FRP fibre orientation, because the primary objective of the current study is to investigate the performance of concrete columns retrofitted with FRP sheets having different fibre orientations.

2.2.1. FRP Jacketing of Columns for Concrete Confinement

Transverse reinforcement, including FRP fibres, improves concrete confinement. This in turn results in improved stress-strain relationship of compression concrete with increased strength and ductility. The fundamental principle of confinement is to impose a limitation on the lateral expansion of concrete and corresponding crack widening under axial compression. Properly confined concrete can sustain axial stresses and strains that are higher than those for unconfined concrete, improving strength and ductility of structural members (Demers and Neale, 1999).

Circular cross-sections benefit the most from lateral confining pressure because of hoop tension that develops in transverse reinforcement. Uniform hoop tension results in uniform confining pressure as illustrated in Figure 2.1a. The confinement of rectangular sections with rectilinear reinforcement or FRP wrapping is less effective than that of circular transverse reinforcement in circular columns. The lateral pressure in this case is not constant around the column section, and the concrete between the corners is not confined nearly as much as the corner regions where the lateral constraining forces provide much higher lateral pressures. The confined area of a rectangular cross-section, externally confined with FRP is illustrated in Figure 2.1b. The loss in confinement efficiency in square and rectangular sections is often expressed through empirical expressions obtained experimentally.

Katsumata et al. (1987, 1988) were among the first who studied the effects of FRP composites in improving ductility and hence seismic performance of reinforced concrete columns. The researchers tested circular and rectangular columns with and without lateral CFRP strands, and illustrated the resulting improvements in column deformability. Subsequently, Seible, Priestley, Hegemier and Innamorato (1997) tested seven full-size bridge columns with CFRP jackets. The results were compared with unretrofitted reference columns. The researchers showed that the FRP jacketing significantly improved column ductility. They then developed design guidelines for FRP jacketing of concrete columns, which was subsequently implemented in California for

retrofitting a large number of bridge columns. Following the 1994 Northridge Earthquake in California, in excess of 500 bridge columns were retrofitted with FRP jackets (Seible et al. 1995, Xiao et al. 1995). Similar research programs were undertaken in Japan by Hoshikuma and Unjoh (1997, 1995).

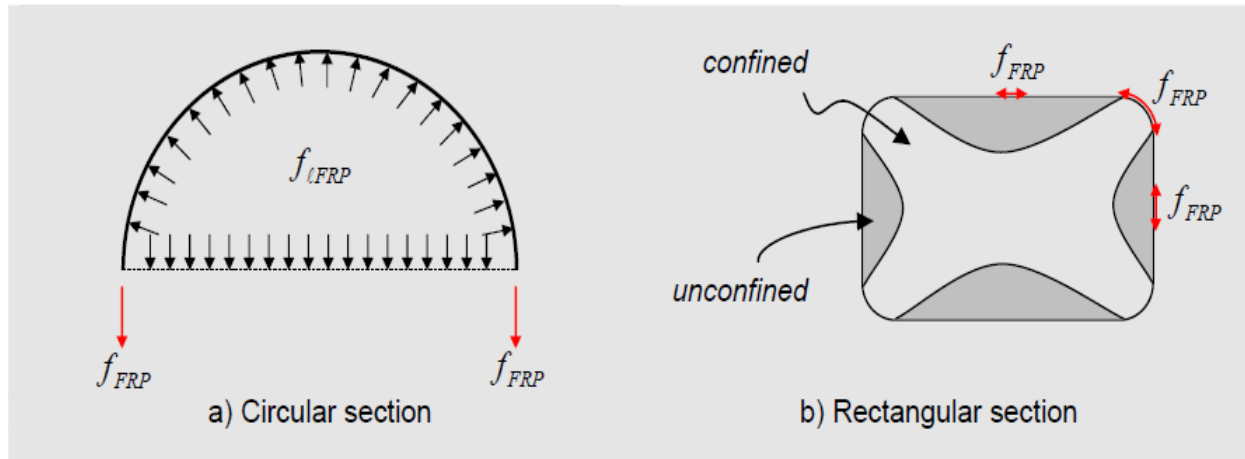


Figure 2. 1: External confinement with FRP (Rahai, et al., 2008)

Xiao and Ma (1997) confined concrete columns with prefabricated composite jackets. The researchers conducted experimental investigation in which they tested concrete columns with and without the FRP jackets. The FRP jackets were produced from uniaxial glass fibres in a factory environment, with improved quality of construction as compared to the field produced wet lay-up method. The fibres were applied such that 90% was in the circumferential direction and the remaining 10% was in the longitudinal direction. The tests demonstrated that significant ductility enhancement was attained. The retrofitted columns maintained stable hysteresis loops up to approximately 4% lateral drift.

Purba and Mufti (1999) tested small-scale circular columns (with a diameter of 190 mm), retrofitted with FRP jackets under monotonically increasing axial compression. CFRP sheets were used to provide the column jacket. The results showed substantial improvements in ultimate load and axial strain capacity. Relatively larger column specimens (305x305x1473 mm) were tested by Lacobucci, Sheikh, and Bayark (2002) to investigate the effectiveness of strengthening seismically deficient undamaged and damaged square columns with CFRP Jackets. The retrofitting technique substantially increased ductility and energy dissipation capacities of

columns. The effectiveness of CFRP jackets for repairing previously damaged columns depended on the severity of damage. The researchers concluded that more CFRP layers would be needed in the jacket for highly damaged columns. They further concluded that high levels of axial compression decreased overall column response, leading to additional demands on CFRP jackets to confine critical regions. As the continuation of the same research program, S.A. Sheikh and G. Yau (2002) evaluated the performance of circular columns under simulated seismic loading. The objective was to evaluate the effectiveness of FRP reinforcement in strengthening otherwise seismically deficient columns. A total of 12 circular columns with a 350 mm cross-sectional dimension were tested. The researchers concluded that the use of CFRP and GFRP resulted in significant improvements in both the column ductility and strength. Furthermore, they found that the behaviour of FRP retrofitted columns was better than that of columns with conventional steel-reinforcement.

Sause, et al. (2004) investigated the effectiveness of CFRP jackets in retrofitting non-ductile concrete columns. Four full-scale square building columns were tested under combined axial and lateral loads. The specimens had 458 mm square sections, and were subjected to 22% of the column concentric capacity during lateral deformation reversals. The results showed that the use of CFRP jackets enhanced the deformation capacity of columns significantly. However, the strength increase was negligible. The CFRP jackets were able to prevent the buckling of compression column bars under large displacement ductility demands.

Another small-scale column test program, including those retrofitted with CFRP wrapping was undertaken by Harajli and Rteil (2004). A total of 12 rectangular specimens with cross-sectional dimensions of 150x300mm and a shear span of 1050 mm were tested under constant axial compression and lateral deformation reversals. The authors concluded that the level of improvement in seismic performance of columns with insufficient longitudinal splice lengths depended on the level of improvement in bond behaviour of longitudinal reinforcement, rather than the total reinforcement ratio in the column.

A comprehensive experimental research program was carried out at the University of Ottawa to investigate the characteristics of FRP retrofitted building and bridge columns. Large-scale columns with circular and square cross-sections were tested under constant axial compression and incrementally increasing lateral deformation reversals (Elnabelsy and Saatcioglu 2004a,

Elnabelsy and Saatcioglu 2004b, Saatcioglu et al, 2008, Ozbakkaloglu and Saatcioglu 2009, Ozbakkaloglu and Saatcioglu 2007, Elnabelsy 2013, Elnabelsy and Saatcioglu 2014). The research program consisted of bridge and building columns, with different levels of axial compression, representing appropriate levels used in practice. The research resulted in a design procedure for FRP wrapping of columns that have been adopted by CSA S804 (2012) and CSA S850 (2013) standards.

2.2.2. FRP Strengthened beams in flexure

The application of CFRP plates for improving flexural capacity of concrete members was studied and investigated at the Swiss Federal Laboratory for Materials Testing and Research (EMPA) in mid 1980s. Since then, a number of researchers investigated the effectiveness of surface bonded and/or mechanically anchored FRP sheets in improving flexural capacity. An important aspect of this series of research was the surface bond capacity of FRP, which tends to be lower than the FRP rupturing strength.

M. Maalej and K.S. Leong (2005) tested simply supported concrete beams with and without CFRP sheets as additional surface bonded reinforcement. All the beams had top and bottom internal steel reinforcement. They concluded that the use of the CFRP sheets increased flexural capacity, while the capacity was controlled by debonding of FRP. The stiffness of the beams was increased as compared to the unretrofitted control beams. Kotynia et al. (2008) also conducted tests of simply supported concrete beams. The strengthening was done by either FRP strips or sheets, with U-shaped or L-shaped transverse reinforcement for improves surface bond. The researchers observed that the beams failed due to the debonding of the FRP from the concrete surface. The use of transverse FRP improved the bond characteristics of longitudinal FRP. Nevertheless, it was recorded that the debonding strain of FRP ranged between 0.62% and 0.86%. Similar debonding failures were also observed by H.B. Pham and R. Al-Mahaidi (2006) and Lee and Moy (2007) in their CFRP retrofitted beam tests.

Eighteen simply supported concrete beams with surface-bonded CFRP sheets were tested by Asduini and Nanni (1997). The parameters considered were; i) number of FRP sheets, ii) FRP development length, iii) width of FRP and iv) the inclination of FRP fibres. The researchers observed that the FRP was effective in increasing the ultimate capacity of beams, though the

failure always occurred by FRP debonding, albeit at different strain levels, ranging between 0.3% at the low end and 1.2% at the high end.

J.F. Bonacci and M. Maalej (2000) also tested simply supported concrete beams with surface bonded CFRP. The beams were subjected to either monotonic or cyclic loading. The failure was observed to take place at longitudinal strains of 0.61% to 0.98% after an increase of 10% to 35% induced in bending moment capacities. Similarly, J. Pan, T.C.F. Chung, and C.K.Y. Leung (2009) tested simply supported reinforced concrete beams, but they were retrofitted with CFRP laminates. The test parameters consisted of number of FRP plies, beam shear span, and number of loading points. While one beam failed through the rupture of FRP, eight other beams failed through bond failure.

The overall conclusion from the above beam tests is that, surface bonded FRP sheets do enhance flexural strength, but depending on their bond characteristics, the FRP suffers from premature bond failure prior to the crushing of concrete or rupturing of CFRP reinforcement.

2.2.3. Behaviour of Fibre Orientation on Column Behaviour

Most of the previous studies focused on use of fibers in the hoop direction due to the anticipated strength and ductility enhancements associated with concrete confinement, as well as in the longitudinal direction for flexural strength enhancement. Although some studies have been conducted on the use of angular fibers [Karbhari et al. 1993, Howie et al. 1995, Picher et al 1996, Mirmiran et al. 1996, Hoppel et al. 1997], and it has been pointed out that the use of angular fibers could possibly improve performance [Howie et al. 1995], the effects of fiber orientation and stack sequence are generally not well understood. Also, extensive efforts have been made to develop ultimate strength models for FRP wrapped columns with different fibre orientations [Lam et al. 2002]. The structural significance of the ultimate state of stress in terms of design and safety was found to be not as critical as the kinking point on the load deformation curve where a sharp reduction in slope occurs at about the point of unconfined concrete failure [Howie et al. 1995].

Au and Buyukozturk (2005) studied the effects of different fiber orientations and mix of ply configurations on the load-deformation characteristics and failure mechanisms of FRP wrapped concrete cylinders subjected to uniaxial loading. A total of twenty-four concrete cylinders with

150 mm diameter and 375 mm height were tested. Concrete cylinders had an average 28-day characteristic compressive strength of 24 MPa. Eighteen specimens were wrapped with FRP while the other six were left without the application of any FRP wrapping. All confined cylinders were wrapped using the wet lay-up technique after the plain concrete cylinders were primed using thickened epoxy. Three types FRP fabrics, made of E-glass fibers, were used. These were; i) 0° hoop, referred to as unidirectional fibers, ii) $0^\circ/90^\circ$ hoop/vertical, referred to as $0^\circ/90^\circ$ bidirectional weaved fibers with equal fiber content in both directions, and, iii) $\pm 45^\circ$ bi-angular fabrics, referred to as $\pm 45^\circ$ bidirectional weaved fibers with equal fiber content in both directions. Wrapping configurations employed were 0° , $0^\circ/90^\circ$, $\pm 45^\circ$, $90^\circ/\pm 45^\circ$, $0^\circ/\pm 45^\circ$, and $\pm 45^\circ/0^\circ$. It was concluded that hoop fibers are efficient in providing confinement, however wrapping in this direction shows a brittle failure in all specimens upon release of the stored energy. The fiber rupture was attributed to low fabric strength, which led to the jacket failure before further strength enhancement due to confinement could be developed. Unlike the hoop confinement, angular FRP confinement tended to fail in a ductile mode owing to the fiber re-orientation mechanism to dissipate energy.

Sadeghian et al. (2008) studied the behaviour of slender concrete columns with high strength CFRP composites having different configurations of fiber orientations and various wrap thicknesses. Thirty concrete cylinders of (150×300 mm) were tested under uniaxial compression up to failure. Twenty three specimens were wrapped with CFRP while the other seven were left without the application of any CFRP wrapping as unconfined control concrete cylinders. The unconfined compressive strength of plain specimens on the test day was measured to vary between 35 MPa and 45 MPa. Four different wrap thicknesses of 0.9, 1.8, 2.7, and 3.6 mm, and four fiber orientations of $0^\circ/90^\circ$, $\pm 45^\circ$ with respect to the hoop direction were investigated. It was concluded that the stress-strain curve of specimens with fibers in the transverse orientation, 0° , has bilinear behaviour and a positive slope; yet, when the angle of fibers is changed to $\pm 45^\circ$, the behaviour is changed to have a larger flat region up to failure. This behaviour is believed to enhance hysteretic damping and energy absorption capacity of columns under seismic loading.

2.3. Previous research on blast performance of FRP retrofitted columns

The cost and safety related issues associated with blast testing of structural components discouraged researchers from conducting extensive testing. Therefore, the previous literature in the area is limited.

J. E. Crawford et al. (2001) conducted different explosive tests on R/C columns and CFRP retrofitted R/C columns to verify the efficiency of composite wrap procedure for blast loading. A full-scale reinforced concrete test office building, simulating an office building was designed and constructed as shown in Figure 2.2. The building was designed for the US Seismic Zone 1 (East Coast of USA). Two blast tests were conducted to compare the response of an FRP wrapped column (DB 8) to that of as built non-retrofitted column (DB 6). Both columns had a 350-mm square section. The retrofitted column (DB 8) had six horizontal CFRP layers to improve shear resistance, and three vertical 102 mm CFRP strips on each side to improve flexural strength. The CFRP had a modulus of elasticity of 230 GPa, tensile strengths of 3 GPa for the wraps and 3.7 GPa for the longitudinal strips. The fibre content in vertical strips was 65%.

The field test indicated that the unretrofitted column (DB 6) failed in shear at the top and bottom, while the middle part of the column remained undamaged and in good shape. The residual deflection at mid-height of the column was 250 mm, as illustrated in Figure 2.3a. The main cause of displacement was attributed to flexural response. In contrast, the retrofitted column (DB 8) remained elastic, and no permanent deflection was evident as illustrated in Figure 2.3b.

An additional set of 20 full-scale specimens was tested under carefully controlled quasi-static load conditions to interpret and validate the observed behaviour of retrofitted and unretrofitted columns tested in the field, as described above (Malvar et al. 2007). A lateral load system, shown in Figure 2.4, was used to simulate the blast load. Lateral loads were applied using three computer-controlled actuators while axial load was applied using only two computer-controlled actuators as shown in Figure 2.5.



Figure 2.2: Test building for field testing and locations of DB 6 and DB 8 columns



(a) Failed unreinforced column (DB 6)



(b) Composite retrofitted column (DB 8)

Figure 2.3: DB 6 and DB 8 columns after field test

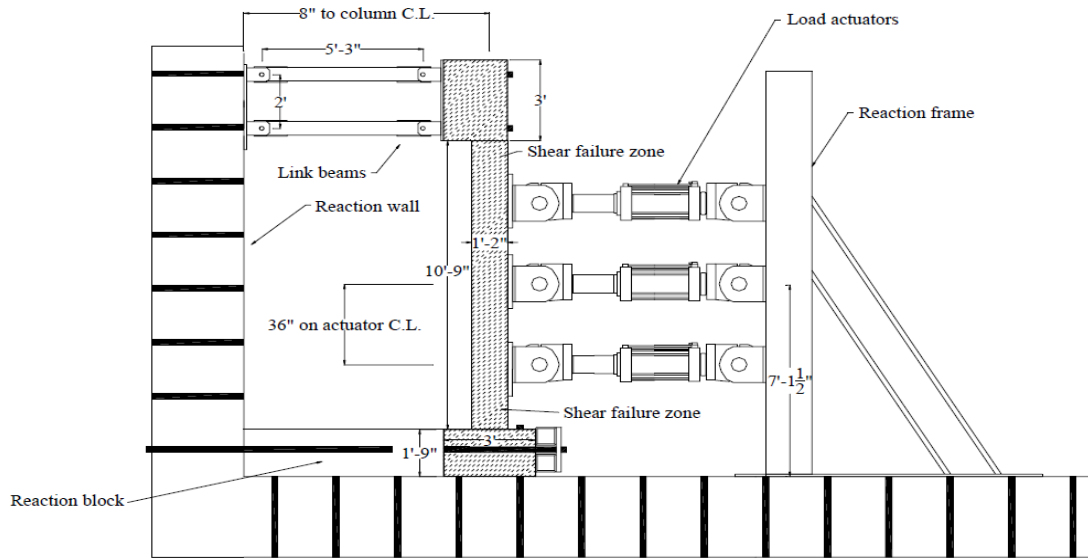


Figure 2.4: Lateral load system used in full-scale laboratory tests (Crawford et al. 2001)



Figure 2.5: Full scale CFRP RC column tested by lateral loading system

Results of the quasi-static test indicated that the laboratory test setup was capable of reproducing similar conditions to those of the field test. Figure 2.6 illustrates the comparison of responses of unretrofitted test specimen (DB6) tested in the field and in the laboratory. The reinforced concrete columns retrofitted with two and six layers of CFRP were also tested in the laboratory and their results were compared with that of the unretrofitted column. The comparisons are shown in Figures 2.7 and 2.8, respectively.



(a) Column from DB 6 test.



(b) Column from laboratory test.

Figure 2.6: Comparison between DB 6 test and laboratory test

The maximum strength obtained for the column wrapped with two layers of CFRP was twice the maximum strength recorded in the unwrapped column, while the maximum deflection at mid height was 114 mm. The shear resistance was enhanced by two layers of CFRP to allow the column to reach its full flexural capacity. Yet, the failure of the column was due to flexural hinging and the insufficient hoop CFRP provided for concrete confinement. The shear capacity and the ductility were increased when the column was strengthened with six layers of CFRP. Moreover, no sign of damage was observed even when the deflection at mid-height increased to 152 mm. The authors concluded that composite wrapping of columns is an effective technique to secure the survivability of existing reinforced concrete buildings subjected to blast loads.

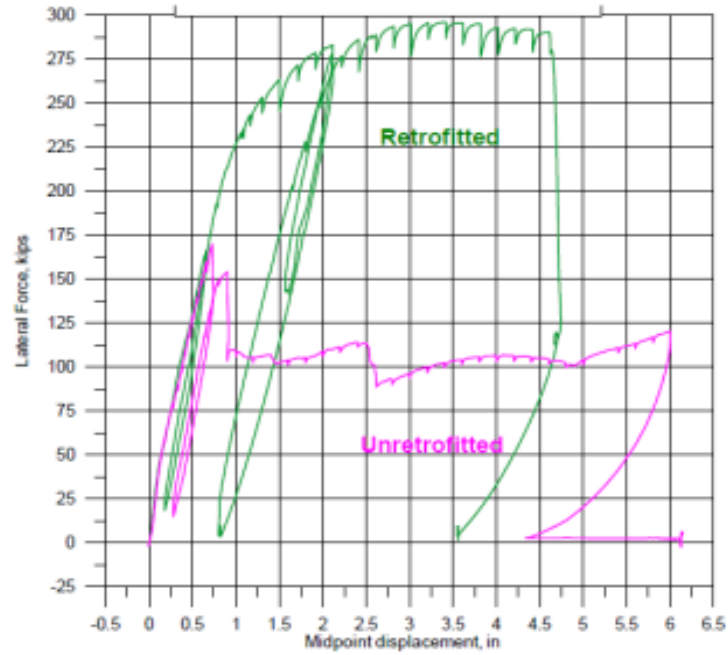


Figure 2.7: Comparison of lateral response of columns with and without two-layers of CFRP hoops (Crawford et al. 2001)

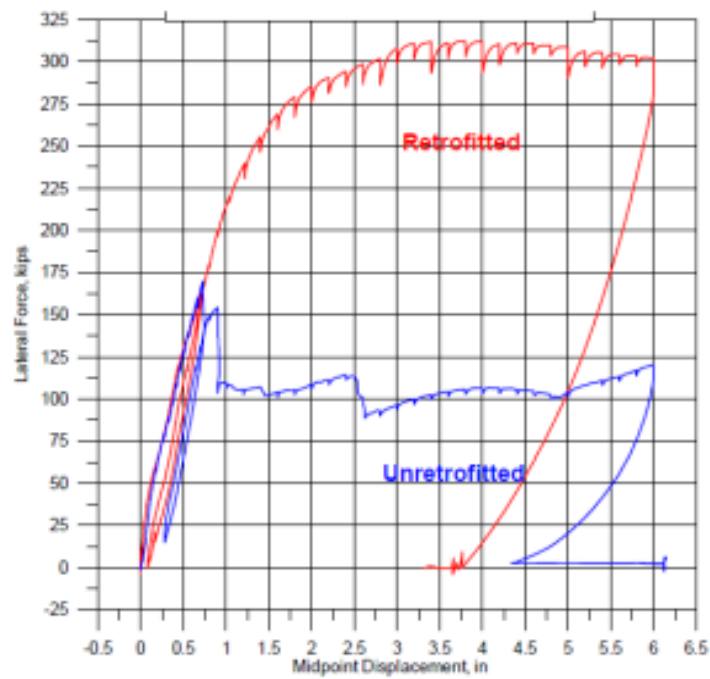


Figure 2.8: Comparison of lateral response of columns with and without six-layers of CFRP hoops (Crawford et al. 2001)

L.C Muszynski and M.R. Purcell (2003) tested reinforced concrete column-wall assemblies with and without CFRP retrofits. Two test assemblies were tested, one with carbon and glass FRP sheets and the other as a reference specimen. The elements were subjected to live blast testing (800 kg TNT at 14 m for retrofitted and 16 m for unretrofitted elements). The retrofitted element remained intact while some of the CFRP laminates ruptured. The unretrofitted element was damaged significantly. It was then concluded that the use CFRP and GFRP laminates increased the capacity against blast loads.

Carroere et al. (2007) tested 10 reinforced concrete beam-columns. The specimens had a 150 mm cross-section with 1.5 m span between fixed ends. Each specimen was reinforced with 4-6 mm diameter bars, one at each corner, tied by steel reinforcement with spacing of 50 mm near the supports and 100 mm within the span. Half of the specimens were retrofitted with steel fibre reinforced polymer (SRP) wraps, whereas the other half were companion unretrofitted specimens. The specimens were subjected to live testing using 15 kg to 50 kg TNT at 2 m standoff distance. The researchers reported that the retrofitted specimens developed higher ductility and strength. They further reported that the axial capacity of the beam-columns increased due to confinement.

Berger, Heffernan, and Wight (2013) tested 18 reinforced concrete columns of the same geometry as the above, but reinforced them with 10 mm diameter longitudinal bars. The specimens were retrofitted with surface-bonded SRP and CPRP sheets. The researchers concluded that the combined use of longitudinal and transverse reinforcement significantly reduced damage. The columns reinforced with SRP showed increased ductility, relative to that with CFRP. Bond failure occurred in a column that did not have a transverse layer of FRP over the longitudinal polymers.

Tests of 10 columns with CFRP jackets were conducted by Rodriguez-Niki et al. (2009) by using a system of hydraulic jacks, simulating blast loading. The detailing of columns corresponded to those for a low seismic region. It was reported that the CFRP jackets significantly improved column response.

A design methodology was developed for column retrofitting with FRP wraps by Crawford et al. (2001). The researchers used both experimental and analytical research findings for developing

their procedure. It was subsequently incorporated into computer software. The researchers reported that columns lacking sufficient transverse reinforcement would develop shear failures under blast loads. The use of FRP jackets improved column strength by enhancing shear capacity. Those columns that were retrofitted with FRP jackets having transverse and longitudinal sheets promoted elastic behavior for otherwise deficient columns.

2.4. CSA Design Standards and the Canadian Design Approach

Two CSA Standards are relevant to FRP retrofit of concrete columns under blast loading. These include CSA S806-12 for use of FRP reinforcement in concrete structures in general, and CSA S850 on design of structures subjected to blast loading. The latter specifically deals with blast-resistant design, and makes reference to the former standard for FRP retrofits.

The CSA Standard S806-12 requires the following FRP jacket thickness, t_j :

$$t_j = 2D \frac{f'_c}{\phi_F f_{Fj}} \frac{P}{P_o} \frac{\delta}{\sqrt{k_c}}$$

Where $\frac{P}{P_o} \geq 0.15$

δ = design lateral drift ratio, which shall not be less than 0.04 for columns in ductile moment-resisting frames ($R_d = 4.0$) and 0.025 for columns in moderately ductile moment-resisting frames ($R_d = 2.5$).

$k_c = 1.0$ for circular and oval jackets.

$k_c = 0.4$ for square and rectangular jackets.

The stress in FRP jacket f_{Fj} shall be determined as follows:

(a) for columns with circular and oval jackets, $f_{Fj} = 0.005E_F$ when $\frac{P}{P_o} > 0.15$ and $f_{Fj} = 0.01E_F$ when $\frac{P}{P_o} \geq 0.30$ with linear interpolation for in-between values of P/P_o ;

(b) for square and rectangular jackets, $f_{Fj} = 0.003E_F$ when $\frac{P}{P_o} > 0.15$ and $f_{Fj} = 0.006E_F$ when $\frac{P}{P_o} \geq 0.30$

with linear interpolation for in-between values of P/P_o ; and

(c) f_{Fj} shall not be more than $\phi_F f_{Fu}$

In the above expressions f_{Fu} is the ultimate rupturing strength of FRP and ϕ_F is the material strength reduction factor for FRP, equal to 0.7.

Chapter 3: Experimental Program

3.1. Introduction

The experimental program involves tests of large-scale reinforced concrete columns retrofitted with carbon fiber reinforced polymer (CFRP) laminates, tested under combined axial service loads and equivalent static blast loads applied uniformly in the lateral direction. Nine half-scale reinforced concrete columns, conforming to the seismic design requirements of CSA A23.3-04 were designed, built, instrumented and tested in the structure laboratory of the National Research Council of Canada (NRC). Seven of the columns were strengthened using four different CFRP laminates, wrapped around the columns over their entire length, as discussed in the following sections. This chapter presents the details of the experimental program. It consists of: (i) material characterization; (ii) column reinforcement and instrumentation, (iii) specimen preparation, including concrete casting and curing; (iv) FRP strengthening details; and (v) test set-up and test procedure.

3.2. Material Properties

3.2.1. Concrete

The concrete used for specimen construction was ordered from a local ready mix company. The only specifications given were a target compressive strength of 25 MPa with 10 mm coarse aggregate size. The mix had adequate workability and the concrete was placed well during the construction of test specimens. No segregation was observed and very slight surface water (bleeding) appeared. Table 3.1 shows the properties for the concrete used.

The compressive strength was obtained by following the ASTM standard C39 275 days after casting, on the first day of testing. The tensile splitting strength was obtained by following the ASTM standard C496. The static modulus of elasticity and Poisson's ratio were obtained by following the ASTM standard C469. Test results were determined from the average of at least three cylinders.

Table 3.1: Concrete properties

Slump mm	210
Density (kg/m ³)	2360
Compressive Strength, f'_c (MPa)	37.31
Strain at Peak Compressive Strength, ϵ'_c (mm/mm)	0.002
Splitting Tensile Strength (MPa)	3.2
Static Modulus of Elasticity (MPa)	30300
Poisson's Ratio	0.21

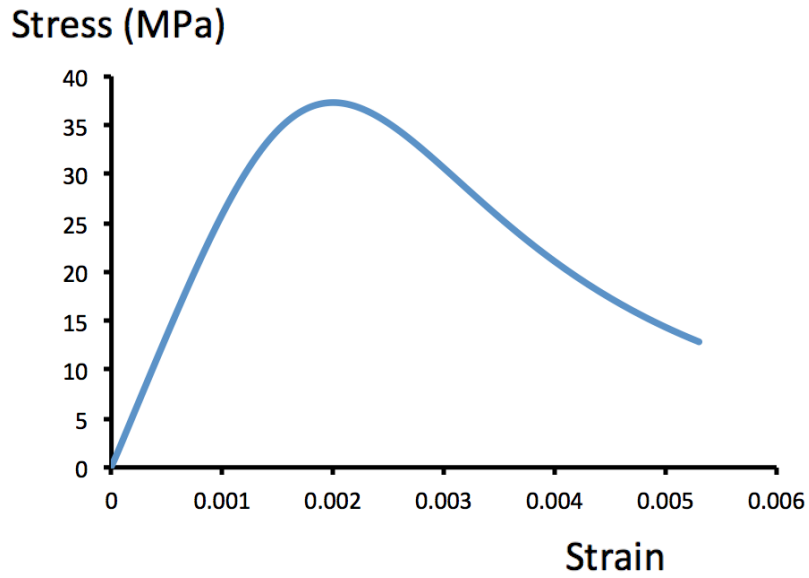


Figure 3. 1. Typical stress-strain relationship for concrete

Figure 3.1 shows the stress-strain relationship reproduced analytically by using the model proposed by Popovics (1973). It is based on the standard cylinder tests performed 275 days after casting the concrete columns.

3.2.2. Steel

All nine columns were reinforced with No. 10M (11.3 mm diameter) longitudinal reinforcement tied together by 6.3 mm diameter transverse steel ties. Standard coupon tests were conducted to determine stress-strain characteristics of steel reinforcement using an MTS materials test frame. Figures 3.2 and 3.3 show the stress-strain relationships, modulus of elasticity and the ultimate tensile stress and strain for the transverse and longitudinal steel, respectively. For the transverse steel the yield strength was established by employing the 0.2% offset method. This is shown in Figure 3.2. For the longitudinal steel, the yield point was observed directly on the stress-strain curve as the onset of the clear yield plateau. Figure 3.3 shows the stress strain relationship of longitudinal reinforcement. Table 3.2 illustrates the mechanical properties of reinforcing steel. The discontinuity shown within the post-yield region of the stress-strain relationship in Figure 3.2 is due to 50,000 microstrain limit of the extensometer, beyond which the strain was obtained from the machine cross-head displacement.

3.2.3. Carbon Fiber Reinforced polymer (CFRP)

The stress-strain relationship of CFRP laminates was established by testing coupons. The coupons were cut from several 450 mm X 450 mm CFRP laminates, prepared using a hand-layup method by impregnating the fibres in epoxy. Each laminate was produced using the same sequence of placement of FRP sheets as those used for the column jackets. The characteristics of the laminates are given in Table 3.3. They were prepared on plastic sheets. The coupons were then cut using a water-jet cutter to obtain 250 mm by 25 mm samples of the same thickness as the corresponding column jackets. They were cut in two orthogonal directions, indicated as X_{Laminate} and Y_{Laminate} in Figure 3.4. Each coupon was labeled according to the direction in which it was cut (0° parallel to the X_{Laminate} axis; and 90° parallel to Y_{Laminate} axis). Coupons are shown in Figure 3.5 and 3.6.

All tension coupon tests were conducted using an MTS-810 test frame, equipped with hydraulic grips. The hydraulic grip pressure was set to the highest possible pressure that would avoid slippage but not damage to the coupon ends during testing. The grip pressure of 10 psi (69 kPa) was found to fulfill this requirement for all coupons. The strains in each coupon were measured using an MTS-634.11F-24-series extensometer. This extensometer had a nominal gauge length of 50 mm. The original length of the coupons prior to the application of load was 150 mm,

measured between the upper and lower grips. Each grip was fitted with a set of aluminum spacer tabs, which ensured the application of uniform pressure on coupons, as per the ASTM D-3039 specification Figure 3.7 shows a typical coupon test.

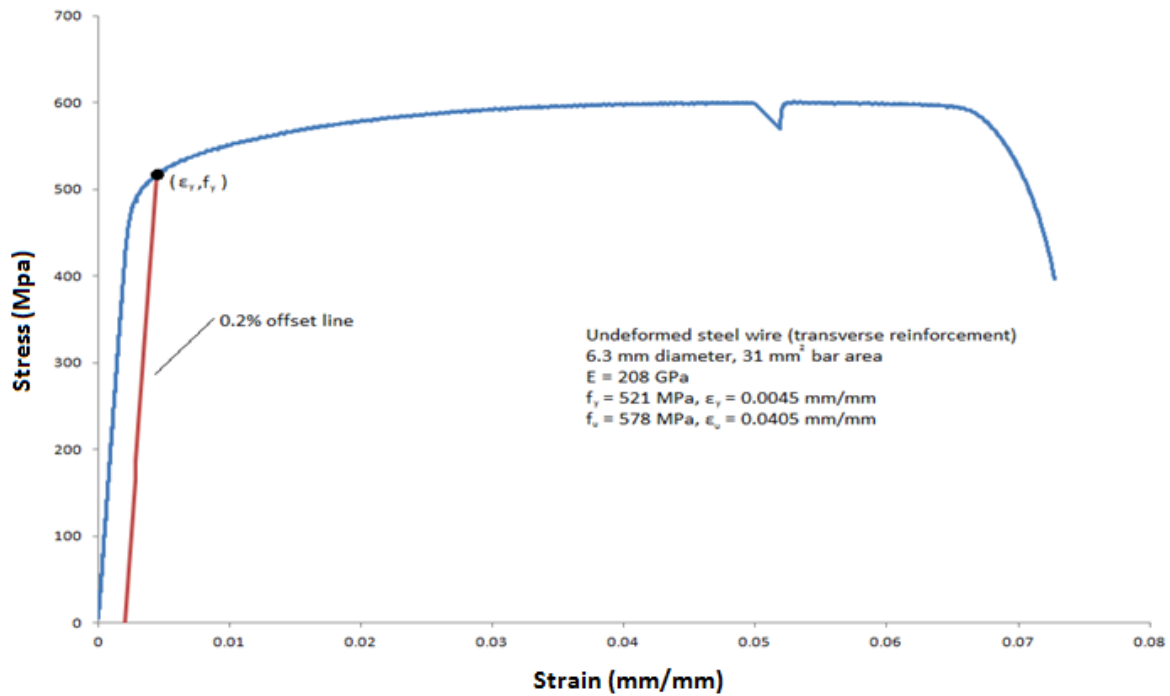


Figure 3. 2. Stress-strain curve for transverse reinforcement and material properties

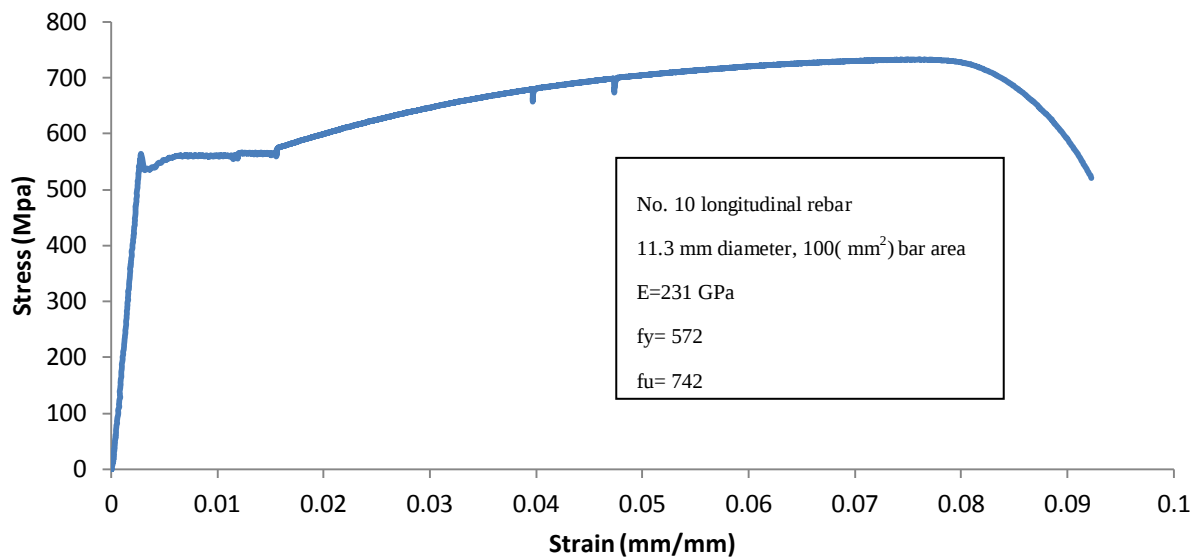


Figure 3. 3. Stress-strain curve for longitudinal rebar and material properties

Table 3.2: Mechanical properties for reinforcing steel

Bar diameter	f_y (MPa)	f_u (MPa)	E (GPa)	ϵ_y (mm/mm)	ϵ_u (mm/mm)
6.3(mm)	521	578	208	0.0045	0.0405
11.3 (mm)	572	742	231	0.0025	0.0771

Table 3.3: Laminate type and properties

Laminate type	Laminas Lay-Up	Laminas Lay-Up description	Laminate Thickness (mm)	Column Group
	Control (No CFRP)	NA	NA	CG-I
CFRP-L1	UD $[0_2 90_2]$	two uni-direction laminas in 0° and two uni-direction laminas in 90°	2.644	CG-II
CFRP-L2	W $[0/90]_4$	four woven 0° - 90° laminas	2.000	CG-III
CFRP-L3	W $[0/90]_2$ W $[\pm 45]_2$	two woven 0° - 90° laminas and two woven $\pm 45^\circ$	2.220	CG-IV
CFRP-L4	UD $[0 90]$ W $[\pm 45]_2$	two uni-direction laminas in 0° and 90° and two woven $\pm 45^\circ$	2.600	CG-V

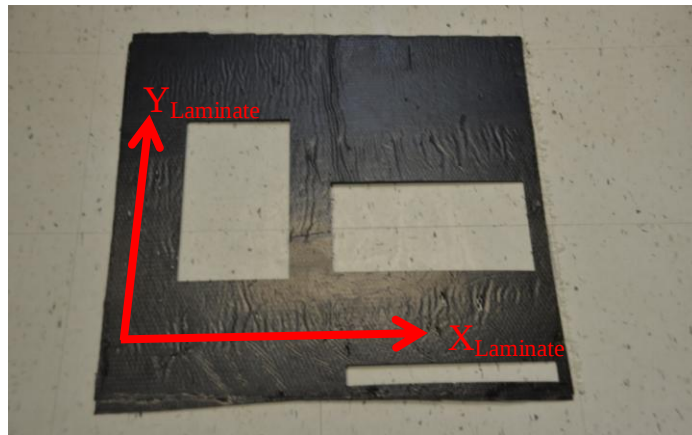


Figure 3. 4. A CFRP laminated sheet with 2 sets of 5 specimens' coupons cut in 0° and 90° directions

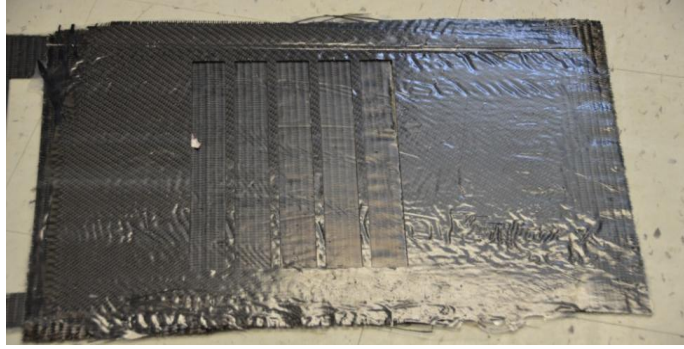


Figure 3. 5. CFRP laminated sheet from which five coupons were cut

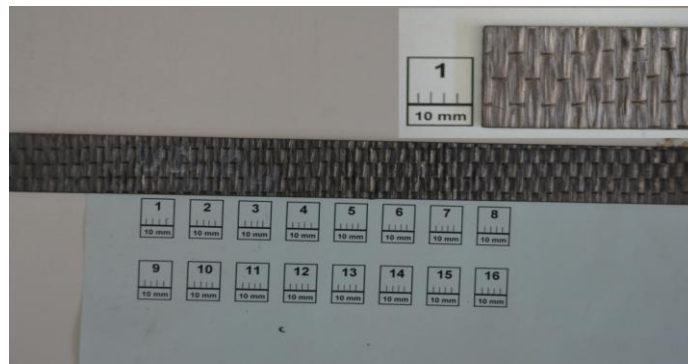


Figure 3. 6. A sample coupon

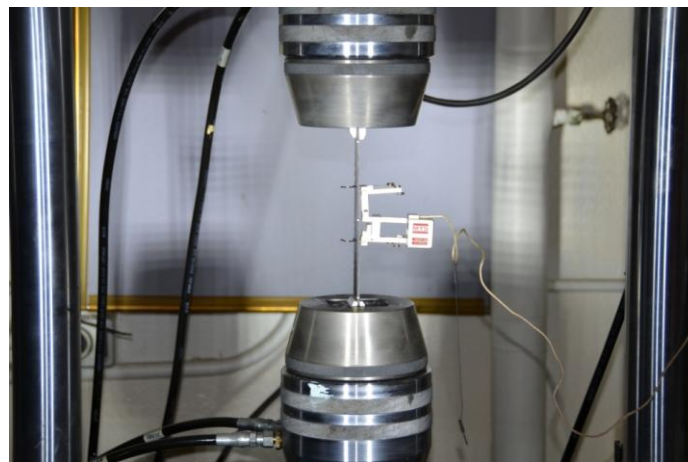


Figure 3. 7. MTS Tensile Load Frame and a test coupon with extensometer installed

Each of the four CFRP laminate types used in strengthening the columns, specified in Table 3.3, was tested in both principle directions. For comparison, two additional laminates of

approximately the same FRP content were also tested. This resulted in: (i) four coupons of UD 0° ; and (ii) five coupons of $W \pm 45^\circ$. The stress-strain relationships of laminate CFRP-L1 in two principle axes are shown in Figure 3.8. The material properties of this laminate are the same in both directions; indicating the same linear stress-strain relationship, the same modulus of elasticity, and the same ultimate stress and strain. The stress-strain relationships of laminate CFRP-L2 in the principle axes are shown in Figure 3.9. The material properties are almost the same in both directions where they have the same linear stress-strain relationship, the same modulus of elasticity, and the same ultimate stress; while the ultimate strain is slightly different (by approximately 5%). The stress-strain relationships of laminate CFRP-L3 in two principle axes are shown in Figure 3.10. The properties of this laminate are different in the two directions. The ultimate stress in X_{Laminate} direction is higher than that in Y_{Laminate} by 21%. Similarly, the ultimate strain in X_{Laminate} direction is higher than that in Y_{Laminate} by 19%. The modulus of elasticity is similar in the two directions (in X_{Laminate} direction 55 GPa, while in Y_{Laminate} direction 59 GPa). The stress-strain relationships of laminate CFRP-L4 in two principle axes are shown in Figure 3.11. The properties of this laminate is different in two directions, with the ultimate stress in X_{Laminate} direction being higher by 19% relative to that in Y_{Laminate} . Similarly, the ultimate strain in X_{Laminate} direction is higher than that in Y_{Laminate} direction by 47%. The modulus of elasticity of the laminate in X_{Laminate} direction is 51.5 GPa while it is 79 GPa in the Y_{Laminate} direction.

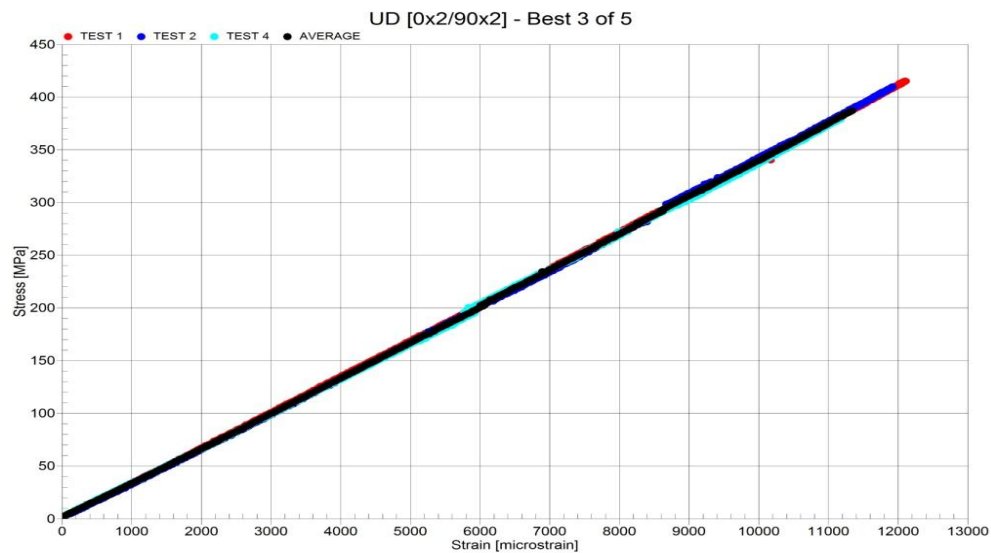
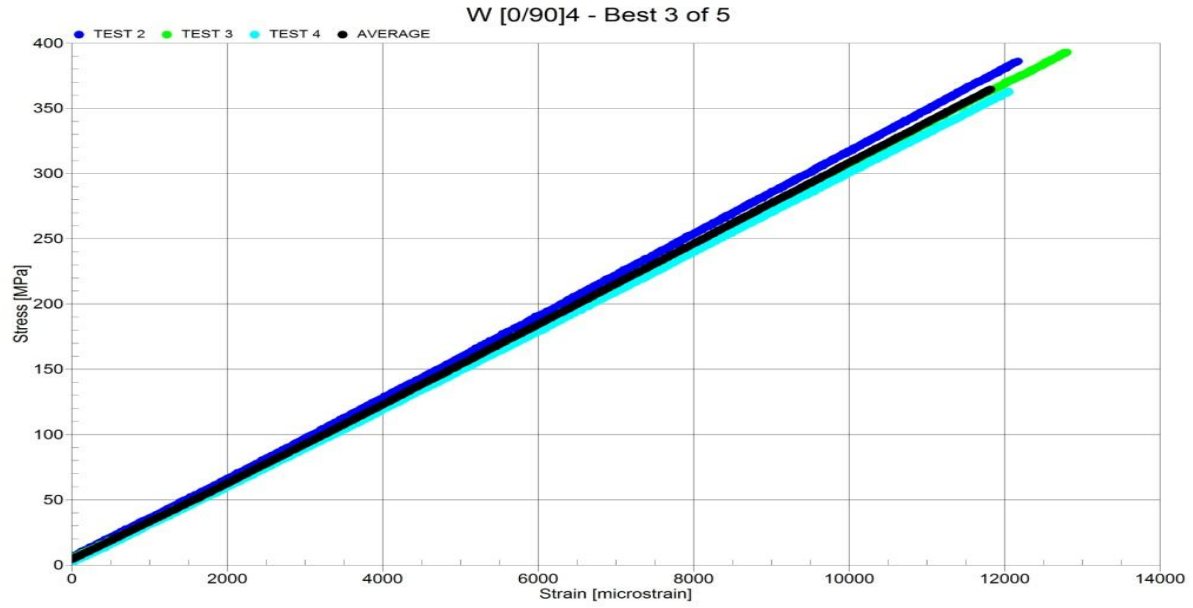
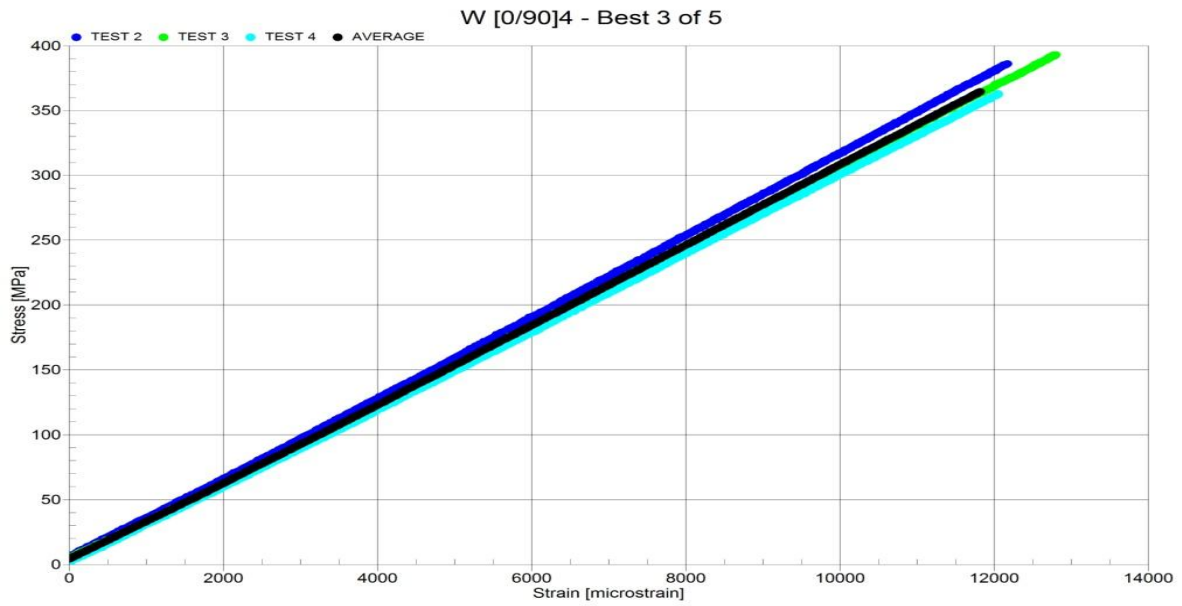


Figure 3. 8. Stress-strain relationships for CFRP-L1 in X_{Laminate} and in Y_{Laminate} directions

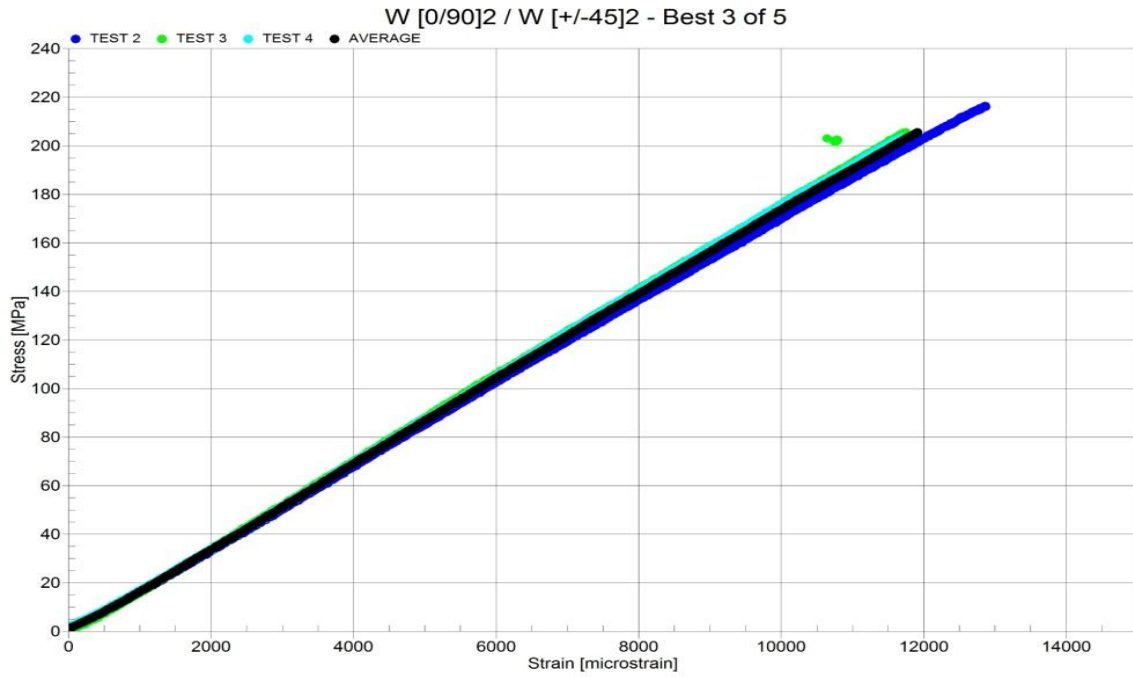


a) In X_{Laminate} direction

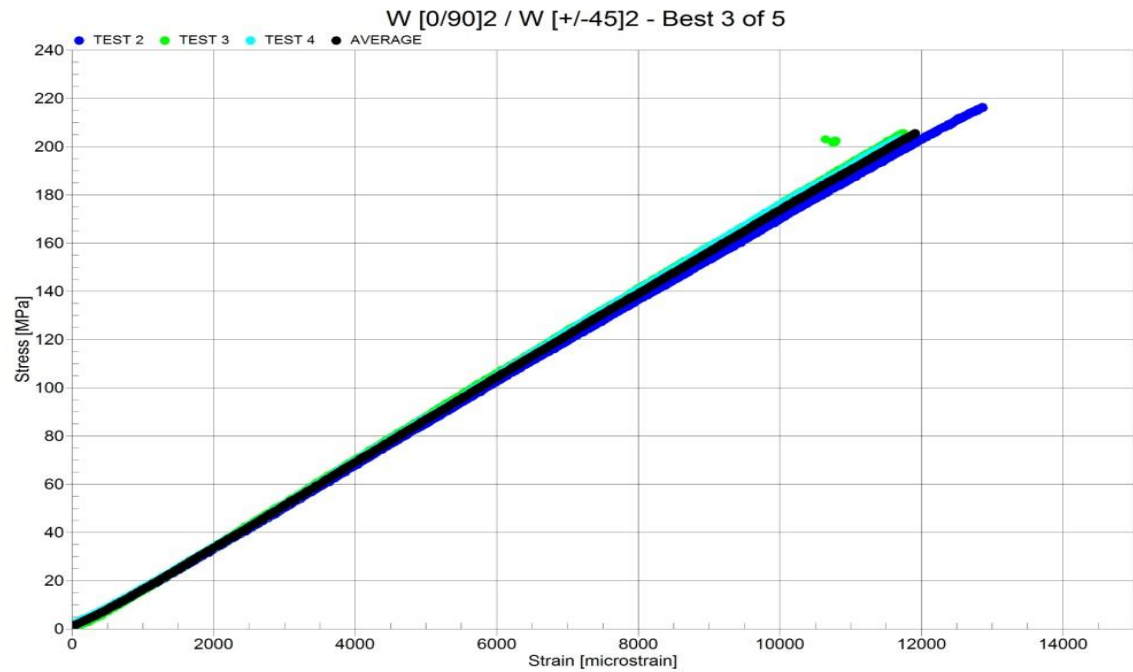


b) In Y_{Laminate} direction

Figure 3. 9. Stress-strain relationship for CFRP-L2

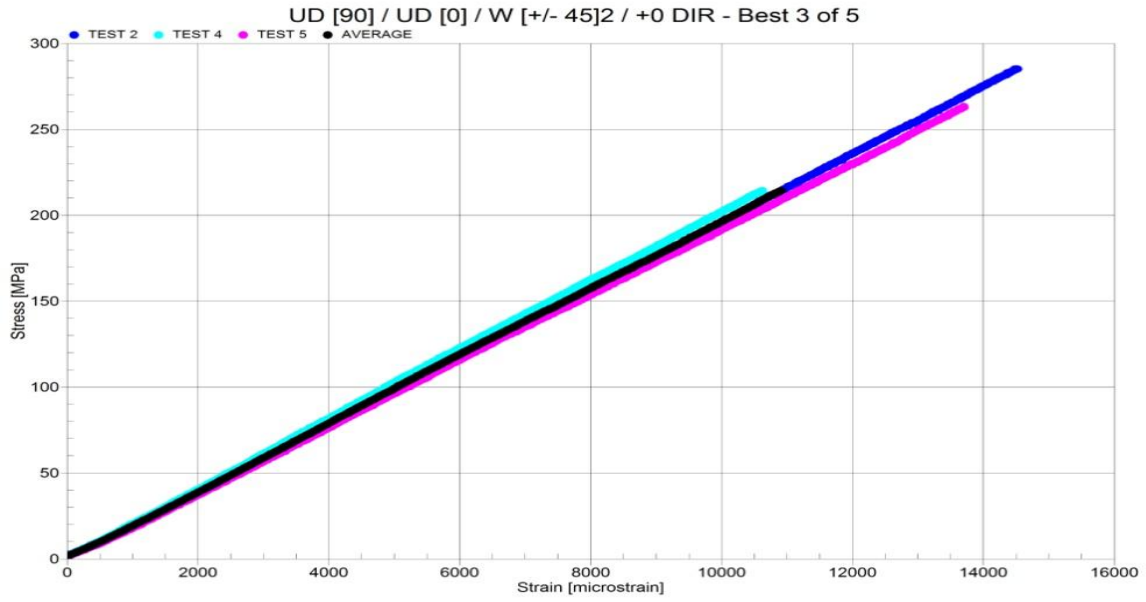


a) In X_{Laminate} direction

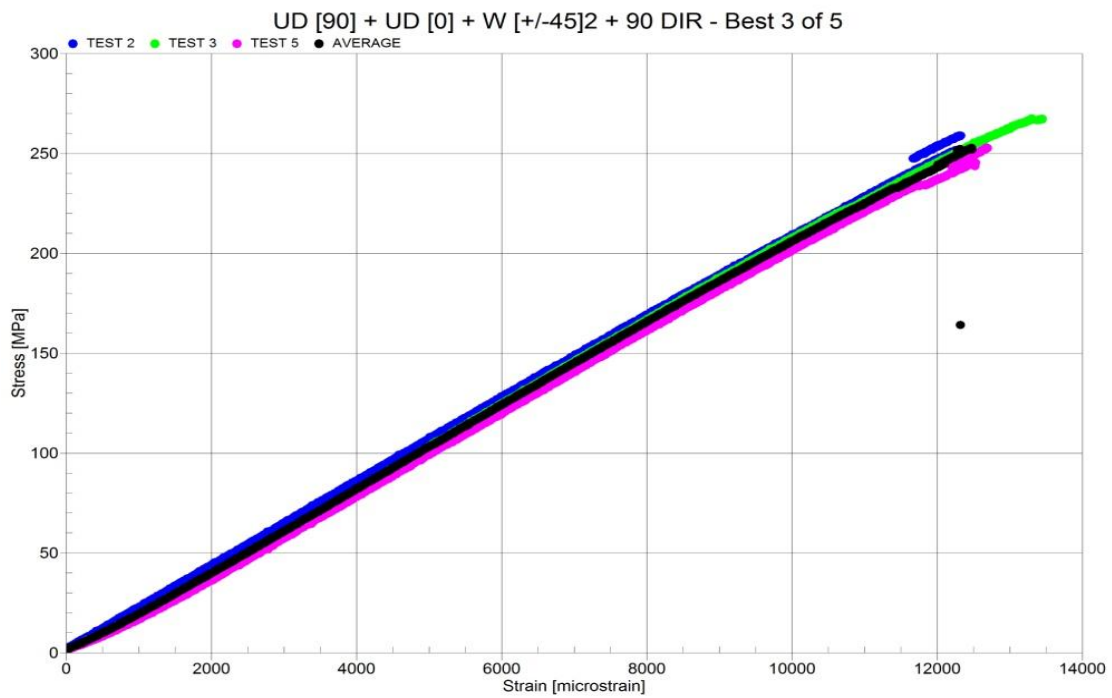


b) In Y_{Laminate} direction

Figure 3. 10. Stress-strain relationship for CFRP-L3



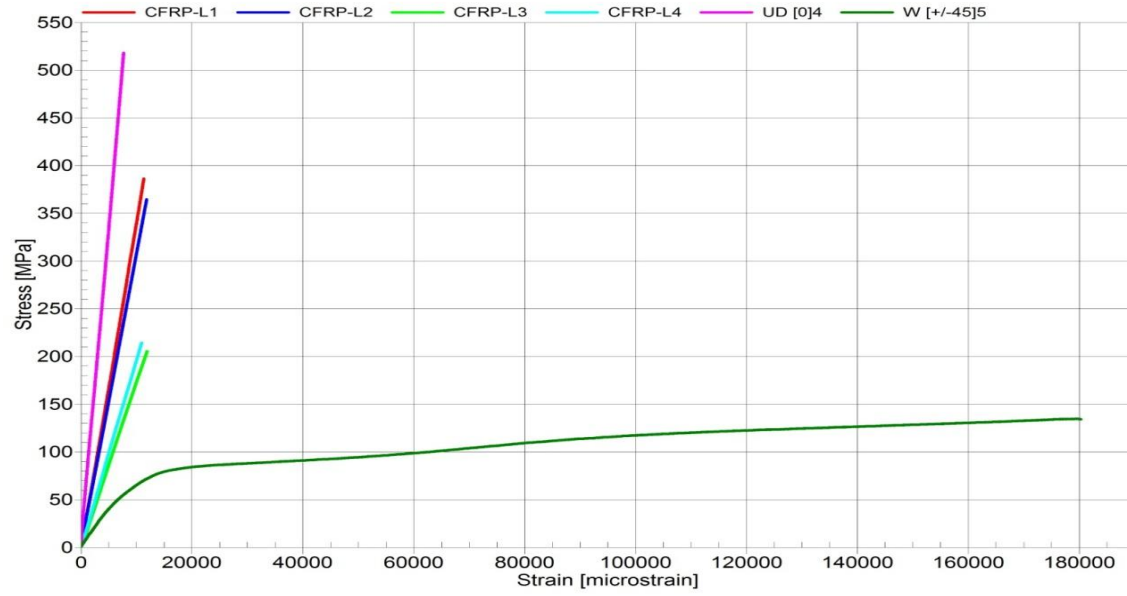
a) In X_{Laminate} direction



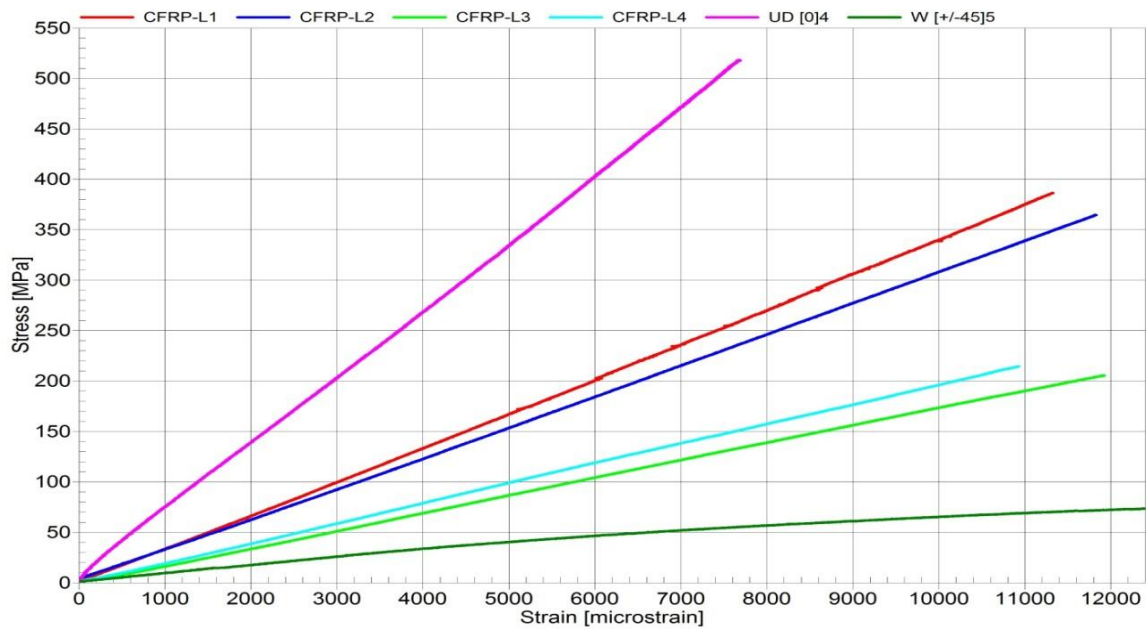
b) In Y_{Laminate} direction

Figure 3. 11. Stress-strain relationship for CFRP-L4

Figure 3.12 shows the stress strain relationships of laminates CFRP-L1 through CFRP-L4 compared with the two laminates formed from repeated single type lamina, i.e., 0° and $\pm 45^\circ$. As shown in the figure, the laminate with 0° gives the highest modulus of elasticity, highest strength and the lowest ultimate strain. On the other hand, the laminate with $\pm 45^\circ$ shows nonlinear stress-strain relationship and gives the lowest modulus of elasticity, lowest ultimate stress and very high ultimate strain. It is clear that the other laminate give mechanical properties that vary between the two extreme laminates. The woven lamina improves the strain capacity and the uni-directional lamina improves the strength and the modulus of elasticity.



(a)



(b)

Figure 3. 12. Stress-strain relationships for different tested CFRP laminates at X_{Laminate} direction
(a) full strain range; (b) strain range up to ultimate strains of the four laminates

3.3. Column Specimens

A total of nine reinforced concrete half-scale column specimens were fabricated and tested in the experimental program. Each column specimen consists of a 150 mm square cross section and a height of 2438 mm. The columns were reinforced longitudinally with four 10M (100 mm²) steel rebars and laterally with 6.3 mm closed steel ties spaced at 37.5 mm, as per the requirement for seismic design. Figure 3.13 shows the reinforcement details of a column specimen. A clear concrete cover of 10 mm was provided in all columns. Seven specimens were strengthened with CFRP and the other two were used as control specimens (no strengthening). The columns were numbered as S1 to S9, where the letter “S” indicates columns meeting the seismic requirements of the current CSA A23.3, and the numeric value indicates the sequence in which they were tested. The columns were divided into five groups according to the CFRP laminate type used as indicated in Table 3.4. The column groups were: (i) Group CG-I for control columns, which included columns S5 and S6; (ii) Group CG-II for the column with laminate type CFRP-L1 having UD [0₂/90₂] FRP, which included column S1; (iii) Group CG-III for two columns with laminate type CFRP-L2 having W [0/90]₄ FRP, including columns S2 and S4; (iv) Group CG-IV for two columns with laminate type CFRP-L3 having W [0/90]₂ and W [±45]₂ FRP, including columns S7 and S8; and (v) Group CG-V for two columns with laminate type CFRP-L4 having UD [0/90] and W [±45]₂ FRP, including columns S3 and S9.

Figure 3.14 shows the lamina fiber orientation relative to column coordinates. The fiber orientation UD 0° indicates fibers parallel to the column X-axis in the lateral direction, in the direction of column ties. The fiber orientation UD 90° indicates fibers parallel to the column Y-axis, which is the longitudinal direction, parallel to the main column reinforcement. When the [±45] woven CFRP lamina are applied, the orientation angles of the two orthogonal fibers are measured from the column X-axis. The flat CFRP laminates prepared for the purpose of obtaining FRP coupons were laid-up horizontally, similar to the fiber laminates on the sides of column specimens.

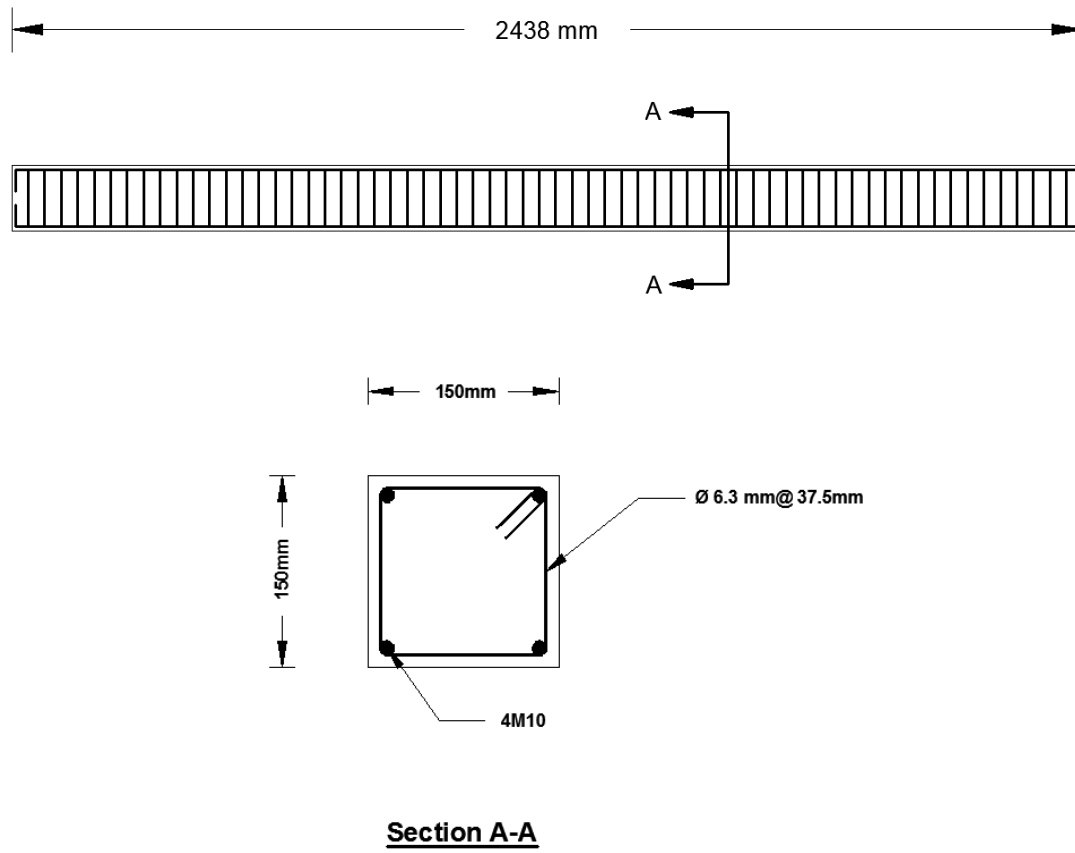


Figure 3. 13. Reinforcement details for all column specimens.

Table 3. 4: Column specimens groups

Column Group	Specimens Name	Laminate type	Number Of Columns
CG-I	S5 & S6		2
CG-II	S1	CFRP-L1	1
CG-III	S2 & S4	CFRP-L2	2
CG-IV	S7 & S8	CFRP-L3	2
CG-V	S3 & S9	CFRP-L4	2

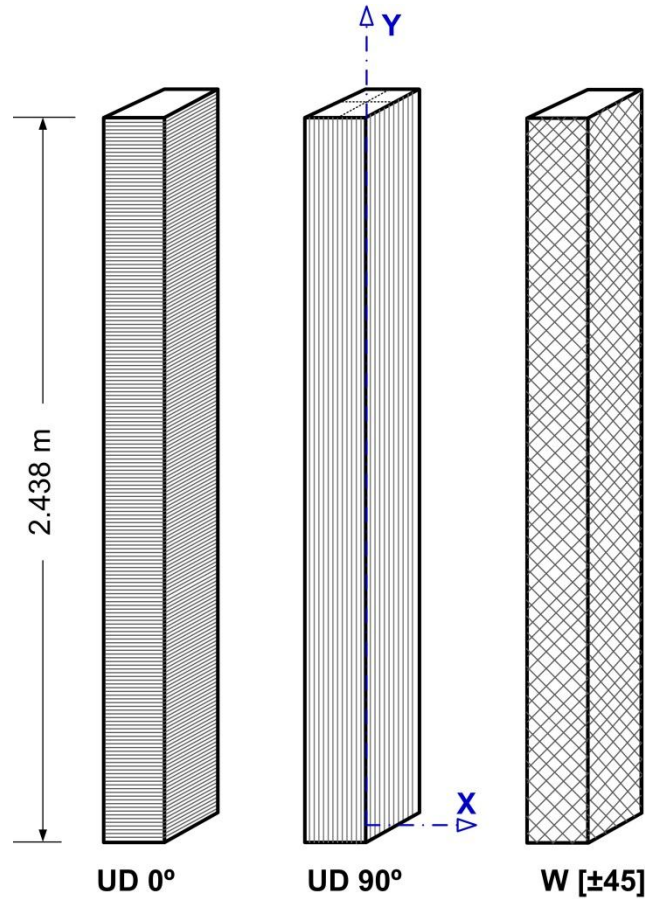


Figure 3. 14. Different fiber orientations relative to column axis

3.4. Construction of Specimens

3.4.1. Reinforcement Cages

The steel reinforcement cages used in the construction of column specimens were built at the NRC Construction Structures Laboratory. The cages were built by: (i) bending stirrups, (ii) tying longitudinal bars to the stirrups bent as per Figure 3.13, (iii) instrumenting the longitudinal bars and the stirrups with strain gauges, and (iv) testing the installed strain gauges and running the lead wires along the reinforcement from the gauges to the exit point located at the quarter height of column specimens. The stirrups were 130 mm square (outside dimensions), which allowed for 10 mm clear concrete cover from all sides. Stirrups were made by cutting 580 mm long bars (6.3 mm diameter) and then bending them to lengths using a steel bending jig. Figure 3.15 shows the bending jig, bending lever arm and a completed stirrup.

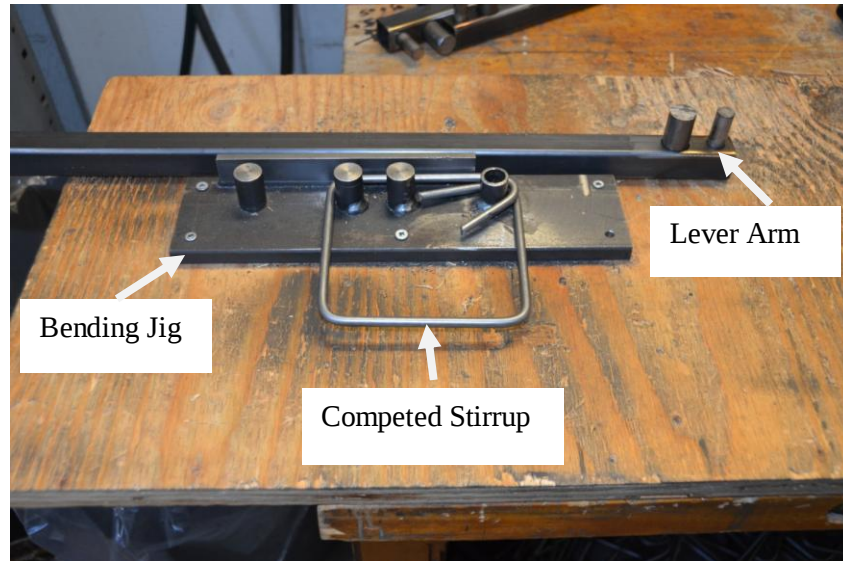


Figure 3. 15. Stirrup bending jig, lever arm, and a completed stirrup

3.4.2. Strain Gauges

Each column specimen was instrumented with a total of eleven strain gauges (see Figures 3.16 and 3.17). Six strain gauges were installed on two of the four longitudinal reinforcement bars. Three of these gauges were placed on one of the two compression bars, whereas the remaining three were installed on one of the two tension bars. Two strain gauges were installed on two legs of a stirrup located at 800 mm from the mid height (in the critical shear zone). In addition, three strain gauges were installed on the concrete surface of the column; one at mid-height, the others at quarter and three-quarters of the height, on compression face of the column (based on the application of uniformly distributed lateral load). All the strain gauges used were 6.35 mm long. The strain gauges were checked individually before and after the cage assembly, using a Gauge Installation Tester to ensure that all strain gauges were working properly. Once all the gauges passed the performance check, the instrumented reinforcement cages were set in the formwork for concrete casting. Figure 3.18 shows the instrumented cages in column formwork.

3.4.3. Casting and Curing of Concrete

The concrete used for casting the columns was supplied by a local ready-mix concrete company. All the specimens were cast horizontally. In an attempt to simplify the grinding of specimen surface and corners after the removal of the formwork, the top surface of all specimens were smoothed with a finishing trowel as illustrated in Figure 3.19. This added step during finishing

did not have the desired effect and the columns required significant grinding afterwards in order to achieve smooth and well-rounded corners.

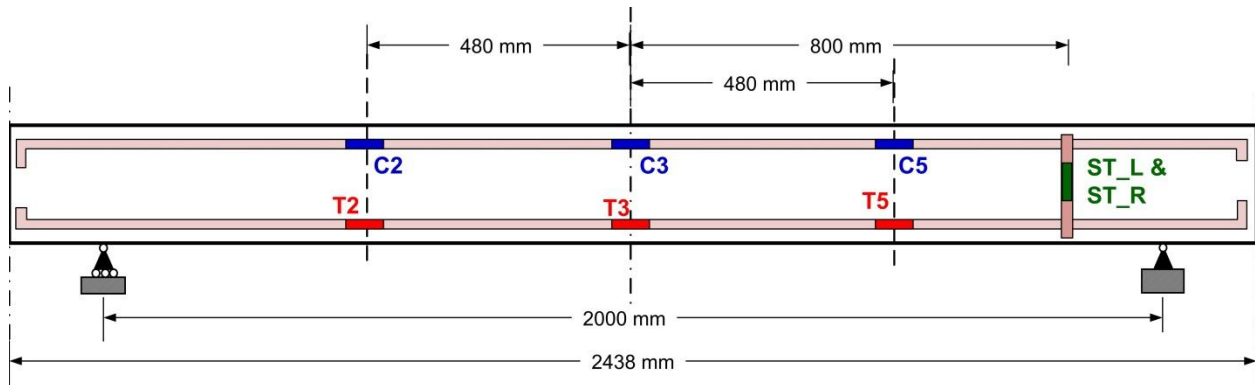


Figure 3. 16. Locations of strain gauges on longitudinal reinforcement

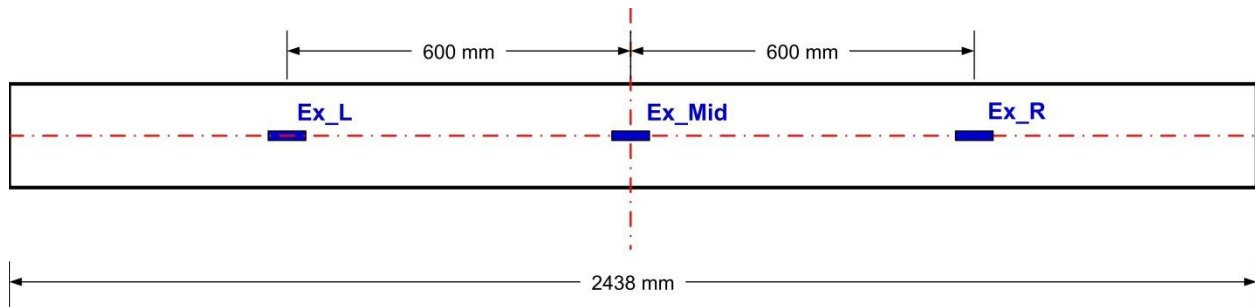


Figure 3. 17. Locations of strain gauges on the surface of column



Figure 3. 18. Protected strain gauge assembly and taped wires to the rebar

The specimens were kept under wet burlap and plastic sheets for the first fourteen days, after which the formwork was removed and the specimens were cured under wet burlap covered by plastic sheets for an additional seven days.



Figure 3. 19. Using of curbing trowel to smooth the top edges of all column specimens

3.4.4. Grinding the Surface of Specimens

The condition of the concrete surface plays an important role on strengthening elements with FRP laminates. The concrete should be capable of transferring flexural (and axial) stresses to the FRP through bond. The stress concentration in sharp concrete column corners can result in premature damage of the CFRP laminate. Hence rounding the corners is essential. In order to have the desired corner rounding, one option is to modify the formwork before casting, using a large bead of latex caulking in the bottom corners of the forms. However, it was further needed to smoothen the rounded corners with a diamond disk grinder.

3.4.5. CFRP strengthening of the column specimens

In this study, the CFRP laminates were applied all-around the column specimens where they were integrated with the column to form a full composite action. This can be envisioned to be similar to the concrete filled FRP tubes where the FRP tube contributes to both the confinement of concrete and the flexural resistance to the column. In such applications the effectiveness of FRP depends primarily on the stiffness and strength of FRP laminated “tube” and the bond between the concrete and the FRP.

The first step in the strengthening process involved patching of the pores on the surface of the concrete to enhance bond. This was performed using a mortar mix. A coat of mortar mix was applied using a brush, which remained 24 hours on the surface to dry before the application of FRP.

The FRP lamina was applied using the “wet layup” method with a resin having 3:1 epoxy to hardener matrix. The FRP fabric was applied to form different lamina on the smoothed and cured concrete surface. First the fabrics, either as uni-directional or woven sheets, were cut to desired widths and lengths. Epoxy resin was then applied to the middle portion of the column. Whether longitudinal or transverse (or bi-directional woven), the fabric was wrapped around the column, allowing at least 25 mm overlap to ensure strong epoxy-to-epoxy bond. Once the middle segment was completed, then the process was repeated for the two end segments. Figures 3.20 and 3.21 show the application of the CFRP lamina on the column ends where the overlap of the end portion over the middle portion was approximately equal to 70 mm. The FRP was cut at the location of exit point of strain gauge extension wires. The epoxy resin was applied carefully and a steel roller was used to bleed the excess resin and remove the entrapped air. Once the first laminate was completed, the second laminate was laid up immediately thereafter following the same procedure. After the application of all CFRP lamina, a sufficient quantity of epoxy resin was applied to achieve full saturation of fibers. Once the entire process was completed the specimens were left to dry for about 24 hours before they were relocated for storage.



Figure 3. 20. FRP is impregnated in resin and the air is rolled out from the saturated fabric



Figure 3. 21. Wrapping the stripped ends of a specimen

3.5. Measurements of Axial and Lateral Displacement

The measurements of axial and lateral displacements of column specimens during loading, under axial and lateral forces, were recorded using LVDTs (linear variable differential transformer). In this study ten LVDTs were used to measure displacements; seven points in the lateral direction ($LVDT_{lat1}$ to $LVDT_{lat7}$) and three points in the axial direction ($LVDT_{ax1}$ to $LVDT_{ax3}$), as shown in Figure 3.22. The displacement measurement points in the longitudinal (axial load) direction were taken at two points near the roller support end, and one at the hinge support side as demonstrated in Figure 3.22. The displacement measurement points in the lateral direction were: (i) one point at the mid span; (ii) two points at 381 mm from the mid span on each side; (iii) two points at 571 mm from the mid span on each side; and (iv) one point at each of the two lateral supports at 1100 mm from the mid span.

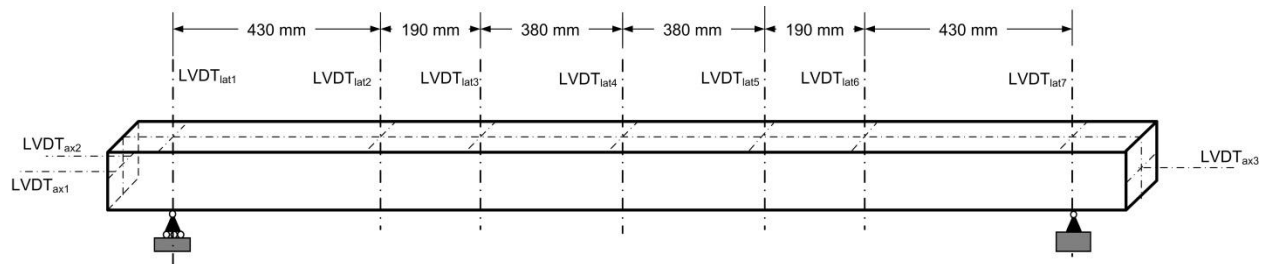


Figure 3. 22. Locations of the displacement sensors (LVDTs)

Chapter 4: Experimental Results

4.1. Introduction

This chapter provides the results of the experimental program, which involves tests of nine half-scale reinforced concrete (RC) columns under combined axial load and uniformly distributed lateral (UDL) load. The tests were performed at the structural laboratory of the National Research Council Canada (NRC Canada). They were tested to study the effects of CFRP laminates on the structural performance of RC columns under constant service gravity loads and incrementally increasing monotonic lateral loads. The following section provides load-displacement relationships, deflection profiles, recorded strains at selected locations, and observed damage for all the specimens. The column performance and comparisons of the effects of test parameters are presented at the end of the chapter.

4.2. Column Specimens and Loading Method

The test columns were designed to satisfy the seismic design provisions of CSA Standard A23.3-2004. The specimens were divided into the following five groups according to the CFRP laminate design:

- (i) Column group I (CG-I) included two control columns (Columns S5 and S6) where no FRP strengthening was applied.
- (ii) Column group II (CG-II) included one column (Column S1) where CFRP-L1 strengthening laminate was used with a fiber alignment sequence of $\{UD [0_2|90_2]\}$.
- (iii) Column group III (CG-III) included two columns (Columns S2 and S4) where CFRP-L2 strengthening laminates were used with a fiber alignment sequence of $\{W [0/90]_4\}$.
- (iv) Column group IV (CG-IV) included two columns (Columns S7 & S8) where CFRP-L3 strengthening laminates were applied with a fiber alignment sequence of $\{W[0/90]_2 W[\pm 45]_2\}$.
- (v) Column group V (CG-V) included two columns (Columns S3 & S9) where CFRP-L4 strengthening laminates were used with a fiber alignment sequence of $\{UD [0|90] W[\pm 45]_2\}$.

Table 4.1 summarizes the column groups with specimen label(s) for each group, and the laminate type used for strengthening. To simulate a column that is axially loaded to its maximum service load, and then subjected to ultimate lateral loads, the load is applied on the column in two stages:

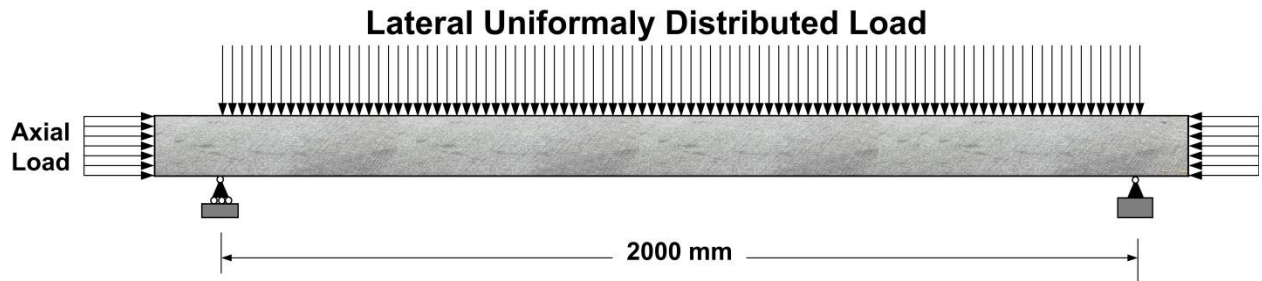
- (i) Axial load up to 45% of the concentric capacity of the control column; and
- (ii) Monotonically increasing uniform load up to failure.

The columns were oriented horizontally for testing, as illustrated in Figure 4.1. Each column was assumed to have pin-pin boundary conditions in the axial direction. The end supports for the lateral loads had a hinge at one end and a roller at the other end. NRC developed a two-dimensional quasi-static loading system, which enables the application of axial and lateral loads with any boundary condition, for a wide range of specimen dimensions and boundary conditions. For the present study, the testing system allowed: (i) free three-dimensional rotations in each of the two column ends; (ii) the roller (of the lateral support) enabled both the translation and the rotation of the column at the support point; and (iii) the hinge enabled free rotations of the column at the point of support while restricting it for translation. Figures 4.1 (a) and (b) show a control column and a CFRP strengthened column subjected to axial and uniformly distributed lateral loads for testing.

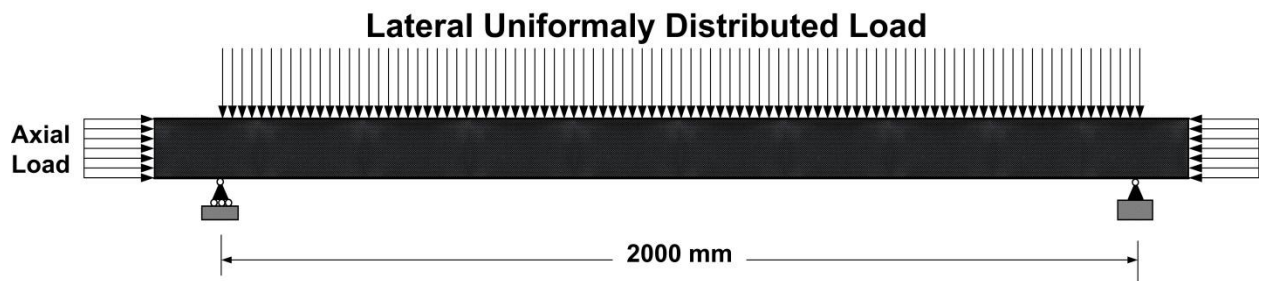
The columns were loaded in two stages: (i) axial load was applied up to the specified level, which was 45% of the concentric load capacity of the control column; and (ii) uniformly distributed lateral load was then applied up to failure. During the axial loading stage, it was observed that a lateral deformation was developed (see Figure 4.2 (a)). As the columns were tested horizontally, the lateral displacement is referred to “camber” in this study. This camber appeared as a negative lateral displacement. In the second stage of loading, the gradual increase in lateral load resulted in a decrease in the camber until the lateral deformation returned back to zero (see Figure 4.2 (b)). Figure 4.3 shows the camber reached 5 mm, and the value of the lateral load that was required to offset the camber and bring the column to its neutral position was equal to 27 kN.

Table 4.1: Columns and their CFRP Laminate protection and column group

Column Group	Column Name	Laminate Type
CG-I	Column-S5	Control Column
	Column-S6	Control Column
CG-II	Column-S1	CFRP-L1
CG-III	Column-S2	CFRP-L2
	Column-S4	CFRP-L2
CG-IV	Column- S7	CFRP-L3
	Column- S8	CFRP-L3
CG-V	Column-S3	CFRP-L4
	Column-S9	CFRP-L4



(a) Control column



(b) RC Column Strengthened by CFRP Laminate Wrapped around the column

Figure 4.1. Column subjected to axial load and uniformly distributed lateral load

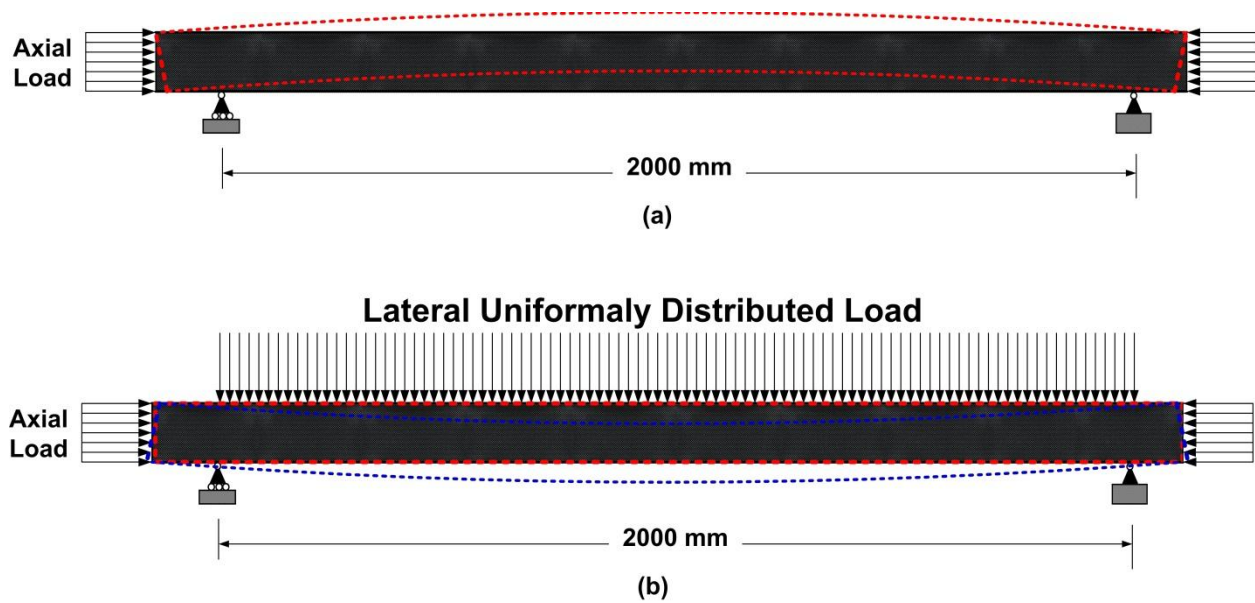


Figure 4.2. Effect of the loading stages on the column: (a) axial load causes camber; (b) initial lateral load is required to bring the column back to the state of zero lateral deformation

4.3. Column Group I: Control RC Column-1 (Column S5)

Figure 4.3 shows “Control Specimen-1” before the load application. An axial load was applied with a rate of 40 kN/min, up to a peak of 430 kN; then the load was held constant at this level. When the axial load reached 340 kN, it was observed that the concrete cover at column ends cracked. This is shown in Figures 4.4, and 4.5. Subsequently, a uniformly distributed lateral load was applied at a rate of 5 kN/min up to the column failure (5 kN is the integration of the UDL over the column length). Onset of spalling of the top concrete cover at mid-span (extreme compressive zone) is shown in Figures 4.6. The column reached its maximum capacity at a total lateral load of 120 kN and a mid-span displacement of 24 mm. The axial load at this stage of loading dropped to 260 kN. Figures 4.6 and 4.7 show the cracks within the mid-span zone when the lateral load was at ultimate. Both figures show that the depths of flexural cracks were less than the typical crack depths expected at ultimate flexural load of concrete elements subjected to lateral UDL only. It was also observed that several flexural cracks were located at mid-span, but shifted slightly towards the roller side. The cracks, however, were not distributed uniformly beyond the plastic hinge zone, as would typical be expected in beams subjected to lateral UDL only. The inclined shear-flexure cracks appeared away from the mid-span cracked region. On the other hand, the size of the critical compression zone at mid span was significantly larger than

that expected in a typical beam without any axial compression (See Figures 4.6 and 4.7). The column damage and the observed crack patterns can be explained by the effects of combined axial compression and flexure, where the compressive stresses covered a wide range of stress field over the column length.

Figure 4.8 shows the variation of lateral load and axial load under increasing lateral displacements. As the axial load increased to its maximum level the column cambered up, developing negative displacement. It was necessary to apply 26 kN of lateral load to attain zero camber as indicated by the horizontal line in Figure 4.8. The results showed an increase in column flexural stiffness after a lateral load of 40 kN and a displacement of 4 mm. The column stiffness decayed up to the yield lateral load of 105 kN corresponding to 11 mm deflection.

Figure 4.9 shows the locations of three strain gauges on tension longitudinal reinforcement; T2, T3, and T5. The strain gauge T3 was located at mid span, while T2 and T3 were located at 480 mm from the mid-span towards the roller and hinge supports, respectively (see Figure 3.16). Beyond yielding (around 2800 $\mu\text{m/m}$), the yield zone propagated towards the ends. Mid-span yielding was recorded at 22 mm of deflection. The maximum load of 120 kN was reached shortly thereafter at a lateral displacement of 24 mm. The lateral load resistance continued to decline until a displacement of 41 mm, at which stage the column was considered to have failed and the load was released. The axial load exhibited almost a linear drop with increasing lateral displacement, where it reduced from 430 kN to 260 kN.

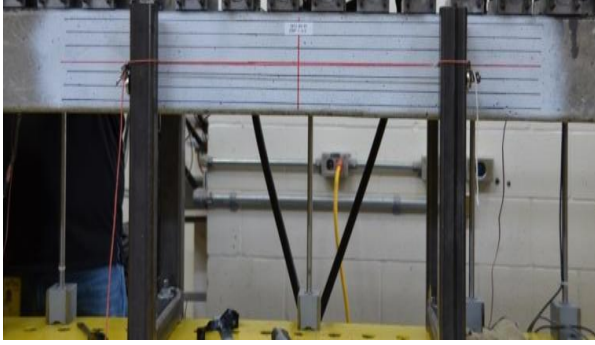


Figure 4.3. Column S5 prior to testing

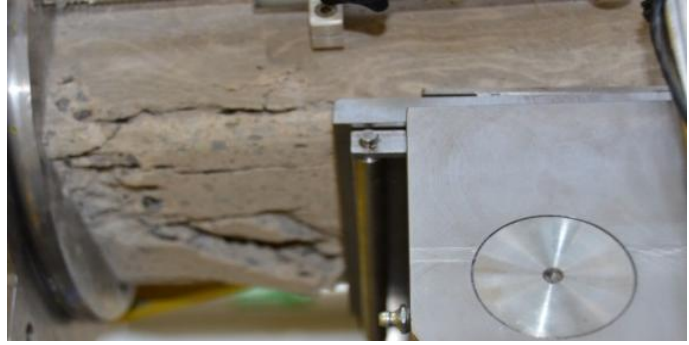


Figure 4.4. Damage to Column S5 near the roller support

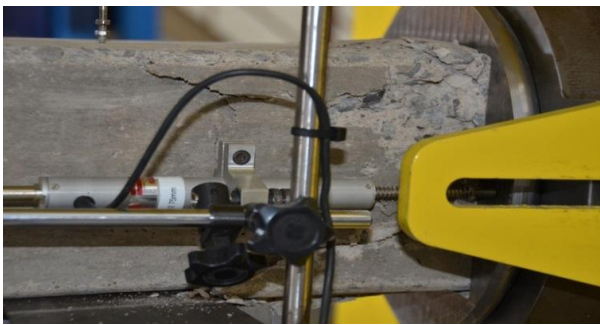


Figure 4.5. Damage to Column S5 near the hinge support

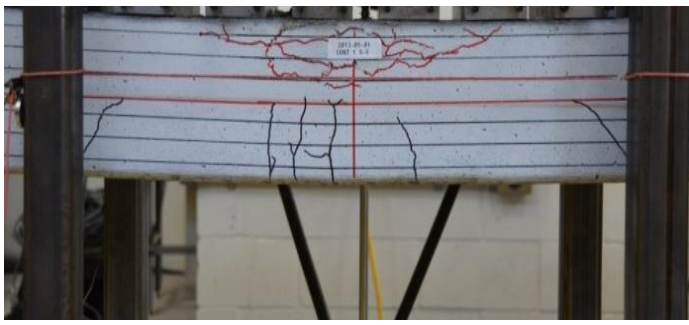


Figure 4.6. Damage on side face of Column S5 at maximum load

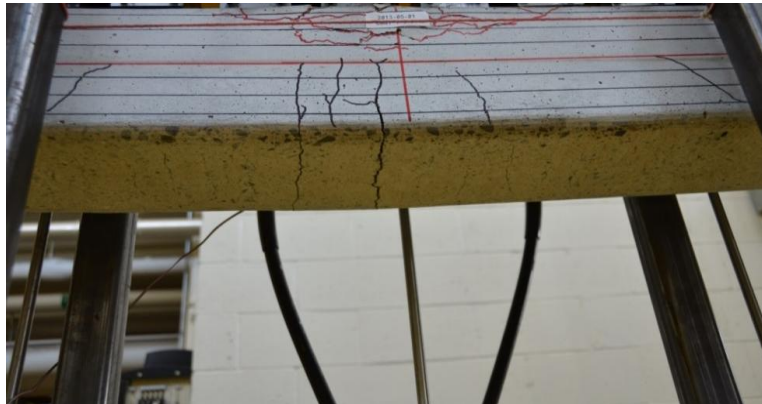


Figure 4.7. Damage to Column S5 at maximum load

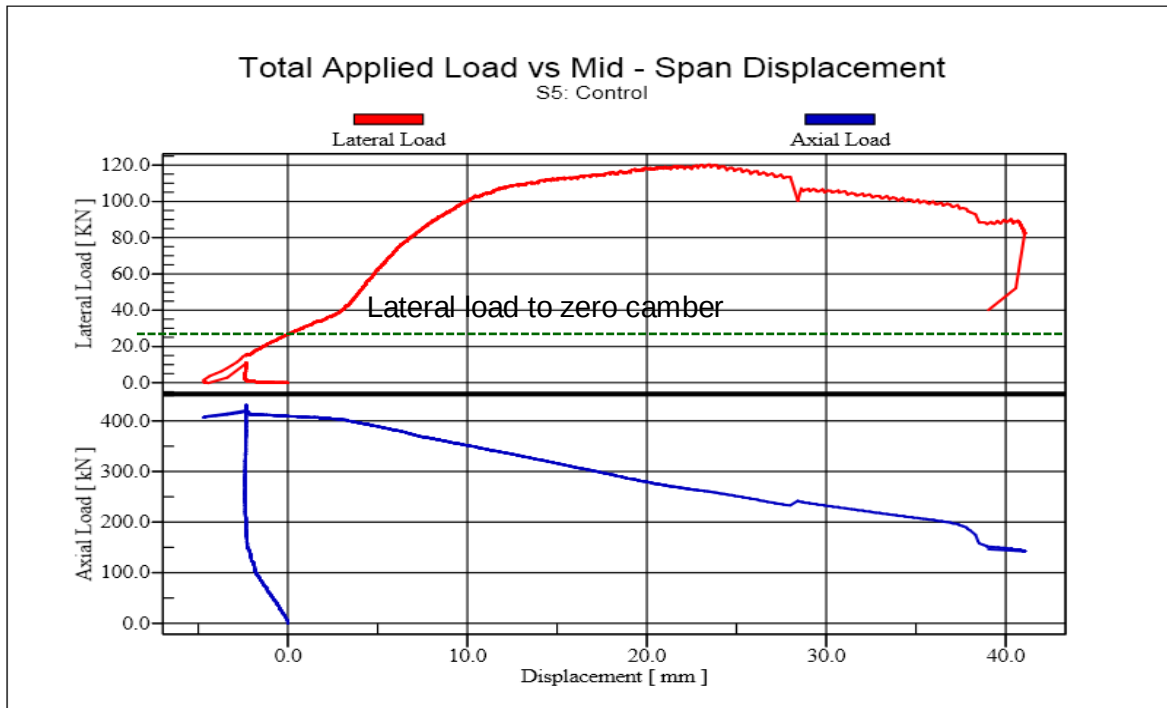


Figure 4.8. Variation of lateral load and axial load with mid-span deflection (Column S5)

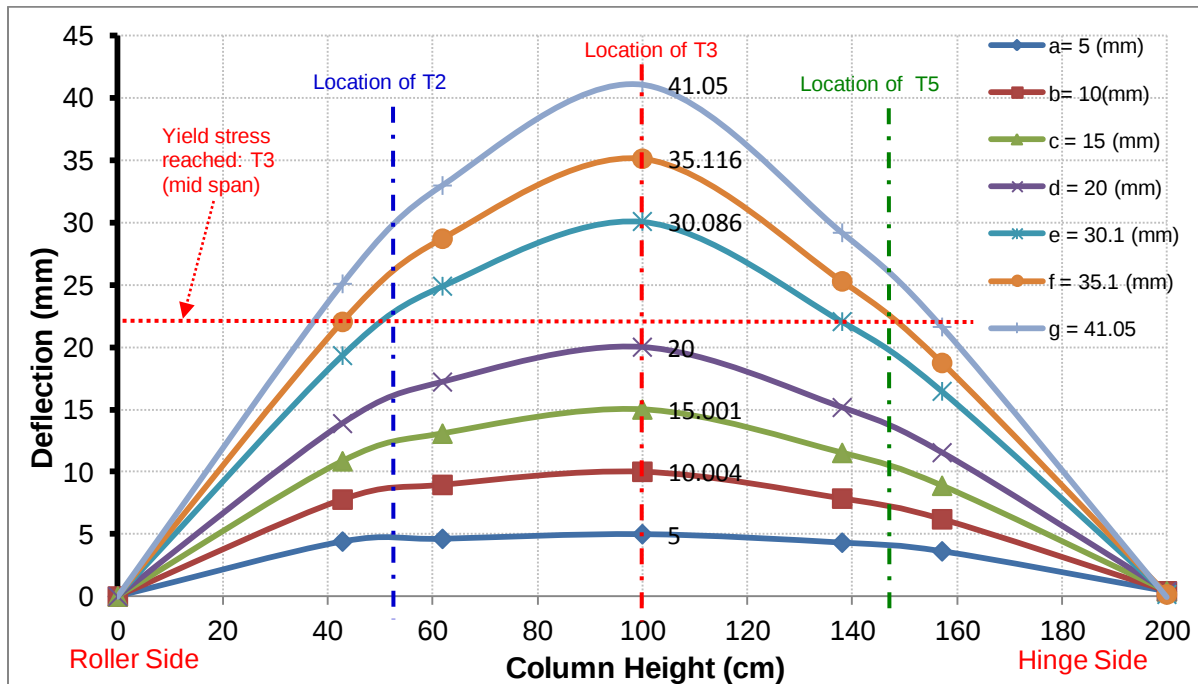


Figure 4.9. Deflection profiles at selected stages of loading (Column S5)

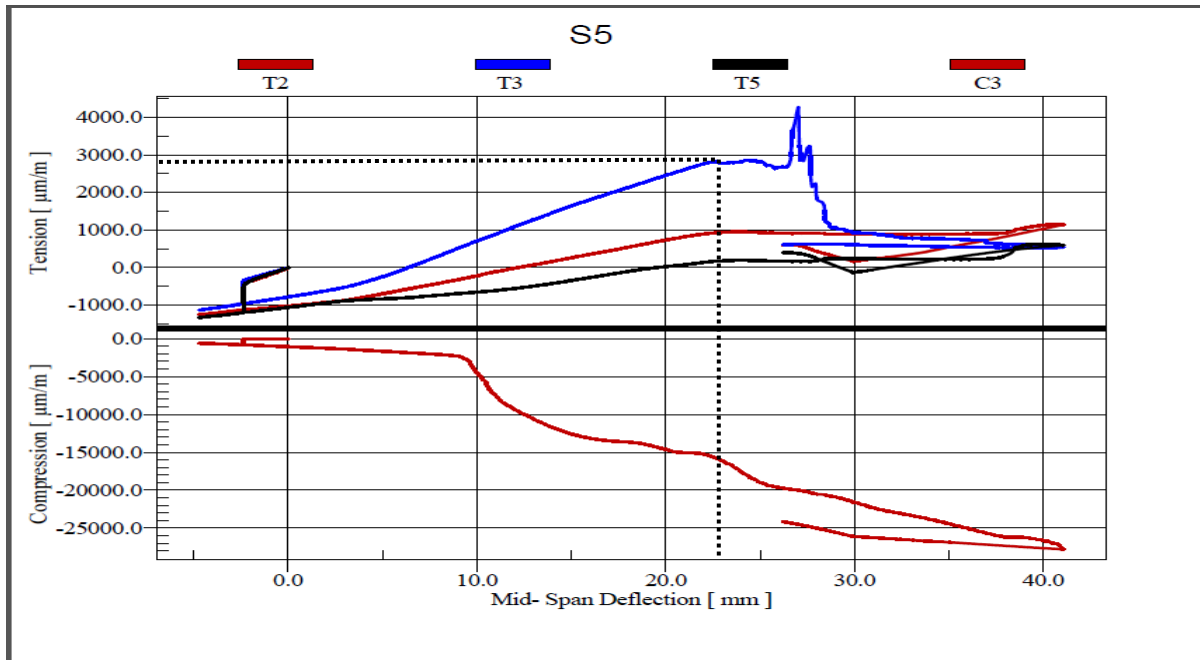


Figure 4.10. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S5)

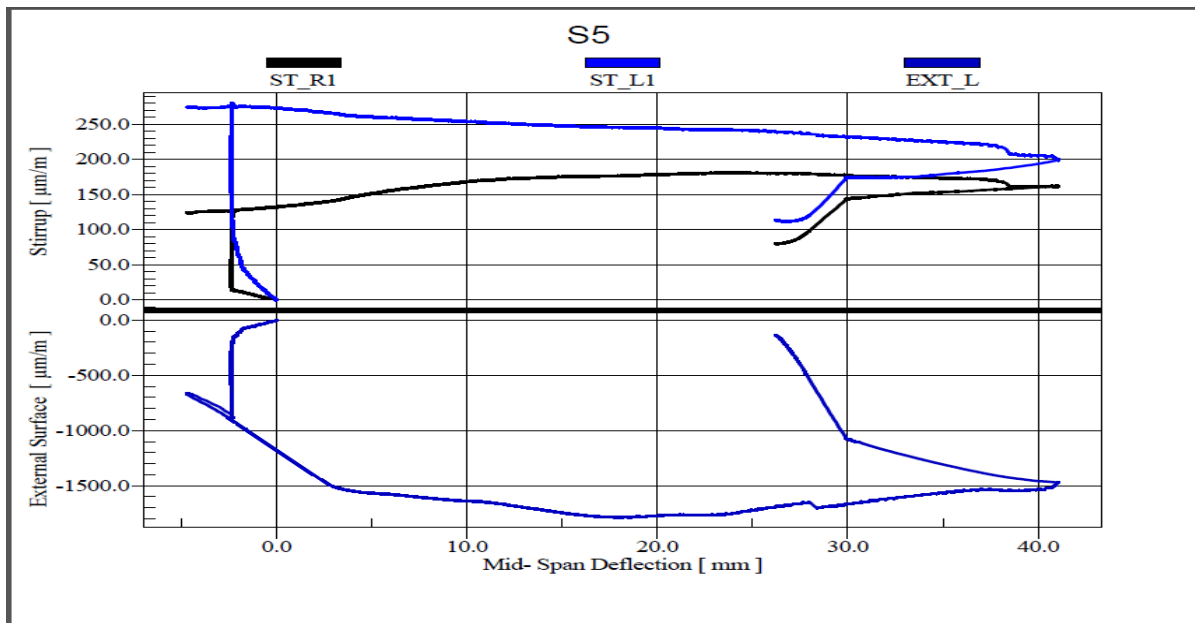


Figure 4.11. Strains in stirrups and top concrete surface at mid-span (Column S5)

Tie strains were recorded by gauges ST-R1 and ST-L1, and are shown in Figure 4.11. The maximum tie strains recorded were very low (only 180.9 $\mu\text{m/m}$ in ST-R1; and 274.2 $\mu\text{m/m}$ in

ST-L1) indicating low shear stresses in the region. The only recorded strain on concrete was at location Ex-L where it showed a strain of 1800 $\mu\text{m/m}$ before the gauge stopped functioning.

4.4. Column Group I: Control Column-2 (Column S6)

Column S6 was the second control column tested. The ends of the column near the axial load application region were wrapped by uni-directional CFRP, in the transverse direction, to confine these regions to avoid damage to the cover concrete, which was observed in the companion control Column S5. Figure 4.12 shows the column prior to the load application. At maximum applied axial load of 430 kN, the column ends remained damage free with the exception of limited cracks observed near the FRP retrofitted regions. This is shown in Figures 4.13, 4.14 and 4.15. Subsequently, a uniformly distributed lateral load was applied at a rate of 5 kN/min up to the column failure (5 kN is the integration of the UDL over the column length). Spalling of concrete cover along the top face near the mid-span was observed in the extreme compression zone. This is shown in Figures 4.16 and 4.17. The column reached its maximum capacity at a total lateral load of 138 kN, and remained fluctuating around 130 kN as the displacements increased from 11 mm to 20 mm. The axial load dropped from 430 kN to 365 kN at 20 mm lateral displacement. It was observed at this load stage that the strain in the strain gauge T3 at mid span had not reached yield, yet the load remained constant. This was attributed to the compression crushing under combined axial and flexural compression.

Figures 4.16 and 4.17 show the damage and crack patterns observed in the mid-span region when the lateral load reached the ultimate level. The column behaviour was similar to that for Control Column-1, except for the wider cracks observed in Column 6. Figure 4.18 shows the variation of lateral load and axial load with mid-span displacements. The initial camber due to the application of axial compression was 5 mm. It was required to apply 35 kN of lateral load to bring the column to its neutral (zero deflection) position. This is indicated in Figure 4.18 by a horizontal line. The results show an increase in column flexural stiffness after a lateral load of 50 kN and a displacement of around 3 mm. Then the column stiffness declined nonlinearly up to yield. The strain gauge T3 located at mid-span reached yield strain at around 22 mm of deflection. The lateral load resistance continued decreasing as displacements increased to 44 mm at mid-span when the column was considered to have failed and the load was released. Figure 4.18 shows the drop in axial from 430 kN to 150 kN. Figure 4.19 shows displacement profiles under increasing

lateral loads. The deflected shape and the ultimate capacity of the column were similar to those observed for the companion control column S5.

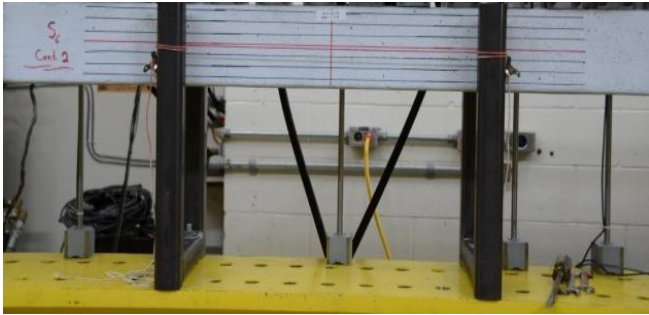


Figure 4.12. Column S6 prior to testing

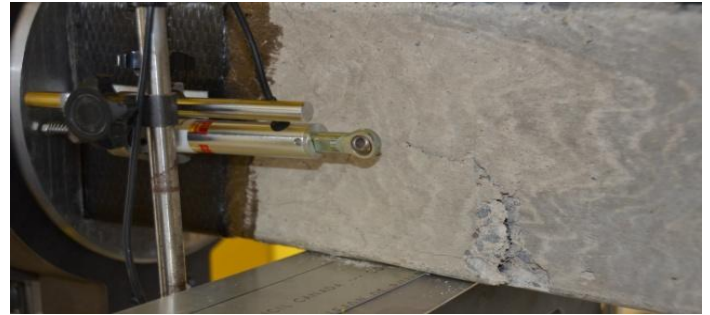


Figure 4.13. Damage to Column S6 near the roller support



Figure 4.14. Damage to Column S6 near the roller support



Figure 4.15. Damage to Column S6 near the hinge support

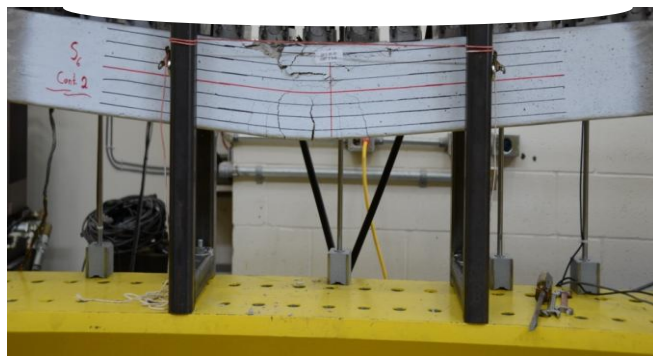


Figure 4.16. Damage on side face of Column S6 at maximum load

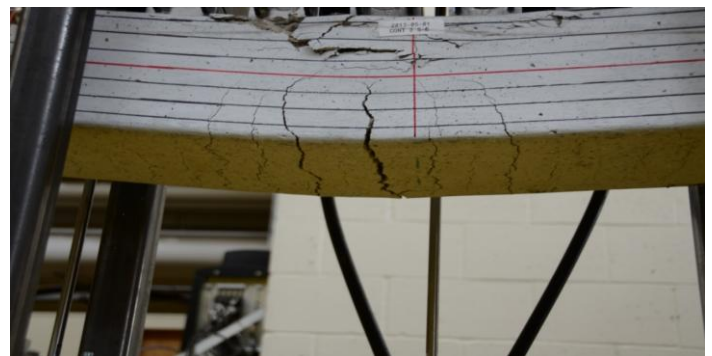


Figure 4.17. Damage to Column S6 at maximum load

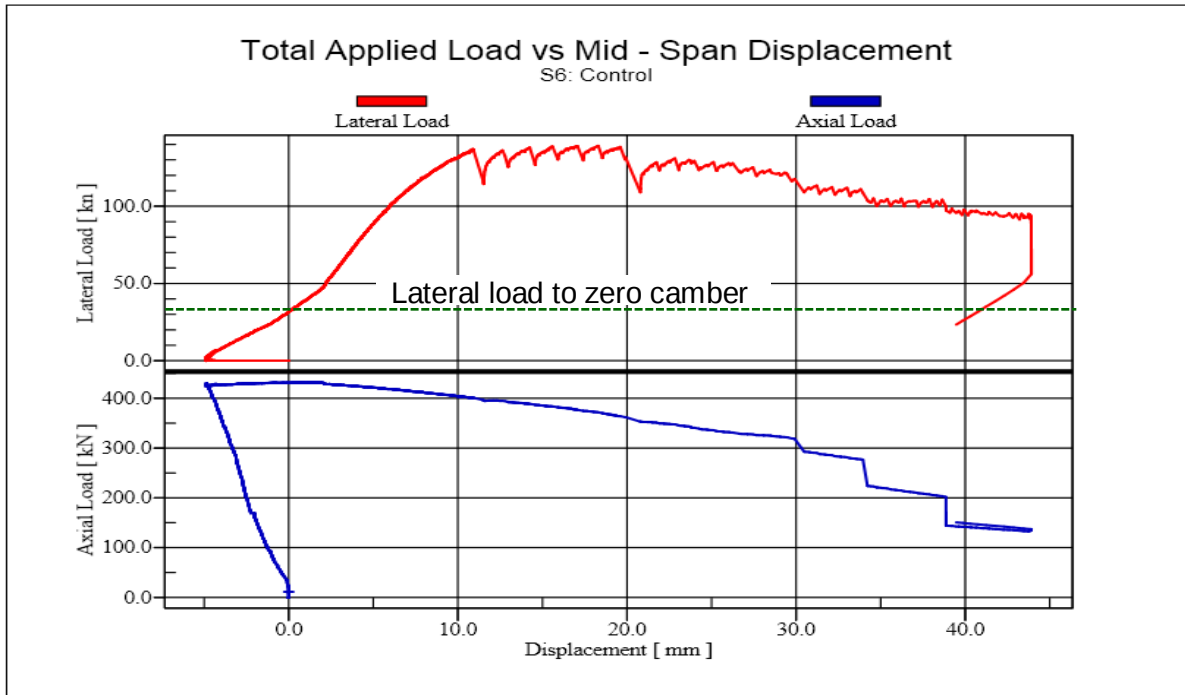


Figure 4.18. Variation of lateral load and axial load with mid-span deflection (Column S6)

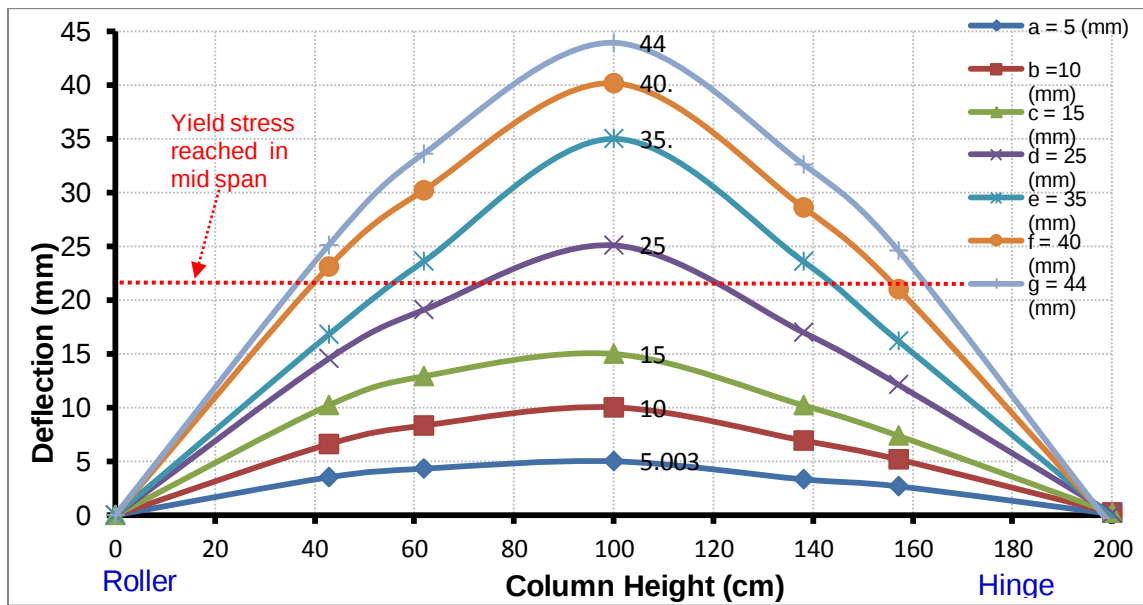


Figure 4.19. Deflection profiles at selected stages of loading (Column S6)

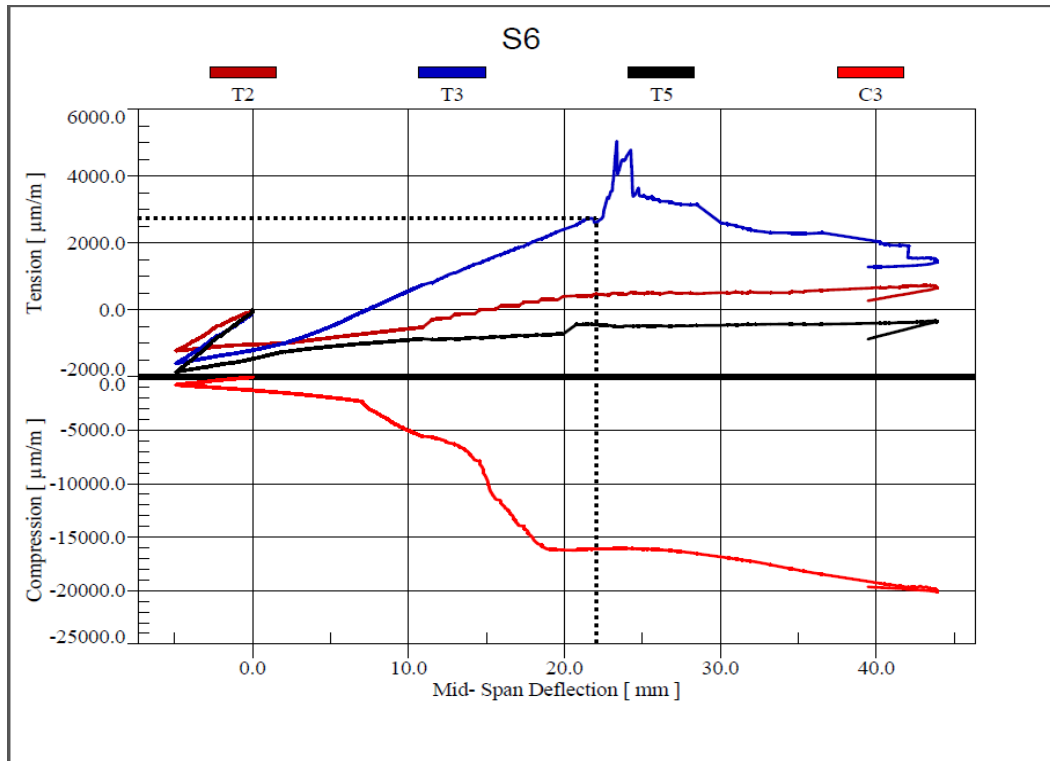


Figure 4.20. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S6)

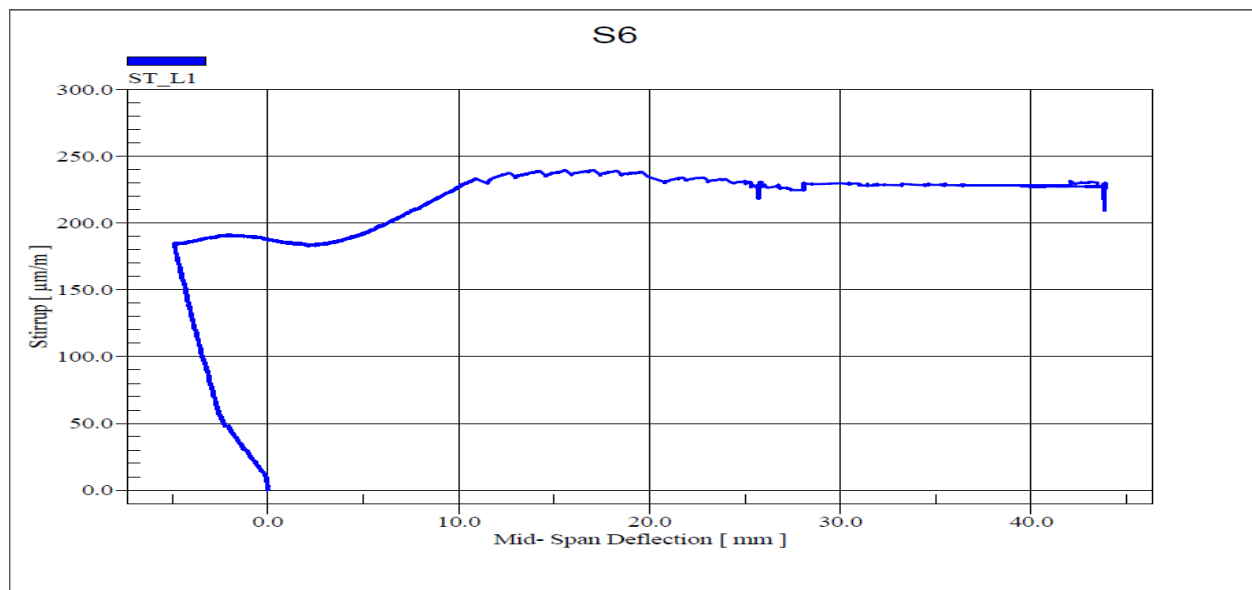


Figure 4.21. Strains in stirrups and top concrete surface at mid-span (Column S6)

Figure 4.20 shows tensile and compressive strains recorded on longitudinal reinforcement. The results indicate that limited yielding was observed in tension, and the buckling of compression reinforcement was evident from the strain readings, which was later confirmed by visual observations. Figure 4.21 shows strain readings taken by gauges ST-L1, indicating very low strains, as also observed in the companion control column S5.

4.5. Column Group II: Column strengthened with CFRP-L1 (Column S1)

This column specimen was retrofitted with CFRP Laminate 1 (or CFRP-L1) which had two layers of unidirectional 0° , and two layers of unidirectional 90° laminates. Figure 4.22 shows the strengthened column specimen before the application of load. An axial load of 430 kN was first applied and held constant. A uniformly distributed lateral load was then applied at a rate of 5kN/min up to column failure. No cracks or damage to concrete was observed. Similarly, no damage to CFRP was observed up to failure. However, loud noise was heard from time to time, indicating the rupturing of FRP. The column failed at a total of 240 kN lateral load at mid-span deflection of 63 mm. The axial load dropped from 430 to 165 kN at a displacement of 20 mm. The failure was sudden, with very loud sound. Figures 4.23 and 4.24 show the failure stage of column where the CFRP laminate ruptured vertically at the mid span. The depth of rupture was 77 mm.

Figure 4.27 shows the variation of lateral load and axial load with mid-span deflection. The initial camber due to the application of axial load was 2 mm. It was required to apply 30 kN lateral load to bring the column back to zero deflection. This level of load is indicated in Figure 4.27 by a horizontal line. The results indicated an increase in column flexural stiffness up to 110 kN of lateral force at a displacement of 6 mm. The column stiffness decreased thereafter in a nonlinear manner as the load approached ultimate.

The strain gauge T3 located on tension bar at mid-span reached yield strain at about 23 mm of deflection. Then the strain increased to a very high level ($16000 \mu\text{m/m}$) at a displacement of 33 mm, when the lateral load reached 200 kN and the axial load dropped to 350 kN. The lateral load resistance was gradually lost as lateral displacement increased to 77 mm. At this stage of loading the column was considered to have failed, and the test was discontinued. Figure 4.27 shows the loss in axial load from 430 kN to 165 kN with an increase in lateral displacement.

Figure 4.28 depicts displacement profiles under increasing lateral load. Figure 4.29 shows the compressive strain measured on compression reinforcement for (C2), as well as the tensile steel strains measured on tension reinforcement under increasing lateral deflection. The strain gauges at mid-span indicated early yielding and increased excessively as the ultimate load was approached. The other two gauges on either side of the columns also reached yield but only near the column failure. It appears that the CFRP laminate contributed significantly to the column strength and the confinement of concrete. The plastic hinge length increased due to the presence of CFRP.

Figure 4.30 shows strain measurements on column ties, as well as on the concrete surface. The strains in transverse reinforcement remained low, but the compressive strain on concrete increased to 8000 $\mu\text{m/m}$. The increased inelastic strains and the propagation of the plastic hinge length were attributed to the presence of CFRP sheets in both directions.

4.6. Column Group III: Column strengthened with CFRP-L2 (Column S2)

This column was retrofitted with CFRP Laminate 2 (or CFRP-L2) which was formed from four woven laminas of 0° - 90° orientation. Figure 4.31 shows the strengthened column specimen before the application of load. Similar to the previous column specimen, an axial load was applied gradually and held at 430 kN. A uniformly distributed lateral load was then applied at a rate of 5 kN/min up to the column failure. No cracks or damage was observed in concrete. Similarly no damage to CFRP was observed up to failure. However, a loud noise was heard near the end of testing. The column reached its ultimate capacity at 160 kN of total lateral load and a mid-span deflection of 40 mm. The lateral load was then dropped to 70 kN with a maximum deflection of 47 mm. Figure 4.35 shows the failure of the column specimen and the rupture of the FRP at failure. The CFRP laminate fracture initiated at the lower face of the column and propagated upward up to point (A) in Figure. 4.35, and then turned horizontally to the right for approximately 170 mm. When the laminate fracture reached point (B) in the figure, it extended vertically at the same time where the fracture at point (A) propagated to the top face of the column (the face of the lateral load application) where it turned again to the left as shown in Figure 4.35. The axial load dropped from 430 kN to 215 kN. Figures 4.34 and 4.35 show the

failure of the column where CFRP-L2 ruptured, forming a wide crack in concrete. No damage was observed near the ends (see Figures 4.32 and 4.33).

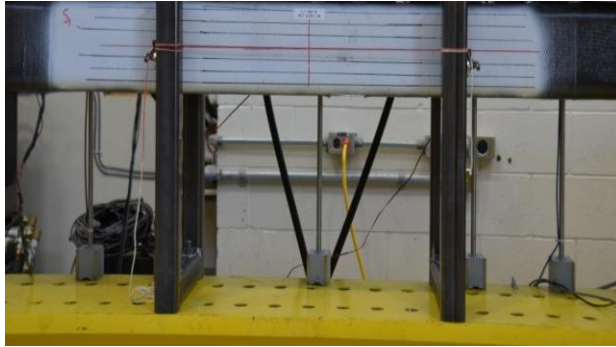


Figure 4.22. Column S1 prior to testing



Figure 4.23. Damage to Column S1 near the roller support

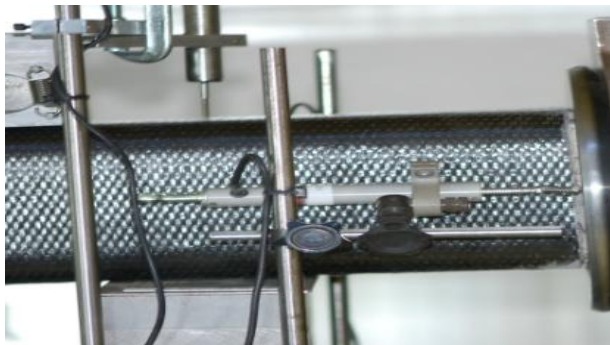


Figure 4.24. Damage to Column S1 near the hinge support

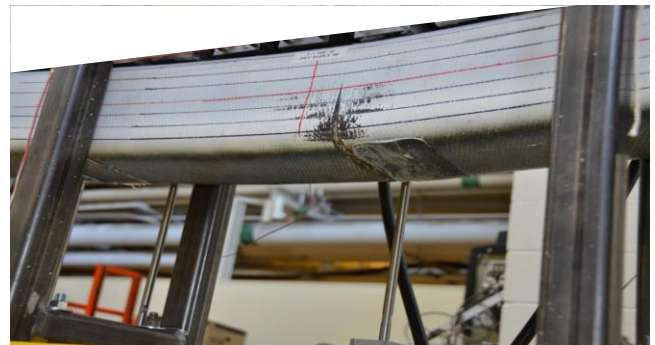


Figure 4.25. Damage on side face of Column S1 at maximum load

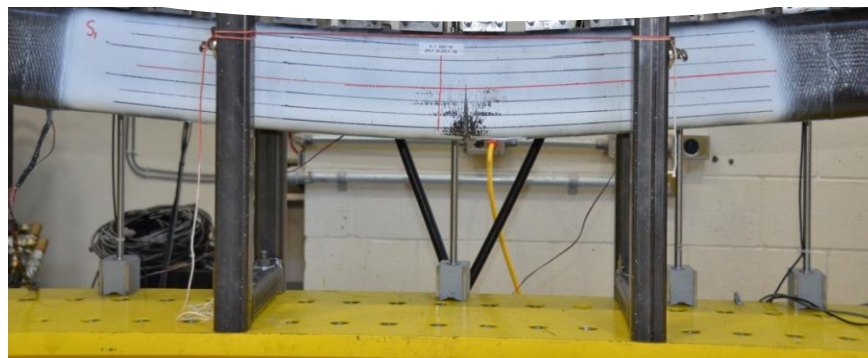


Figure 4.26. Damage to Column S1 at maximum load

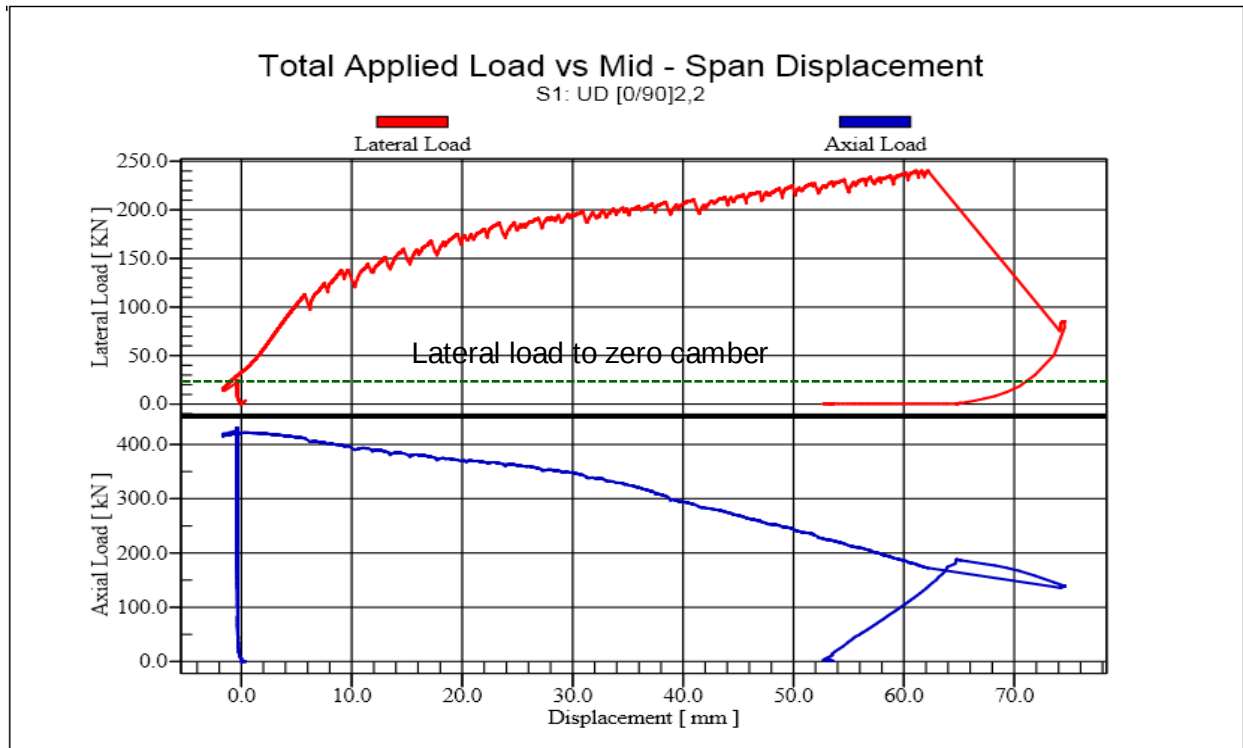


Figure 4.27. Variation of lateral load and axial load with mid-span deflection (Column S1)

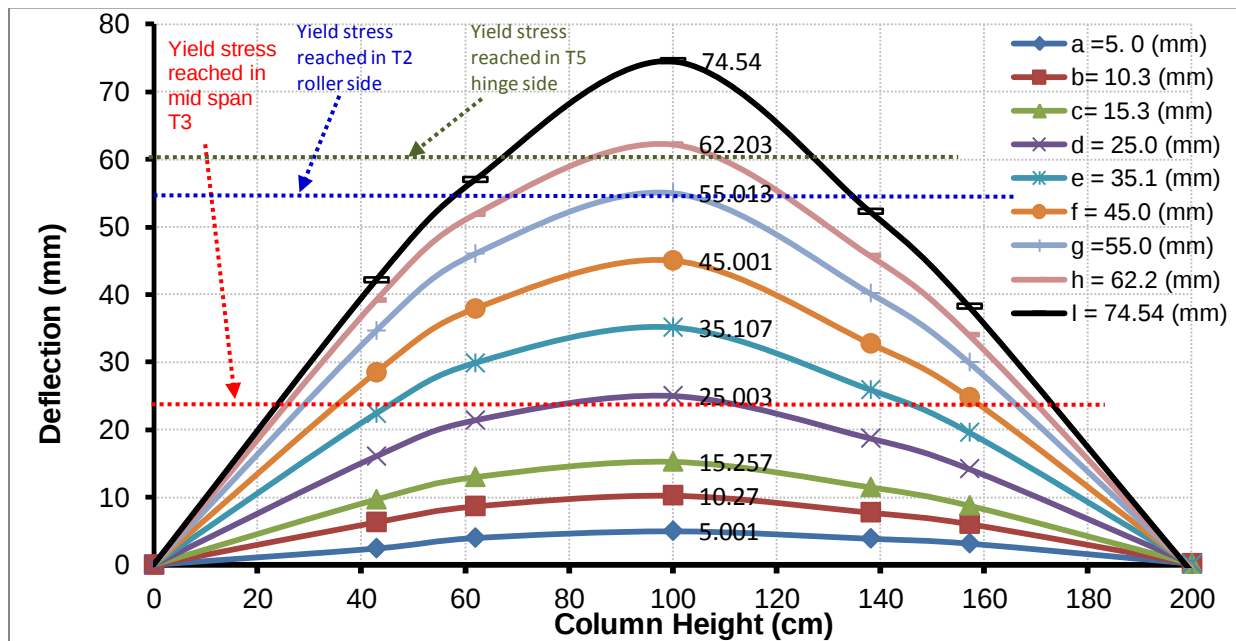


Figure 4.28. Deflection profiles at selected stages of loading (Column S1)

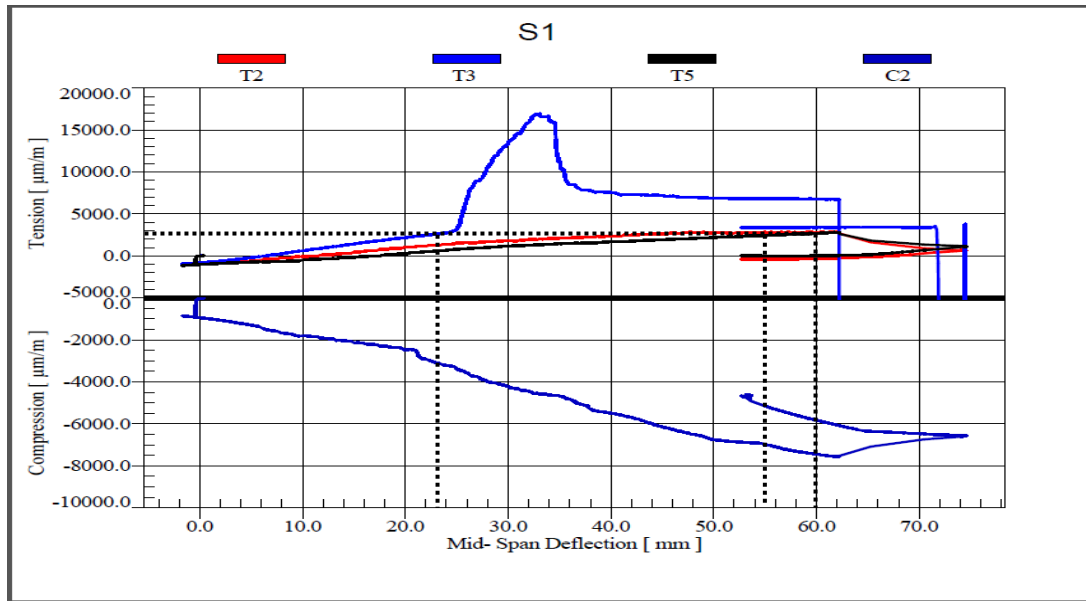


Figure 4.29. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S1)

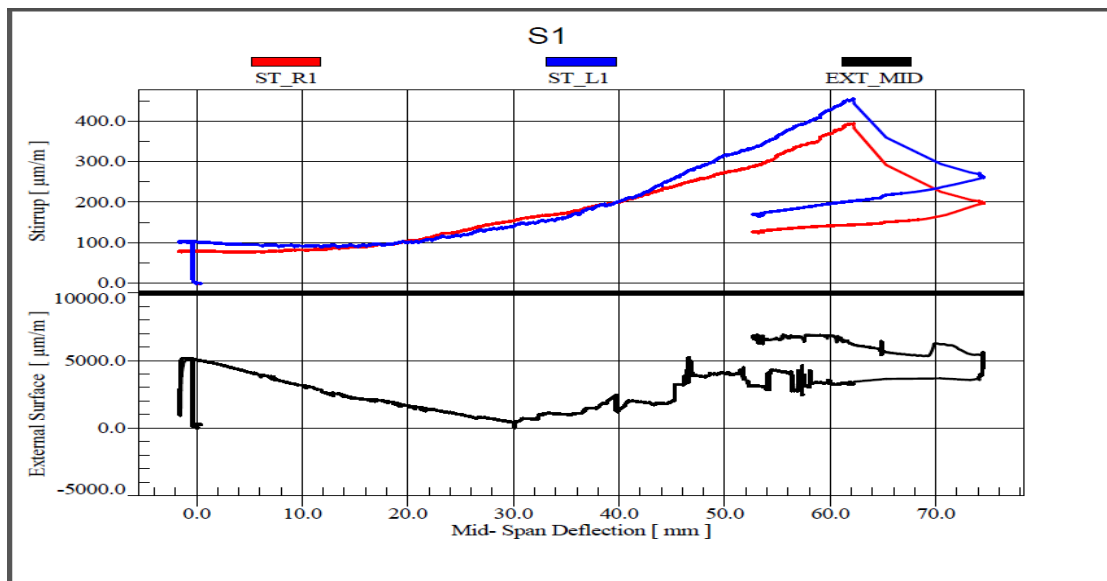


Figure 4.30. Strains in stirrups and top concrete surface at mid-span (Column S1)

Figure 4.36 shows the variation of lateral loads and axial load with mid-span deflection. The initial camber due to the axial load was relatively small, and was measured to be 1.5 mm. The required lateral load to bring the column deflection back to zero was 18 kN. Similar to the column strengthened with CFRP-L1, the load deflection relationship showed an increase of column flexural stiffness up to a load of 120 kN corresponding to a displacement of 10 mm. The column stiffness showed a sharp decrease after a mid-span deflection of about 40 mm. The strain gauge T3 located at the mid-span was lost at this stage. The strain gauge readings of T3, taken on the identical companion column, are shown in Section 4.7. The lateral load resistance continued dropping with a large increase in lateral displacements up to 47 mm of mid-span deflection where the load was then released and the column was considered to have reached failure. Figure 4.36 shows approximately linear reduction in axial load from 430 kN to 210 kN with increasing lateral displacement.

Figure 4.37 shows displacement profiles of column at different stages of loading, up to failure. Figure 4.38 shows tension and compression strains on longitudinal reinforcement as a function of lateral deflection, recorded by gauges T2 and T5 on the tension side, and C2, C3, and C5 on the compression side. The strains on tension reinforcement started with small compressive strains (-1000) $\mu\text{m/m}$, and then became tensile strains as the lateral load increased. The strains in tension never reached yield strain, and remained at a maximum of 2300 $\mu\text{m/m}$. On the other hand the compressive strain at mid span (C3) reached very high strains (-15000) $\mu\text{m/m}$ at column failure. The strain gauge on the compression reinforcement, near the roller support (C2) reached (-6000) $\mu\text{m/m}$. The strain gauge C5 showed approximately constant readings during the entire test (at -2000 $\mu\text{m/m}$).

Figure 4.39 shows tie strains (gauges ST-R1 and ST-L1), with very low values, as well as the surface strains recorded on the compression side, reaching -3500 micro strains.

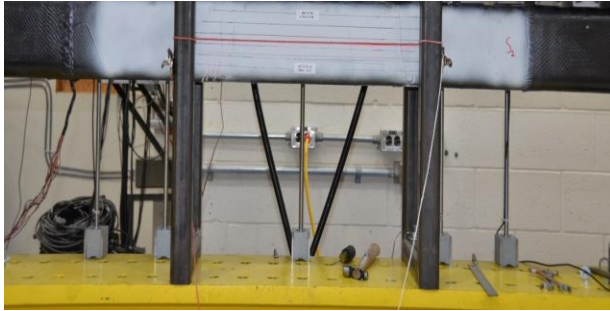


Figure 4.31. Column S2 prior to testing

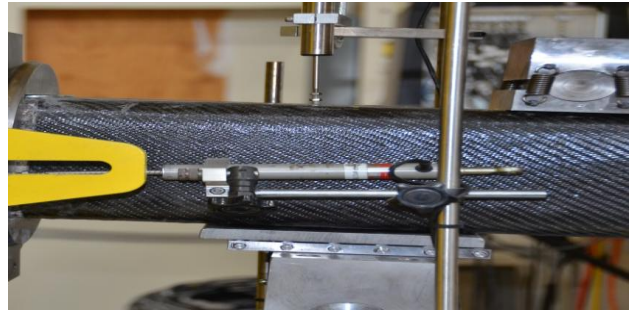


Figure 4.32. Damage to Column S2 near the roller support

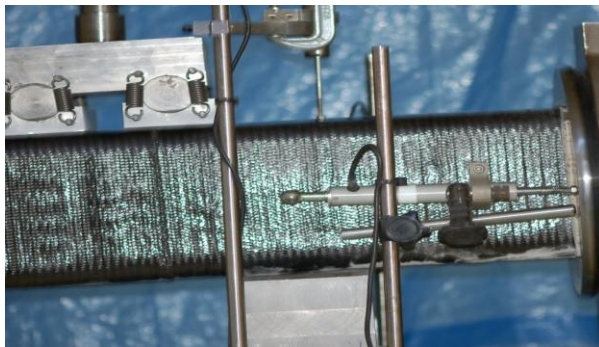


Figure 4.33. Damage to Column S2 near the hinge support

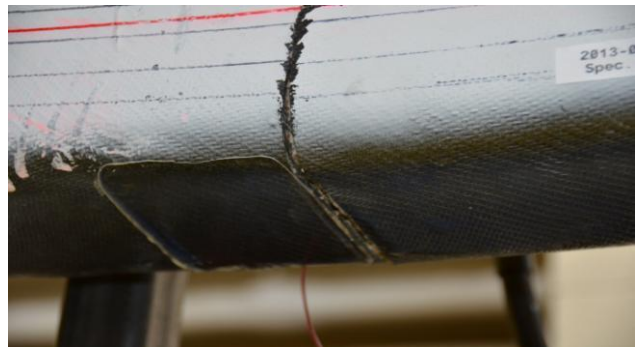


Figure 4.34. Damage on side face of Column S2 at maximum load

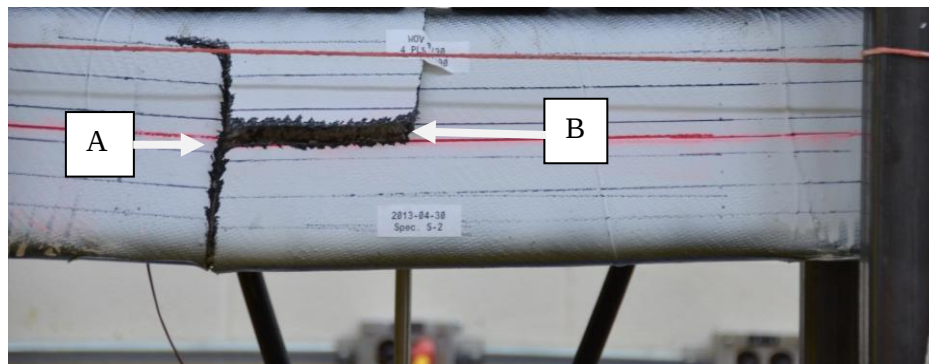


Figure 4.35. Damage to Column S2 at maximum load

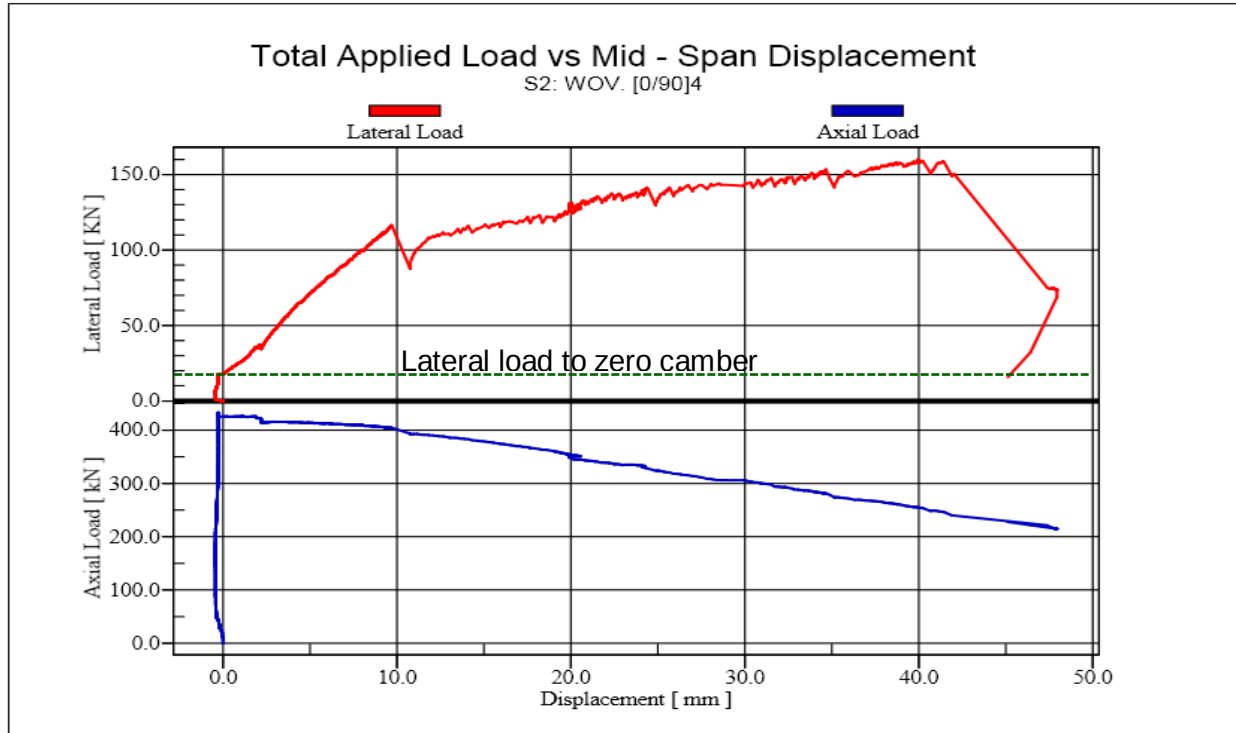


Figure 4.36. Variation of lateral load and axial load with mid-span deflection (Column S2)

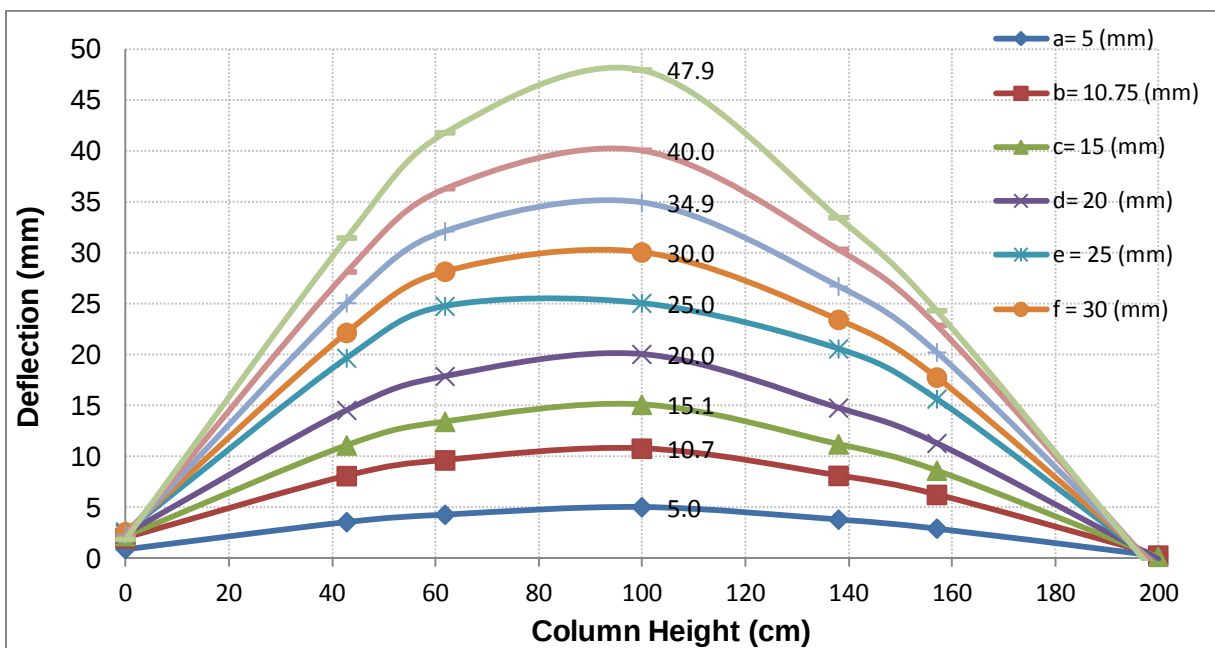


Figure 4.37. Deflection profiles at selected stages of loading (Column S2)

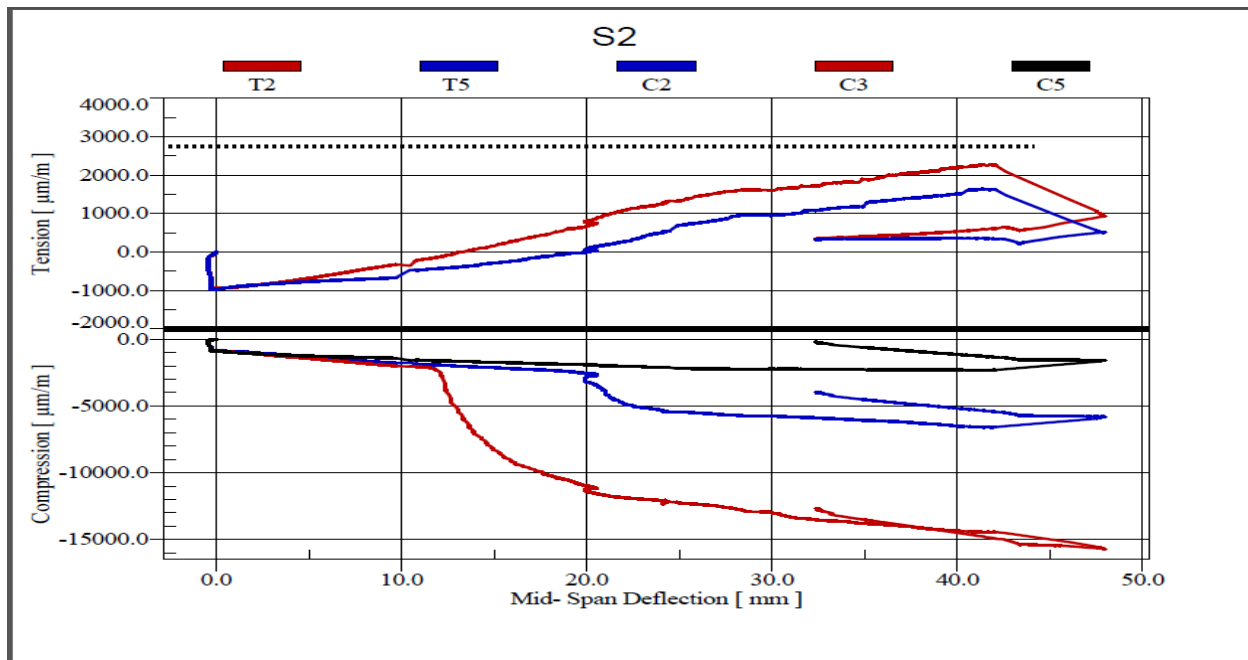


Figure 4.38. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S2)

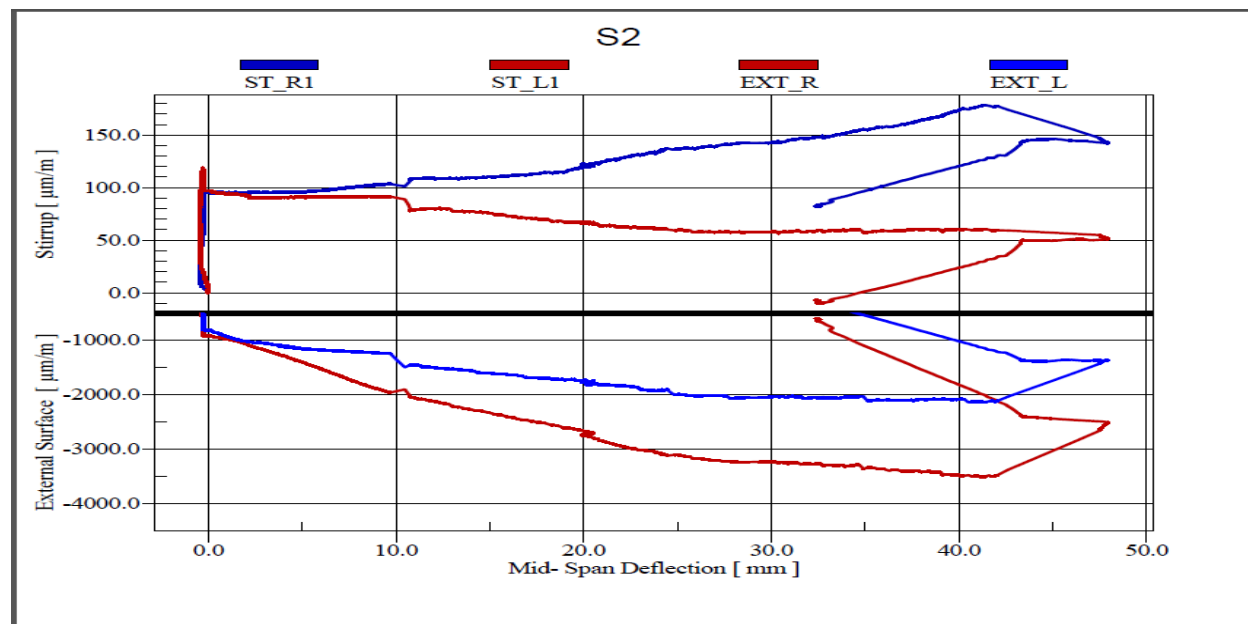


Figure 4.39. Strains in stirrups and top concrete surface at mid-span (Column S2)

4.7. Column Group III: Column strengthened with CFRP-L2 (Column S4)

This column was retrofitted with CFRP Laminate 2 (or CFRP-L2) which consisted of four woven laminas in 0° - 90° orientation. Figure 4.40 shows the strengthened column specimen prior to testing. Similar to other columns, an axial load was applied first and held constant at 430 kN. A uniformly distributed lateral load was then applied at a rate of 5 kN/min up to the column failure. No cracks or damage to CFRP was observed. Similarly, there was no damage to the concrete inside until failure. However, loud noise was heard from time to time. The column reached its ultimate capacity at a lateral load of 165 kN and the corresponding mid-span deflection of 31 mm. The lateral load then dropped to 98 kN, which subsequently recovered to 115 kN as the mid-span deflection increased to 39 mm, but gradually dropped back to 102 kN. The CFRP started rupturing vertically as shown in Figures 4.43 and 4.44. The axial load dropped from 430 kN to 216 kN. The column was not damaged near the ends. The end regions at column failure are illustrated in Figures 4.41 and 4.42.

Figure 4.45 shows lateral and axial load versus mid-span displacement relationships. In the first phase of loading, the axial load was applied without any camber in the column due to any accidental eccentricity, as was the case in some of the earlier columns. The column flexural stiffness increased up to 120 kN of lateral load at a corresponding displacement of 7 mm. The column experienced stiffness decay and an associated sudden reduction of load at a deflection of 8-9 mm, at which time a loud noise was heard, followed by a load recovery up to 130 kN.

The strain gauge T3, located at mid-span, reached yield strain at around 25 mm of deflection. Then the strain increased to $11500 \mu\text{m/m}$ at a displacement of 27 mm when the lateral load reached 200 kN and the axial load dropped to 300 kN. At this stage of loading, the extreme tensile strain reduced but remained constant at $5000 \mu\text{m/m}$ up to a deflection of 30 mm. It then dropped to the yield level at a deflection of 35 mm, and stayed at this level up to failure. The lateral load resistance continued decreasing with large increases in displacement, reaching up to 58.2 mm at mid-span. The column reached failure at this stage of loading. Figure 4.43 indicates approximately linear reduction in axial load (from 430 kN to 216 kN) with increasing lateral displacement.

Figure 4.46 shows displacement profiles at selected stages of loading. Figure 4.47 shows: (i) tensile steel strains on longitudinal tension steel, T2, T3, and T5; and (ii) three compressive strains on compression reinforcement (C2, C3, and C5) as a function of mid-span deflection. The three strain gauges on tension reinforcement started at $-1500 \mu\text{m/m}$ (compression) due to the cambering of the specimen, and became tension as the lateral load increased. The mid-span deflection at which mid span tensile strain (T3) reached the yield strain level is shown as horizontal red dotted line in Figure 4.46. While the strain at mid-span strain gauge (T3) increased at a faster rate, the others (T2 and T5) at the two ends increased at a lower rate and never reached the yield strain, though they approached the yield value. On the other hand, the compressive strain at mid span (C3) reached a very high strain value of $-10000 \mu\text{m/m}$ when the column capacity was reached, and remains at this level. The strain gauge on compression reinforcement on the roller support side (C2) reached $-8000 \mu\text{m/m}$ and fluctuated between this value and $-6000 \mu\text{m/m}$ at a deflection of 20 mm, and dropped to $-4000 \mu\text{m/m}$ at a deflection of 35 mm. The strain gauge C5 remained below yield, with a strain value of around $-2000 \mu\text{m/m}$.

Figure 4.48 shows that the strains in gauges ST-R1 and ST-L1 on column ties (the locations of the strain gauges are shown in Figure 3.16) recorded very low values. The Gauge EXT-Mid on the exterior compression face was lost during testing; the other two gauges on FRP indicated reading of about $-3500 \mu\text{m/m}$.

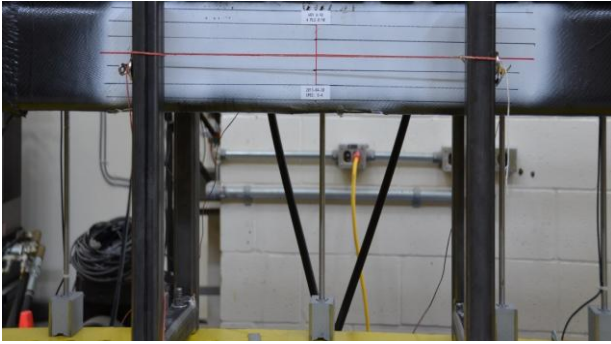


Figure 4.40. Column S4 prior to testing

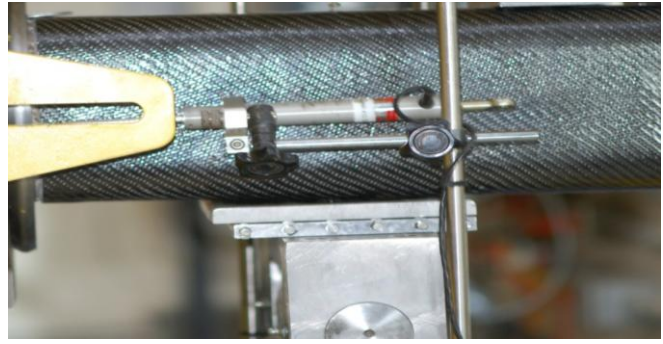


Figure 4.41. Damage to Column S4 near the roller support



Figure 4.42. Damage to Column S4 near the hinge support

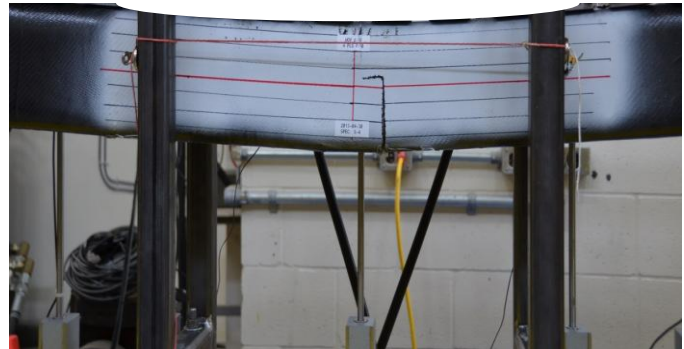


Figure 4.43. Damage on side face of Column S4 at maximum load



Figure 4.44. Damage to Column S4 at maximum load

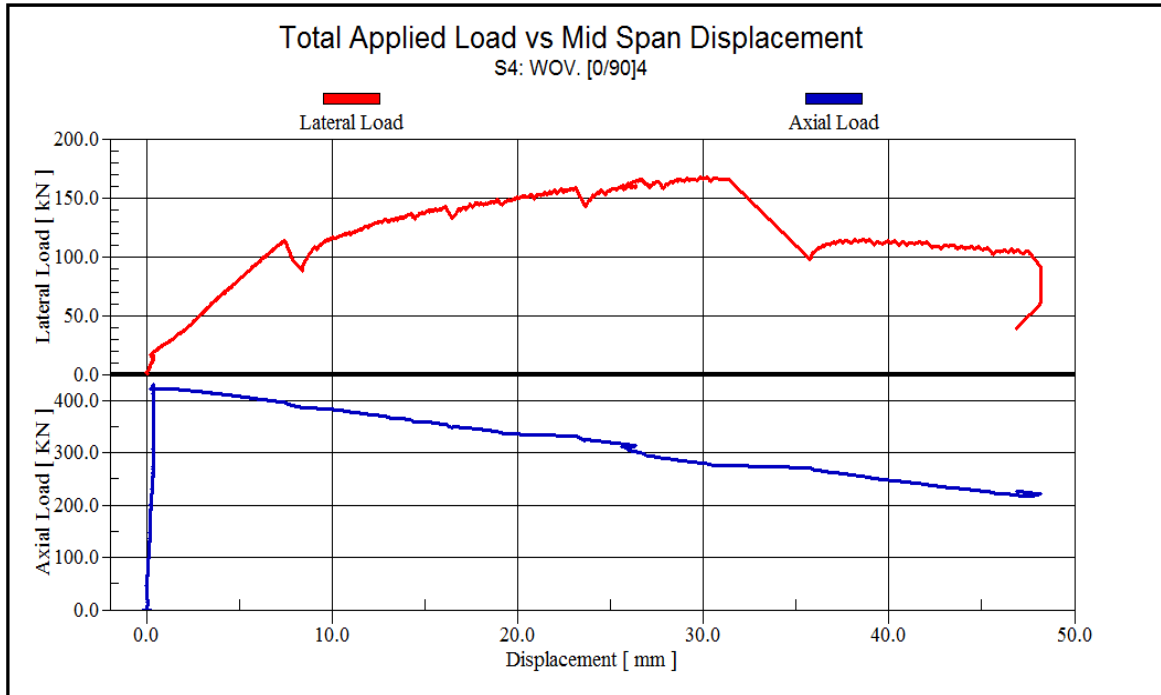


Figure 4.45. Variation of lateral load and axial load with mid-span deflection (Column S4)

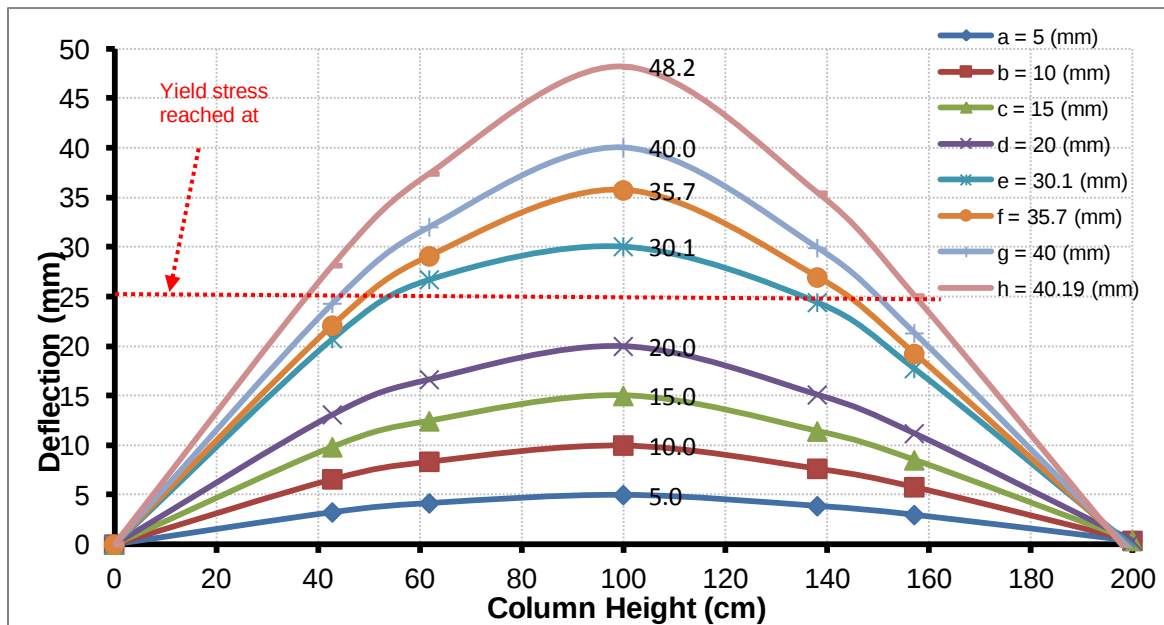


Figure 4.46. Deflection profiles at selected stages of loading (Column S4)

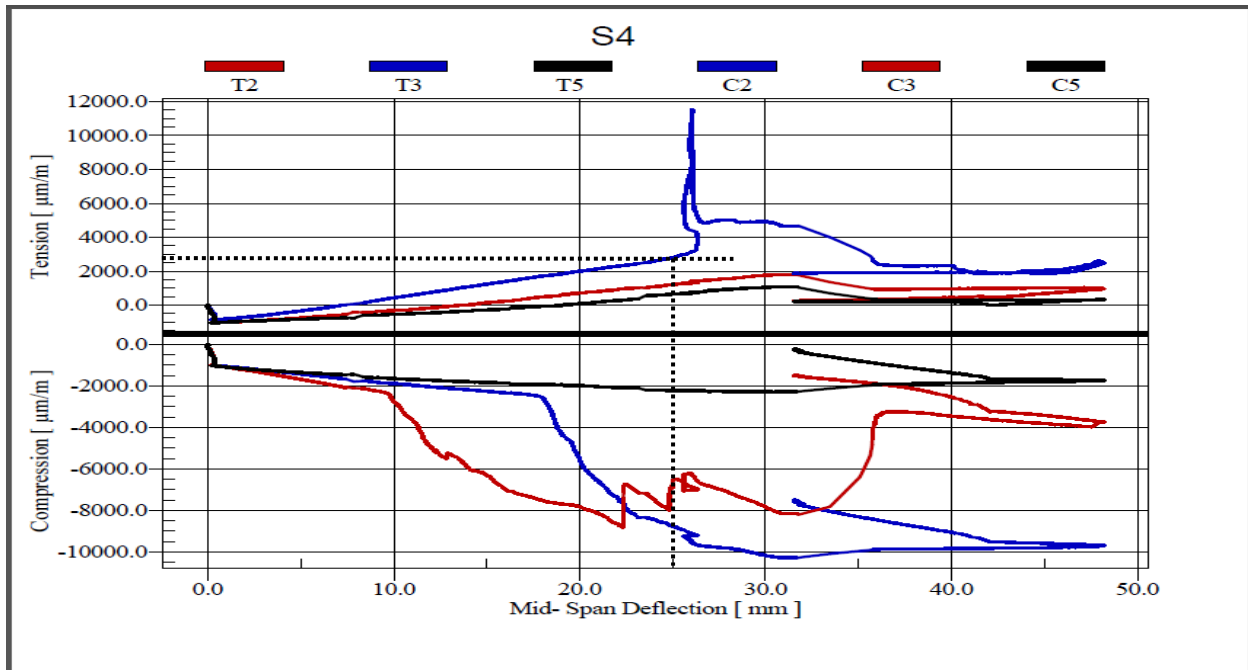


Figure 4.47. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S4)

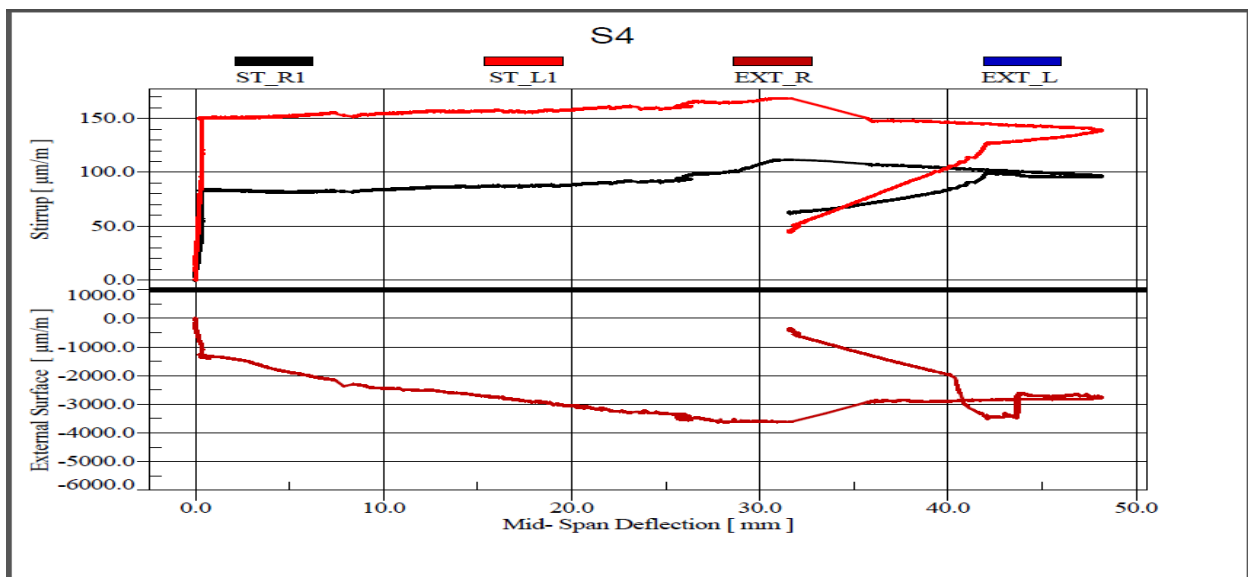


Figure 4.48. Strains in stirrups and top concrete surface at mid-span (Column S4)

4.8. Column Group IV: Column Strengthened with CFRP-L3 (Column S7)

Column S7 was retrofitted with CFRP Laminate 3 (or CFRP-L3) which consisted of two laminas of woven 0° - 90° and two laminas of woven $\pm 45^{\circ}$. Figure 4.49 shows the strengthened column specimen prior to testing. As in the case of previous columns, the axial load was applied gradually and held at 430 kN before the uniformly distributed lateral load was applied at a rate of 5 kN/min up to the column failure. At maximum applied axial load of 430 kN, the column ends remained damage free as illustrated in Figures 4.50 and 4.51. No cracks or damage to the CFRP or the concrete was observed during loading prior to failure. However, loud noise was heard from time to time, while the crippling of CFRP was observed on the top compression face.

The column reached its ultimate capacity at a total of 154 kN lateral load, at a mid-span deflection of 55 mm. The lateral load then dropped until the maximum deflection of 57 mm was reached. The vertical rupture of the CFRP-L3 started perpendicular to the column length at the bottom face, near the mid-span, propagated towards the column end after forming a -45° inclination and then continuing horizontally. Figures 4.52 and 4.53 show the crack pattern. The axial load dropped from 430 kN to 170 kN as the mid-span deflection increased. Figures 4.52 and 4.53 show the failure of the column upon the formation of wide crack at the rupture zone.

Figure 4.54 shows the variation of lateral and axial loads with mid-span deflection. When the axial load was applied the column developed an upward camber, which was brought back to zero with the application of 20 kN of lateral load. The load deflection relationship shows an ascending branch up to a load of 110 kN and a displacement of 8 mm. The stiffness became nonlinear after passing three small decrease-recovery steps between 8mm and 21 mm deflections with a loud noise heard at each load drop followed by recovery.

The strain gauge T3 was lost immediately after developing 28 mm of deflection. The strain gauge T2 reached yield strain at 55 mm of deflection. The axial load dropped linearly up to column failure, reaching 180 kN at failure. The lateral load resistance in this column continued increasing at a lower rate up to a mid-span deflection of 57 mm, at which point the test was discontinued.

Figure 4.55 shows the displacement distributions over column length at selected stages of loading. Figure 4.56 shows that tensile steel strains recorded by gauges T2, T3, and T5; and

compressive strains recorded by C2, C3, and C5 on tension and compression reinforcement, respectively. The three strain gauges on tension reinforcement started developing compressive strains of $-1300 \mu\text{m/m}$ initially, but they changed to tension readings as lateral load increased. The yield level of mid-span strain gauge (T2) is illustrated with a horizontal blue dotted line in Figure 4.55. The strains recorded by gauge (T5) did not show yielding, though they were very close to the yield level. On the other hand the compressive strain at mid span (C3) reached a relatively low strain of $-3000 \mu\text{m/m}$ at a deflection of 8 mm. The strain gauge on compression reinforcement near the roller support side (C2) only recorded $-2000 \mu\text{m/m}$ at a deflection of 20 mm, and then stayed constant up to column failure. The strain gauge C5 reached $-2500 \mu\text{m/m}$ at a deflection of 21 mm, and stayed constant up to column failure.

Figure 4.57 depicts strain gauge recordings for ST-R1 and ST-L1 on column ties (the locations of which are shown in Figure 3.16). These readings were very low, indicating little straining in column ties. The external strains recorded on column surface by gauge EXT_Mid increased linearly up to $-6000 \mu\text{m/m}$ at a displacement of 28 mm and stayed constant up to the column failure. The two other exterior strain gauges, Ext_R and Ext_L, developed an increase in strains up to $-3300 \mu\text{m/m}$ and $-1900 \mu\text{m/m}$, respectively.

4.9. Column Group IV: Column Strengthened with CFRP-L3 (Column S8)

Column S8 was retrofitted with CFRP Laminate 3 (or CFRP-L3), which consisted of two laminas of woven 0° - 90° and two laminas of woven $\pm 45^\circ$. Figure 4.58 shows the strengthened column specimen prior to loading. An axial load was applied gradually up to 430 kN, followed by a uniform lateral load at a rate of 5 kN/min up to the column failure. No cracks or damage to CFRP and the concrete inside was observed until failure. Loud noise was heard from time to time and the crippling of CFRP laminate was observed on the compression face. The column reached its ultimate capacity at a total load of 158 kN and a mid-span deflection of 56 mm. Subsequently, the lateral load dropped as deflections increased to 58 mm. The rupturing of CFRP-L3 followed the same pattern as that for Column S7, starting at the bottom face, as a vertical crack was formed and propagated towards one end of the column with a -45° degree inclination, followed by a horizontal pattern. Figures 4.61 and 4.62 illustrate the crack pattern. The axial load dropped from 430 kN to 174 kN. No damage was observed near the column

support regions. This is illustrated in Figures 4.59 and 4.60. As the compression FRP continued rippling, three small bumps of FRP (of about 3 mm height) appeared on the compression face near mid-span as shown in Figure 4.63. This signified local loss of bond between the CFRP and the concrete surface.

Figure 4.64 shows the variation of lateral and axial loads with mid-span deflection. The column cambered up by about 2 mm after the application of axial load due to the presence of accidental eccentricities. A total lateral load of 35 kN was required to bring the initial deflection back to zero.

The load-deflection relationship shows elastic behaviour up to 90 kN load and 6 mm mid-span deflection. The column then experienced inelastic behaviour after experiencing two sudden loss of strength at about 20 mm and 28 mm deflections, accompanied by loud noise and immediate force recoveries of 115 kN and 130 kN, respectively.

The strain gauge T3 on tension reinforcement reached yield strain at around 25 mm of deflection. Subsequently, the strain increased significantly to 13000-14000 $\mu\text{m/m}$ as the displacements increased while the axial load decreased from 350 kN to 174 kN at column failure. The lateral load resistance increased slightly up to a mid-span displacement of 56 mm, at which stage the load started dropping suddenly, marking the failure point of the column.

Figure 4.65 shows column displacement profiles at selected stages of loading with yield strains recorded on tension bars indicated by red and blue dotted lines. Figure 4.66 shows strains recorded on tension reinforcement by gauges T2, T3, and T5, as well as those recorded on compression reinforcement through gauges C2, C3, and C5. The gauges on tension reinforcement initially recorded about -1000 $\mu\text{m/m}$ compressive strain, and then measured tensile strains as the lateral load increased. While the maximum tensile strain at mid span increased significantly, the strains recorded by gauges T2 and T5 developed yield strains only after a deflection of 50 mm, just before column failure. On the other hand the compressive strain at mid span recorded by gauge C3 reached very high values, in excess of -20000 $\mu\text{m/m}$ before the load capacity was reached.

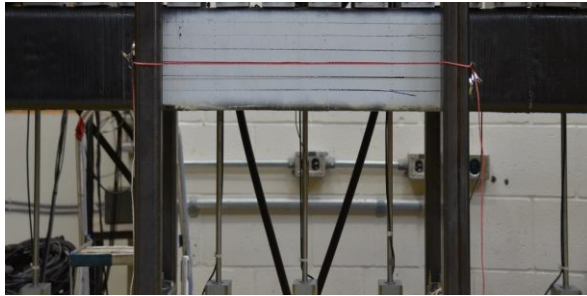


Figure 4.49. Column S7 prior to testing



Figure 4.50. Damage to Column S7 near the roller support

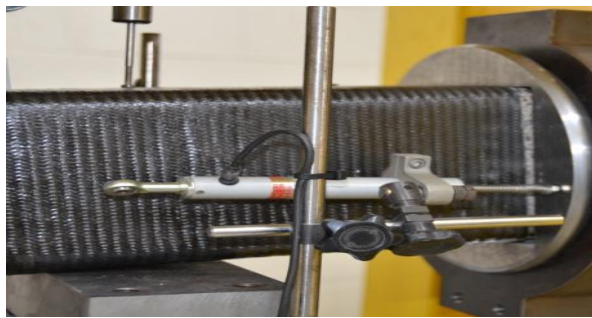


Figure 4.51. Damage to Column S6 near the hinge support

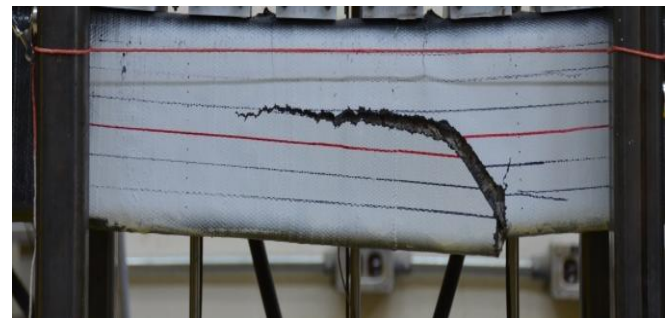


Figure 4.52. Damage on side face of Column S7 at maximum load

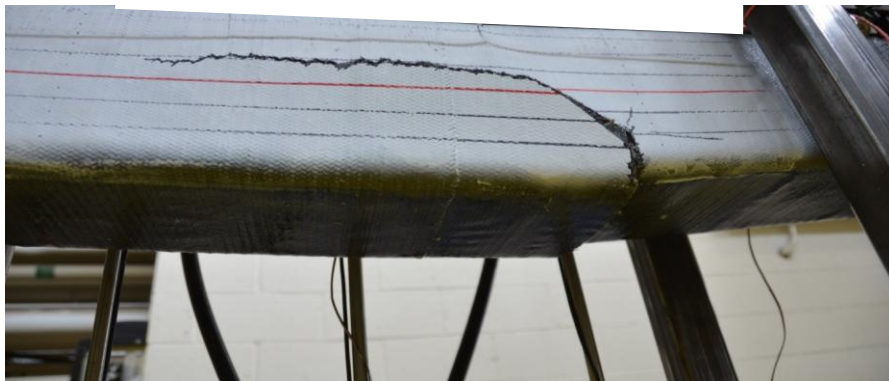


Figure 4.53. Damage to Column S7 at maximum load

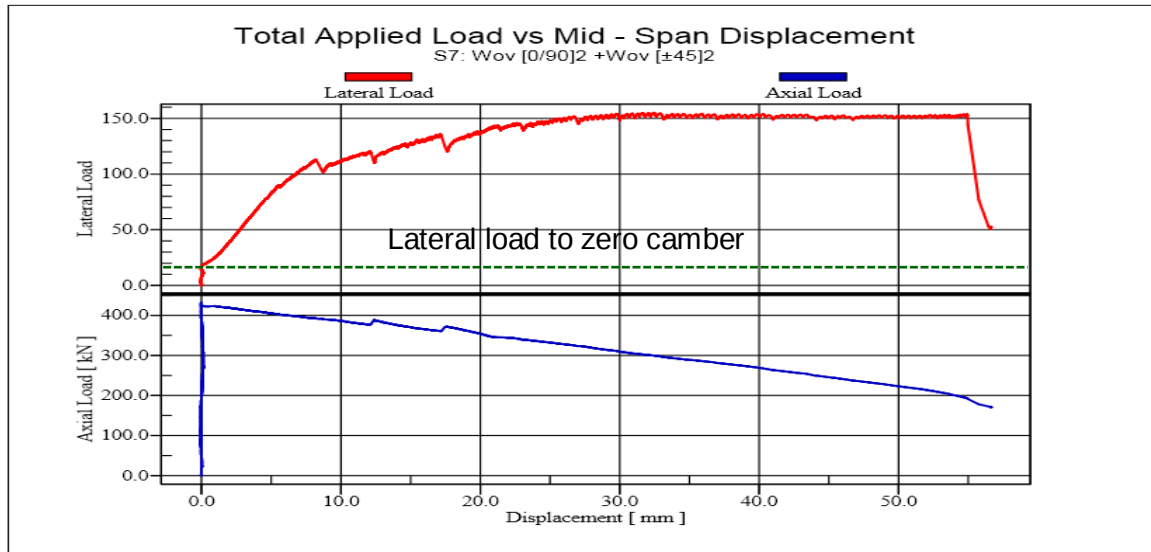


Figure 4.54. Variation of lateral load and axial load with mid-span deflection (Column S7)

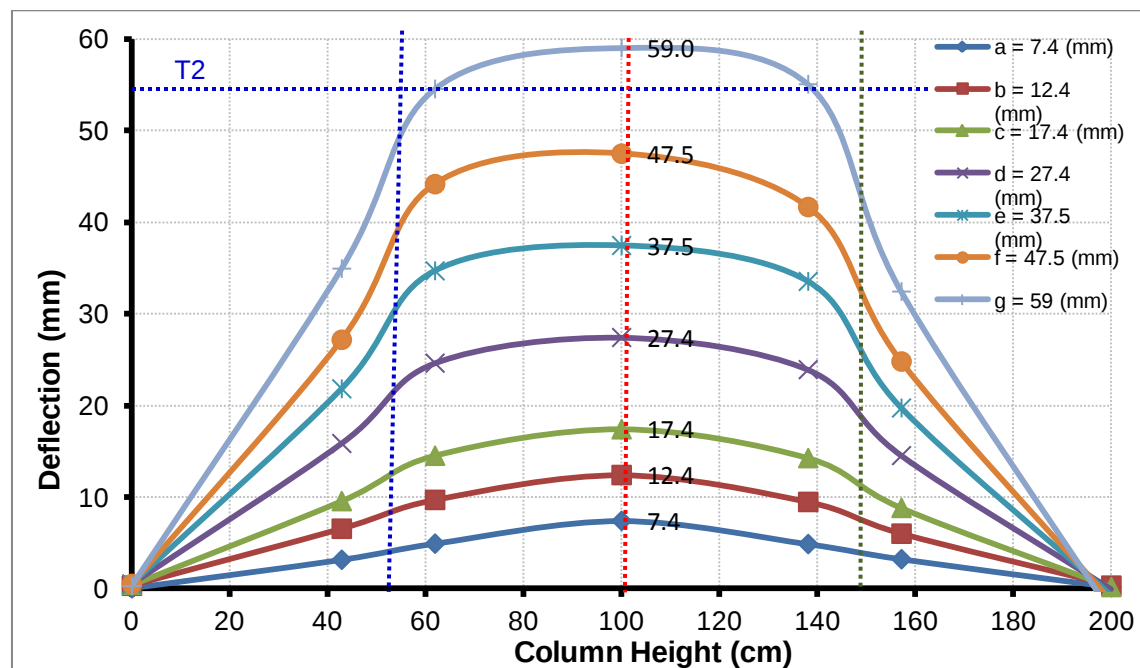


Figure 4.55. Deflection profiles at selected stages of loading (Column S7)

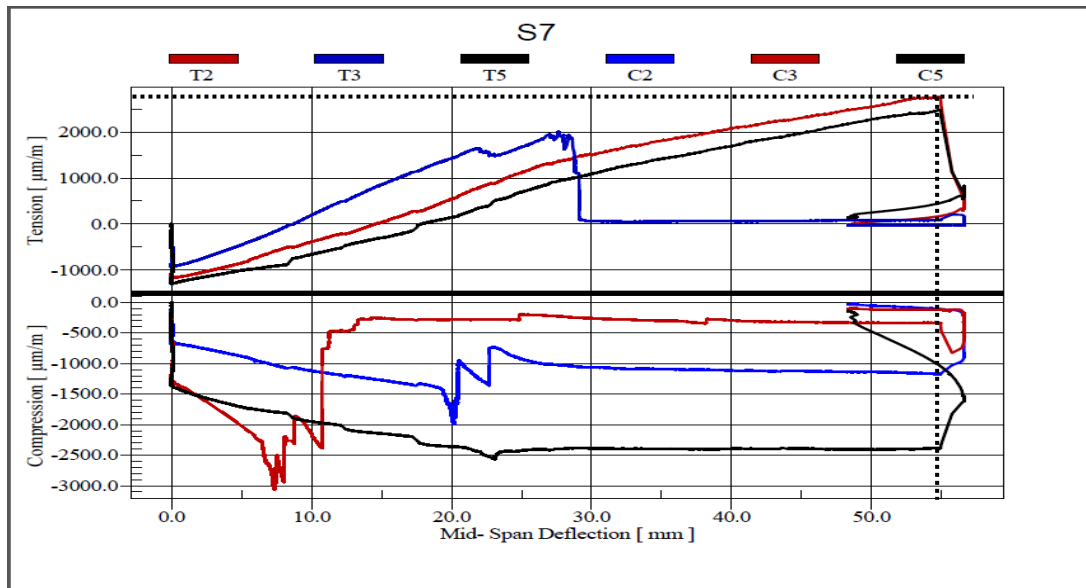


Figure 4.56. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S7)

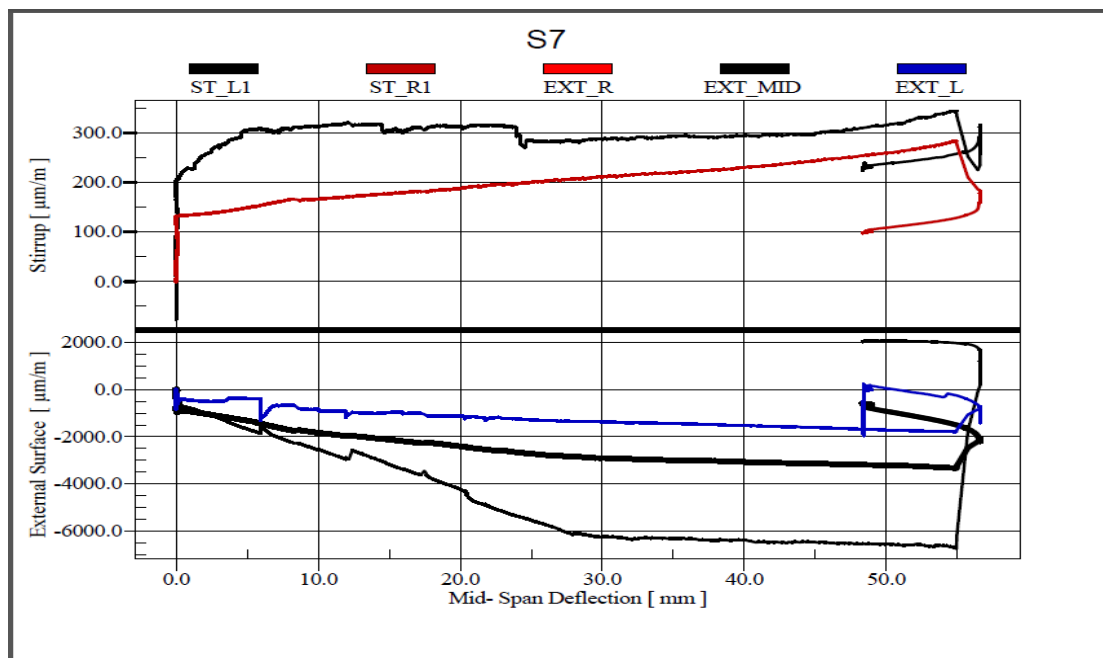


Figure 4.57. Strains in stirrups and top concrete surface at mid-span (Column S7)

The compressive strain recorded near the roller support (C2) developed $-5000 \mu\text{m/m}$ at a deflection of 30 mm and remained constant up to column failure. At the same deflection level, the strain gauge C5 reached $-8000 \mu\text{m/m}$ and remained constant up to column failure.

Figure 4.67 shows recorded strains on column ties by gauges ST-R1 and ST-L1. These strain values are very low, as was the case for the other columns discussed earlier. The gauge placed on column surface at mid-span was lost during testing (strain gauge EXT_Mid). The other two gauges on the surface developed a maximum strain of about $2700 \mu\text{m/m}$.



Figure 4.58. Column S8 prior to testing



Figure 4.59. Damage to Column S8 near the roller support

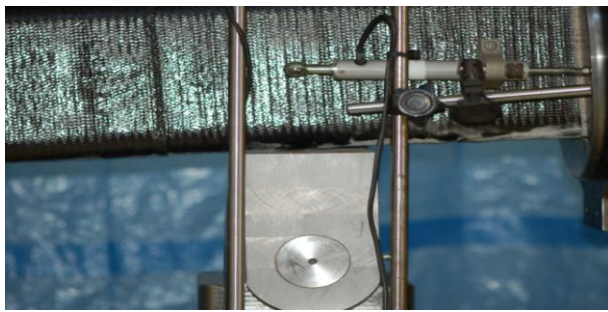


Figure 4.60. Damage to Column S8 near the hinge support



Figure 4.61. Damage on side face of Column S8 at maximum load



Figure 4.62. Damage to Column S8 at maximum load

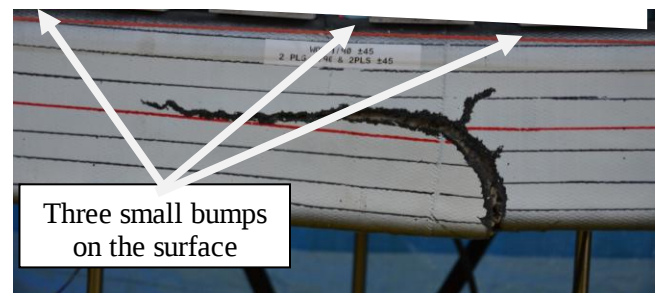


Figure 4.63. local buckling of the CFRP with three bumps on the top surface of S8

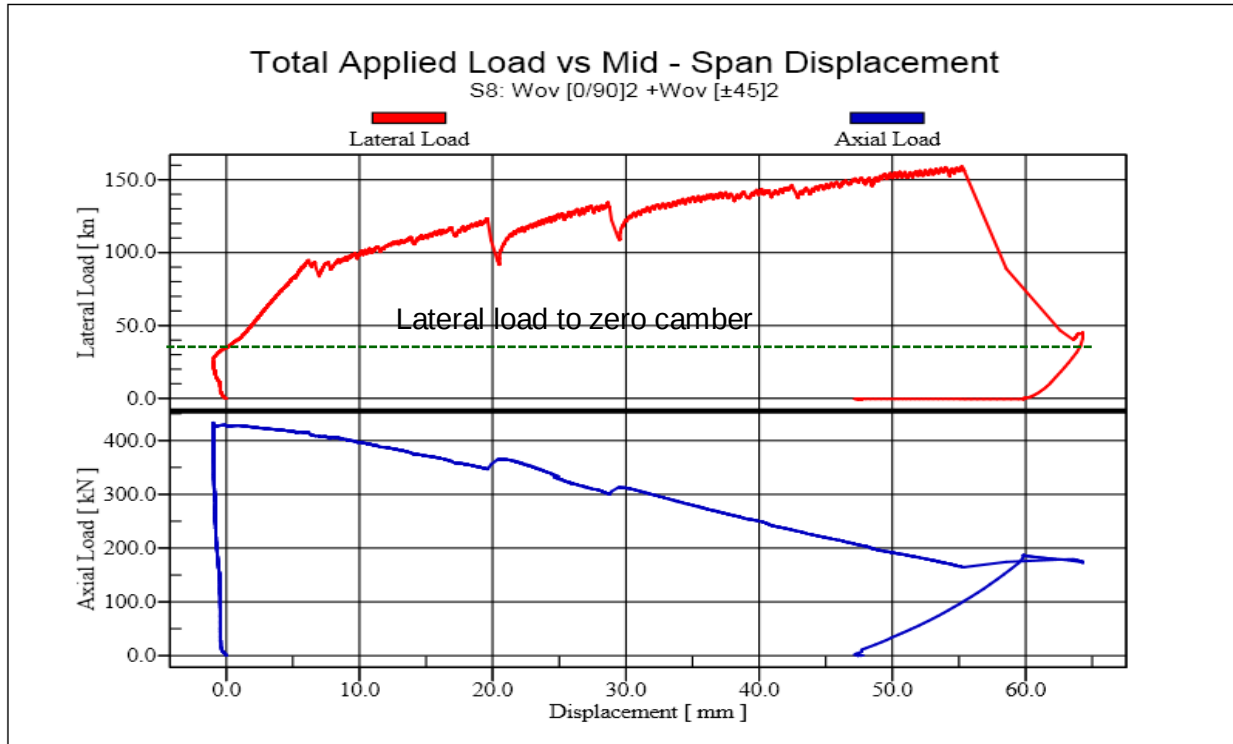


Figure 4.64. Variation of lateral load and axial load with mid-span deflection (Column S8)

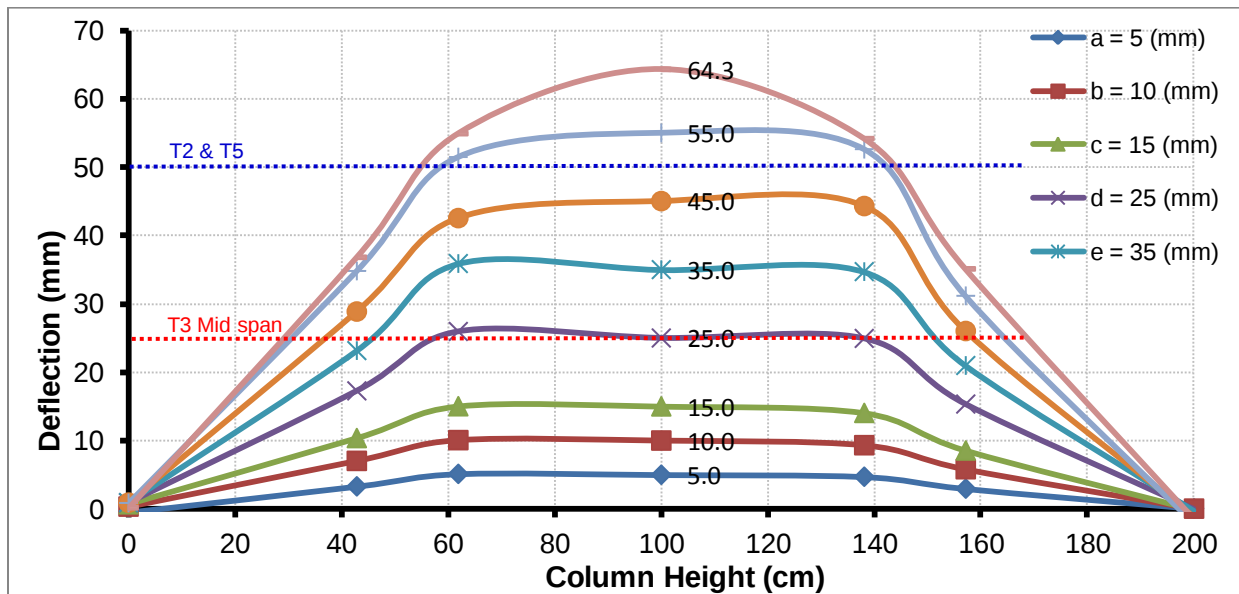


Figure 4.65. Deflection profiles at selected stages of loading (Column S8)

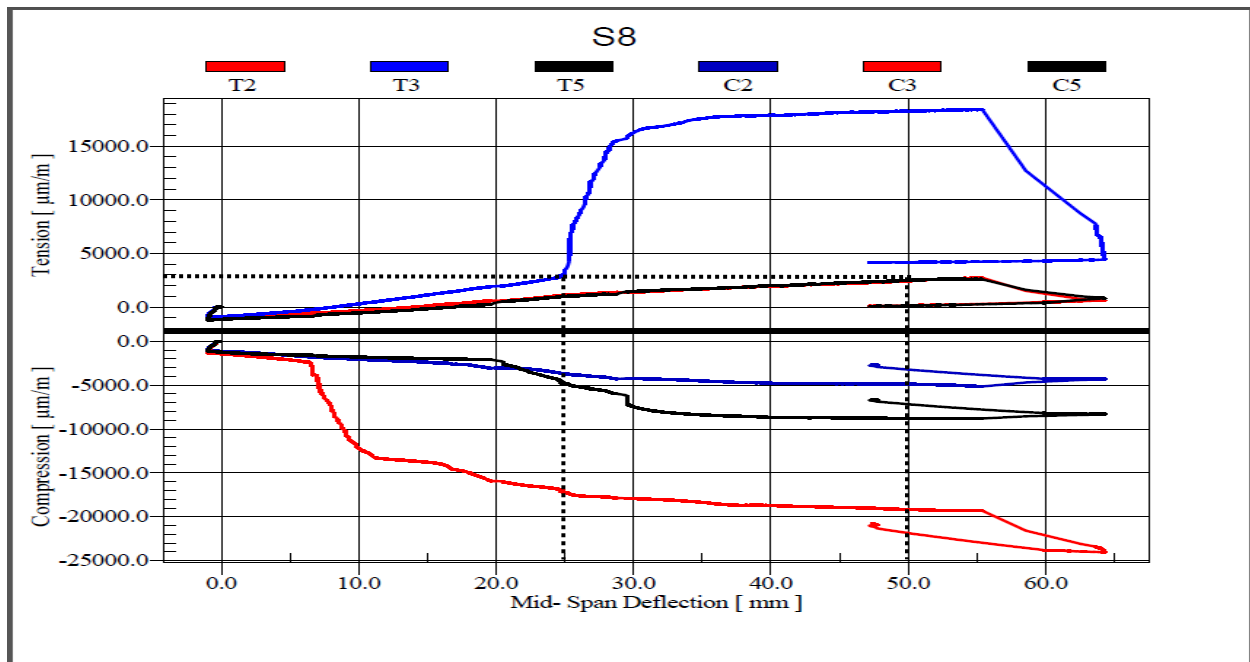


Figure 4.66. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S8)

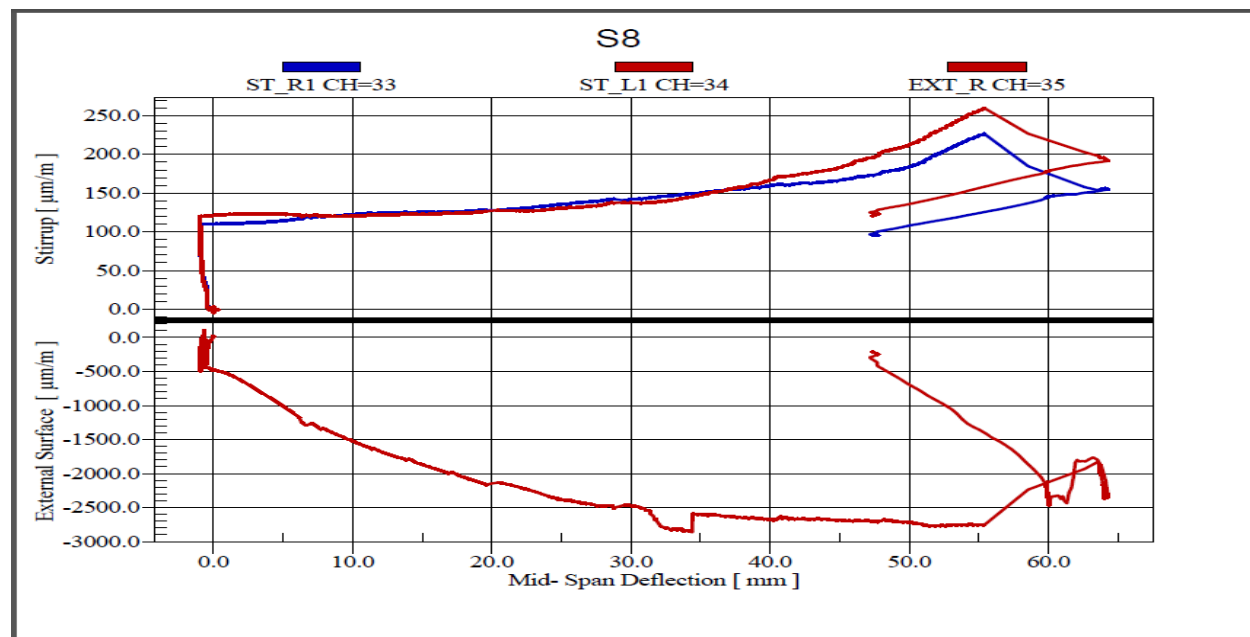


Figure 4.67. Strains in stirrups and top concrete surface at mid-span (Column S8)

4.10. Column Group V: Column Strengthened with CFRP-L4 (Column S3)

Column S3 was retrofitted with CFRP Laminate 4 (or CFRP-L4), which consisted of one lamina of uni-directional 0° , one lamina of uni-directional 90° , and two laminas of woven $\pm 45^\circ$. Figure 4.68 shows the column specimen prior to testing. Similar to previous column specimens, the axial load was applied gradually and held constant at 430 kN, followed by lateral loading with uniform distribution, applied at a rate of 5 kN/min up to the column failure. No cracks or damage to CFRP or the concrete inside was observed until failure. The load dropped and regained strength several times during loading, generating loud noise each time. This can be explained by the failure of the bond between the concrete and CFRP, and the inter-lamina shear stress redistribution. The column reached its ultimate capacity at a total lateral load of 216 kN at 58 mm mid-span deflection. The lateral load resistance started decaying beyond this load level as deflections continued increasing up to 65 mm. At failure, the CFRP ruptured with $\pm 45^\circ$ degree inclinations in a region between column mid-span section and the roller support end. The crack towards -45° degree inclination stopped, but the $+45^\circ$ crack continued as shown in Figures 4.71 and 4.72. The axial load dropped from 430 kN at the beginning of testing to 177 kN at column failure. No other damage was observed, as shown in Figures 4.69 and 4.70.

Figure 4.73 shows the variation of lateral and axial loads with increasing mid-span displacement. The initial camber due to the application of axial load and the accidental eccentricity in column was 6 mm, which required a total lateral force of 38 kN to bring the column to the neutral position. The load-deflection relationship shows an elastic region up to a lateral load of 150 kN and a lateral deflection of 11 mm. The column then started showing inelastic response up to a lateral load of 216 kN and a lateral displacement of 58 mm, beyond which it failed suddenly.

The strain gauge T3 located on tension reinforcement at mid-span showed yield strain at around 23 mm of lateral column deflection. The strain then increased to $20000 \mu\text{m/m}$ at a displacement of 38 mm, when the lateral load increased to 190 kN. The lateral load resistance continued to increase slightly with further increases in lateral displacement up to 58 mm at mid-span when it reached its maximum value of 216 kN. The column then lost its resistance rapidly, marking the end of testing. During this period the axial load dropped from 280 kN to 177 kN.

Figure 4.70 shows deflection profiles recorded at selected stages of loading. The same figure illustrates the deflections at which yielding of tension reinforcement recorded with dotted lines. Figure 4.71 shows the variation of strains recorded on tension and compression reinforcement with gauges T2, T3, and T5; and C2, C3, and C5, respectively. The initial strain on tension reinforcement was about $-3000 \mu\text{m/m}$ compression. The strains became tension as the lateral load increased putting the bottom bars in tension. The mid span gauge (T3) recorded high strains well above yielding as the strains recorded towards the supports increased only slightly, reaching yield strains at deflections of 40 mm and 58 mm, as recorded by gauges T2 and T5, respectively. The compressive strain at mid span, recorded by gauge C3, reached a value of $-20000 \mu\text{m/m}$ at a displacement of 39 mm. The strain gauge on compression reinforcement on the roller support side (C2) showed $-5000 \mu\text{m/m}$ at a deflection of 40 mm and remained constant up to column failure. The strain gauge C5 was lost during this test. Figures 4.74 and 4.75 indicate that the observed plastic hinge zone extends beyond the locations of the side strain gauges. The formation of a relatively long plastic hinge zone did not trigger a plastic collapse mechanism or instability. Figure 4.76 shows that the strains in the strain gauges ST-R1 and ST-L1 on column ties. These strain values are typically very low, as are the cases for all other columns tested.

4.11. Column Group V: Column strengthened with CFRP-L4 (Column S9)

Column S9 was retrofitted with CFRP Laminate 4 (or CFRP-L4), which consisted of one lamina of uni-directional 0° , one lamina of uni-directional 90° , and two laminas of woven $\pm 45^\circ$. Figure 4.77 shows the column before testing. An axial load was applied first and held constant at 430 kN. A uniformly distributed lateral load was applied next at a rate of 5 kN/min up to the column failure. No cracks or damage to the CFRP or the concrete inside was observed during until strength decay. At several stages of loading in the inelastic range of deformations small drops in load was observed with subsequent load recovery. When these occurred, loud noise was heard due to the failure of bond between the concrete surface and the CFRP laminate and the inter-lamina shear stress redistribution.

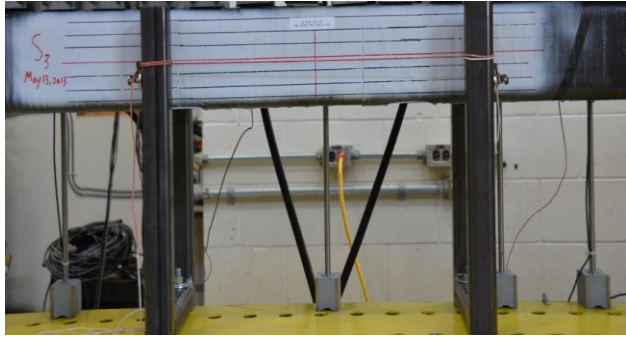


Figure 4.68. Column S3 prior to testing

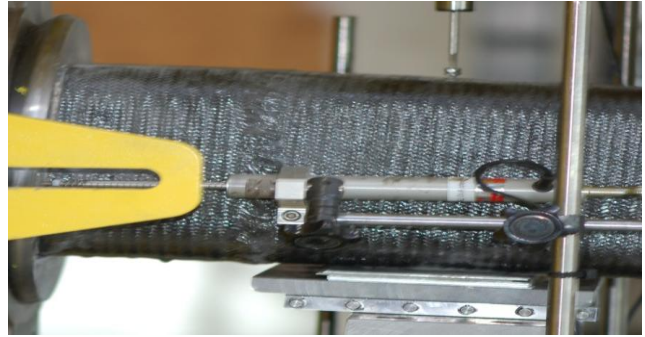


Figure 4.69. Damage to Column S3 near the roller support

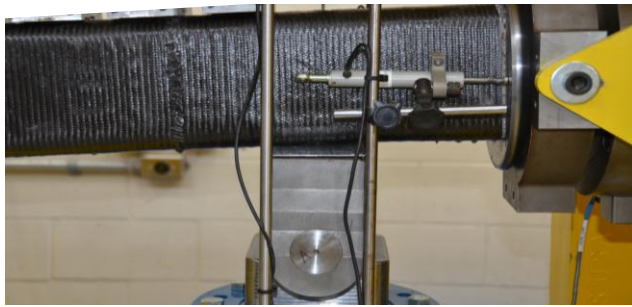


Figure 4.70. Damage to Column S3 near the hinge support

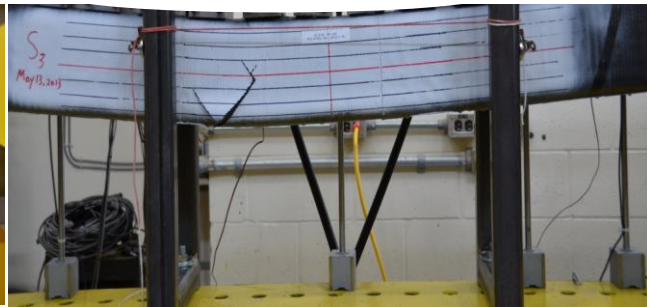


Figure 4.71. Damage on side face of Column S3 at maximum load



Figure 4.72. Damage to Column S3 at maximum load

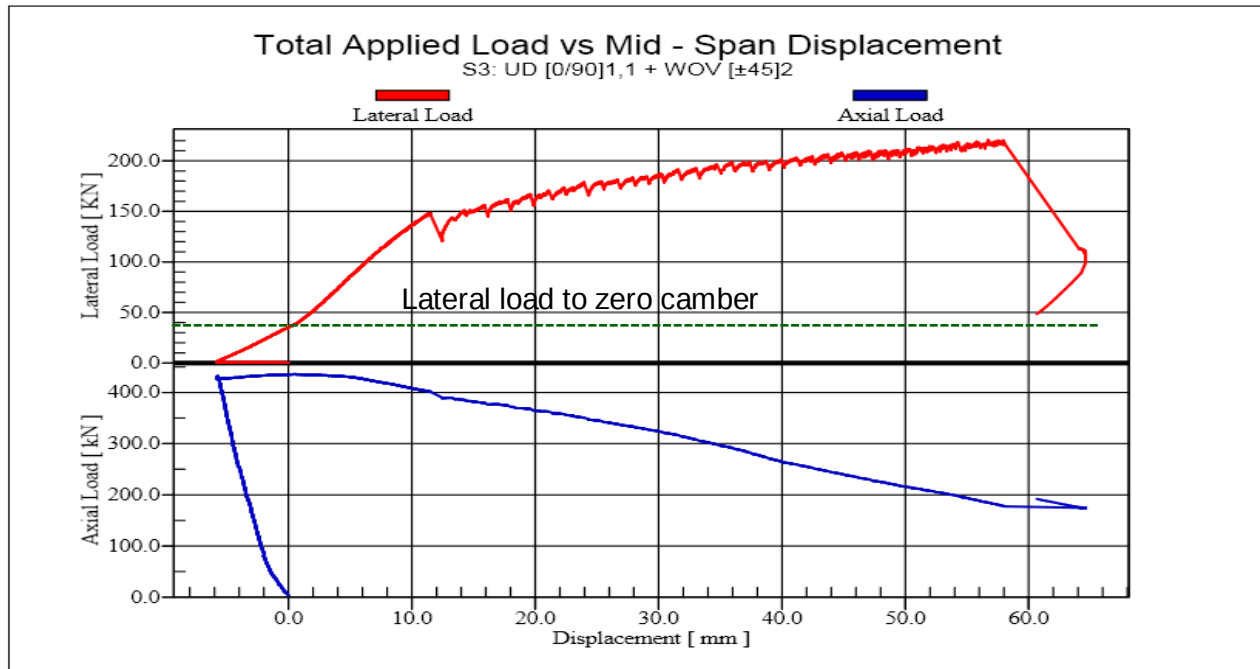


Figure 4.73. Variation of lateral load and axial load with mid-span deflection (Column S3)

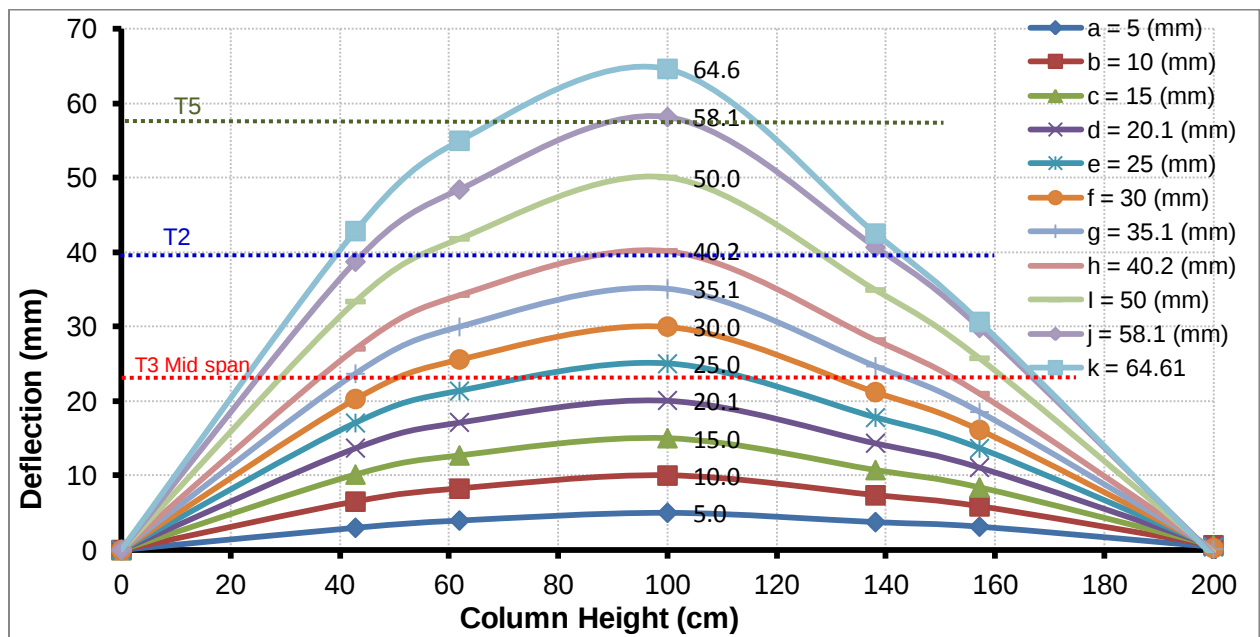


Figure 4.74. Deflection profiles at selected stages of loading (Column S3)

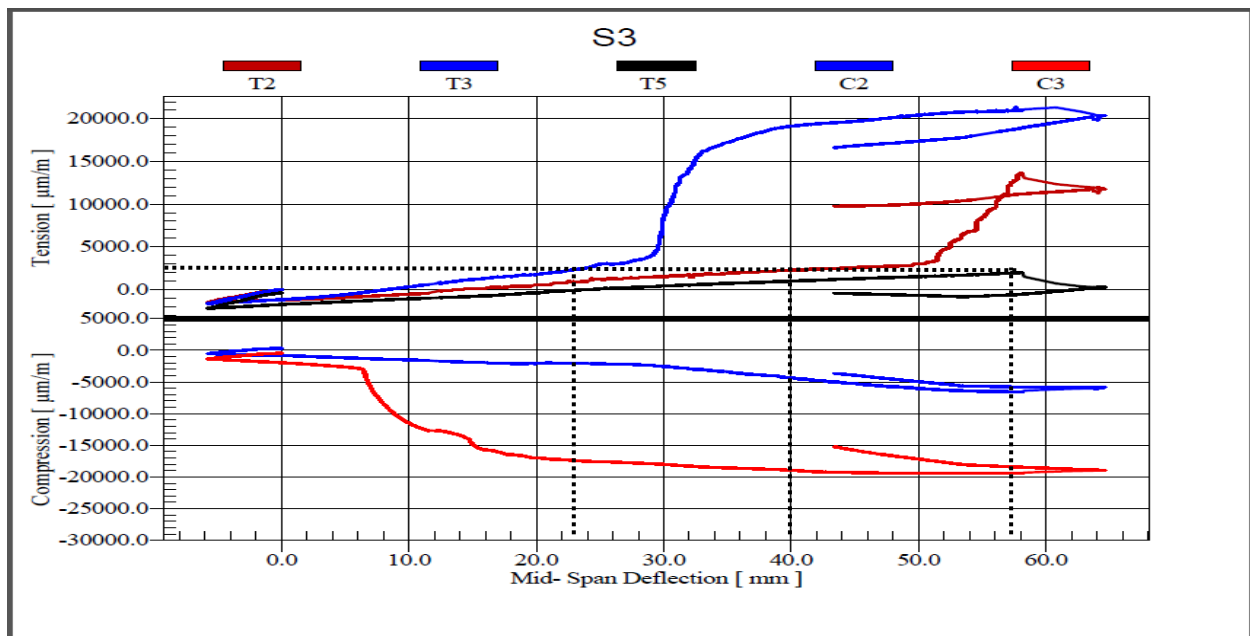


Figure 4.75. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S3)

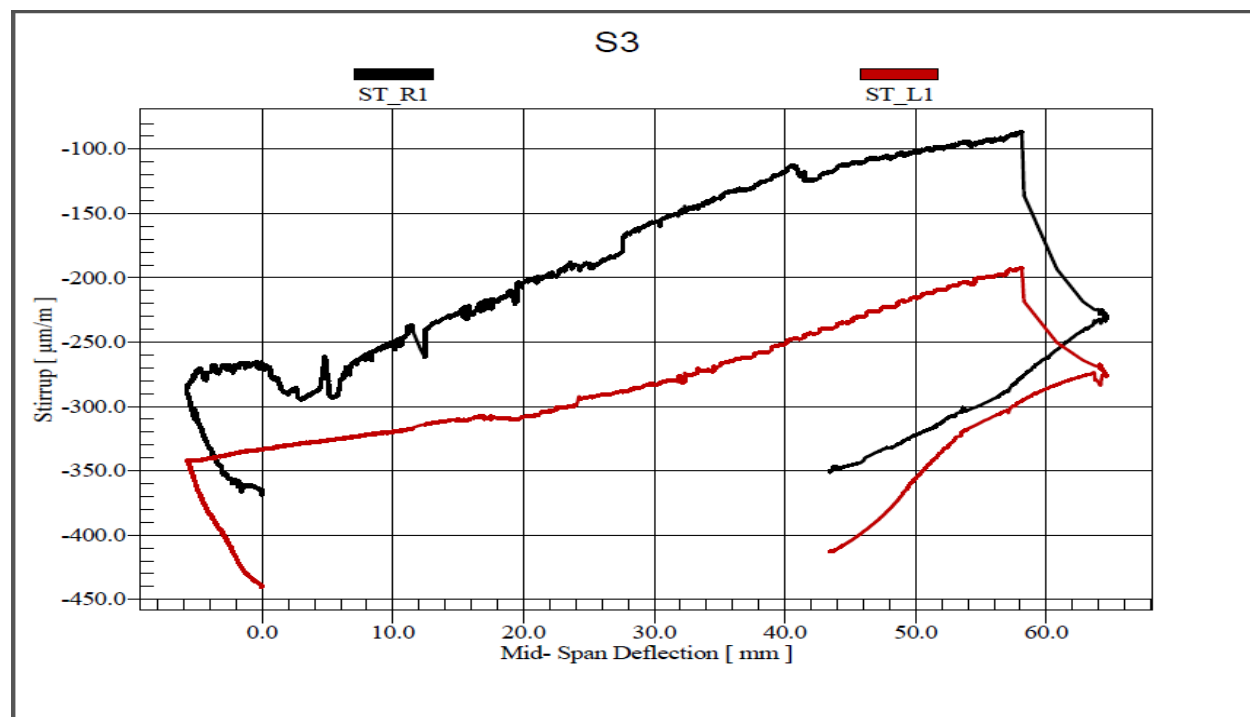


Figure 4.76. Strains in stirrups and top concrete surface at mid-span (Column S3)

The column reached its ultimate capacity at a total load of 218 kN and a mid-span deflection of 69 mm. Subsequently the lateral load dropped as the mid-span deflection continued increasing up to 82 mm. The CFRP-L4 started rupturing at peak load, where the rupture initiated in the bottom fibre of specimen. Propagating with a 45-degree inclination, as illustrated in Figures 4.80, 4.81 and 4.82. The axial load dropped from 430 to 65 kN at the end of the test. Although this developed the highest deformation among all the columns tested, there were no observations of damage. The support regions did not show any distress, as illustrated in Figures 4.78 and 4.79. Figure 4.83 shows the lateral load and axial load variations during testing. When the axial load was applied, the column cambered up due to the accidental eccentricity of load. The negative deflection was recorded to be 1.5 mm, which required 36 kN of downward load to bring the deflection back to zero. The load deflection relationship shows elastic behaviour up to the load of 110 kN and a mid-span displacement of 7 mm. As the deformations increased, the column exhibited inelastic behaviour, developing higher deformations with gradually increasing load resistance.

The strain gauge T3 located on tension steel at mid-span recorded yield strain at about a mid-span deflection of 23 mm. Then the strain increased to 20000 $\mu\text{m/m}$ beyond the mid-span displacement of 27 mm. The lateral load resistance in this column continued to increase slightly with further increase in lateral displacement up to 69 mm before the test was discontinued. Figure 4.83 shows the approximately linear reduction of axial load from 430 kN to 65 kN. The deflection profiles for selected stages of loading are shown in Figure 4.84, which also indicates the onset of yielding detected by two strain gauges on tension reinforcement.

The strain readings on tension and compression reinforcement are plotted in Figure 4.85. The initial readings on two strain gauges placed on tension reinforcement show compression strains (-500 $\mu\text{m/m}$) because of the initial camber under axial load. Subsequent increases in tensile strains are depicted in Figure 4.85 with high values at mid-span, and low values away from the high moment region, reaching yielding at 36 mm deflection. The compressive strain at mid span on the compression reinforcement developed strains reaching -18000 $\mu\text{m/m}$ at 42 mm lateral deflection. The compression strain gauge placed towards the roller support side (C2) reached a strain value of -8000 $\mu\text{m/m}$ at a deflection of 50 mm, and remained constant up to column failure. The strain gauge C5 reached -3000 $\mu\text{m/m}$ at 20 mm deflection and stayed constant up to

the column failure. Figure 4.84 and 4.85 indicate that the plastic zone of the reinforcement expands beyond the location of the two side strain gauges. As was the case in all other columns, the strain gauge readings on column ties were very low. This is shown in Figure 3.16.

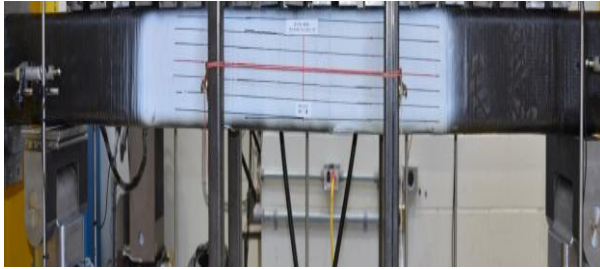


Figure 4.77. Column S9 prior to testing

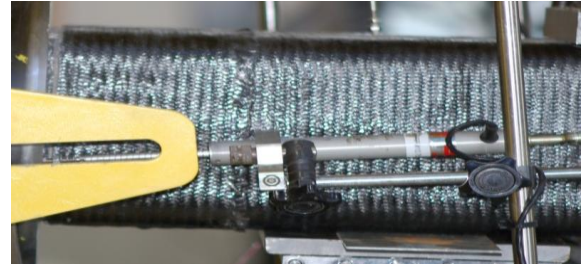


Figure 4.78. Damage to Column S9 near the roller support

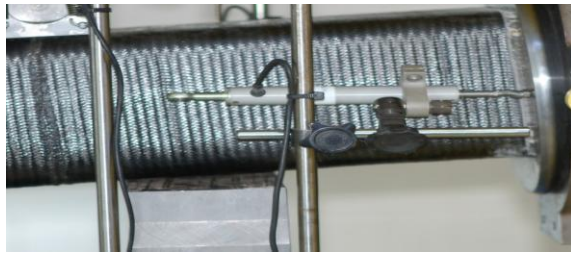


Figure 4.79. Damage to Column S9 near the hinge support



Figure 4.80. Damage on side face of Column S9 at maximum load

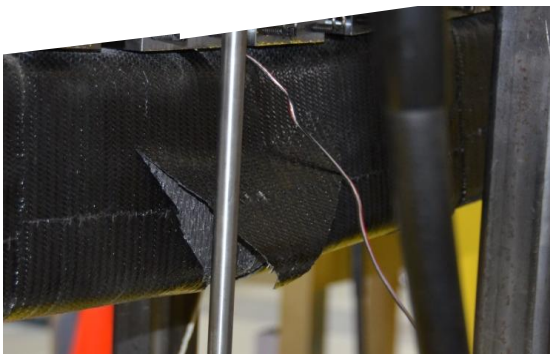


Figure 4.81. Damage on back side of Column S9 at maximum load

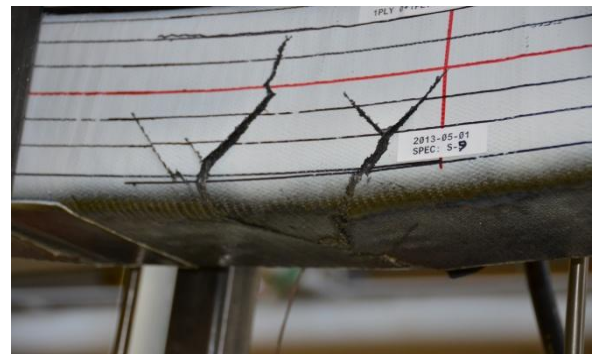


Figure 4.82. Damage to Column S9 at maximum load

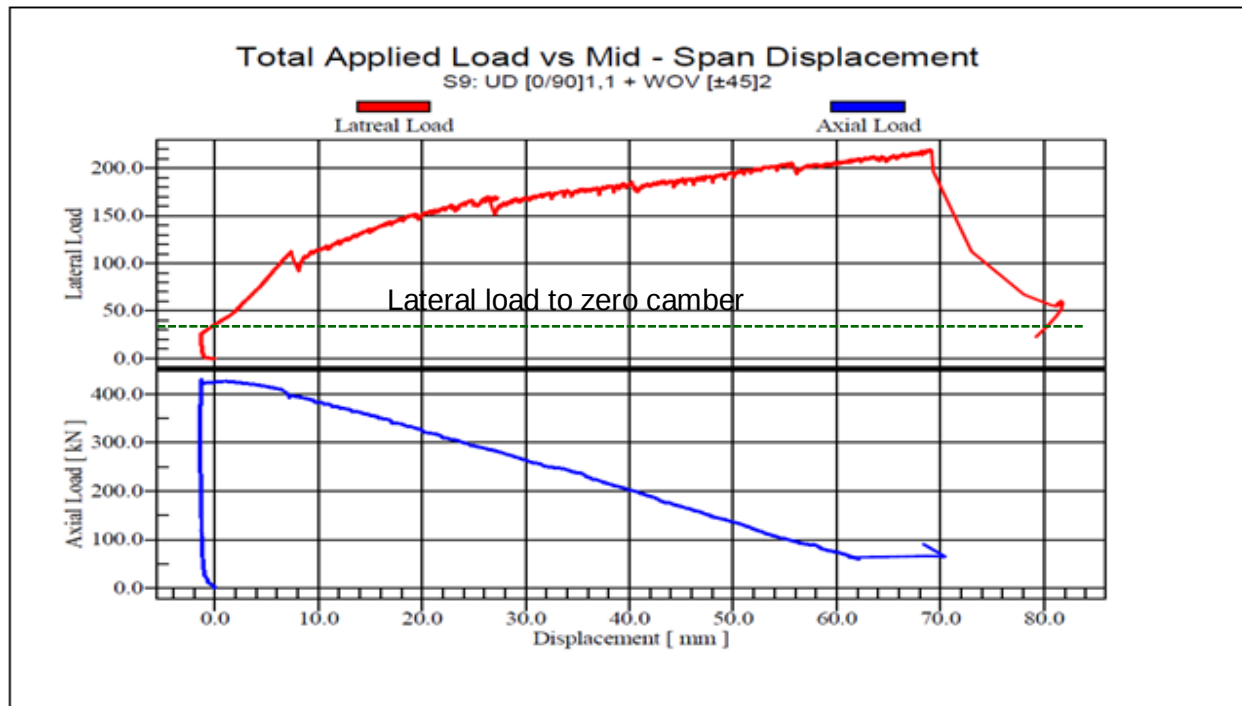


Figure 4.83. Variation of lateral load and axial load with mid-span deflection (Column S9)

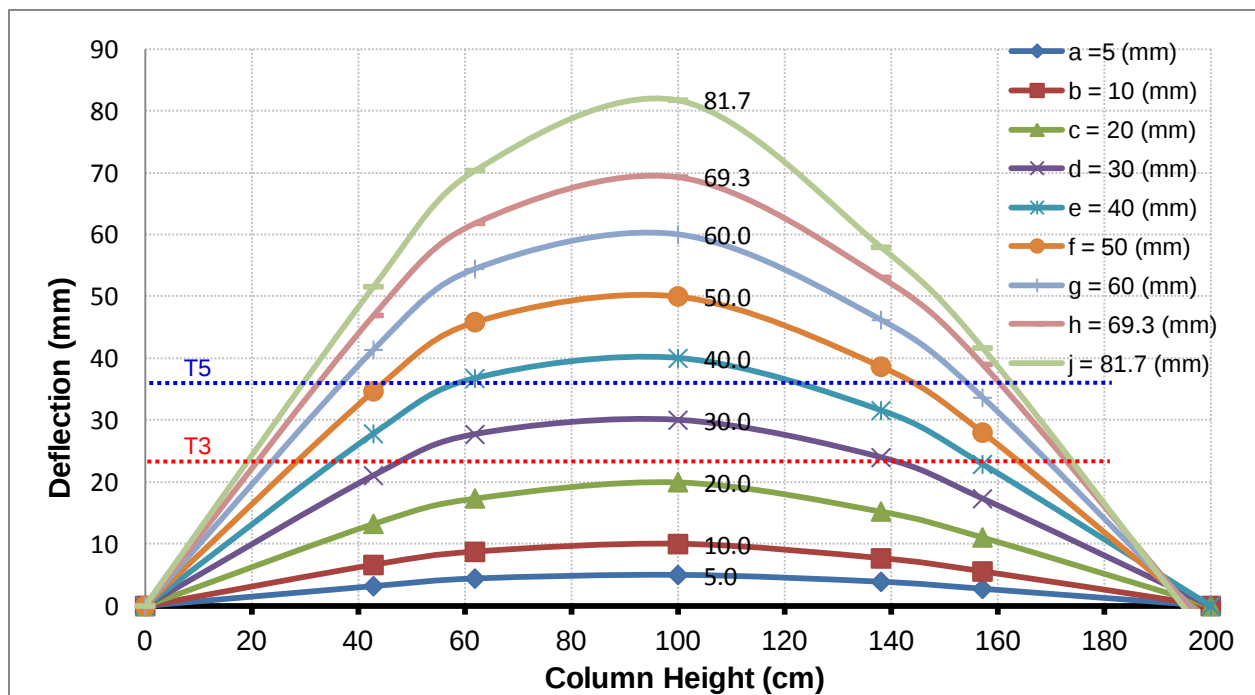


Figure 4.84. Deflection profiles at selected stages of loading (Column S9)

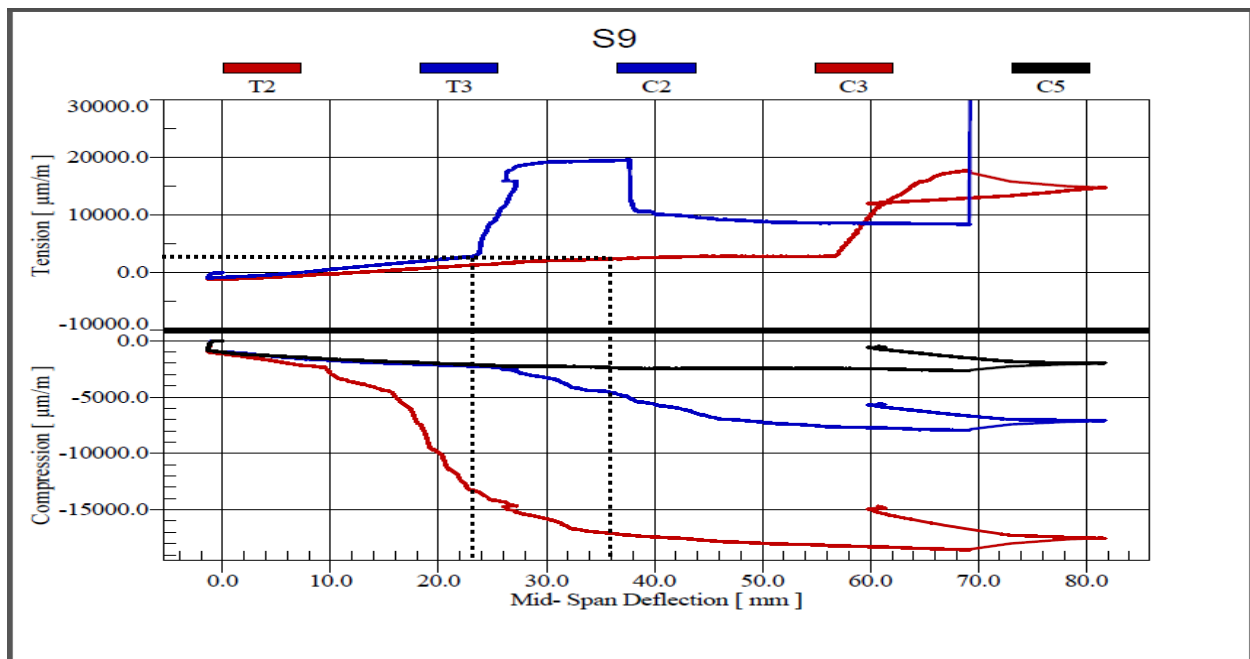


Figure 4.85. Variation of strains in bottom and top longitudinal reinforcement with mid-span deflections (Column S9)

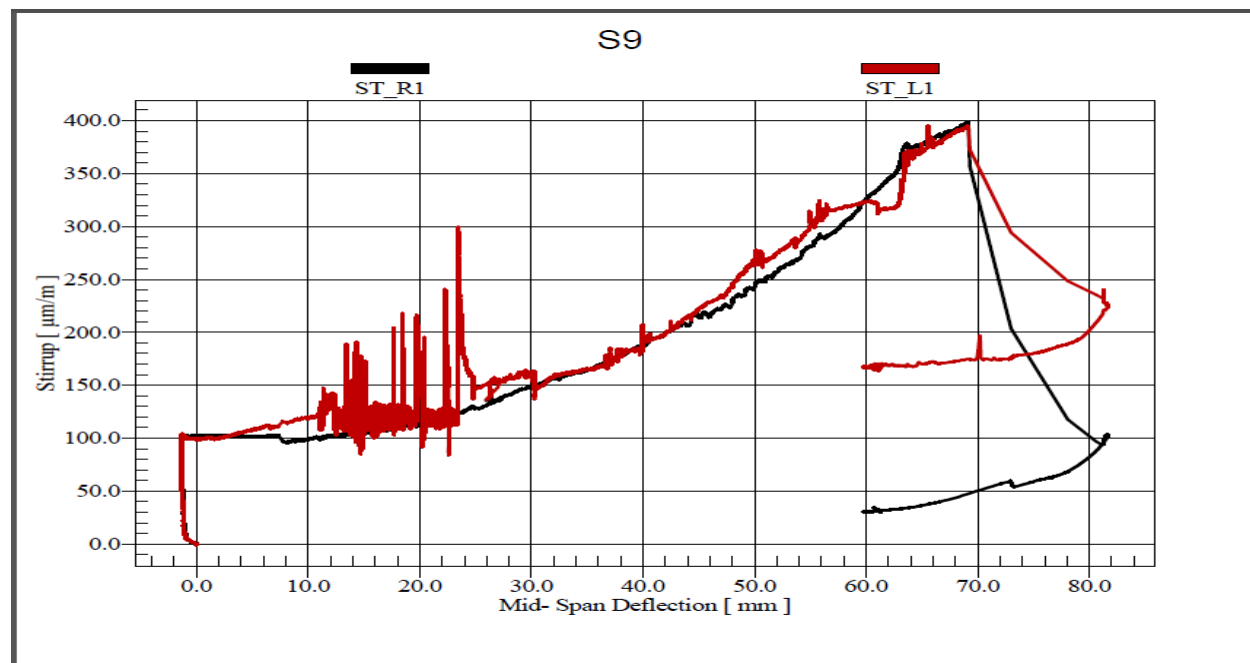


Figure 4.86. Strains in stirrups and top concrete surface at mid-span (Column S9)

Chapter 5: Effects of Test Parameters and Analysis of Columns

5.1. Introduction

The experimental research reported in the preceding Chapters was devised to investigate the effects of selected parameters related to the application of externally placed FRP sheets on column performance. This Chapter provides a comparison of the effects of each parameter selected, while also illustrating the significance of FRP laminates on column performance. In addition to the comparisons of experimental data and test observations, the columns were analysed under the same loading conditions used in the experimental program, including the inelastic range of deformations. These incremental inelastic static analysis results are compared to provide further explanation of the significance of each parameter, while also providing a verification of the applicability of the analysis techniques employed to columns retrofitted with the FRP laminates of the types considered.

The CFRP laminates were wrapped continuously, layer after layer, around the columns over their entire length using the wet layup method, as described in Chapter 3. This application process resulted in five different column groups with four different CFRP laminate tubes around the columns. The test results presented in Chapter 4 show that the composite action between the concrete columns and the external CFRP tube was affected by the laminate type used. The structural behaviour and the failure mechanisms observed can be explained by the procedure used in the laminate lay-up, fiber orientation, and fiber fabric characteristics (i.e. woven or unidirectional). The significance of these parameters is explained using; (i) lateral load versus mid-span lateral displacement relationships, (ii) deflection profiles and their significance on plastic hinge formation, (iii) variations in axial loads with lateral displacements, and (iv) axial load versus axial shortening relationships. Furthermore, a comparison is made between the experimental and analytical results to assess the ability of simple analysis techniques in reproducing test results in the elastic and inelastic range of deformations.

The assessment of test parameters and the analytical study are presented in three major steps:

- Comparisons between the behaviours of identically built companion columns within the same group; and between the columns of each group with the control columns in Column Group I (CG-I), as well as with the column in Column Group II (CG-II) which was the only column with FRP sheets having unwoven unidirectional fibres.
- Investigation of the effects of laminate design (type of the laminas and their lay-up) on the structural behaviour of reinforced concrete columns through the comparison of columns from different groups.
- Comparisons of test results with those obtained from incremental non-linear static analysis incorporating simple models for the FRP laminates.

5.2. Comparisons of Columns within Each Group and Relative to Control Columns

In this section companion columns within each group, built identically as pairs, are compared to assess whether the test results can be reproduced, and columns in each group behave in a similar manner. The comparisons are made in terms of strength, stiffness and deformation characteristics of columns, as well as observed damage patterns and failure modes. The comparisons are extended to include CFRP retrofitted columns from other groups relative to the control columns in CG-I and the retrofitted column with unwoven unidirectional fibers (Column S1 in CG-II) to assess the effectiveness of CFRP laminates in terms of strength, stiffness and ductility. As explained in Chapter 4, the first column group CG-I includes two control columns (S5 & S6) without any CFRP laminate. The second column group CG-II includes only one column (S1), retrofitted with CFRP-L1 having unidirectional unwoven fibers with an alignment sequence of UD $[0_2/90_2]$. The third column group CG-III includes two columns (S2 and S4) retrofitted with CFRP-L2, having woven fibers with an alignment sequence of W $[0/90]_4$. The fourth column group, CG-IV, includes two columns (S7 and S8) retrofitted with CFRP-L3, having a fiber alignment sequence of $\{W [0/90]_2 W [\pm 45]_2\}$. The fifth column group CG-V includes two columns (S3 and S9) retrofitted with CFRP-L4, having a fiber alignment sequence of $\{UD [0/90] W [\pm 45]_2\}$.

The failure of control columns were generally due to the crushing of compression concrete in the cover, followed by the buckling of compressive rebars. Figure 5.1 shows the lateral load-lateral

displacement and axial load- lateral displacement relationships for the control columns (S5 and S6). As can be seen, both columns showed similar behaviour. Column S5 was tested first. It developed some local crushing near the support region, outside the test region, prior to the column failure at the mid-span. Therefore, Column S6 was wrapped with CFRP sheets near the support regions, prior to testing. The two columns were otherwise identical. The results show a slight increase in the lateral capacity at failure. The maximum lateral load-carrying capacity of S5 and S6 were 120 kN and 138 kN respectively. This difference may be attributed to a combination of the FRP wrapping at the ends of S6 and the experimental scatter that is expected. The residual axial load in each column is approximately 150 kN.

The lateral load and axial load versus mid-span displacement relationships for Column S1 in the second column group CG-II (retrofitted with unidirectional fibres) are shown in Figure 5.2. The failure of the column initiated by sudden rupturing of the CFRP laminate at the tension face (or at the lower face during testing) of the column. The figure shows high flexural stiffness at the beginning of loading relative to the control column up to ultimate. The column reached a peak lateral load capacity of 240 kN at a mid- span deflection of 63 mm. The strength of this column is almost double the strength of the control columns. The lateral displacement at peak load is almost three times the value recorded for control columns. The increased strength of the specimen can be explained by the use of uni-directional fibers in all the laminates, provided both in the longitudinal and lateral directions; with the former providing increased flexural strength and the latter providing enhanced concrete confinement. In addition, the continuous lay-up approach employed resulted in full bond between the concrete and the CFRP. As a result, debonding of the CFRP was not observed, and the failure occurred when the longitudinal fibers on the tension surface reached the rupturing strength. The yield strain in the tension longitudinal bar at mid-span was developed at a slightly higher displacement than that for the control columns, indicating a small contribution of CFRP at this stage of loading. However, as the load continued to increase, the CFRP laminate tube became more effective and provided higher resistance. As the column capacity increased, both the tension and compression yielding in bottom and top reinforcement, respectively developed higher strains, indicating yield penetration towards the support regions. Indeed, the strain in compression reinforcement reached to 7800 $\mu\text{m/m}$ at column quarter length location, and the tension reinforcement started yielding at quarter-span

locations in tension at displacements of 55 mm and 60 mm. This signified that the CFRP laminates resulted in the extension of the plastic hinge region towards the column ends.

The axial load and lateral load versus lateral displacement relationships for column group CG-III (Columns S2 and S4), wrapped with laminate CFRP-L2, are shown in Figure 5.3. They indicate almost identical response between these two companion columns. The failure of both columns initiated by the FRP rupture. The maximum force resistance was around 160 kN (159 kN for Column S2 and 165 kN for Column S4), which is about 25% higher than the peak capacity of the control columns, but much lower than that of Column S1. The maximum deflections of columns are slightly higher (around 10% higher) than those of the control columns. Similarly, the residual load capacities for both lateral and axial loads are somewhat higher than the respective values of the control columns and Column S1.

Comparisons of column behaviour between CG-III (with woven 0/90 fibres of lesser thickness) and CG-II (with unidirectional fibres of higher thickness) show similar pattern. However, the propagation of yielding towards the support regions was limited in CG-III columns. This can be explained by the relatively smaller enhancement of strength and ductility associated with thinner layer of laminate in CG-III. In these columns maximum tensile steel strains of about 12,000 $\mu\text{m/m}$ were recorded at mid-span, which were higher than those of control columns, but lower than that of Column S1. The tension steel at quarter-span lengths did not reach yielding, though they were close to it. This means that the plastic zone of CG-III is shorter than that of CG-II, but it is much more than that of the control columns.

The axial force and lateral load versus lateral displacement relationships for column group CG-IV (Columns S7 and S8), which were retrofitted with laminate CFRP-L3 (consisting of two laminates of woven 0° - 90° and two laminates of woven $\pm 45^\circ$), are shown in Figure 5.4. The two companion columns showed almost identical behaviour up to the peak load, at which stage the rupturing of CFRP-L3 started from the column tension face. The columns reached a maximum lateral load carrying capacity of 154 kN for column S7 and 158 kN for Column S8, which is around 30% higher than that of the control columns, but lower than that of Column S1. The maximum deflections were about 35 % higher than those of the control columns.

The comparison of force-displacements relationships for the columns of CG-IV with the column of CG-II indicates ductile response for both groups. However, Columns S7 and S8 of CG-IV developed extended hinging region at column mid-span. The deflection profile showed a region within which the curvatures were almost constant. The steel tensile strain at mid-span reached a maximum value of 19000 $\mu\text{m/m}$, which was significantly higher than those for the control columns, as well as that for Column S1. The yielding propagated towards the quarter-span locations, where strain gauges indicated yielding at a displacement of 50 mm. This was another indication of a long plastic region, extending to cover about half the column length. This behaviour can be explained by the type of laminates employed, which consisted of woven fibres at $0^\circ/90^\circ$ orientation and an additional woven fibres having $\pm 45^\circ$ orientation. This fibre arrangement resulted in lower modulus elasticity, and allowed higher elongation of the material as the fibres aligned themselves in tension. The thickness of this laminate is also less than that of laminate CFRP-L1 used in CG-II.

The axial load and lateral load versus lateral displacement relationships for column group CG-V (Columns S3 and S9), retrofitted with laminate CFRP-L4 (consisting of one laminate of uni-directional 0° , one laminate of uni-directional 90° , and two laminates of woven $\pm 45^\circ$) are shown in Figure 5.5. The figure indicates similar behaviour among the two companion columns. They developed maximum lateral loads of 216 kN for Column S3 and 218 kN for Column S9. These values are about 80% higher than the peak capacities recorded for control columns, but they are 10% lower than the peak lateral load capacity of Column S1. The maximum deflections of this column group are 85% and 15% higher than those for the control columns and CG-II, respectively. The force-displacement relationships indicate similar behaviour as Column S1 in CG-II, but the deflection profile and the strain gauge readings show a very significant propagation of the plastic hinge region towards the column ends. This behaviour can be explained by the type of laminates employed, consisting of unidirectional $0^\circ/90^\circ$ and woven $\pm 45^\circ$, the combination of which enabled high elastic modulus with increased elongation characteristics under higher loads.

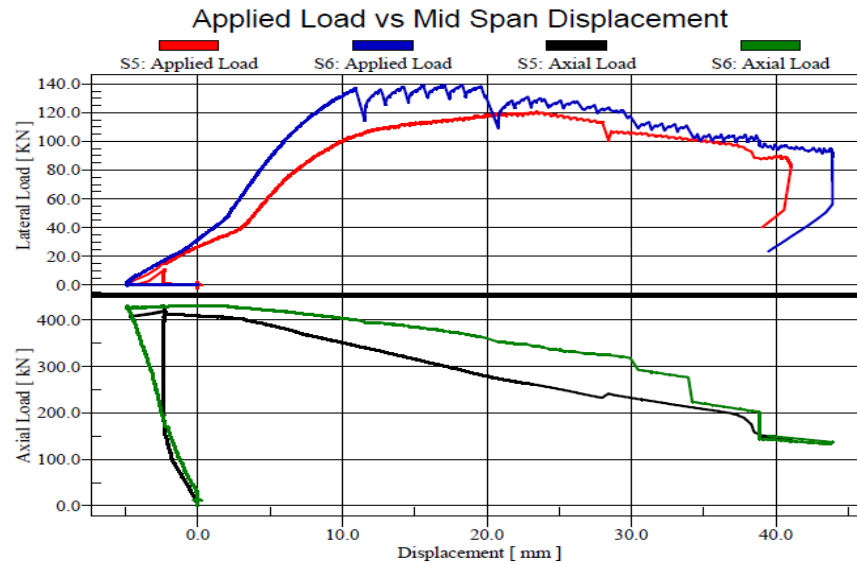


Figure 5.1. Applied load and lateral reactions vs mid-span deflection for Column Group CG-I
(control columns S5 and S6)

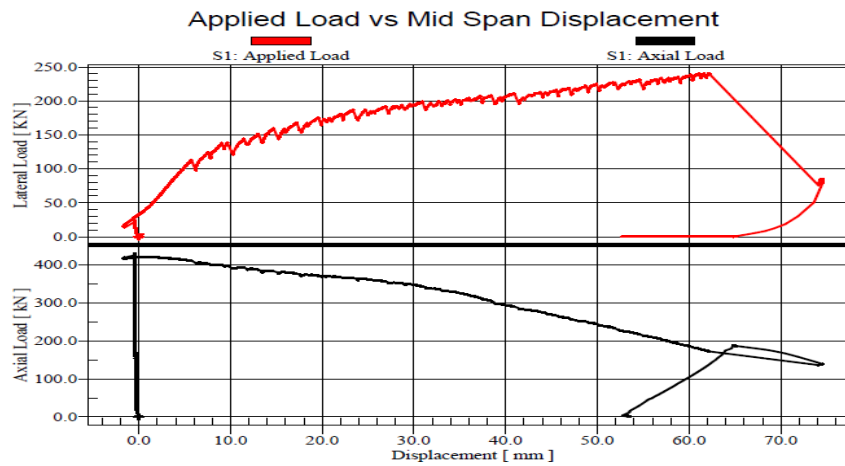


Figure 5.2. Applied load and lateral reactions vs mid-span deflection for Column Group CG-II
(Column S1)

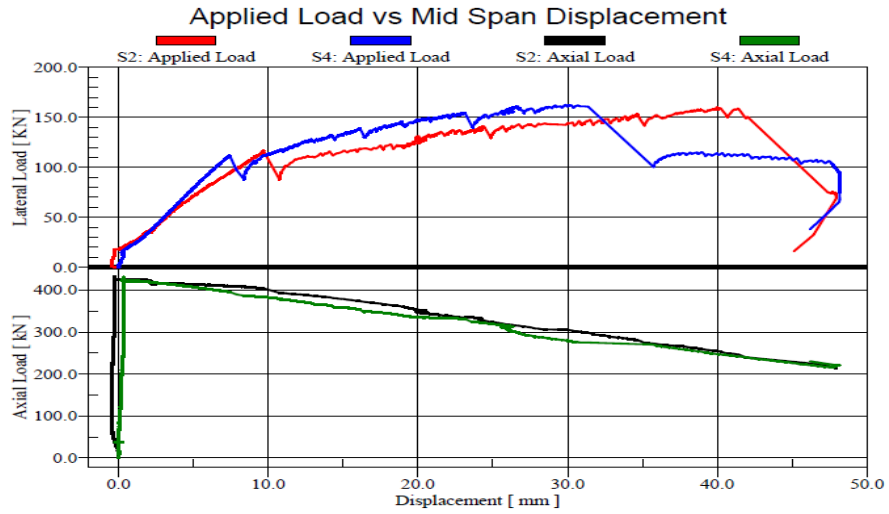


Figure 5.3. Applied load and lateral reactions vs mid-span deflection for Column Group CG-III
(Columns S2 and S4)

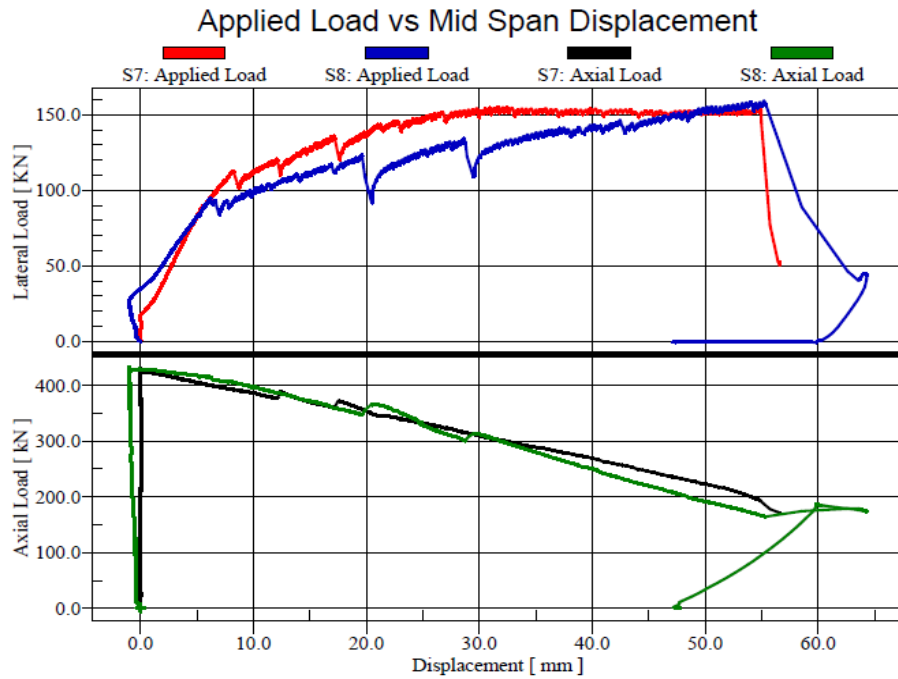


Figure 5.4. Applied load and lateral reactions vs mid-span deflection for Column Group CG-IV
(Columns S7 and S8)

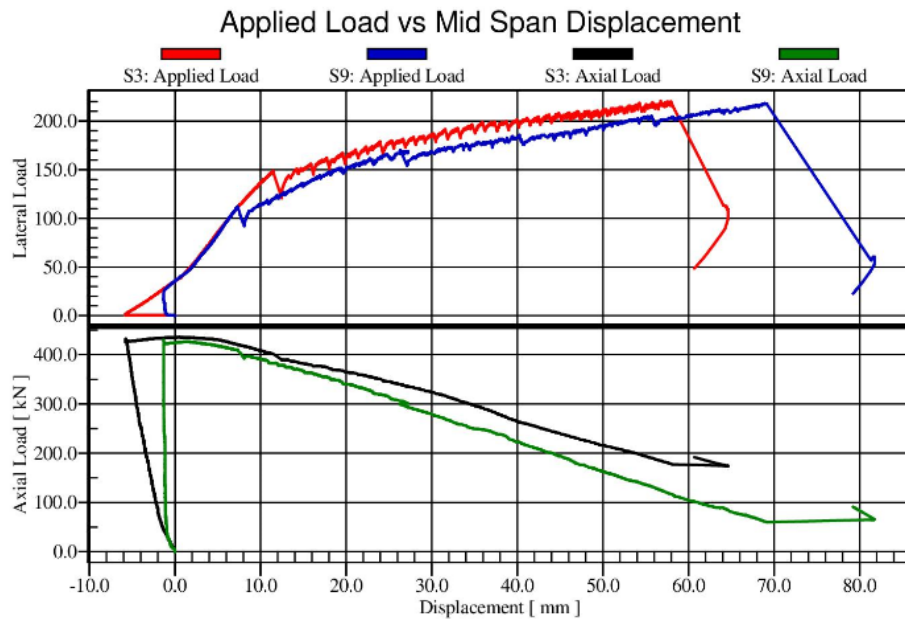


Figure 5.5. Applied load and lateral reactions vs mid-span deflection for Column Group CG-V (Columns S3 and S9)

5.3. Effects of CFRP Laminate Type on Column Performance

In this section selected columns from each group are compared to assess the performance of columns retrofitted with different types of CFRP laminates. Figure 5.6 shows lateral force-lateral deflection relationships of selected columns with different CFRP laminates. The comparison of these experimental relationships, along with the experimental data and behaviour discussed earlier in Section 5.2 result in the following observations in terms of the effectiveness of the CFRP retrofits employed:

- (i) All CFRP retrofitted columns developed higher strength and deformability than unretrofitted control columns. The increase in strength depended on the ultimate rupturing strength of the laminates used, and varied between 20% and 90%. The increase in deformability depended on the fibre orientation and the resulting changes in the elastic modulus of laminates, and ranged between 68% and 176%.

- (ii) The use of inclined fibre orientation ($\pm 45^\circ$) in any laminate reduced the modulus of elasticity and strength of CFRP. Therefore, these columns (CG-IV and CG-V) showed reduced stiffness relative to that with unidirectional fibres (CG-II).
- (iii) The laminate thickness is another parameter that affects the effectiveness of the CFRP jacket used for column retrofitting, in addition to the laminate modulus. Columns in CG-III and CG-IV, though had either entirely or partially higher modulus unidirectional woven fibres, they showed reduced column stiffness because of the relatively small laminate thickness (2.00 to 2.22 mm as opposed to 2.60 mm and 2.64 mm in other columns).

During the materials testing it was observed that the strength of laminates with inclined fibres resulted in reduced strength and stiffness. This was attributed to the realignment of fibres in tension. To assess the significance of inclined fibres on inelastic behaviour of columns, column deflection profiles were re-assessed and compared. Figure 5.7 shows the deflection profiles of columns with different laminate configurations at failure. The figure shows that CG-V gives the highest deformations compared to all other groups. The comparisons of deflected shapes shows that the use of woven $\pm 45^\circ$ laminates reduces the slope of the deflection curve at the mid-span, where plastic hinging occurs. This is especially evident in Column S7 with woven fabrics, where two of the four layers had fibres with $\pm 45^\circ$ orientation. Column groups CG-I, CG-II, and CG-III show higher slope at mid-span, with sharper deflected shapes. The use of unidirectional laminates (CG-II and CG-V) results in large force and deformation capacity; however the use of the woven $\pm 45^\circ$ laminates improves the distribution of inelastic column deformations within the critical region, resulting in extended hinging regions.

5.4. Analysis of Columns and Comparison with Experimental Data

5.4.1. General

Non-linear static analyses of columns with and without CFRP jackets were conducted to validate the analysis techniques for CFRP retrofitted columns and to generate further data on the effectiveness of the CFRP laminates employed. A modified version of the computer software developed by Mohammed (2014) was used for this purpose. The analyses consisted of sectional and member analyses under incrementally increasing static loads. This section provides the details of the material constitutive models used, the nonlinear sectional analysis algorithm

employed and the column member analysis procedure used. The analytical results are presented and compared with experimental data.

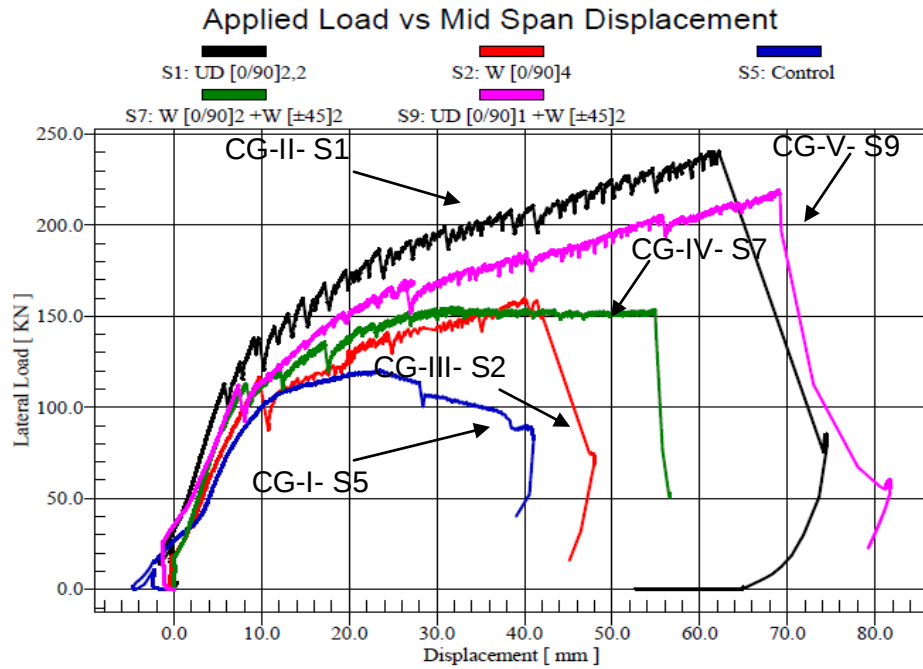


Figure 5.6. Comparison of the lateral load versus lateral deflection relationships of selected columns from different column groups

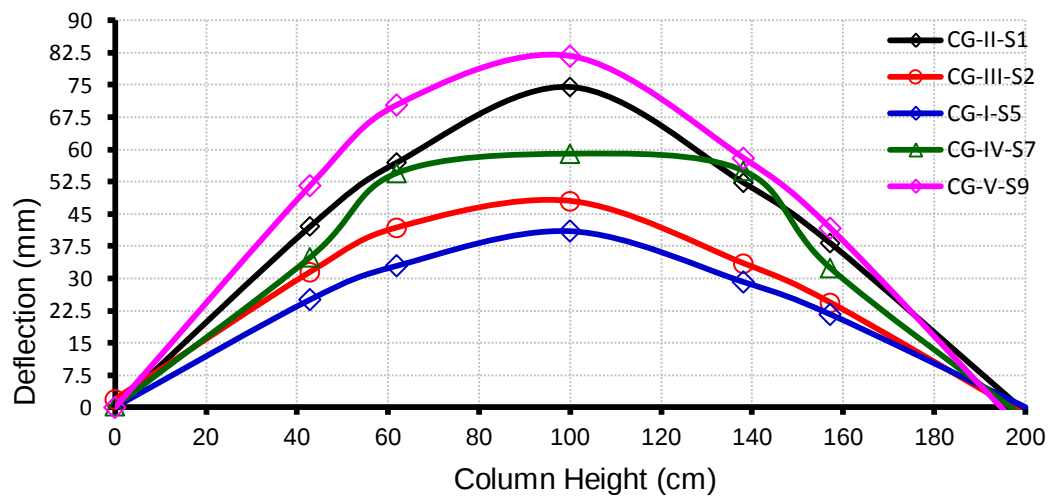


Figure 5.7. Comparison of deflection profiles over column unsupported length for selected columns in different column groups

5.4.2. Stress-strain models for concrete and reinforcing steel

Modeling the stress-strain relationship of concrete in compression has been widely researched and numerous models have been developed by previous researchers. One of the well accepted models was developed by Hognestad (1951), which consists of a second-order stress-strain relationship (parabola) up to the ultimate stress, followed by a linear stress drop with relatively large increase of the strain up to collapse. Kabaila (1964) and Basu (1967) proposed fourth-degree polynomials with more precise consideration of the post ultimate stage. Although such models have been widely used by many researchers, they are not capable to simulate the change in concrete properties under different states of confinement.

Saatcioglu and Razvi (1992) proposed a stress-strain model for concrete with different levels of confinement based on reinforcement details. The researchers verified the applicability of their approach to almost all popular cross-sectional shapes and reinforcement details. In a column sectional analysis under axial and flexure loads, the cross-section is divided into two parts: (i) unconfined concrete cover; and (ii) confined core concrete. When the axial pressure is relatively high and the beam-column has adequate lateral reinforcement properly detailed, Saatcioglu and Razvi's approach shows that the concrete strength and ductility are enhanced significantly due to the well-developed lateral confinement pressure. The Saatcioglu and Razvi (1992) stress-strain model approaches to that proposed by Hognestad when the effect of confinement becomes negligible, following an ascending part described by a second degree parabola, and a descending part linearly changing to a strain corresponding to 20% of the peak (see Figure 5.8). The slope of the descending part is defined by specifying the strain at 85% of strength. This stress-strain relationship is used for unconfined concrete (e.g., for the concrete cover or for the entire RC section when the axial load is less than 10% of the column axial capacity). The following expressions are used to describe the stress-strain relationship for unconfined concrete:

$$f_c = f'_{co} \left[2 \frac{\epsilon_c}{\epsilon_{o1}} - \left(\frac{\epsilon_c}{\epsilon_{o1}} \right)^2 \right] \text{ for } 0 \leq \epsilon_c \leq \epsilon_{o1} \quad (5.1)$$

$$f_c = f'_{co} - (\epsilon_c - \epsilon_{o1}) \left(\frac{0.15f'_{co}}{\epsilon_{o85} - \epsilon_{o1}} \right) \text{ for } \epsilon_{o1} \leq \epsilon_c \leq \epsilon_{cu} \quad (5.2)$$

where f'_{co} is the in-place strength of unconfined concrete; ϵ_{o1} and ϵ_{o85} are the strain at peak and 85% of the peak unconfined strength respectively; and ϵ_c and ϵ_{cu} are strain and ultimate strain

in concrete compression respectively. Concrete confinement is introduced through coefficient “k”. The following expression is used for the parabolic ascending part, where coefficient k is based on material and geometric properties of columns (Saatcioglu and Razvi 1992).

$$f_{cc} = f'_{cc} \left[2 \frac{\varepsilon_c}{\varepsilon_1} - \left(\frac{\varepsilon_c}{\varepsilon_1} \right)^2 \right]^{1/(1+2k)} \quad \text{for } \varepsilon_c \leq \varepsilon_{o1} \quad (5.3)$$

Where f_{cc} refers to the compressive stress in confined concrete.

The coefficient k in equation 5.3 is expressed below:

$$k = \frac{k_1 f_1}{f'_{co}} \quad (5.4)$$

$$k_1 = 0.67 (f_{le})^{-0.17} \quad (5.5)$$

$$f_{le} = k_2 f_l \quad (5.6)$$

$$k_2 = 0.15 \sqrt{\left(\frac{b_c}{s} \right) \left(\frac{b_c}{s_\ell} \right)} \quad (5.7)$$

$$f_l = \frac{A_t f_s}{s b_c} \quad (5.8)$$

Where, b_c is the core dimension perpendicular to the direction of loading, s is the spacing of transverse ties, s_ℓ is the spacing between laterally supported longitudinal reinforcement (supported by cross ties or perimeter hoop), A_t is the total area of transverse steel reinforcement in the direction of lateral load and f_s is the effective stress in transverse reinforcement, which is equal to the yield strength f_y for ordinary Grade 400 MPa steel. The above expressions are applicable to core concrete confined by internal steel reinforcement, as in the case of hoop steel used in the columns tested in the current investigation.

When the concrete is confined by the FRP jacket, then the confinement becomes a function of the FRP with fibres in the hoop direction (E_{fy}), FRP laminate thickness, t_{frp} , and the effective stress of the laminate in the lateral column direction f_{hfrp} (or the column hoop direction), which is limited to the rupture strength of f_{frpy} .

$$f_l = \frac{2 t_{frp} f_{hfrp}}{D} \quad (5.9)$$

Where D is the column sectional dimension perpendicular to the lateral load direction, and

$$f_{hfrp} = 0.006 E_{fy} \text{ or } f_{frpy} \text{ whichever is less.} \quad (5.10)$$

The confinement effectiveness parameter k_2 becomes 1.0 for circular columns and 0.4 for square and rectangular columns (Saatcioglu et al. 2008). When a column is confined internally by

internal hoop steel and externally by CFRP jacket, then the internal core concrete enclosed by the hoops is under the confinement effects of both the internal steel and external CFRP. In this case the stress strain relationship for the confined internal core concrete can be established using the same model described above, but through the superposition of lateral confinement pressures resulting from both types of confinement reinforcement. This requires the computation of total confinement pressure as indicated below:

$$f_{le} = (k_2 f_{\ell})_{Hoop} + (k_2 f_{\ell})_{CFRP} \quad (5.11)$$

The RC-Blast software (Jacques 2014) used in the current study does include the effects of both the internal steel hoop and external CFRP confinement. However, the program developed by Mohammed (2014) ignores the contributions of the internal hoop steel.

The strain quantities for confined concrete can be computed from the following expressions:

$$\varepsilon_1 = \varepsilon_{01} + 5k \quad (5.12)$$

$$\varepsilon_{85} = 260\rho_c\varepsilon_1 + \varepsilon_{085} \quad (5.13)$$

Figure 5.8 shows the stress-strain relationships of unconfined and confined concrete with key stress and strains indicate.

The contribution of tension concrete to sectional resistance is neglected in RC-Blast (Jacques 2014), whereas it is considered in the Nonlinear Sectional Analysis of RC Beam-Column program (NLSABC) Mohammed (2014). When considered, the stress-strain relationship of concrete in tension is adopted from Gilbert and Warner (1978). As shown in Figure 5.9, the concrete in tension is assumed to behave linearly up to the cracking strain, where the modulus in tension is assumed to be equal to the initial tangent modulus of the concrete in compression, followed by a linear descending curve that represents strain softening.

Reinforcing steel is assumed to have the same stress-strain relationship in both tension and compression provided that no local buckling of the compression bars take place. For an accurate representation of the steel behaviour the actual stress-strain data collected from coupon tests were used. The stress-strain model for steel in tension and compression has been adopted from Yalcin and Saatcioglu (2000), as described below and illustrated in Figure 5.10:

$$f_s = E_s \varepsilon_s \quad \text{for } 0 \leq \varepsilon_s \leq \varepsilon_y \quad (5.14)$$

$$f_s = f_y + \left(\varepsilon_s - \varepsilon_y \right) \left(\frac{f_{sh} - f_y}{\varepsilon_{sh} - \varepsilon_y} \right) \quad \text{for } \varepsilon_y \leq \varepsilon_s \leq \varepsilon_{sh} \quad (5.15)$$

$$f_s = f_y + (f_u - f_y) \left[2 \frac{\epsilon_s - \epsilon_{sh}}{\epsilon_u - \epsilon_{sh}} - \left(\frac{\epsilon_s - \epsilon_{sh}}{\epsilon_u - \epsilon_{sh}} \right)^2 \right] \quad \text{for } \epsilon_{sh} \leq \epsilon_s \leq \epsilon_u \quad (5.16)$$

Where f_y , f_{sh} and f_u are yield, strain hardening and ultimate stress, respectively; ϵ_y , ϵ_{sh} , ϵ_u are the corresponding strains; f_s , E_s and ϵ_s are stress, modulus of elasticity and strain in reinforcing steel, respectively.

The CFRP jacket material in tension is modelled as a linearly elastic material with experimentally established rupturing strength in tension and elastic modulus, as demonstrated by the coupon tests reported in Chapter 3. In compression, however, the CFRP is assumed to have reduced strength and modulus of elasticity. When RC-Blast was used, the compression strength and elastic modulus were taken as 25% of those reported in tension. This was observed by Sharbatdar and Saatcioglu (2004) in their tests. The same reduction was taken as one third of the corresponding tensile values by Mohammed (2014) in her computer program.

5.4.3. Nonlinear sectional analysis procedure

Both computer software employed utilize a standard flexural analysis based on strain compatibility and numerical integration. The procedure is based on: (i) linear strain distribution across the depth of the section (Bernoulli-Navier hypothesis of plane sections before bending remaining plane after bending), (ii) the nonlinear material stress-strain models described earlier, (iii) perfect bond between steel reinforcement and concrete, and (iv) the equilibrium of internal and external forces throughout the analysis. The effect of variable axial load is incorporated in the sectional analysis with linearly varying axial compression between the two experimentally observed forces.

The analysis involves numerical integration of sectional stresses and the equilibrium of forces in every load increment. It takes into account the progressive changes in loading and material properties. Figure 5.11 shows external forces (axial force and moment) acting on a column section with all possible strain distributions resulting from various combinations of axial forces and bending moments. Sectional forces are calculated using numerical integration. The cross section is divided into finite number of strips of equal width (the same as the width of the cross-section) and equal thickness (shown as t_{ci} in Figure 5.12-a). The FRP is identified as; (i) the

bottom FRP laminate strip, which is assumed to undergo tension, (ii) the top FRP Laminate strip, which is assumed to develop compression, and (iii) the two side FRP laminates. The strain in the extreme tension FRP strip is denoted by ε_{FRPi} , and the strain in the extreme compression FRP strip is denoted by ε_{FRPci} . Figure 5.12-c shows the stress distribution corresponding to the strain distribution in Figure 5.12-b; however, the stresses in the FRP laminate or the reinforcing steel are not shown. The forces acting on the section are shown in Figure 5.12-d. Equilibrium of internal forces and external compressive force and bending moment identify the strain distribution and the corresponding curvature. The presence of surface-bonded and wrapped FRP was accounted in the analysis. When RC-Blast was used for the sectional analysis, the FRP was included all around the section, with respective jacket thickness and material properties as described in Sec. 5.4.2. When the software developed by Mohammed was used, three different cases were considered, as illustrated in Figure 5.13 and described below.

1. **Case CFRP-C₁:** The CFRP laminate was assumed to have contribution only as tension element at the extreme tension fibre.
2. **Case CFRP-C₂:** The CFRP laminate was assumed to contribute as a tension element at the extreme tension face, as well as the side faces up to the neutral axis, which varied during the analysis.
3. **Case CFRP-C₃:** The CFRP laminate was assumed to have contribution all around the perimeter of the section as tension and compression elements.

5.4.4. Nonlinear member analysis procedure

Once the sectional analysis was completed, as described above, the variation of curvatures were numerically integrated to obtain rotations and deflections. The member was divided into small segments and the curvature in each segment was assumed to be constant, and was assigned a value corresponding to the moment imposed at that section as computed by the sectional analysis. RC Blast allowed the formation of plastic hinges beyond the peak resistance of the critical column section at mid-span. Beyond this point a pre-specified plastic hinge length was used, within which the curvature was assumed to be constant but vary according to the computed sectional response. The computer software by Mohammed followed the same procedure, except it did not incorporate plastification of the hinging region. Accordingly, the curvatures were integrated until the peak resistance of the column was encountered. Comparisons of sectional

moment-curvature and member force-displacement relationships with experimentally obtained values are presented in Sec. 5.4.5.

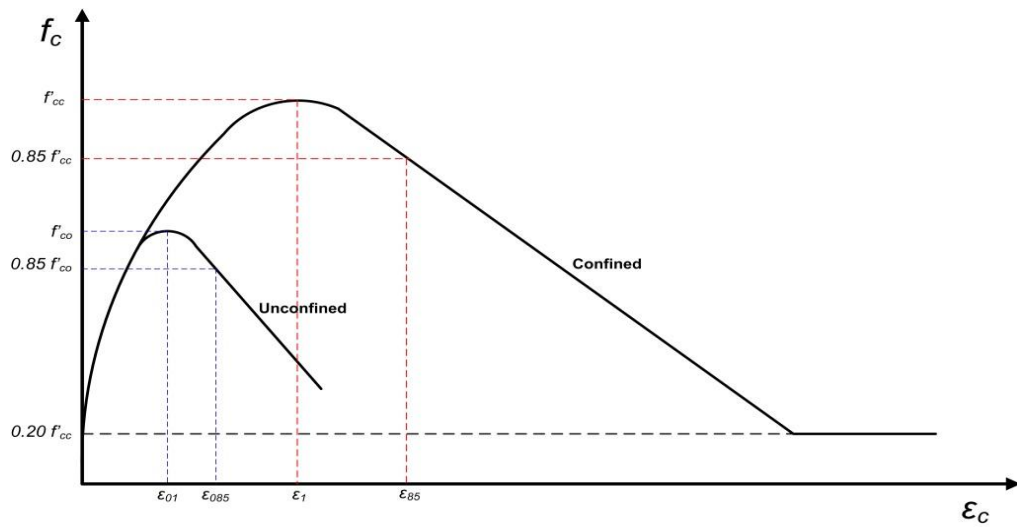


Figure 5.8. Stress-strain relationship for unconfined and confined concrete
(after Saatcioglu and Razvi 1992)

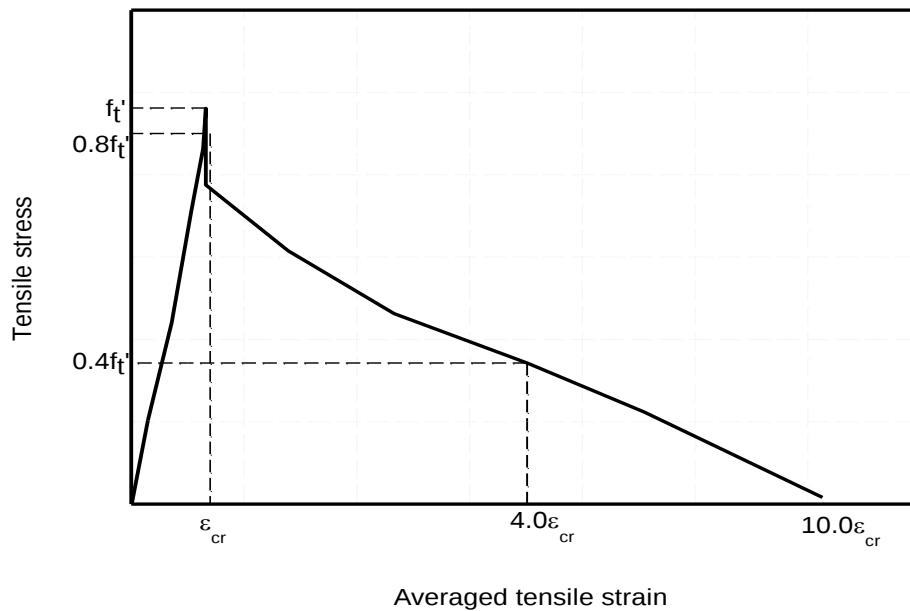


Figure 5.9. Tensile stress-average tensile strain relation (after Gilbert and Warner 1978)

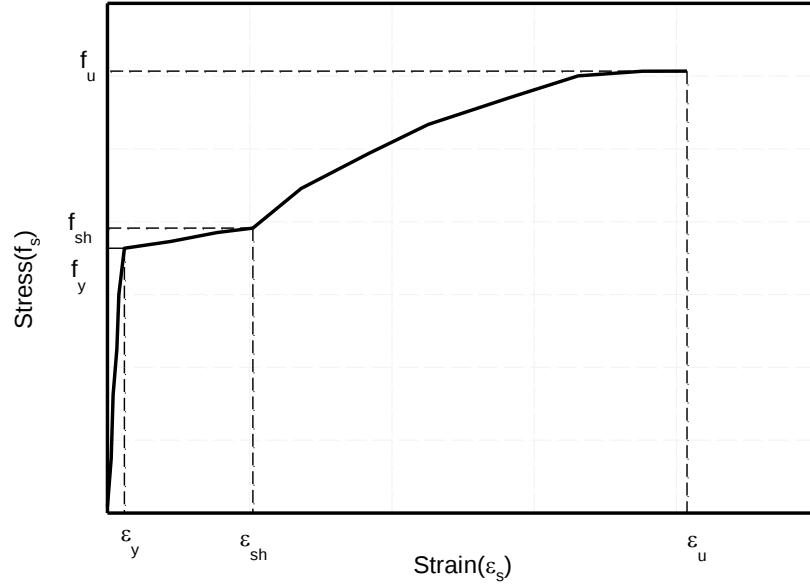


Figure 5.10. Stress-strain relationship for reinforcing steel (after Yalcin and Saatcioglu 2000)

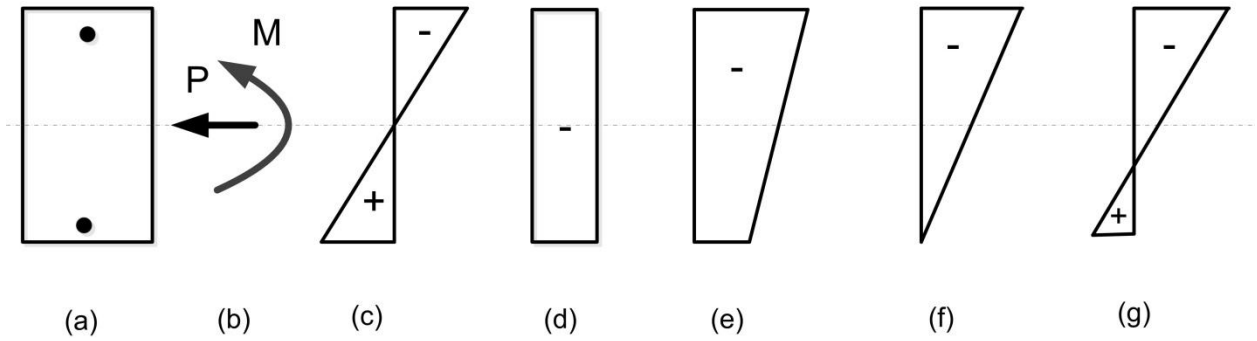


Figure 5.11. External forces on the section and possible strain distributions: (a) cross section; (b) forces on the section; (c) strain distribution due to flexure: case(i); (d) strain distribution due to axial load & flexure: case(ii); (e) strain distribution due to axial load & flexure: case(iii-1); (f) strain distribution due to axial load & flexure: case(iii-2); (g) strain distribution due to axial load & flexure: case(iii-3)

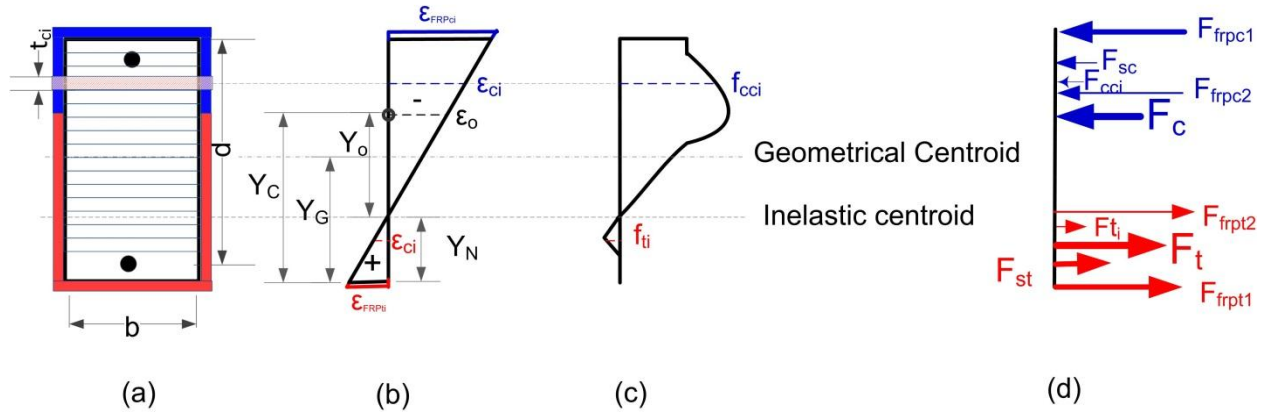


Figure 5.12. Numerical integration of nonlinear stresses over the cross section: (a) cross section divided into strips; (b) strain distribution due to axial load & flexure: ϵ_{ci} concrete strain, ϵ_{sti} steel strain; (c) stress distribution in concrete & steel: f_{cci} concrete stress at compressive level, f_{cti} concrete stress at tensile level, and f_{sti} steel stress; (d) internal forces in concrete & steel: F_c summation of internal forces of concrete (F_{cc}) and steel (F_{sc}) in compressive level, F_t summation of internal forces of concrete (F_{ct}) and steel (F_{st}) in tension level

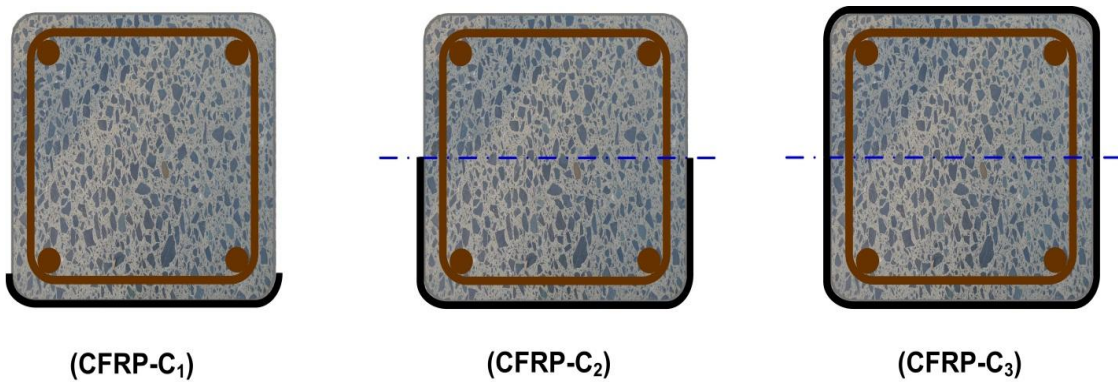


Figure 5.13. Three different alternatives for the contribution of the CFRP laminate in the nonlinear sectional analysis

5.4.5. Comparisons with experimental data

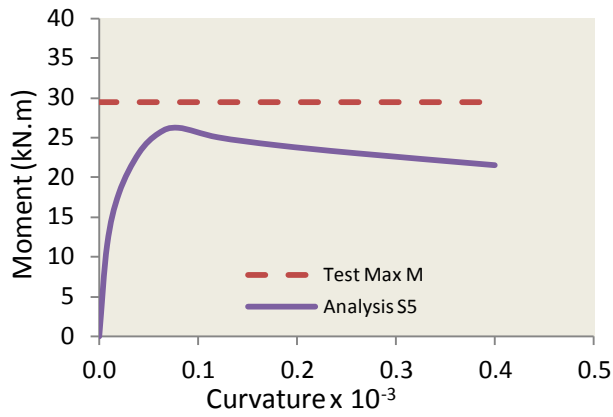
The moment curvature relationships, analytically generated as described in the preceding section, are compared with experimentally recorded flexural strength values in Figure 5.14 and Figure 5.15. The analyses were conducted with external CFRP jackets covering the entire column surface, when present, including tension and compression sides (CFRP-C3 in Figure 5.13). The

analytical results show good agreement with moment capacities, for most columns. This is especially true for the RC-Blast predictions involving CFRP jacketed columns, as these analyses were conducted with the confining effects of CFRP on the cover concrete and the combined confinement effects of external CFRP and internal steel hoops on the core concrete, where the comparisons in Figure 5.15 neglects the effects of internal hoops. Furthermore both approaches reflect the increase in strength associated with the use of CFRP jacket, while the RC-Blast also captures the point at which the CFRP ruptures, as observed in the tests, beyond which the curves drop down to the strength level associated with the internal steel reinforcement. The lower modulus of CFRP, relative to internal steel reinforcement, is also reflected in the sectional rigidities. Accordingly, the slopes of the ascending branches of the moment-curvature relationships for CFRP retrofitted columns are smaller than that for the control column (Column S5) beyond the yielding of internal steel. It is also noteworthy that the CFRP jackets with woven fabrics, and especially those that have inclined fibres ($\pm 45^\circ$) have lower strength and elastic modulus. This is reflected on the slopes of the ascending segments of the moment-curvature relationships; these columns also exhibiting lower flexural capacities than that of Column S1 with unwoven unidirectional fibres. Similar observations can be made in Figure 5.14(f) where the moment curvature relationships for all columns are compared.

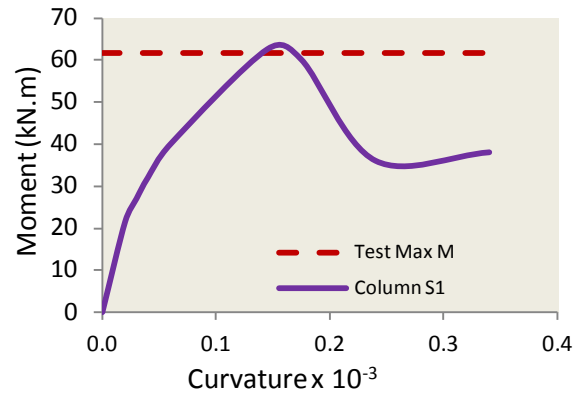
The comparisons of analyses results with experimental data extend to force-displacement relationships in Figures 5.16 and 5.17, where the results of analytically generated force displacement relationships are compared. The force term represents the total load applied on the column as lateral load (vertical during testing) in kilonewtons. The displacement term represents the mid-span deflection. The analyses were conducted with due considerations given to inelastic behaviour of materials, under variable axial load, as recorded during testing. The results shown in Figure 5.16, generated by the RC-Blast software, include a user-specified hinge length within which curvatures were assumed constant. A hinge length equal to the cross-sectional dimension of $h = 150$ mm was used for the analysis of the control column (S5). The tests indicated that there was a significant extension of the hinging region in CFRP retrofitted columns, beyond the formation of initial hinge, as the FRP becomes effective and contribute the load carrying capacity of the column. This phenomenon was also evident in the deflection profiles of test columns. Therefore, the analyses were repeated with different hinge lengths for Column S1, starting with a hinge length of $h=150$ mm, and repeating the analyses with $1.5h = 225$ mm as well

as $2h = 300$ mm. It was found that the increase in hinge length made the column softer under increasing loads, as expected. Though the impact on the analysis results was not very high, the use of twice the cross-sectional dimension produced closer to recorded test data in most cases. Therefore, this value was used in the analyses of other CFRP retrofitted columns. The results shown in Figure 5.16 show a reasonably good agreement with test data. A similar comparison is made with the results generated by the computer software developed by Mohammed (2014). However, this program did not allow plastic analysis with propagation of hinging beyond the peak moment resistance, and hence underestimated response in most cases. Figures 5.16(f) and 5.17(f) include analytical comparisons of force-displacement relationships for columns with different CFRP jackets used in the experimental program. Also included in the figures is the force-displacement relationship for the unretrofitted control column. The comparisons indicate the same trends indicated by the moment curvature relationships, discussed earlier. The use of externally applied CFRP jackets increased strength significantly. The level of increase was dictated by the strength and thickness of the laminate. Unidirectional unwoven fibres showed the highest effectiveness for strength increase.

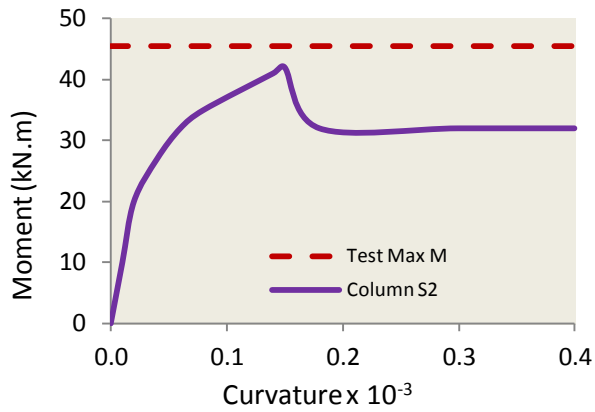
The significance of the use of different arrangement of CFRP in sectional analysis, as illustrated in Figure 5.13, and its effect on member behaviour was also investigated. The columns were analysed using the software developed by Mohammed (2014) with three different sectional modelling of CFRP; i) CFRP only on the tension face (C1), ii) CFRP extending as tension element up to the neutral axis (C2), and iii) CFRP covering the entire column, including the compression zone (C3). The comparisons of force-displacement behaviour are shown in Figures 5.18 through 5.21. The results clearly indicate that the consideration of the entire CFRP wrap all around the section (C3), with contributions to tension and compression, produce the best results in relation to the experimentally recorded force-displacement response. The same figures also provide comparisons between the retrofitted and control columns, indicating improved performance and increased capacity in the latter group, as expected.



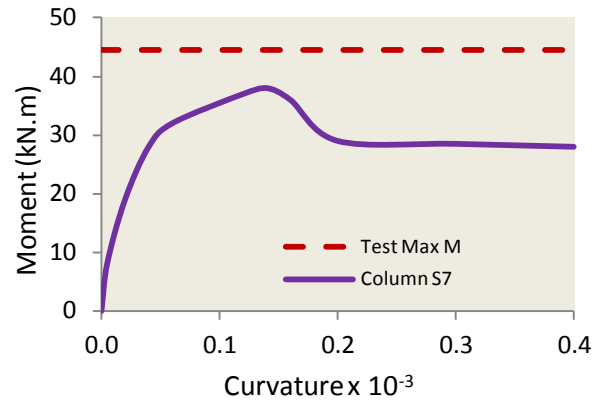
a) Column S5



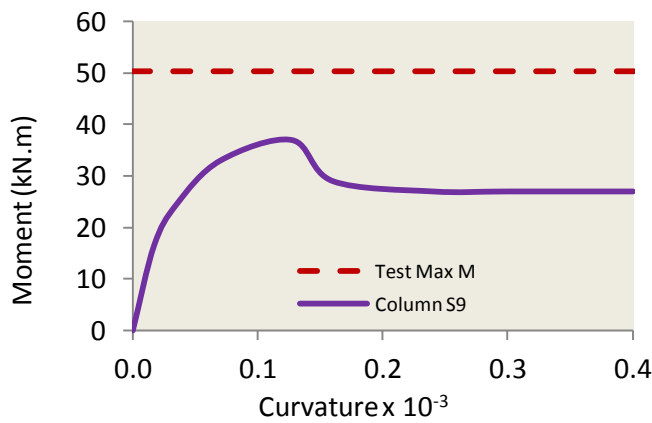
b) Column S1



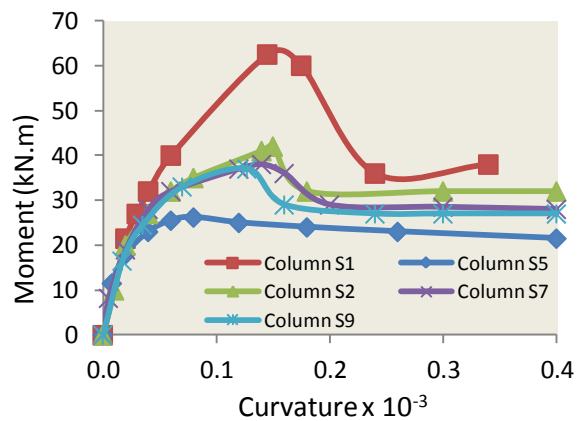
c) Column S2



d) Column S7

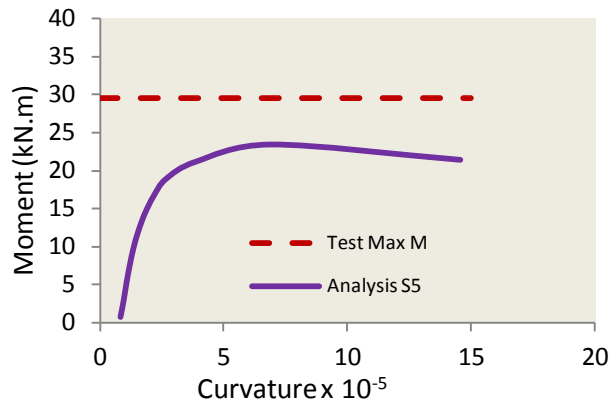


e) Column S9

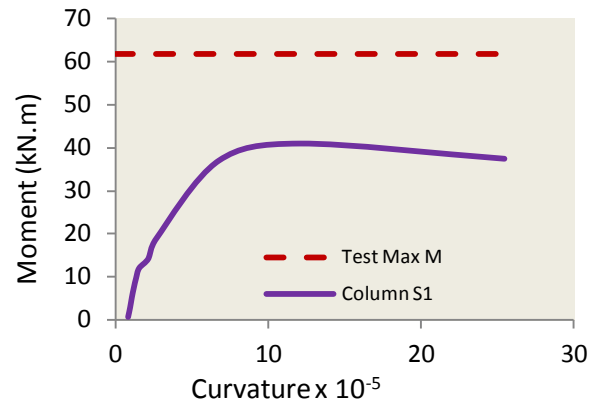


f) All columns

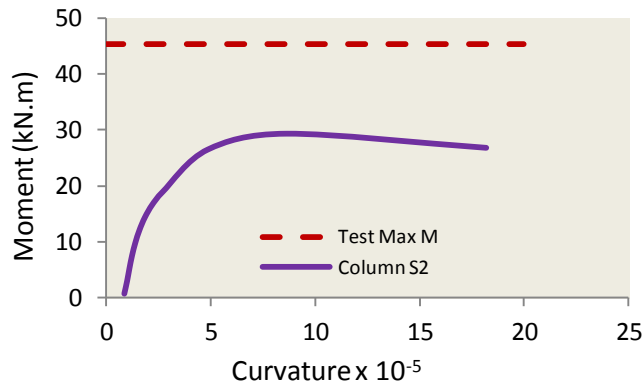
Figure 5.14. Comparisons of analytically generated moment-curvature relationships computed using RC-Blast software with experimentally recorded flexural capacities



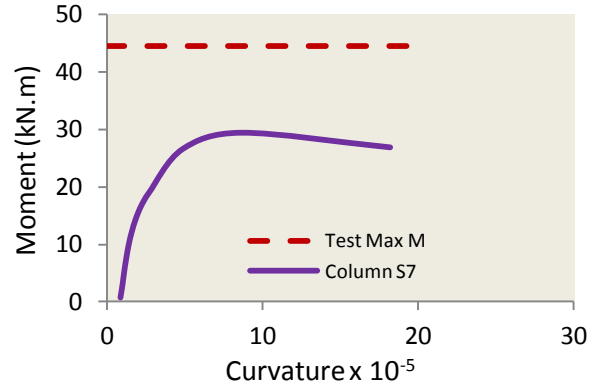
a) Column S5



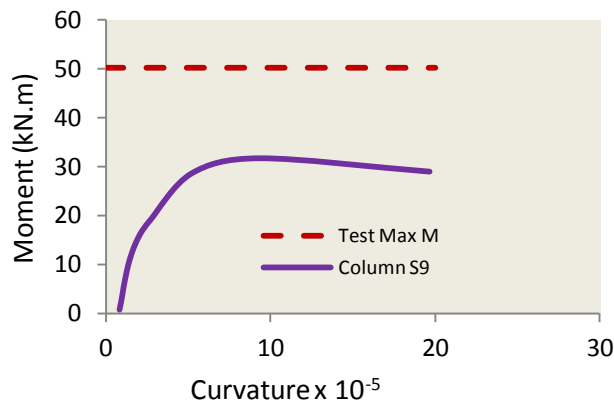
b) Column S1



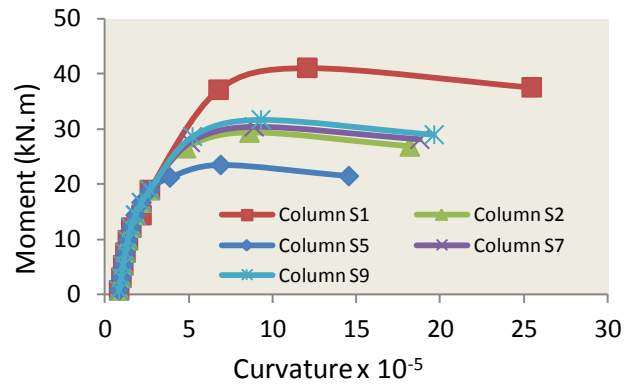
c) Column S2



d) Column S7

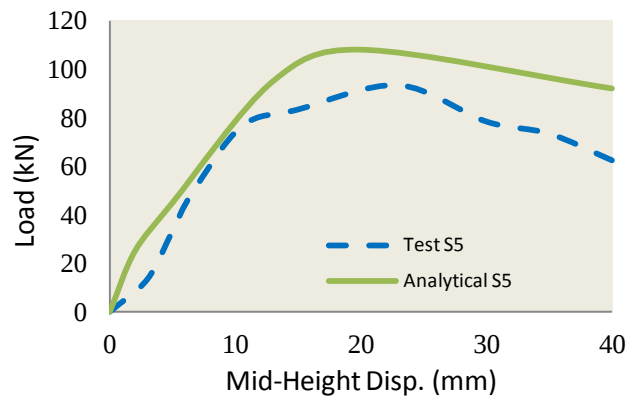


e) Column S9

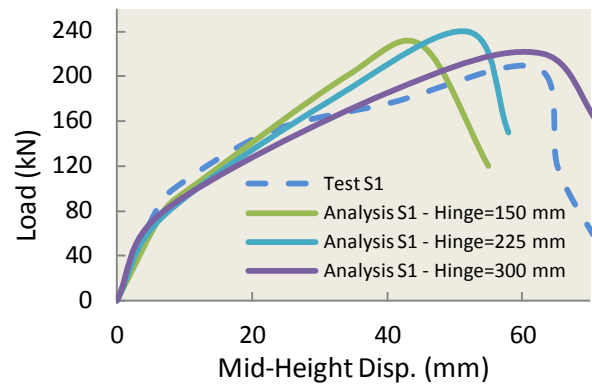


f) All Columns

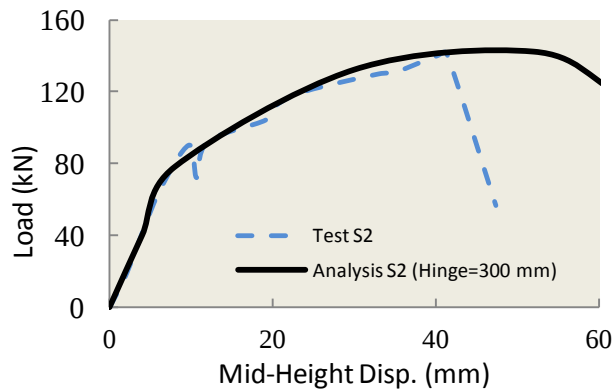
Figure 5.15. Comparisons of analytically generated moment-curvature relationships computed using Mohammed's software with experimentally recorded flexural capacities



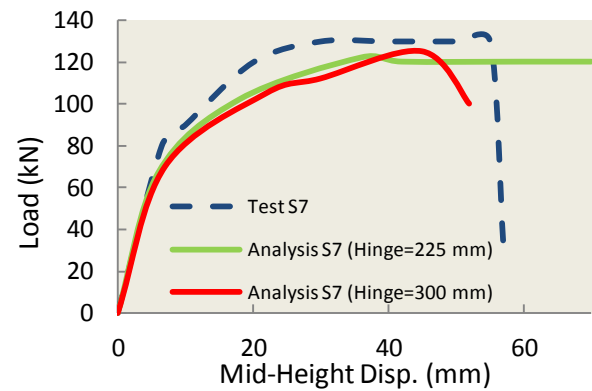
a) Column S5



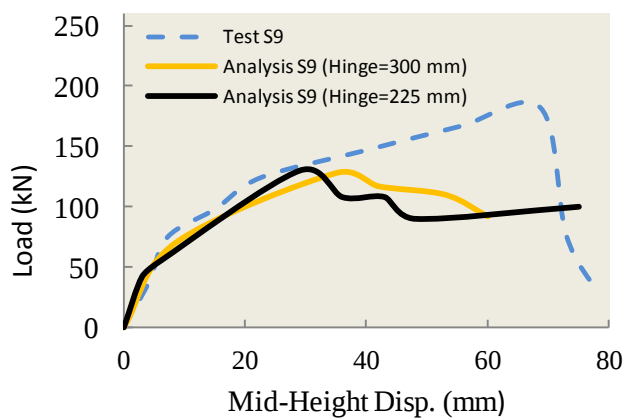
b) Column S1



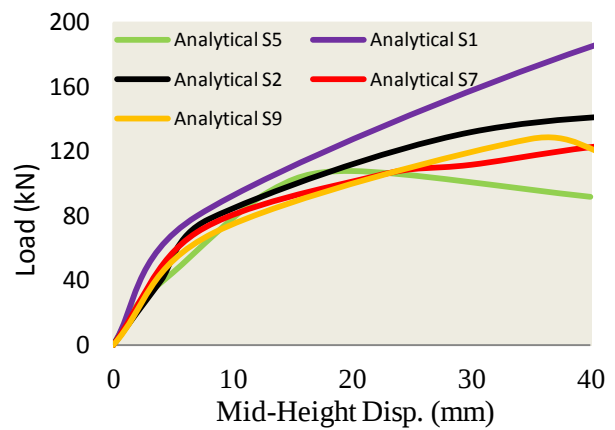
c) Column S2



d) Column S7

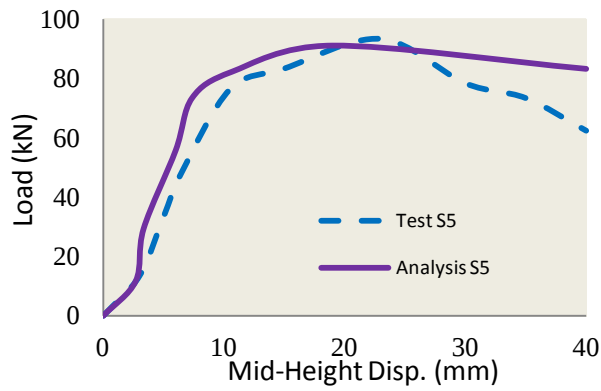


e) Column S9

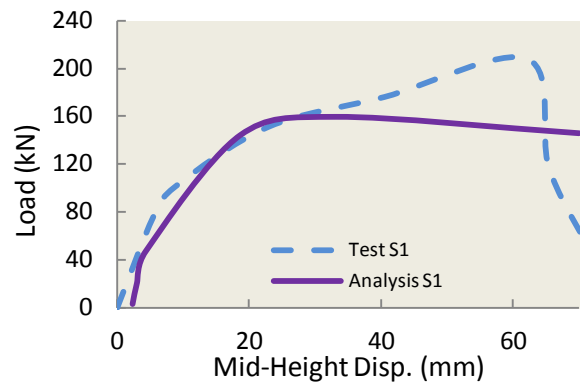


f) All columns

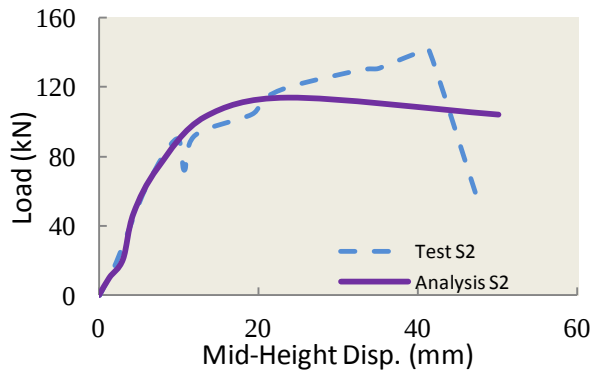
Figure 5.16. Comparisons of analytically generated force-displacement relationships computed using RC-Blast software with experimentally recorded data



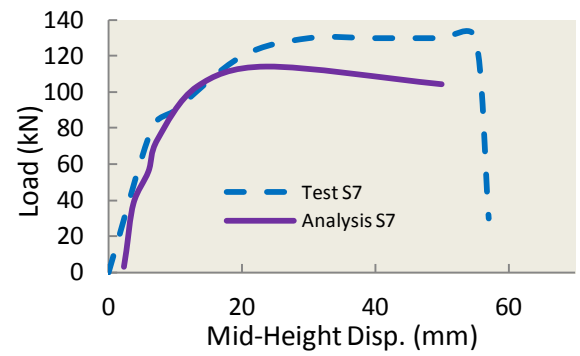
a) Column S5



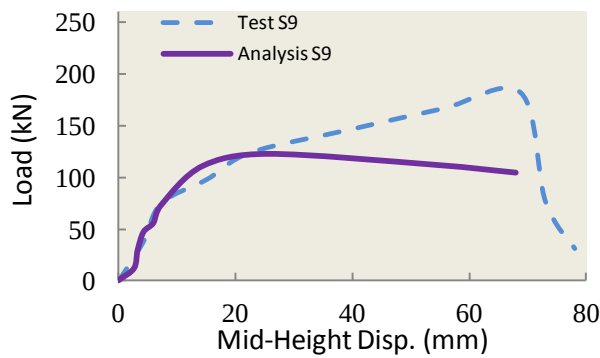
b) Column S1



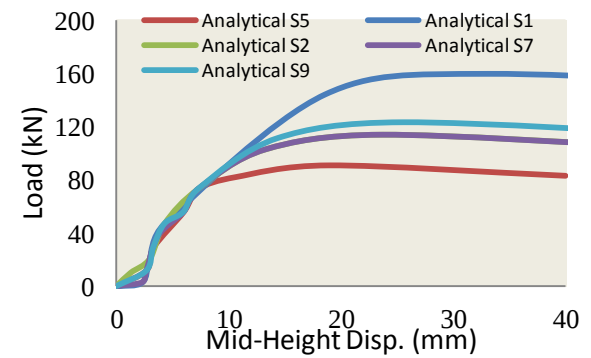
c) Column S2



d) Column S7



e) Column S9



f) All columns

Figure 5.17. Comparisons of analytically generated force-displacement relationships computed using Mohammed's software with test results

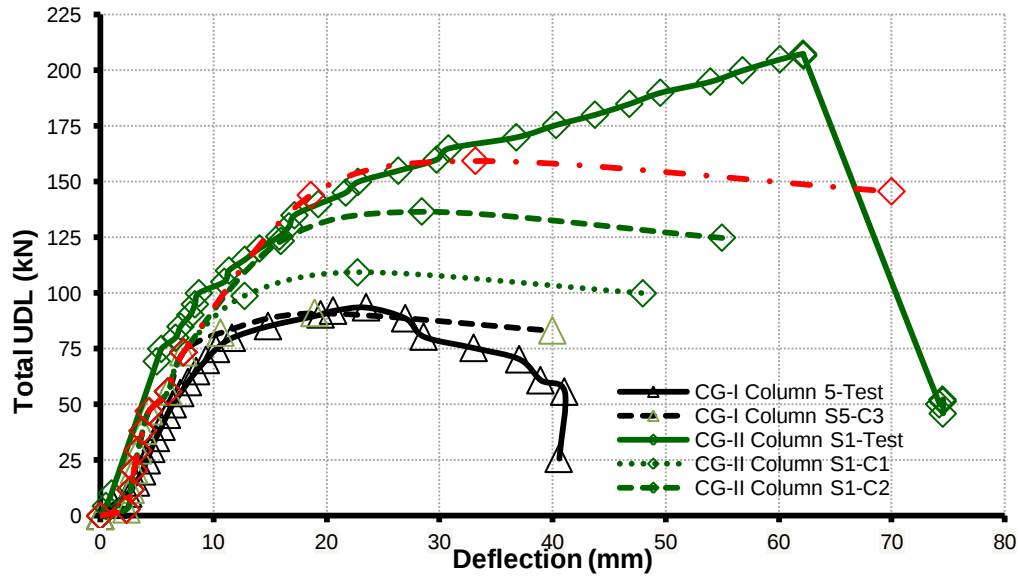


Figure 5.18. Comparison of the experimental with the analytical results of the total lateral UDL versus the deflection for Columns S1 and S5

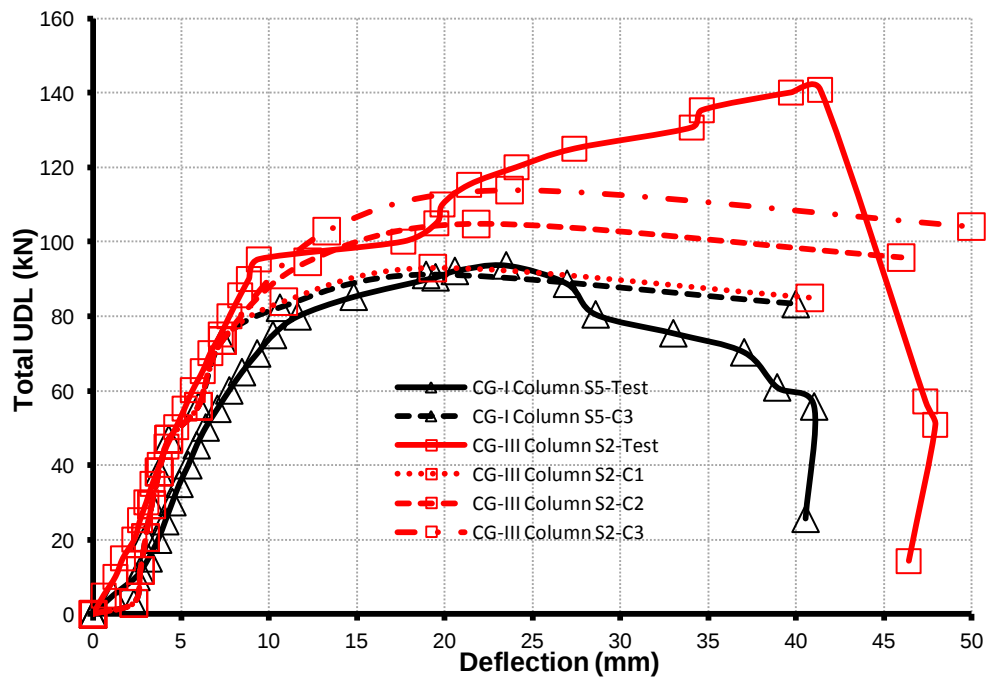


Figure 5.19. Comparison of the experimental with the analytical results of the total lateral UDL versus the deflection for Columns S2 and S5

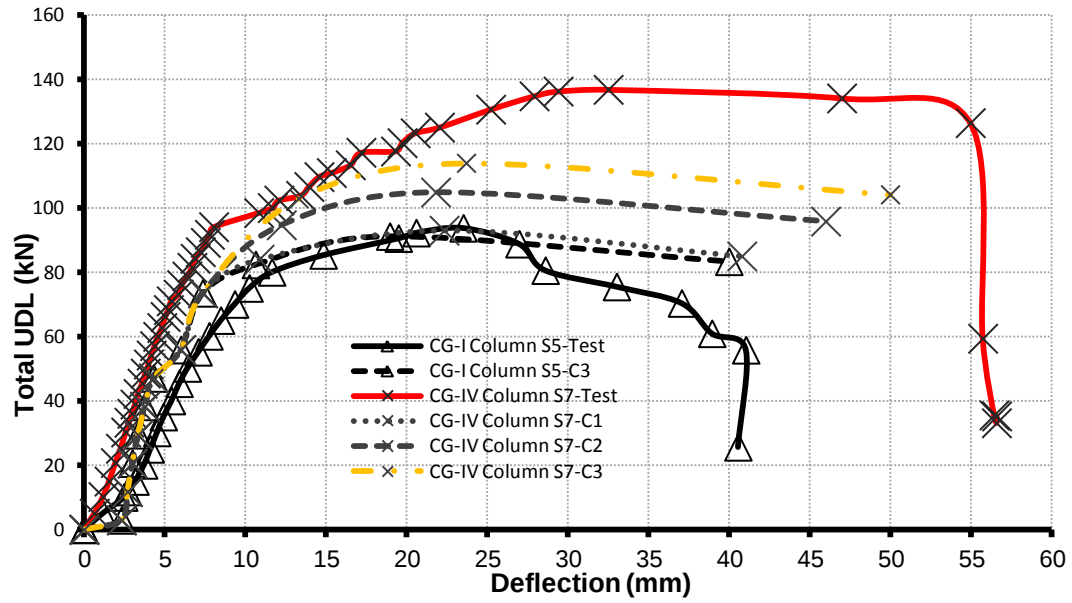


Figure 5.20. Comparison of the experimental with the analytical results of the total lateral UDL versus the deflection for Columns S7 and S5

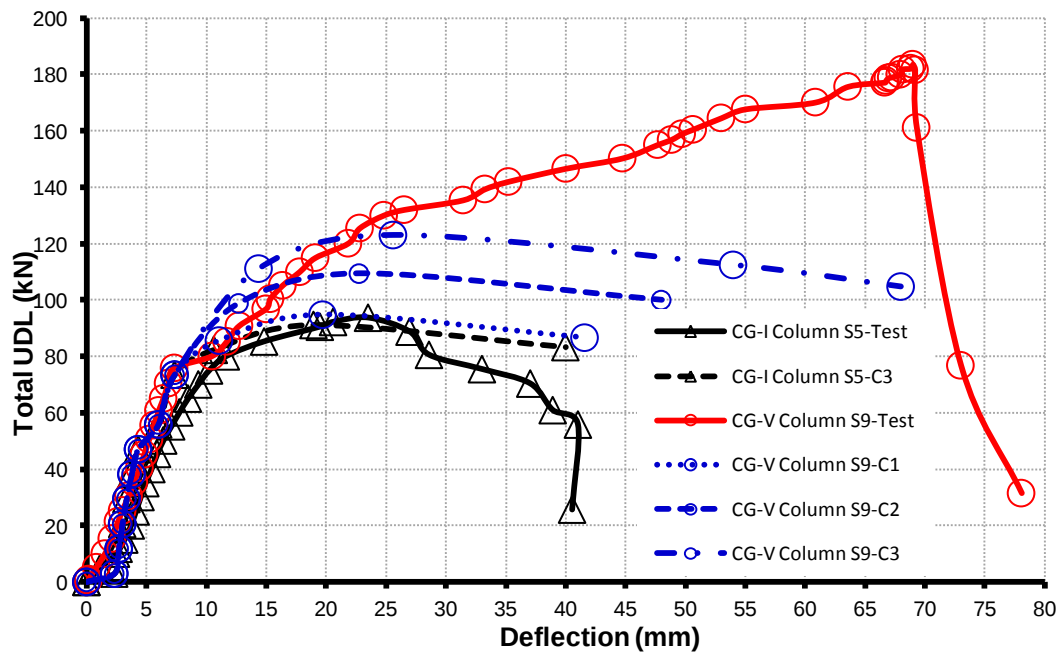


Figure 5.21. Comparison of the experimental with the analytical results of the total lateral UDL versus the deflection for Columns S9 and S5

Chapter 6: Summary and Conclusions

6.1. Summary

In this research project the effectiveness of CFRP laminates as externally applied column jacketing material was investigated experimentally. The experimental program consisted of nine half-scale seismically designed reinforced concrete columns, tested under combined axial compression and simulated lateral blast loading, applied statically. Seven columns were strengthened by four types of CFRP laminates. The columns were first subjected to service axial loads of 45% of the design axial load capacity of the control columns, followed by the lateral UDL load, up to failure. The tests were performed under static load conditions to be compared with the columns of a companion research projects that were tested under dynamic blast loads using the University of Ottawa's unique shock tube.

The columns were divided into five column groups; control column group and four additional retrofitted column groups with four different CFRP laminate designs. The CFRP laminate was laid-up around the columns over their entire length. The first column group CG-I included two control columns (S5 & S6) without any CFRP laminate applied. The second column group CG-II included only one column (S1) strengthened with laminate CFRP-L1, having unidirectional fibre with alignment sequence of $\{UD [0_2|90_2]\}$. The third column group CG-III included two columns (S2 and S4) strengthened with laminate CFRP-L2, having woven fibres aligned with alignment sequence of $\{W [0/90]_4\}$. The fourth column group CG-IV included two columns (S7 and S8) strengthened with laminate CFRP-L3, having a combination of 0/90 and ± 45 woven fibres with alignment sequence of $\{W [0/90]_2 W [\pm 45]_2\}$. The fifth column group CG-V included two columns (S3 and S9) strengthened with laminate CFRP-L4, having a combination of unidirectional 0, unidirectional 90 and ± 45 woven fibre with alignment sequence of $\{UD [0|90] W [\pm 45]_2\}$.

The columns were analyzed under incrementally increasing static loads, well into the inelastic range of deformations. This static push-over analysis consisted of two parts; i) sectional analysis, and ii) member analysis. The sectional analysis included a standard strain compatibility analysis. Both the internal reinforcing steel and the external FRP were considered in the analysis, with

post-yield behaviour of internal steel modelled for tension and compression, with strain hardening and buckling properties incorporated. The FRP was modelled as a linear material up to rupturing in tension. The behaviour of CFRP in compression was modelled with reduced strength and modulus of elasticity. The member analysis consisted of numerical integration of curvatures along the length of the members. Plastic hinge length was assumed to model the plastic behaviour. The analyses were carried out using two different computer software; i) RC-Blast (Jacques 2014) and ii) a program developed by Mohammed (2014). The experimental and analytical results were plotted in terms of force-displacement and moment-curvature relationships. Experimental results included column deflection profiles and strain gauge data. Observations made during the experimental phase were recorded. The results were analyzed and assessed to better understand the effectiveness of different laminate designs as CFRP column jackets, especially for applications to blast-resistant construction.

6.2. Conclusions

The following conclusions can be drawn from the results of the experimental investigation reported in Chapter 4 as well as the comparisons of test results and the column analyses performed in Chapter 5:

- (i) The use CFRP laminates as column jacketing improves strength and deformability of members significantly. The improvements are proportional to the area, strength and the elastic modulus of the FRP employed.
- (ii) The laminate CFRP-L1 that includes longitudinal and transverse unidirectional fibres (Column S1) showed the highest strength and second highest inelastic column deformability. This type of laminate also showed the highest elastic modulus, albeit, much lower than that of steel reinforcement. However, the failure was sudden with deflection pattern similar to the control column indicating no change of the plastic hinge length.
- (iii) Strengthening of RC columns with laminates formed from woven CFRP resulted in slight increases in column stiffness, strength and deformability. This type of a laminate, though

had fibres in the longitudinal and transverse direction, developed a lower elastic modulus associated with the process of wowing.

- (iv) The use of woven CFRP lamina with $\pm 45^\circ$ orientation in the “all woven CFRP laminate” resulted in improvements of column ductility, where a long plateau of the lateral load versus deflection relationship could be developed.
- (v) When the CFRP laminate, formed from the use of unidirectional fibres was employed, the strains in tension and compression reinforcement dropped with the increase of lateral loads and deformations. This indicates that the FRP provided a large percentage of total column resistance, reducing the contribution of steel reinforcement to flexure. The sudden failure of longitudinal fibres resulted in the loss of the additional strength gained, resulting in steel reinforced concrete behaviour, similar to that of the control column.
- (vi) The use of laminate formed from unidirectional CFRP and woven CFRP lamina with $\pm 45^\circ$ orientation appears to give the best compromise, increasing column flexural capacity while also improving deformation at failure. This may be a preferred lamina for blast resistance of columns. However, further research is needed, with additional testing, to substantiate this observation.
- (vii) The use of woven $\pm 45^\circ$ laminas results in well distribution of strains within the critical hinging region at higher levels of inelastic action, promoting redistribution of stresses with increased plastic hinge length, as evidenced by the deflection profiles tested in the experimental program.
- (viii) The use of standard plane-section analysis, with the effects of external fibres providing additional tension reinforcement and the confinement of compression concrete in the entire column section provides good estimates of column flexural capacity and moment curvature relationships that can be used in generating member response.
- (ix) Standard flexural member analysis, based on numerical integration of sectional response with an appropriate selection of hinging length provides force-displacement response that agrees reasonably well with recorded experimental force-displacement relationships. The comparisons indicate that the use of a hinge length equal to the column cross-sectional dimension “h” for unretrofitted columns, and an increased hinge length of up to twice the

column cross-sectional dimension ($2h$) provides good correlations with experimental results.

- (x) The analytically generated force-displacement relations can be used as resistance functions for blast load analysis under dynamic loads, with the appropriate dynamic increase factors incorporated.

6.3. Future Work

The present study opens the door for the following future research work,

1. Further experimental research both under equivalent static and dynamic impulsive forcing functions.
2. Investigation of the effect of the laminate thickness by increasing the lamina thickness and keeping the lay-up sequence.
3. Study of the effect of column scale on laminate thickness for all four laminate types.
4. Study of the effect of column axial load and lateral boundary conditions (e.g. fixed-pin, or fixed-fixed, etc) on the efficiency of column strengthening with laminates CFRP-L1 and CFRP-L4.
5. Dynamic analysis using the resistance functions from this study for the same columns when they are subjected to the same axial load and lateral blast pressure.
6. Investigation of the effect of using laminate CFRP-L4 on the column behaviour when subjected to seismic loading.
7. Development of a simplified design approach for FRP strengthening of columns subjected to different types of loading using laminate CFRP-L4.

References

- Arduini, M., & Nanni, A. (1997). "Behavior of precracked RC beams strengthened with carbon FRP sheets." *Journal of Composites for Construction*, 1(2), pp. 63-70.
- Berger, J.O., Heffernan, P.J., & Wight, R.G. (2013). Blast testing of CFRP and SRP strengthened RC columns. In *Design Against Blast: Load definition and structural response*, Ashurst Lodge, Southampton: WIT Press, pp. 189-198.
- Bonacci, J.F., & Maalej, M. (2000). Externally bonded fiber-reinforced polymer for rehabilitation of corrosion damaged concrete beams, *ACI Structural Journal*, 97(5), pp. 703-711.
- Buyukozturk, O. & Au, C. (2005). Effect of Fiber Orientation and Ply Mix on FRP Confined Concrete. Department of Civil and Environmental Engineering, Massachusetts Institute of Technology, Cambridge.
- Carroere, M., Wight, R.G., Heffernan, P.J. & Braimah, A. (2007). Effect of Blast Loads on Reinforced Concrete Strengthened with Steel Reinforced Polymer Sheets. FRPRCS-8. Patras, Greece.
- Crawford, J. E., Malvar, L. J., Morrill, K. B., & Ferritto, J. M. (2001). Composite retrofit to increase the blast resistance of reinforced concrete buildings. 10th International Symposium on Interaction of the Effects of Munitions with Structures, pp. 1-13.
- Demers, M., and Neale, K. W. (1999). Confinement of reinforced concrete columns with fiber-reinforced composite sheets an experimental study. *Can. J. of C. E.* 26, 226-241.
- Demers, M., Neale, W. K., & Sheikh, S. (2008). *ISIS Design Manual No. 4: FRP Rehabilitation of Reinforced Concrete Structures*. Canada: ISIS CANADA Research Network, Canada.
- Elnabelsy, G., and Saatcioglu, M., (2004a). Design of FRP Jackets for seismic retrofit of circular concrete columns. *Emirates Journal for Engineering Research*, 9(2), pp. 65–69. Published by United Arab Emirates University, (Issn-1022-9892).
- Elnabelsy, G., and Saatcioglu, M., (2004b). Seismic Retrofit of Circular and Square Columns with CFRP Jacket. *Proceedings of the 4th International Conference on Advanced Composite Materials in Bridges and Structures*, Calgary.
- Elnabelsy, G. and Saatcioglu, M. (2014). Seismic Behaviour of Concrete Bridge Columns Confined with Stay-in-Place Formwork. *Proceedings of the 7th International Conference on FRP Composites in Civil Engineering*, Vancouver.
- Harajli, M. H., and Rtiel, A. A. (2002). Effect of confinement using fiber-reinforced polymer or fiber-reinforced concrete on seismic performance of gravity load-design columns. *ACI Structural Journal*, 101(1), pp. 74-86.
- Hoshikuma, J., and Unjoh, S. (1997). Seismic retrofit of existing reinforced concrete columns by steel jacketing. *Proc., Italy-Japan Workshop on Seismic Design of Bridges*, pp.413-428.

- Howie, I., and Karbhari, V. M. (1994). Effect of materials architecture on strengthening efficiency of composite wraps for deteriorating columns in the North-East. *Infrastructure: New Materials and Methods of Repair*, Proc. 3rd Materials Conf., Material Engineering Division, ASCE, New York, pp. 199-206.
- Jacques, E. (2014). "RCBLAST version. 0.5.1." <<http://www.rcblast.ca/> (Nov. 1, 2014).
- Katsumata, H., Kobatake, Y., and Takeda, T. (1987). A study on strengthening with carbon fiber for earthquake-resistance capacity of existing reinforced concrete columns. *Proc. Seminar on Repair and Retrofit of Structures, Workshop on Repair and Retrofit of Existing Structures*, Tsukuba, Japan US-Japan Panel on Wind and Seismic Effects.
- Katsumata, H., Kobatake, Y., and Takeda, T. (1988). A study on strengthening with carbon fiber for earthquake-resistance capacity of existing reinforced concrete columns, *Proc. 9th World Conference on Earthquake Engineering*, Tokyo-Kyoto, VII, pp. 517-522.
- Kotynia, R., Baky, H.A., Neale, K.W., & Ebead, U.A. (2008). Flexural strengthening of RC beams with externally bonded CFRP systems: test results and 3D nonlinear FE analysis. *Journal of Composites for Construction*, pp. 190-201.
- Lacobucci, R. D., Sheikh, S. A., & Bayark, O. (2002). Retrofit of square concrete columns with carbon fiber reinforced polymer for seismic resistance. *ACI Structural Journal*, 100(6), pp. 785-794.
- Lee, S., & Moy, S. (2007). A Method for Predicting the Flexural Strength of RC Beams Strengthened with Carbon Fiber Reinforced Polymer. *Journal of Reinforced Plastics and Composites*, 26(14), pp. 1383-1401.
- Maalej, M., & Leong, K.S. (2005). Effect of beam size and FRP thickness on interfacial shear stress concentration and failure mode of FRP-strengthened beams. *Composites Science and Technology*, pp. 1148-1158.
- Mirmiran, A., Shahaway, M., and Samaan, M. (1997). Behavior of concrete columns confined by fiber composites. *Journal of Structural Engineering*, ASCE, 123(5), 583-590.
- Mirmiran, A., Shahawy, M., & Samaan, M. (1999). Strength and Ductility of Hybrid FRP-Concrete Beam-Columns. *Journal of Structural Engineering*, 125(10), pp. 1085-1093.
- Mohammed, A.M (2014). Semi-Quantitative Assessment Framework for Corrosion Damaged Slab-on-Girders Bridge Columns Using Simplified Nonlinear Finite Element Analysis, Ph.D. Thesis, University of Ottawa. Retrieved from: https://www.ruor.uottawa.ca/bitstream/10393/31020/1/Mohammed_Amina_2014_Thesis.pdf
- Muszynski, L.C., & Purcell, M.R. (2003). Composite Reinforcement to Strengthen Existing Concrete Structures Against Air Blast. *Journal of Composites for Construction*, ASCE, 7(2), pp. 93-97.

- Pan, J., Chung, T.C.F., & Leung, C.K.Y. (2009). FRP Debonding from Concrete Beams under Various Load Uniformities. *Advances in Structural Engineering*, 12(6), pp. 807-819.
- Pham, H.B., & Al-Mahaidi, R. (2006). Prediction Models for Debonding Failure Loads of Carbon Fiber Reinforced Polymer Retrofitted Reinforced Concrete Beams. *Journal of Composites for Construction*, ASCE, 10(1), pp. 48-59.
- Popovics, S. (1973). Analytical approach to complete stress-strain curves. *Cement and Concrete Res.*, 3(5), 583–599.
- Purba, B. K., and Mufti, A. A. (1999). Investigation of the behavior of circular concrete columns reinforced with carbon fiber reinforced polymer (CFRP) jackets. *Can. J. C. E.*, 26, 590-596.
- Rahai, A.R., Sadeghian, P., & Ehsani, M.R. (2008). Experimental Behavior of Concrete Cylinders Confined with CFRP Composites. *The 14th World Conference on Earthquake Engineering*, Beijing, China, pp. 1-9.
- Rodriguez-Nikl, T., Kobayashi, T., Oesterle, M.G., Lan, S., Morrillm, K.B., Hegemier, G.A., & Seible, F. (2009). Carbon Fiber Composite Jackets to Protect Reinforced Concrete Columns against Blast Damage. *Structures Congress 2009: Don't Mess with Structural Engineers*, pp. 2775-2783.
- Saatcioglu, M., Ozbakkaloglu, T. and Elnabelsy, G. (2008). Seismic Behavior and Design of Reinforced Concrete Columns Confined with FRP Stay-in-place Formwork, American Concrete Institute, ACI-SP 257(9), 145-165.
- Saatcioglu, M., & Razvi, S.R. (1992). Strength and Ductility of Confined Concrete. *Journal of Structural Engineering*, 118 (6), pp. 1590- 1606.
- Sause, R., Harries, K. A., Walkup, S. L., Pessiki, S., and Ricles, J. M., (2003). Flexural behaviour of concrete columns retrofitted with carbon fiber-reinforced polymer jackets. *ACI Structural Journal*, 101(5), Sept.-Oct. 2004, 708 716.
- Seible, F., Priestley, M. J. N., Hegemier, G. A., and Innamorato, D. (1995). Earthquake retrofit of bridge columns with continuous fiber jackets. Vol.II, Des. Guidelines, Advanced Composites.Technol. Transfer Consortium,Rep. No. ACTT-95/08, U of Calif., San Diego, La Jolla, Calif.
- Seible, F., Priestley, M. J. N., Hegemier, G. A., and Innamorato, D. (1997). Seismic retrofit of RC columns with continuous carbon fiber jackets. *J. Composites Const.*, ASCE, 1(2), 52-62.
- Sheikh, S. A., and Yau, G., (2002). Seismic behaviour of concrete columns confined with steel and fiber reinforced polymers. *ACI Structural Journal*, 99(1), 72 80.
- Teng, J. G., Chen, J. F., Smith, S. T., and Lam, L. (2002). *FRP strengthened structures*. John Wiley & Sons, Ltd., Chichester, England.

- Xiao, Y., and Ma, R. (1997). Seismic retrofit of RC circular columns using prefabricated composite jacketing. *Journal of Structural Engineering*, ASCE, 123(10), 1357-1364.
- Xiao, Y., Martin, G.R., Yin, Z., and Ma, R. (1995). Retrofit design of existing reinforced concrete bridge columns using prefabricated composite jacketing. *Proc., National Seismic Conference on Bridges and Hwy.*