

Seismic Retrofit of Concrete Columns by Transverse Prestressing

by

Amirreza Sabri

A thesis submitted to the Faculty of Graduate and Postdoctoral
Affairs in partial fulfillment of the requirements for the degree of
Master of Applied Science
in
Civil Engineering

University of Ottawa
Ottawa, Ontario

© Amirreza Sabri, Ottawa, Canada, 2013

Abstract

Performance of buildings and bridges during past earthquakes has indicated that many of these structures are vulnerable to seismic damage and structural collapse. The deficiencies in pre-1970s design codes have resulted in poor performance of reinforced concrete structures during seismic excitations. The Richter Magnitude 6.6 - 1971 San Fernando Earthquake raised awareness for seismic retrofit needs of existing buildings for the first time. The majority of deficiencies of vulnerable concrete columns can be overcome through seismic retrofits that involve additional transverse reinforcement. This can be done either by providing reinforced concrete, steel, or fibre-reinforced polymer (FRP) jackets around existing columns; or by applying transverse prestressing to columns (RetroBelt System).

The research project presented in this thesis involves a seismic retrofit methodology for seismically deficient building and bridge columns, utilizing the use of high-strength packaging straps as external reinforcement for transverse prestressing. The emphasis in the project is placed on experimental research. Three seismically deficient full-size reinforced concrete columns, with a circular, a square and a rectangular cross-section, either critical in shear or flexure, were designed, built and tested under simulated seismic loading. The results indicate that external prestressing of columns in transverse direction with high-strength steel straps improves ductility and energy dissipation capacity of seismically deficient columns. They further indicate that current analytical techniques can be used to predict the force-displacement relationships of columns. A design approach is presented for the retrofit methodology investigated.

Acknowledgements

I wish to express my gratitude to my supervisor Dr. Murat Saatcioglu for his endless academic and moral support. Appreciations are extended to the Structures Laboratory supervisor Dr. Muslim Majeed, and the Machine Shop supervisor Mr. John Perrins for their never-ending assistance during the experimental research. I am grateful to my graduate student peers for helping me during different stages of specimen preparation.

I am indebted to my precious family for their life-long support, including their continuous moral support during my research. Last but not least, I would like to express my appreciations to my dear girlfriend for her limitless moral support and assistance during my research.

Once again, I am grateful to my supervisor, my colleagues, my family and friends, without whose continuous support the completion of this thesis would have been much more difficult.

Table of Contents

Abstract.....	ii
Acknowledgements	iii
Table of Contents	iv
List of Figures.....	vii
Notations	xii
Chapter 1: Introduction	1
1.1 Background	1
1.2 Objective	3
1.3 Scope	3
1.4 Literature Review	4
1.4.1 Steel Jacking	4
1.4.2 Reinforced Concrete Jacketing	8
1.4.3 Fiber Composites	13
1.4.4 External Prestressing (Retrobelt)	20
1.4.5 Conclusion from literature review	25
Chapter 2: Experimental Program	27
2.1 Introduction	27
2.2 Column Details.....	27
2.2.1 Flexure-Dominant Circular Column (F-CR)	28
2.2.2 Flexure-Dominant Rectangular Column (F-RC)	28
2.2.3 Shear-Dominant Square Column (S-SQ).....	29
2.3 Preparation of Column Specimens.....	29
2.3.1 Footings	29

2.3.2	Columns	30
2.4	Retrofitting Phase	30
2.4.1	Retrofit Materials and the Installation of Packaging Straps	31
2.4.2	Concrete Raiser Units	32
2.5	Material Properties	34
2.5.1	Concrete	34
2.5.2	Reinforcing Steel	35
2.5.3	High Strength Steel Straps	36
2.6	Test Setup	37
2.7	Loading Program	37
2.8	Instrumentation	38
Chapter 3: Test Results		69
3.1	Introduction	69
3.2	Flexural Dominant Circular Column (F-CTR)	70
3.2.1	Test Observations	70
3.2.2	Data Analysis	72
3.3	Flexural Dominant Rectangular Column (F-CR)	73
3.3.1	Test Observations	73
3.3.2	Data Analysis	75
3.4	Comparisons of Retrofitted Circular and Rectangular Columns with Other Flexural Dominant As-built Columns	76
3.5	Shear Dominant Square Column (S-SQ)	79
3.5.1	Test Observations	79
3.5.2	Data Analysis	81
3.6	The effect of Column Retrofit	82

Chapter 4: Analytical Investigation	115
4.1 Introduction	115
4.2 Flexural Moment-Deflection Behavior of Columns	115
4.2.1 Moment-curvature Relationship	116
4.2.2 Confined Concrete Model.....	117
4.2.3 RC-Section Software	122
4.3 Flexural Deformations of Concrete Columns	122
4.4 Shear Capacity of Concrete Columns	123
4.5 Comparison of Experimental Results with Analytical Results	125
4.5.1 Circular Building Column.....	126
4.5.2 Rectangular Building Column	127
4.5.3 Square Column.....	128
4.6 Effect of Transverse Prestressing on Column Deformability	129
Chapter 5: Design Recommendations and Conclusions.....	138
5.1 Introduction	138
5.2 Design Procedure	139
5.2.1 Shear Design	139
5.2.2 Confinement Design	140
5.2.3 Design for Splice Clamping.....	142
5.3 Conclusion.....	142
5.4 Recommendations for Future Research	144
References.....	146

List of Figures

Figure 2-1	Geometric and reinforcement details for circular column F-CR	40
Figure 2-2	Geometric and reinforcement details for rectangular column F-RC.....	41
Figure 2-3	Geometric and reinforcement details for square column S-SQ	42
Figure 2-4	Footing formwork	43
Figure 2-5	Footing formwork and reinforcement cages	43
Figure 2-6	Footings after concrete casting.....	44
Figure 2-7	Column reinforcement cages.....	44
Figure 2-8	Column formwork after casting	45
Figure 2-9	Pneumatic tensioner	45
Figure 2-10	Pneumatic sealer machine	46
Figure 2-11	Sealer.....	46
Figure 2-12	Prestressing process	46
Figure 2-13	Retrofit details of circular column	47
Figure 2-14	Retrofit details of rectangular column.....	47
Figure 2-15	Retrofit details of square column	48
Figure 2-16	Retrofit details of square column	48
Figure 2-17	Concrete raiser unit formwork built around columns	49
Figure 2-18	Casting of the first layer of concrete raiser units for square column	49
Figure 2-19	Placement of plywood on top of the first layer, and casting of the second layer.....	50
Figure 2-20	Third layer of concrete raiser units after concrete casting	50

Figure 2-21	Opening of sonotubes after one week	51
Figure 2-22	Removing the concrete raiser units out of the formwork.....	51
Figure 2-23	Placement of cement paste between concrete raiser units and columns	51
Figure 2-24	Temporary prestressing of concrete raiser units as the cement paste dries	52
Figure 2-25	All three columns after final prestressing, from left to right; square column, rectangular column, and circular column.....	52
Figure 2-26	Concrete compression testing machine.....	53
Figure 2-27	Stress-strain relationship of No. 10 steel bar used as transverse reinforcement	53
Figure 2-28	Stress-strain relationship of No. 25 steel bar used as longitudinal reinforcement	54
Figure 2-29	From left to right: Coupon Type 1 - Solid Strap (no sealer), Coupon Type 2 - Two straps connected by a sealer, and Coupon Type 3 - Three straps connected by a sealer	54
Figure 2-30	From left to right: Galdabini testing machine, and Coupon Types 1, and 2 after the rupturing of steel.....	55
Figure 2-31	Stress-strain relationship of Coupon Type 1 (solid strap - no sealer)	55
Figure 2-32	Stress-strain relationship of Coupon Type 2 (Two straps connected by a sealer).....	56
Figure 2-33	Stress-strain relationship of Coupon Type 3 (Three straps connected by a sealer).....	56
Figure 2-34	Details of loading beam assembly.....	57
Figure 2-35	Side view of test setup.....	58

Figure 2-36	Front view of test setup	59
Figure 2-37	Photos of test setup.....	60
Figure 2-38	Loading program.....	61
Figure 2-39	Strain gauge locations on reinforcing steel for circular column	62
Figure 2-40	Locations of straps with strain gauges for the circular column (each strap has three strain gauges on east, north, and south sides).....	63
Figure 2-41	Strain gauge locations on reinforcing steel for the rectangular column.....	64
Figure 2-42	Locations of straps with strain gauges for the rectangular column (each strap has three strain gauges on east, north, and south side directions).....	65
Figure 2-43	Strain gauge location on reinforcing steel for the square column.....	66
Figure 2-44	Locations of straps with strain gauges for the square column (each strap has three strain gauges on east, north, and south side directions).....	67
Figure 2-45	Location of LVDTs and cable transducers.....	68
Figure 3-1	Behaviour of circular column.....	84
Figure 3-2	Behaviour of circular column.....	85
Figure 3-3	Moment-displacement and Force-displacement relationships for circular column.....	86
Figure 3-4	Moment-Rotation relationship for circular column	87
Figure 3-5	Transverse reinforcement steel strains for circular column	88
Figure 3-6	Longitudinal reinforcement steel strains for circular column	89
Figure 3-7	High strength steel strap strains for circular column	90
Figure 3-8	High strength steel strap strains for circular column	91
Figure 3-9	High strength steel strap strains for circular column	92

Figure 3-10	Behaviour of rectangular column.....	93
Figure 3-11	Behaviour of rectangular column.....	94
Figure 3-12	Moment-displacement and Force-displacement relationship for rectangular column.....	95
Figure 3-13	Moment-rotation relationship for rectangular column.....	96
Figure 3-14	Transverse reinforcement steel strains for rectangular column	97
Figure 3-15	Longitudinal reinforcement steel strains for rectangular column	98
Figure 3-16	High strength steel strap strains for rectangular column.....	99
Figure 3-17	High strength steel strap strains for rectangular column.....	100
Figure 3-18	High strength steel strap strains for rectangular column.....	101
Figure 3-19.....		102
Figure 3-20.....		103
Figure 3-21	Rupture of 4 th strap and development of new cracks on the concrete raiser units after 6% drift	104
Figure 3-22.....		105
Figure 3-23	Behavior of square column	106
Figure 3-24	Moment-displacement and Force-displacement relationships for square column.....	107
Figure 3-25	Moment-rotation relationship for square column	108
Figure 3-26	Transverse reinforcement steel strains for square column.....	109
Figure 3-27	Longitudinal reinforcement steel strains for square column.....	110
Figure 3-28	High strength steel straps strains for square column.....	111
Figure 3-29	High strength steel straps strains for square column.....	112

Figure 3-30	High strength steel straps strains for square column.....	113
Figure 3-31	Moment-displacement relationship of non-retrofitted square column, tested by Yalcin (1997)	114
Figure 4-1	Lateral pressure in circular columns: (a) Uniform buildup of pressure; and (b) Computation of lateral pressure from hoop tension	131
Figure 4-2	Lateral pressure in square column: (a) Lateral pressure buildup in square columns; and (b) Pressure distribution resulting from different reinforcement arrangements	131
Figure 4-3	Distributions of lateral pressures: (a) Distributions of lateral pressure along member length; and (b) Actual, average, and equivalent lateral pressures	132
Figure 4-4	Lateral pressure distribution in rectangular column.....	133
Figure 4-5	Stress-strain relationship of unconfined and confined concrete	133
Figure 4-6	Relationship between unidirectional ductility and concrete shear contribution.....	134
Figure 4-7	Definition of column core dimension	134
Figure 4-8	Axial force shear contribution.....	135
Figure 4-9	Comparison of analytical and experimental results for circular column ...	136
Figure 4-10	Comparison of analytical and experimental results for rectangular column	136
Figure 4-11	Comparison of analytical and experimental results for square column ...	137

Notations

A_e	Effective concrete shear transfer area
A_v	Area of shear reinforcement within a distance s
A_h	Area of one leg of transverse reinforcement
A_{ps}	Nominal area of prestressing straps
A_s	Area of one reinforcing bar
b_c	Core dimension measured center to center of perimeter hoop
b_{cx}	Core dimension measured center to center of perimeter hoop in x direction
b_{cy}	Core dimension measured center to center of perimeter hoop in y direction
d	Effective depth of tension reinforcement
D'	Core dimension of column in the loading direction
E_c	Modulus of elasticity of plain concrete
E_p	Modulus of elasticity of prestressing strap
f_c	Stress in concrete
f'_c	Cylinder compressive strength of concrete specimens
f'_{cc}	Confined concrete compressive strength in member
f'_{co}	Unconfined concrete strength in member
f_l	Average confinement pressure
f_{le}	Equivalent lateral pressure that produces the same effect as uniformly

	applied pressure
f_{pe}	Effective stress in prestressing straps
f_{pi}	Initial stress in straps
f_{pu}	Ultimate stress in straps
f_{py}	Yield stress in straps
f_s	Maximum steel stress in the elastic range
f_y	Yield strength of reinforcing steel
f_{yt}	Yield capacity of transverse steel
h	Average of cross-sectional dimensions in two orthogonal directions
K	Coefficient used in Eq.4.12
k	Strength enhancement coefficient
k_1	Coefficient that relates confinement pressure to strength enhancement
k_2	Coefficient that reflects efficiency of confined reinforcement
l_p	Plastic hinge region
P	Axial force on column
P_o	Axial capacity under pure compression
s	Spacing of transverse reinforcement
s_{ps}	Spacing of prestressing straps along the column height
s_l	Spacing of longitudinal reinforcement, laterally supported
V_c	Normal shear strength provided by concrete
V_e	Seismic shear force
V_p	Total shear resistance provided by external prestressing

V_{pi}	Shear contribution of initial prestressing force to nominal shear strength
V_{ps}	Total shear contribution of prestressing strands to nominal shear strength
V_s	Shear provided by transverse reinforcement
α	Angle formed between column axis strut from application point of axial load to center of compression zone in plastic hinge region
ε_c	Concrete strain
ε_1	Strain corresponding to peak stress of confined concrete
ε_{01}	Strain corresponding to peak stress of unconfined concrete
ε_{085}	Strain at 85% of peak unconfined stress
ε_s	Strain in reinforcing steel
ε_{sh}	Strain hardening strain of reinforcing steel
ε_y	Yield strain of reinforcing steel
δ	Design drift ratio
μ_Δ	Displacement ductility of column
φ	Capacity reduction factor for axial compression
ρ	Total transverse steel area in two orthogonal directions divided by corresponding concrete area

Chapter 1: Introduction

1.1 Background

Performance of buildings and bridges during past earthquakes has indicated that many of these structures are vulnerable to seismic damage and structural collapse. The deficiencies in pre-1970s design codes result in undesirable behaviour of reinforced concrete structures during seismic excitations. The Richter Magnitude 6.6 - 1971 San Fernando Earthquake raised awareness for seismic retrofit needs of existing buildings for the first time. As a result, several researches assessed seismic risk in existing infrastructure, while others identified seismic deficiencies in structural elements, particularly in foundations, columns, beams, beam-column connections, slab-column connections. This resulted in the modification of building and bridge design codes. In addition, seismic retrofit methodologies were developed. The Richter Magnitude 7.1 - 1989 Loma Prieta Earthquake showed the effectiveness of improved design codes, and acceptable behaviour of seismically retrofitted elements.

A significant proportion of seismic damage in existing infrastructure is attributed to poor column behaviour. The main problem of pre-1970s columns is the use of insufficient transverse reinforcement. Transverse reinforcement in columns serves for three main function; i) improves shear resistance, ii) confines compression concrete, and iii) prevents buckling of compression reinforcement. Lack of properly designed transverse reinforcement results in column failures due to diagonal tension caused by shear, crushing of concrete due to lack of confinement, and premature buckling of

reinforcement. Furthermore, it has been a common practice to provide lap splices for column reinforcement in the potential plastic hinge regions of columns near the ends in older columns. This results in another important failure mode.

Shear failure is the most undesirable failure mode in columns, resulting in brittle behaviour and sudden failure. This failure mode is characterized either by opening of wide diagonal cracks due to diagonal tension or crushing of diagonal struts under high diagonal compressive stresses. In flexure-dominant columns, while properly confined columns can yield and develop significant ductility, dissipating earthquake-induced energy, lack of concrete confinement can result in brittle crushing of compression concrete and sudden failure of column. The third type of column failure is triggered by the slippage of longitudinal reinforcement in the lap-splice region, if this region is located at or near plastic hinging and does not have sufficient transverse reinforcement to confine concrete and improve bond. Seismically deficient columns are therefore retrofitted with steel, reinforced concrete or fibre reinforced polymer (FRP) jackets to minimize damage associated with lack of transverse reinforcement. A new column retrofit methodology was developed at the University of Ottawa earlier, involving transverse prestressing of columns by means of 7-wire strands. This technique, known as “RetroBelt,” provides additional transverse reinforcement while applying active lateral pressure for improved concrete confinement, delayed diagonal cracking due to shear, and improved bond and lap splice performance. The effectiveness of the technology has been demonstrated through extensive experimental research. While it is relatively easy to implement external prestressing using 7-wire strands in large size bridge columns, it creates challenges in smaller size building columns. Furthermore, prestressing often requires specialized

labour adding additional complexities, even though the application of RetroBelt is significantly less labour and material intensive when compared with steel, reinforced column and FRP jackets. An alternative to use of 7-wire strands, in the form of high-strength packaging straps is explored in the current research project as a viable column retrofit strategy.

1.2 Objective

The objective of the current research project is to develop a seismic retrofit methodology for seismically deficient building and bridge columns, involving the use of high-strength packaging straps as external reinforcement for transverse prestressing. The objective includes the verification of the technology through testing and the development of a design methodology for the approach.

1.3 Scope

The scope of research is outlined below:

- Design of three seismically deficient full-size reinforced concrete columns, with a circular, a square and a rectangular cross-section, either critical in shear or flexure.
- Assembly of steel cages, building of formwork, casting of concrete and the instrumentation of specimens for data measurement.
- Retrofitting the specimens with externally placed transverse steel straps, and prestressing them laterally for improved concrete confinement and shear strength enhancement.
- Tests of columns under constant axial compression and incrementally increasing lateral deformation reversals.

- Data analysis and assessment of test variables.
- Analysis of columns and comparison with experimental results.
- Development of a design procedure for seismic retrofit of columns by transverse prestressing through the use of high-strength steel straps.
- Presentation of results.

1.4 Literature Review

The available literature on previous research and existing column retrofit methodologies are reviewed prior to experimental research. The following is the review of literature, organized based on the retrofit technique studied/implemented. The review consists of four major retrofit techniques, involving; i) steel jacketing, ii) reinforced concrete jacketing, iii) FRP jacketing, and iv) transverse prestressing (RetroBelt).

1.4.1 Steel Jacketing

Chai et al. (1991) conducted tests of 0.4 scale retrofitted circular columns to investigate the improvements obtained due to steel jacketing, on bridge columns. Potential plastic hinge regions of columns near the base were enclosed with site-welded cylindrical steel jackets. A gap of 0.25 in (6.35-mm) was left between the exterior surface of the column concrete and the steel jacket for ease of installation. This gap was subsequently filled with water-cement grout. The steel jacket was placed 25 mm above the footing to prevent the bearing of jacket on foundation concrete during load reversals. All the columns in the experimental program were subjected to a constant axial load of 400 kips (1779 KN). The lateral force was applied in the form of incrementally increasing lateral drift cycles. Tests indicated that, steel jackets with 3.2% volumetric ratio of confinement steel in plastic hinge regions increased ultimate compressive strain,

flexural strength, and ductility of columns for both lap-spliced columns and columns with continuous longitudinal reinforcements. The lateral stiffness of steel jacketed column was also increased by 10 to 15 percent. The retrofitting technique was found to be effective in restoring the flexural strength and ductility capacity of previously damaged columns.

Chai et al (1992) investigated the effects of steel jacketing on shear and flexural dominant, rectangular and circular bridge columns. Since rectangular shape steel jacketing of column cannot provide uniform hoop tension, elliptical steel jackets were used. The space between the rectangular column and the oval steel jacket was filled with concrete. Unlike the flexure dominant columns, which were retrofitted only in plastic hinge regions, the steel jackets in shear-dominant column were extended to cover the full column length. Both flexure- and shear-dominant columns showed acceptable performance. Stable hysteresis loops were obtained with high drift capacity, indicating high energy absorption capacity.

Chai (1995) also investigated the influence of steel jacketing on structural characteristics of bridge columns. He concluded that compressive strength, curvature ductility capacity, and lateral stiffness of steel-jacketed columns increase with the jacket thickness. Steel jacketing improved the compressive strength of column by restricting the dilation of concrete, and it led to increased curvature ductility capacity. The researcher reported that the ultimate limit state of steel jacketed columns depended on the jacket thickness ratio (thickness of steel jacket to column section dimension). For jacket thickness ratios of 0.0033 to 0.0098, the failure was governed by concrete compression failure, while for larger thickness ratios of 0.0131 to 0.0164, the failure was governed by tension failure of longitudinal steel. Lateral stiffness of steel jacketed columns increased

with thickness and height of steel jacket, and also the bond strength between the jacket and the column. He assessed the performance of steel jacketed bridge columns by performing series of inelastic dynamic analyses, using ground motions from the 1989 Loma Prieta Earthquake. He concluded that the jacket thickness used by the California Transportation Department (Caltrans) was adequate for protecting bridge columns.

Aboutaha (1994) studied the effectiveness of different types of rectangular steel jackets for seismic retrofit of non-ductile reinforced concrete columns. He tested columns with inadequate shear strength or inadequate lap splice length. The columns represented typical building columns designed in the 1950's and 1960's in the United States. All specimens were tested without axial load. He constructed and tested 28 large-scale columns, designed according to the provisions of ACI 318-56 and ACI 318-63. Seventeen flexural columns with inadequate longitudinal steel lap splices, and eleven columns with inadequate shear strength were constructed and tested before or after retrofitting with steel jackets. The splice length, when used, was equal to 24 bar diameters. Flexure-dominant columns were built in four different groups with different shapes and rebar patterns. Shear dominant columns were constructed in three different groups with different transverse rebar patterns, but the same rectangular cross section and height. Eight shear columns were loaded in the weak direction, with an aspect ratio of 2.67, and three columns were loaded in the strong direction, with an aspect ratio of 1.33. Both flexure- and shear-dominant columns were retrofitted with $\frac{1}{4}$ -inch thick rectangular steel jackets with enough height and adequate number of anchor bolts. They showed acceptable behaviour with significant improvements in ductility, strength, and energy dissipation capacity. Simplified analytical models were developed for design and

evaluation of rectangular steel jackets based on the principles of mechanics and the experimental results.

Xiao et al (2003) conducted tests to evaluate the efficiency of partially stiffened rectangular steel jackets. He tried to develop a method to increase confinement and shear strength of older, seismic-deficient columns without substantially changing the column size and shape. Five 1/3 scale reinforced concrete columns were constructed reflecting common deficiencies in pre-1971 codes. The first column was tested as a reference as-built specimen, and the other columns were retrofitted. All four retrofitted columns were provided with 3.2 mm thick rectilinear steel jackets; three of them were also stiffened at potential plastic hinges. Thick steel plates, angles, and square tubes were used as stiffeners. The as-built column showed very poor performance with brittle shear failure at 1.5% drift ratio. The axial load capacity of column diminished after the shear failure of column. Retrofitted column with plain rectilinear steel jacket had improved behaviour in comparison with as-built column; but it did not provide sufficient confinement. Steel jacket alone increased the shear capacity but it could not provide adequate flexural capacity and ductility. On the other hand, columns retrofitted with partially stiffened steel jackets had excellent seismic behaviour. Partially stiffened steel jackets in potential plastic hinge regions substantially increased shear capacity and ductility of rectangular concrete columns.

Y-F Li et al. (2005) proposed a model for concrete confined by a steel jacket and lateral steel reinforcement. They tested 60 reinforced concrete cylinders encased in steel jackets to obtain constitutive parameters required for modeling the stress-strain curve of confined concrete. Since the confined concrete behavior is depending on

unconfined concrete strength, the concrete cylinders were constructed in two different categories according to their compressive strength. Group A represented low strength concretes with compressive strength of 7.06 MPa; and Group B represented higher strength concretes with compressive strength of 19.91 MPa. Each group consisted of concrete cylinders with no lateral reinforcement, circular steel reinforcement, spiral steel reinforcement, and spiral steel wire cable. Steel jackets with 2 and 5 mm thicknesses were placed on the cylinders in each group. Cylinder tests showed that the steel jacket could provide efficient concrete confinement. It was also concluded that the confined strength of concrete was highly affected by the type of internal transverse reinforcement. Spiral steel reinforcement showed the highest effectiveness, followed by circular steel hoops, and spiral steel wire. A systematic method was recommended by the authors to find constitutive parameters needed for modeling the concrete confined by steel reinforcements and steel jackets. The stress-strain curves of proposed model were perfectly matched with the stress-strain curves of tested cylinders.

1.4.2 Reinforced Concrete Jacketing

John Bett et al. (1987) conducted experimental research to evaluate repair and retrofitting of short concrete columns by reinforced concrete jackets. Three two-third-scale identical reinforced concrete columns were constructed based on the design provisions of ACI 318-63 to represent common detailing deficiencies before 1970s in the U.S. Column 1-1 was tested as a reference column representing as-built columns in practice. The column was then repaired by RC jacketing, and retested as Column 1-1R. Two other columns, Column 1-2 and Column 1-3 were each retrofitted with RC jackets with different details and then tested. Tests were conducted under constant axial load and

lateral displacements applied at the top of columns. All three specimens in the test program were strengthened by shotcrete jackets. Jacket reinforcement for Column 1-2 consisted of US No. 3 longitudinal bars (9.5 mm diameter) at the corners, and special 6 mm deformed ties with a spacing of 2.5 in (64 mm). Column 1-3 had the same jacket reinforcement as Column 1-2, except for additional US No. 6 longitudinal bars (19 mm diameter) at each midface and US No. 3 crossties (9.5 mm diameter) spaced at 9 in (230 mm). Crossties were placed in pre-drilled holes and fixed with epoxy adhesive. The longitudinal reinforcements were not extended into the foundation, so they did not have any effect on flexural strength of critical end sections. All column surfaces were roughened by light sandblasting before shotcreting. For repairing of Column 1-1, all the loose cover was removed by a chipping hammer and the jacket reinforcement and shotcrete were applied similar to Column 1-3.

Column 1-1 demonstrated poor performance under reversed cyclic lateral deformations, beyond 0.5 % lateral drift. Low aspect ratio and weak detailing were responsible for column brittle failure. Retrofitted and repaired specimens didn't show any strength degradation up to 2.5 % drift. Although the shear capacity of strengthened columns was increased, the flexural bars didn't yield. Extra longitudinal bars at column midface and crossties didn't have any significant effect on column stiffness and strength. Repaired column with reinforced shotcrete jacket had an acceptable performance similar to the undamaged retrofitted columns.

Rodriguez and Park (1994) conducted tests of four reinforced concrete columns to evaluate concrete jacketing as a retrofit method. Concrete columns were constructed based on typical building column design prior to 1970 in New Zealand. All four as-built

columns were constructed similarly with 350 mm square cross section and a column height of 3.3 m. The column units were representative of the mid-height region of a typical column in practice between two stories of a building. Eight evenly distributed plain round bars were used as longitudinal reinforcements in each column. Transverse reinforcements consisted of sets of overlapping square hoops. The amount of transverse reinforcements was less than that required for confinement in the ACI Building Code.

Two as-built columns were first damaged by simulated seismic loading, and then repaired by concrete jackets. Two other undamaged columns were retrofitted by concrete jackets for testing. The surface of concrete columns was roughened before the placement of jackets. Two different concrete jackets were investigated; one with 100 mm thickness and eight longitudinal bars at the corners with square hoops, and the other with 100 mm thickness and twelve longitudinal bars distributed around the perimeter with square and octagonal overlapping hoops. Longitudinal and transverse reinforcements in the concrete jackets were designed according to the New Zealand concrete design code NZS 3101.

Non-retrofitted as-built columns showed poor seismic behavior. Low displacement ductility and inter-story drift ratios were observed. In some regions bond failure was observed between the longitudinal round bars and the surrounding concrete. Retrofitted and repaired columns showed three times the strength and stiffness obtained in as-built columns. Jacketed columns also provided very good energy dissipation and ductility. Results of the simulated seismic tests indicated that imposing previous damage to the as-built column did not make any significant change on the seismic performance of the jacketed column. In addition, providing larger quantity and different patterns of

longitudinal bars in concrete jackets did not have any considerable improvement in the seismic behavior of columns.

Eduardo et al. (2005) studied the effect of interface treatment on structural behaviour of retrofitted concrete columns by reinforced concrete jackets. Seven full-scale concrete columns were constructed and tested to evaluate the influence of surface preparation with sandblasting and application of steel connectors. Original concrete columns had 200mm square cross section and 1350mm height. Concrete columns were reinforced with three 10mm diameter longitudinal bars at each face, and 6mm ties spaced at 150 mm. All the columns were tested under monotonic horizontal forces applied by a hydraulic jack, and an axial load of 170 kN applied through a set of prestressing tendons. The first model (M1) was tested without a jacket to serve as a reference as-built column. A thin, hard, and greased layer was applied to the interface of second column (M2) to create a non-adherent RC jacket. Third column (M3) and its jacket were cast monolithically. Column M2 and M3 were designed to represent the lower and upper limits of expected structural behaviour in jacketed columns, respectively. Fourth column (M4) was jacketed with no surface preparation. Interface surface of the fifth column (M5) was prepared by sandblasting. Sixth column (M6) was jacketed after surface preparation with sandblasting and application of steel connectors. For the last column (M7), the axial load was applied prior to the application of RC jacket on sandblasted surface. All reinforced concrete jackets were cast by a commercial grout with characteristics of SCC and HSC. Concrete jackets were reinforced with three 10mm diameter longitudinal bars on each side of the column, and 6 mm ties spaced at 75 mm, out of phase with the original column stirrups. All the longitudinal bars of the jacket were anchored to the

foundation by placing them in predrilled holes and filling the holes with commercial epoxy resin. Test results indicated that the jacketed models were considerably stronger than the original column. Resistance of model M6 was slightly higher than the monolithic model, and this was attributed to the higher strength concrete that was used in the concrete jacket compare to the regular concrete that was used in monolithic columns. The stiffness of the jacketed columns was also much higher than the original column. Different interface preparation methods did not show considerable influence on the column behaviour. Except for Model M2, all retrofitted columns behaved monolithically. Structural response of Column M7 indicated that the application of axial load before or after retrofitting did not have any effect on column behavior.

Konstantinos et al. (2006) investigated different methods of concrete jacketing. Four half-height full-scale ground floor columns were designed according to 1950s Greek Building Codes. Mild steel reinforcement, widely spaced stirrups, and 90-degree hooks at the ends of stirrups were applied to simulate the common deficiencies of old codes. Three columns were retrofitted with three different types of concrete jacket techniques, and one column was tested as an as-built column. They also constructed a monolithic column with the same cross section and detailing of jacketed columns for comparison. For all the retrofitted columns the thickness of the jacket was the same. There was no treatment at the interface between the column and the jacket. The longitudinal and transverse reinforcements were applied similarly in all three jacketed columns. The difference between the retrofit techniques was in the construction details. In the first retrofitted column (denoted as N), jacket stirrups ends were welded together and poured concrete was used. In the second retrofitted column (denoted as E) in addition to the Column N

detailing, some dowels were placed to connect the jacket reinforcement to the original column. In the third retrofitted column (denoted as W) they used bent down steel connector bars to connect the original column longitudinal reinforcement to the jacket reinforcement. Column W was also shotcreted. Test results showed an overall improvement in jacketed column behaviour even without the treatment of the interface. An acceptable increase in strength, stiffness, and dissipated energy were observed in the jacketed columns, compared to the as built column. All the three jacketed columns showed lower stiffness and strength in comparison with the monolithic column. Nevertheless, column W with special bent down steel connector bars and shotcrete concrete had better energy dissipation than the monolithic column.

1.4.3 Fiber Composites

The use of FRP jacketing was investigated by many researchers in the past. Saadatmanesh et al. (1994) performed analytical studies on circular and rectangular concrete columns strengthened with composite straps under monotonic loading to evaluate the increase in strength and ductility. A parametric study was also conducted to assess the influence of concrete compressive strength, thickness and spacing of straps, as well as the type of straps used. Carbon fiber strap and E-glass strap were the two types of resin-impregnated unidirectional composite straps that were investigated in the study. For analysis of circular and rectangular columns confined by composite straps, they used the stress-strain model for confined concrete developed by Mander, Priestley, and Park (1988). The results indicated that compressive strength and strain of retrofitted columns increased dramatically when compare with non-retrofitted columns. Based on the area under stress stress-strain diagrams, carbon fiber showed a higher energy-absorption

compare to E-glass. Consequently, it was concluded that a column retrofitted with carbon fibres had higher ultimate axial load and ductility when compared with those retrofitted with glass fibres. The rate of increase in ductility for strengthened columns depended on the strap thickness and spacing. It increased with increasing strap thickness, and decreased with increasing strap spacing. External strengthening of columns with fibres did not have a considerable improvement on maximum moment capacity. This, however, produced ductile flexural failure, which is desirable for seismic retrofitting. The increase in concrete compressive strength resulted in decreased rate of increase in ultimate axial load, ductility, and maximum moment capacity.

Saadatmanesh et al. (1996) tested five 1/5 scale concrete column-footing specimens to evaluate the seismic strengthening of circular bridge piers with fiber reinforced plastic composite straps. Circular reinforced concrete columns were designed according to pre-1970 bridge design codes for high seismic zones. The diameter of the columns was 305 mm and 14 No. 4 longitudinal bars were used in each column. Steel wire hoops, with a diameter of 3.5 mm, were spaced at 89 mm as transverse reinforcement. Lap-spliced longitudinal bars with a lap length of 20 bar diameter were used in specimens C-1, C-2, and C-3. For specimens C-4, and C-5, the longitudinal bars extended into the foundation with a 90-degree hook at the end. Specimens C-1 and C-4 were representative of as-built columns with no retrofitting. All retrofitted columns were wrapped with six layers of FRP composite straps from the top of the foundation to a height of 635 mm, which was considered as the plastic hinge region of the column. A passive retrofit scheme was applied in specimens C-2 and C-5, while an active retrofit scheme was used in specimen C-3. The combination of axial load and lateral load were

applied to simulate seismic loading. Axial load of 445 kN was applied through two high-strength steel rods on each side of the column. The lateral load was applied by a MTS ± 489 KN hydraulic actuator. Test results indicated brittle failure of the non-retrofitted lap-spliced column. The use of continuous reinforcement in specimen C-4 slightly improved the seismic behaviour. On the other hand, FRP retrofitted columns showed acceptable improvement in ductility and strength. Both active and passive jacketing methods significantly increased the confinement of core concrete, and consequently prevented bond failure and longitudinal bar buckling. Active method did not have enough advantage over passive method to justify its additional cost.

Saadatmanesh et al. (1997) investigated the effectiveness of FRP composite wrapping by repairing seismically damaged columns. Two circular (C-1 and C-2) and two rectangular (R-1 and R-2) columns were laterally loaded to the level of failure, and then repaired with FRP composite wraps and pressurized epoxy. All concrete columns were 1/5 scale bridge columns, which were designed based on pre-1970 design codes. Column C-1 and R-1 were longitudinally reinforced with lap-spliced bars having lap length of 20-bar diameter. While, for column C-2 and R-2, longitudinal reinforcement was extended into the foundation, largely spaced 9-gage steel wires were used as transverse reinforcement.

As the first step of repairing, the damaged columns were pushed back to their initial position. Then all the loose concrete was removed and the gaps were filled by fresh concrete. Active retrofit scheme was applied by placing slightly oversized FRP straps around the columns in their potential plastic hinge regions. The space between the concrete and the strap was filled with pressurized epoxy. The number of FRP straps was

selected according to the required confinement for column design by current design codes. The test results indicated that the repair of seismically damaged columns with FRP composite wraps could effectively restore the flexural strength and ductility capacity of columns. All repaired columns had lower stiffness deterioration as compare to the original columns.

Seible et al. (1995) developed a design guideline for seismic retrofit of bridge columns for continuous carbon fiber jackets. Three different design methods were recommended based on three possible types of failure modes for concrete columns. The failure modes included; shear failure, confinement failure of flexural plastic hinge region, and lap splice failure at lower column end. Two separate design criteria were developed for shear retrofit of rectangular and circular columns with carbon fiber jackets. Column dilation was limited to 0.004 to control the aggregate interlock and crack width in concrete. This value is well below the ultimate strain limit of 0.01 for carbon jacket, and higher than the yield strain of transverse reinforcement which allow for the contribution of reinforcement to shear resistance. Design equations for confinement of flexural plastic regions were derived for circular columns. It was recommended that rectangular jackets with a side aspect ratio of 1.5 or less should be retrofitted with a jacket thickness of twice that required thickness for equivalent circular columns. It was also recommended that the corners of rectangular column should be rounded before the installation of jackets. For rectangular columns with side aspect ratios of higher than 1.5, oval or circular jackets were recommended on precast concrete bolsters. It was concluded that rectangular jackets could not provide enough confinement to prevent debonding of the reinforcement lap

splices. In this case, jackets with convex curvature were recommended to provide continuous lateral clamping pressure.

Seible et al. (1997) tested three 40% scale concrete bridge columns to evaluate previously developed methods for seismic retrofit of concrete columns with continuous carbon fiber jackets. Three concrete columns with rectangular and circular cross sections were constructed at the Powell Structural Research Laboratory at the University of San Diego. All columns were designed based on pre 1971 seismic design codes to represent common deficiencies of old columns. Transverse reinforcement was 6 mm diameter with 127 mm spacing, for all three columns. Specimen one was a rectangular shear dominant column bending in double curvature. Specimen two was a flexure-dominant rectangular column bending in single curvature and specimen three was a circular flexural cantilever column. Continuous carbon/epoxy tows were applied on the columns through an automated system. Based on the shear strength and flexural confinement requirements of specimens, the height of the column was divided into different sections, and carbon fiber jackets with different thickness were applied on each section. All the retrofitted columns showed higher deformation capacities, similar to the columns previously retrofitted with steel jackets. Test results indicated lower stiffness and capacity increases in columns relative to those with steel jacket retrofits. This was indicated to be a desirable seismic behavior that would lead to reduced earthquake inertia forces.

Xiao et al. (1999) evaluated prefabricated composite jackets for seismic shear resistance of RC columns. Three identical half-scale circular concrete columns were constructed with a shear aspect ratio of 1.5 in double curvature. All the columns were reinforced longitudinally with 2% reinforcement. Circular hoops consisting of 6.4 mm

diameter smooth reinforcement were placed at every 150 mm. Grade 300 steel bars and concrete with target compressive strength of 34.5 MPa were used to represent common practice in bridge structures built in the 1960s. First column was tested without any retrofit as an as-built column. The other two columns were jacketed by multiple layers of prefabricated composite shells. The second column was retrofitted with four individual single layer shells with a longitudinal slit for each layer. Third column was retrofitted by three layers of continuous shells. Prefabricated shells were cured and cut in a quality-controlled environment before they were applied on the columns. The workability of the method was also investigated through a field construction test. The retrofitting process consisted of the following steps:

1. Surface cleaning of the columns.
2. Spraying thixotropic urethane adhesive.
3. Clamping a composite shell around the column.
4. Repeating steps 2 and 3 for required number of layers.
5. Fastening steel straps around the layers to fix them in their place.
6. Repeating steps 2 to 5 for the required jacket height.
7. Removing the straps after 1 to 2 days of curing, and then applying surface coating.

The test setup consisted of high-strength steel rods to apply constant axial load and a double acting actuator to apply cyclic horizontal forces through an L-shaped loading arm. Test results indicated brittle failure with low ductility for as built columns as predicted by the researchers. Both retrofitting methods have effectively improved the seismic behaviour. Prefabricated composite jackets successfully changed the brittle shear failure

of the columns to ductile flexural failure, with high-energy dissipation characteristics. Individual shell jacket and continuous shell jacket did not show a significant difference in behaviour.

Pantelides et al. (1999) conducted in situ tests of two identical bridge bents to measure the lateral load and ductility capacity of existing bridges, while also evaluating the effectiveness of CFRP composite jackets for retrofitting cap beam-column joints and columns. Each bridge bent consisted of three rectangular concrete columns and a concrete cap beam with girders and a deck on top. The bridge bents were constructed in the 1960s which were just designed for wind loads and not for earthquakes. Lack of insufficient transverse reinforcement in plastic hinge regions, as well as in lap splice regions, and inadequate anchorage of longitudinal bars in the pile cap and cap beam-column joint were some of the seismic deficiencies of the bridge bents. No transverse hoops were used in the column cap-beam joint. The first bridge bent was tested as an as-built specimen and the second one was retrofitted with a CFRP composite. The retrofitting was designed based on the method proposed by Seible et al. (1997) to double the ductility of the columns. The evaluation of shear force in the joints and also the retrofit design of joints were carried out based on the recommendations made by Gergely et al. (1998). Simulated seismic loads were applied as horizontal forces on the cap beam by a hydraulic actuator and a reaction steel frame. The axial load for each bent was half of the original dead load of the deck, which was transferred to the cap beam by means of welded plate girders, bearing on eight reinforced concrete pedestals. The non-retrofitted bent showed an undesirable seismic behaviour with a displacement ductility of 2.8. On the other hand, externally bonded CFRP sheets effectively controlled the overall damage

of the bent, resulting in a displacement ductility of 6.3. Delamination of CFRP composite was properly postponed by the using high-strength adhesive in the retrofitted bent.

1.4.4 External Prestressing (Retrobelt)

A series of experimental investigations have been conducted at University of Ottawa under supervision of Dr. Murat Saatcioglu in order to develop a new seismic retrofit method involving external prestressing of columns in transverse direction. Series of tests were conducted since 1995 to expand the method to different types of columns with different cross-sectional geometry and mode of behaviour. Yalcin (1997) was the first to test seven full-scale concrete bridge columns with shear deficiencies in the first phase of the overall research program. Post tensioned external hoops, consisting of 7-wire prestressing strands were used to apply active lateral pressure on column in one column the prestressing was applied by means of high-strength steel straps to provide shear strength and confinement for the columns. A comprehensive survey of the Canadian highway bridges was also conducted to categorize bridges based on their types. The computer software COLA and DRAIN-RC were developed to calculate the drift capacity and drift demand of reinforced concrete columns. The test specimens consisted of five circular and two square columns, all representing on pre-1970s design practices with common deficiencies of that era. Internal transverse reinforcement was widely spaced (305 mm on centres). Each specimen consisted of a concrete foundation with a half height reinforced concrete column extending to the point of inflection of column. Two of the columns, one circular and one square, were tested as reference columns with no retrofitting. The other columns were retrofitted with seven-wire strands, stressed to different levels of initial prestressing at different hoop spacings. For circular columns the

strands were placed directly on the surface of the column. For square columns, steel spreader frames and raiser discs were used to shape the strands to develop hoop tension and provide uniform confinement pressure rather than generating stress concentration at the corners. One of the circular columns was also retrofitted by steel packaging straps, used for prestressing, fastened by manual anchoring device. The test setup consisted of three computer-servo-controlled MTS hydraulic actuators with a 1,000 kN capacity. Two actuators were placed vertically one on each side of the column to apply constant axial load through a steel loading beam assembly. The lateral force was also applied by a horizontally positioned actuator in displacement-controlled mode, incrementally increasing displacement reversals. The as-built columns had shear failure starting at 1% lateral drift ratio. They could not survive 2% lateral drift, and developed brittle failure, which was initiated with diagonal cracks, followed by buckled longitudinal bars. Test results of retrofitted columns showed acceptable behaviour. Circular and square columns with strands placed at 150 mm spacing and transversely prestressed to 300 MPa showed ductile response and failed at 5% lateral drift. Other retrofitted columns with higher spacing or lower prestressing provided less improvements in drift capacity, while also providing reasonably ductile behaviour. The tests showed the importance of the level of prestressing and spacing as the main design parameters. The steel packaging straps used in one of the circular columns showed only 2% lateral drift as the straps did not have sufficiently high capacity and failed prematurely under laterally expanding concrete.

Mess (1999) tested six full-scale half height concrete columns, in Phase II of the above research program to expand the retrofit methodology to flexural dominant columns with confinement deficiencies. Three pairs of concrete columns were designed based on

pre-1970s design codes. The first pair of concrete columns had a circular cross section with 508 mm diameter and a 2.0 m shear span. The second pair of columns had a 500 mm square cross-section with a 2.0 m shear span. The third pair of concrete columns had a 610 mm circular cross-section and a 1.55 m shear span, with spiral transverse reinforcement. In each column pair, one column was tested as reference column with no retrofitting, and the other was retrofitted by external prestressing. The required confinement was provided by seven-wire strands placed with 150 mm spacing. The strands were prestressed up to 25% of the strand capacity. Three raiser discs with different diameters were attached to the HSS steel spreader sections placed on each face of the square column to apply well distributed lateral pressure. The test setup used in the experimental program was similar to that used earlier by Yalcin (1997). Test results indicated that non-retrofitted flexural dominant concrete columns could develop up to 2% later drift. The retrofitted columns demonstrated ductile behaviour with improved lateral drift ratio capacity of 4% to 5%.

In Phase III of the same research program, Pascal (2000) tested three pairs of full-scale half height concrete bridge columns to study the effectiveness of external prestressing method on controlling lap splice slippage in bridge columns with splice deficiencies. All the columns were designed and constructed based on pre-1970s practice. The test columns had spliced longitudinal bars in the potential plastic hinge region, with inadequate splice length (equal to 20 times the bar diameter). This resulted in a total lap length of 390 mm. The columns were prepared in pairs with a column in each pair representing as-built conditions and the other column build for retrofitting by external prestressing. The first pair was flexure-dominant circular columns with a 508 mm cross-

sectional size and a 2.0 m shear span. The second pair was flexure-dominant square column with a 500 mm cross-sectional dimension and a 2.0 meter shear span. The third pair consisted of shear-dominant circular columns with a 610 mm diameter and a 1.5 m shear span. The flexure-dominant circular column was retrofitted with 9 mm diameter 7-wire strands, spaced at 150 mm over the entire column height. The strands were prestressed up to 25% of the ultimate strand capacity. The flexure dominant square column was retrofitted with closely spaced strands within the splice region. Accordingly, the strands were placed with a 100 mm spacing, each prestressed to 50% of the strand capacity. The spacing and prestressing of the strands outside the splicing region were 150 mm and 25% of ultimate, respectively. The raiser assembly that was used to provide well-distributed confinement pressure in the square column was similar to that used by Mess (1999). The shear dominant circular column was also retrofitted by strands that were spaced at every 100 mm with a level of prestressing reaching up to 50% of ultimate within the splice region. The spacing and prestressing level of strands outside the splice region were 150 mm and 25% of ultimate, respectively. The test results indicated that the spacing and the level of prestressing of strands used in the first pair was not sufficient to improve the column failure mode substantially. Both retrofitted and reference columns failed due to the slippage of spliced reinforcement, with retrofitting increasing the lateral drift capacity from 1% to 2%. This was judged to be insufficient for confinement and displacement ductility. On the other hand, closely spaced and highly prestressed strands in square columns changed the mode of behaviour from a brittle failure with 1% drift capacity to a ductile failure with 4% drift capacity. The retrofitted shear dominant circular column also demonstrated ductile behaviour.

Phase IV of the same investigation involved the extension of the retrofit methodology to rectangular columns. Yarandi (2007) tested three pairs of rectangular concrete columns to investigate the effectiveness of external prestressing in rectangular bridge columns. Each column pair consisted of an as-built column and a retrofitted column. Three additional shear dominant concrete columns were tested to explore the possibility of using the same retrofit technology as a quick seismic repair methodology. The columns, once again, were designed following the old design provisions with common seismic deficiencies, reflecting pre-1970s practice. The first pair consisted of shear dominant rectangular columns with a 350×700 cross-section and a 1.5 m shear span. The second pair consisted flexural dominant rectangular columns with a 350×700 cross-section and a 2.0 m shear span. The third pair consisted of flexural dominant rectangular columns with same dimensions as those of the second pair, but having lap-spliced longitudinal bars. The lap splice length was 20 times the bar diameter, located in the potential plastic hinge region, and hence was seismically deficient. The shear dominant column was retrofitted with prestressing strands with 150 mm spacing and 25% of ultimate strand strength as the level of prestressing. The same retrofit was also applied to the flexure-dominant column. The lap-splice deficient column had reduced strand spacing of 100 mm and increased level of initial prestressing of 50% of the strand rupturing strength within the splice region. The spacing of strands was increased to 150 mm and the level of prestressing was reduced to 25% of the strand capacity outside the splice zone. All three retrofitted columns showed very ductile behaviour with high-energy dissipation. The retrofit strategy changed the mode of failure from a brittle shear

failure to a ductile flexure failure, even in shear-dominant columns. The proposed method has significantly improved the seismic behavior of the flexural dominant columns.

The research program was extended to develop a quick seismic repair methodology through external transverse prestressing. Three shear-dominant columns with a circular, a square, and a rectangular cross section were constructed for damaging and then repairing. All three columns had an effective length of 2.0 m. The rectangular column had spliced longitudinal bars with insufficient lap length within 390 mm from the column base. This splice length corresponded to 20 times the longitudinal bar diameter. The circular column was tested up to 1.5% lateral drift, but the test could not be stopped prior to a complete, brittle failure. Hence this column could not be repaired. The square and rectangular columns were tested up to 1% lateral drift, developed wide diagonal cracks and the test was stopped for repair. The repairing was implemented through external transverse prestressing. The results indicated that the shear failure was suppressed and the columns developed ductile behavior with high energy dissipation capacity.

1.4.5 Conclusion from literature review

The following conclusions can be drawn from the literature review:

- Performance of buildings and bridges during past earthquakes has indicated that many of these structures are vulnerable to seismic damage and structural collapse. The deficiencies in pre-1970s design codes have resulted in poor performance of reinforced concrete structures during seismic excitations.
- Most seismically vulnerable concrete columns lack either sufficient shear capacity or confinement. This problem is attributed to inadequate transverse reinforcement.

- The majority of deficiencies of vulnerable concrete columns can be overcome through seismic retrofits that involve additional transverse reinforcement. This can be done either by providing reinforced concrete, steel, or fibre-reinforced polymer (FRP) jackets around existing columns; or by applying transverse prestressing to columns (RetroBelt System).
- Among the retrofitting techniques considered for concrete columns, transverse prestressing using 7-wire strands (RetroBelt System) satisfies all the requirements. Transverse prestressing increases the shear capacity of columns and provides enough confinement in the plastic hinge region. It is the most practical and economical method of retrofitting for concrete columns.
- While it is relatively easy to implement external prestressing using 7-wire strands in large size bridge columns, it creates challenges in smaller size building columns due to its bending limit.
- Retrofitting of concrete columns with high strength packaging straps can be an alternative technique to use of 7-wire strands since it is more flexible material and it can be used for small diameter columns as well as large one.

Chapter 2: Experimental Program

2.1 Introduction

Experimental phase of the research project was conducted at Structural Laboratory of the University of Ottawa. The primary aim was to investigate the effectiveness of using high-strength packaging straps as external prestressing elements in retrofitting seismically deficient columns. This required tests of large-scale reinforced concrete columns under simulated seismic loading. The test program consisted of two flexural-dominant building columns and one shear-dominant bridge column, designed following the pre-1970's design and construction practices. The columns represented common seismic deficiencies associated with lack of sufficient transverse reinforcement. Design and construction of column specimens, as well as the column retrofit methodology investigated in the research project, including material properties, test setup, and instrumentation is presented in this chapter. Illustrative pictures and sketches on test set-up and specimen preparation are provided at the end of the chapter.

2.2 Column Details

Three full-scale concrete columns were designed based on pre-1970's building codes, representing common seismic deficiencies of that era. Two flexure-dominant building columns with circular and rectangular cross sections were designed to have insufficient confinement in the plastic hinge region. A shear dominant bridge column was also designed to have less shear capacity than its shear demand. All three column specimens were half-height cantilever columns representative of the segment between

column footing and the point of inflection. Seismic forces were applied at the top, and the shear span was equal to the specimen height.

2.2.1 Flexure-Dominant Circular Column (F-CR)

A circular building column with a 350 mm cross-sectional dimension and a 1345 mm cantilever height was designed and constructed for testing. The effective height of the column, measured from the column footing to the point of application of the force (shear span) was 1630 mm. The shear span included the height of the column plus the mid-height of the loading beam, which had a total height of 285 mm. The aspect ratio (ratio of shear span to column diameter) was 4.6, representative of flexural-dominant columns. Six No. 25 longitudinal bars were distributed uniformly along the section perimeter. No. 10 circular ties with overlapping ends were located at every 175 mm, starting from 50 mm above the footing. The clear concrete cover was 27 mm. The column had a reinforced concrete footing having the dimensions of 1400×1735×520mm. The footing was designed to be connected to the strong floor of the Structural Laboratory to provide full fixity. Figure 2.1 illustrates the column geometry and the reinforcement details.

2.2.2 Flexure-Dominant Rectangular Column (F-RC)

A rectangular building column with cross-sectional dimensions of 400×200 mm was designed and constructed with a cantilever column height of 1595 mm. The specimen represented the segment of a first-storey column between the footing and the point of inflection. The height between the surface of the footing and the point of application of the force (shear span) was equal to 1880 mm. This translates into an aspect ratio of 4.7, to develop primarily flexure-dominant behaviour. Six No. 25 longitudinal

bars were distributed uniformly along the section perimeter. No. 10 rectangular ties with 90 degree bents were placed as transverse reinforcement at every 175 mm, starting from 50 mm above the footing. Clear concrete cover was 35 mm. The footing was similar to that built for the circular column described in the previous section. The column geometry and reinforcement details are illustrated in Figure 2.2.

2.2.3 Shear-Dominant Square Column (S-SQ)

A bridge column, with a 550×550 mm square cross section was designed and built for testing. The column height was 1200 mm. The shear span was 1485 mm, measured to the point of application of the horizontal force, resulting in an aspect ratio of 2.7. This small aspect ratio increased the influence of shear during response. Twelve No. 25 longitudinal bars were distributed uniformly along the section perimeter. No. 10 square ties with 90 degree bents, were located at every 275 mm, starting from 50 mm above the footing. The clear concrete cover was 45 mm. A 1400×2080×520 mm rectangular footing was designed and built. The footing was secured to the laboratory strong floor to ensure full fixity of the column. The column geometry and reinforcement details are illustrated in Figure 2.3.

2.3 Preparation of Column Specimens

2.3.1 Footings

The footing for each column was built first. The construction of footings consisted of the preparation of formwork and assembly of pre-ordered reinforcing bars. The formwork was built from plywood and lumber. Plastic pipes were fixed inside each formwork to provide twelve holes inside the concrete after casting. Four large holes were used to anchor the footing to the strong floor, and four small holes on each side of the

column were used to fix two vertical actuators to the footing. These actuators were used to apply a constant axial load, under which the columns were tested. The footing reinforcement cage consisted of No. 25 pre-bent bars. The footings were over-designed to ensure that they remain essentially elastic while the columns were subjected to lateral deformation reversals. The footings were cast after the placement of reinforcement cages into the formwork. The construction process of formworks and reinforcement cages is illustrated in Figure 2.4-2.6.

2.3.2 Columns

Column cage was built by assembling the longitudinal and transverse reinforcement. Strain gauges were first installed on the bars before the cage was assembled. The column formwork for square and rectangular columns were constructed using plywood and lumber. Sonotube was used for the circular column. Once assembled, the column cages were secured in the column formwork. High-strength steel packaging straps were placed around the formworks to stiffened the formwork during casting. Ready mix concrete was used to cast the columns. Steel bolts were cast in concrete at the top of each column to be used at loading stage. All three columns were casted from a same concrete batch, and they were cured for one week. The construction process of concrete columns is illustrated in Figure 2.7-2.8.

2.4 Retrofitting Phase

The retrofit methodology is the extension of previously developed RetroBelt system of column retrofitting, involving external transverse prestressing of columns to induce active lateral pressure for concrete confinement and improved bond for spliced reinforcement, while controlling diagonal tension cracks caused by shear forces and

providing additional shear reinforcement. The prestressing in the RetroBelt system is done by means of 7-wire strands and a specially designed ring anchors that connect the two ends of the prestressed strands, thereby forming a tensioned hoop. In the current application, the prestressing is done by means of packaging straps having a rectangular cross-section.

2.4.1 Retrofit Materials and the Installation of Packaging Straps

Protective packaging system provided by Signode Company was used for transverse prestressing of columns. This was done by forming external column hoops, consisting of prestressed high-strength steel packaging straps, sealed (connected) at their ends. Magnus steel strapping was used as the strap material. Magnus is cold-reduced medium-carbon high-manganese steel strapping which is heat-treated. This product has higher ductility and shock resistance when compare with cold-rolled straps. The cross-sectional dimensions of the specific strap employed in the current investigation had 19 mm width and 0.74 mm thickness, with a nominal cross-sectional area of 14 mm². Steel straps were prestressed by means of a pneumatic tensioner, and then fixed (connected) at the ends in their stressed state by means of a pneumatic sealer. Push-type seals were used for this purpose.

A pneumatic tensioner and a pneumatic sealer, provided by Signode Company, were used for prestressing the straps around the columns. Figure 2.9-2.11 illustrates the equipment that was used during the prestressing process, including the pneumatic tensioner and sealer, individual seals, and the high-strength packaging straps. The prestressing process is shown in Figure 2.12. The amount of prestressing provided by the pneumatic tensioner was not adjustable, and it was relatively constant. The average level

of prestressing in each strap was approximately 250 MPa, which was calculated based on the measured strains on straps during and after the prestressing operation. The strap spacing and the level of prestressing force on concrete, which was dictated by the number of strap hoops placed at a given location, were based on the retrofit design and the desired level of active pressure for concrete confinement, splice clamping or diagonal crack control. For the columns retrofitted in the current experimental program, two layers of straps were used (one on top of the other) within the plastic hinge region of circular and rectangular columns. Figure 2.13-2.15 illustrates the strap spacing and the number of layers of straps used in each column.

The packaging straps were directly applied around the circular column. The direct application of straps around rectangular and square columns could produce undesirable stress concentrations at the corners, while loosing their effectiveness as confinement reinforcement. Therefore, precast concrete raiser units were manufactured as filler materials around square and rectangular columns to complete them to circular and elliptical shapes, respectively to generate hoop tension in the straps. The details of the prefabricated raiser units are presented in the next section.

2.4.2 Concrete Raiser Units

In order to generate uniform hoop tension in the straps and to provide uniform confinement pressure around square and rectangular columns, it's desirable to complete the column cross section to an ellipse or a circle. In this research program it was decided to use prefabricated concrete pieces, placed on each face of the column as filler materials to generate the desired shape. This was done as a convenient way of developing hoop

tension, as an alternative to steel raiser frames and circular discs employed in earlier phases of the research program for the RetroBelt system.

Concrete raiser units were manufactured as semicircular arc shaped concrete pieces to be attached on each face of square and rectangular columns. These elements were manufactured in smaller pieces for ease in handling, rather than attempting cover each column face with one unit. Twelve concrete raiser units (four pieces in three rows) were cast for each of the square and rectangular columns. The individual units were placed on the columns with some gap between them along the column height to allow restraint free bending of the columns. A gap of 25 mm was left between the foundation and the first row of concrete raiser units to prevent any contact between the units and the foundation on the compression side. The same spacing of 25 mm was also provided between each row of concrete raiser units along the column height for the same reason. The major benefit of using individual prefabricated concrete raiser units is its flexibility. They can be cast and cured at precast plants and brought to the job site for installation. This technique requires less labour than the other methods used in earlier phases of research, and permits easy and quick assembly.

For the purpose of the current research program, the construction of concrete raiser units was done using the as-built columns as part of the formwork. This would also permit perfect matching of pieces during the prestressing stage. Plastic sheets were placed around the columns to prevent bonding between the columns and concrete units. Two 32 inch (813 mm) and 18 inch (457 mm) sonotubes were used to shape the outside face of the units for square and rectangular columns, respectively. This is shown in Figure 2.16. Sonotubes were cut into short pieces and then secured by wood stud frames.

For the rectangular column, the sonotube pieces were forced to fit on the column, thereby forcing them to assume the required elliptical shapes. Three segmental formwork were constructed for each column to cast concrete in three stages. Plywood plates were used to separate the formwork at three different elevations. This is depicted in Figure 2.17.

Three batches of concrete were mixed using the concrete mixer in the laboratory. Steel fibres, with 0.75% fibre content was used in the mix to control cracking in concrete. Concrete raiser units were cast in three layers from bottom to top as illustrated in Figure 2.18-2.22.

After the concrete pieces were cured and removed, they were placed back on column surface with a very thin layer of cement paste between the concrete raiser units and column concrete. The units were held in place by temporary prestressing of a few steel straps until the mortar dried and provided a perfectly matching surface. After one day, the permanent prestressing straps were applied. Placing the mortar between the concrete raiser units and columns eliminated prestress loss that would develop from sequential prestressing of straps. This process is illustrated in Figure 2.23-2.25.

2.5 Material Properties

Properties of materials used in the experimental program were established from standard tests. Standard concrete cylinder tests and reinforcement coupon tests were conducted to experimentally establish their stress-strain relationships, as discussed in subsequent sections.

2.5.1 Concrete

Different batches of concrete were used for footings, columns, and concrete raiser units. The compressive capacity of concrete for footings and columns were ordered to be

50 Mpa and 30 Mpa respectively. Nine standard concrete cylinders were casted from each batch of concrete to measure the average strength of concrete on the day of testing, for each column. Figure 2.26 shows the concrete compression machine that was used to conduct the tests. For the circular column the average concrete capacity for the column was 36 MPa on the day of testing. The footing of the same column had 60 MPa concrete. For the rectangular column the average concrete capacity of the column was 39 MPa on the day of column testing. The footing of the rectangular column had 71 MPa concrete. The concrete used to produce the raiser units had 24 MPa strength. The average concrete capacity of the square column on the day of testing was 42 MPa, with the strength of concrete in the footing measured as 82 MPa. The raiser units had 24 MPa concrete strength.

2.5.2 Reinforcing Steel

Coupon tests were performed to establish the stress-strain relationship of No. 10M and 25M reinforcement. Galdabini Sun 60 universal testing machine, with 600 kN nominal capacity, was used for coupon testing. Figure 2.27-2.28 shows the stress-strain diagram of No. 10M and 25M reinforcement. The yield stress of No. 10M reinforcement was 415 MPa at a strain of 0.28%, and the ultimate tensile strength was 628 MPa, which occurred at 13.2% strain. For No. 25 reinforcement, the yield stress was 465 MPa at 0.25% strain, and the ultimate tensile strength was 647 MPa at 15.4% strain. As can be seen in Figure 2.27 and Figure 2.28, the yield plateau region for No.10M and 25M were different.

2.5.3 High Strength Steel Straps

Three types of coupon test were performed to establish the stress-strain relationship of high-strength straps used to prestress the columns. Three coupons were tested for each coupon type, and the average of the three was used to establish the stress strain characteristics of the material. The first series of coupon tests were conducted on steel coupons, without the sealers (Coupon Type 1). Each coupon had a steel cross-sectional area of 14 mm². These coupons provided the stress strain relationship of steel alone. Figure 2.31 shows the stress-strain relationship with 813 MPa yield strength at 0.42% strain, followed by a yield plateau up to the ultimate rupturing strength of 973 MPa at 6.69% strain. The second series of coupons consisted of two layers of strap connected with a sealer (Coupon Type 2) as in the case of the straps used outside the column hinging regions. These coupons showed the effect of sealer on strength. It was established that the effect of placing the seal on the strap through crimping of the steel was to reduce the area by approximately 20%. Figure 2.32 shows the stress-strain relationship of Type 2 coupon based on the reduced area, indicating approximately the same strength. The extensometer used to measure the strains was placed outside the yield region, and hence recorded lower strains, as the strap was yielding at the seal location. The third type of coupons had three layers of strap and a sealer (Coupon Type 3) as in the case of double strap hoops used in column hinging regions. Having accounted for the area reduction due to crimping, Figure 2.33 indicates about the same strength level. Once again, in these coupon tests the extensometer was placed outside the yield region, hence showed low strain reading. The results indicate that the use of sealers weakens the straps, as the sealers provide connection through the crimping of steel.

2.6 Test Setup

The test setup consists of three servo-controlled MTS hydraulic actuators; each with 1000 kN capacity in compression and tension. Two actuators were mounted vertically to apply the axial load, and the third one was located horizontally to apply the lateral load reversals. The vertical actuators were secured on either side of the column through the bolts that had been cast in the foundation concrete, and the bolts used to connect the top to the loading beam. The ends of the horizontal actuator were fixed to the supporting steel frame at one end and the loading beam on the column at the other end. The details of the loading beam assembly are shown in Figure 2.34. A concrete spacer block was placed between the loading beam and the loading box to extend the height of the column to allow the placement of the vertical actuators. Two sizes of concrete spacer blocks were used for short and long columns. The column footing was fixed to the laboratory strong floor by means of four large steel bolts. Figure 2.35-2.37 shows the details of the test setup from different views.

2.7 Loading Program

A constant level of axial compression was applied to the columns by two vertical actuators, one on each side of column. The level of axial load for the circular and rectangular building columns was approximately 50% of column concentric capacity P_o . This level of axial load is representative of axial load on typical first-storey columns in multi-storey buildings. The level of axial compression for the square bridge column was approximately 15% P_o , which is typical of axial load on bridge columns.

The lateral load was applied in deformation-control mode as incrementally increasing lateral displacement reversals. Cyclic lateral deformations were based on the drift ratio of column. Three cycles of lateral deformations were first applied at about 0.5% lateral drift ratio, as representatives of elastic load cycles. Lateral displacements were then increased in 1% drift ratio, and continued with 3 cycles at each of the increasing lateral drift ratio (1%, 2%, 3%, 4% etc.) until failure. The failure was defined as lateral load corresponding to 20% strength decay. However, the tests continued beyond failure to observe the column behavior until, and sometimes beyond 50% decay in lateral load resistance. Figure 2.138 shows the lateral load program that was used in column tests.

2.8 Instrumentation

Three types of instruments were used in the test program, as follows:

- Electrical Resistance Strain Gauges.
- Linear Variable Differential Transducers (LVDT),
- Cable Transducers (compact string pot SP2 from Cescor).

The electrical resistance strain gauges were placed on longitudinal reinforcement, transverse reinforcement, and high strength steel straps to measure strain during testing. Figure 2.39-2.44 shows the locations of strain gauges for each column. LVDTs were used to measure flexural rotations of columns. Two LVDT were located on either side of the column at a height of h (h is the cross sectional dimension of the column). They were connected to the column by aluminum handles that were bolted on the column. Figure 2.45 shows the location of LVDTs. Cable transducers were used to measure lateral displacements of columns. They were connected to a steel frame that was fixed on the foundation concrete to measure displacements relative to the footing. Figure 2.45 shows

the locations of cable transducers. All the instruments were connected to two data acquisition systems to record data continuously.

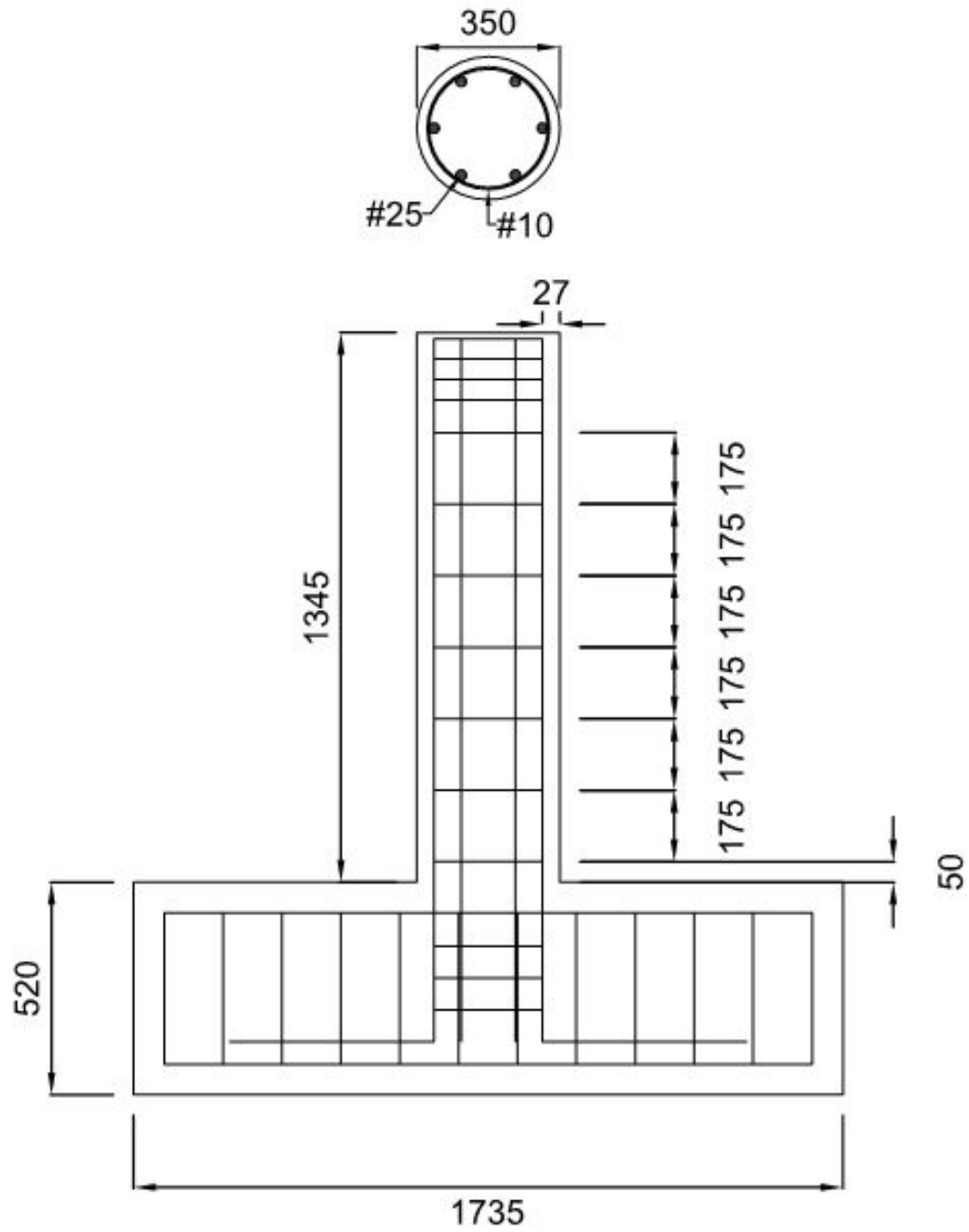


Figure 2-1 Geometric and reinforcement details for circular column F-CR

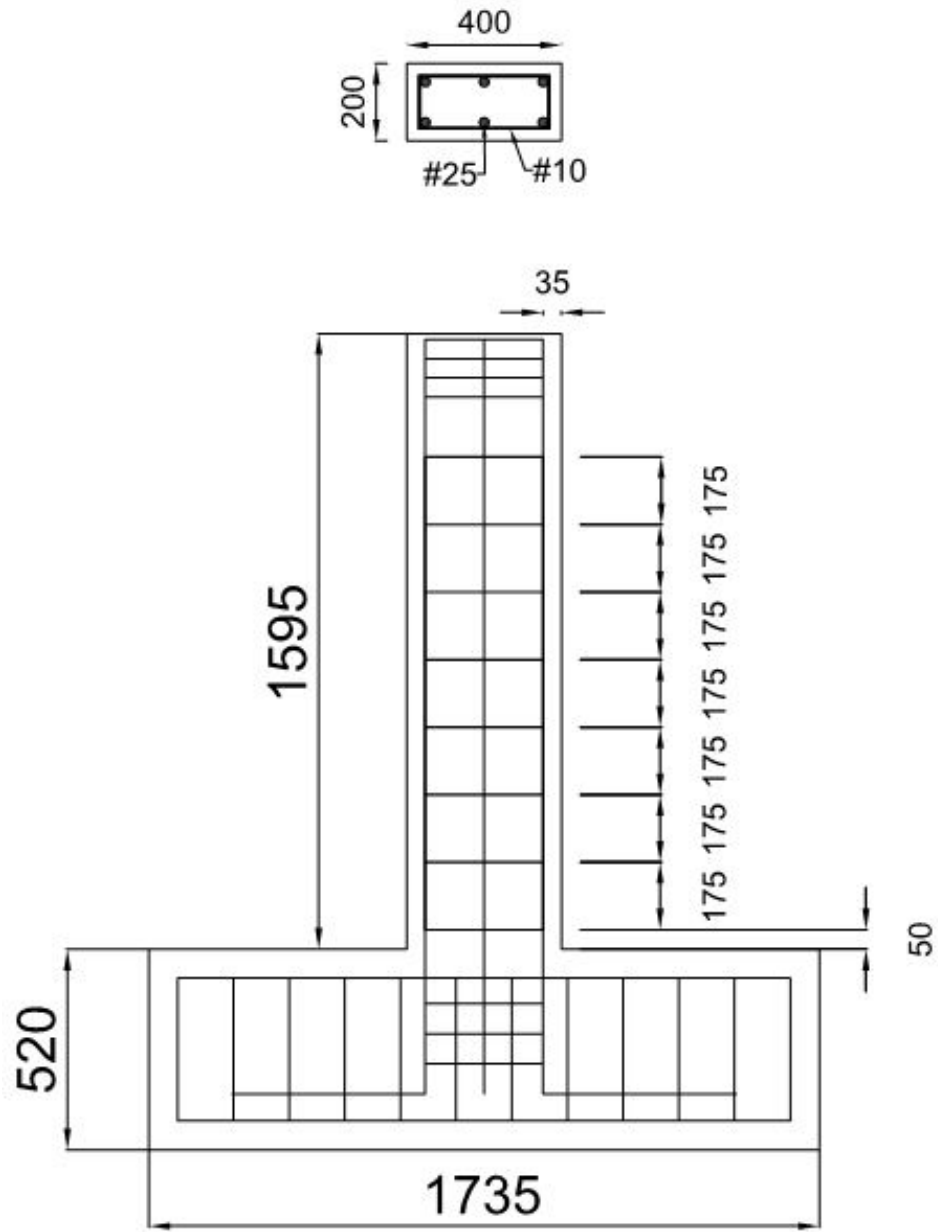


Figure 2-2 Geometric and reinforcement details for rectangular column F-RC

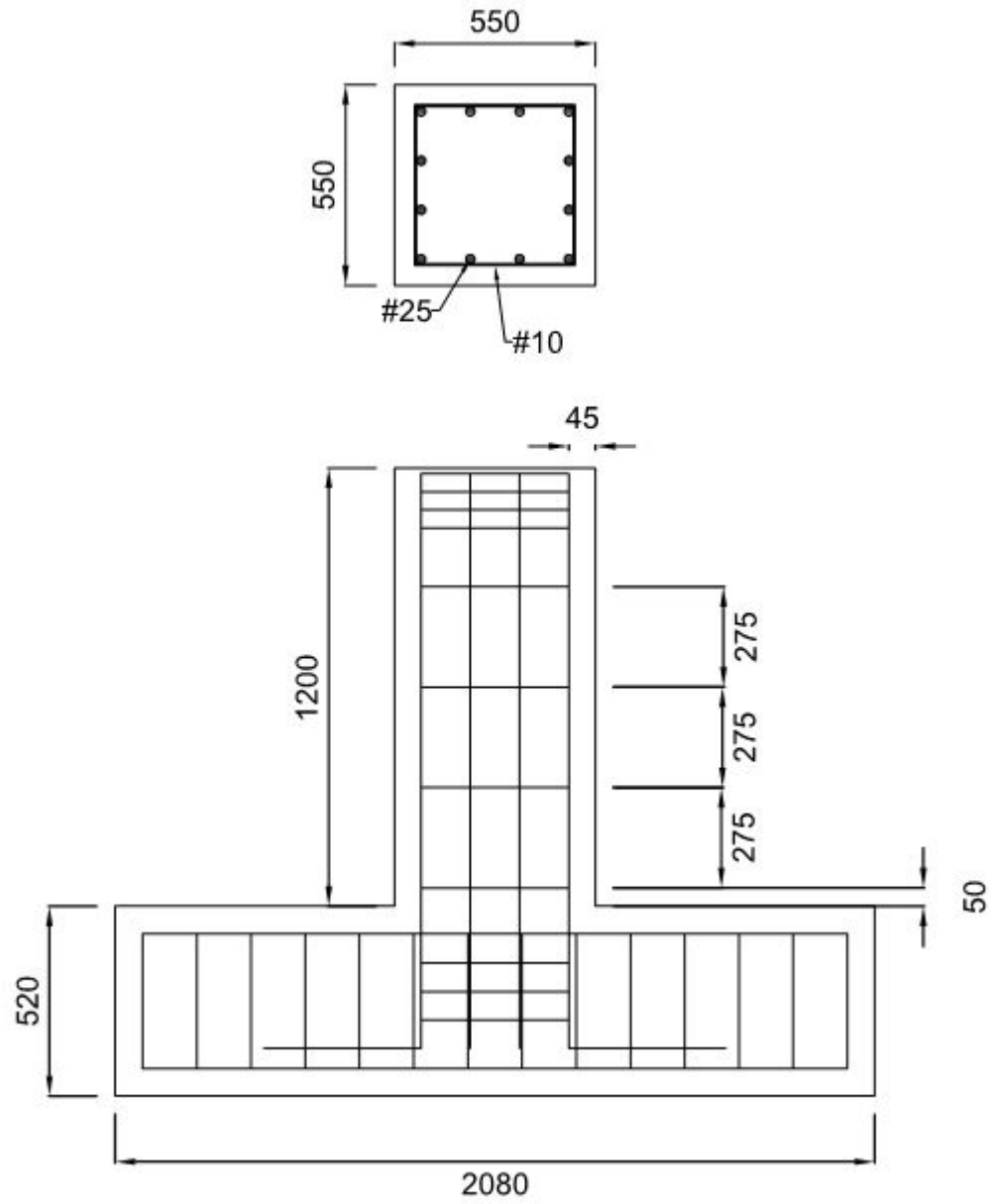


Figure 2-3 Geometric and reinforcement details for square column S-SQ



Figure 2-4 Footing formwork

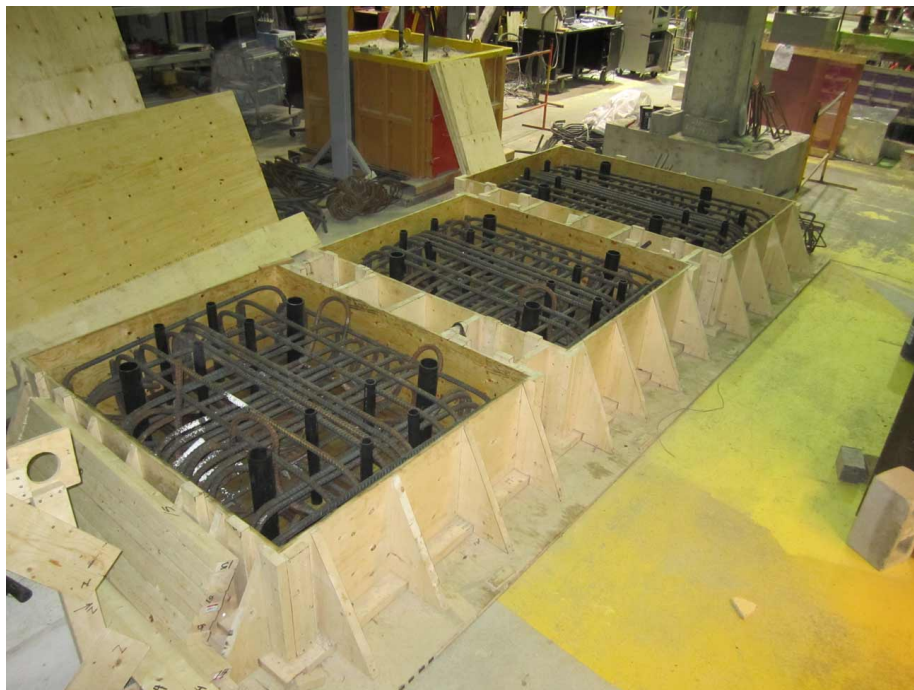


Figure 2-5 Footing formwork and reinforcement cages



Figure 2-6 Footings after concrete casting



Figure 2-7 Column reinforcement cages



Figure 2-8 Column formwork after casting



Figure 2-9 Pneumatic tensioner



Figure 2-10 Pneumatic sealer machine



Figure 2-11 Sealer



Figure 2-12 Prestressing process

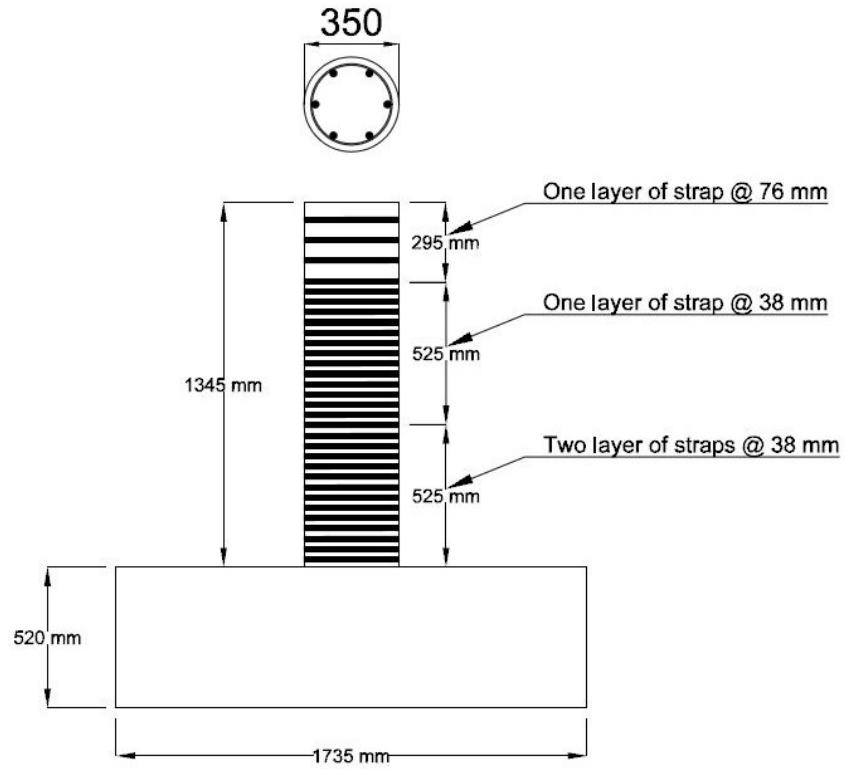


Figure 2-13 Retrofit details of circular column

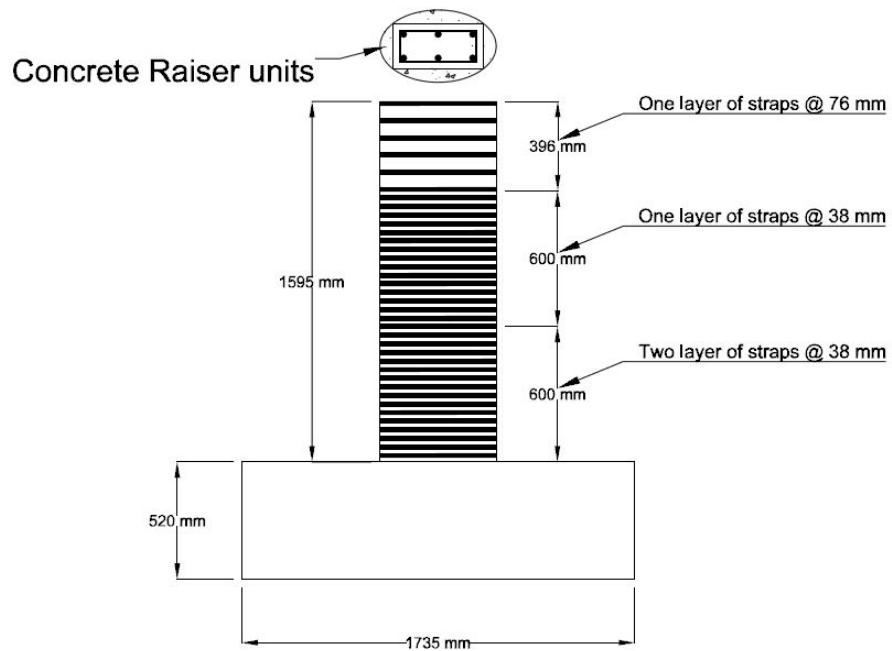


Figure 2-14 Retrofit details of rectangular column

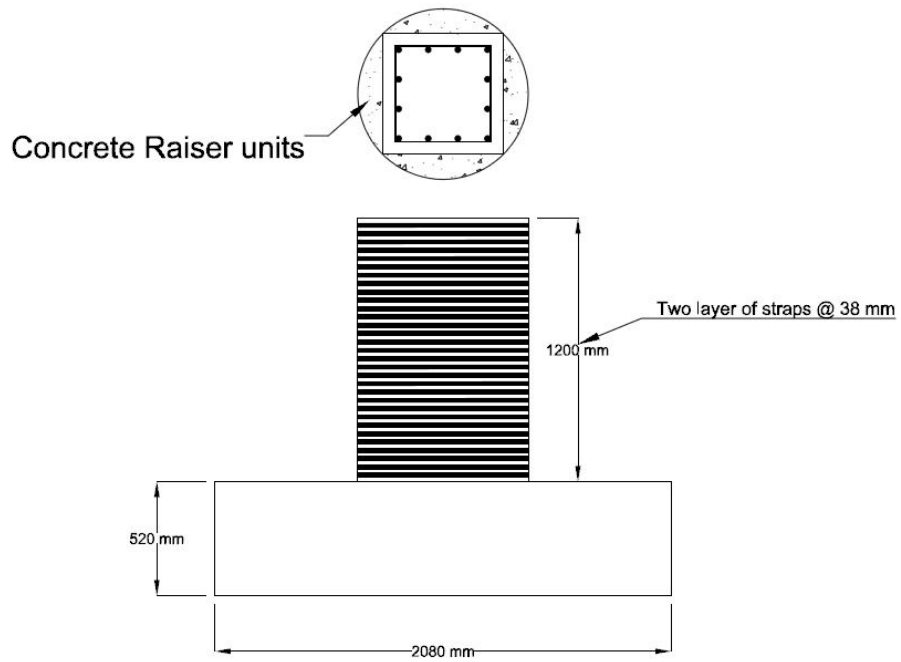


Figure 2-15 Retrofit details of square column



Figure 2-16 Retrofit details of square column



Figure 2-17 Concrete raiser unit formwork built around columns



Figure 2-18 Casting of the first layer of concrete raiser units for square column



Figure 2-19 Placement of plywood on top of the first layer, and casting of the second layer



Figure 2-20 Third layer of concrete raiser units after concrete casting



Figure 2-21 Opening of sonotubes after one week



Figure 2-22 Removing the concrete raiser units out of the formwork



Figure 2-23 Placement of cement paste between concrete raiser units and columns

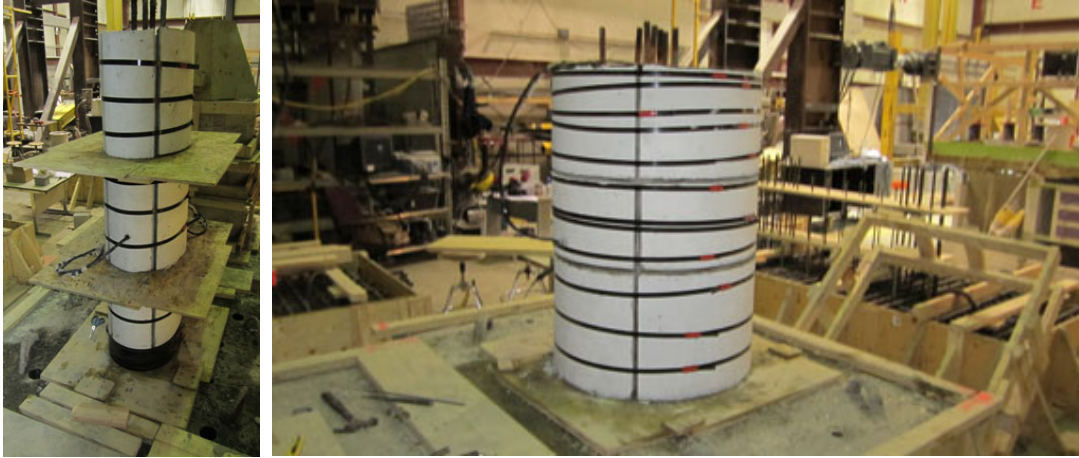


Figure 2-24 Temporary prestressing of concrete raiser units as the cement paste dries

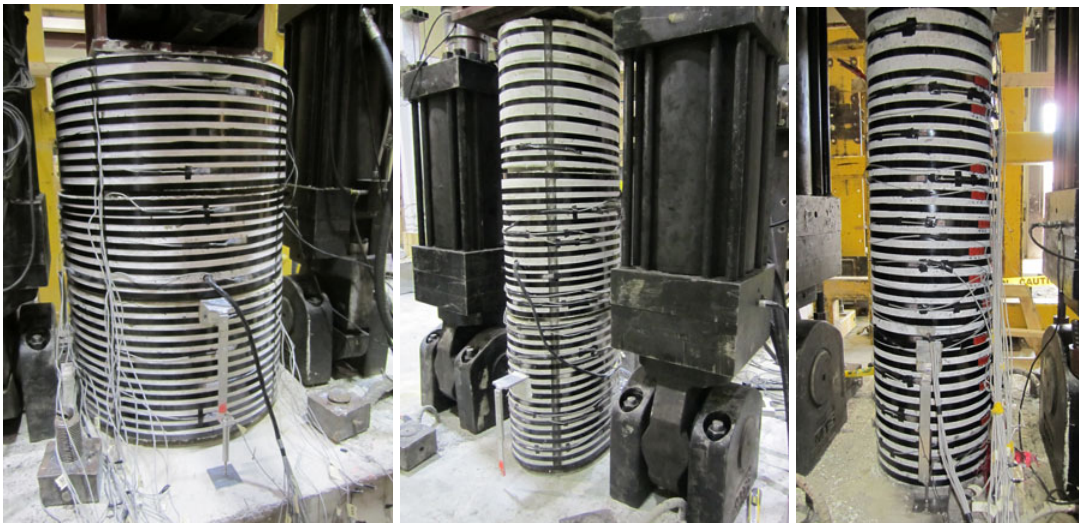


Figure 2-25 All three columns after final prestressing, from left to right; square column, rectangular column, and circular column



Figure 2-26 Concrete compression testing machine

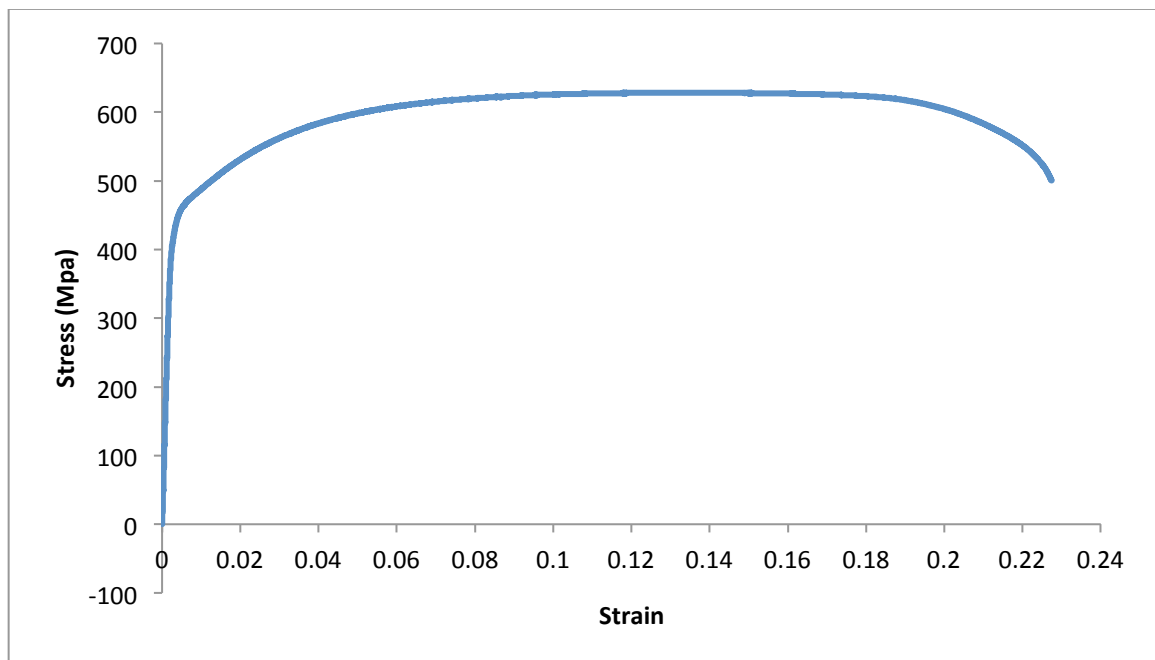


Figure 2-27 Stress-strain relationship of No. 10 steel bar used as transverse reinforcement

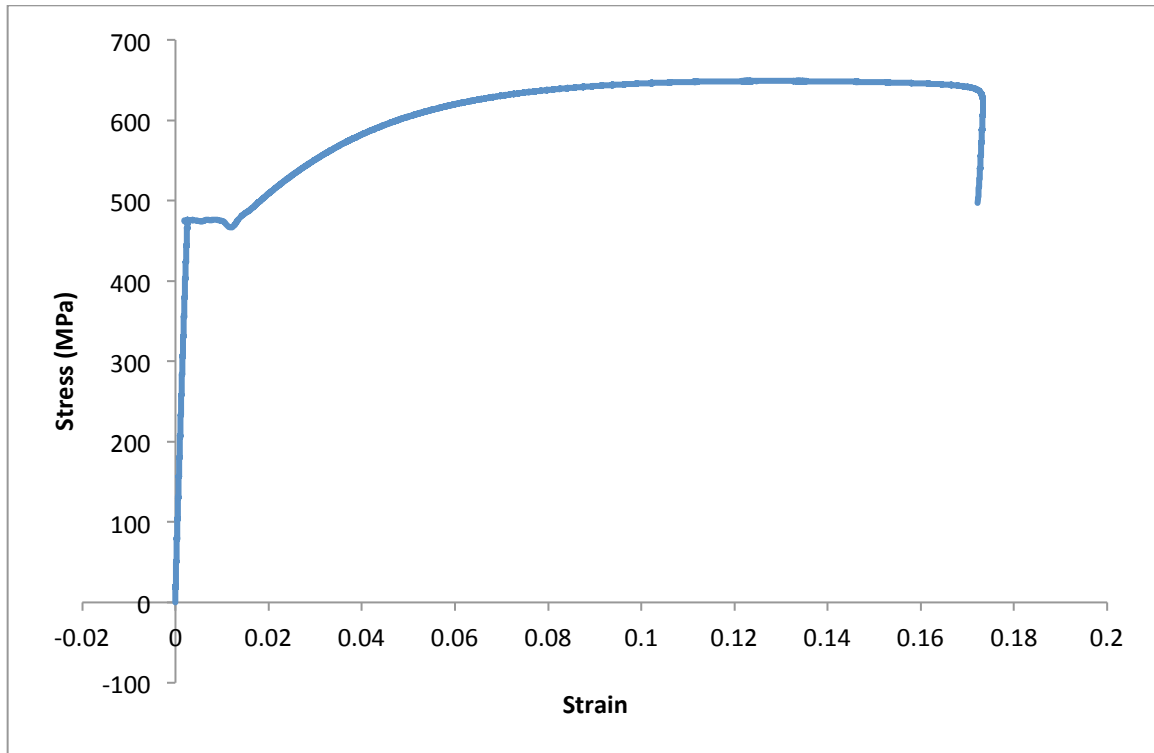


Figure 2-28 Stress-strain relationship of No. 25 steel bar used as longitudinal reinforcement



Figure 2-29 From left to right: Coupon Type 1 - Solid Strap (no sealer), Coupon Type 2 - Two straps connected by a sealer, and Coupon Type 3 - Three straps connected by a sealer



Figure 2-30 From left to right: Galdabini testing machine, and Coupon Types 1, and 2 after the rupturing of steel.

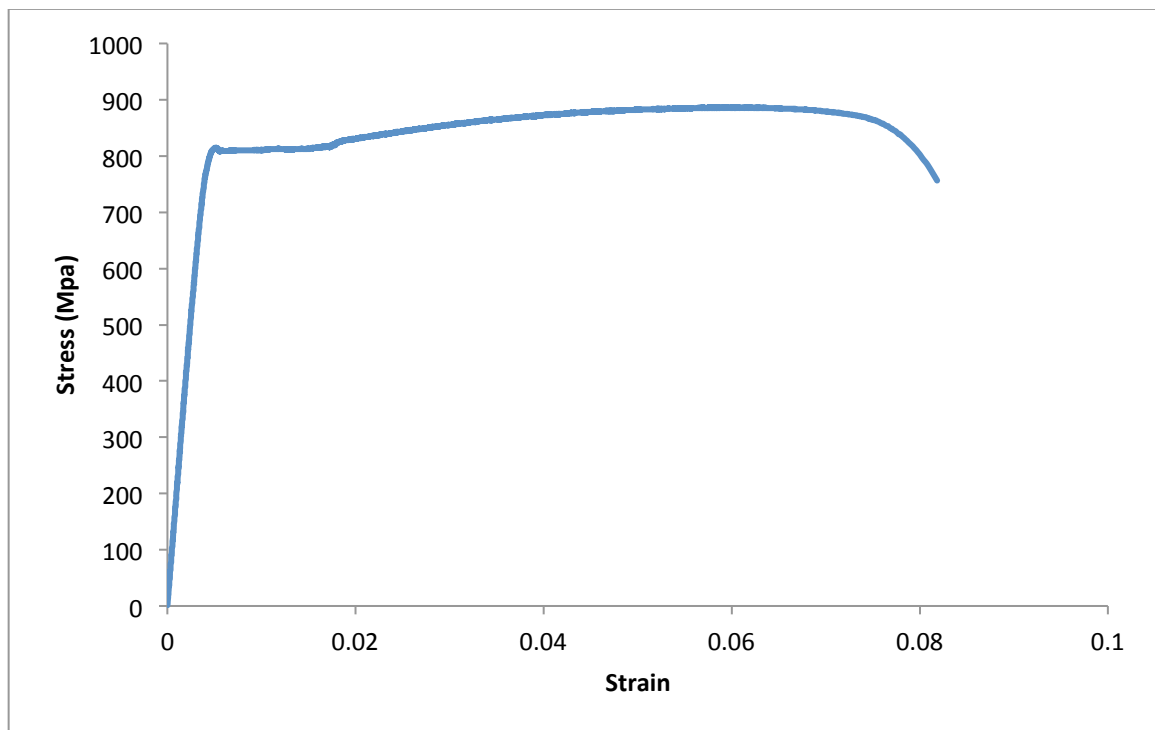


Figure 2-31 Stress-strain relationship of Coupon Type 1 (solid strap - no sealer)

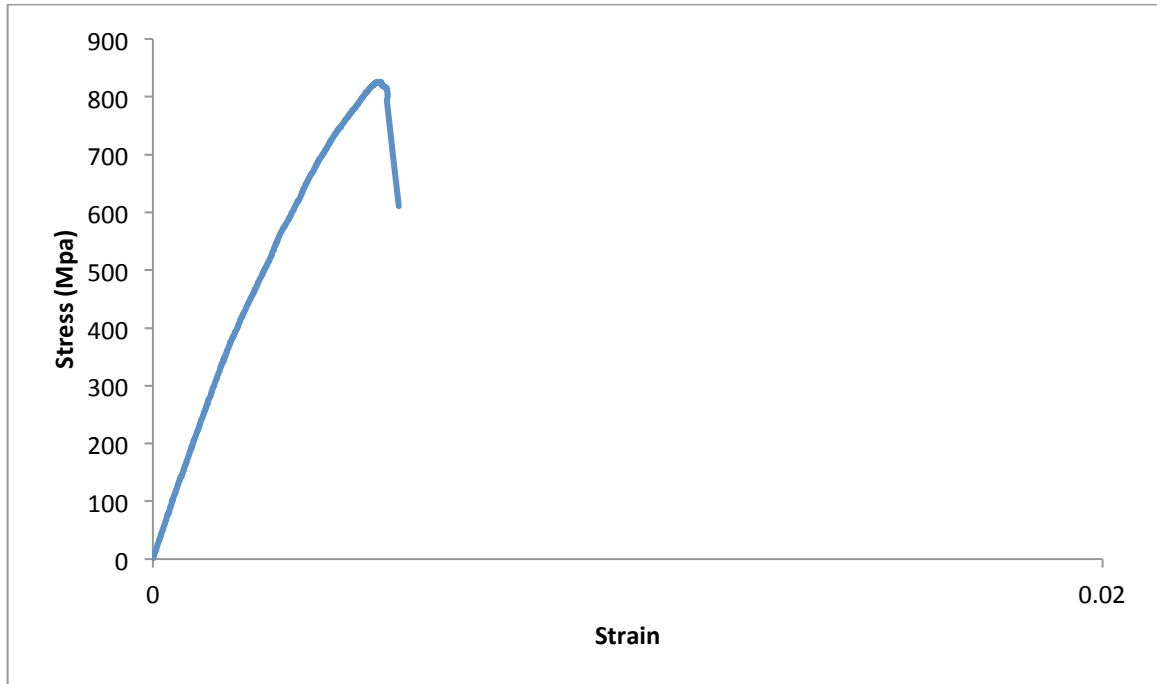


Figure 2-32 Stress-strain relationship of Coupon Type 2 (Two straps connected by a sealer)



Figure 2-33 Stress-strain relationship of Coupon Type 3 (Three straps connected by a sealer)

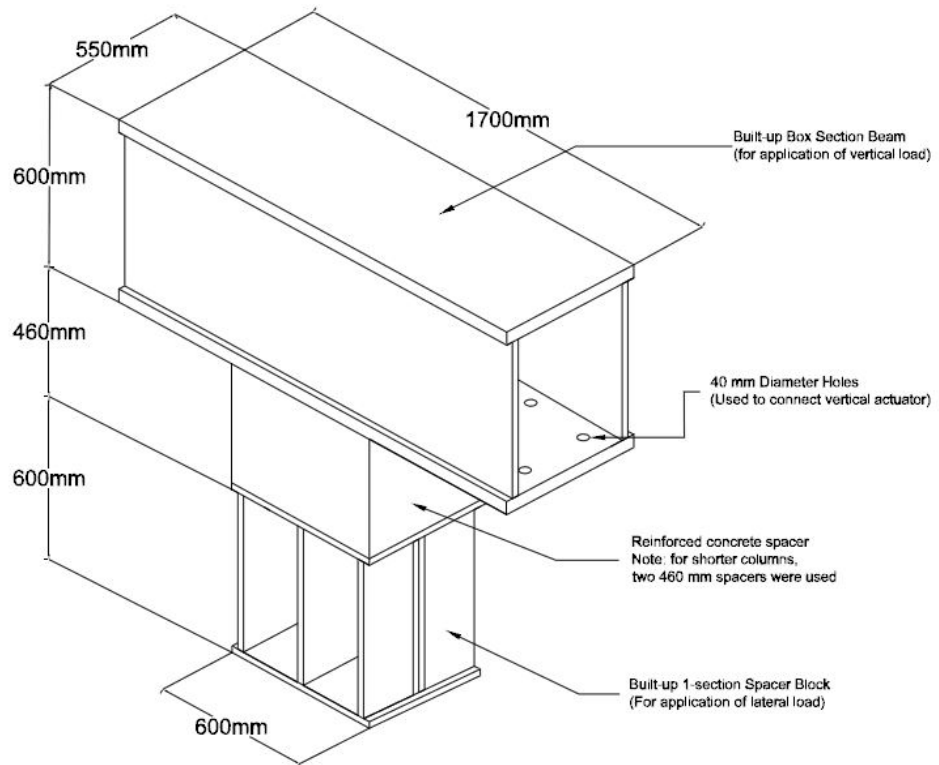


Figure 2-34 Details of loading beam assembly

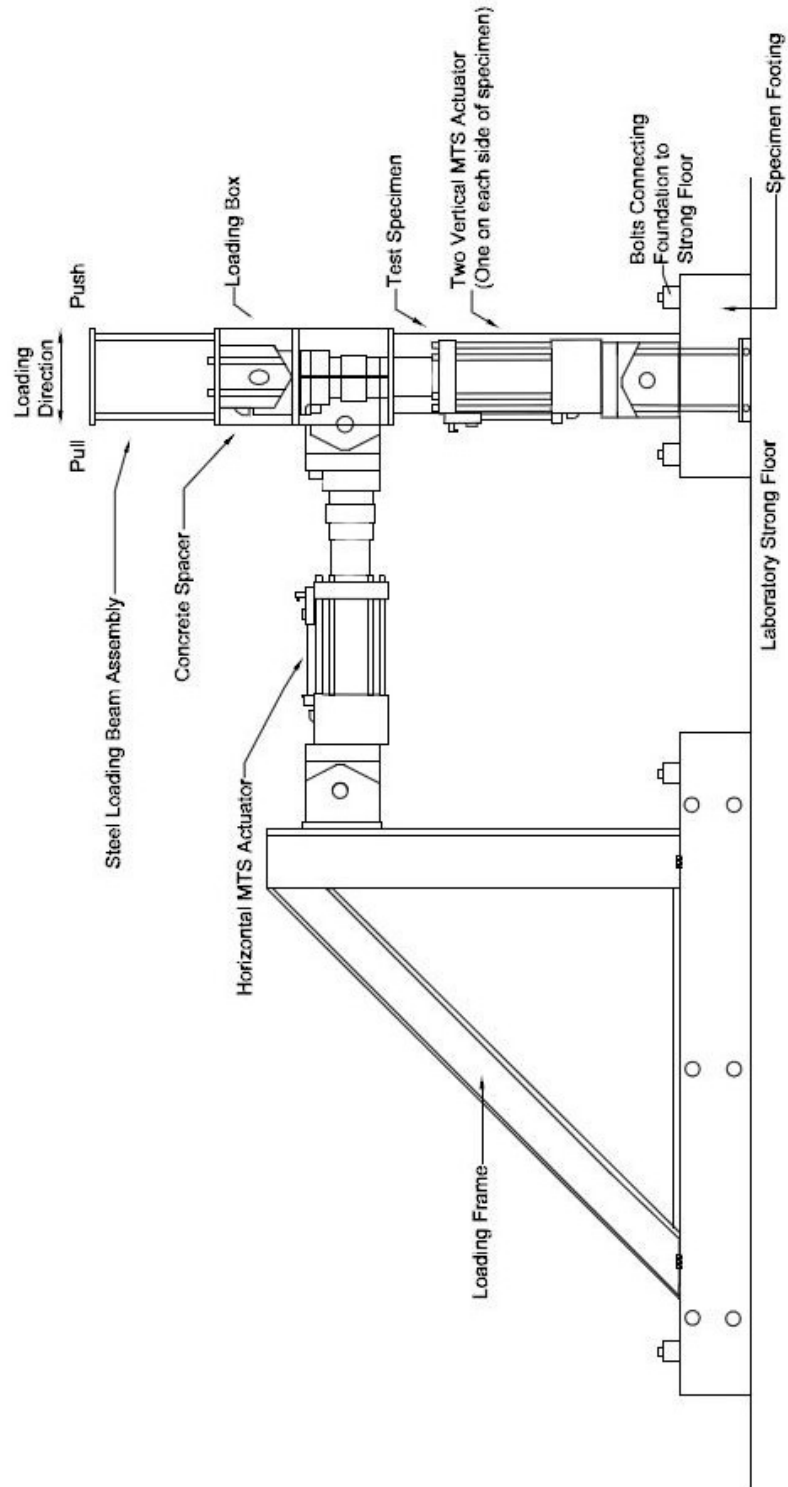


Figure 2-35 Side view of test setup

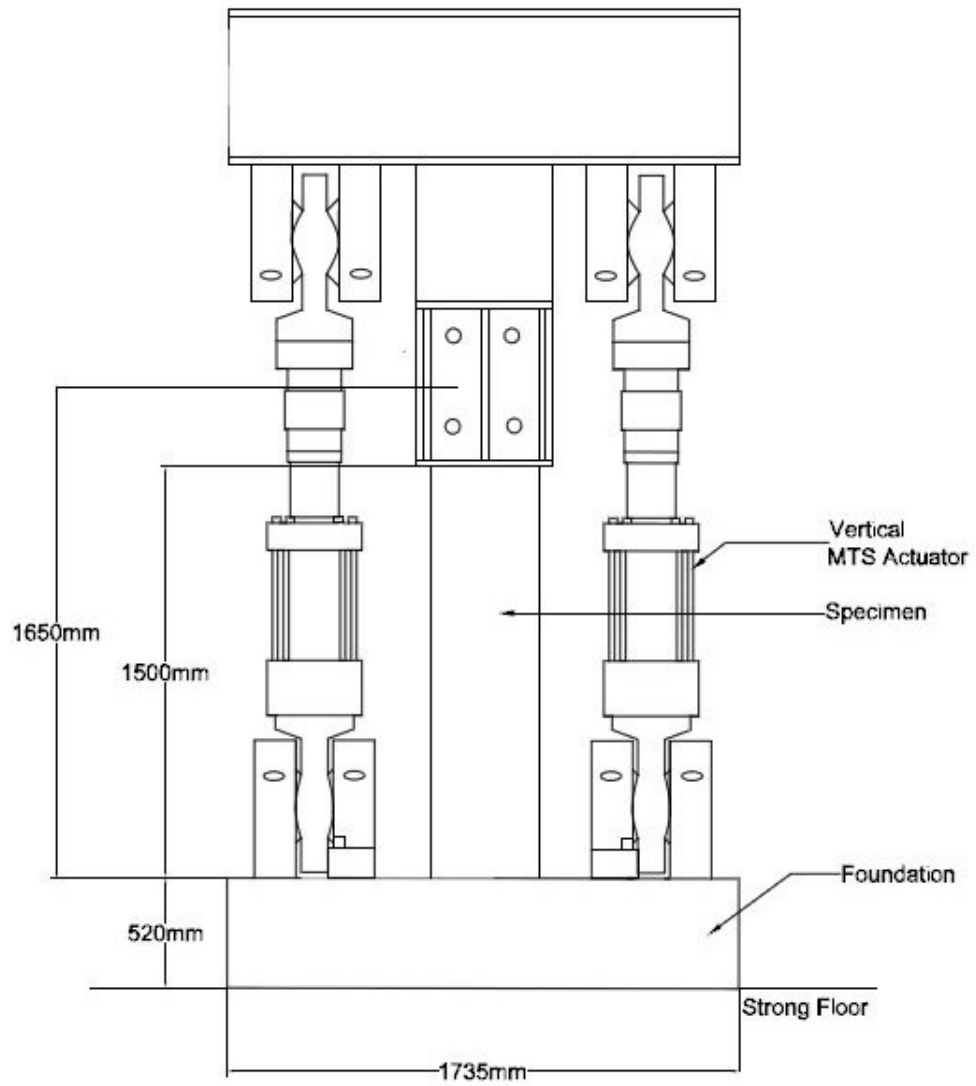


Figure 2-36 Front view of test setup

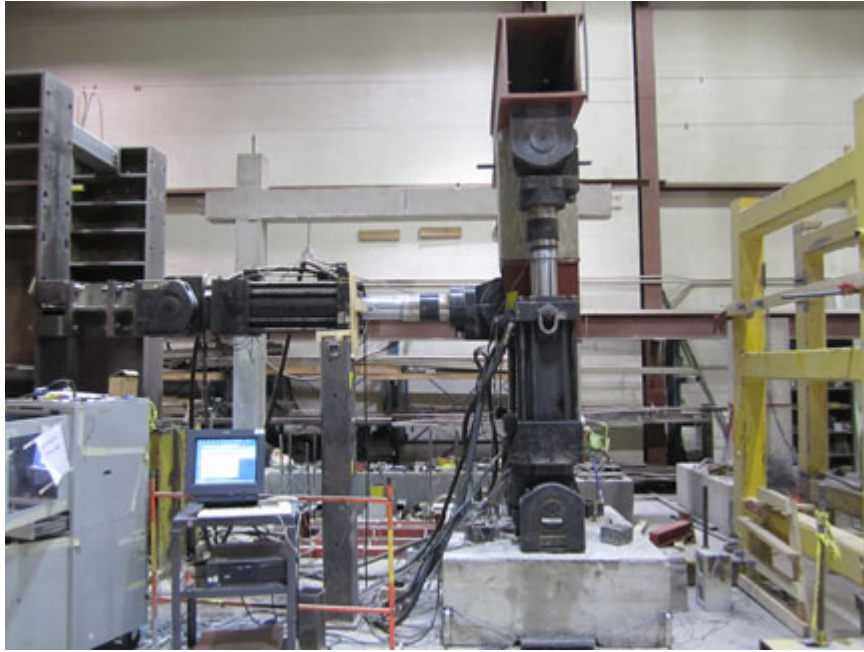


Figure 2-37 Photos of test setup

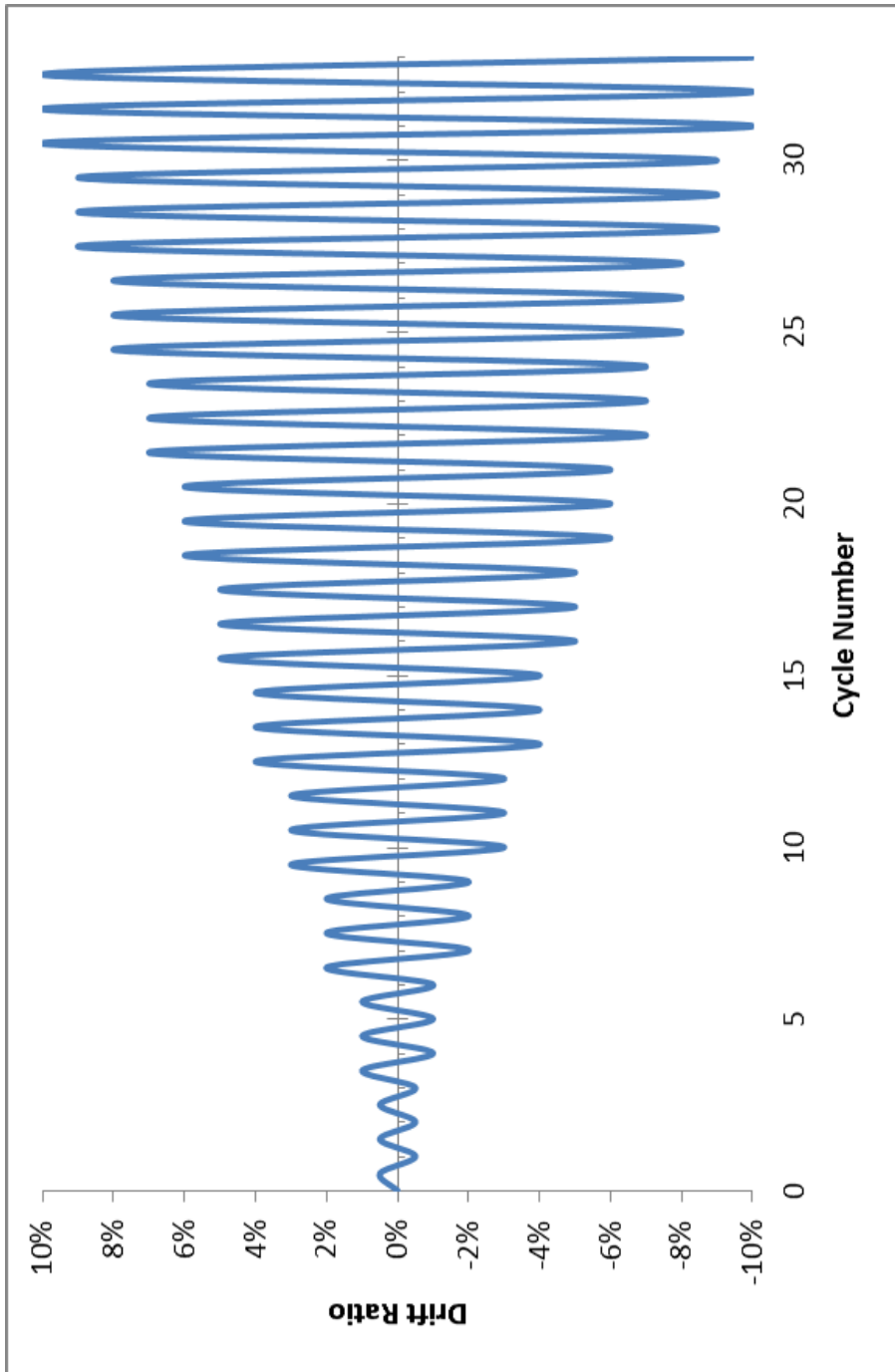


Figure 2-38 Loading program

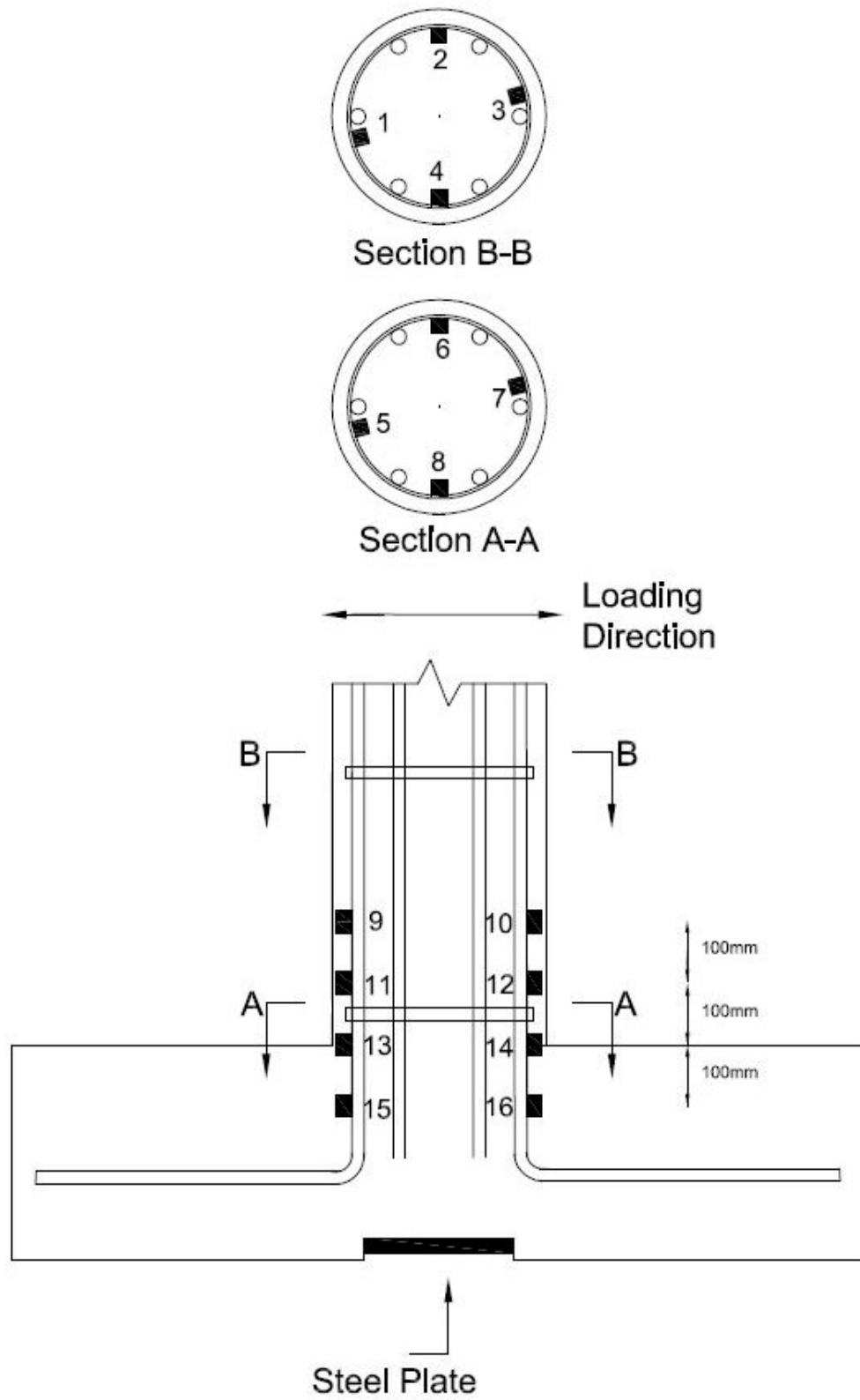


Figure 2-39 Strain gauge locations on reinforcing steel for circular column

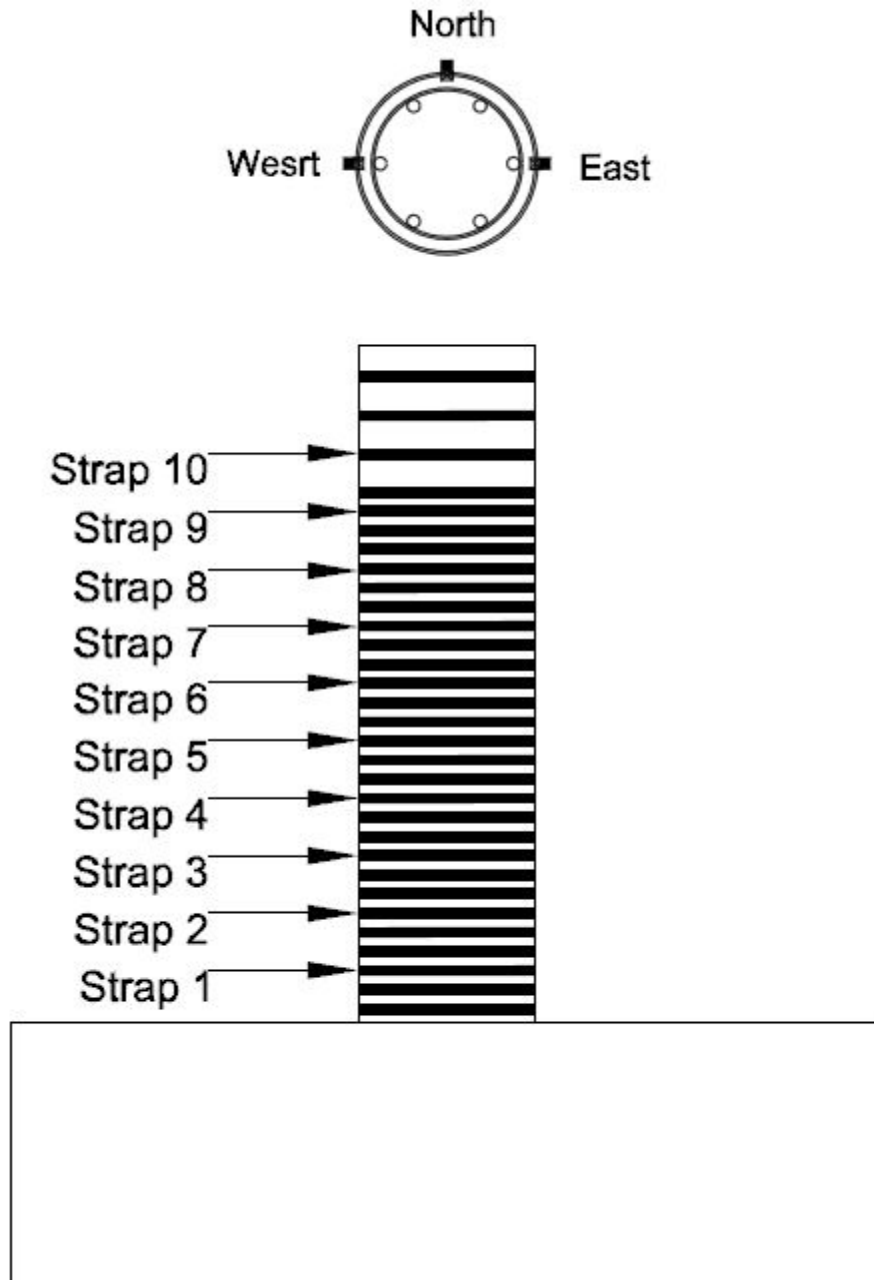


Figure 2-40 Locations of straps with strain gauges for the circular column (each strap has three strain gauges on east, north, and south sides)

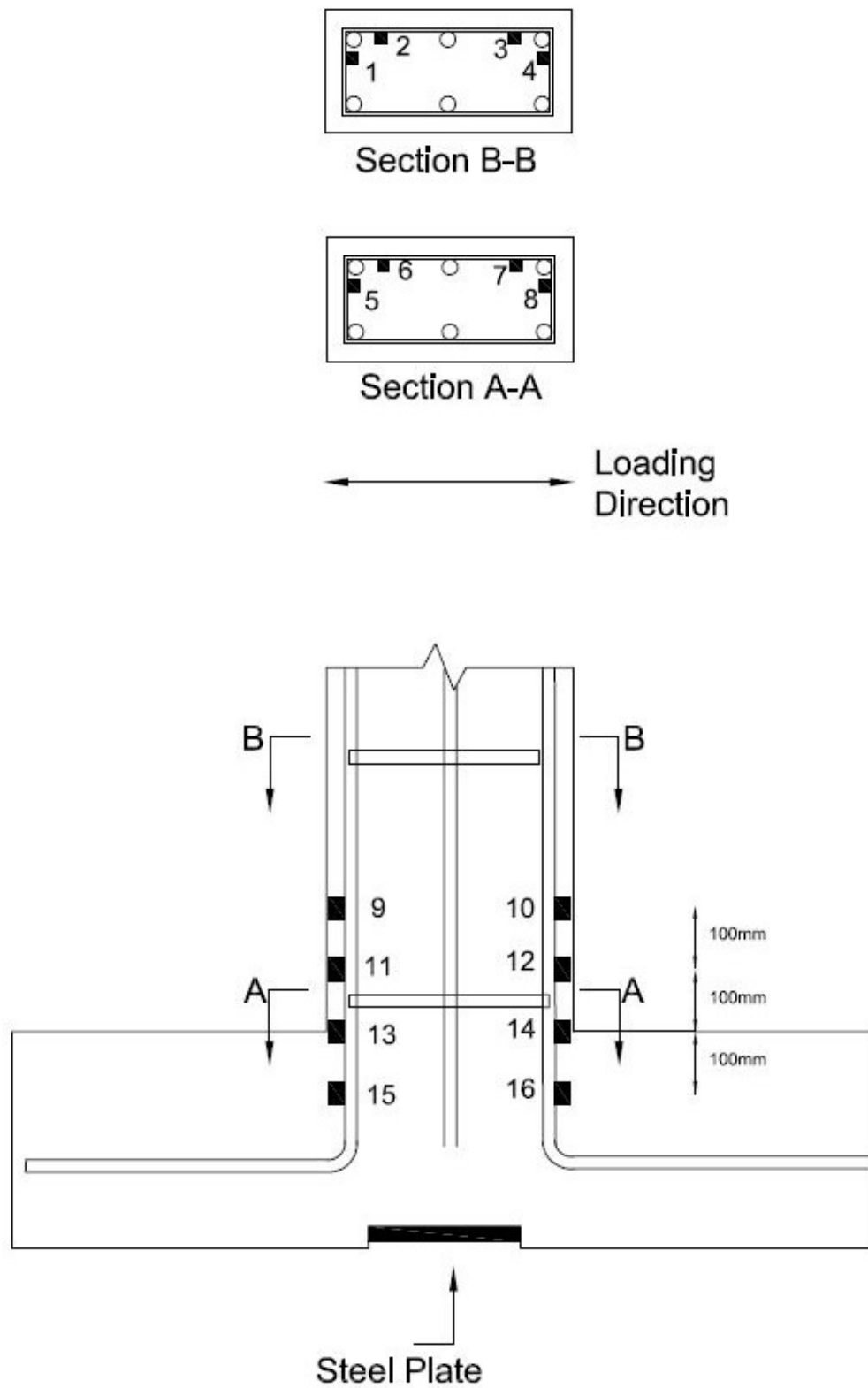


Figure 2-41 Strain gauge locations on reinforcing steel for the rectangular column

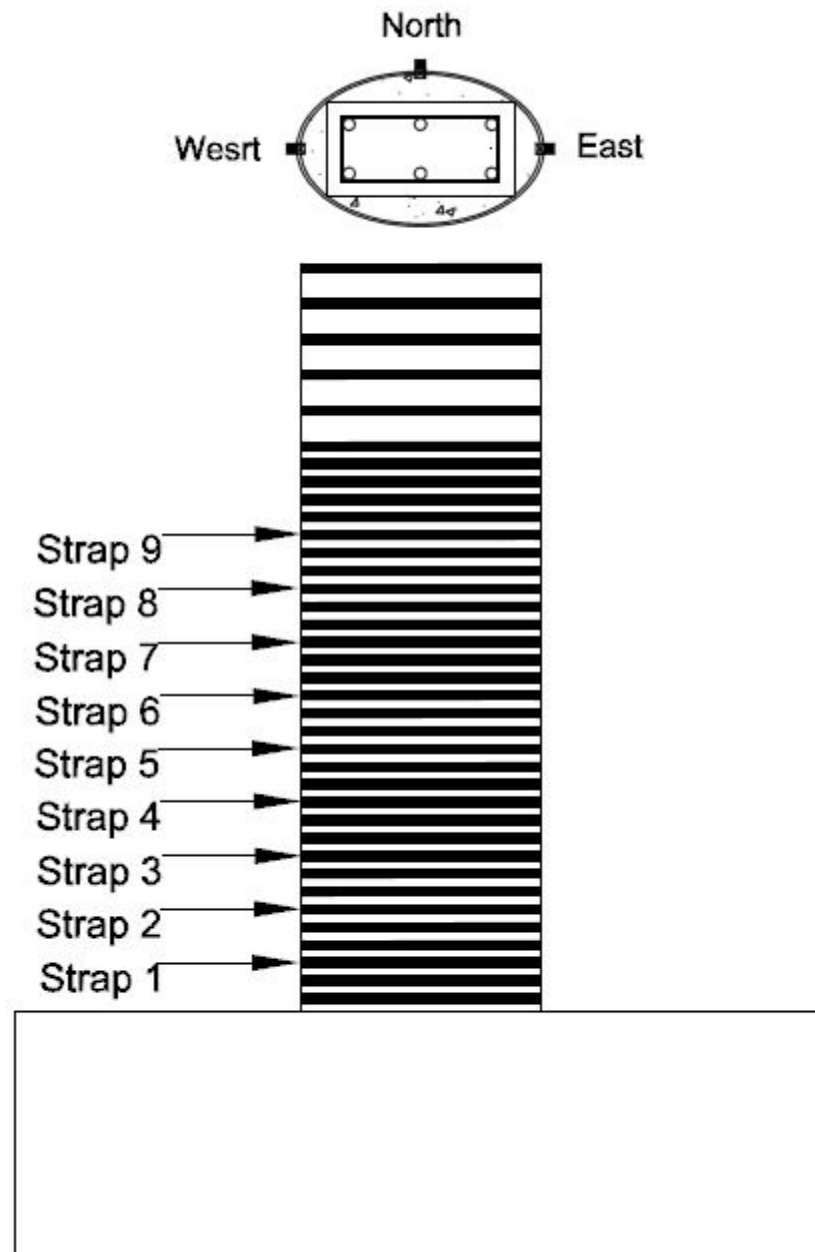


Figure 2-42 Locations of straps with strain gauges for the rectangular column (each strap has three strain gauges on east, north, and south side directions)

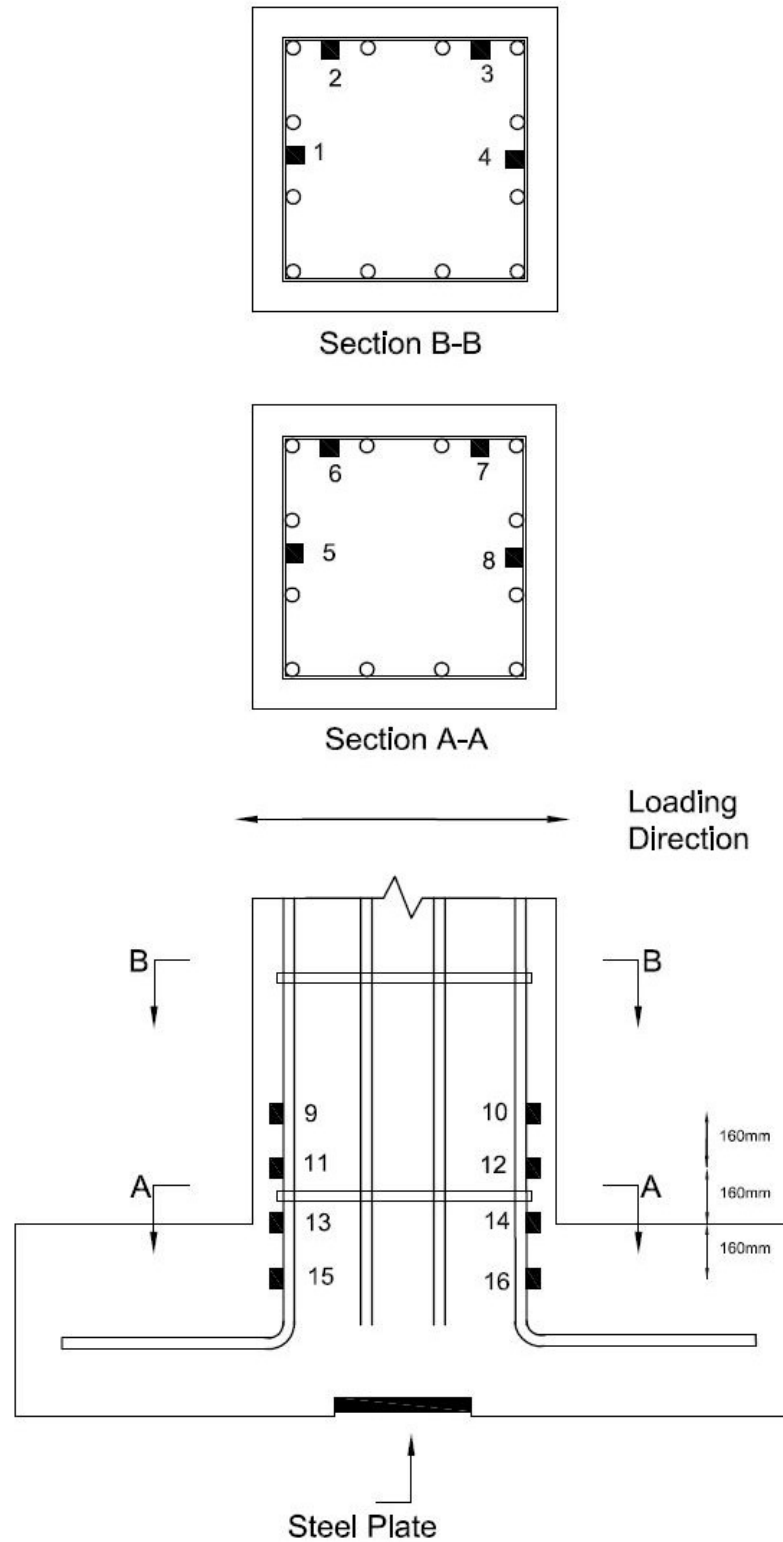


Figure 2-43 Strain gauge location on reinforcing steel for the square column

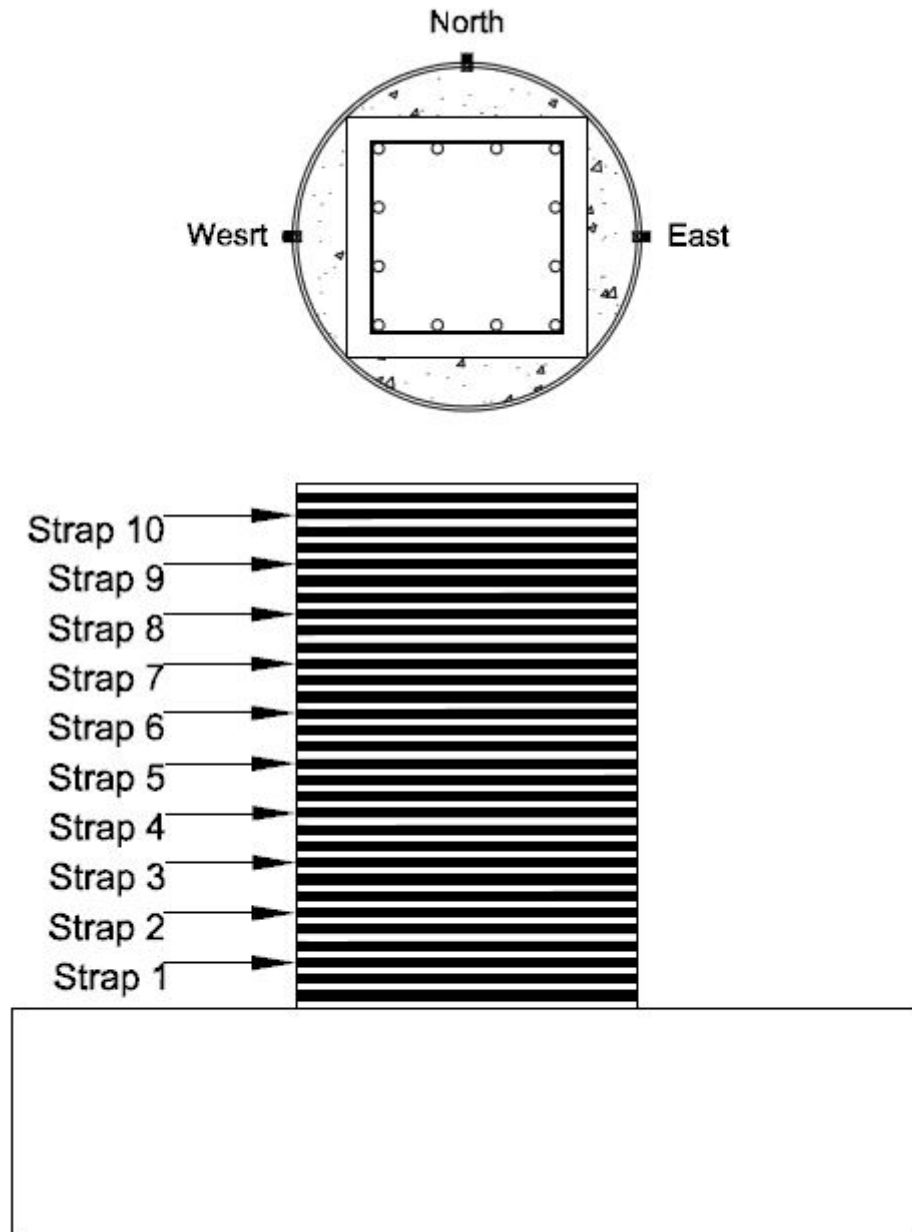


Figure 2-44 Locations of straps with strain gauges for the square column (each strap has three strain gauges on east, north, and south side directions)

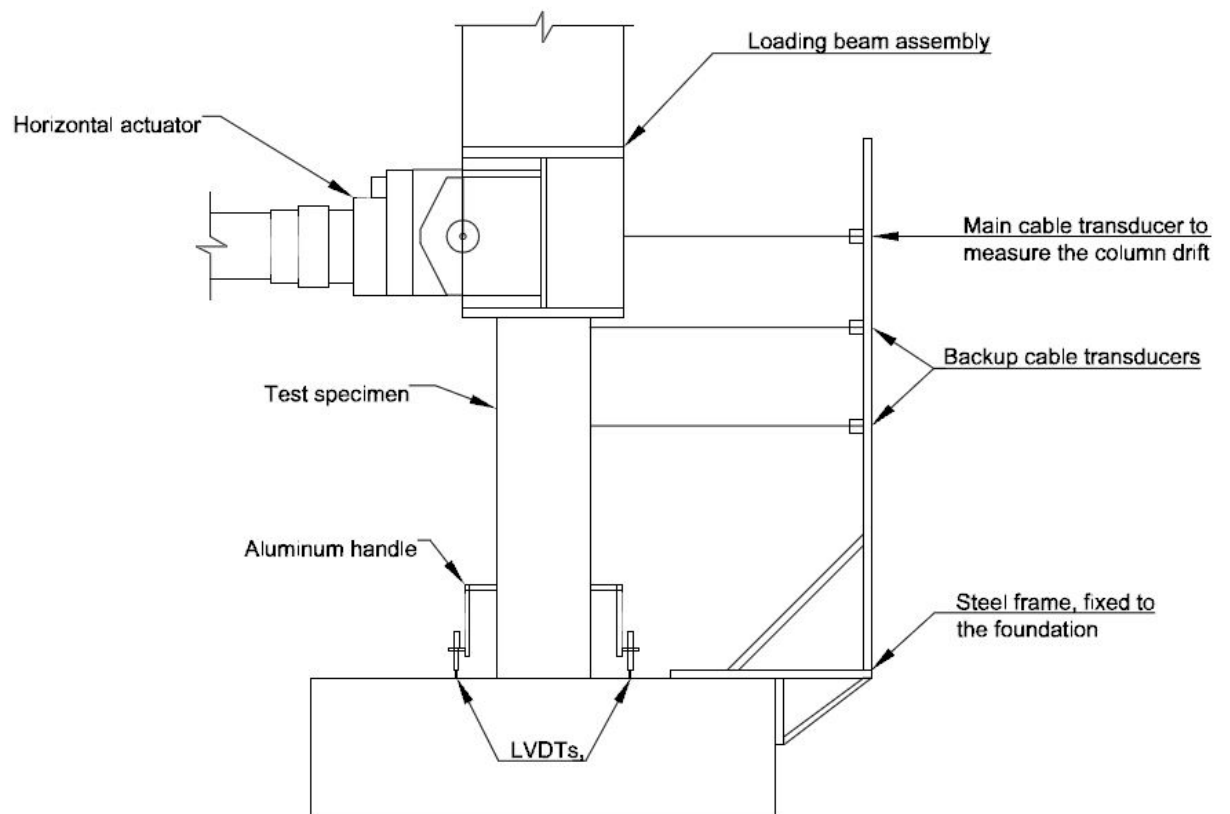


Figure 2-45 Location of LVDTs and cable transducers

Chapter 3: Test Results

3.1 Introduction

The results of three retrofitted concrete columns, tested under simulated seismic loading as described in Chapter 2, are presented in this chapter. Test observations, as well as data analysis are provided for each column. The experimental results are compared with those obtained from testing companion/equivalent non-retrofitted columns reported in the literature to assess the significance of the retrofit strategy employed. The test results for circular and rectangular building columns are compared with similar non-retrofitted building columns tested elsewhere by other researchers. The results of the shear dominant square column are compared with those obtained from a companion non-retrofitted column tested by Yalcin and Saatcioglu (1998) in the first phase of the overall research program at the University of Ottawa.

The lateral load drop down of 80% and more is considered as the failure of the column. The drift capacity of column is the drift level that the column has completed three cycles in it with less than 20% strength decay. The lateral force-drift and moment-drift hysteretic graphs as well as strain-drift graphs for strain gauges are presented at the end of the chapter. Column photos at different drift levels are also presented at the end of chapter.

Some of the measuring instruments like LVDTs and strain gauges have damaged during the test and couldn't provide data throughout the test.

3.2 Flexural Dominant Circular Column (F-CTR)

3.2.1 Test Observations

The flexural dominant circular building column, with 50% P_o as the concentric axial load showed excellent seismic behaviour during testing, which is attributed to the retrofit technique employed. Both ductility and energy dissipation capacities were observed to be favorable, as discussed later in this section. Figure 3.1-3.2 shows the progression of damage observed at different stages of lateral deformation.

Elastic force reversals at 0.5% drift ratio did not result in surface cracks or deterioration of concrete. The lateral force resistance at this stage of loading was 60 kN during the first push, and -50 kN during the first pull of the initial three deformation cycles. Three cycles at 1% drift ratio resulted in small flexural cracks on the column surface at a height of h (cross sectional dimension of the column) and $1.5h$ above the foundation. The lateral load increased to 92 kN and -94 kN during the first cycle at 1% lateral drift. The observed cracks expanded and widened after three cycles at 2% drift ratio. Many new cracks formed, most of them above the height of h . The maximum lateral load resistance of the column was 130 kN and -143 kN during the first cycle at 2% drift ratio. The maximum load resistance in the push direction of this load stage was also the maximum lateral load capacity of the column in the push direction. During the application of three cycles at 3% drift ratio, the previous cracks enlarged and extended over the surface of the column. A wide crack became obvious at the bottom of the column, approximately 60 mm above the footing surface. Some spalling of cover concrete occurred on the east side of the column, 100 mm above the foundation. At this stage of loading, the lateral load resistance of the column was 121 kN and -158 kN in the

push and pull directions, respectively. This was the maximum lateral load capacity of the column in pull. The 37 kN difference in force resistance between the push and the pull sides was attributed to the spalling and loss of cover concrete on the east side. During 4% drift cycles, the previously formed cracks enlarged dramatically, and the concrete spalling increased at the bottom of the column. The lateral load resistance of the column dropped by 17% and 11% (relative to the maximum load resistances) in push and pull directions, respectively. At the end of the third cycle of 5% drift ratio, the exterior layer of the third steel strap ruptured due to the lateral expansion of column concrete. At this drift level, the lateral load resistance of the column dropped by 27% and 14% relative to the maximum load capacity in the push and pull directions, respectively. During the third cycle of 6% lateral drift, the exterior layer of steel strap 2 and 4 ruptured during push, and the interior layer of strap 1 and interior layer of strap 3 ruptured during pull. The lateral load resistance of the column dropped to 86 kN and -112 kN during the first cycle of 6% drift, corresponding to 66% and 70% of maximum resistances in push and pull, respectively. During the first push cycle at 7% drift ratio, the inside layer of Strap 2 ruptured. Similarly, during the first pull cycle at 7% drift ratio, the inside layer of Strap 4 and outside layer of Strap 5 ruptured. The straps, when ruptured, always ruptured at the sealer location. This is due to the decreased cross sectional area of the strap in that area, caused by the crimping of steel. During the second cycle at 7% drift ratio the column suddenly collapsed under axial compression, and the test was stopped. At the end of the test it was observed that all the longitudinal bars were buckled and the core concrete was completely crushed.

3.2.2 Data Analysis

Experimentally recorded force-displacement and moment-displacement hysteretic relationships are plotted in Figure 3.2. The $P-\Delta$ effect is included in the moment-displacement relationship. The stable hysteresis loops shown in the figure indicate that the prestressing straps were effective in improving the seismic performance of the column, even under a high level of axial compression. The column was able to sustain more than 80% of its maximum lateral force resistance up to a lateral drift ratio of 5%, developing its peak resistance of 158 kN at 3% lateral drift ratio.

The rotation of the assumed column hinging region was instrumented and measured at the column base, within a distance equal to the column cross-sectional dimension in the direction of loading “h.” The rotation measurements were taken by means of two LVDTs, each located on one side of the column vertically, measuring the column rotation within “h.” Figure 3.4 shows the moment-rotation hysteretic behaviour. The measured rotations consist of two components; i) rotations within “h” due to flexure, and ii) rotations associated with the extension of longitudinal reinforcement within the footing (anchorage slip) and the resulting wide crack at column-footing interface. However, the latter component is believed to be negligibly small under high axial compression prior to the strain hardening of longitudinal reinforcement in tension.

The strains in reinforcement are monitored and recorded during the test by means of strain gauges. The measured strains recorded by strain gauges are shown in Figure 3.5-3.9. Steel strain measurements include longitudinal bars, transverse reinforcement, and steel straps. These measurements were recorded up to 5% lateral drift. Maximum strains on longitudinal reinforcement were recorded at 100 mm above the foundation. Based on

the recorded strains, the longitudinal bars started to yield shortly after 1% drift. The strains recorded on Stirrup 1 near the footing were higher than the strains recorded on Stirrup 2, second level above the footing. On both stirrups, the strains recorded on east and west sides were higher than strains recorded on north and south sides. This is because of the direction of load reversals, which was east-west. The strains on Stirrup 2 never passed the yield point, but Stirrup 1 started yielding after 4% lateral drift.

Strains were also measured on the prestressing steel straps. Figure 2.39 shows the location of strain gauges on steel straps. Three strain gauges were attached on each strap to measure the strains on east, west, and north sides of the column. The strain-drift diagram of each strain gauge is shown in Figure 3.4. Some of the strain gauges didn't work properly or they stopped working after some point. The straps that were located within the plastic hinge region show higher strains as compared to the straps that were located at higher elevations. The highest strains occurred in strap 1 (Figure 2.40), which explains its early rupture.

3.3 Flexural Dominant Rectangular Column (F-CR)

3.3.1 Test Observations

The rectangular building column, tested under 50% of concentric compression, performed well when retrofitted with prestressed transverse steel straps, placed on elliptical concrete raiser units. The column showed improved deformability up to about 3.6% lateral drift, and failed during the subsequent 4.5% drift cycles. The progression of damage under increasing lateral deformation reversals is shown in Figure 3.10-3.11.

The main cable transducer used to keep track of lateral drift cycles during testing had minor calibration-related inaccuracies. Therefore, lateral deformations were obtained

subsequently from the back-up instrumentation. This resulted in some discrepancies between the intended drift levels and the actual imposed on the column. For example 1% drift cycles were actually imposed at 1.3% drift ratio. This did not affect the test, but explains slight differences between the intended and actual drift values.

This column was covered with elliptical concrete raiser units, and hence it was not possible to monitor and observe crack patterns on the column. However, the cracking of concrete units and some cracking between the units could be observed under increasing lateral drift cycles. No cracks and damage to concrete was anticipated during the elastic load cycles at 0.5% drift ratio. The lateral load resistance of the column at 0.5% drift was 87 kN at first push, and -64 kN at first pull load. The lateral load increased to 145 kN and -142 kN during the first cycle at 1.3% drift. This was the maximum lateral load capacity of the column on the push side. After three cycles at 1.3% drift, a horizontal crack occurred on the east side of the column at a height of 250 mm above the footing. After cycling the column at 2.5% drift level, similar horizontal cracks occurred on the west, north, and south sides of the column at a height of 250 cm above the foundation. At 2.5% drift level, the lateral load resistance of the column was 130 kN in the push direction, and -147 kN in the pull direction. This was the maximum lateral load capacity of the column in the pull direction. After three complete cycles at 3.6% drift ratio, the previously formed cracks enlarged, and additional new cracks were developed. At this drift level the lateral load capacity of the column dropped to 87% and 96% of the maximum load capacity of the column, in the push and pull directions, respectively. When the column was loaded to 4.5% drift cycles, all the steel straps in the plastic hinge region ruptured (both layers) at the column failed at the end of the first cycle at 4.5% drift ratio. During

this load stage the concrete raiser units were cracked and bulged out within the plastic hinge region. The lateral load resistance during the first pull force at 4.5% drift was 60% of the peak load, and during the second push force it was only 9% of the respective peak load. The sudden drop in load resistance, and the complete rupture of the steel straps in the plastic hinge region indicate that the amount of steel straps provided was not sufficient to develop increased inelastic deformability beyond 3.6% drift capacity. This could potentially be achieved if straps with larger cross sectional area were used, or if the number of strap layers were increased to more than the two used in the test.

3.3.2 Data Analysis

Figure 3.12 illustrates the hysteretic behavior of the rectangular column in terms of force-displacement and moment-displacement relationships. The P- Δ effect is included in the moment-displacement relationship. The hysteresis loops show stable loops with energy dissipation capacity up to 3.6% lateral drift, and failure at 4.5% drift. The maximum load capacity of the column occurred at 1.3% drift ratio with a gradual strength decay thereafter, up to 4.5% drift ratio, at which level the rupturing of external straps resulted in a sudden and brittle failure.

The column was instrumented with vertical LVDTs within the assumed plastic hinge region for rotation measurements, as in the case of the previous column. The moment-rotation hysteretic relationship is shown in Figure 3.13 and indicates a similarly good behaviour as those exhibited earlier in Figure 3.12 for the lateral drift. The rotations included in Figure 3.13 consist of those caused by flexure and anchorage slip, though the latter component is believed to be negligibly small in comparison with flexural rotations that took place in the plastic hinge region.

Steel strains were measured and monitored during testing. This was done through strain gauges that were attached to the longitudinal bars, transverse reinforcement, and the external prestressing straps. Figure 3.14-3.18 shows the strain-drift relationships for all the strain gauges. The highest strains on longitudinal bars were recorded by strain gauges that were placed at the column-footing interface. The yielding of longitudinal bars started at 1% lateral drift. The recorded strains on transvers hoops show higher strain on the second hoop from the base. The strain gauges that were located on the long side of the hoops (east-west direction), recorded higher strains, reaching up to 0.005. The recorded strains on one of the short sides of hoops were higher than those on the other side. This is because of the 90-degree hooks at one end, which were not effective in resisting the expansion of concrete.

Strains on steel straps were also measured by strain gauges located on the east, west, and north sides of the column. Strain-drift diagrams of un-damaged strain gauges are depicted in Figure 3.16-3.18. The strains that were measured on lower straps were higher than those measured at higher elevations. Higher strains were recorded on Straps 1 and 2 closer to the footing (Figure 2.42), which explains why they ruptured sooner. In most straps, the measured strains on the east and west sides of the column are higher than the strains measured on the north side of the column. This is attributed to the lateral expansion of concrete in the compression toe.

3.4 Comparisons of Retrofitted Circular and Rectangular Columns with Other Flexural Dominant As-built Columns

The two retrofitted building columns in the current experimental investigation were compared with observed performance of seismically deficient, un-retrofitted

columns. Performance of reinforced concrete columns with widely spaced transverse reinforcement, without the seismic hooks at the ends, shows premature crushing of compression concrete when subjected to combined axial compression and flexure. These columns suffer from lack of confinement in plastic hinge regions, and they exhibit brittle failure with low displacement ductility. While many test data exist in the literature, hence extensive research on concrete confinement, a few examples will be discussed in this section to demonstrate the improvements attained through the retrofit methodology implemented.

Saatcioglu and Mes (1999) tested two non-retrofitted flexural dominant columns as part of Phase II of the current research program. The first column was a 508 mm diameter circular column with a shear span of 2000 mm. It consisted of 12-#20 longitudinal bars with #10 transverse reinforcement spaced at 300 mm. The second column had a 500 mm square cross-section with a shear span of 2000 mm. It consisted of 12-#20 longitudinal bars with #10 transverse reinforcement spaced at 300 mm. Both columns were designed based on pre-1971 design codes. The test setup was similar to the setup used in the present study, but the axial load was 15% of the column concentric capacity. The lateral load resistance of the circular column reached its peak capacity of 246 KN at the first cycle at 2% drift. At the first cycle at 3% drift the column capacity dropped to 85% of its peak load capacity and at the second cycle at 3% drift it dropped to 176 KN, which was the failure of the column. The lateral load resistance of the square column reached its peak capacity of 308 KN at the first cycle at 2% drift. At the first cycle at 3% drift the lateral load capacity of the column started to deteriorate and at the third cycle the column was failed.

Saatcioglu and Yarandi (2007) tested a flexural dominant rectangular (350x700 mm) column with highly spaced transverse reinforcements and poor detailing. The effective height (shear span) of the column was 2000 mm. It was consisting of twelve #20 longitudinal bars with transverse reinforcement (#10) spacing of 300 mm and the cover concrete of 45 mm. The test setup was similar to the presented research and the axial load was 15% of the concentric capacity of the column.

The lateral load resistance of the rectangular column reached its peak capacity of 415 KN at the first cycle at 1% drift. At the first cycle at 2.5% drift the lateral load capacity dropped to 93% of its peak load capacity and at the second cycle it dropped to 287 KN, which was the failure of the column.

Moehel and Sezen (2002) tested two identical as-built columns in double curvature, behaving in the flexure mode. The columns were subjected to different level of axial compression, accompanied by lateral displacement reversals. The first column was tested with a low level of axial load (15% of concentric compressive capacity of the column), and the second column was tested with a high level of axial load (60% of concentric compressive capacity of the column). The specimens were loaded axially using two 400 kip-capacity vertical hydraulic actuators. The lateral load was applied by a 500-kip-capacity horizontal hydraulic actuator in displacement-control mode. Both columns had insufficient lateral reinforcement and poor seismic detailing. The drift capacity of the first column with low axial load was 3.4%, while the drift capacity of the second column with high axial load was 1.7%. The specimen with low axial load lost its lateral strength at a low level of lateral drift, but it tolerated the axial load at higher drift levels. The specimen with high axial load had higher lateral strength at low drift levels,

but it had a sudden axial failure. Their research clearly indicated that columns without sufficient transverse reinforcement had limited drift capacity.

The Comparisons of presented research studies with test results of retrofitted columns show that the seismic behavior of retrofitted columns was improved, due to lateral prestressing. The drift capacity of both circular and rectangular retrofitted columns was higher than 3% as a result of provided confinement by steel straps. However, the rectangular column could have more desirable behavior if higher amount of straps were applied.

3.5 Shear Dominant Square Column (S-SQ)

3.5.1 Test Observations

A square bridge column, with deficient shear reinforcement was retrofitted by external steel straps and tested under a relatively low level of axial load, corresponding to 15% of column concentric capacity, which is typical in bridge column design. Transverse prestressing through the use of externally placed steel straps improved concrete resistance to diagonal tension, while also providing additional shear reinforcement and associated shear capacity. This retrofit scheme improved the seismic behaviour of column relative to non-retrofitted column and effectively converted the brittle shear mode to a ductile flexure mode. The lateral load capacity of the column was increased, with a tremendous improvement in ductility and energy dissipation capacity. Figure 3.19-3.23 shows the progression of damage observed at higher levels of deformation reversals. The square column was covered with raiser concrete units to complete the cross-section to a circle, before the external prestressing was applied. Therefore, earlier cracking and limited

damage that may have taken place during the initial stages of testing are not visible from outside, until the concrete units covering the column are cracked and damaged.

After three cycles at 0.25% drift ratio no visible damage was observed. The lateral load resistance at this stage of loading was 403 kN and -356 kN during the first push and pull, respectively. The lateral load increased to 656 kN and -624 kN during 1% drift cycles. At 2% lateral drift, the lateral load resistance further increased to 702 kN and -720 kN. This was the peak load capacity of the column. There was no crack on concrete raiser units, but some small cracks could be seen on the column concrete through the gap between the units. The lateral load resistance of the column at 3% drift was 682 kN and -719 kN, slightly reduced from the resistance provided at the previous deformation level. No crack was observed on concrete raiser units, but some could be seen on the column, with increased crack width. At 4% drift, the column maintained its capacity with little or no sign of damage with load resistances of 696 kN and -690 kN during push and pull, respectively. Small hairline shear and flexure cracks were observed on concrete raiser units within the plastic hinge region. At 5% drift ratio, the lateral load resistance of the column dropped to 97% and 92% of the peak load capacity in push and pull, respectively. New cracks occurred on the first row of concrete raiser units, as previous flexure cracks extended and widened, especially on the west side of the units. After three complete cycles at 6% drift, steel strap No. 4 ruptured completely. Flexural cracks enlarged drastically on the first row of concrete raiser units. It was observed that, because of the gap provided between the rows of concrete raiser units, they were able to deform freely, without having any contact among them. This prevented crack development on the concrete raiser units. At the end of 6% drift cycles, the lateral load resistance dropped to

94% and 90% of the maximum load capacities in push and pull, respectively. At the first cycle of 7% drift, the lateral load resistance of the column decayed to 88%, and 77% of the maximum load capacity in push and pull, respectively. All steel straps around the first row of concrete raiser units ruptured during the first cycle at 7% drift ratio. The column suffered failure as it was forced to develop the second cycle at 7% drift. At this stage of loading, concrete raiser units on the east and west sides of the column broke into two pieces. The core concrete in the plastic hinge region was completely crushed.

3.5.2 Data Analysis

Force-displacement and moment-displacement hysteretic relationships are shown in Figure 3.24. The P- Δ effect is included in the moment-displacement relationship. The relationships indicate improved behaviour and ductile response, with lateral drift capacity as high as 6%, prior to failure at 7% drift ratio. The externally placed steel straps increased shear capacity, changed the mode of behaviour from shear to flexure and confined the compression concrete, enhancing inelastic deformability very significantly.

The column rotations within the assumed hinging region were measured by vertically placed LVDTs with a distance equal to the column section (h). Figure 3.25 illustrates the hysteretic moment-rotation relationship, showing stable hysteresis loops and ductile response. The rotations measured include contributions both from flexural rotations within the plastic hinge region, and anchorage slip of longitudinal reinforcement within the footing and the resulting column-footing interface crack. However, as in the previous two column tests, the contribution of anchorage slip to total rotations is believed to be small. The relationship shown in Figure 3.25 was obtained by plotting moments computed from lateral forces that were recorded by one data acquisition system against

rotations measured by another data acquisition system, which were not necessarily in sync during the test. Hence the resulting hysteresis loops are not as smooth as one would expect.

Steel strains were monitored and recorded during testing. Electric resistant strain gauges were placed on longitudinal bars, transverse reinforcement, and externally placed steel straps. The results are shown in Figure 3.26-3.30. The maximum strains on longitudinal bars were recorded at 160 mm above the foundation. Based on the recorded strains, the longitudinal bars started to yield shortly after 1% drift ratio. The strains recorded on Stirrup 1 were considerably higher than those recorded on Stirrup 2. The strains on Stirrup 2 never passed the yield point, but those on Stirrup 1 started yielding after 4% lateral drift.

Strains were also measured on prestressing steel straps. Strain-drift relationships for un-damaged strain gauges are depicted in Figure 3.28-2.30. Strains that were measured on lower straps were higher than strains measured at higher elevations. The maximum strain of 0.0035 is measured on Strap 1 (Figure 2.44), which explains its early rupture. The measured strains on the east, north, and west sides of column were approximately equal. This is because of the circular shape of the column.

3.6 The effect of Column Retrofit

It was clear from the observed behaviour of the column that the retrofit strategy implemented improved the column behavior tremendously. To quantify this improvement, the test results are compared with those obtained by Yalcin (1997) from a companion column tested without the retrofit. The comparisons of moment-displacement relationships clearly demonstrate the improvements attained in column behaviour. The

moment-displacement hysteretic relationship of the un-retrofitted square column, tested by Yalcin is depicted in Figure 3.31. As it is shown, the un-retrofitted column developed brittle failure shortly after 1% lateral drift. The maximum lateral load capacity of retrofitted column had 24% higher value as compare with the un-retrofitted column. The retrofitted column had 6% drift capacity, which is 6 times higher than that obtained from the companion un-retrofitted column. This shows that the proposed retrofitting technique was very effective in changing the failure mode from brittle shear failure to ductile flexural failure.



Figure 3-1 Behaviour of circular column



Figure 3-2 Behaviour of circular column

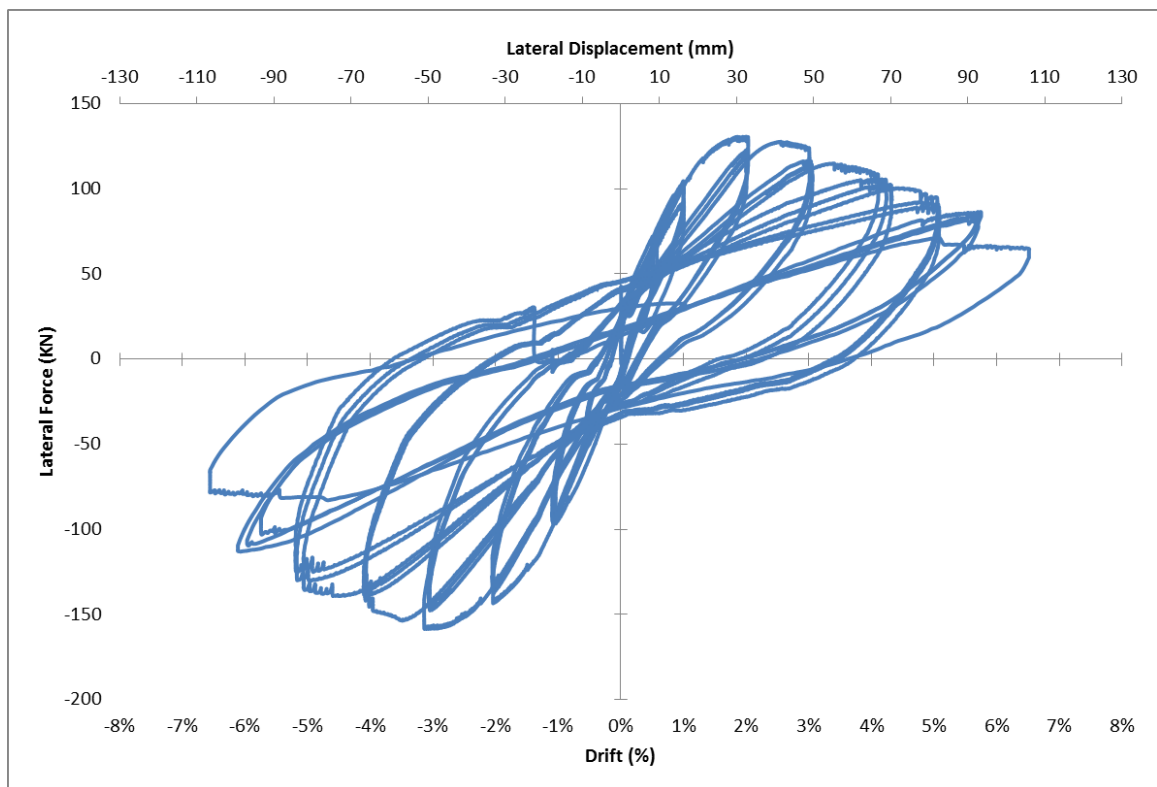
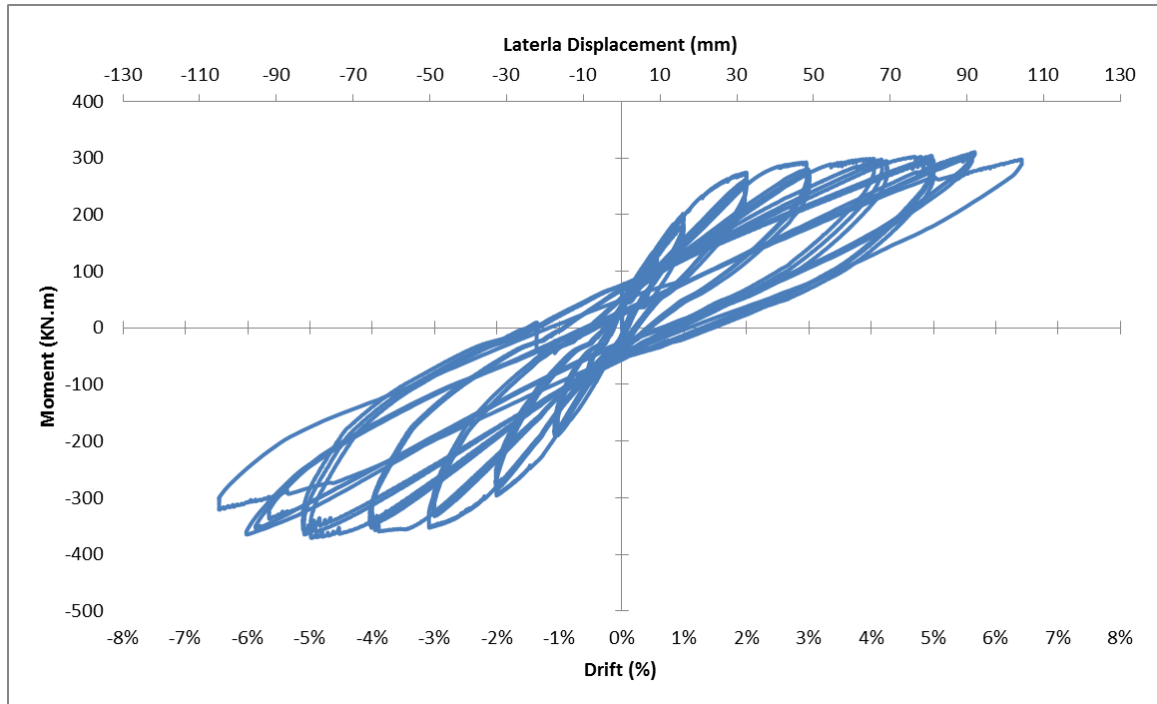


Figure 3-3 Moment-displacement and Force-displacement relationships for circular column

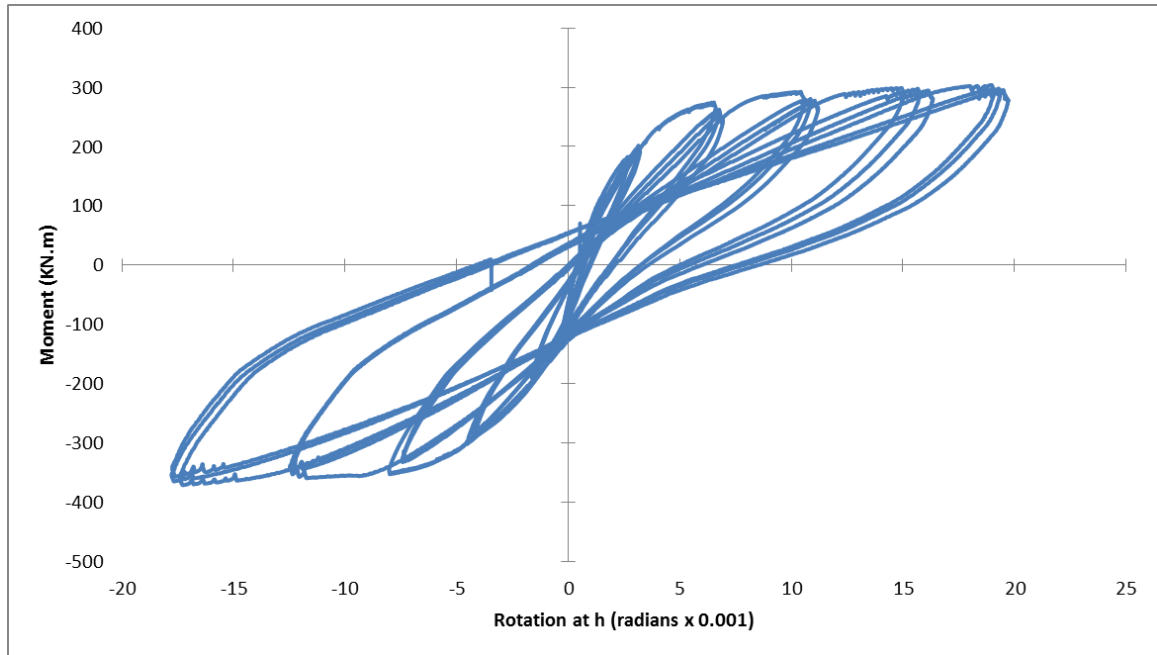


Figure 3-4 Moment-Rotation relationship for circular column

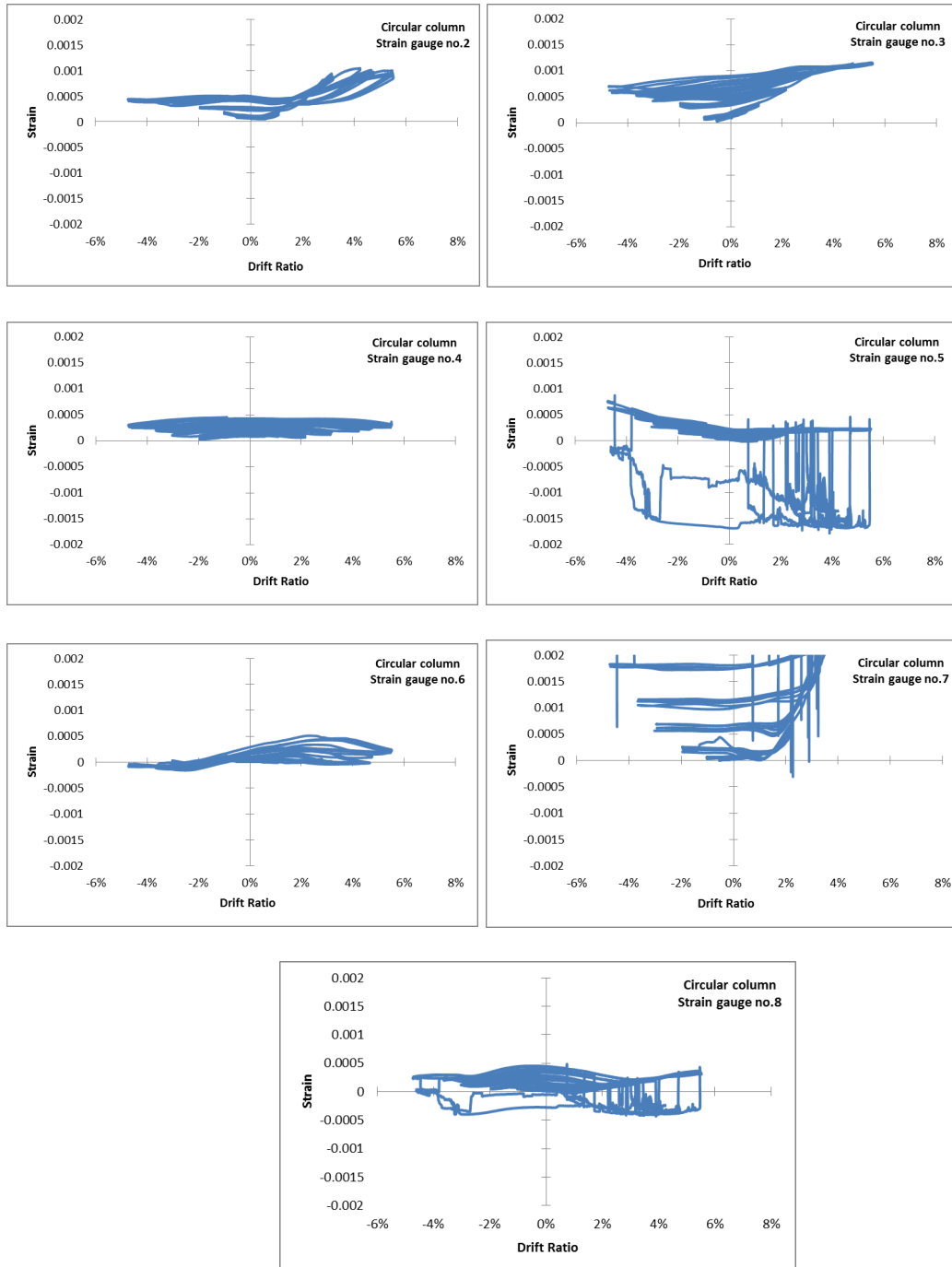


Figure 3-5 Transverse reinforcement steel strains for circular column

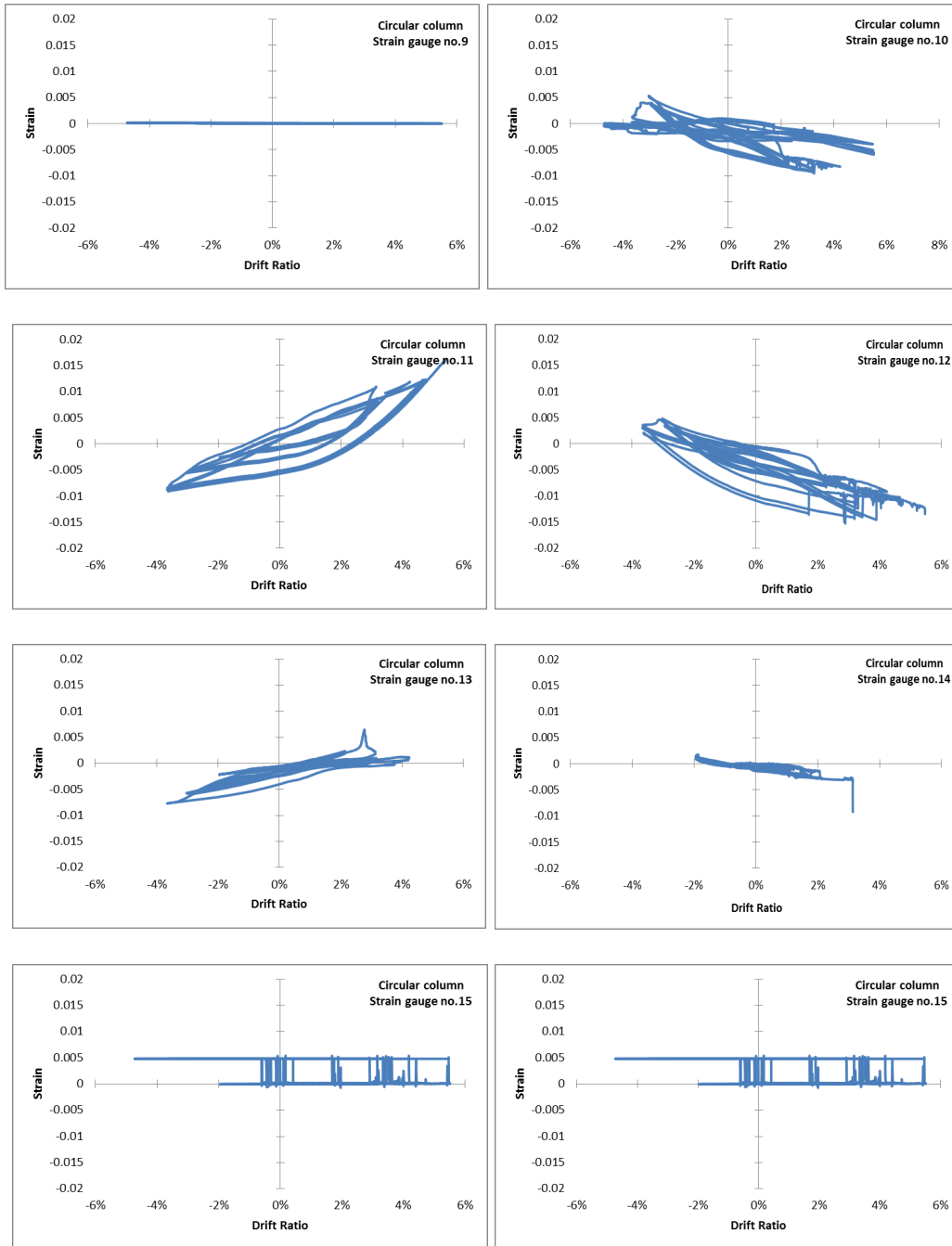


Figure 3-6 Longitudinal reinforcement steel strains for circular column

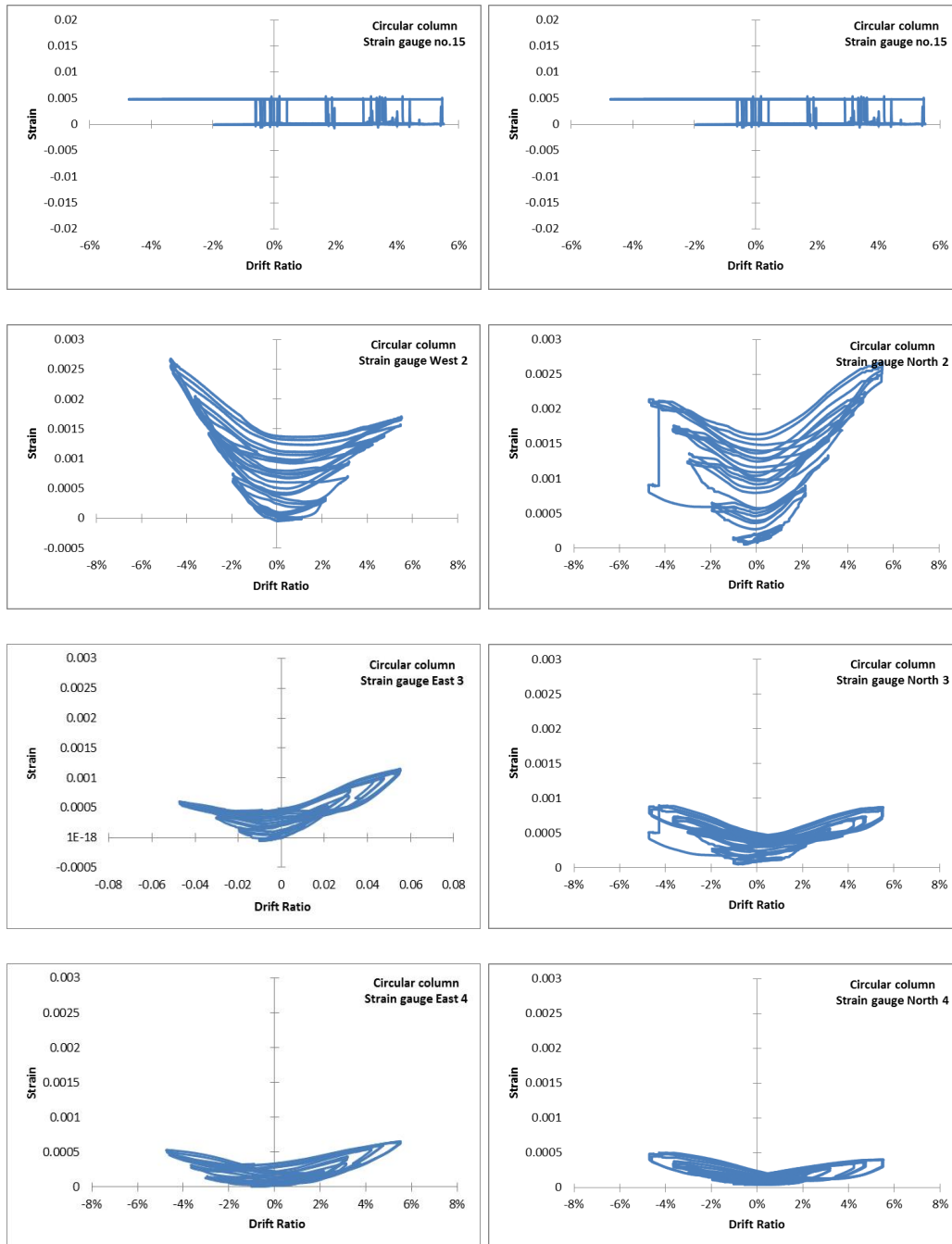


Figure 3-7 High strength steel strap strains for circular column

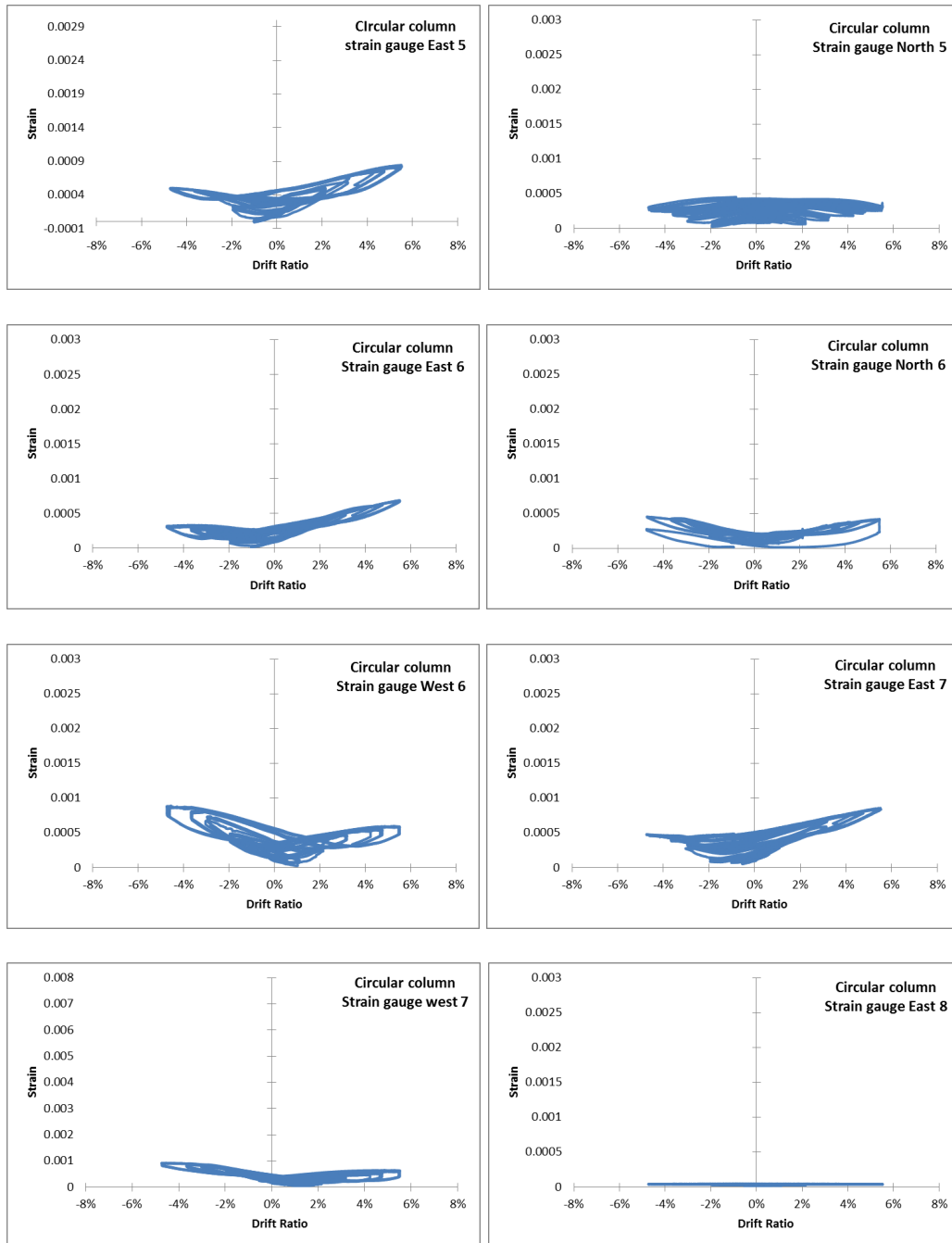


Figure 3-8 High strength steel strap strains for circular column

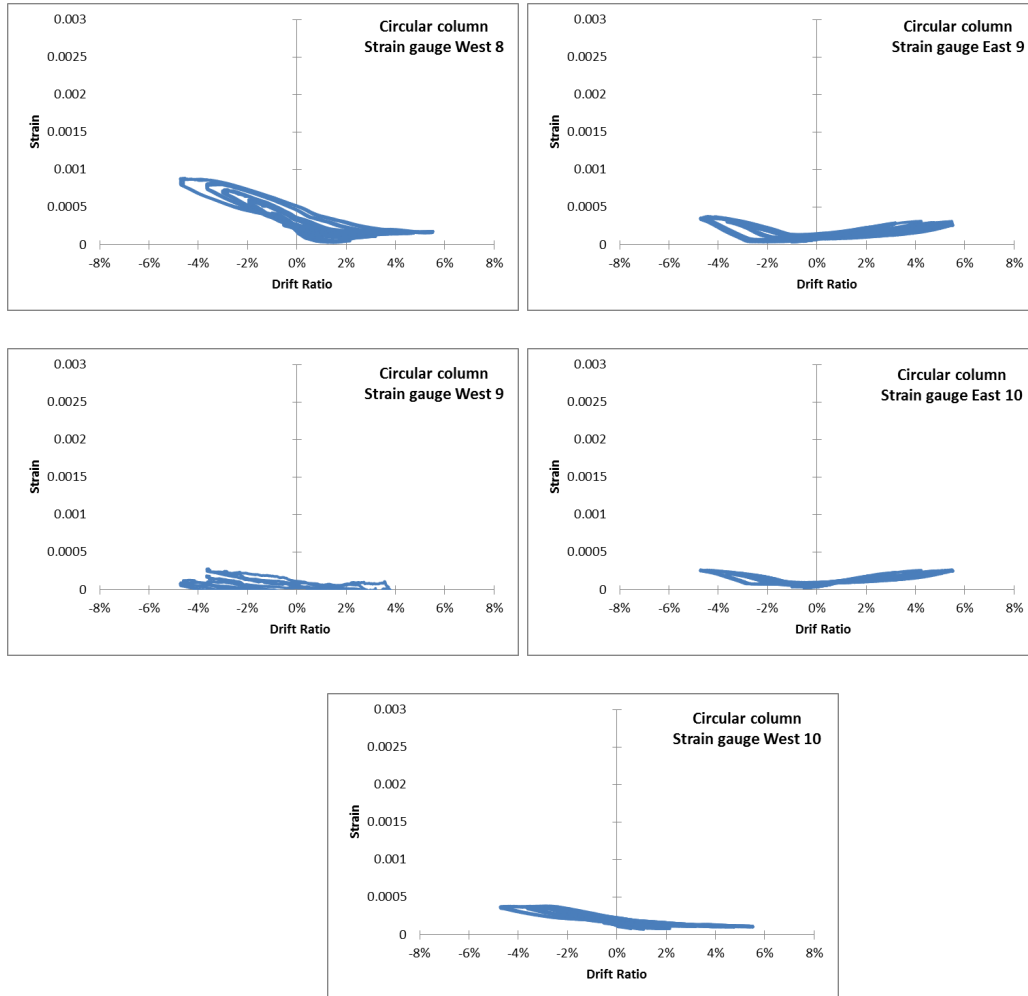


Figure 3-9 High strength steel strap strains for circular column

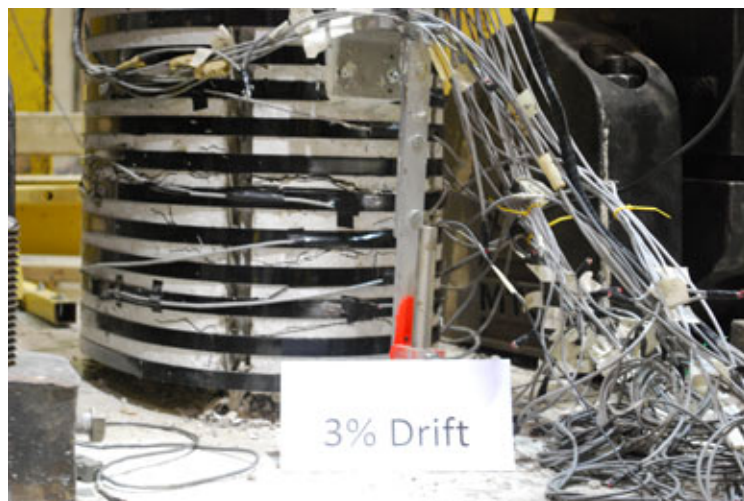
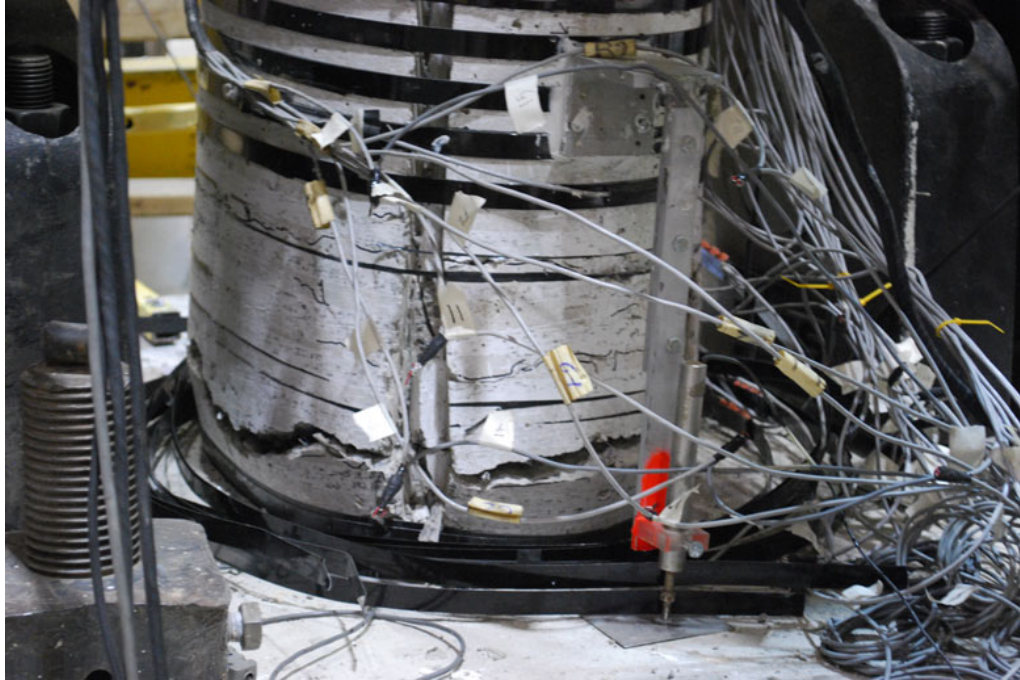


Figure 3-10 Behaviour of rectangular column



Failure of the column at first cycle of 4.5% drifts



Figure 3-11 Behaviour of rectangular column

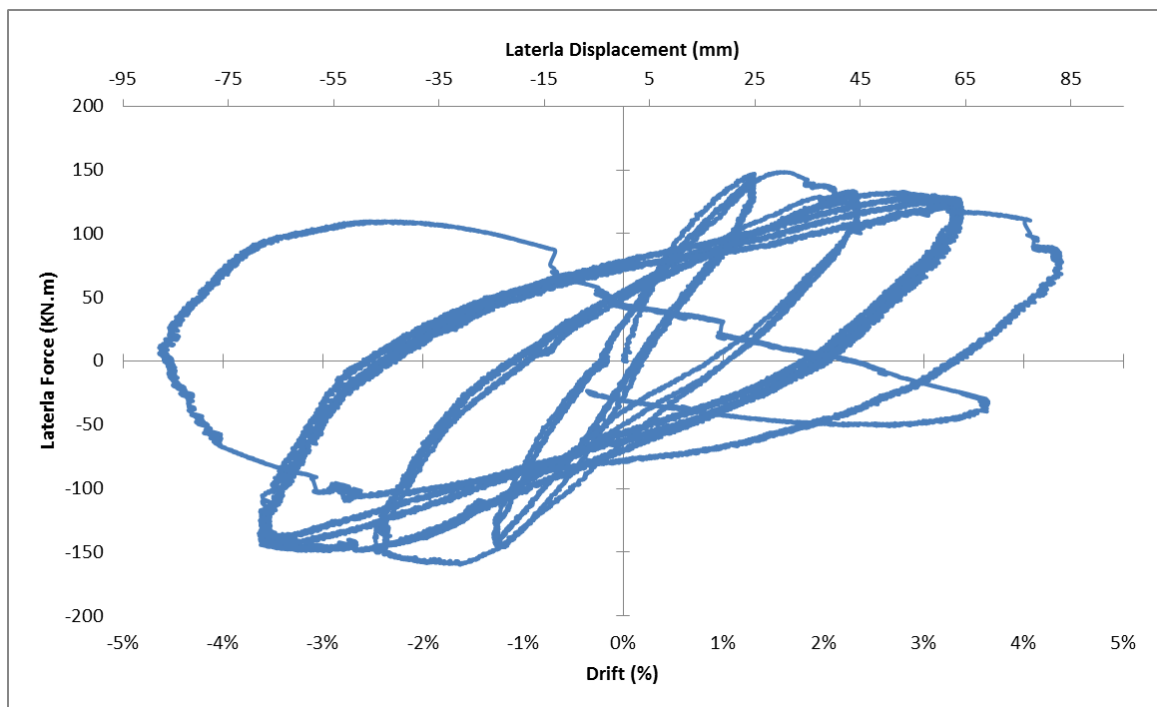
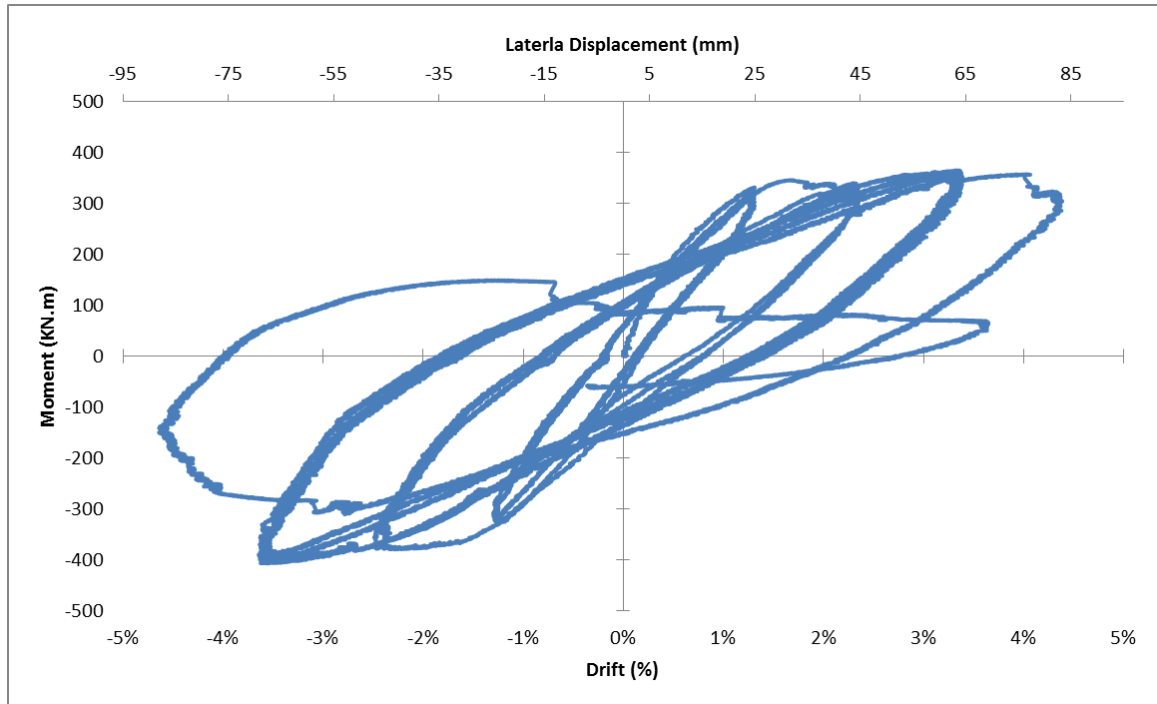


Figure 3-12 Moment-displacement and Force-displacement relationship for rectangular column

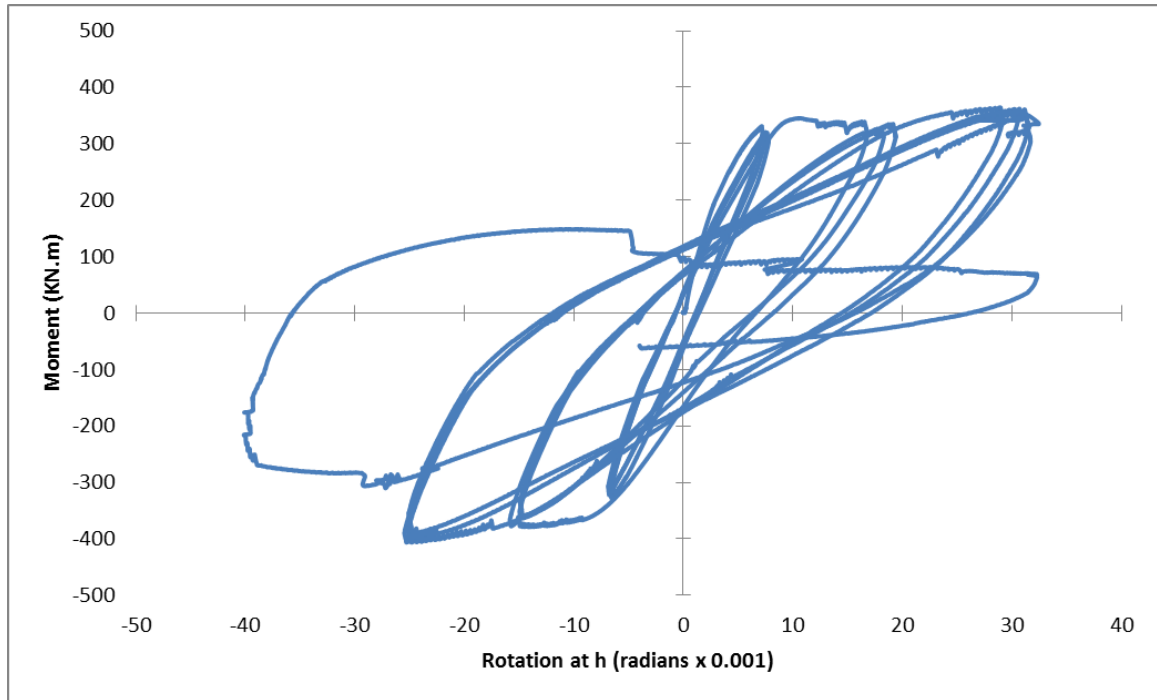


Figure 3-13 Moment-rotation relationship for rectangular column

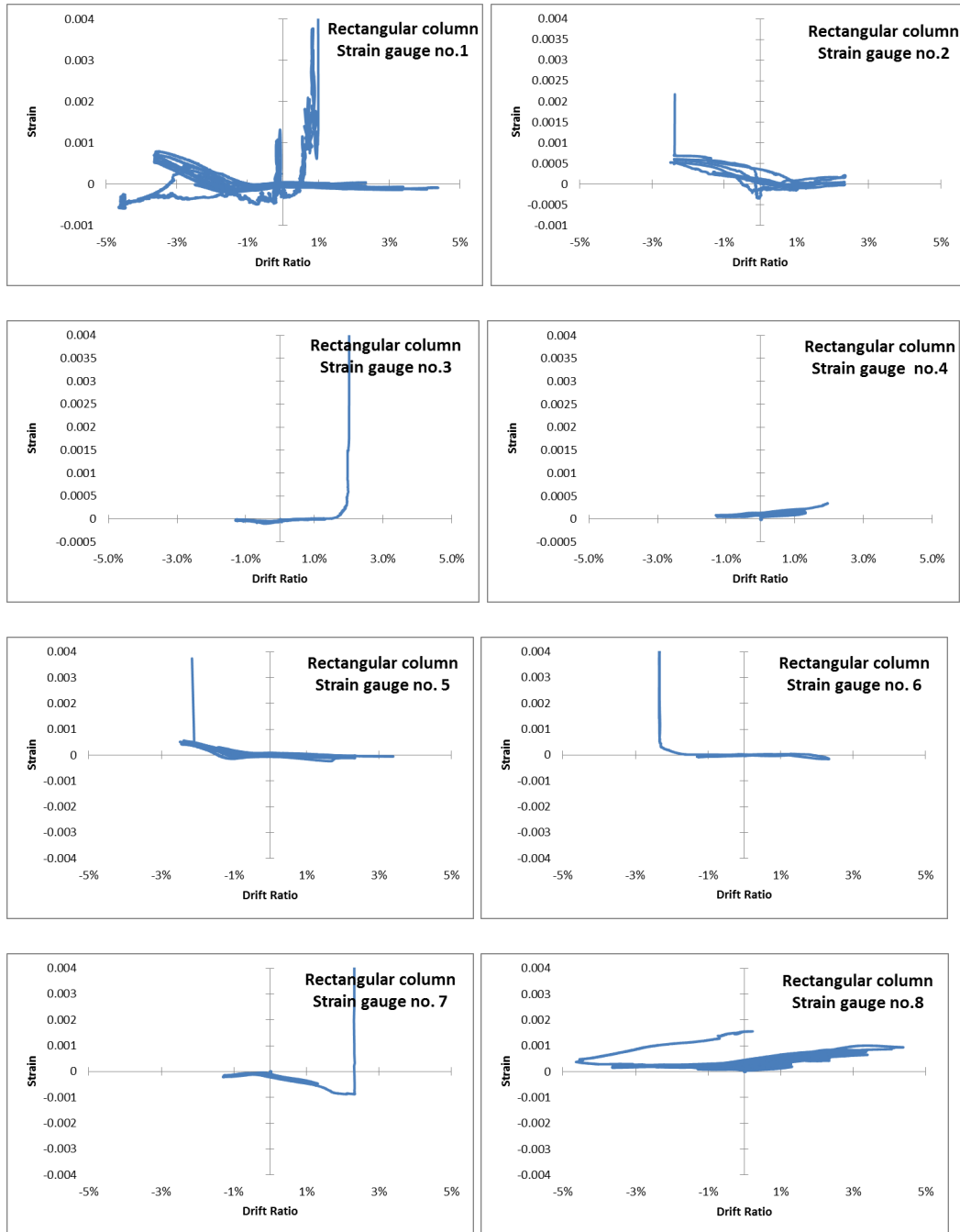


Figure 3-14 Transverse reinforcement steel strains for rectangular column

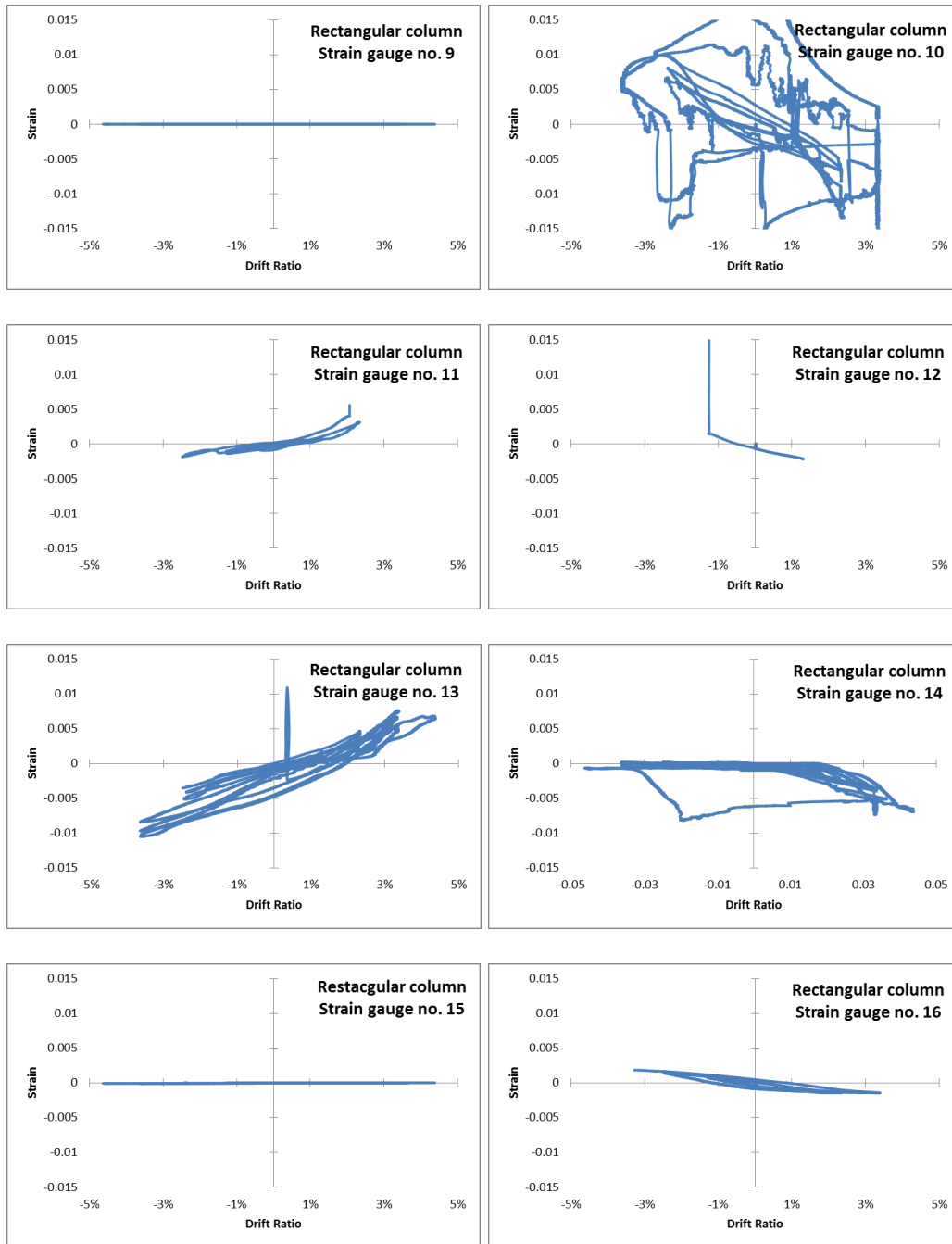


Figure 3-15 Longitudinal reinforcement steel strains for rectangular column

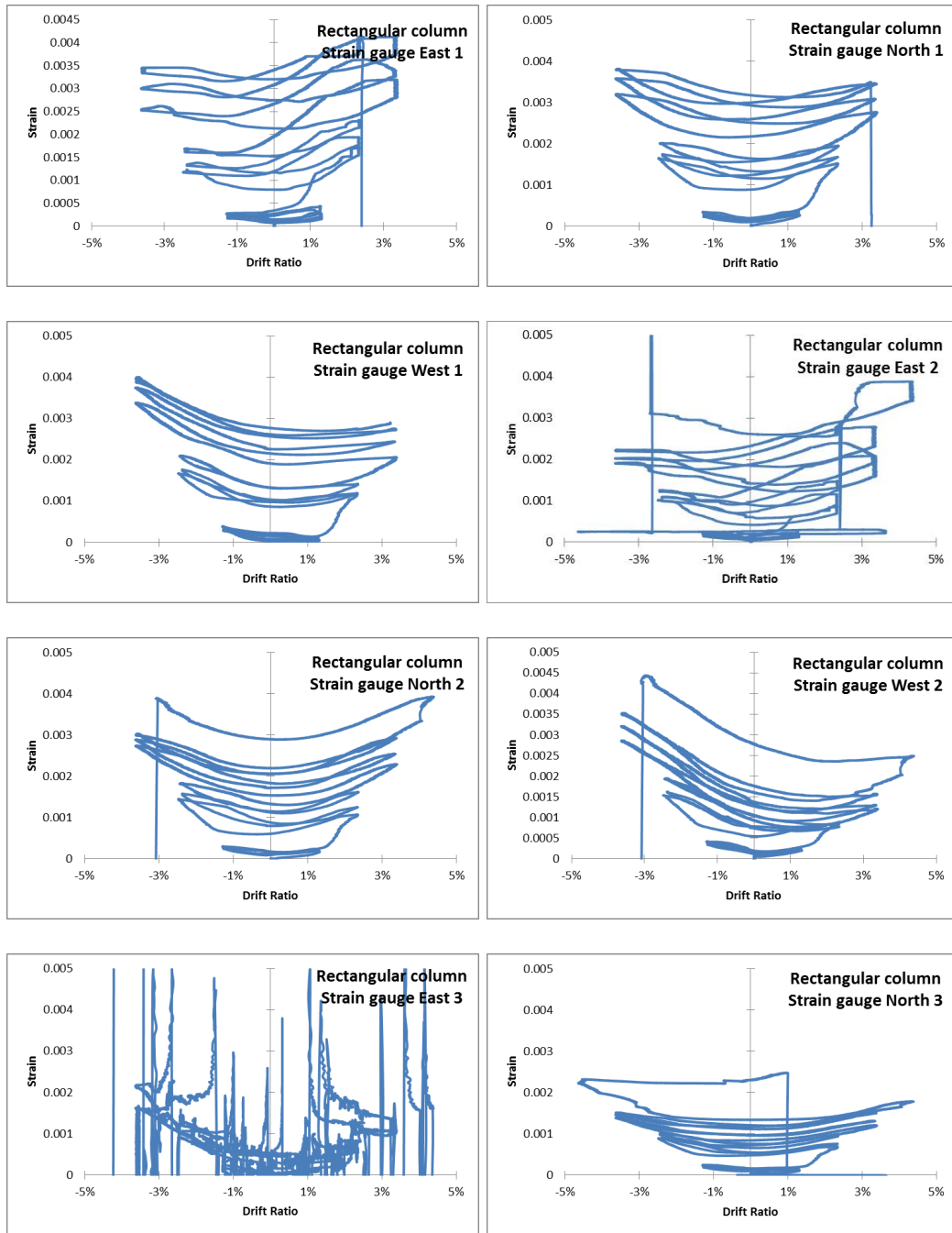


Figure 3-16 High strength steel strap strains for rectangular column

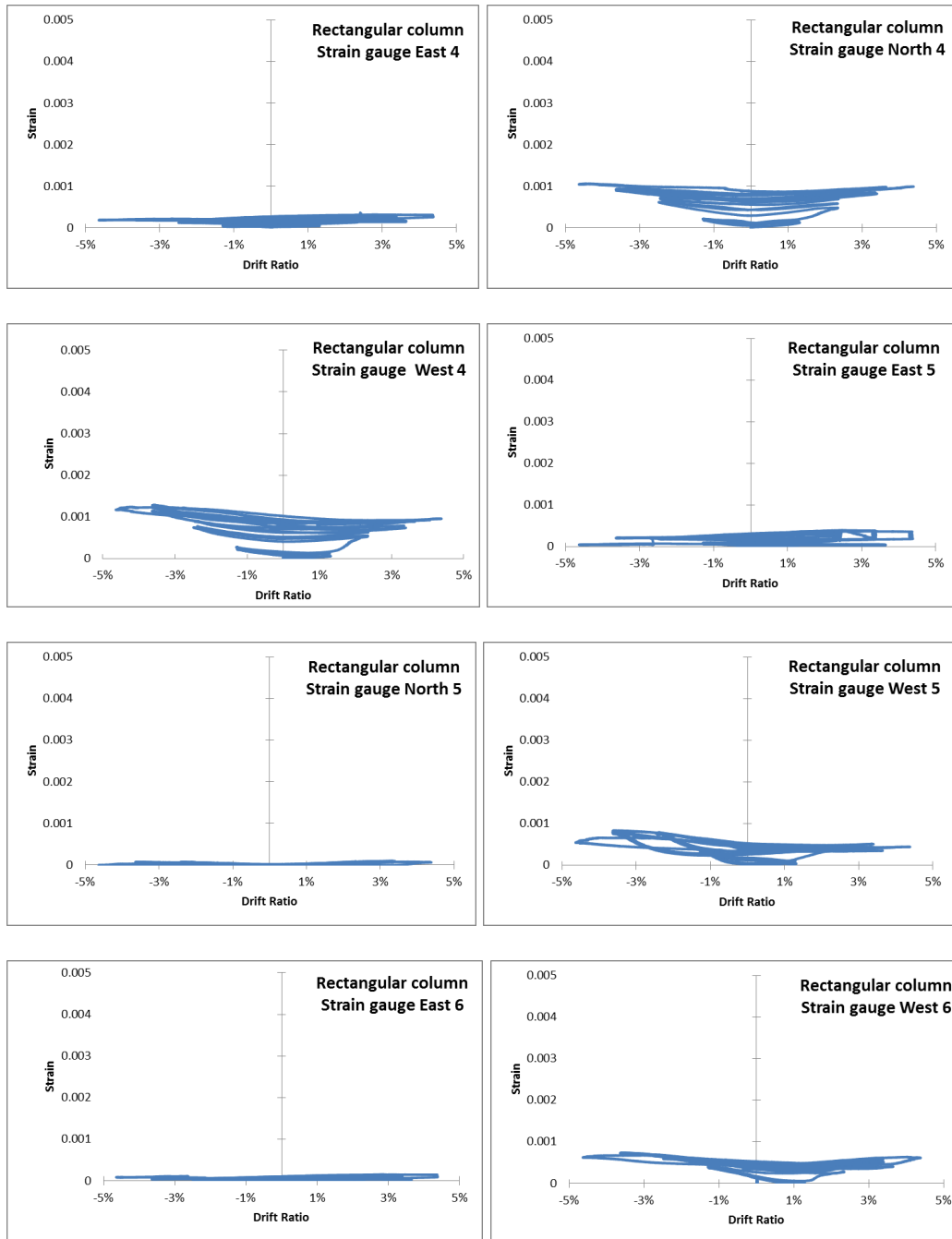


Figure 3-17 High strength steel strap strains for rectangular column

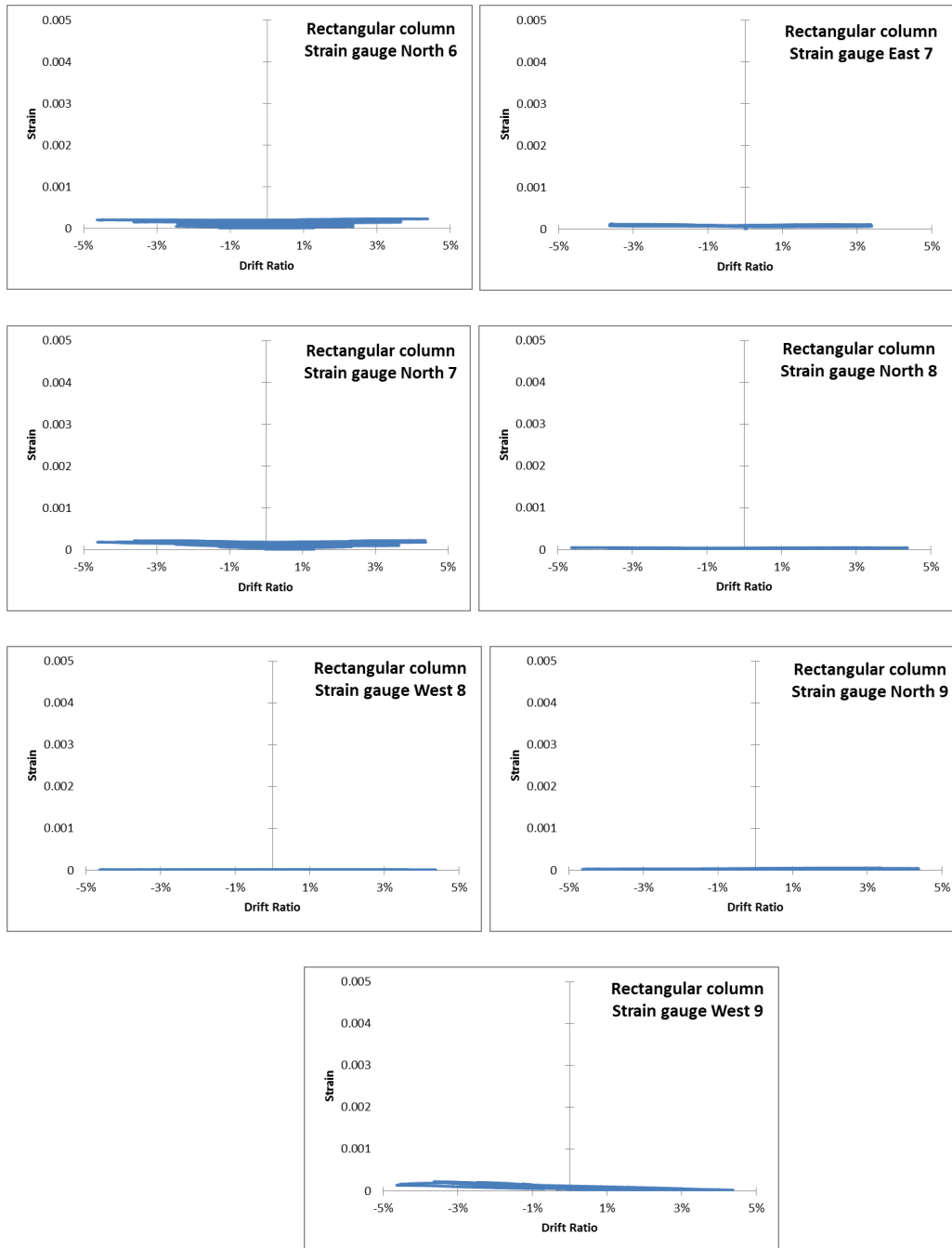
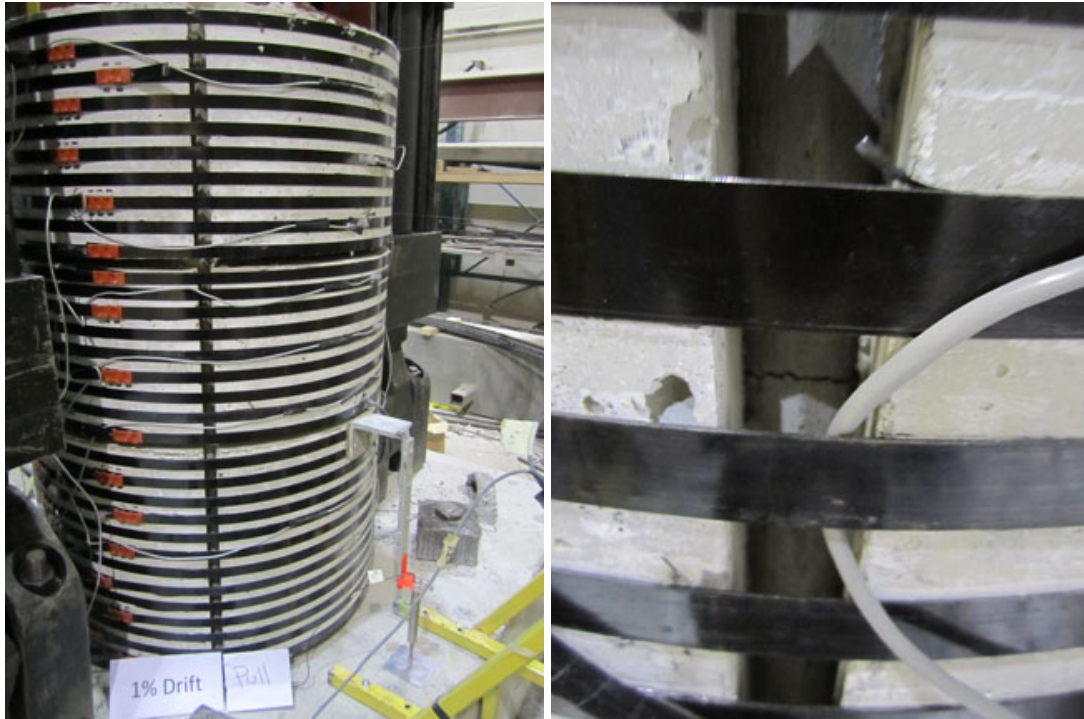


Figure 3-18 High strength steel strap strains for rectangular column



Left: Square column after 1% drift, Right: Observed crack on the main column after 2% drift



Delamination of main column at column-footing location after 3% drift

Figure 3-19



Cracks of the concrete raiser units after 4% drift



Expansion of cracks on the concrete raiser units after 5% drift

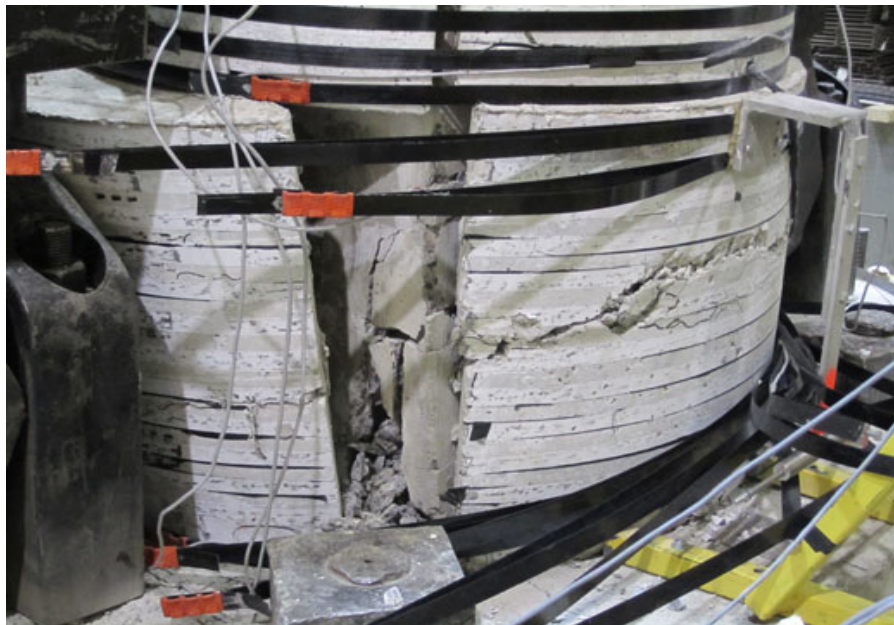
Figure 3-20



Figure 3-21 Rupture of 4th strap and development of new cracks on the concrete raiser units after 6% drift



Rupture of 6 strap after first push at 7% drift



Rupture of all the straps on the first row of concrete raiser units after first pull at 7% drift

Figure 3-22



Failure of the column at second push at 7% drift



Figure 3-23 Behavior of square column

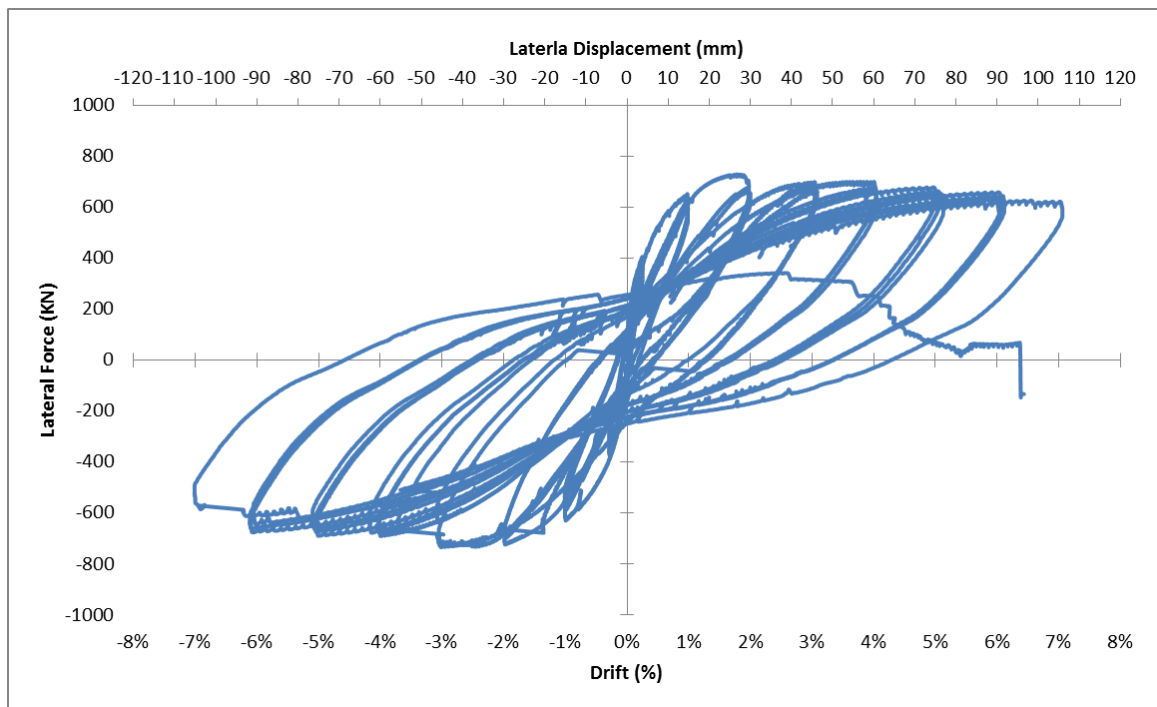
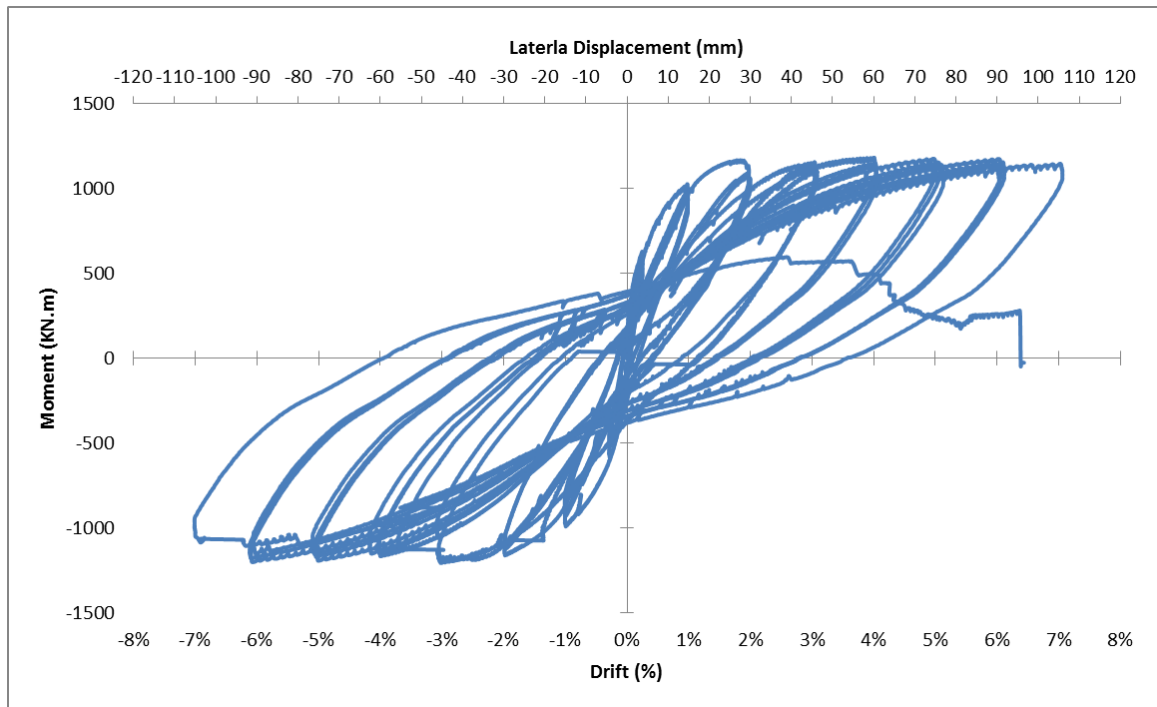


Figure 3-24 Moment-displacement and Force-displacement relationships for square column

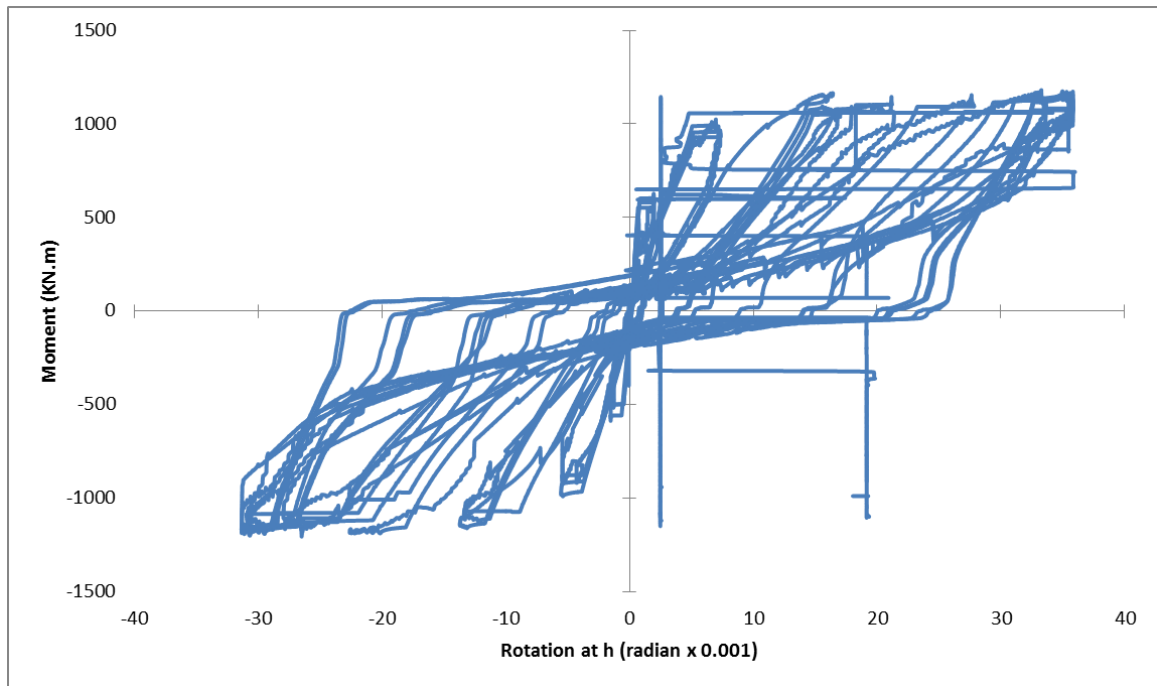


Figure 3-25 Moment-rotation relationship for square column

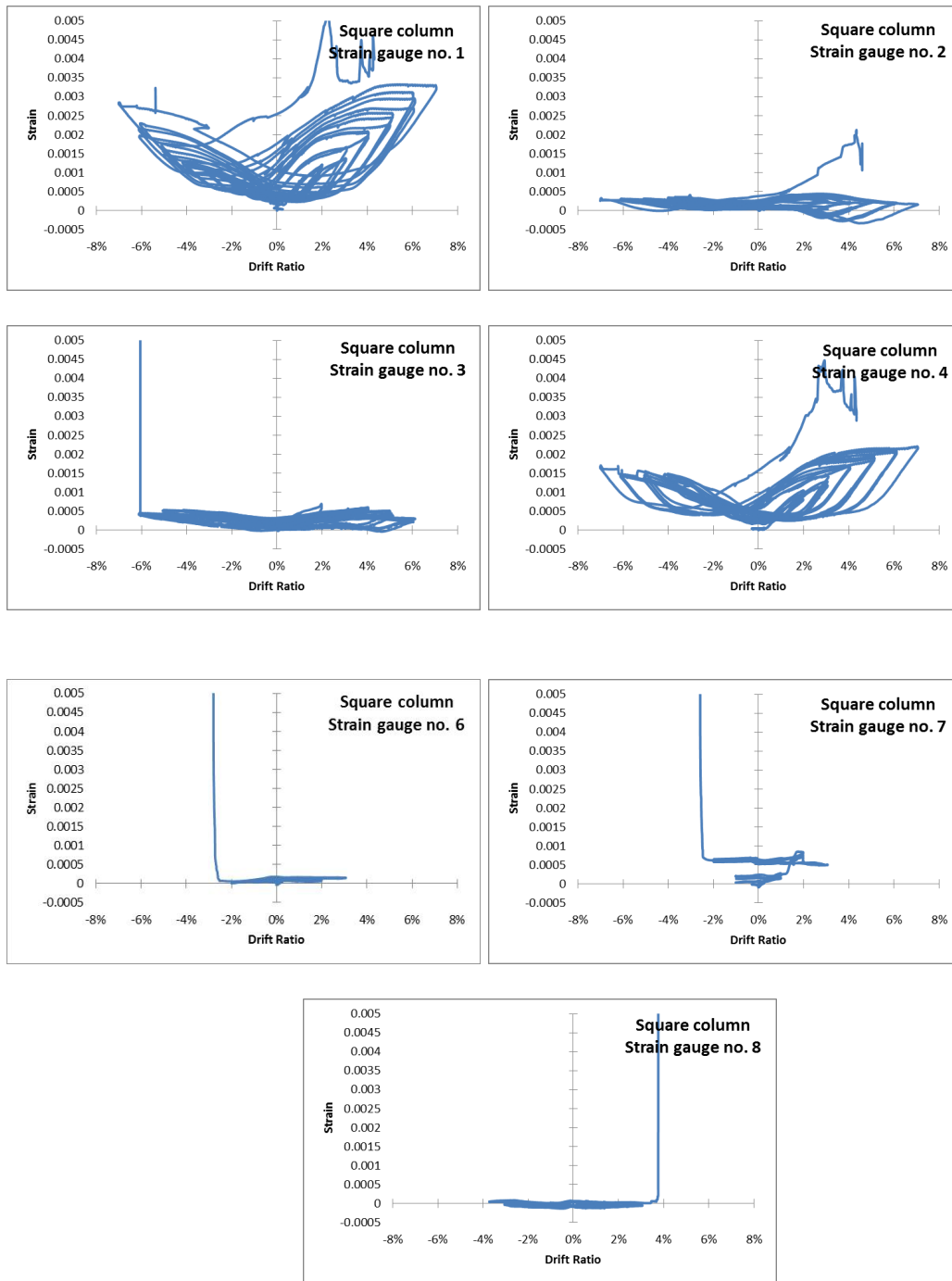


Figure 3-26 Transverse reinforcement steel strains for square column

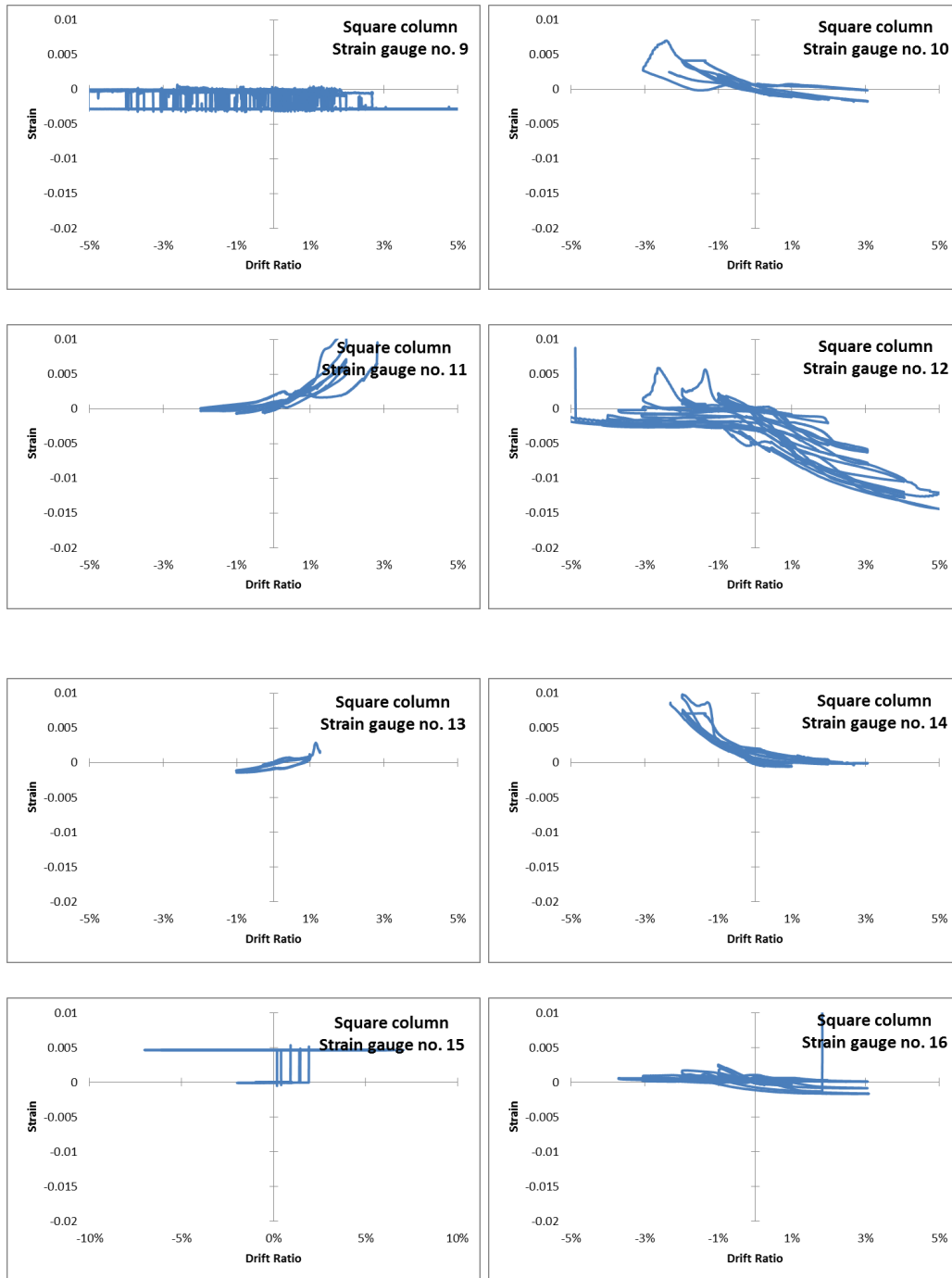


Figure 3-27 Longitudinal reinforcement steel strains for square column

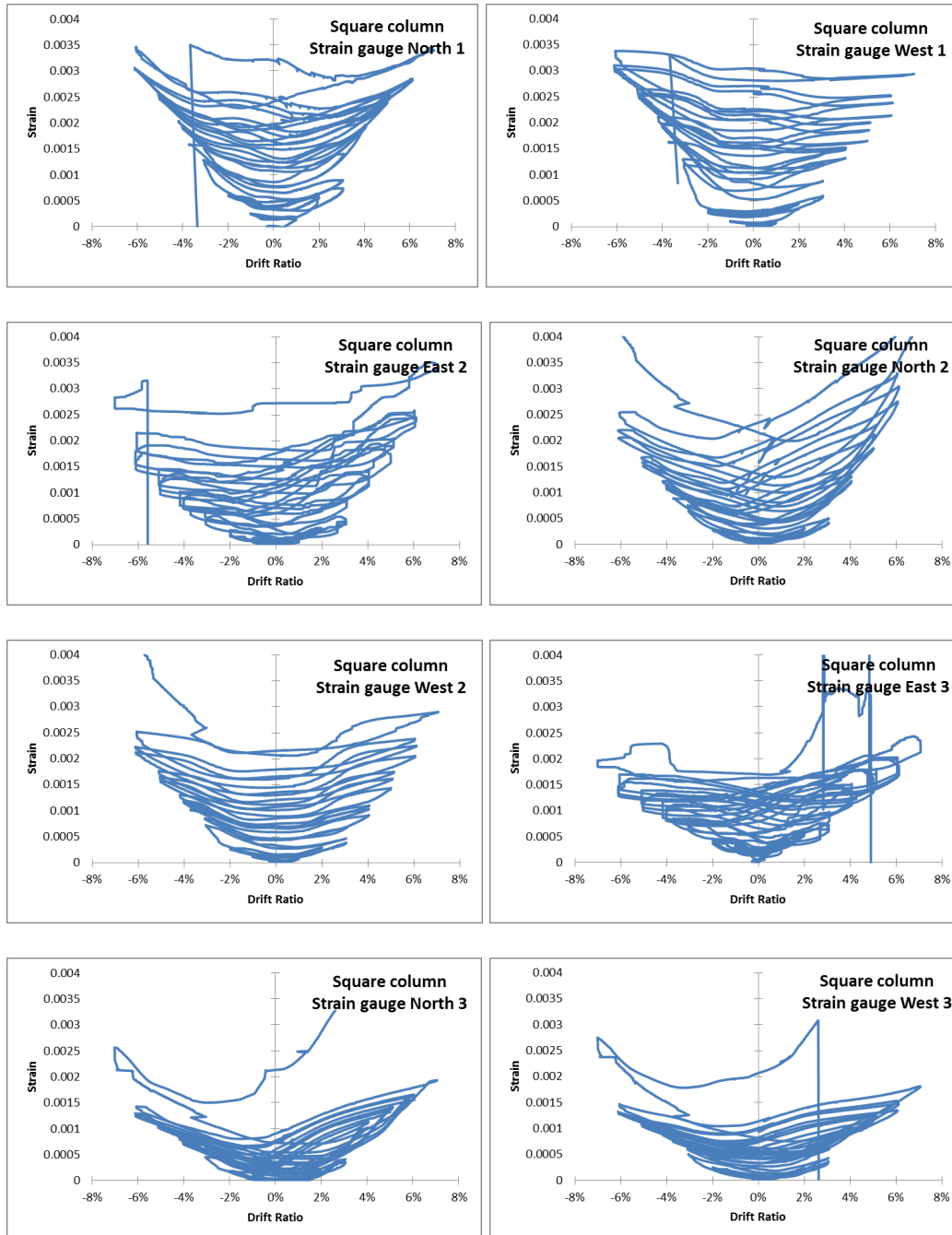


Figure 3-28 High strength steel straps strains for square column

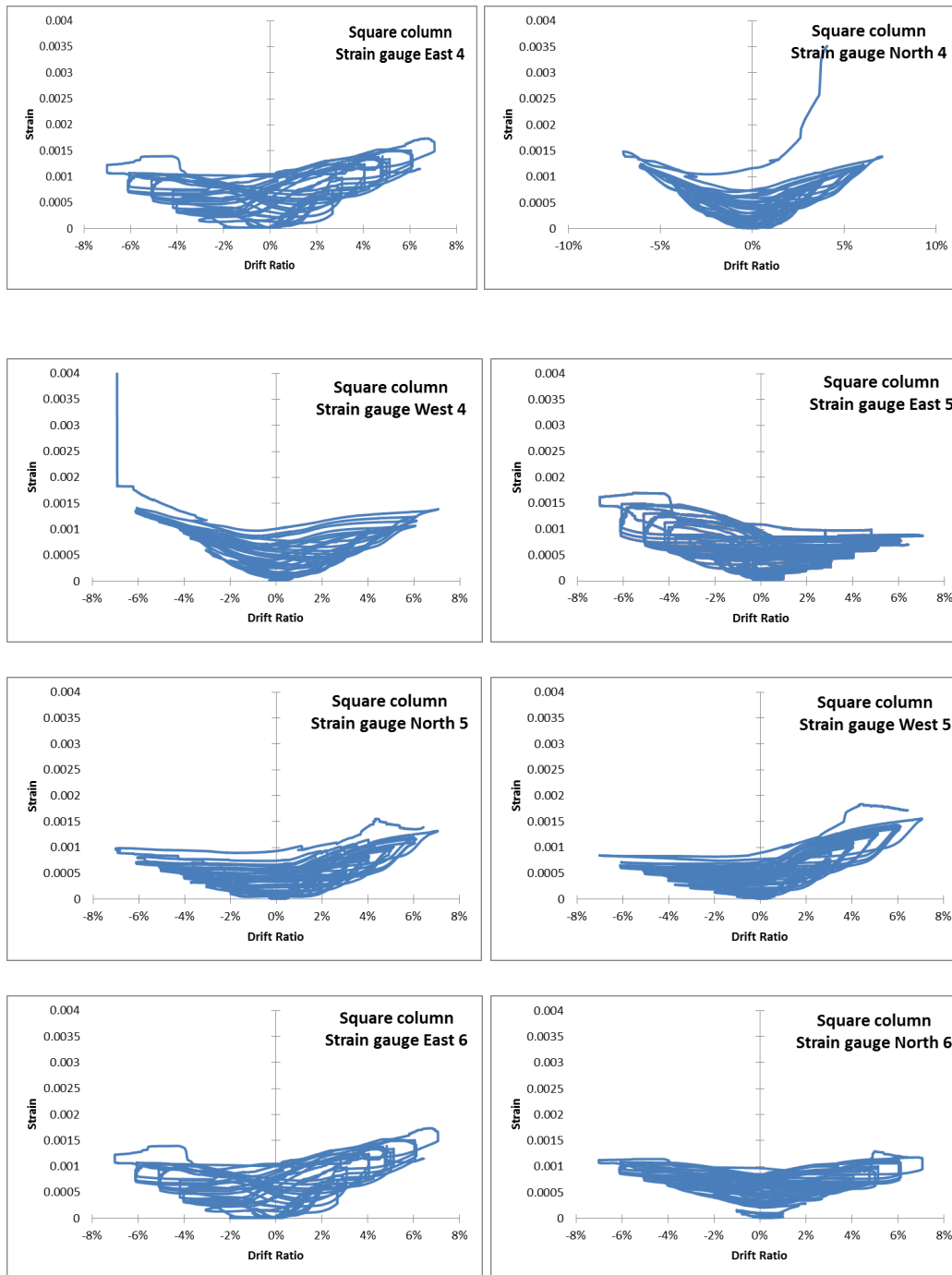


Figure 3-29 High strength steel straps strains for square column

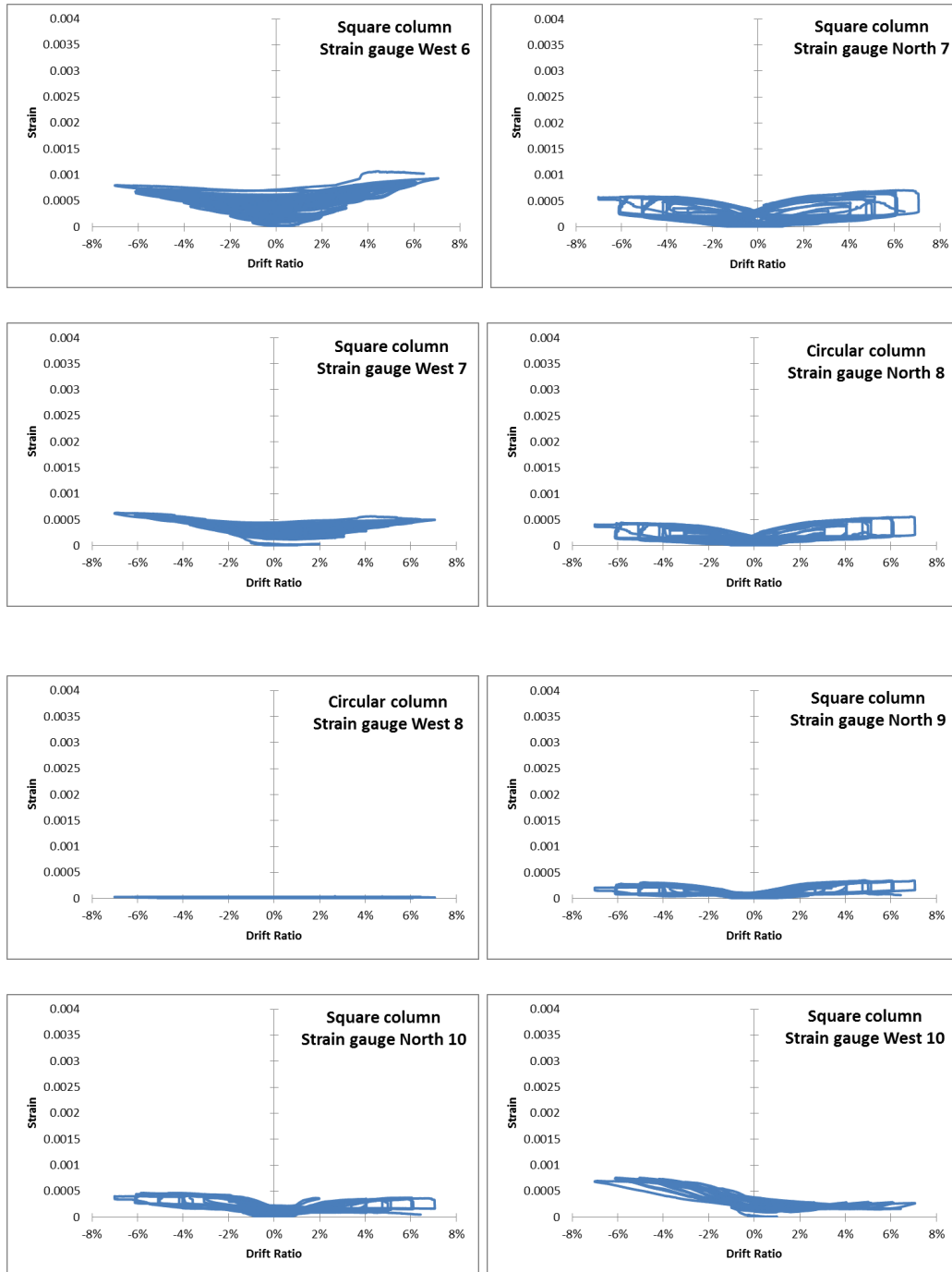


Figure 3-30 High strength steel straps strains for square column

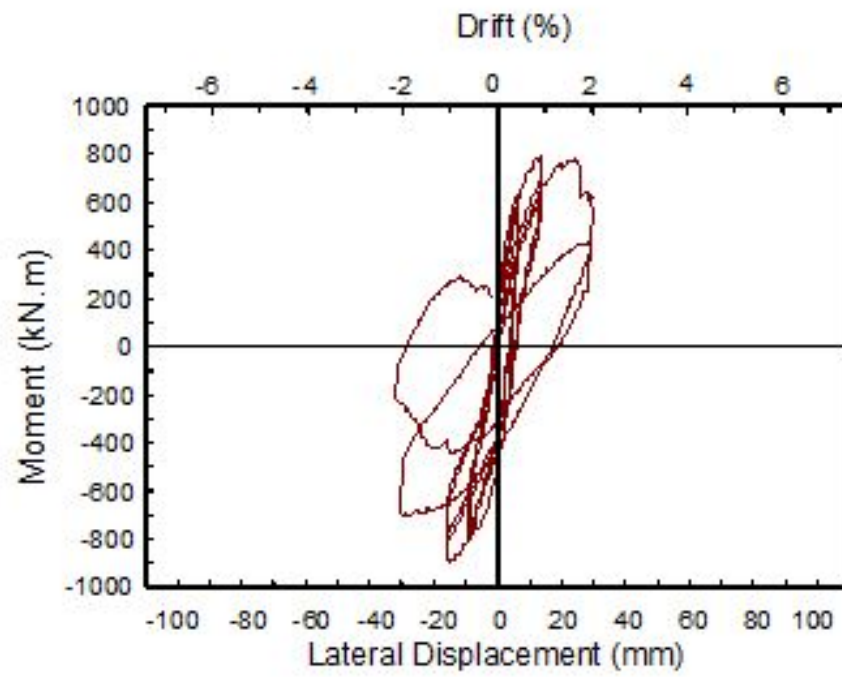


Figure 3-31 Moment-displacement relationship of non-retrofitted square column, tested by Yalcin (1997)

Chapter 4: Analytical Investigation

4.1 Introduction

Analyses of the three column specimens tested were conducted to assess the significance of the retrofit methodology employed, and to develop a retrofit design procedure. The latter also involved the verification of the applicability of analyses techniques commonly used for structural design to the type of retrofitting implemented. This necessitated sectional moment-curvature analysis with due considerations given to concrete confinement and member analysis in flexure for establishing moment-lateral displacement relationships under monotonically increasing inelastic flexural displacements. The analysis was extended to include shear capacity predictions, as one of the columns was shear-critical, and also to ensure that flexural retrofitting would not trigger brittle shear behaviour. The details of the analyses conducted are presented in this chapter.

4.2 Flexural Moment-Deflection Behavior of Columns

Inelastic deformations in reinforced concrete columns consist of components caused by flexure, shear, and anchorage slip (Saatcioglu and Ozcebe 1989). Most of the deformations of members of normal proportions arise from strains associated with flexure (Park and Paulay 1974). Study of inelastic flexural behaviour of reinforced concrete members starts with sectional analysis. Sectional analysis of reinforced concrete columns is discussed in the following section. Shear deformations become considerable in concrete columns that are under-designed in shear. For flexure-dominant columns, or

short columns with high shear force capacity, shear deformation may be neglected. Inelastic deformations caused by anchorage slip result from the slippage and/or extension of longitudinal reinforcement in the attached member (column footing in the case of first-storey buildings or bridge columns). In building columns that have high levels of axial compression, the anchorage slip deformations are usually small as compared to bridge columns that have low levels of axial compression, especially prior to the strain hardening of reinforcement.

A typical flexural analysis of a column starts with a column sectional analysis, incorporating proper material models for confined and unconfined concrete, and reinforcing steel. Sectional moment-curvature analysis usually referred to as “plane-section analysis” leads to the force-deflection relationship, which when extended to the inelastic range of deformations also provides ductility and inelastic drift capacities. The stiffness and peak lateral load capacity of a column can be calculated through this method. The following sections explain the different steps involved in a typical concrete column analysis.

4.2.1 Moment-curvature Relationship

Calculation of moment-curvature relationship is the first step towards establishing force-deflection behaviour of a concrete member in flexure. Moment-curvature diagram of a concrete column reveals its flexural resistance at different stages of loading (cracking, yielding and post yield loading) and corresponding curvatures, up to the maximum moment capacity. Member behaviour, including elastic and plastic rotations and corresponding horizontal deflections can be calculated from curvatures. The basic assumption in the calculation of theoretical moment-curvature relationship is that “plane

sections before bending remain plane after bending.” The stress-strain curves for concrete and steel should be established before such an analysis could be undertaken. Several models were recommended by researchers to idealize the stress-strain diagrams for unconfined and confined concrete, as well as reinforcing steel.

The moment-curvature relationship is computed by considering different strains in the extreme fibre of the column section, and calculating corresponding moments and curvatures from the equilibrium of internal forces. An idealized moment-curvature relationship can be established as a trilinear relationship with a linear line segment up to the initial cracking of concrete, followed by another linear segment up to the yielding of tension reinforcement, and a linear segment up to crushing of compression concrete or until reaching the assumed material limit. The calculation of moment-curvature relationship may be time consuming unless done by computer software. The computer software, RC-Section developed at the University of Ottawa, was used for the calculation of moment-curvature diagrams of the three retrofitted columns considered in the current investigation. Detailed explanation of the software and the analysis results are presented in the following sections.

4.2.2 Confined Concrete Model

Stress-strain relationship of confined concrete is substantially different than that of unconfined plain concrete. Analytical models were recommended by researchers to predict the stress-strain characteristics of confined concrete. The confined concrete model proposed by Saatcioglu and Razvi (1992) has been adopted for use in the current research program. The proposed model provides was shown to produce good predictions of observed behaviour for various cross-sectional shapes and reinforcement arrangements. It

is relatively simple to use for members having circular, square, and rectangular sections with different types of lateral reinforcement, providing lateral confinement pressure. It is especially useful for the current research project, as unlike other models, it can introduce the effects of active, as well as passive lateral confinement pressures.

The confined concrete model is established based on equivalent uniform confinement pressure produced by the reinforcement cage or any external lateral pressure. Transverse strains caused by lateral pressure counteract the tendency of material to expand laterally, and result in increased strength (Saatcioglu and Razvi 1992). The following expressions describe the triaxial strength of confined concrete.

$$f'_{cc} = f'_{co} + k_1 f_l \quad (4.1)$$

$$k_1 = 6.7(f_l)^{-0.17} \quad (4.2)$$

Where f'_{cc} and f'_{co} are confined and unconfined strengths of concrete, and f_l is the lateral confining pressure in MPa. Equation (4.2) shows the variation of coefficient k_1 with lateral pressure, which was obtained from a regression analysis of experimental data. The relationship in equation (4.1) is valid for uniform lateral pressures. The lateral pressure provided by closely spaced circular hoops can be considered as uniform pressure. The following expression can be employed to compute the lateral uniform pressure provided by closely spaced circular hoops, as illustrated in Figure 4.1.

$$f_l = \frac{2A_s f_{yt}}{b_c s} \quad (4.3)$$

The confining pressure provided by a square or a rectangular hoop is not easy to calculate. As it is shown in Figure 4.2, rectilinear hoops develop high restraining forces at the corners and nodal points where they have hooks engaging in longitudinal bars; and

low restraining force in locations away from the nodes. In order to use the expression given in Eq. (4.1), an equivalent uniform pressure, with the same effect of actual non-uniform pressure, should be considered. The following expressions describe the computation of equivalent uniform pressure and the strength of confined concrete for rectilinear hoops.

$$f'_{cc} = f'_{co} + k_1 f_{le} \quad (4.4)$$

$$f_{le} = k_2 f_l \quad (4.5)$$

$$f_l = \frac{\sum A_s f_y \sin \alpha}{s b_c} \quad (4.6)$$

$$k_1 = 6.7(f_{le})^{-0.17} \quad (4.7)$$

Where f_{le} and f_l are equivalent uniform pressure and average pressure provided by rectilinear hoops, respectively. Coefficient k_2 , expressed in Eq. (4.8) enables the conversion from average confinement pressure f_l to equivalent uniform pressure f_{le} .

$$k_2 = 0.26 \sqrt{\left(\frac{b_c}{s}\right) \left(\frac{b_c}{s_l}\right) \left(\frac{1}{f_l}\right)} \leq 1.0 \quad (4.8)$$

Where b_c is the concrete core dimension, measured center to center of perimeter hoops, s is the spacing of transverse reinforcement, and s_l is the spacing of laterally supported longitudinal reinforcement.

Rectangular columns may provide different confinement pressures in two orthogonal directions. Based on experimental data, the confinement pressure on the short and long side of a column can be correlated to column cross-sectional dimensions. The following expression describes this correlation.

$$f_{le} = \frac{f_{lex} + f_{ley} b_{cy}}{b_{cx} + b_{cy}} \quad (4.9)$$

Figure 4.4 illustrates the parameters used in the above equation.

The proposed model for confined concrete reduces to Hognestad's model (1951) when the effect of confinement is negligible. Figure 4.5 shows the stress-strain relationship of unconfined and confined concretes. The ascending and descending branch of unconfined concrete stress-strain diagram is formulated through the following expressions.

$$f_c = f'_{co} \left[2 \frac{\epsilon_c}{\epsilon_{01}} - \left(\frac{\epsilon_c}{\epsilon_{01}} \right)^2 \right] \quad \epsilon_c \leq \epsilon_{01} \quad (4.10)$$

$$f_c = f'_{co} - (\epsilon_c - \epsilon_{01}) \left(\frac{0.15 f'_{co}}{\epsilon_{085} - \epsilon_{01}} \right) \quad \epsilon_{01} < \epsilon_c \leq \epsilon_{cu} \quad (4.11)$$

Where f'_{co} is the unconfined concrete compressive strength in MPa, ϵ_{01} and ϵ_{085} are concrete strains corresponding to the maximum stress, and 85% of maximum stress, respectively. The descending branch of the diagram continues to the strain corresponding to 20% of the peak capacity.

As it is shown in Figure 4.5, lateral confinement improves concrete strength and ductility. The following equation is recommended for the parabolic ascending branch of confined concrete.

$$f_c = f'_{cc} \left[2 \left(\frac{\varepsilon_c}{\varepsilon_1} \right) - \left(\frac{\varepsilon_c}{\varepsilon_1} \right)^2 \right]^{\frac{1}{(1+2K)}} \leq f'_{cc} \quad (4.12)$$

The relationship between the unconfined and confined concrete strains at peak strength is described by the following expressions. This is also previously used by Balmer (1949), and Mander et al. (1988).

$$\varepsilon_1 = \varepsilon_{01}(1 + 5K) \quad (4.13)$$

$$K = \frac{k_1 f_{le}}{f'_{co}} \quad (4.14)$$

The strain corresponding to the peak stress of unconfined concrete (ε_{01}), should be determined experimentally under the same rate of loading as that for confined concrete. The strain value of 0.002 can be used in the absence of experimental data. Eq. (4.12) is recommended for the ascending branch of confined concrete stress-strain relationship. The amount of transverse reinforcement plays a significant role on the slope of the descending branch. The strain at 85% of strength can be computed by the following expressions.

$$\varepsilon_{85} = 260\rho\varepsilon_1 + \varepsilon_{085} \quad (4.15)$$

$$\rho = \frac{\sum A_s}{s(b_{cx} + b_{cy})} \quad (4.16)$$

Where $\sum A_s$ is the total transverse steel area in both orthogonal directions. ε_{085} and ε_{85} are unconfined and confined concrete strains at 85% of strength, respectively. The

unconfined concrete strain at 85% strength ($\epsilon_{0.85}$) should be determined experimentally under the same rate of loading as that for confined concrete. The strain value of 0.0038 can be used in the absence of experimental data. Eq. (4.14) is recommended for the slope calculation of linear descending branch of confined concrete stress-strain relationship.

4.2.3 RC-Section Software

The computer software RC-Section, developed at University of Ottawa, was used to calculate the moment-curvature relationship of retrofitted and non-retrofitted columns. The software is capable of drawing moment-curvature relationships of concrete columns with different cross sections based on specified stress-strain relationships of steel and concrete. Various material models including that proposed by Saatcioglu and Razvi (1992) are provided in the software. It can handle sections with confined core and unconfined cover concrete. The steel model can be specified by the user by defining points on the diagram. The software requires the specification of section geometry and the reinforcement arrangement. In addition to the moment-curvature analysis, the software can calculate unfactored and factored flexural capacities of sections, as well as the computation of column interaction diagrams.

4.3 Flexural Deformations of Concrete Columns

The rotation of a column can be calculated by integrating curvatures along its height. By knowing the moment-curvature relationship and the distribution of bending moment, the curvature diagram along the column height can be obtained. The area under the curvature diagram between two sections gives the rotation between these sections. The deflection amount between two sections is also the moment of the area under the

curvature diagram between these sections. The following expressions describe the rotation and deflection between any two sections A and B.

$$\theta_{AB} = \int_A^B \varphi dx \quad (4.16)$$

$$\Delta_{AB} = \int_A^B x \varphi dx \quad (4.17)$$

Eqs. (4.16) and (4.17) are based on moment-area theorems and they can be used for elastic and plastic curvatures.

4.4 Shear Capacity of Concrete Columns

Many different models were recommended by previous researchers to describe the transfer of shear forces in a reinforced concrete member. The shear model presented in this section is recommended by Seible and Priestly (1995). The proposed model describes the shear capacity as a combination of three components. These components are concrete contribution V_c , transverse reinforcing steel contribution V_s , and axial load contribution V_p . Thus;

$$V_n = V_c + V_s + V_p \quad (4.18)$$

The concrete contribution V_c is a function of aggregate interlock. It decreases with increasing crack width and flexural ductility. In the plastic hinge region of a concrete member, the displacement ductility level μ_Δ has a direct effect on V_c . Therefore, the concrete contributions to shear should be investigated separately within the plastic hinge region and outside the plastic hinge region. The following expressions are recommended for calculation of V_c .

i) Within the plastic hinge region:

$$V_c = 0.083k\sqrt{f'_c}A_e \quad (4.19)$$

ii) Outside the plastic hinge region:

$$V_c = 0.25\sqrt{f'_c}A_e \quad (4.20)$$

Where A_e is the effective concrete shear transfer area, and it's equal to 80% of the gross area of column, and k is the concrete strength reduction factor which is defined based on column displacement ductility ratio μ_Δ . Figure 4.6 describes the correlation between k and μ_Δ for design and assessment. Recommended reduction factors for shear design at different ductility levels are as follows.

$$k = 3 \quad \mu_\Delta < 2 \quad (4.21)$$

$$k = 5 - \mu_\Delta \quad 2 \leq \mu_\Delta < 4$$

$$k = 1.5 - \frac{\mu_\Delta}{8} \quad 4 \leq \mu_\Delta < 8$$

$$k = 0.5 \quad \mu_\Delta \geq 8$$

The following expressions are recommended for horizontal reinforcing steel contribution V_s for circular and rectangular columns.

$$V_s = \frac{\pi A_h f_{hy} D'}{2 s} \cot \theta \quad (\text{Circular}) \quad (4.22)$$

$$V_s = \frac{n A_h f_{hy} D'}{s} \cot \theta \quad (\text{Rectangular}) \quad (4.23)$$

Where A_h , is the area of one leg of transverse reinforcement, and n is the number of legs of transverse reinforcement in the loading direction. f_{hy} is the yield capacity of transverse reinforcement, and D' is the core dimension of column in the loading direction. Figure 4.7 shows D' in circular and rectangular columns. θ is the angle of principal compression strut to the column axis which is taken as 30° for bridge columns and 45° for pier walls.

The axial load contribution to the shear is coming from horizontal component of inclined compression strut and it is simply defined by the following expression.

$$V_p = P \tan \alpha \quad (4.24)$$

Where, α is the angle between the inclined compression strut and the neutral axis of column section. Figure 4.8 describes the axial force shear contribution for single bending and double bending columns.

4.5 Comparison of Experimental Results with Analytical Results

Flexural and shear analyses were performed for retrofitted and equivalent as-built columns based on the procedures presented in the previous sections. The confined and unconfined stress-strain properties of concrete were calculated using the Saatcioglu and Razvi confined concrete model, and the moment-curvature relationships for columns were established by using the computer software RC-Section. The flexural behaviour of columns was developed based on their moment-curvature diagrams. The lateral force

corresponding to the nominal flexural capacity of the companion non-retrofitted column and shear force capacities with and without retrofitting, were calculated for each column. The force-displacement relationship up to yield was computed by finding the moment of the area under the curvature distribution along the height of each column. The post-yield region was obtained by computing the ultimate deflection and connecting the yield deflection with ultimate deflection. The ultimate deflection was computed by using a hinging length equal to the column cross-sectional dimension in the direction of loading (h), and assuming a constant curvature within the plastic hinge region. The ultimate curvature value was obtained from the column test experimentally. Shear force capacities were computed under a constant concentric compression, at; i) nominal flexural capacity of un-retrofitted column, ii) nominal shear capacity of un-retrofitted column, and iii) nominal shear capacity of retrofitted column. The following sections describe the analytical results and their comparisons with the experimental hysteretic relationship for each column, i.e.; circular building column, rectangular building column, and square bridge column.

4.5.1 Circular Building Column

Figure 4.9 shows the results of analytically computed lateral force capacities, and the comparison of analytically generated force-displacement envelop curve with experimentally recorded hysteretic force-displacement relationship. The results indicate a very good match between the analytically generated force-displacement envelop under monotonically increasing lateral forces and the experimentally recorded hysteretic relationship. The lateral force corresponding to the nominal flexural capacity of the companion non-retrofitted column is also plotted, indicating a very little effect of

retrofitting on strength (124 kN computed for un-retrofitted column versus 152 kN computed for retrofitted column). This is expected, as the retrofit methodology was intended to increase flexural deformability, rather than the flexural strength. The force-displacement relationships show gradual reduction in lateral force capacity due to the P- Δ effect caused by the accompanying axial compression, which produces additional moments at increasing magnitudes as lateral displacements increase, consuming a portion of the flexural capacity, resulting in a reduction in the moment associated with the lateral force. The same figure also depicts the nominal shear capacities of a companion un-retrofitted column and the tested retrofitted column, which are 302 kN, and 486 kN, respectively. These shear capacities are well above the shear forces corresponding to the nominal flexural capacities un-retrofitted and retrofitted columns, which are 124 kN and 152 kN, respectively, indicating that the column is not critical in shear with or without retrofitting. Indeed, this column was designed to behave predominantly in the flexure mode of deformations.

4.5.2 Rectangular Building Column

Figure 4.10 shows the results of analytically computed lateral force capacities, and the comparison of analytically generated force-displacement envelop curve with experimentally recorded hysteretic force-displacement relationship. The results indicate a very good match between the analytically generated force-displacement envelop under monotonically increasing lateral forces and the experimentally recorded hysteretic relationship. The lateral force corresponding to the nominal flexural capacity of the companion non-retrofitted column is also plotted, indicating little effect of retrofitting on strength (125 kN computed for un-retrofitted column versus 158 kN computed for

retrofitted column). This is expected, as the retrofit methodology was intended to increase flexural deformability, rather than the flexural strength. The force-displacement relationships show gradual reduction in lateral force capacity due to the P- Δ effect caused by the accompanying axial compression, which produces additional moments at increasing magnitudes as lateral displacements increase, consuming a portion of the flexural capacity, resulting in a reduction in the moment associated with the lateral force. The same figure also depicts the nominal shear capacities of a companion un-retrofitted column and the tested retrofitted column, which are 241 kN, and 378 kN, respectively. These shear capacities are well above the shear forces corresponding to the nominal flexural capacities un-retrofitted and retrofitted columns, which are 125 kN and 158 kN, respectively, indicating that the column is not critical in shear with or without retrofitting. Indeed, this column was designed to behave predominantly in the flexure mode of deformations.

4.5.3 Square Column

Figure 4.11 shows the results of analytically computed lateral force capacities, and the comparison of analytically generated force-displacement envelop curve with experimentally recorded hysteretic force-displacement relationship. The results indicate a very good match between the analytically generated force-displacement envelope under monotonically increasing lateral forces and the experimentally recorded hysteretic relationship. The lateral force corresponding to the nominal flexural capacity of the companion non-retrofitted column is also plotted, indicating little effect of retrofitting on strength (648 kN computed for un-retrofitted column versus 714 kN computed for retrofitted column). This is expected, as the retrofit methodology was intended to increase

flexural deformability, rather than the flexural strength. The force-displacement relationships show very little reduction in lateral force capacity due to the $P-\Delta$ effect, because of the low level of axial compression applied on this column (15% of column concentric capacity). This column was designed to be shear critical, with a nominal shear capacity lower than the shear force associated with flexural capacity of the column. Figure 4.11 depicts the nominal shear capacity of the column prior to retrofit as 605 kN, which is below the experimentally recorded maximum lateral force capacity of 693 kN. Hence, if the column had not been retrofitted, it would have failed in shear. Retrofitting increased the column shear capacity (computed as 914 kN after retrofitting), changed the behaviour from shear-critical to flexure dominant ductile behaviour.

4.6 Effect of Transverse Prestressing on Column Deformability

The performance of flexure-dominant building columns under high axial compression, and the performance of shear-critical bridge column under low axial compression both indicate that the column retrofit strategy adopted in this research project achieved the objective of increasing strength (in shear critical column) and inelastic deformability. High strength steel straps were able to provide active and passive confinement pressures, while also controlling diagonal tension and providing additional shear reinforcement, thereby increasing the column shear capacity. The improvement in flexural deformability resulted from the confinement of compression concrete that resulted from combined axial load and flexural bending. The active pressure generated by transverse prestressing, in addition to the reserve passive confinement pressure capacity produced high level of ductility, improving lateral drift capacity to approximately 6%, which is a deformation capacity that is well above what is expected from fully ductile

columns. This should be compared with un-retrofitted and seismically deficient-building columns of pre-1970's era, which were able to develop 1% to 2% drift capacities depending on the level of axial compression. The contribution of high-strength steel straps to shear capacity was twofold; first, controlling diagonal tension cracks resulting from seismic shear forces, thereby improving the aggregate interlock mechanism and increasing concrete shear resistance, and second, providing additional shear reinforcement, thereby increasing the contribution of reinforcement to shear resistance. In the case of shear deficient bridge column that was tested, the mode of behaviour changed from a brittle shear mode to a ductile flexure mode. Within the flexural mode of response, the straps improved concrete confinement.

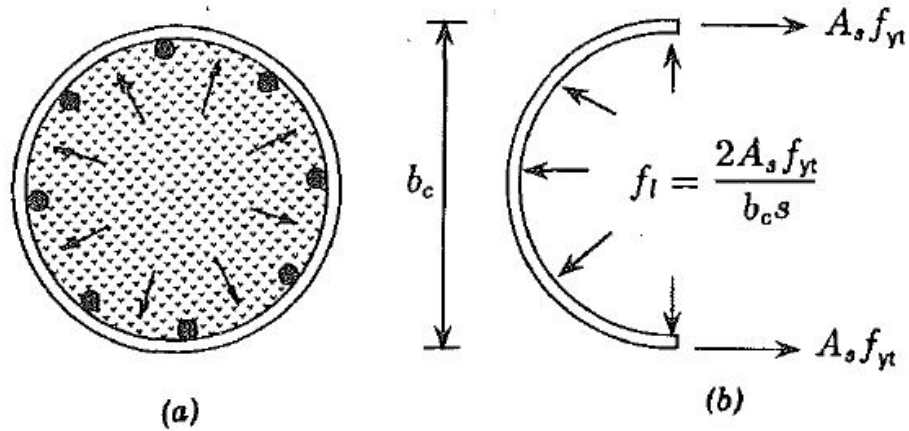


Figure 4-1 Lateral pressure in circular columns: (a) Uniform buildup of pressure; and (b) Computation of lateral pressure from hoop tension
(Saatcioglu and Razvi 1992)

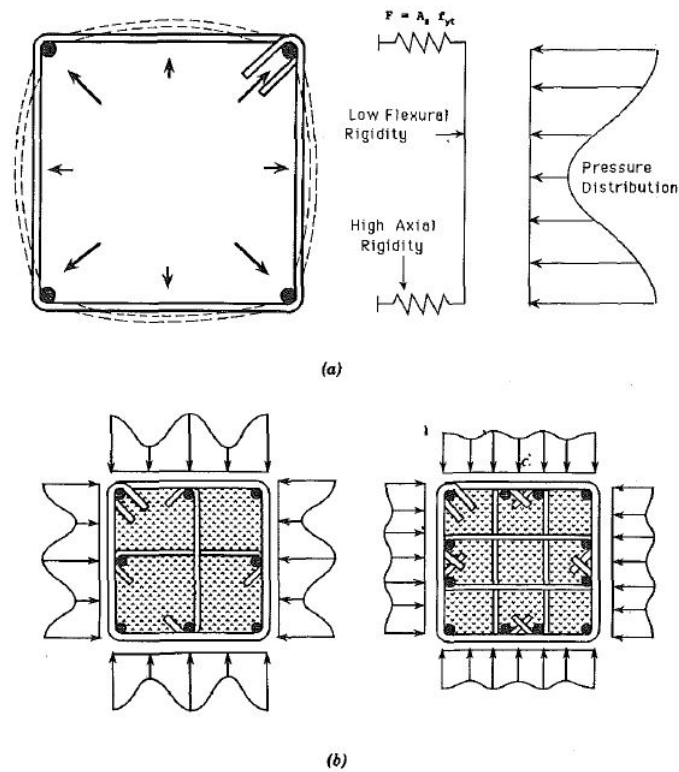
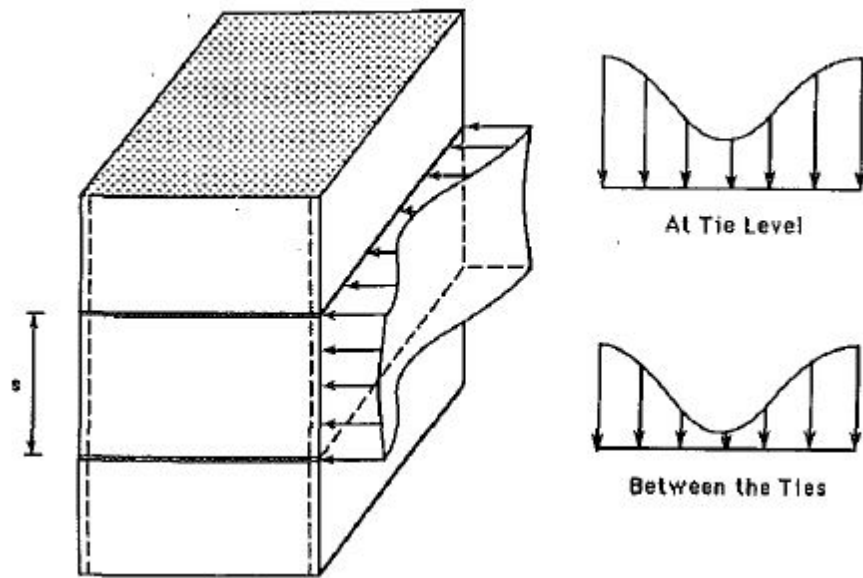
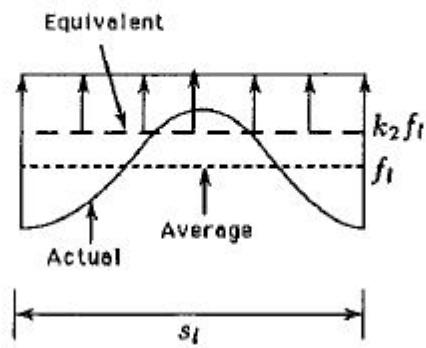


Figure 4-2 Lateral pressure in square column: (a) Lateral pressure buildup in square columns; and (b) Pressure distribution resulting from different reinforcement arrangements
(Saatcioglu and Razvi 1992)



(a)



(b)

Figure 4-3 Distributions of lateral pressures: (a) Distributions of lateral pressure along member length; and (b) Actual, average, and equivalent lateral pressures

(Saatcioglu and Razvi 1992)

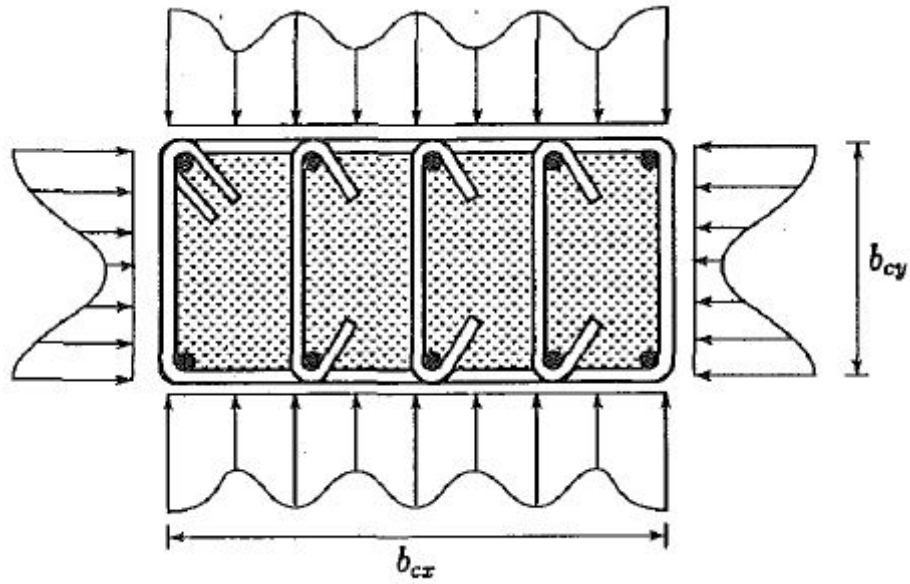


Figure 4-4 Lateral pressure distribution in rectangular column
(Saatcioglu and Razvi 1992)

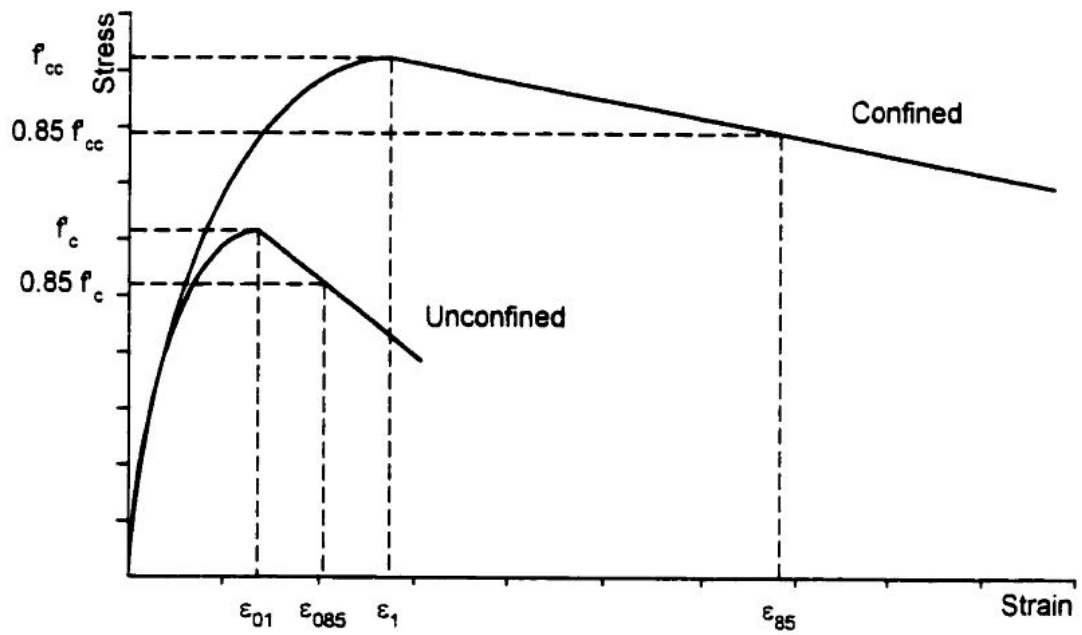


Figure 4-5 Stress-strain relationship of unconfined and confined concrete
(Yalcin 1997)

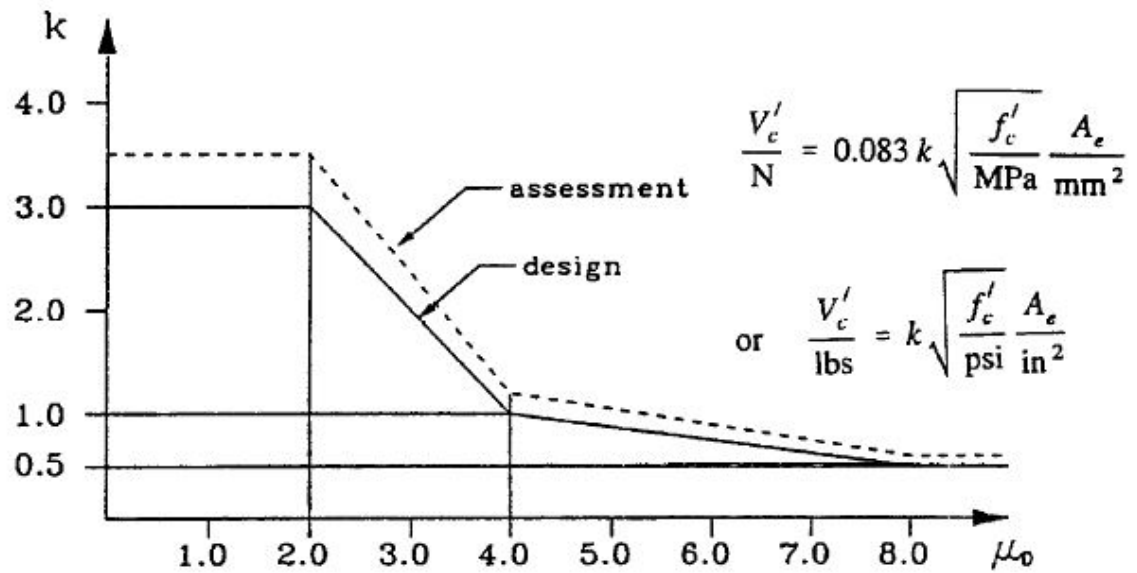


Figure 4-6 Relationship between unidirectional ductility and concrete shear contribution

(Seible and Priestley 1995)

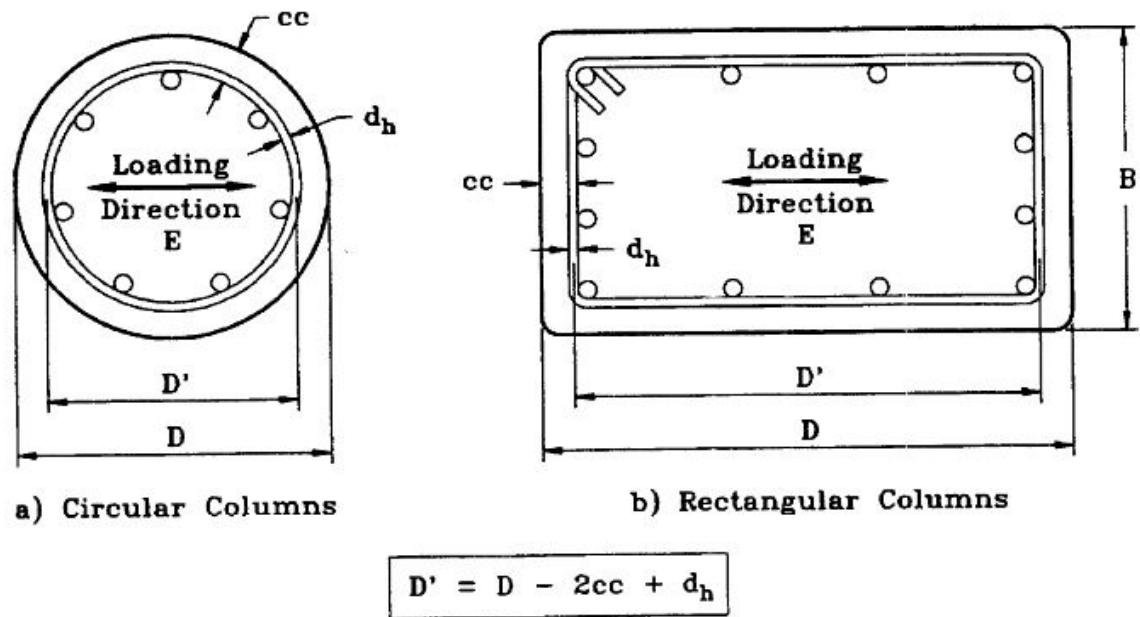
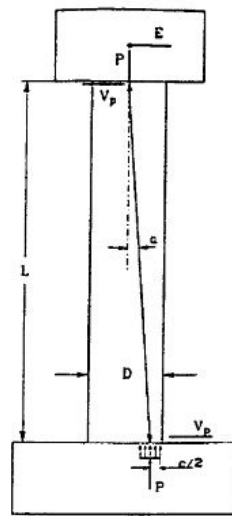


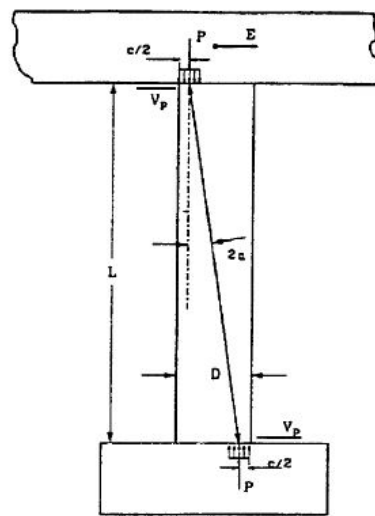
Figure 4-7 Definition of column core dimension

(Seible and Priestley 1995)



(a) Single Bending

$$V_p = P \frac{(D - c)}{2L}$$



(b) Double Bending

$$V_p = P \frac{(D - c)}{L}$$

Figure 4-8 Axial force shear contribution

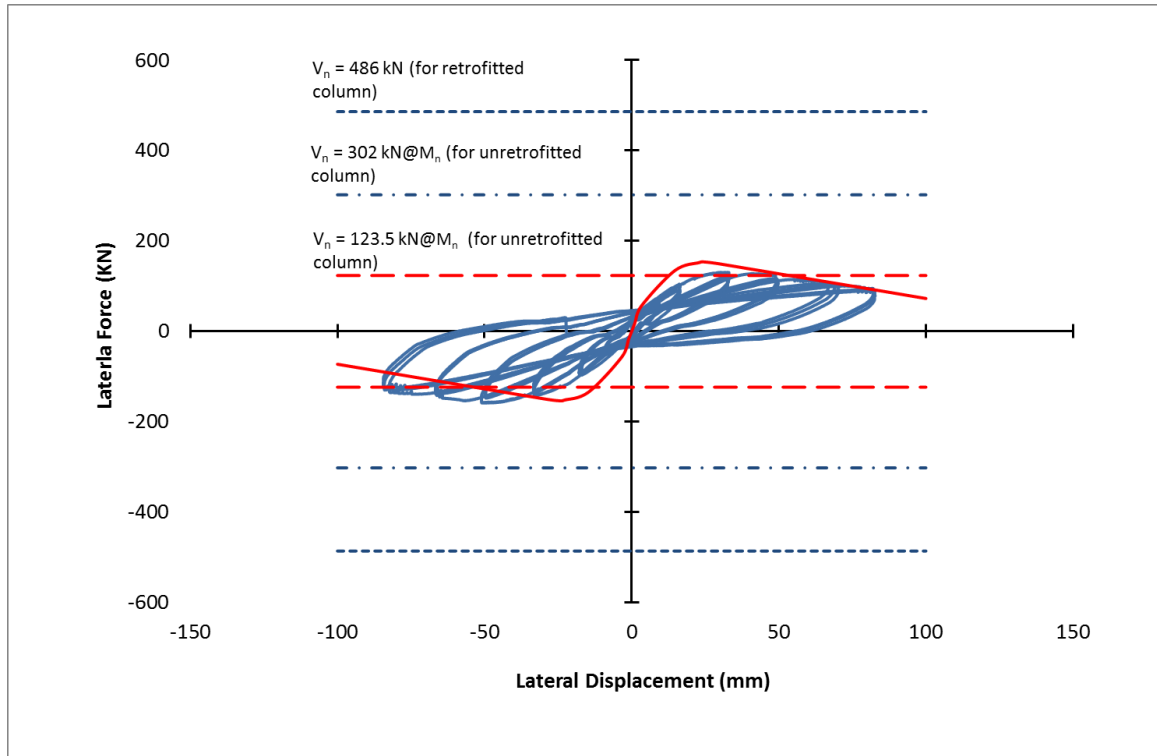


Figure 4-9 Comparison of analytical and experimental results for circular column

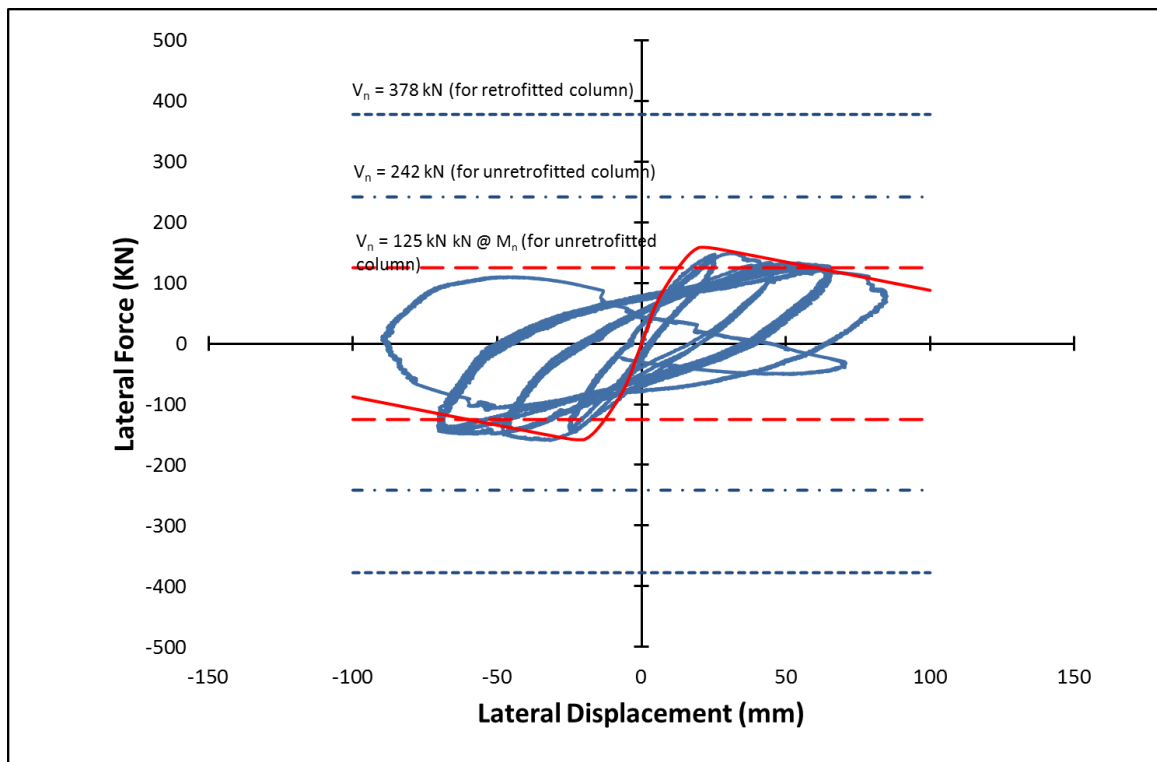


Figure 4-10 Comparison of analytical and experimental results for rectangular column

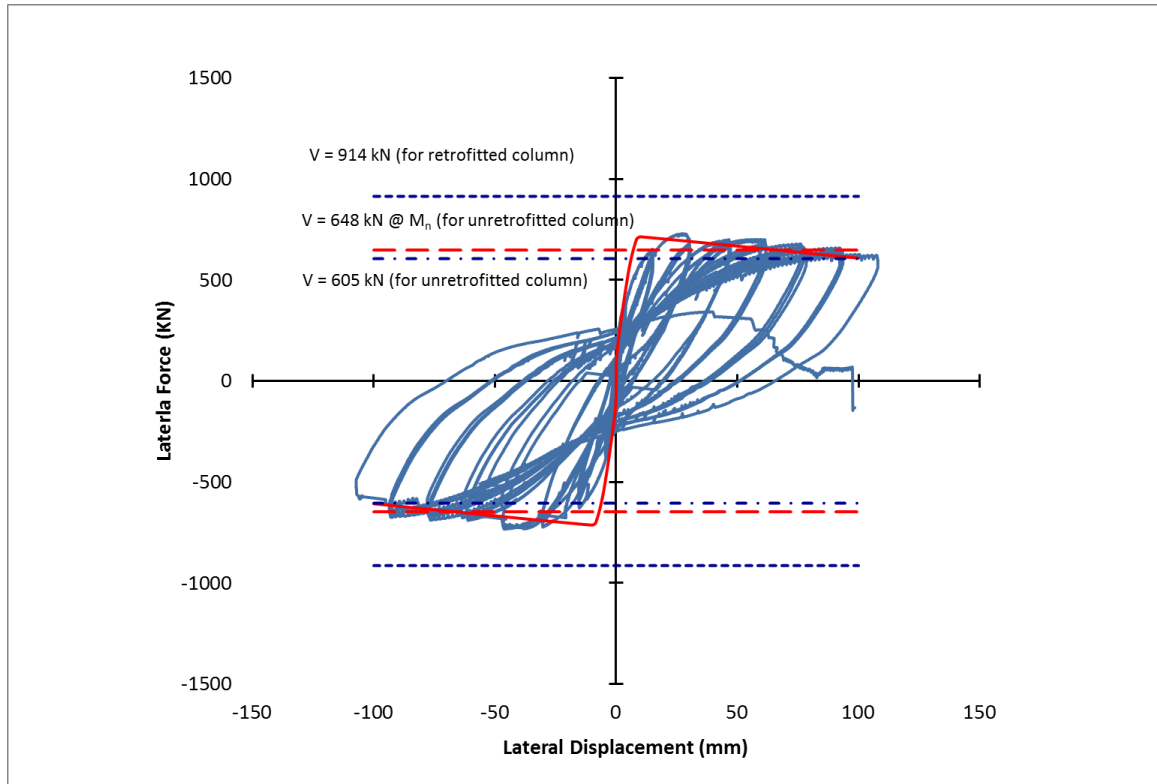


Figure 4-11 Comparison of analytical and experimental results for square column

Chapter 5: Design Recommendations and Conclusions

5.1 Introduction

Seismic deficiencies in reinforced concrete columns typically occur due to; i) lack of confinement reinforcement, ii) lack of sufficient shear capacity, or iii) insufficient splicing of longitudinal reinforcement. All three deficiencies can be remedied through seismic retrofits that involve additional transverse reinforcement. This can be done either by providing reinforced concrete, steel, or fibre-reinforced polymer (FRP) jackets around existing columns or by applying transverse prestressing to the column (RetroBelt System). Column jackets provide passive lateral pressure for concrete confinement, improve bond between spliced reinforcement and concrete, and provide additional reinforcement against shear. Transverse prestressing provides the same, while also providing active lateral pressure that further enhances concrete confinement, reinforcement bond and shear capacity of concrete. The original concept of transverse prestressing was developed at the University of Ottawa using 7-wire prestressing strands around columns, forming steel hoops (RetroBelts) that either bear against column concrete for circular column applications, or applied through raiser steel frames and disks for square and rectangular column applications. Design procedure was developed previously for the RetroBelt system (Saatcioglu et al. 2002, Saatcioglu and Yalcin 2003). The current research extends the same concept to columns prestressed by means of high-

strength packaging straps in the transverse direction. The design procedures developed earlier is adapted in this Chapter to the use of packaging straps. The Chapter also presents the conclusions obtained from the current research project.

5.2 Design Procedure

5.2.1 Shear Design

Shear retrofitting of reinforced concrete column is needed if the shear capacity of column is lower than its flexural capacity. These types of columns fail in brittle shear mode due to diagonal tension, with wide diagonal cracks forming in concrete, showing low energy dissipation. Shear retrofitting is implemented to increase the shear capacity of column above its flexural capacity, and allow the column to fail in ductile flexural mode with high energy dissipation characteristics.

The proposed retrofit design method is based on the transverse strain limit below which damage to concrete is controlled and the mechanism of aggregate interlock is maintained. The limiting transverse strain is dependent on the lateral drift demand of column and conservatively taken as the yield strain of Grade 400 MPa steel which is used in practice as transverse reinforcement. This implies that a transverse strain limit of 0.2% can be used as the upper limit. The following expression can be used to calculate the cross sectional area A_{ps} of packaging straps required within spacing s_{ps} for shear retrofitting.

$$V_e \leq V_c + V_s + V_p \quad (5.1)$$

$$V_c = 0.2\sqrt{f'_c}bd \quad (5.2)$$

$$V_s = A_v f_s \frac{d}{s} \quad (5.3)$$

$$f_s = 0.002 E_s \leq f_y \quad (5.4)$$

$$V_p = 2 A_{ps} f_{pe} \frac{h}{s_p} \quad (5.5)$$

$$f_{pe} = f_{pi} + 0.002 E_p \leq f_{py} \quad (5.6)$$

$$A_{ps} = \left(\frac{V_e - 0.2 \sqrt{f'_c} b d - A_v f_s \frac{d}{s}}{2 f_{pe}} \right) \frac{s_p}{h} \quad (5.7)$$

$$50 \text{ MPa} < f_{pi} \leq 0.5 f_{pu} \quad (5.8)$$

$$s_p \leq \frac{h}{4} \quad (5.9)$$

Where, V_e is the maximum seismic shear force that column may be subjected to during earthquake, V_c is shear force resistance provided by tensile stresses in concrete, V_s is shear resistance provided by internal shear reinforcement, and V_p is shear strength enhancement provided by external prestressing.

5.2.2 Confinement Design

Retrofit design for confinement of reinforced concrete columns by externally prestressed packaging straps is based on the design drift ratio δ . Retrofitting is required if the drift capacity of column is less than its drift demand. Lack of confinement is the

result of wide spacing and poor detailing of transverse reinforcements in concrete columns. Proposed method for confinement retrofit design of concrete columns is adopted from the design procedure developed by Saatcioglu and Razvi (2002) for internally placed confinement reinforcement in new concrete columns, by extending it to include the additional confinement imposed by the active confinement pressure due to prestressing. The following expression can be used to calculate the cross sectional area A_{ps} and spacing s_{ps} of transverse prestressing straps.

$$A_{ps} = 2 \frac{f'_c}{f_{pe}} \frac{P}{P_0} h s_{ps} \delta \quad (5.10)$$

Where, f'_c and f_{pe} are concrete compressive strength and effective stress in prestressing straps, h is the average of cross-sectional dimensions in two orthogonal directions, $\frac{P}{P_0}$ is the ratio of maximum axial compression on the column due to the earthquake effects to column concentric capacity. The effective stress in prestressing straps consists of active and passive components. The active component is the initial prestress f_{pi} and the passive component is the stress caused by expanding concrete under earthquake loads. The following expression describes the effective stress in prestressing straps.

$$f_{pe} = f_{pi} + 0.002 E_p \leq f_{py} \quad (5.11)$$

Where, $0.002 E_p$ is the passive confinement provided by straps, and may be taken conservatively as stress corresponding to 0.002 strain in steel.

The design drift ratio δ is based on performance level that is expected from a column during earthquake. Design drift ration of 4.0% is recommended for achieving a fully ductile column.

5.2.3 Design for Splice Clamping

The current research program does not include columns that are deficient in longitudinal reinforcement splicing. However, the previous research on columns with splice deficiencies retrofitted with RetroBelt indicated that the confinement design procedure is equally applicable to retrofit design for splice clamping, except for the limiting transverse strain used (Saatcioglu et al. 2002). It was experimentally observed that longitudinal reinforcement with splice lengths no less than 20 times the bar diameter can maintain their bond to concrete and transfer the tensile forces if the surrounding concrete is kept to a maximum transverse expansion strain of 0.001. This value is recommended for the design of prestressing packaging straps for splice clamping in calculating the effective prestress, f_{pe} in Sec. 5.2.2, i.e, $f_{pe} = f_{pi} + 0.001 E_p \leq f_{py}$.

5.3 Conclusion

The following conclusions can be drawn from the current experimental research project:

- Seismically deficient reinforced concrete columns can be retrofitted through transverse prestressing by means of high-strength packaging straps. The straps provide active and passive confinement pressures, improving ductility of compression concrete, enhancing flexural response of columns. The same technique also improves shear capacity of columns, suppressing premature shear failure, promoting ductile flexural behaviour in otherwise shear-critical columns.

- Circular building columns under high levels of axial compression (up to 50% of concentric column capacity), can develop a minimum of 6% lateral drift with little or no strength decay in moment capacity if externally prestressed with high-strength packaging straps in the transvers direction. Circular steel hoops provide efficient confinement pressure resulting from hoop tension.
- The rectangular building column tested under 50% of concentric column capacity was able to develop 3.5% lateral drift ratio with little or no strength decay in moment capacity when externally prestressed with high-strength packaging straps in the transvers direction. The raiser concrete units used to complete the rectangular column to an elliptical shape, in an effort to eliminate stress concentration at the corners and promote uniform hoop tension, were effective in achieving near uniform confinement pressure. However, in this column, the amount of transverse steel was not sufficiently high to attain further improvements in ductility.
- Shear deficient square columns can be retrofitted through external prestressing by means of high-strength packaging steel straps. The prestressing overcomes diagonal tensile stresses associated with seismic shear force reversals, and enhances the shear resistance of concrete. The straps further enhance the column shear resistance by providing additional shear reinforcement. The retrofitting of the square bridge column tested in the current investigation was able to suppress premature shear failure and changed the mode of behavior from brittle shear to ductile flexure, with a drift capacity of 6%. The use of pre-fabricated concrete spacers to change the cross-sectional shape of a square column to a circular

column to promote the development of uniform hoop tension in the straps is effective.

- The high-strength steel straps used to prestress columns, with 800 MPa yield strength and 8% elongation strain capacity can be used as prestressing steel for column retrofitting. The reduction in steel area due to the crimping of steel, which was recorded to be approximately 20%, must be considered in design.
- Conventional sectional analysis for flexure and flexural member analysis with appropriate material models accounting for concrete confinement can be used to generate force-deformation behavior of columns under monotonically increasing lateral loads. The analytically generated curves showed excellent agreement with the envelopes of the hysteresis loops recorded during column tests.
- The design procedure presented in this Chapter can be employed to provide a displacement based design procedure for column retrofitting involving external transverse prestressing.

5.4 Recommendations for Future Research

The current research project was the first attempt to explore the possibility of using the type of packaging straps employed on concrete columns for seismic retrofitting. Only a limited number of column tests were conducted. Additional research is needed in the future for the full development of the retrofit methodology. The following is recommended for future research:

- Additional tests are needed to expand the proposed retrofitting technique to columns with different seismic deficiencies. The effectiveness of external prestressing on columns with lap splice problem should be studied.

- Additional experimental and analytical research is required to derive relationship between the prestressing level and spacing of straps with lateral drift capacity of columns.
- More study and experimental research are needed to find the most effective shape for elliptical raiser units for rectangular columns.
- More study is required on the performance of the sealers used as connector devices.

References

1. Aboutaha, Riyad S. *Seismic retrofit of non-ductile reinforced concrete columns using rectangular steel jackets*. Diss. University of Texas at Austin, 1994.
2. Beauséjour, Pascal. *Seismic Retrofit of Concrete Columns with Splice Deficiencies by External Prestressing*. Diss. Masters Thesis, University of Ottawa, 2000.
3. Bett, B. John, Richard E. Klingner, and James O. Jirsa. "Lateral load response of strengthened and repaired reinforced concrete columns." *ACI Structural Journal* 85.5 (1988).
4. Chai, Yuk Hon, MJ Nigel Priestley, and Frieder Seible. "Analytical model for steel-jacketed RC circular bridge columns." *Journal of Structural Engineering* 120.8 (1994): 2358-2376.
5. Chai, Yuk Hon, MJ Nigel Priestley, and Frieder Seible. "Seismic retrofit of circular bridge columns for enhanced flexural performance." *ACI Structural Journal* 88.5 (1991).
6. Julio, E. N. B. S., Fernando AB Branco, and Vítor D. Silva. "Reinforced concrete jacketing—interface influence on monotonic loading response." *ACI structural journal* 102.2 (2005): 252-257.
7. Karbhari, V. M., and F. Seible. "Design considerations for the use of fiber reinforced polymeric composites in the rehabilitation of concrete structures." *Proceedings of NIST Workshop on Standards Development for the Use of Fiber Reinforced Polymers for the Rehabilitation of Concrete and Masonry Structures*. 1998.
8. Li, Y. F., et al. "A constitutive model of concrete confined by steel reinforcements and steel jackets." *Canadian Journal of Civil Engineering* 32.1 (2005): 279-288.
9. Mander, John B., Michael JN Priestley, and R. Park. "Theoretical stress-strain model for confined concrete." *Journal of structural engineering* 114.8 (1988): 1804-1826.

10. Mes, Derek. *Seismic retrofitting of concrete bridge columns by external prestressing*. Diss. Masters Thesis, University of Ottawa, 1999.
11. Pantelides, Chris P., et al. "Retrofit of RC bridge pier with CFRP advanced composites." *Journal of Structural Engineering* 125.10 (1999): 1094-1099.
12. Priestley, M. J. N., F. Seible, and Y. H. Chai. "Seismic retrofit of bridge columns using steel jackets." *Tenth World Conference on Earthquake Engineering*. Vol. 9. 1992.
13. Razvi, Salim, and Murat Saatcioglu. "Confinement model for high-strength concrete." *Journal of Structural Engineering* 125.3 (1999): 281-289.
14. Rodriguez, Mario, and Robert Park. "Seismic load tests on reinforced concrete columns strengthened by jacketing." *ACI Structural Journal* 91.2 (1994).
15. Saadatmanesh, Hamid, Mohammad R. Ehsani, and Limin Jin. "Repair of earthquake-damaged RC columns with FRP wraps." *ACI Structural Journal* 94 (1997): 206-215.
16. Saadatmanesh, Hamid, Mohammad R. Ehsani, and Limin Jin. "Seismic strengthening of circular bridge pier models with fiber composites." *ACI Structural Journal* 93.6 (1996): 639-738.
17. Saadatmanesh, Hamid, Mohammad R. Ehsani, and Mu-Wen Li. "Strength and ductility of concrete columns externally reinforced with fiber composite straps." *ACI Structural Journal* 91.4 (1994): 434-447.
18. Saatcioglu, M., Chakrabarti, S., Selbuy, R. and Mes, D. "Improving Ductility and Shear Capacity of Reinforced Concrete Columns with the Retro-Belt Retrofitting System." *Proceedings of the 7th U.S. National Conference on Earthquake Engineering*, Earthquake Engineering Research Institute, 2002.
19. Saatcioglu, Murat, and Salim R. Razvi. "Strength and ductility of confined concrete." *Journal of Structural Engineering* 118.6 (1992): 1590-1607.

20. Sadoughi Yarandi, Majid. Seismic Retrofit of Existing Reinforced Concrete Bridge Columns by Transverse Prestressing. Diss. University of Ottawa, 2007.
21. Seible, Frieder, et al. "Seismic retrofit of RC columns with continuous carbon fiber jackets." *Journal of Composites for Construction* 1.2 (1997): 52-62.
22. Seible, Frieder, et al. "Seismic retrofit of RC columns with continuous carbon fiber jackets." *Journal of Composites for Construction* 1.2 (1997): 52-62.
23. Sezen, Halil. Seismic Behavior and Modeling of Reinforced Concrete Building Columns. Diss. University of California, Berkeley, 2000.
24. Vondoros, Konstantinos G., and Stephanos E. Dritsos. "Concrete jacket construction detail effectiveness when strengthening RC columns." *Construction and Building Materials* 22.3 (2008): 264-276.
25. Xiao, Yan, and Hui Wu. "Retrofit of reinforced concrete columns using partially stiffened steel jackets." *Journal of Structural Engineering* 129.6 (2003): 725-732.
26. Xiao, Yan, Hui Wu, and G. R. Martin. "Prefabricated composite jacketing of RC columns for enhanced shear strength." *Journal of structural engineering* 125.3 (1999): 255-264.
27. Yalcin, Cem. *Seismic evaluation and retrofit of existing reinforced concrete bridge columns*. Diss. University of Ottawa, 1998.