

Behaviour of High Performance Fibre Reinforced Concrete Columns under Axial Loading

By

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Thesis submitted to the

Faculty of Graduate Studies and Research

In partial fulfillment of the requirements for the degree of

Master of Applied Sciences

In Civil Engineering

Under the auspices of the Ottawa-Carleton Institute for Civil Engineering



uOttawa

University of Ottawa

January 2014

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I would like to dedicate this work to my family.

Acknowledgements

For every valuable work in human history, there exists a true organized procedure that is obtained by hardworking of so many interested participants who made their marks before. In fact any work is valuable and is the product and one part of an evolution process that never ends. At this point, I realize that there are so many people who have kindly helped me throughout this thesis. First, I would like to express my greatest appreciation to Dr. Hassan Aoude, my hardworking supervisor for his guidance and encouragement in all phases of this research.

The experimental research in this thesis was conducted in the Jamieson Structural Laboratory at McGill University. I would like to express my great appreciation to Professor Denis Mitchell and Dr. William D. Cook for their discussions, recommendations and support throughout the experimental research program. I would like to greatly thank Sarah De Carufel, Aram Torikian, Heidy Botero, Abram Wasef who helped me in construction and testing of the specimens.

I am thankful to the professors of the Department of Civil Engineering of University of Ottawa. And the assistance of office staff is sincerely appreciated. I also gratefully acknowledge the support of material supplies from Bekaert Steel Wire Co for providing steel fibres, King Pre-Packaged Materials for providing SCC material and CRC Tech for providing materials for UHPFRC specimens. In my Graduate studies in University of Ottawa, I had chance to be supervised by Dr. Nove Naumoski and learn from his experience. I always remember him for his kindly guidance and may he rest in peace.

Finally, I would like to thank my family, my father, mother and sister for their encouragement, love and support. It is hard to imagine that I could complete this work without them.

Milad Mohammadi Hosinieh

January, 2014

Abstract

When compared to traditional concrete, steel fibre reinforced concrete (SFRC) shows several enhancements in performance, including improved tensile resistance, toughness and ductility. One potential application for SFRC is in columns where the provision of steel fibres can improve performance under axial and lateral loads. The use of SFRC can also allow for partial replacement of transverse reinforcement required by modern seismic codes. To improve workability, self-consolidating concrete (SCC) can be combined with steel fibres, leading to highly workable SFRC suitable for structural applications. Recent advances in material science have also led to the development of ultra-high performance fibre reinforced concretes (UHPFRC), a material which exhibits very high compressive strength, enhanced post-cracking resistance and high damage tolerance. In heavily loaded ground-story columns, the use of UHPFRC can allow for reduced column sections.

This thesis presents the results from a comprehensive research program conducted to study the axial behaviour of columns constructed with highly workable SFRC and UHPFRC. As part of the experimental program, twenty-three full-scale columns were tested under pure axial compressive loading. In the case of the SFRC columns, columns having rectangular section and constructed with SCC and steel fibres were tested, with variables including fibre content and spacing of transverse reinforcement. The results confirm that use of fibres results in improved column behaviour due to enhancements in core confinement and cover behaviour. Furthermore, the results demonstrate that the provision of steel fibres in columns can allow for partial replacement of transverse reinforcement required by modern codes. The analytical investigation indicates that confinement models proposed by other researchers for traditional RC and SFRC can predict the response of columns constructed with SCC and highly workable SFRC. In the case of the UHPFRC columns, variables included configuration and spacing of transverse reinforcement. The results demonstrate that the use of appropriate detailing in UHPFRC columns can result in suitable ductility. Furthermore, the results demonstrate the improved damage tolerance of UHPFRC when compared to traditional high-strength concrete. The analytical investigation demonstrates the need for development of confinement models specific for UHPFRC.

Table of Contents

1	Introduction	1
1.1	Introduction to Research Program	1
1.2	Thesis Overview	2
2	Literature Review	4
2.1	Chapter Overview	4
2.2	Overview of SCC and Previous Research on SCC Columns	4
2.2.1	Introduction to SCC	4
2.2.2	Previous Research on SCC Columns Under Compression	6
2.3	Overview of SFRC and Previous Research on SFRC Columns	9
2.3.1	Introduction to SFRC	9
2.3.2	Factors Affecting SFRC Fresh-State Behaviour	9
2.3.3	Mechanical Properties of SFRC:	11
2.3.4	Previous Research on SFRC Columns Under Compression	21
2.4	Overview of UHPFRC and Previous Research on UHPFRC Columns	25
2.4.1	Introduction to UHPFRC	25
2.4.2	Compressive Behaviour of UHPFRC	25
2.4.3	Previous Research on UHPFRC Columns	26
2.5	Review of Code Requirements for Detailing of Reinforced Concrete Columns	29
3	Experimental Program	33
3.1	Introduction	33
3.2	Description of Test Specimens	33
3.3	Material Properties	40
3.3.1	Reinforcing Steel	40
3.3.2	Steel Fibres	42

3.3.3	Concrete Mixture	43
3.3.4	Concrete Properties	45
3.4	Instrumentation and Testing	51
3.4.1	Instrumentation	51
3.4.2	Test Setup Procedure	54
3.4.3	Loading Procedure	56
4	Experimental Results	57
4.1	Introduction	57
4.2	Results - SFRC Test Series (Specimens With 30 mm Cover)	58
4.2.1	Specimen # AR0	58
4.2.2	Specimen # AR1	62
4.2.3	Specimen # AR15	65
4.2.4	Specimen # BR0	68
4.2.5	Specimen # BR1	71
4.2.6	Specimen # BR15	74
4.2.7	Specimen # CR0	77
4.2.8	Specimen # CR1	80
4.2.9	Specimen # CR15	83
4.3	Results - SFRC Test Series (Specimens Without Cover)	86
4.3.1	Specimen # AR0nc	86
4.3.2	Specimen # AR1nc	89
4.3.3	Specimen # AR15nc	92
4.3.4	Specimen # BR0nc	95
4.3.5	Specimen # BR1nc	98
4.3.6	Specimen # BR15nc	101
4.3.7	Specimen # CR0nc	104
4.3.8	Specimen # CR1nc	107
4.4	Ultra High Strength Specimens.....	110
4.4.1	Specimen # C3-120	110
4.4.2	Specimen # C3-80	113
4.4.3	Specimen # C3-60	116
4.4.4	Specimen # C3-40	119

4.4.5	Specimen # C4-120	122
4.4.6	Specimen # C4-60	125
5	Discussion of Results	128
5.1	Introduction	128
5.2	Summary of Results - SFRC Series.....	128
5.3	Comparison of Load-Strain Curves – SFRC Series	133
5.3.1	Effect of Transverse Reinforcement Spacing	133
5.3.2	Effect of Fibres on Column Performance	138
5.3.3	Ability of Fibres to Partially Substitute For Transverse Reinforcement.....	141
5.3.4	Effect of Fibres on Core Behaviour	143
5.3.5	Effect of Fibres on Cover Behaviour	150
5.3.6	Summary of Effect of Fibres on Core and Cover Concrete	154
5.4	Discussions of Failure Mechanisms – SFRC Series	156
5.4.1	Effect of Fibres on Crack Pattern, Cover Spalling and Crushing of the Core	156
5.4.2	Effect of Fibres on Bar Buckling	156
5.4.3	Effect of Fibres on Development of Failure Zone	161
5.4.4	Hook Deformation	162
5.5	Summary of Results - UHPFRC Series	163
5.5.1	Effect of Tie Spacing	165
5.5.2	Effect of Tie Configuration	168
5.5.3	Further Discussion.....	171
5.6	Discussions of Failure Process – UHPFRC Columns	174
6	Analytical Evaluation.....	177
6.1	Introduction	177
6.2	Analysis Methodology	178
6.3	Review of Existing Models	179
6.3.1	Models for Plain Unconfined Concrete	180

6.3.2	Models for Unconfined Steel Fibre Reinforced Concrete	180
6.3.3	Models for Plain Confined Concrete	182
6.3.4	Models for Confined SFRC	185
6.3.5	Longitudinal Reinforcement in Compression	186
6.3.6	Accounting for Cover Spalling	188
6.4	Analytical Comparisons	195
6.4.1	Overview	195
6.4.2	Analysis - SCC Columns (0% Fibres)	197
6.4.3	Analysis - SFRC Columns (1% and 1.5% Fibres)	210
6.4.4	Analysis - UHPFRC Columns	229
7	Conclusions and Recommendations	235
7.1	Conclusions	235
7.2	Recommendations for Future Research	237
8	Reference	238
9	Appendix	245
	Appendix A – Detailed Tables of Database for SCC	246
	Appendix B – Detailed Tables of Database for SFRC	249
	Part I – Square Specimens	249
	Part II – Circular Sections	256

List of Figures

Figure 1.1 Thesis Organisation.	3
Figure 2.1 Sample Comparison of SCC and Normal Concrete Proportioning	5
Figure 2.2 Sample Slump Test for SCC	5
Figure 2.3 Effect of Aggregate Size on the Fibre Distribution	10
Figure 2.4 Inclination Angle of Fibres Relative to Cracking Plane	12
Figure 2.5. Fracturing Process and Toughening Mechanism in Brittle Composites	13
Figure 2.6. Pull-Out Load vs. Slip Curve for a Straight Fibre	14
Figure 2.7. Pull-Out Process For Hooked-End Steel Fibres	15
Figure 2.8. Pull-Out Process for Hooked-End Fibres with Plastic Deformation along Fibre Length	16
Figure 2.9. Variability of Tensile Behaviour of SFRC Depending on Fibre Content.	17
Figure 2.10 Uniaxial Behaviour of Concrete under Compression – Compaction and Dilatation.....	19
Figure 2.11 Bi-Axial Behaviour of Concrete under Compression – Compaction and Dilatation.....	19
Figure 2.12 Typical Compressive Stress-Strain Response of SFRC	20
Figure 3.1 Column Details (SFRC Test Series)	36
Figure 3.2 Column Details (UHPFRC Test Series)	37
Figure 3.3 Reinforcing Cages and Formwork for the SFRC Test Series	38
Figure 3.4 Reinforcing Cages and Formwork for the UHPFRC Test Series	39
Figure 3.5 Typical Tension Test Results for 10M Rebar Samples	41
Figure 3.6 Typical Tension Test Results for 15M Rebar Samples	41
Figure 3.7 Fibres Used in the SFRC and UHPFRC Series	42
Figure 3.8 Pre-Packed Concrete Mixtures	43
Figure 3.9 Mixers Used in This Study	45
Figure 3.10 Typical Slump Flows for SCC and Highly Workable SFRC	46
Figure 3.11 UHPFRC Mixing Procedure.....	47
Figure 3.12 Typical UHPFRC Slump Flow without and with Fibres.....	47
Figure 3.13 Compressive Stress-Strain Response for SFRC and UHPFRC Used in This Study	49

Figure 3.14 Flexural Beam Load-Deflection Response for SFRC and UHPFRC Used in This Study	50
Figure 3.15 LVDT Placement for the SFRC and UHFRC Columns	51
Figure 3.16 Strain Gauge Instrumentation for the SFRC Columns	52
Figure 3.17 Strain Gauge Instrumentation for the UHPFRC Columns	53
Figure 3.18 Pre-Test Setup Procedure	54
Figure 3.19 Instrumentation Scheme	55
Figure 3.20 Columns under MTS Testing Machine Ready for Testing.....	55
Figure 3.21 Loading Rate Scheme for Experimental Specimens.....	56
Figure 4.1 Staged Failure Process for Specimen # AR0.....	60
Figure 4.2 Specimen # AR0 Failure Snapshots During the Test	60
Figure 4.3 Close Look at Specimen # AR0.....	61
Figure 4.4 Results for Specimen # AR0.....	61
Figure 4.5 Staged Failure Process for Specimen # AR1	63
Figure 4.6 Specimen # AR1 Failure Snapshots During the Test	63
Figure 4.7 Close Look at Specimen # AR1.....	64
Figure 4.8 Results for Specimen # AR1.....	64
Figure 4.9 Staged Failure Process for Specimen # AR15	66
Figure 4.10 Specimen # AR15 Failure Snapshots During the Test	66
Figure 4.11 Close Look at Specimen # AR15.....	67
Figure 4.12 Results for Specimen # AR15.....	67
Figure 4.13 Staged Failure Process for Specimen # BR0	69
Figure 4.14 Specimen # BR0 Failure Snapshots During the Test.....	69
Figure 4.15 Close Look at Specimen # BR0.....	70
Figure 4.16 Results for Specimen # BR0.....	70
Figure 4.17 Staged Failure Process for Specimen # BR1	72
Figure 4.18 Specimen # BR1 Failure Snapshots During the Test.....	72
Figure 4.19 Close Look at Specimen # BR1	73
Figure 4.20 Results for Specimen # BR1	73
Figure 4.21 Staged Failure Process for Specimen # BR15	75
Figure 4.22 Specimen # BR15 Failure Snapshots During the Test.....	75

Figure 4.23 Close Look at Specimen # BR15	76
Figure 4.24 Results for Specimen # BR15	76
Figure 4.25 Staged Failure Process for Specimen # CR0	78
Figure 4.26 Specimen # CR0 Failure Snapshots During the Test.....	78
Figure 4.27 Close Look at Specimen # CR0.....	79
Figure 4.28 Results for Specimen # CR0.....	79
Figure 4.29 Staged Failure Process for Specimen # CR1	81
Figure 4.30 Specimen # CR1 Failure Snapshots During the Test.....	81
Figure 4.31 Close Look at Specimen # CR1	82
Figure 4.32 Results for Specimen # CR1	82
Figure 4.33 Staged Failure Process for Specimen # CR15	84
Figure 4.34 Specimen # CR15 Failure Snapshots During the Test.....	84
Figure 4.35 Close Look at Specimen # CR15	85
Figure 4.36 Results for Specimen # CR15.....	85
Figure 4.37 Staged Failure Process	87
Figure 4.38 Column Failure Snapshots During the Test.....	87
Figure 4.39 Close Look at Specimen # AR0nc.....	88
Figure 4.40 Results for Specimen # AR0nc.....	88
Figure 4.41 Staged Failure Process for Specimen # AR1nc	90
Figure 4.42 Specimen # AR1nc Failure Snapshots During the Test.....	90
Figure 4.43 Close Look at Specimen # AR1nc.....	91
Figure 4.44 Results for Specimen # AR1nc.....	91
Figure 4.45 Staged Failure Process for Specimen # AR15nc	93
Figure 4.46 Specimen # AR15nc Failure Snapshots During the Test.....	93
Figure 4.47 Close Look at Specimen # AR15nc.....	94
Figure 4.48 Results for Specimen # AR15nc.....	94
Figure 4.49 Staged Failure Process for Specimen # BR0nc	96
Figure 4.50 Specimen # BR0nc Failure Snapshots During the Test.....	96
Figure 4.51 Close Look at Specimen # BR0nc	97
Figure 4.52 Results for Specimen # BR0nc	97
Figure 4.53 Staged Failure Process for Specimen # BR1nc	99

Figure 4.54 Specimen # BR1nc Failure Snapshots During the Test.....	99
Figure 4.55 Close Look at Specimen # BR1nc	100
Figure 4.56 Results for Specimen # BR1nc	100
Figure 4.57 Staged Failure Process for Specimen # BR15nc	102
Figure 4.58 Specimen # BR15nc Failure Snapshots During the Test.....	102
Figure 4.59 Close Look at Specimen # BR15nc	103
Figure 4.60 Results for Specimen # BR15nc	103
Figure 4.61 Staged Failure Process for Specimen # CR0nc	105
Figure 4.62 Specimen # CR0nc Failure Snapshots During the Test.....	105
Figure 4.63 Close Look at Specimen # CR0nc	106
Figure 4.64 Results for Specimen # CR0nc	106
Figure 4.65 Staged Failure Process for Specimen # CR1nc	108
Figure 4.66 Specimen # CR1nc Failure Snapshots During the Test.....	108
Figure 4.67 Close Look at Specimen # CR1nc	109
Figure 4.68 Results for Specimen # CR1nc	109
Figure 4.69 Staged Failure Process for Specimen # C3-120	111
Figure 4.70 Specimen # C3-120 Failure Snapshots During the Test.....	111
Figure 4.71 Close Look at Specimen # C3-120	112
Figure 4.72 Results for Specimen # C3-120	112
Figure 4.73 Staged Failure Process for Specimen # C3-80	114
Figure 4.74 Specimen # C3-80 Failure Snapshots During the Test.....	114
Figure 4.75 Close Look at Specimen # C3-80	115
Figure 4.76 Results for Specimen # C3-80	115
Figure 4.77 Staged Failure Process for Specimen # C3-60	117
Figure 4.78 Specimen # C3-60 Failure Snapshots During the Test.....	117
Figure 4.79 Close Look at Specimen # C3-60	118
Figure 4.80 Results for Specimen # C3-60	118
Figure 4.81 Staged Failure Process for Specimen # C3-40	120
Figure 4.82 Specimen # C3-40 Failure Snapshots During the Test.....	120
Figure 4.83 Close Look at Specimen #C3-40	121
Figure 4.84 Results for Specimen # C3-40	121

Figure 4.85 Staged Failure Process for Specimen # C4-120	123
Figure 4.86 Specimen # C4-120 Failure Snapshots During the Test	123
Figure 4.87 Close Look at Specimen #C4-120	124
Figure 4.88 Results for Specimen # C4-120	124
Figure 4.89 Staged Failure Process for Specimen # C4-60	126
Figure 4.90 Specimen # C4-60 Failure Snapshots During the Test	126
Figure 4.91 Close Look at Specimen # C4-60	127
Figure 4.92 Results for Specimen # C4-60	127
Figure 5.1 Specimens in the SFRC Test Series.....	128
Figure 5.2 Definition of Toughness Parameters	129
Figure 5.3 Definition of Softening Rates K_{sd} and K_{pd}	131
Figure 5.4 Comparison of 0% Series for Specimens with Cover	135
Figure 5.5 Comparison of 0% Series for Specimens without Cover	135
Figure 5.6 Comparison of 1% Series for Specimens with Cover	136
Figure 5.7 Comparison of 1% Series for Specimens without Cover	136
Figure 5.8 Comparison of 1.5% Series for Specimens with Cover	137
Figure 5.9 Comparison of 1.5% Series for Specimens without Cover	137
Figure 5.10 Comparison of Response for the AR-Series Specimens with Cover.....	139
Figure 5.11 Comparison of Response for the BR-Series Specimens with Cover	140
Figure 5.12 Comparison of Response for the CR-Series Specimens with Cover	140
Figure 5.13 Ability of Fibres to Substitute Transverse Reinforcement in Moderately Ductile Columns	142
Figure 5.14 Ability of Fibres to Substitute Transverse Reinforcement in Ductile Columns	142
Figure 5.15 Comparison of Response for the AR-Series Specimens without Cover.....	146
Figure 5.16 Effect of Fibres on Core Improvement in Columns with Nominal Confinement ...	146
Figure 5.17 Comparison of Response for the CR-Series Specimens without Cover	147
Figure 5.18 Effect of Fibres on Core Improvement in Highly-Confined Columns	147
Figure 5.19 Comparison of Response for the BR-Series Specimens without Cover	148
Figure 5.20 Effect of Fibres on Core Improvement in Moderately-Confined Columns.....	148
Figure 5.21 Comparison of the Core-Effect in the AR, BR and CR Series Columns	149
Figure 5.22 Comparison of Cover Response for 0% Fibres	152

Figure 5.23 Comparison of Cover Response in the AR-Series Columns	152
Figure 5.24 Comparison of Cover Response in the CR-Series Columns	153
Figure 5.25 Comparison of Cover Response for 1% Fibres	153
Figure 5.26 Fibre Effect on Columns with Low Confinement (Individual Improvements)	155
Figure 5.27 Fibre Effect on Columns with High Confinement (Individual Improvements)	155
Figure 5.28 Staged Failure Process for Columns with 0%-Fibres	157
Figure 5.29 Staged Failure Process for Columns with 1%-Fibres	158
Figure 5.30 Visual Comparison of Core Crushing and Bar Buckling	159
Figure 5.31 Examples of Effective Cover Spalling in SFRC Columns	160
Figure 5.32 Effect of Fibre in Development of Failure Zone	161
Figure 5.33 Hook Deformation	162
Figure 5.34 UHPFRC Column Configurations	164
Figure 5.35 Comparison of UHPFRC Responses for Specimens with Configuration C3	166
Figure 5.36 Comparison of UHPFRC Responses for Specimens with Configuration C4	167
Figure 5.37 Comparison of UHPFRC Responses for Specimens with Spacing of 120 mm	169
Figure 5.38 Comparison of UHPFRC Responses for Specimens with Spacing of 60 mm	170
Figure 5.39 Comparison of UHPFRC Responses for Specimens with Moderate Ductility	172
Figure 5.40 Comparison of UHPFRC Responses for Specimens with High Ductility	173
Figure 5.41 Staged Failure and Damage Tolerance in the UHPFRC Columns	175
Figure 5.42 Cover Spalling in the UHPFRC Columns	176
Figure 5.43 Bar Buckling and Core Integrity in the UHPFRC Columns	176
Figure 6.1 Definition of Cover, Core and Reinforcement for Analysis	178
Figure 6.2 Comparison of Predicted Stress-Strain Response from Various Unconfined SFRC Models Proposed in the Literature (for $f'_c = 50$ MPa, $V_f = 1\%$)	181
Figure 6.3 Model Predictions for Post Peak Buckling of Longitudinal Reinforcement	187
Figure 6.4 Spalling Factor Proposed by Aoude 2008	189
Figure 6.5 Typical Stress-Strain Curves with and without Spalling Factor	189
Figure 6.6 Parameters for Evaluation of SCC & SFRC Models	196
Figure 6.7 Parameters for Evaluation of UHPFRC Models	196
Figure 6.8 Comparison of Existing Models with Experimental Results – A0nc	202
Figure 6.9 Comparison of Existing Models with Experimental Results – A0	202

Figure 6.10 Comparison of Existing Models with Experimental Results – B0nc	203
Figure 6.11 Comparison of Existing Models with Experimental Results – B0	203
Figure 6.12 Comparison of Existing Models with Experimental Results – C0nc	204
Figure 6.13 Comparison of Existing Models with Experimental Results – C0	204
Figure 6.14 Comparison of Existing Models with Experimental Results – AR0nc	207
Figure 6.15 Comparison of Existing Models with Experimental Results – AR0	207
Figure 6.16 Comparison of Existing Models with Experimental Results – BR0nc.....	208
Figure 6.17 Comparison of Existing Models with Experimental Results – BR0	208
Figure 6.18 Comparison of Existing Models with Experimental Results – CR0nc.....	209
Figure 6.19 Comparison of Existing Models with Experimental Results – CR0	209
Figure 6.20 Comparison of Existing Models with Experimental Results – A1nc	215
Figure 6.21 Comparison of Existing Models with Experimental Results – A1	215
Figure 6.22 Comparison of Existing Models with Experimental Results – B1nc	216
Figure 6.23 Comparison of Existing Models with Experimental Results – B1	216
Figure 6.24 Comparison of Existing Models with Experimental Results – C1	217
Figure 6.25 Comparison of Existing Models with Experimental Results – A15nc	218
Figure 6.26 Comparison of Existing Models with Experimental Results – A15	218
Figure 6.27 Comparison of Existing Models with Experimental Results – B15nc	219
Figure 6.28 Comparison of Existing Models with Experimental Results – B15	219
Figure 6.29 Comparison of Existing Models with Experimental Results – C15	220
Figure 6.30 Comparison of Existing Models with Experimental Results – AR1nc	223
Figure 6.31 Comparison of Existing Models with Experimental Results – AR1	223
Figure 6.32 Comparison of Existing Models with Experimental Results – BR1nc.....	224
Figure 6.33 Comparison of Existing Models with Experimental Results – BR1	224
Figure 6.34 Comparison of Existing Models with Experimental Results – CR1nc.....	225
Figure 6.35 Comparison of Existing Models with Experimental Results – CR1	225
Figure 6.36 Comparison of Existing Models with Experimental Results – AR15nc	226
Figure 6.37 Comparison of Existing Models with Experimental Results – AR15	226
Figure 6.38 Comparison of Existing Models with Experimental Results – BR15nc.....	227
Figure 6.39 Comparison of Existing Models with Experimental Results – BR15	227
Figure 6.40 Comparison of Existing Models with Experimental Results – CR15	228

Figure 6.41 Comparison of Existing Models with Experimental Results –C3-120.....	232
Figure 6.42 Comparison of Existing Models with Experimental Results –C3-80.....	232
Figure 6.43 Comparison of Existing Models with Experimental Results –C3-60.....	233
Figure 6.44 Comparison of Existing Models with Experimental Results –C3-40.....	233
Figure 6.45 Comparison of Existing Models with Experimental Results – C4-120.....	234
Figure 6.46 Comparison of Existing Models with Experimental Results –C4-60.....	234

List of Tables

Table 2.1 Previous Tests on SCC Columns Reported in the Literature	8
Table 2.2 Fibre-Matrix Bond Relations	12
Table 2.3 Examples of Fibre Shape Factor in Literature	14
Table 2.4 Previous Tests on SFRC Columns Reported in the Literature.....	24
Table 2.5 Previous Tests on Steel Reinforced UHPFRC Columns Reported in the Literature	28
Table 2.6 CSA A23.3 2004 Transverse Reinforcement Spacing Requirements.....	31
Table 2.7 ACI 318 2008 transverse reinforcement spacing requirements	31
Table 3.1 Detail of Experimental Specimens.....	35
Table 3.2 Properties of Reinforcing Steel	40
Table 3.3 Properties of Steel Fibres	42
Table 3.4 SCC Mix Properties as Specified by Manufacturer	44
Table 3.5 CRC Mix Properties as Specified by Manufacturer.....	44
Table 3.6 SCC/SFRC Slump and Slump Flow Values	46
Table 3.7 Hardened Cylinder Strength.....	48
Table 5.1 Summary of Results for SCC and SCFRC Specimens	132
Table 5.2 Comparison of specimens for effect of transverse reinforcement ratio	133
Table 5.3 Summary of Results for UHPFRC Specimens	164
Table 6.1 Summary of Columns Considered in the Analytical Evaluation	177
Table 6.2 Constitutive Models Reviewed in Section 6.3	179
Table 6.3 Equation Forms for Unconfined Plain Concrete (Stress-Strain Curve).....	190
Table 6.4 Equation Forms for Plain Concrete Confinement Models (Stress-Strain Curve).....	191
Table 6.5 Relationships for Peak Confined Stress and Strain in RC Confinement Models	192

Table 6.6 Relationships for Peak Unconfined Stress and Strain in SFRC Models	192
Table 6.7 Equation Forms for Unconfined SFRC (Stress-Strain Curve).....	193
Table 6.8 Relationships for Peak Confined Stress and Strain in SFRC Models	194
Table 6.9 Equation Forms for SFRC Confinement Models (Stress-Strain Curve).....	194
Table 6.10 Constitutive Models Considered for Each Column Series.....	195
Table 6.11 Ratio of Analytical /Experimental Results – 0% Square Sections.....	200
Table 6.12 Average Ratio of Analytical /Experimental Results – 0% Square Sections	201
Table 6.13 Ratio of Analytical /Experimental Results – 0% Rectangular Sections	205
Table 6.14 Average Ratio of Analytical /Experimental Results – 0% Rectangular Sections	206
Table 6.15 Ratio of Analytical /Experimental Results – 1% & 1.5% Square Sections	213
Table 6.16 Average Ratio of Analytical /Experimental Results – 1% & 1.5 % Square Sections	214
Table 6.17 Ratio of Analytical /Experimental Results – 1% & 1.5% Rectangular Sections	221
Table 6.18 Average Ratio of Analytical /Experimental Results – 1% & 1.5 % Rectangular Sections	222
Table 6.19 Ratio of Analytical /Experimental Results – UHPFRC Columns.....	230
Table 6.20 Average Ratio of Analytical /Experimental Results – UHPFRC Columns	231

LIST OF SYMBOLS

A_u	= Area under the load strain curve up to ϵ_{Au}
A_g	= Gross column cross-section area
A_{sh}	= Total cross-sectional area of transverse reinforcement perpendicular to one direction
A_{st}	= Area of longitudinal steel in the column cross-section
A_R	= Area of an equivalent rigid material
A_{ch}	= Cross sectional area of the core
A_{co}	= Cross sectional area of the cover
c_x	= Core dimension perpendicular to minor axis (“y” axis)
c_y	= Core dimension perpendicular to major axis (“x” axis)
d_b	= Diameter of longitudinal steel rebar
d_h	= Diameter of transverse steel rebar
d_f	= Diameter of the fibre
E_{cr}	= Crack-release energy
E_{int}	= Internal energy
ϵ_y	= Yield strain of reinforcing steel
E_{sec}	= Secant modulus of concrete
E_{it}	= Initial tangent modulus
E_{ct}	= Tangent modulus of elasticity of unconfined concrete
ϵ_{sh}	= Strain corresponding to beginning of strain hardening in reinforcing steel
ϵ_u	= Strain corresponding to ultimate strength of reinforcing steel
ϵ_{cf}	= Strain associated to peak unconfined SFRC strength

ϵ_r	= Maximum strain (rupture) of reinforcing steel in tension test
ϵ_{cc}	= Strain associated to peak confined concrete
ϵ_{Au}	= Maximum strain in Load strain curve of column response which Au is considered
ϵ_{co}	= Strain associated with peak stress of unconfined concrete
ϵ_t	= Strain corresponding to total column response
ϵ_{peak}	= Strain associated with peak total column strength
ϵ_{ccf}	= Strain associated to peak confined SFRC
ϵ_{c85}	= Strains corresponding to 85% drop from peak concrete share
ϵ_{c50}	= Strains corresponding to 50% drop from peak concrete share
ϵ_c	= Strain corresponding to peak strength of column from concrete share
f'_c	= Cylinder strength of concrete
f'_{cu}	= Unconfined concrete strength (usually 85 % of the concrete cylinder strength)
f_u	= Ultimate strength of reinforcing steel
f_{lfib}	= Lateral confining pressure due to fibres
$f_{cuf_{cover}}$	= Stress of the cover concrete in SFRC columns
f_{cuf}	= Stress in unconfined SFRC
f_{yh}	= Yield strength of transverse reinforcement
f_{yb}	= Yield strength of longitudinal reinforcement
f_{fy}	= Yield strength of steel fibres
f_y	= Yield strength of steel
f_{ccf}	= Peak confined SFRC stress
f_t	= Tensile capacity of concrete

F_s	= Pull-out strength of straight fibres
f_l	= Confinement lateral pressure due to transverse reinforcement
f_{le}	= Effective confinement lateral pressure due to transverse reinforcement
f_{cc}	= Peak confined concrete stress
f_{hcc}	= Stress in transverse reinforcement at peak concrete confined stress
f_{cf}	= Unconfined SFRC strength
h_x	= Maximum horizontal centre-to-centre spacing between longitudinal bars on all faces of the column that are laterally supported by seismic hoops or crosstie legs
h_c	= Dimension of concrete core of rectangular section measured perpendicular to the direction of the hoop bars to outside of peripheral hoop
l_f	= Length of the fibre
n_l	= Total number of longitudinal bars in the column cross-section that are laterally supported by the corner of hoops or by hooks of seismic crossties
P_o	= Nominal axial resistance at zero eccentricity
P_f	= Factored axial load
P_c	= Total concrete share in columns load response = Peak column response considering only concrete share
P_{occ}	= Normalized concrete share based on cross section of the core
P_{oc}	= Normalized concrete share based on column gross area
P_t	= Column total load
P_{max}	= Peak column response in load – strain curves
$P_{n,cover}$	= Normalized cover concrete load
P_s	= Load in longitudinal steel
$P_{\varepsilon=0.01}$	= Load at strain equal to 0.01

$P_{\varepsilon=0.02}$	= Load at strain equal to 0.02
R_d	= Ductility-related force modification factor
R.I.	= Reinforcing index
S	= Spacing from center to center of ties
s'	= Clear spacing between inner side of ties (center to center minus one diameter of ties)
T. I.	= Toughness index
V_f	= Fibre content by volume of fibres
W_f	= Fibre content by weight of fibres
w_i	= Distance between longitudinal rebar
ρ_s	= Volumetric ratio of transverse reinforcement
ρ_c	= Ratio of area of longitudinal rebar to area of the core
τ_{bond}	= Fibre-matrix bond shear strength
η_l	= Length factor which represents the average embedded length of the fibre in the matrix
α_m	= Fibre shape factor
φ	= Cover spalling factor (Aoude 2008)

1 Introduction

1.1 Introduction to Research Program

When compared to traditional concrete, steel fibre reinforced concrete (SFRC) shows several enhancements in performance, including improved tensile strength, post-cracking resistance, toughness and ductility. These improvements arise due to the ability of steel fibres to arrest and redistribute cracks under tensile stresses. One potential application for SFRC is in columns where the provision of steel fibres can improve performance under axial and lateral loads. Furthermore, the use of SFRC can potentially allow for partial replacement of transverse reinforcement required by modern seismic codes. Current industrial use of SFRC has been limited to non-structural applications, where fibres are used to replace secondary reinforcement. Structural applications and replacement of primary transverse reinforcement in columns would require use of higher fibre contents, which can result in difficulty in concrete placement. One solution to this problem may be in the combined use of self-consolidating concrete (SCC) and fibres, which can result in highly workable SFRC. While research exists on the axial load behaviour of traditional concrete and SFRC columns, only limited research exists on the behaviour of columns constructed with SCC and steel fibres. There is also a lack of published research in the case of SFRC columns having rectangular cross-section.

Recent advances in material science have also led to the development of ultra-high performance fibre reinforced concretes (UHPFRC). Through careful mix proportioning, UHPFRC exhibits high compressive strength, while provision of steel fibres allows for enhanced post-cracking resistance leading to superior damage tolerance. In heavily loaded columns, the use of UHPFRC can allow for reduced sections. Furthermore, the provision of fibres in UHPFRC can allow for increased ductility and reduced brittleness when compared to traditional high-strength concrete. Current published research on the use of UHPFRC in columns is scarce and there is a need to verify if the ductility needed for earthquake applications can be achieved in UHPFRC columns.

This thesis presents the results from a comprehensive research program conducted to study the axial load behaviour of columns constructed with highly workable SFRC and UHPFRC. As part of the experimental program, twenty-three full-scale columns are tested under pure axial compressive loading.

In the first phase of the research program, SCC and SFRC columns having rectangular cross-section are tested, and variables include fibre content and spacing of transverse reinforcement. The effect of steel fibres on core confinement, cover spalling and bar buckling is studied and the potential of using steel fibres to replace transverse reinforcement is examined. The analytical investigation examines the suitability of using models proposed in the literature to predict the response of SCC and highly workable SFRC columns.

In the second phase of the research program, UHPFRC columns having square cross-section are tested under axial load. The study examines the effect of transverse reinforcement detailing on column performance. The analytical investigation examines the suitability of using models proposed in the literature for high-strength concrete/SFRC to predict the axial response of UHPFRC columns.

1.2 Thesis Overview

The thesis consists of seven chapters as follows (see Figure 1.1):

- Chapter 1 introduces the thesis and presents a literature review on the thesis subject matter;
- Chapter 2 provides a literature review on topics relevant to the thesis including a brief overview of SCC, SFRC and UHPFRC, and a review of published research on columns;
- Chapter 3 summarizes the details of the experimental program, including: column properties, material properties, instrumentation and test setup;
- Chapter 4 presents the results for the SFRC and UHPFRC columns tested in the experimental program;
- Chapter 5 discusses the influence of various parameters on the axial load performance of the SFRC and UHPFRC columns tested in the experimental program;

- Chapter 6 presents an analytical investigation aimed at evaluating the suitability of using existing models in the literature to predict the performance of the SCC, SFRC and UHPFRC columns tested in the experimental program;
- Chapter 7 presents the concluding remarks regarding the research program, and provides recommendations for future research.

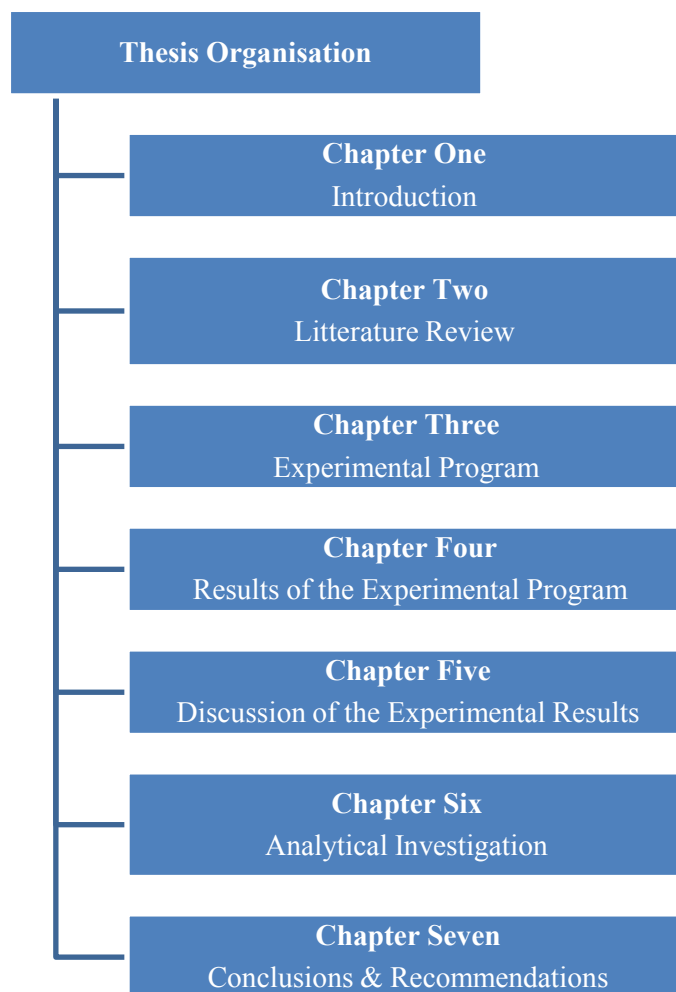


Figure 1.1 Thesis Organisation.

2 Literature Review

2.1 Chapter Overview

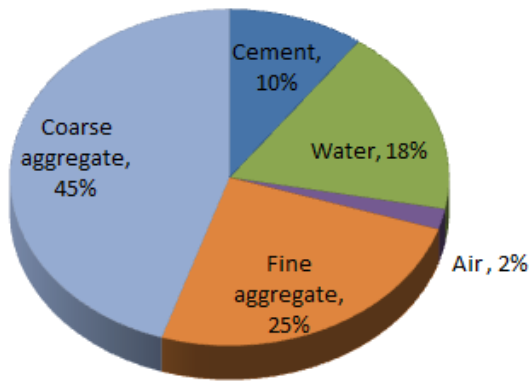
This chapter provides a literature review relevant to the three types of concrete materials studied in the experimental program: SCC, SFRC and UHPFRC. For each concrete type, material properties are briefly reviewed and previous experimental studies on columns are summarized. The chapter also reviews of the transverse reinforcement detailing requirements for columns in the CSA A23.3-04 standard and ACI 318-08 code.

2.2 Overview of SCC and Previous Research on SCC Columns

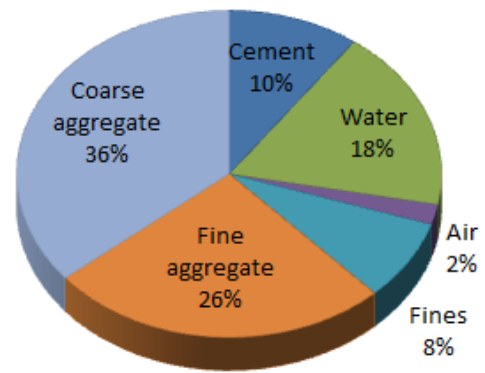
2.2.1 Introduction to SCC

In the 1980s reductions in available labour force and the construction of structures with increased complexity led to the development of self-consolidating concrete (SCC). SCC is defined as being a non-segregating, self-compacting concrete that can flow into formwork without the need for external vibration (Ozyildirim & Lane, 2003). SCC is made using the same constituents of plain concrete (cement, water, fine and coarse aggregates) but at different ratios (see - Figure 2.1). High workability is obtained by limiting the maximum size of coarse aggregate and addition of water reducing admixtures and super plasticizers. Viscosity modifying admixtures (VMA) are also typically used to enhance mix stability (Ozyildirim & Lane, 2003).

SCC is highly flow able and thus special testing methods such as the slump flow test, V-funnel, J-Ring and L-Box, are used in order to measure its required characteristics such as workability, stability and passing ability. A typical job site test is the slump flow test, where a slump cone is filled with SCC without vibration and then the cone is lifted and slump diameter (spread) is measured (see Figure 2.2). CSA A23.1/.2 defines requirements related to the use of SCC on construction projects. According to the CSA A23.1/.2, a diameter in range of 500-800 mm is required in order to accept SCC for construction (RMCAO, 2009). One important issue about SCC is that it should generally not be vibrated for consolidation. SCC is designed to be self-consolidating and therefore further vibration can result in segregation of the material.



(a) Regular Mix



(b) SCC

Figure 2.1 Sample Comparison of SCC and Normal Concrete Proportioning
Kosmatka et al (2002)



Figure 2.2 Sample Slump Test for SCC

2.2.2 Previous Research on SCC Columns Under Compression

Research has shown that properly detailed and closely spaced transverse reinforcement in columns can ensure ductile behaviour during earthquakes. As a result, modern design codes impose stringent requirements for the detailing of transverse reinforcement in columns. In regions of high seismicity, these requirements can result in highly congested sections that are difficult to construct. The use of self-consolidating concrete (SCC) can facilitate placement of concrete in these situations.

There have been a limited number of studies on SCC columns in the literature. Khayat et al. (2001) studied the axial load behaviour of normal-strength SCC columns. Later, Paultre et al. (2005) tested columns with similar properties but having higher compressive strength. Taking both series into account, the columns were 235 x 235 x 1400 mm in size and variables included compressive strength (f'_c , 40-87 MPa), as well as configuration, spacing and yield strength of transverse reinforcement (f_{yh} , 410-820 MPa). In total twelve columns were tested and the results were compared to companion specimens cast with conventional concrete. The results showed that the SCC columns had reduced axial capacity but improved ductility when compared to companion columns constructed with traditional concrete. As has been noted for traditional RC columns, confinement was more effective in columns constructed with normal-strength SCC when compared to high-strength SCC. The use of high-strength transverse steel was also found to be more effective in improving SCC column ductility when compared to SCC columns with normal strength transverse reinforcement. The authors noted that use of SCC ensured uniform placement of concrete in the heavily congested columns when compared to columns constructed with traditional concrete. They concluded that using SCC can reduce casting time and also ensure increased homogeneity of in-place properties along column height.

Lin et al. (2008), tested a series of SCC and normal-strength concrete (NC) column specimens under pure axial loading. The test series included 16 columns made with SCC, and 16 companion columns made with NC. The columns were 300 x 300 x 600 mm and variables included concrete strength (29-56 MPa), amount of longitudinal reinforcement (1.7% ~ 3.4%), as well as strength (340~560 MPa), volumetric ratio ($\rho_s f_{yh}$: 3.5 ~ 13.2

MPa), and configuration of transverse reinforcement. The experiments showed that the SCC columns had better crack control and ductility compared to the NC columns.

Bullo et al. (2009) studied the behaviour of large-scale SCC columns (dimensions of 220 x 220 mm x 1800 mm) detailed with three different levels of confinement reinforcement. The behaviour of the SCC columns was compared to a companion set of columns constructed with rheoplastic concrete (RpC). The results demonstrated that SCC had no distinct effect on cracking or ductility of the columns when compared to RpC.

Aoude et al. (2009) studied the axial load behaviour of thirteen columns constructed with SCC and steel fibres. The columns were 300 x 300 x 1200 mm in size and included four specimens constructed with plain SCC and nine specimens constructed with SCC and steel fibres. Hooked end steel fibres at fibre contents ranging from 0-2% by volume of concrete were added to the SCC mix, which had a compressive strength of 50 MPa. The confinement details were selected using the provisions of the CSA A23.3-04 Standard. The A-series columns were detailed in accordance with the requirements for "conventional construction" (ductility related force modification factor, $R_d = 1.5$), resulting in a tie spacing, s , of 240 mm, while the B-series and C-series columns were detailed in accordance with the requirements for "moderately ductile" ($R_d = 2.5$) and "ductile" ($R_d = 4.0$) columns, resulting in tie spacing of $s = 120$ mm, and $s = 65$ mm, respectively. The results confirmed that the use of closely spaced and well-detailed transverse reinforcement leads to enhanced load carrying capacity and ductility in SCC columns.

Table 2.1 summarizes the range of properties of the SCC columns in these studies. Appendix A presents a more detailed database of the column properties.

There is a need for further research on SCC columns in general, and SCC columns with rectangular cross-section in particular.

Table 2.1 Previous Tests on SCC Columns Reported in the Literature

	Number of specimens	Cross- section	Section Dimensions mm x mm	f'_c MPa	V_f (%)	Cover mm	d_b mm	F_{yb} MPa	d_h mm	F_{yh} MPa	s mm
Khayat et al. 2001	12	Square	235 x 235	40-87	--	20	12	469	6	410-820	60 -240
Paultre et al. 2005											
Lin et al. 2008	16	Square	300 x 300	28-56	--	30	##	552	10	330-560	60-150
Bullo et al. 2009	3	Square	220 x 220	40	--	##	14-20	533	10	##	50-150
Aoude et al. 2009	13*	Square	300 x 300	50	0-2.0	30	16	510	11.3	410	65-240

* 4 specimens constructed with SCC and 9 specimens constructed with SCC and steel fibres

= information not available

2.3 Overview of SFRC and Previous Research on SFRC Columns

2.3.1 Introduction to SFRC

Concrete has poor tensile strength and post-cracking resistance. The addition of steel fibres transforms this behaviour due to the ability of fibres to arrest and redistribute cracking. Modern fibre reinforced concrete (FRC) was first studied by researchers in the early 1960s (Li, 2002) and SFRC was introduced commercially for the first time in 1973 for use in pavements and the tunnel industry (Kobayashi, 1983).

Addition of fibres to concrete affects both hardened-state and fresh-state properties. In the hardened state, the ability of fibres to bridge across cracks and control cracking results in improvements in strength and ductility. On the other hand, from a fresh-state perspective, addition of fibres reduces workability which affects concrete placement. In this section, the fresh state and hardened-state properties of SFRC are reviewed.

2.3.2 Factors Affecting SFRC Fresh-State Behaviour

2.3.2.1 Workability and Maximum Fibre Content

Fibres are needle-shaped elements and have greater surface area when compared to aggregates. Furthermore, the stiffness of fibres allows them to push apart particles that are relatively large compared to the fibre length. Thus, factors such as the length, aspect-ratio and physical shape of the fibres affect the workability of concrete (Grünwald and Walraven 2010). The "maximum" fibre content is defined as the critical fibre content which ensures uniform fibre distribution. If this fibre content is exceeded, "balling-up" of fibres can occur, where fibres clump together, and segregation and non-uniform distribution of fibres results. Another important factor affecting uniform distribution of fibres is aggregate size. Figure 2.3 schematically depicts the effect of aggregate size on fibre – aggregate distribution; research has shown that as aggregate size is increased fibre distribution is affected and the maximum fibre content is reduced.

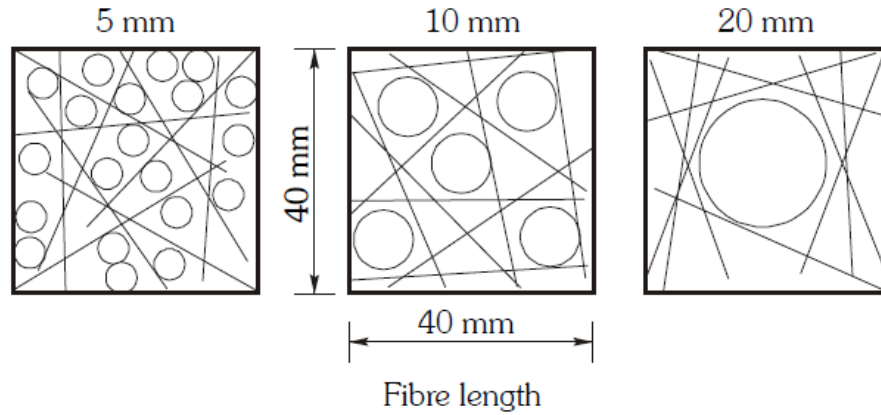


Figure 2.3 Effect of Aggregate Size on the Fibre Distribution
[Adopted from Grünewald and Walraven 2010]

2.3.2.2 *SFRC Passing-ability and Blocking Risk*

According to Grünewald and Walraven (2010), passing-ability is the ability of concrete to pass through the reinforcing cage and thus fill the framework homogeneously. This factor depends on several mixture properties including distribution, shape and maximum size of aggregate, as well as fibre properties and content.

2.3.2.3 *Combined Use of SCC and Steel Fibres (SCFRC)*

The addition of steel fibres to concrete results in reductions in workability and difficulties in concrete placement, particularly when higher fibre contents are used or in situations when SFRC is poured in heavily congested sections. Recently researchers have studied the possibility of using self-consolidating concrete (SCC) to enhance the workability of SFRC. When SCC is combined with steel fibres, self-consolidating fibre reinforced concrete (SCFRC) or highly-workable SFRC results (with the former term describing fully self-compacting SFRC, and the latter term being more appropriate for cases where high workability is achieved but SCC properties are not fully maintained). It is noted that even when SCC is used, there are limits to the quantity of fibres one can use. According to Grünewald (2004), fibre content should be limited to less than 2% to maintain adequate workability in SCFRC.

2.3.3 Mechanical Properties of SFRC:

The crack bridging mechanism in SFRC is dependent on several parameters including fibre properties, bond properties and fibre pull-out behaviour. These parameters are reviewed in the following subsections. In addition the behaviour of SFRC in tension and compression is briefly discussed.

2.3.3.1 Reinforcing Index

The term reinforcing index (R.I.), which is widely used by researchers, is defined as fibre content times the aspect ratio of fibres $[(V_f \text{ or } W_f) \times l_f/d_f]$. The reinforcing index is dependent on fibre content (either volume or weight fraction by volume of concrete) and fibre aspect ratio, which is function of length and diameter of the fibre (Nataraja et al. 1999, Aoude 2008). Fibre length and diameter typically vary between 10- 60 mm and 0.4-1 mm (Cohen, 2012). Shorter fibres influence early cracking in concrete and can result in strength increase, whereas longer fibres play a more important role in the post-cracking stage (Bhargava et al. 2006). For fibre content, even though volume fractions up to 3% have been reported in the literature, it has been suggested that fibre contents greater than 1.5% considerably decrease workability and can result in segregation, which can negatively affect mechanical properties (Aoude 2008). It should be noted that the "maximum" fibre content discussed previously is highly dependent on fibre properties (length, aspect ratio, etc.).

2.3.3.2 Fibre-Matrix Bonds

Fibre-matrix bond (τ_{bond}) relates to the frictional resistance between the fibre and the surrounding concrete matrix. Previous research shows that the fibre-matrix bond is a function of concrete strength, and as the concrete strength increases so does the bond strength. Typical bond strength values as a function of concrete strength are reported in Table 2.2 (Cohen, 2012). Other researchers have suggested that bond strength can be estimated based on concrete tensile strength (Marti et al 1999)

Table 2.2 Fibre-Matrix Bond Relations

Fibre Shape	Concrete Strength f'_c (MPa)	Fibre – Matrix Bond Shear Strength
		τ_{bond} (MPa)
Straight Fibre	$f'_c \leq 50$	2.0-3.0
	$50 \leq f'_c \leq 70$	3.4-4.5
	$70 < f'_c$	5.0-6.0
Hooked-End Fibre	---	$2f_t$ or $(0.6 f'_c)^{2/3}$

f_t =concrete tensile strength

2.3.3.3 *Fibre-Orientation Factor*

Fibres are randomly oriented in the matrix and therefore can have varying inclination angles relative to the cracking plane (see Figure 2.4). Researchers have proposed the use of “fibre orientation factors” to account for the random distribution of fibres in concrete. According to Dupont (2003), by assuming that a fibre is not limited by any boundary and can rotate freely in all directions (0 to 90 degrees), a constant value of 0.5 can be used. Based on research that suggests fibre efficiency reduces at inclination angles less than 30 degrees, Foster (2001) suggested a value of 3/8 by considering only fibres at angles 30 to 90 degree being effective.

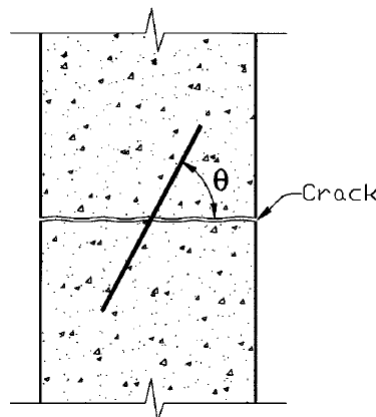


Figure 2.4 Inclination Angle of Fibres Relative to Cracking Plane
[Adopted from Foster 2001]

2.3.3.4 *Pull-Out Behaviour of Fibres*

The propagation of cracks in fibre reinforced concrete is delayed because of the ability of fibres to bridge across cracks. Generally, since the concrete matrix is weaker than the tensile strength of the fibres, fibres pull out rather than fracture across cracks (see Figure 2.5). The pull-out behaviour is directly related to the physical shape of the fibre. For straight (smooth) fibres, pull-out is characterized by two stages: deboning of the fibre from the surrounding concrete matrix followed by frictional pull-out (see Figure 2.6). Therefore for straight fibres this pull-out strength is only function of bond strength and the pull-out force can be taken as:

$$F_s = \tau_{bond} \cdot (\pi d_f) \frac{l_f}{2} \cdot \eta_l \quad \text{Equation 2-1}$$

Where, τ_{bond} is the fibre-matrix bond strength, d_f is the diameter of the fibre, l_f is the length of the fibre and η_l is a length factor which represents the average embedded length of the fibre in the matrix (the embedment of the fibre across the crack varies from 1.0 to 0, therefore a value of $\eta_l = 0.5$ can be taken to represent the intermediate case) (Aoude 2008).

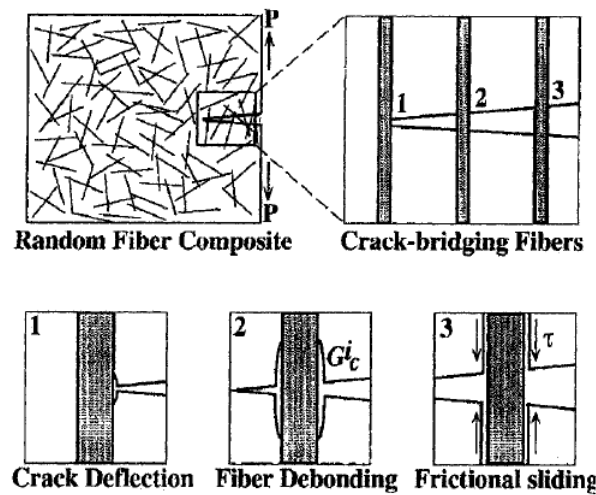


Figure 2.5. Fracturing Process and Toughening Mechanism in Brittle Composites
[Adopted from Tehrani 2008 - based on Easley et al. 1999]

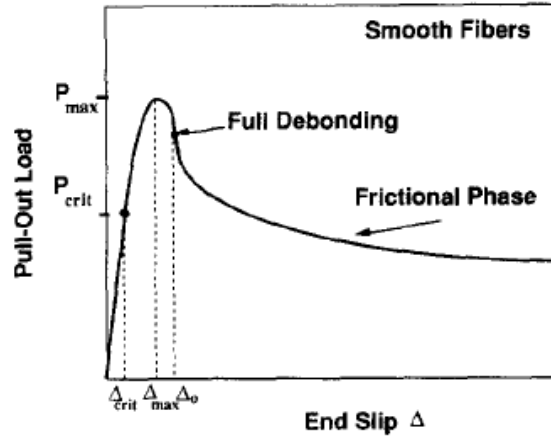


Figure 2.6. Pull-Out Load vs. Slip Curve for a Straight Fibre
[Adopted from Naaman et al. 1991]

Deformed steel fibers (e.g. hooked-end fibers) have superior pullout strength when compared to straight fibers, because of added “mechanical” pullout contributions provided by the anchorage of the fibers. Some authors suggest using a fibre factor(α_m) which scales the pull-out strength of straight fibres to account for improved pull-out resistance (see Equation 2-2 and Table 2-3).

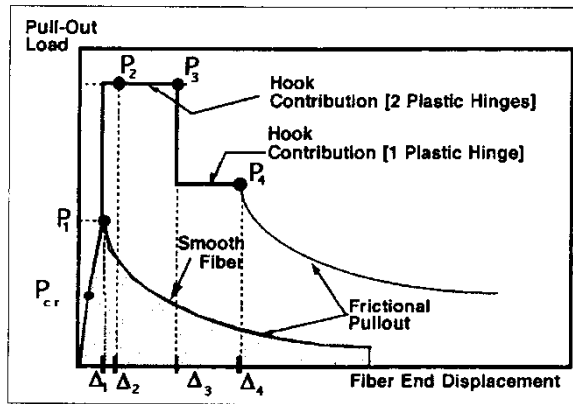
$$F_s = \alpha_m \tau_{bond} \cdot (\pi d_f) \frac{l_f}{2} \cdot \eta_l \quad \text{Equation 2-2}$$

Table 2.3 Examples of Fibre Shape Factor in Literature

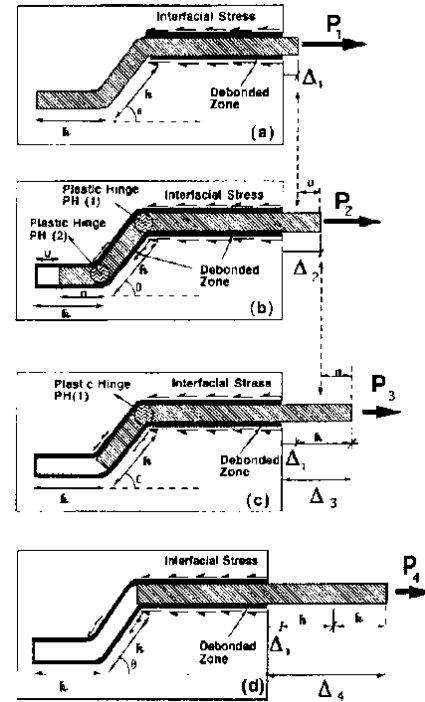
	Fibre Type	Fibre Shape Factor (α_m)
Cohen 2012	Crimped	2.84
	Hooked-end	2

For fibres that have hooked-ends, some models suggest an additional pull-out strength value (ΔP) due to straitening of the hooks. Alwan et al. (1999) mentions that the pull-out of hooked-end steel fibres is characterized by four distinct stages which include an initial stage where the fibre is bonded to the matrix, a second stage where the fibre-matrix bond is

gradually reduced until de-bonding, a third stage associated with the formation of plastic hinges required to straighten the hooks, followed by frictional pull-out of the straightened fibre (see Figure 2.7). Recent models (e.g. Laranjeira et al. 2010) consider the lateral pressure and shear force exerted on the fibre to cause plastic fibre deformation along the length of the fibre, in addition to the formation of plastic hinges at the hooks (see Figure 2.8).



(a) Pull-out load-displacement model



(b) Plastic hinging of the hooks

Figure 2.7. Pull-Out Process For Hooked-End Steel Fibres

[Adopted from Alwan et al . 1999]

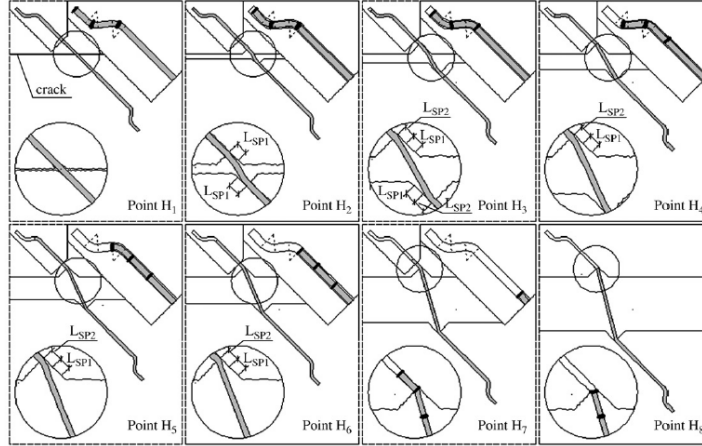


Figure 2.8. Pull-Out Process for Hooked-End Fibres with Plastic Deformation along Fibre Length
[Adopted from Laranjeira et al. 2010]

2.3.3.1 *Behaviour of SFRC in Tension*

Figure 2.9 schematically shows the response of concrete and SFRC under tensile stress. In the case of traditional concrete, when ultimate tensile capacity is reached first cracking occurs, and the tensile load path is “cut” and thus rapid rupture is observed. Addition of steel fibres helps concrete to maintain the load path through bridging of the cracks and thus post-cracking response is improved. The tensile stress-strain curve consists of an ascending branch up to first-cracking, and thereafter depending on fibre content, different post-peak behaviour is observed. According to Fantilli et al (2009), for SFRC having low fibre contents ($V_f < 1\%$), the post-cracking behaviour is characterized by strain-softening response, and as fibre content is increased ($V_f \geq 2\%$ or 1.5%) post-cracking response improves and strain-hardening behaviour becomes possible (where peak strength exceeds initial cracking strength).

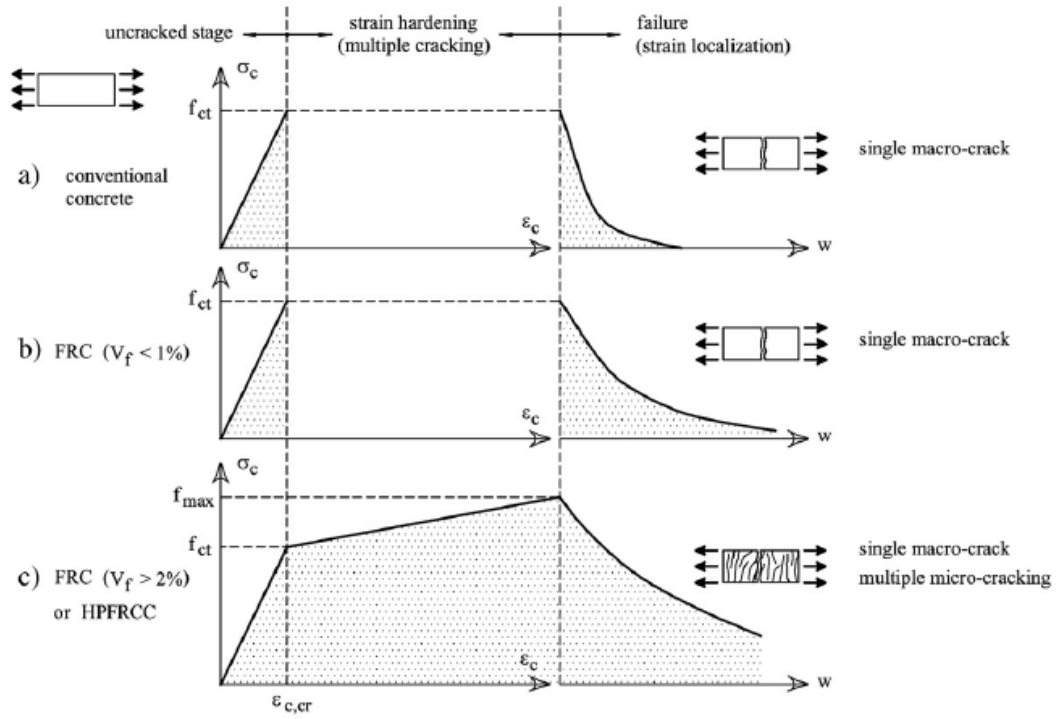


Figure 2.9. Variability of Tensile Behaviour of SFRC Depending on Fibre Content.

[Adopted from Fantilli et al. 2009]

2.3.3.2 *Behaviour of SFRC in Compression*

Previous research has shown that addition of steel fibres improves the compressive behaviour of concrete. In order to understand the mechanisms which lead to improved compressive response, the failure mechanism of concrete in compression can be examined.

The formation and propagation of micro-cracks is believed to be the cause of failure of concrete under compressive stresses. By comparing the crack-release energy (E_{cr}) with internal energy (E_{int}), failure is classified into three stages (Chen 1982, Hsu 1984). In the first stage, up to loading corresponding to 30% of ultimate capacity (f'_c) the stress-strain curve shows an approximately linear response. Prior to reaching 30% f'_c , E_{int} is less than E_{cr} , and existing cracks inside the concrete remain unchanged in length and width (these existing cracks occur before loading due to shrinkage of cement either by hydration, drying or carbonation).

For stresses above 30% f'_c the stress-strain behaviour shows a gradual increase in curvature up to 75% f'_c . During this second stage E_{int} reaches E_{cr} which initiates the onset of localized cracking (Chen, 1982). In between 30% f'_c -50% f'_c , bond cracks between the mortar and aggregates initiate and begin to extend due to stress concentration at the crack tips. For this stage the internal energy is approximately balanced with the needed crack-release energy ($E_{int} \cong E_{cr}$) and therefore crack propagation is stable (Chen, 1982). For stresses in between 50% f'_c -75% f'_c , cracks in the mortar are developed and crack propagation continues in a stable manner. At a stress of $\sim 75\%f'_c$ the internal energy exceeds the crack release energy ($E_{int} > E_{cr}$) and the system becomes unstable. The stress at this point is termed the *critical stress* and corresponds to the point where volumetric strain reaches its minimum value (see Figure 2.10). After the *critical stress* the concrete starts to dilate. In this stage propagation of mortar cracks accelerates, micro-cracks in the mortar join bond-cracks at the surface of nearby aggregates, and internal damage occurs and thus concrete passes its ultimate capacity. Figure 2.10 depicts the compressive behaviour of concrete under uniaxial compression; it is clear that first, compaction of concrete occurs, and as the critical stress is surpassed the concrete starts to dilate (Chen 1982, Hsu 1984).

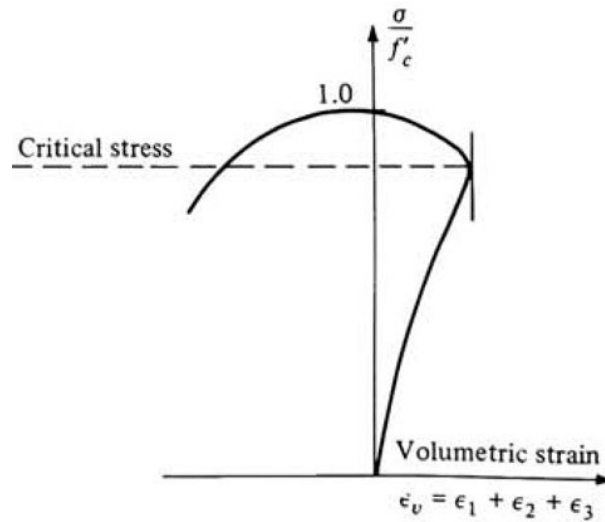


Figure 2.10 Uniaxial Behaviour of Concrete under Compression – Compaction and Dilatation
[Adopted from Chen 1982]

The described failure mechanism is affected by the existence of lateral pressure (confinement). Figure 2.11 shows the difference between uniaxial and biaxial loaded concrete. Lateral loading resists the dilatation of concrete and thus the critical strength of concrete increases. This means that concrete can be compacted more and therefore more volumetric strain is observed.

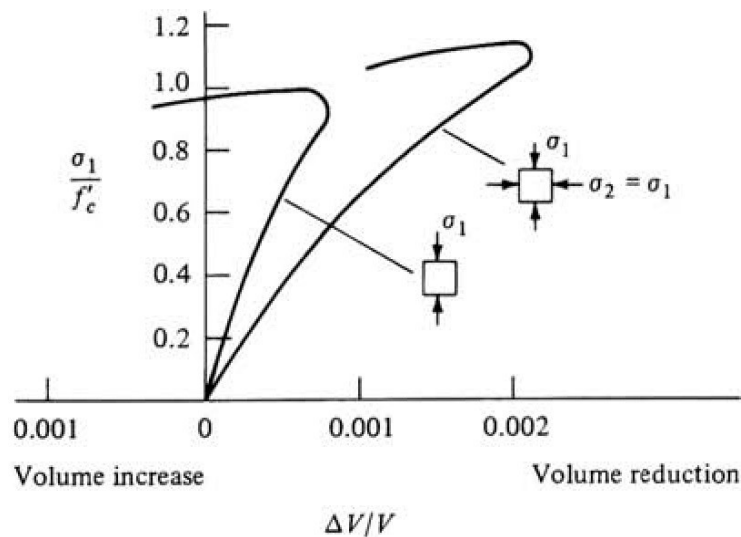


Figure 2.11 Bi-Axial Behaviour of Concrete under Compression – Compaction and Dilatation
[Adopted from Chen 1982]

The additional lateral pressure can result from either active confinement, as in the case of hydrostatic pressure or by passive confinement, as in the case of transverse reinforcement in columns. The level of confinement plays an important role in controlling this failure process. In columns with transverse reinforcement, confinement will vary depending on tie spacing and configuration (Saatcioglu and Razvi 1992). In the case of columns with low confinement, dilatation of the concrete occurs rapidly and no significant residual post-cracking response is expected. As the amount of transverse reinforcement increases so does the effective confinement, and therefore dilatation of the concrete is better controlled and improved post-cracking response is observed (Attard & Setunge, 1996).

Research has shown that addition of steel fibres affects the compressive behaviour of concrete (Nataraja et al. 1999, Bhargava et al. 2006, Ou et al. 2012). This research shows that fibres can improve peak strength and post-peak ductility. As failure progresses in concrete, fibres bridge the cracks and delay crack development which leads to improvement in response of the concrete. This improvement is affected by fibre properties (fibre length, aspect ratio and content). Regarding the findings of Bhargava et al. (2006), short (micro) fibres arrest smaller cracks that occur before peak capacity is reached and thus increase peak strength. Longer (macro) fibres are more effective after peak, when macro-cracks are developed, and therefore affect post-peak ductility, with a lesser effect on strength.

Examples of compressive response of SFRC reinforced with macro-fibres are shown in Figure 2.12. As it can be seen, post-peak response and ductility improves as fibres are added.

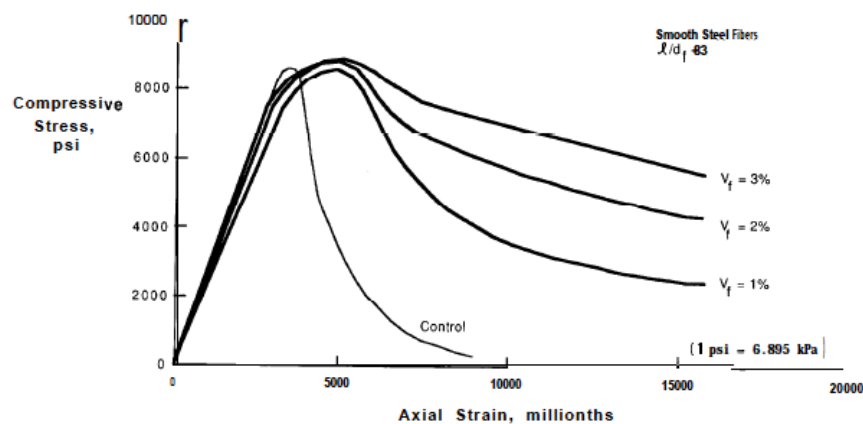


Figure 2.12 Typical Compressive Stress-Strain Response of SFRC
[Adopted from ACI-544 2008]

2.3.4 Previous Research on SFRC Columns Under Compression

Ganesan and Ramana Murthy (1990) tested eight 200 x 200 mm square-section short columns where four specimens were constructed with SFRC. Fibre content ($V_f = 0$ or 1.5%) was kept constant while transverse reinforcement spacing was varied in order to investigate the effect of confinement on strength and ductility. The results showed that addition of steel fibres to plain concrete improves peak strength and post-peak ductility in columns.

Massicotte et al. (1999) tested eighteen reinforced columns having square section. Twelve of the specimens had reinforcement and steel fibres. Among these specimens, six were cast without cover (170 x 170 mm section) and the remaining with cover (250 x 250 mm). The variables in this test series included fibre content, amount of transverse reinforcement and amount of cover. The results showed that adding fibres increases the ductility of columns and could be used to reduce required amount of transverse reinforcement, particularly in moderate seismic zones. Furthermore, the authors mentioned that although crack progression and cover failure was affected, addition of fibres did not significantly affect core confinement.

Foster and Attard (2001) tested twenty-one high-strength steel fibre reinforced columns having square section under both concentric and eccentric loading. Fibre content was kept constant in this study (2% by weight of concrete) with the main variables being loading and steel transverse reinforcement ratio. The authors of this study concluded that addition of fibres prevents cover spalling, increases load-carrying capacity and improves post-peak failure response.

Campione and La Mendola (2004) studied the behaviour of small-scale SFRC columns having square and circular cross-section. The base mix consisted of light-weight concrete composed of either pumice stone or expanded clay. Variables included fibre content, column height and transverse reinforcement spacing. The authors of this study concluded that addition of steel fibres improves the bearing capacity and transforms the brittle behaviour of light-weight concrete columns by increasing energy absorption capacity and post-peak ductility. Moreover they concluded that steel fibres can partially substitute for transverse

reinforcement. Finally they concluded that column height affects the compressive behaviour of SFRC columns.

Zaina and Foster (2005) tested 56 high-strength fibre reinforced concrete columns constructed with steel and polypropylene fibres under concentric or eccentric loading. They concluded that in contrary to polypropylene fibres, addition of steel fibres to concrete increases ductility and therefore steel fibres can partially substitute for the transverse reinforcement in highly congested columns. They further mentioned that addition of 2% steel fibres prevents early cover spalling in high-strength columns.

Sharma et al. (2006) tested twenty-eight SFRC short circular columns. In the test series fibres with two aspect ratios were used and fibre content was varied. They found early cover spalling was prevented in the SFRC columns. They further mentioned that when aspect ratio is kept constant, increase in fibre content results in improvement of strength and ductility. Conversely, when fibre content is kept constant, increasing fibre aspect ratio results in a reduction in strength gain, although relative toughness increases.

Sharma et al (2007) studied ninety-six square section high-strength steel fibre reinforced concrete columns under concentric loading. Variables in the test program included fibre content (1%, 1.5% and 2%), fibre aspect ratio (20 and 40), tie configurations and spacing, and concrete strength. As in previous studies the authors of this study found that early cover spalling is prevented by addition of fibres and although fibres increase strength, improvement of ductility is more important. They further mentioned that by keeping the concrete strength and tie spacing constant, strength and ductility increase with increasing fibre content and fibre effect is more important in columns with low amount of transverse reinforcement. Moreover, they suggested that short (micro) fibres have a more important effect on peak strength while long (macro) fibres improve column ductility, and therefore a blend of micro and macro fibres can be used for optimised improvement of column behaviour. In addition, they mentioned that as concrete strength is increased, ties and fibres become less effective and therefore, either fibre content or tie confinement should be increased.

Djumbong et al. (2008) tested twelve large-scale high-strength fibre reinforced concrete columns having circular cross-section and spiral transverse reinforcement under concentric loading. Test variables included concrete strength, spiral spacing and volume fraction of synthetic fibres. As reported in the literature, reducing spacing of transverse reinforcement was found to increase strength and ductility in columns, with ductility reducing as concrete strength increases. Furthermore, although the authors noted fibres can be used to increase spiral pitch, synthetic fibres may not be effective at large deformations.

Paultre et al. (2010) tested twelve large-scale high-strength steel fibre reinforced concrete columns having square cross-section under concentric loading. Test variables included fibre content, as well as configuration, yield strength and spacing of transverse reinforcement. The authors of this study found that addition of fibres prevents premature cover spalling in high-strength concrete columns and results in improvements in column strength and ductility. They further developed an analytical model (see chapter 6).

A brief summary of the aforementioned studies is presented in Table 2.4. Appendix B presents a more detailed database of the column properties.

Table 2.4 Previous Tests on SFRC Columns Reported in the Literature

	Cross-section Shape	Section Dimensions mm x mm	f'_c MPa	V_f (%)	Cover mm	d_b mm	F_{yb} MPa	d_h mm	F_{yh} MPa	s mm
Ganesan & Murthy 1990	Square	200x200	---	1.5	25	12	469	6	469	60 ~240
Massicotte et al. 1999	Square	170x170 250x250	36 ~ 43	0.5~ 1	40	16	525	11.3	475	63 ~250
Foster & Attard 2001	Square	155x155 200x200	70 ~ 90	2	30	12.2	450	4 ~ 6.1	470 ~600	30 ~100
Campione & La Mendola 2004	Square	100x100	21.7	0 ~ 2	7.5	4	520	4	520	20~ 40
	Circular	100	20 ~ 22	0 ~ 2	7.5	4	520	4	520	25~ 50
Zaina & Foster 2005	Square	190x190 200x200	86 ~ 91	1 ~ 2.6	20	12 ~ 16	560 ~ 576	4 ~ 6	660	50 ~150
Sharma et al. 2006	Circular	150	82 ~ 83	0 ~ 2	10	8	412	8	412	30 ~ 75
Sharma et al. 2007	Square	150x150	62 ~ 83	0 ~ 2	10	12	395	8	412 ~520	30 ~ 75
Djumbong et al. 2008	Circular	300	67 ~ 88	0 ~ 1	25	19.5	494	11.3	459	50 ~100
Paultre et al. 2010	Square	235x235	88 ~ 100	0 ~ 1	20	14 ~ 18	405 ~ 524	8~9.5	392 ~856	50 ~100

Note: d_b and F_{yb} represent diameter and yield strength of longitudinal reinforcement; d_h , F_{yh} and s represent diameter, yield strength and spacing of transverse reinforcement.

Note: 1 MPa = 0.145 ksi, 1 mm = 0.0394 in.

2.4 Overview of UHPFRC and Previous Research on UHPFRC Columns

2.4.1 Introduction to UHPFRC

Ultra-high performance fibre reinforced concrete (UHPFRC) is a new class of concrete material with high compressive strength, high tensile resistance and increased durability. The components of UHPFRC include cement, silica fume, quartz flour, fine silica sand, water, admixtures and steel fibres. High strength ($f_c \geq 150$ MPa) results from low water-cement ratio ($w/c \leq 0.2$), introduction of extremely fine particles (such as silica fume) and careful mix proportioning that ensures dense particle packing. Tensile post-cracking resistance and toughness is achieved through addition of short (micro) steel fibres at fibre contents ranging from 2-6% by volume of concrete. Despite low water-cement ratio and provision of fibres, high workability and self-compacting properties are achieved by using super plasticizers. Examples of commercial concretes in this class include Compact Reinforced Composite (CRC), produced by CRC Technology (Denmark) and Reactive Powder Concrete (RPC) produced by Lafarge under the *Ductal* trademark.

2.4.2 Compressive Behaviour of UHPFRC

Nielsen (1995) studied the compressive behaviour of UHPFRC and examined the effect of reinforcing index ($V_f \frac{l_f}{d_f}$) on response. In this study Compact Reinforced Composite (CRC) was used as the base mixture. Without provision of fibres, the material was shown to have brittle and explosive response, with linear behaviour up to peak followed by failure. By addition of fibres capacity increased, the ascending branch became more non-linear and post-peak failure became more stable and less brittle. Nielsen (1995) showed that the parameters of the stress-strain curve (peak stress, peak strain, modulus of elasticity, etc.) are directly related to the reinforcing index and the effect can be considered using the following linear relationship:

$$X_f = X + a \left(V_f \frac{l_f}{d_f} \right) \quad \text{Equation 2-3}$$

Where "X" is the parameter being studied (stress, strain, etc.) for the case without fibres and " X_f " is parameter after addition of fibres, "a" is a constant depending on the parameter being studied, while $V_f \frac{l_f}{d_f}$ represents the reinforcing index (fibre content times fibre aspect ratio). Nielsen found that increasing the reinforcing index, results in increases in peak stress and overall toughness (represented by area under the stress-strain curve), but does not significantly affect the post-peak branch of the stress-strain curve.

Curing procedure has an important effect on the compressive strength of UHPFRC, and higher strengths can be achieved using special curing procedures. Graybeal (2006, 2007) studied the performance of the proprietary UHPFRC, *Ductal*, under various curing conditions: steam curing (at varying temperatures and durations) and under laboratory (ambient) conditions. In the test program over 1000 specimens were tested. Based on this study Graybeal concluded that steam curing significantly increases compressive strength, along with positive effects on long-term durability. He also noted that the compressive strength of steam-cured UHPFRC is effectively stabilized after 48 hours, while UHPFRC treated under ambient conditions shows strength gain for at least 8 weeks after casting. An equation for the strength gain in UHPFRC under normal laboratory conditions was also proposed. In addition, Graybeal found that the linearity of ascending branch of the stress strain curve depends on treatment method and the relationship becomes more linear in the case of steam-treated UHPFRC. Based on this observation, a linear model was proposed for the ascending branch of the stress-strain curve. Graybeal also investigated the effect of cylinder and cube size on measured compressive strength and reported that results were similar for all specimen types. He also indicated that loading rate between 0.24 and 1.74 MPa/sec did not have a significant effect on compressive behaviour.

2.4.3 Previous Research on UHPFRC Columns

There is limited published research in the literature on the performance of UHPFRC columns. Sugano et al (2007) tested nine UHPFRC columns with square section (200 mm x 200 mm) which were reinforced with high strength steel ties. The main variables in this study included concrete strength (120, 160, 200 MPa), fibre content (0 or 2%) as well as strength (700 MPa and 1400 MPa) and spacing of transverse reinforcement (35, 45, 55 mm).

For the case of the fibre-reinforced specimens the authors of this study observed increase in confined strength when compared to cylinder strength as the amount/strength of transverse reinforcement ($\rho_s f_{yh}$) increased, while the increase in strength was negligible in the case of the columns without fibres. Compression ductility was discussed in terms of ratio of peak confined strain and peak unconfined strain and it was found that ductility improved with increase in the amount/strength of transverse reinforcement. Behaviour of UHPFRC under simulated earthquake loading was also studied by testing six companion columns under reverse-cyclic loading. The results showed that an increase in the level of confinement and provision of fibres improved the ductility of the columns under lateral loads. In addition to the column tests several mechanical properties of UHPFRC were studied using small-scale specimens tested under compression, tension and bending. For the cylinder tests the authors reported explosive and brittle failure in the case of the plain specimens, while provision of fibres prevented explosive failure and allowed for some improvement in post-peak response. The authors of this study further proposed simple relationships for the stress-strain curves of UHPFRC in unconfined compression and tension.

Empelmann et al. (2008) reported results of twelve UHPFRC columns having square section of 200 x 200 mm tested under axial loading. The variables in the test program included configuration and spacing of transverse reinforcement. Very little details were given regarding the test results, however the authors noted that ductile performance can be achieved in UHPFRC columns if adequate transverse steel reinforcement and steel fibres are provided. The authors also noted that provision of fibres prevented cover spalling in the columns. In addition, the study proposed design guidelines based on the experimental and further numerical investigations. The authors also discuss a case study where traditional concrete and UHPFRC are considered for columns in a 42-story high-rise building. In the study the cross-section was limited to 762 mm and the column was to carry a design axial load of 30,000 kN. Using regular concrete the design required use of a steel-concrete composite section, while use of UHPFRC allowed for a column with regular reinforcement, resulting in savings in material and construction costs.

Heshe and Nielsen (1992) studied the compressive behaviour of UHPFRC through several triaxial-stress tests on cylinders and uniaxial tests on short columns. Triaxial tests were

conducted on twenty-four cylinder specimens under varying lateral pressures (0, 20, 140 MPa) to examine the suitability of the Coulomb criterion. In the second part of the experimental program twenty-four small-scale rectangular section (130 x 158 mm) columns were tested under axial load. The test variables included transverse reinforcement spacing and reinforcement configuration. The authors of this study concluded that although confinement influences axial capacity in UHPFRC, it does not significantly enhance ductility. In addition they mentioned that direction of concrete placement has a noticeable effect on capacity due the influence of fibre orientation, with the capacity being higher when concreteing is parallel relative to the loading axis.

The test parameters in the aforementioned studies are briefly summarized in Table 2.5 .

Table 2.5 Previous Tests on Steel Reinforced UHPFRC Columns Reported in the Literature

	Cross-section Shape	Section Dimensions mm x mm	f'_c MPa	V_f (%)	Cover mm	d_b mm	F_{yb} MPa	d_h mm	F_{yh} MPa	s mm
Heshe and Nielsen 1992	Rectangular	130x158	173~184	---	---	16~25	550	5	550	31~620
Sugano et al 2007	Square	200x200	159~222	0~2	10	10	685	6.8	700~1400	35~55
Empelmann et al 2008	Square	200x200	150~160	0~1.25	---	14~28	500~850	8	---	41~84

2.5 Review of Code Requirements for Detailing of Reinforced Concrete Columns

Research has shown that the detailing of transverse reinforcement in columns plays an important role in ensuring adequate performance during earthquakes. It is now well known that ductility is influenced by the configuration and spacing of the transverse reinforcement. Consequently modern codes such as the CSA A23.3 standard and the ACI 318 code impose various requirements for the detailing of this reinforcement in columns.

The detailing requirements in the CSA A23.3-04 standard are function of the design ductility level, taken into account using the ductility-related force modification factor (R_d). This factor reflects the capability of a structure to dissipate energy through inelastic behaviour. The requirements are summarized briefly in Table 2.6.

Clause 7.6.5.2 clarifies the limitation for tie spacing in columns in frames designed using *conventional construction* ($R_d=1.5$), and specifies that tie spacing shall not exceed the following:

$$\text{Cl.7.6.5.2} \qquad \min\{16d_b, 48d_h, b, 300 \text{ mm}\} \qquad \text{Equation 2-4}$$

For specified concrete compressive strengths exceeding 50 MPa, the determined tie spacing shall be multiplied by 0.75.

For columns in *Moderately ductile* ($R_d=2.5$) moment resisting frames, clause 21.7.2.2.3, clarifies that ties should be provided at both ends of the columns over a length equal to the largest of one-sixth of the clear height, the maximum cross-sectional dimension, or 450 mm, with a spacing not exceeding the following:

$$\text{Cl.21.7.2.2.3} \qquad \min\{8d_b, 24d_h, b/2\} \qquad \text{Equation 2-5}$$

In addition, clause 21.7.2.2.5 clarifies that columns that can develop plastic hinges (because of their connection to rigid members such as foundations or discontinued walls or because of their position at the base of the structure) shall be provided with transverse reinforcement

over their clear height. In addition the total effective area in each of the principal directions of the cross-section, A_{sh} within spacing s , shall not be less than the following (for rectangular columns):

$$\text{Cl.21.7.2.2.5} \quad A_{sh} > \max \left\{ 0.15 k_n k_p \frac{A_g}{A_{ch}} \frac{f'_c}{f_{yh}} s h_c, 0.09 \frac{f'_c}{f_{yh}} s h_c \right\} \quad , \quad \text{Equation 2-6}$$

where $f_{yh} < 500 \text{MPa}$

Where

$$k_n = \frac{n_l}{n_l - 2} \quad \text{Equation 2-7}$$

$$k_p = \frac{P_f}{P_o} \quad \text{Equation 2-8}$$

For columns in *Ductile* moment resisting frames ($R_d=4.0$), clause 21.4.4.2 mentions that the A_{sh} and spacing requirements below shall be met unless a larger amount of transverse reinforcement is required either by Equation 2-10 (clause 21.4.4.3) or by shear demand:

$$\text{Cl. 21.4.4.2} \quad A_{sh} > \max \left\{ 0.2 k_n k_p \frac{A_g}{A_{ch}} \frac{f'_c}{f_{yh}} s h_c, 0.09 \frac{f'_c}{f_{yh}} s h_c \right\} \quad , \quad \text{Equation 2-9}$$

where $f_{yh} < 500 \text{MPa}$

$$\text{Cl.21.4.4.3} \quad \text{Spacing} < \min \left\{ 6d_b, \frac{b}{4}, s_x = 100 + \left(\frac{350 - h_x}{3} \right) \right\} \quad \text{Equation 2-10}$$

The provisions in the ACI 318-08 codes are similar, with detailing dependant on seismic design category (SDC). Requirements are specified for three classes: ordinary, intermediate and special moment frame members. A summary of the requirements is provided in Table 2.7.

Table 2.6 CSA A23.3 2004 Transverse Reinforcement Spacing Requirements

	R_d	Requirements
Conventional construction moment resisting frames	1.5	$s < \min\{16d_b, 48d_h, b_{min}, 300 \text{ mm}\}$
Moderately ductile moment resisting frames	2.5	$s < \min\left\{8d_b, 24d_h, \frac{b_{min}}{2}\right\}$ $A_{sh} > \max\left\{0.15 k_n k_p \frac{A_g}{A_{ch}} \frac{f'_c}{f_{yh}} sh_c, 0.09 \frac{f'_c}{f_{yh}} sh_c\right\},$ $f_{yh} < 500 \text{ MPa}$
Ductile moment resisting frames	4.0	$A_{sh} > \max\left\{0.2 k_n k_p \frac{A_g}{A_{ch}} \frac{f'_c}{f_{yh}} sh_c, 0.09 \frac{f'_c}{f_{yh}} sh_c\right\},$ $f_{yh} < 500 \text{ MPa}$ $s < \min\left\{6d_b, \frac{b_{min}}{4}, s_x = 100 + \left(\frac{350 - h_x}{3}\right)\right\}$

Table 2.7 ACI 318 2008 transverse reinforcement spacing requirements

	Clauses	
Ordinary moment frames	7.10.5.2	$s < \min\{16d_b, 48d_h, b_{min}\}$
Intermediate moment frames	21.3.5.2	$s < \min\left\{8d_b, 24d_h, \frac{b_{min}}{2}, 300\right\}$
Special moment frame members subjected to bending and axial load	21.6.4.3	$s < \min\left\{6d_b, \frac{b_{min}}{4}, s_x\right\},$ $100 \text{ mm} < s_x = 100 + \left(\frac{350 - h_x}{3}\right) < 150 \text{ mm}$
	21.6.4.4	$A_{sh} > \max\left\{0.3 \left(\frac{A_g}{A_{ch}} - 1\right) \frac{f'_c}{f_{yh}} sh_c, 0.09 \frac{f'_c}{f_{yh}} sh_c\right\}$

2.6 Research Needs and Objectives

As discussed in literature review, while research has been conducted on the behaviour of SCC columns, published research on the behaviour of SCC columns with rectangular sections is rare. Research has shown that provision of fibres (SFRC) changes the behaviour of concrete under compression, however there is a need to examine the specific contribution of fibres to the mechanisms governing the behaviour of columns under axial loading. In addition, to the best knowledge of the author, there is no research on the behaviour of SFRC columns having rectangular section.

In the current research program, considering the behaviour of SCC and SFRC, several parameters have been chosen in order to clarify the compressive behaviour of columns constructed with these materials. For this purpose, part of specimens are designed with steel reinforcement and concrete cover with various tie spacing and fibre content and the second group of columns are designed without any concrete cover with the same reinforcement configuration and fibre content . By considering various amount of tie spacing combined with varying fibre content, it was possible to investigate the individual contribution of these parameters to the improvement of the compressive behaviour of columns. Furthermore, introduction of specimens without cover facilitated the study of the effect of fibres and tie spacing on “core concrete” and “cover concrete” individually so that the failure phenomenon can be explained in detail.

Since UHPFRC is a new material, its behaviour in columns having traditional steel confinement reinforcement is not quite clear. In the current research program, the configuration of transverse reinforcement and the amount of tie spacing in columns were varied in order to study the compressive behaviour of UHPFRC under confinement.

Finally, there is a need to verify if existing models for compressive behaviour of normal and high strength concrete are applicable to SCC, SFRC and UHPFRC columns.

In conclusion, the current research aims to further the knowledge on the compressive behaviour of SCC, SFRC and UHPFRC materials in columns both with experimental and analytical methods and thus investigate and contribute to the current literature.

3 Experimental Program

3.1 Introduction

This research study was undertaken in order to study the axial load behaviour columns constructed with steel fibre reinforced concrete (SFRC) and ultra-high performance fibre reinforced concrete (UHPFRC). This chapter summarizes the details of the research program, including specimen and material properties and an overview of construction, instrumentation and testing procedure.

3.2 Description of Test Specimens

As part of the research program, a total of twenty three full-scale columns were tested under pure axial loading. Seventeen columns were constructed with self-consolidating concrete (SCC) and reinforced with steel fibres, resulting in highly workable SFRC. In addition, six UHPFRC columns made of compact reinforced composite (CRC) were also constructed. Table 3.1 summarizes the properties of the various specimens.

The columns in the SFRC series had rectangular cross-section and the variables included fibre content and spacing of transverse reinforcement. The columns were constructed with self-consolidating concrete (SCC) and reinforced with steel fibre at contents ranging from 0% to 1.5% by volume of concrete. All columns had a height of 1200 mm, with nine of the columns having a cross-section 400 mm x 225 mm and eight of the specimens having a cross-section of 375 mm x 175 mm. The columns had the same core area, however a 30 mm clear cover was used in the first set of columns, while the remaining columns were constructed without cover (a minimal 5 mm cover was used to facilitate placement of the reinforcing cages in the formwork). The longitudinal reinforcement consisted of 8–15M reinforcing bars ($d_b = 16$ mm and $A_s = 200$ mm²) while the transverse reinforcement was provided by 10M double hoops ($d_b = 11.3$ mm and $A_s = 100$ mm²), anchored with seismic hooks (to ease construction, the interior "hoops" were fabricated using two interior cross-ties). The SFRC columns in this test program were divided into three series, with transverse reinforcement detailing corresponding to the three ductility levels specified in the CSA A23.3-04 Standard (CSA 2004). The AR-series columns were detailed in accordance with the basic confinement provisions of the CSA A23.3-04 Standard, for columns having a

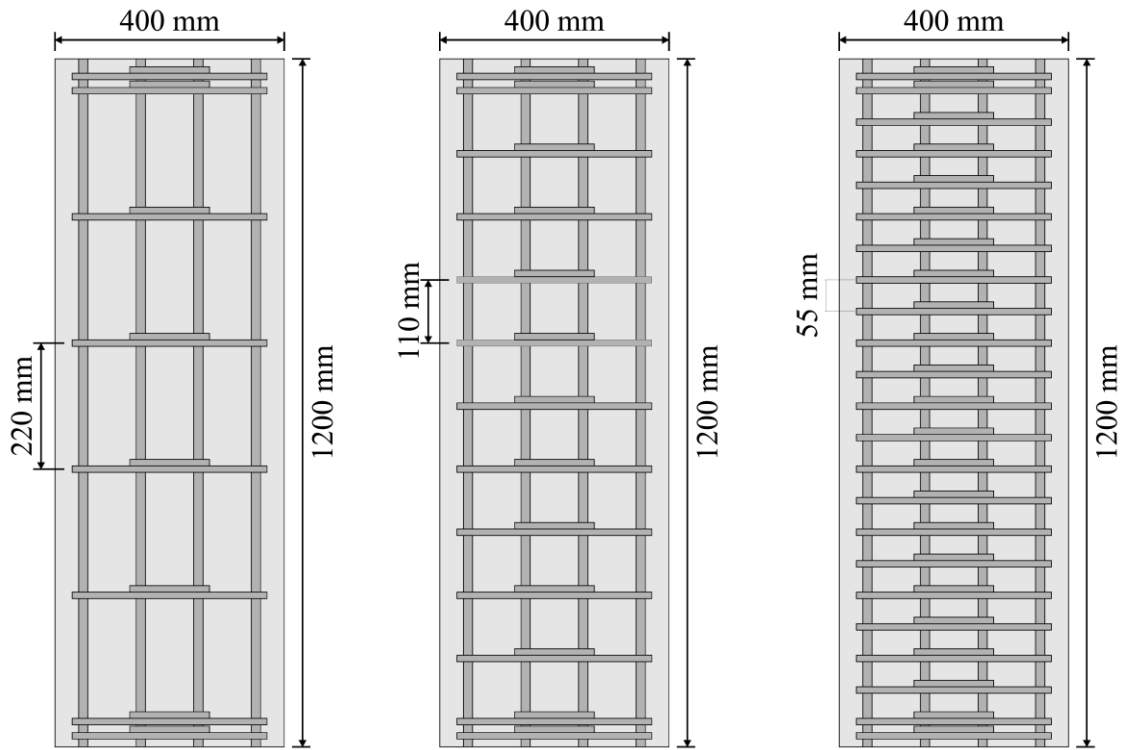
ductility-related force modification factor, R_d of 1.5 (conventional construction). The spacing, s , of the 10M hoops was governed by the bar buckling requirements, resulting in a spacing of 220 mm. The requirements for columns having R_d of 2.5 (moderately ductile columns) were used to detail the BR-series specimens and resulted in $s = 110$ mm. Finally, the CR-series specimens were detailed with the provisions for ductile columns ($R_d = 4.0$) and had a hoop spacing $s = 55$ mm. In all cases, the hoop spacing was extended over the full height of the columns, with additional hoops provided at the column ends. The specimen details are summarized in Table 3.1 and Figure 3.1. The specimen nomenclature reflects the test variables in the test series: ductility level (AR, BR, CR) and fibre content (0%, 1%, 1.5%). For example specimen AR0 had hoop spacing of 220 mm and was constructed with plain SCC, while specimen BR1 had hoop spacing of 110 mm and was constructed with SCC and 1% steel fibres. The specimens without cover are identified with the designation "nc".

The six UHPFRC columns in the experimental program were constructed with compact reinforced composite (CRC), and were reinforced with steel fibres at a fibre content of 2.5%. The columns had a total height of 1000 mm, with a cross-section of 250 mm x 250 mm and a clear cover of 10 mm, and the variables included configuration and spacing of the transverse reinforcement. In configuration C3, the longitudinal reinforcement consisted of 8–15M reinforcing bars, while the transverse reinforcement consisted of 10M hoops and cross-ties, anchored with seismic hooks. In configuration C4, the longitudinal reinforcement consisted of 12–15M reinforcing bars, while the transverse reinforcement consisted of 10M double hoops, anchored with seismic hooks (to ease construction the interior "hoops" were fabricated using two interior cross-ties). Specimens C3-120 ($s=120$ mm) and C4-120 ($s=120$ mm) were detailed to meet the requirements for moderately ductile columns ($R_d=2.5$) in the CSA A23.3-04 Standard. The requirements for ductile columns ($R_d=4.0$) were used to detail specimens C3-40 ($s= 40$ mm) and C4-60 ($s= 60$ mm). The remaining specimens had intermediate spacing between these values to investigate the effect of hoop spacing on confinement in UHPFRC. It is noted that specimen nomenclature reflects configuration and spacing of transverse reinforcement (for example specimen C3-80 had configuration

corresponding to C3, with $s = 80$ mm). Details of the specimens are shown in Table 3.1 and Figure 3.2.

Table 3.1 Detail of Experimental Specimens

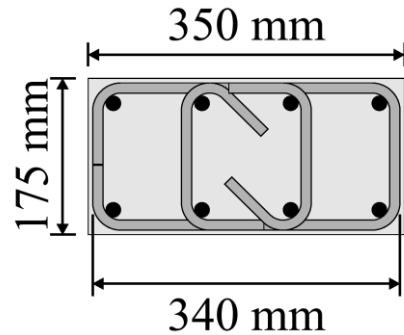
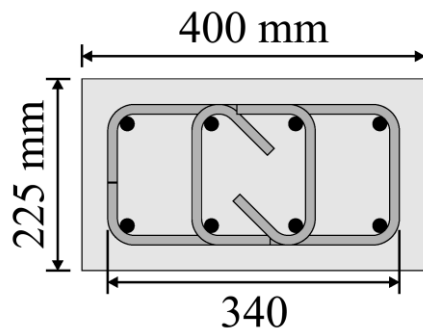
Series	Columns	Concrete Mix	Cross-section (mm)	Tie Spacing (mm)	Cover	Longitudinal Reinforcement	Transverse Reinforcement	Fibre Type	Fibre Content (%)
SCC & SFRC	AR0	KING SCC	400 x 225	220	30	8 – 15 M	10 M (4 legs in wider dimension, 2 legs in shorter dimension)	Dramix ZP-305 Hooked end steel fibres	0
	AR1			220					1
	AR15			220					1.5
	BR0			110					0
	BR1			110					1
	BR15			110					1.5
	CR0			55					0
	CR1			55					1
	CR15			55					1.5
	AR0nc		350 x 175	220	5	8 – 15 M	10 M (4 legs in wider dimension, 2 legs in shorter dimension)		0
	AR1nc			220					1
	AR15nc			220					1.5
	BR0nc			110					0
	BR1nc			110					1
	BR15nc			110					1.5
	CR0nc			55					0
	CR1nc			55					1
UHPRC	C3-120	CRC Joint Cast	250x250	120	10	8 – 15 M	10 M (3 legs)	Stratec 13/0.3 Smooth steel fibres	2.5
	C3-80			80					2.5
	C3-60			60					2.5
	C3-40			40					2.5
	C4-120			120		12 – 15 M	10 M (4 legs)		2.5
	C4-60			60					2.5



(a)AR Series

(b)BR Series

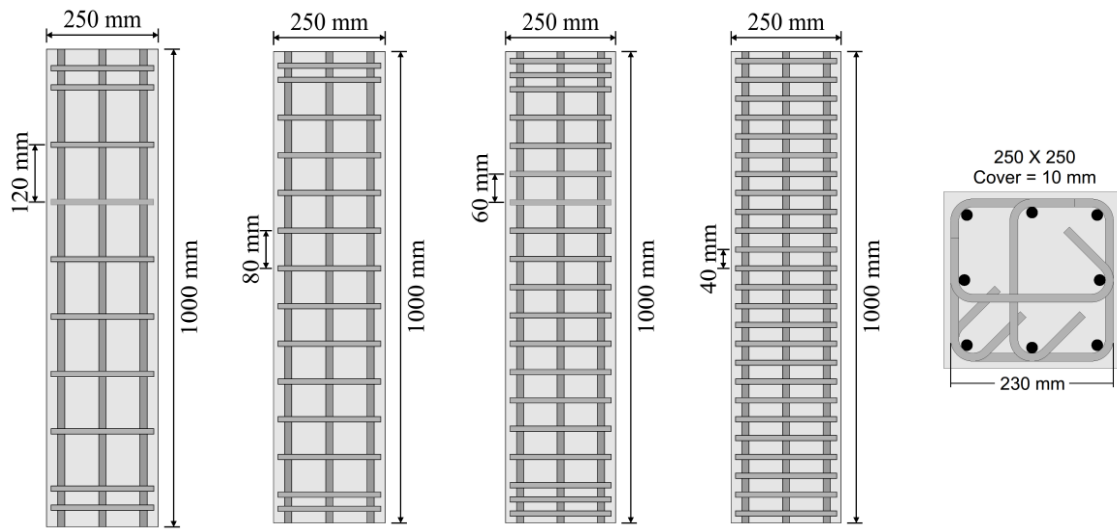
(c)CR Series



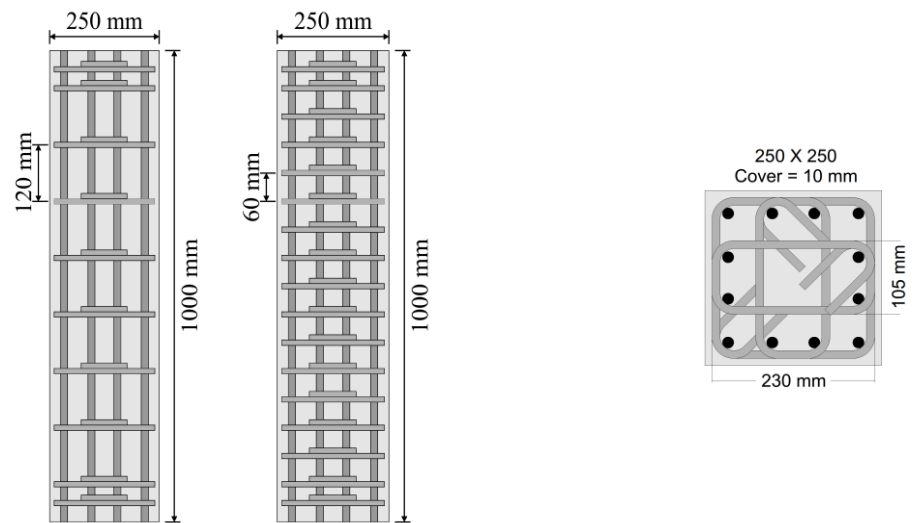
(d)Cross-Section for Specimens With
Cover

(e) Cross-Section for Specimens “Without
Cover”

Figure 3.1 Column Details (SFRC Test Series)



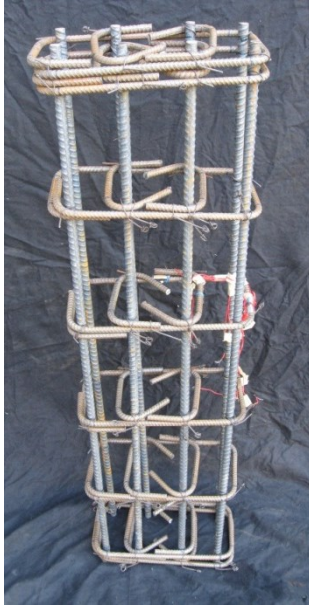
(a)C3-120 (b)C3-80 (c)C3-60 (d)C3-40 (e) Cross-Section
Details for C3
Series



(f)C4-120 (g)C4-60 (h) Cross-Section
Details for C4
Series

Figure 3.2 Column Details (UHPFRC Test Series)

Typical cages after construction and in the formwork for the SFRC and UHPFRC test series are shown in Figure 3.3 and Figure 3.4, respectively.



(a)AR Series



(b)BR Series



(c)CR Series

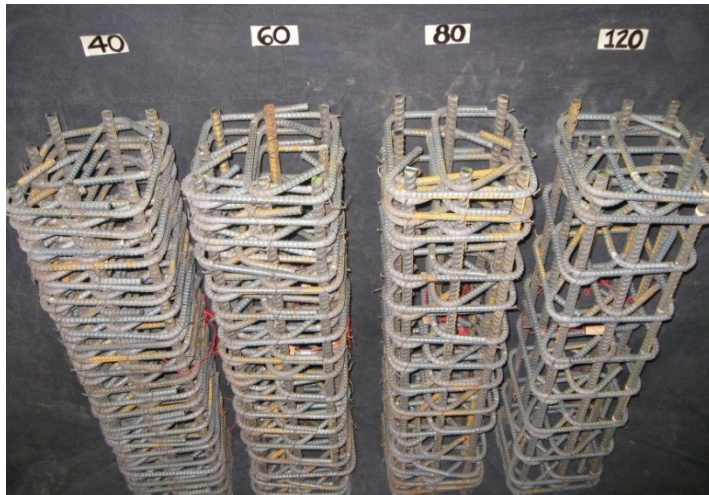


(d)Specimens “With Cover” in Formwork



(e) Specimens “Without Cover”
in Formwork

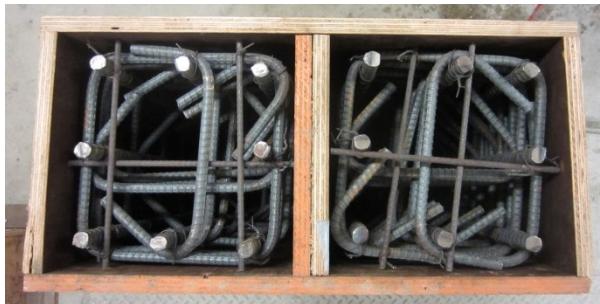
Figure 3.3 Reinforcing Cages and Formwork for the SFRC Test Series



(a) C3-Series Column Cages



(b) C4-Series Column Cages



(c) UHPFRC Specimens in Formwork
(Top View)



(d) UHPFRC Specimens in
Formwork (Side View)

Figure 3.4 Reinforcing Cages and Formwork for the UHPFRC Test Series

3.3 Material Properties

3.3.1 Reinforcing Steel

The reinforcement that was used in construction of specimens was weldable grade steel. The longitudinal reinforcing were size 15M ($d_b = 16$ mm and $A_s = 200$ mm²), while the transverse steel ties were fabricated from 10M bars ($d_b = 11$ mm and $A_s = 100$ mm²). Standard tensile coupon tests were performed on three samples for both the 15M and 10M bars. It is noted that two batches of 10M and 15M were used to fabricate the specimens (bars having designation 10M-1 and 15M-1 were used in the SFRC test series; bars with designation 10M-2 and 15M-2 were used in the UHPFRC columns). Table 3.2 summarizes the average properties of the reinforcing steel bars, including strain and stress at yield (ϵ_y , f_y), strain at the onset of strain hardening (ϵ_{sh}), as well as strain and stress at ultimate (ϵ_u , f_u). Typical stress-strain relationships are shown in Figure 3.5 and Figure 3.6. As it is clear from Figure 3.5, the 10M-1 rebar does not show a yield plateau and the stress gradually increases up to rupture.

Table 3.2 Properties of Reinforcing Steel

Bar#	f_y (MPa) [S.D.]	ϵ_y (mm/mm) [S.D.]	ϵ_{sh} (mm/mm) [S.D.]	f_u (MPa) [S.D.]	ϵ_u (mm/mm) [S.D.]	ϵ_r (mm/mm) [S.D.]
10M-1	516.8	0.0044*		652.3	0.0878	0.1337
	[14.23]	[0.0001]		[3.28]	[0.0087]	[0.0156]
10M-2	453.3	0.0025	0.0147	645.1	0.1235	0.1454
	[1.82]	[0.0001]	[0.0002]	[0.24]	[0.0084]	[0.0024]
15M	457.8	0.0023	0.0115	644.5	0.1196	0.1582
	[6.47]	[0.0001]	[0.0016]	[1.78]	[0.0034]	[0.0028]
15M-2	455	0.0024	0.0112	643	0.1166	0.1466
	[4.92]	[0.0001]	[0.0013]	[2.87]	[0.0047]	[0.0033]

* for bar 10M-1 no yield plateau is observed, yield stress and strain are obtained using the 2% offset method.

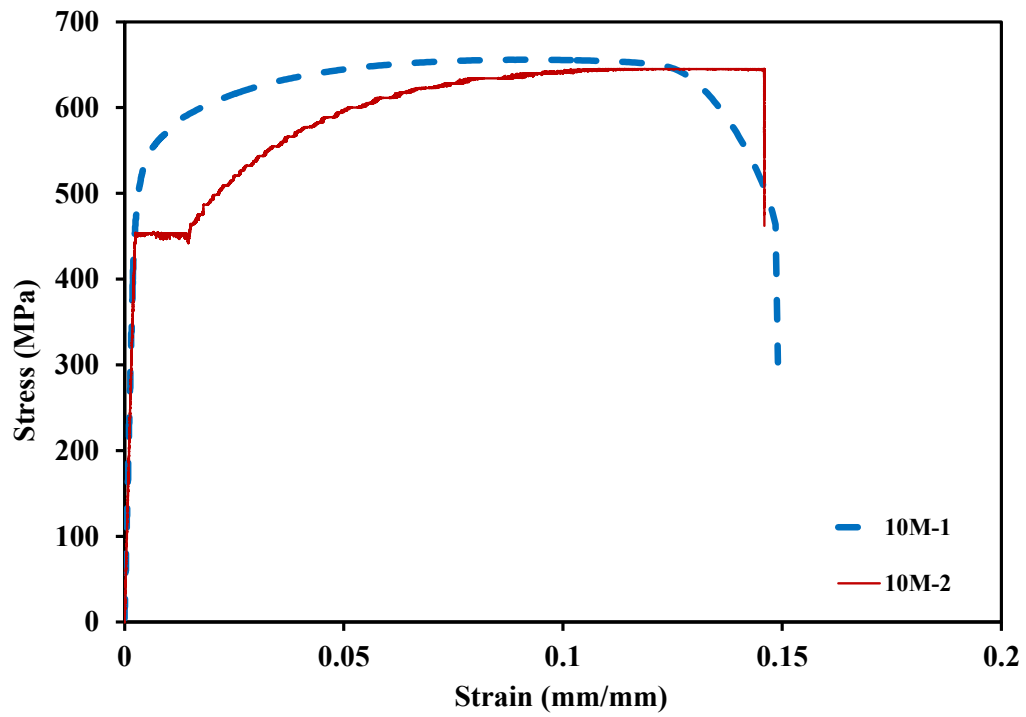


Figure 3.5 Typical Tension Test Results for 10M Rebar Samples

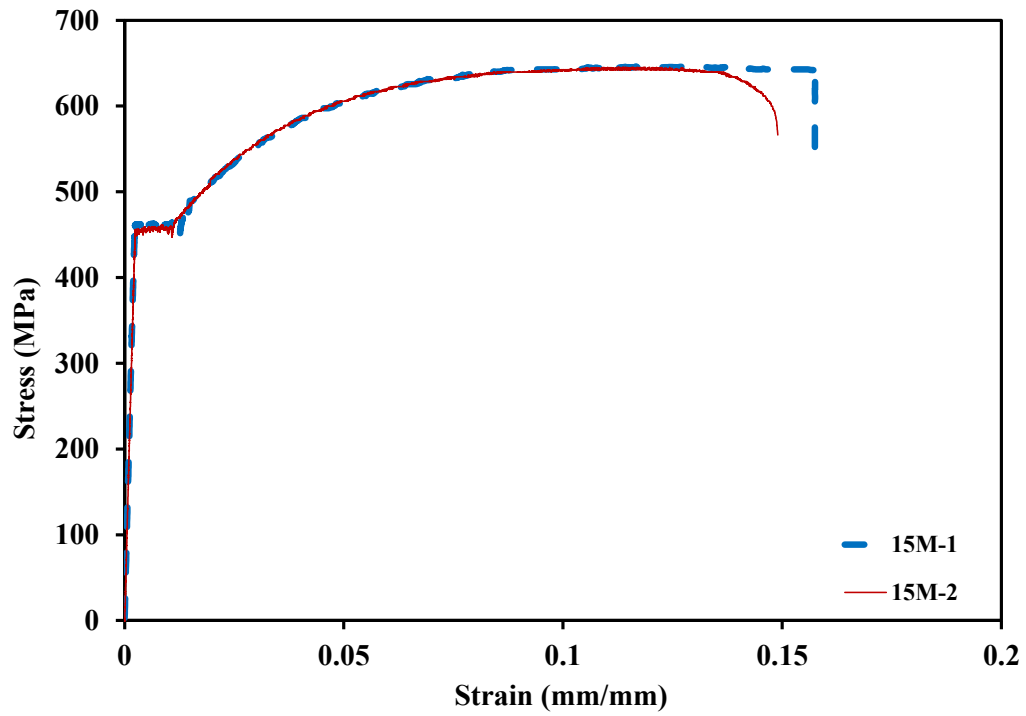


Figure 3.6 Typical Tension Test Results for 15M Rebar Samples

3.3.2 Steel Fibres

Two types of steel fibres were used in the experimental program (see Figure 3.7). The columns in the SFRC series was reinforced with *Dramix* ZP-305 steel fibres, manufactured by *Bekaert* (Belgium). These hooked-end fibres were 30 mm in length, with aspect-ratio (l_f/d_f) of 55, and tensile strength of 1100 MPa. Three different fiber contents were used: 0.5% (40 kg/m³), 1.0% (80 kg/m³) and 1.5% (120 kg/m³), per volume of concrete.

In the case of the UHPFRC series, the CRC mix was reinforced with high-strength, brass-coated, smooth steel fibres having length of 13 mm and diameter of 0.3 mm, resulting in aspect-ratio (l_f/d_f) of 43. These fibres had a tensile strength of 2500 MPa and were manufactured by *Stratec GMBH* (Germany).

Properties of the two fibres are reported in Table 3.3.

Table 3.3 Properties of Steel Fibres

Fibre type	Shape	Length l_f (mm)	Diameter d_f (mm)	Aspect ratio l_f/d_f (mm/mm)	Tensile strength f_{fy} (MPa)
ZP-305	Hooked-end	30	0.55	55	1100
ML-13/0.3	straight	13	0.30	43	2500



(a) Dramix ZP-305 Fibres (SFRC)



(b) Stratec ML-13/0.3 Fibres (UHPFRC)

Figure 3.7 Fibres Used in the SFRC and UHPFRC Series

3.3.3 Concrete Mixture

To improve workability of SFRC, an SCC mix was used in the construction of the columns in the SFRC series. The SCC mix consisted of a pre-packaged mixture with a specified strength of 50 MPa, manufactured by *KING Packaged Materials Company*. The mix contained a maximum aggregate size of 10 mm with a sand-total aggregate ratio of approximately 0.55 and a water-cement ratio of approximately 0.42. Furthermore, an air-entraining admixture, a super plasticizer and a viscosity modifying admixture (VMA) are incorporated in the mix in the form of dry powder. Table 3.4 lists the mix properties as specified by the manufacturer. This mix was used to cast both the “control” SCC specimens (05% fibres) and the SFRC specimens that were constructed with SCC and fibres. In the case of the UHPFRC series, the columns were constructed with compact reinforced concrete (CRC), a proprietary UHPFRC product. The mix contains a maximum aggregate size of 4 mm, a binder which consists of approximately 20% micro silica and Portland cement (*Densit Binder*), and a super plasticizer which is incorporated in the mix in the form of a dry powder (Nielsen 1995). The constituents of CRC as reported by the manufacturer are listed in Table 3.5.



(a) Pre-Packaged *KING* SCC Mix



(b) Pre-Packaged *CRC Jointcast* Mix

Figure 3.8 Pre-Packed Concrete Mixtures

Table 3.4 SCC Mix Properties as Specified by
Manufacturer

Component	Content
HSF Cement	500 (Kg/m ³)
Coarse Aggregate	765 (Kg/m ³)
Fine Aggregate	915 (Kg/m ³)
Other Properties	
W/C	0.42
Fine/Total Aggregate	0.55
Mass Density	2300 (Kg/m ³)
Air Content	7(%)

Table 3.5 CRC Mix Properties as Specified by Manufacturer

Ingredients		Fraction by Weight (%)	Density (Kg/m ³)	Content (Kg/m ³ concrete)
Densit	Cement	34	3150	1146
Binder	microsilica		2250	
	0-0.25mm	7	2640	236
Quartz sand	0.25-1mm	12	2640	404
	1-4mm	24	2740	809
Water	---	6	1000	202
Water/Binder		0.18		

3.3.4 Concrete Properties

The subsections that follow provide details regarding the fresh-state and hardened-state properties of the mixtures used in the construction of the columns.

3.3.4.1 *Mixing and Fresh-State Properties*

The concrete for all columns was mixed in the Jamieson Structures Laboratory at McGill University, in Montreal. Two types of mixers were used (see Figure 3.9): the majority of the SCC and SFRC was mixed using a 8 cu. ft drum mixer with the concrete for each column mixed in a single batch, while the UHPFRC was mixed using a 6 cu. ft mortar mixer with two batches required in the construction of each column (the batch size was governed by the capacity of the mixer's motor as the UHPFRC mix requires significant forcing action).



(a) Drum Mixer Used to Cast SCC and SFRC



(b) Mortar Mixer Used to Cast UHPFRC

Figure 3.9 Mixers Used in This Study

The SCC and SFRC mixtures were prepared by adding the drymix to the mixer, dry mixing for 30 seconds, and then adding water in several stages (half at the start, and the remaining in several stages over 4 minutes). After a uniform mix was attained, the steel fibres were added gradually over 2 minutes and mixing continued until uniform distribution was attained. Typical slump and slump flow values are reported in for 0%, 1% and 1.5% fibre contents in Table 3.6. Examples of slump flows are shown in (photos are from another study where the same mix was used). In general the combined use of SCC and fibres resulted in mixtures with high workability. During casting no vibration was required for the specimens with 0%

and 1% fibres, while some vibration was required in the case of specimens with 1.5 % fibres, particularly in the case of the heavily congested CR series specimen.

It is noted that the drum mixer was unavailable at the time of casting specimens BR1nc, BR15nc and CR1nc, and therefore the mortar mixer shown in Figure 3.9b was used for the construction of these columns. Although the same mix procedure was followed, the use of the mortar mixer resulted in larger than expected slump flows, as well as an increase in entrapped air in the case of the fibre reinforced specimens (visible upon inspection of specimens after casting).

Table 3.6 SCC/SFRC Slump and Slump Flow Values

SFRC Series	Typical Slump (mm)	Typical Slump Flow (mm)
0.0%	290	622
1.0%	247	436
1.5%	218	367



(a) SCC and 0% fibres



(b) SCC and 1% fibres



(c) SCC and 1.5% fibres

Figure 3.10 Typical Slump Flows for SCC and Highly Workable SFRC (Burrell, 2012)

In the case of the UHPFRC mixtures the mix procedure is outlined in Figure 3.11 and typical results from slump flow tests are shown in Figure 3.12. The procedure involved first adding the required amount of pre-packaged CRC to the mortar mixer, dry-mixing for 30 seconds, adding half the required water and mixing for 2 minutes and then gradually adding the remaining water over 3 minutes. It is noted that the dry super plasticizer in the CRC requires significant forcing action before the CRC becomes fluid and this limited the batch size used

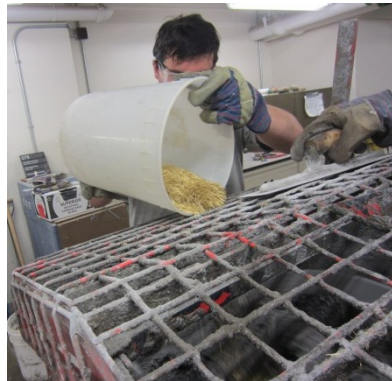
in casting the columns. After the super plasticizer was activated, the fibres were added over a period of approximately 1 minute. All ingredients were then mixed together for an additional 2 minutes. In general, while the CRC is relatively “sticky” high fluidity was observed at the high fibre contents used in this study and generally only external vibration of the forms was required during column construction, even in the case of the more heavily congested specimens.



(a) Addition Pre-Packaged CRC and Dry Mixing



(b) Adding Water in Several Stages.

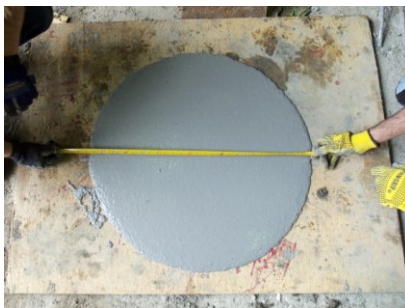


(c) Adding Fibres



(d) Placing the Concrete into the Formwork

Figure 3.11 UHPFRC Mixing Procedure



(a) Typical Slump Flow without Fibres (700 mm)



(b) Typical Slump Flow with Fibres (540 mm)

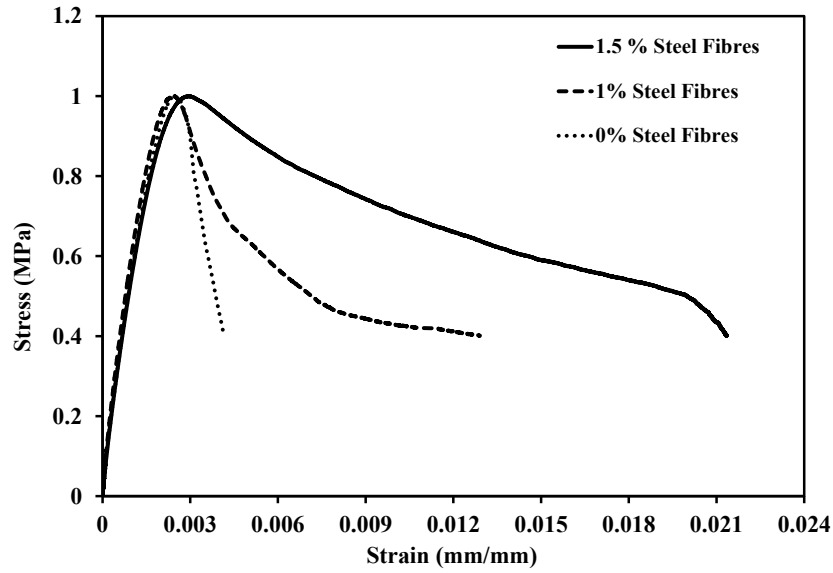
Figure 3.12 Typical UHPFRC Slump Flow without and with Fibres

3.3.4.2 *Hardened-State Properties*

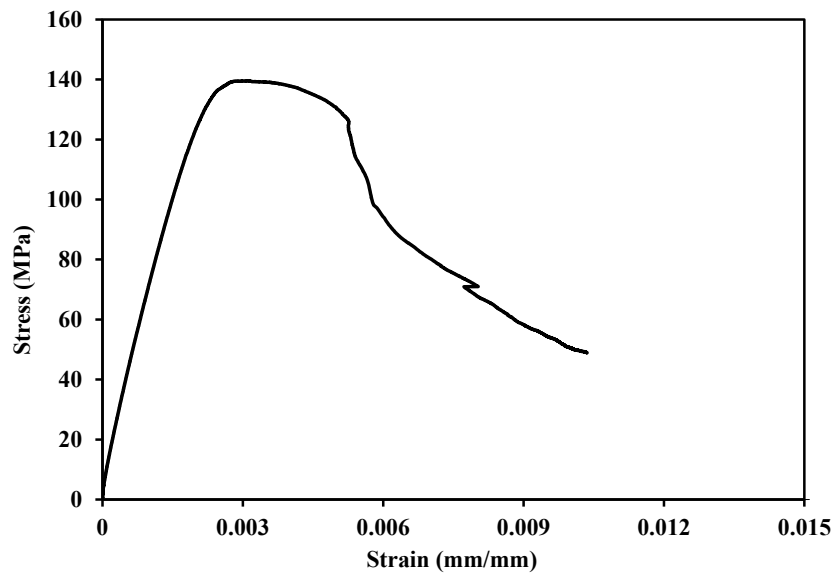
For each column, a series of concrete cylinders and flexural beams were prepared. The standard cylinders had diameter of 100 mm and height of 200 mm while the flexural beams had dimensions of 100x100x400 mm³. On the day the columns were tested, three cylinders were tested under axial compression to measure peak strength (f'_c) and associated strain (ϵ_{co}) and average values are reported in Table 3.7. Figure 3.13(a) shows typical normalized compressive stress-strain plots for the SFRC and illustrates the effect of fibre content on toughness in compression. It is noted that the concrete used in specimens BR1nc, BR15nc and CR1nc had lower compressive strength than expected and this returns to the use of mortar mixer as discussed previously. The results from the compression tests on the UHPFRC are reported in Table 3.7, and Figure 3.13(b) shows a typical stress strain plot. It is interesting to note that the UHPFRC shows a small plateau after reaching peak stress and this is followed by a gradual decrease in post-peak stress.

Table 3.7 Hardened Cylinder Strength

SCFRC Specimens						UHPFRC Specimens		
Series	f'_c (MPa)	ϵ_{co} (mm/mm)	Series	f'_c (MPa)	ϵ_{co} (mm/mm)	Series	f'_c (MPa)	ϵ_{co} (mm/mm)
AR0	51.8	0.0024	AR0nc	52.8	0.0026	C3-120	114.1	0.0028
AR1	44.2	0.0027	AR1nc	43.0	0.0023	C3-80	124.6	0.0028
AR15	38.0	0.0024	AR15nc	45.4	0.0026	C3-60	137.8	0.0028
BR0	48.9	0.0026	BR0nc	51.6	0.0022	C3-40	120.5	0.0028
BR1	46.7	0.0024	BR1nc	30.3	0.0022	C4-120	134.9	0.0028
BR15	50.8	0.0028	BR15nc	27.1	0.0011	C4-60	129.1	0.0028
CR0	51.7	0.0023	CR0nc	50.8	0.0022			
CR1	45.2	0.0023	CR1nc	38.0	0.0016			
CR15	43.6	0.0022						



(a) Typical Normalized Stress-Strain Behaviour for SFRC

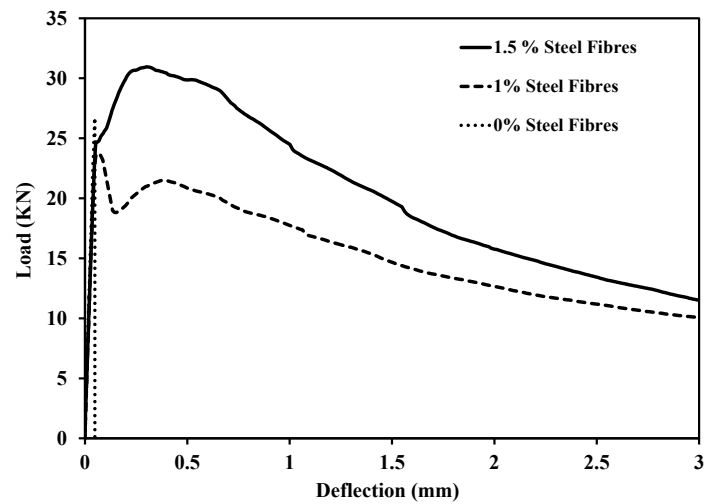


(b) Typical Stress-Strain Behaviour for UHPFRC

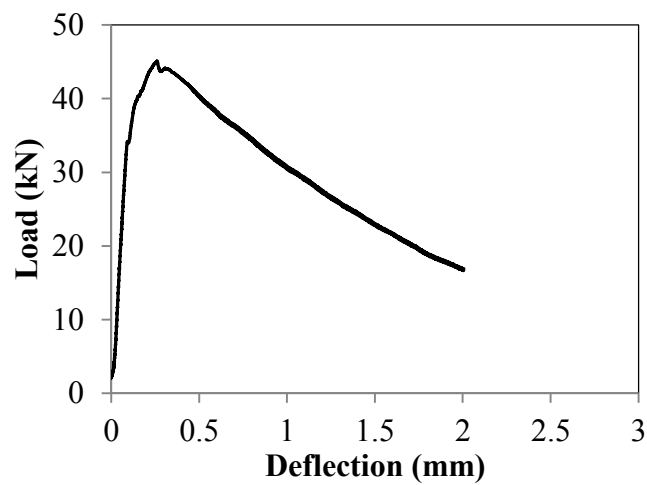
Figure 3.13 Compressive Stress-Strain Response for SFRC and UHPFRC Used in This Study

Standard beams were also cast and tested in accordance with the ASTM C1609 test method (ASTM, 2011). Figure 3.14(a) shows typical load-deflection responses for SFRC beams having 0%, 1% and 1.5% fibre contents. The addition of 1% and 1.5% results in significant

improvement in post-cracking toughness. A typical load-deflection response for a typical UHPFRC beam is also shown in Figure 3.14(b).



(a) Typical Load-Deflection Response for SFRC



(b) Typical Load-Deflection Response for UHPFRC

Figure 3.14 Flexural Beam Load-Deflection Response for SFRC and UHPFRC Used in This Study

3.4 Instrumentation and Testing

All specimens were tested under pure axial loading using an 11.4 MN MTS machine located at Jamieson Structures Laboratory at McGill University. Details regarding instrumentation, test setup, and loading procedure are provided in the subsections that follow.

3.4.1 Instrumentation

During testing the MTS machine head displacement was used to monitor axial displacements. However in order to measure axial strains a series of Linear Variable Differential Transducers (LVDTs) were installed along the column height (see Figure 3.15). In the case of the SFRC test series columns two LVDTs were placed on two opposite faces (wider dimension) and the axial strains were measured over a gauge length of 900 mm. In the case of the UHPFRC columns, special steel collars were constructed to confine the end regions of the columns and four LVDTs were mounted on the collars to measure strains on all four faces over a gauge length of 925 mm. In all cases the average strain recorded by the LVDTs was used to plot the load-deformation response of the columns.

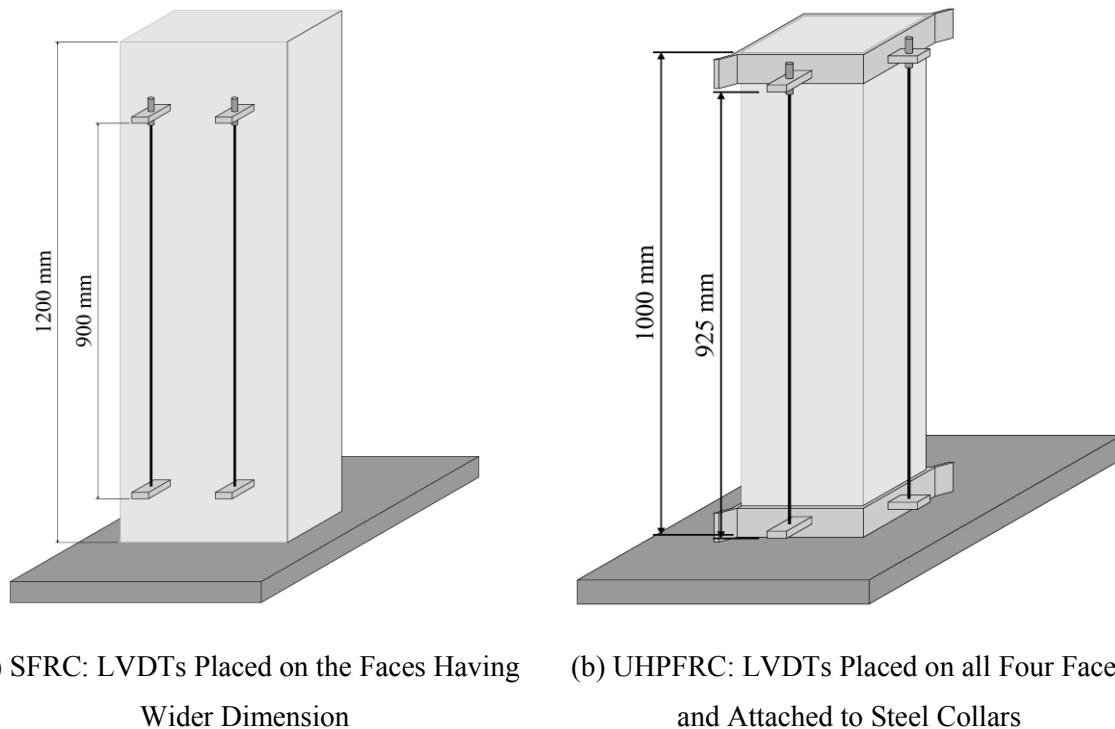


Figure 3.15 LVDT Placement for the SFRC and UHPFRC Columns

Each column was instrumented with electric resistance strain gauges located on the longitudinal reinforcing bars and transverse ties.

Figure 3.16 and

Figure 3.17 show the locations of the gauges for the SFRC and UHPFRC columns. For all columns, four strain gauges were installed on the longitudinal bars to monitor strains and to capture the onset of bar buckling. Two gauges were installed on the inner and outer surface of one corner bar and the remaining two gauges were installed on the inner and outer surface of a mid-section bar. All four gauges were installed at column mid-height. In addition, gauges were placed on the mid-height transverse ties to study the strains in the transverse reinforcement. In the case of the rectangular SFRC columns, two gauges were installed on the perimeter ties, with a third installed on the inner crosstie. In the case of the UHPFRC columns, 4 gauges were placed on the perimeter and inner ties.

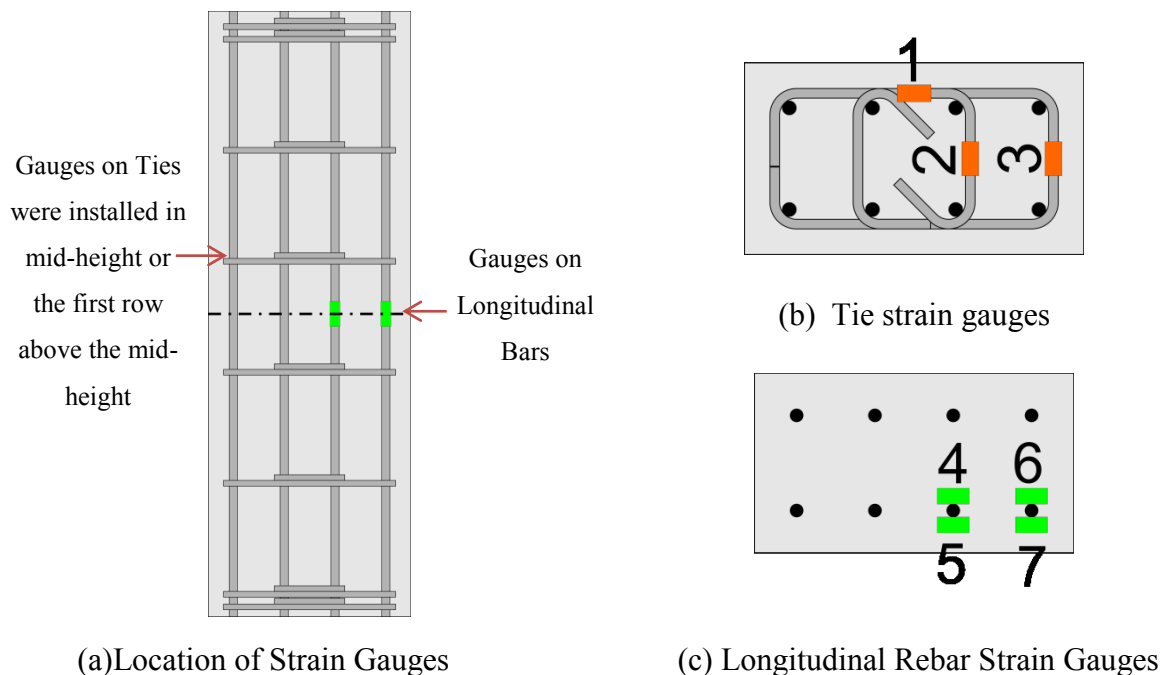


Figure 3.16 Strain Gauge Instrumentation for the SFRC Columns

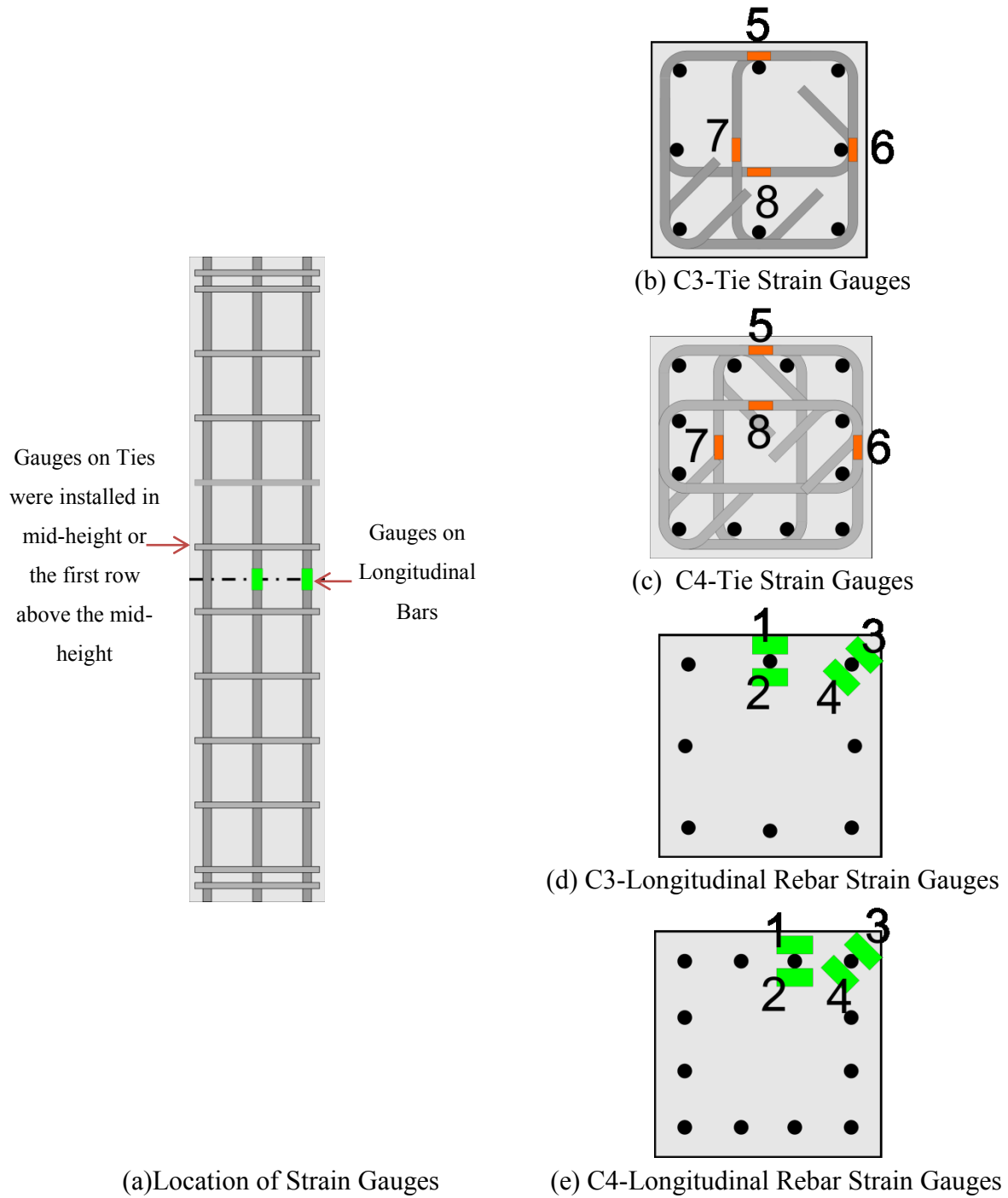
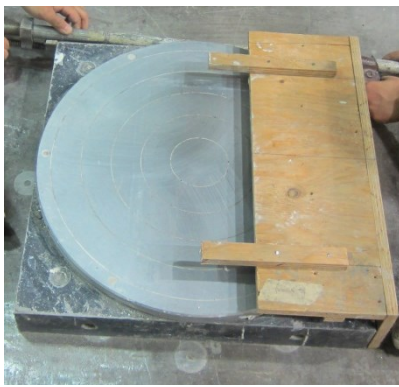


Figure 3.17 Strain Gauge Instrumentation for the UHPFRC Columns

3.4.2 Test Setup Procedure

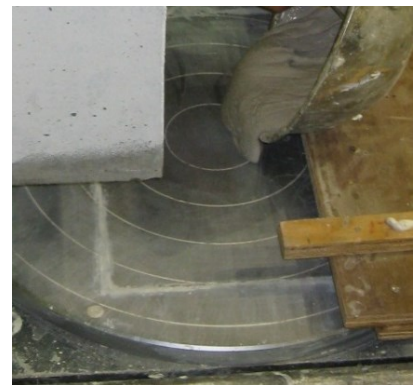
All specimens were tested under pure axial loading using an 11.4 MN MTS machine. The pre-test setup for each column is illustrated in Figure 3.18. First, the top surface of the columns was ground to remove any roughness. The columns were then placed under the MTS machine using a specially built form to avoid eccentricity during loading. Prior to placing the column, a thin layer of high-strength capping compound was employed on both the top and bottom surfaces of the columns to ensure a uniform application of stresses from the MTS machine.



(a) Bottom steel plate and special form used to ensure concentric column placement



(b) Transferring the column with forklift



(c) employment of high-strength capping compound on top and bottom surfaces

Figure 3.18 Pre-Test Setup Procedure

As discussed previously, additional ties were employed in the SFRC column end regions to avoid failure at the column ends. In the case of the UHPFRC columns a pair of steel collars were used to provide additional confinement in the end regions (the collars were also used to mount the LVDTs). After the column was concentrically placed under the MTS machine the strain gauge and LVDT wires were connected to the computer and all the instrumentation was calibrated. Figure 3.19 shows the instrumentation scheme used in a typical rectangular SFRC column and Figure 3.20 shows typical SFRC and UHPFRC columns ready for testing.

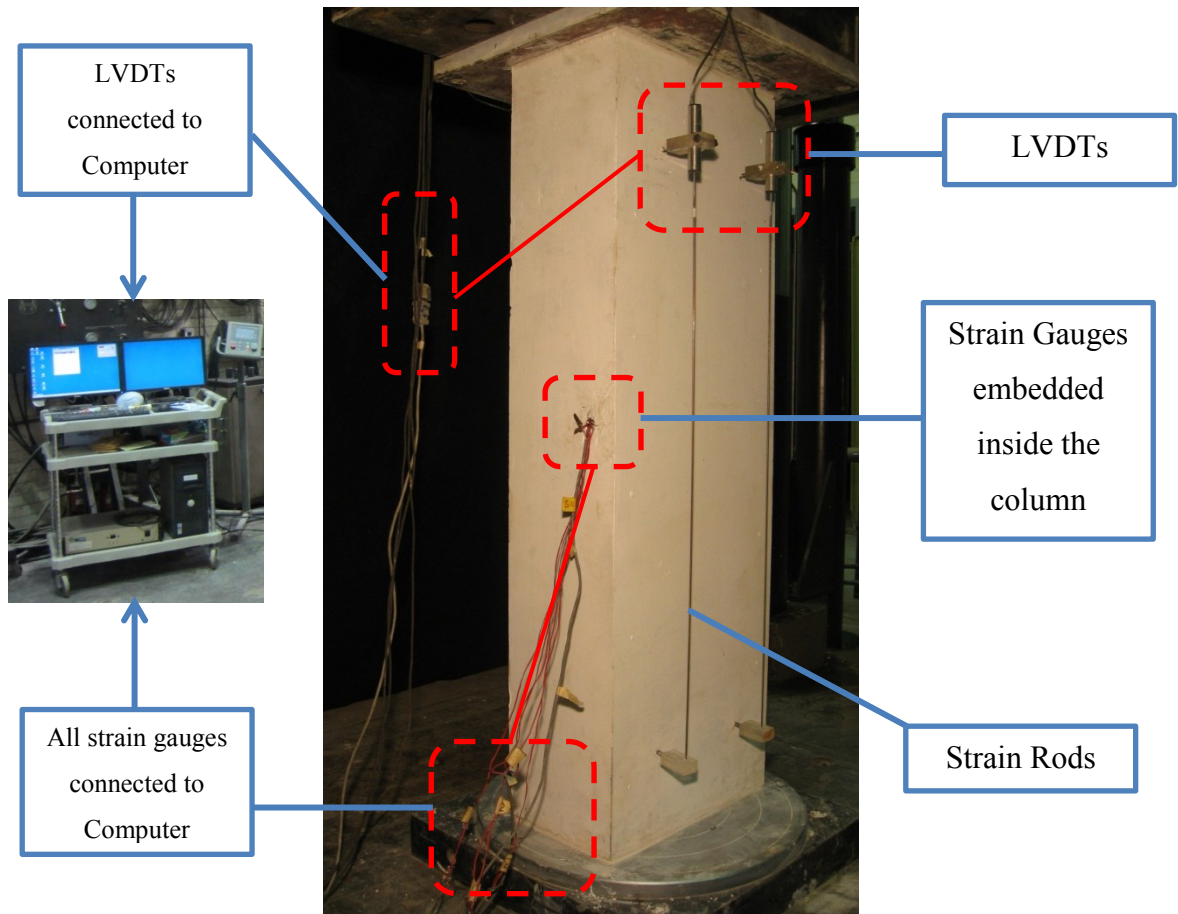


Figure 3.19 Instrumentation Scheme



(a) SFRC column ready for testing

(b) UHPFRC column ready for testing

Figure 3.20 Columns under MTS Testing Machine Ready for Testing

3.4.3 Loading Procedure

As shown Figure 3.21, the loading procedure for the SFRC series included four stages. First, the specimens were tested under load-control at a rate of 2.5 KN/s up to a load of 3000 KN for the specimens with cover and 2500 KN for specimens without cover. Next, loading changed to displacement-control, at a rate of 0.002 mm/s up to machine displacement of 8 mm. Thereafter, the loading rate increased to 0.004 mm/s up to 18mm the specimens with cover and 16 mm for specimens without cover. Finally, the loading rate increased to 0.006 mm/s to the end of testing which was around 25 mm stroke. For the nominally confined specimens loading was stopped earlier if the load dropped to 20% of peak capacity.

A similar procedure was used for the UHPFRC series (see Figure 3.21). Testing began under load-control at a rate of 3KN/s up to a load of 7000 KN after which loading was changed to displacement-control at a rate of 0.0024 mm/s up to 9 mm. The rate increased to 0.0036 mm/s up to 12 mm, 0.0054 mm/s up to 16 mm and finally 0.0072 mm/s until the end of the test (typically at MTS stroke of 20 mm).

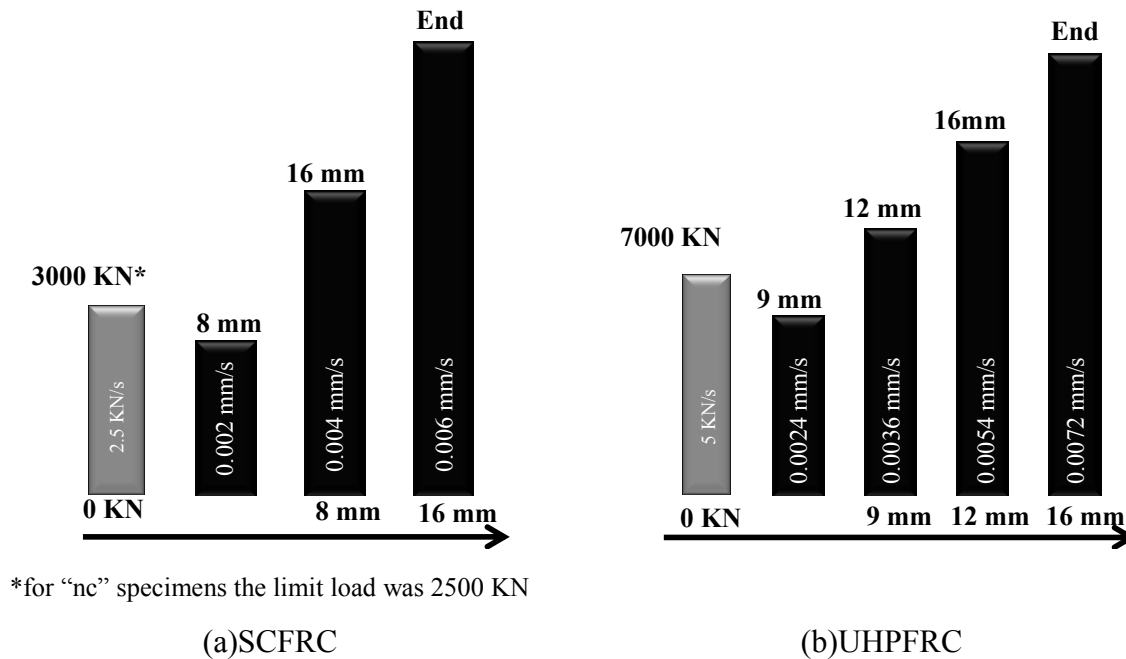


Figure 3.21 Loading Rate Scheme for Experimental Specimens

4 Experimental Results

4.1 Introduction

This chapter presents the experimental results from the twenty-three columns tested in the experimental program. Sections 4.2 and 4.3 summarize the individual response of the SFRC columns constructed without and with cover, respectively. The response of the UHPFRC columns is presented in section 4.4. It is noted that the column results are compared and discussed in Chapter 5.

The following results are presented for each column:

1. The response in terms of axial load versus machine head (stroke) displacement is first presented, along with snap-shots of failure progression in the columns (taken at the load stages defined in section 3.4.3);
2. Next, a set of plots are presented as follows:
 - Total axial load (P_t) versus axial strain response;
 - Concrete load (P_c) versus axial strain response;
 - Normalized load versus axial strain response;
 - Strain development in the longitudinal reinforcement;
 - Strain development in the transverse reinforcement.

It is noted that the total axial load (P_t) corresponds to the load recorded by the MTS machine, whereas the concrete load (P_c) is obtained by subtracting the expected contribution of the longitudinal steel reinforcement in compression (P_s) while taking into account buckling using the model proposed by Dhakal and Maekawa (2002) (see Equation 4-1):

$$P_c = P_t - P_s \quad \text{Equation 4-1}$$

In the response plots, the axial strain corresponds to the average strain recorded by the 4 LVDTs attached to each column.

Since some of the concrete strengths varied, the normalized response is also presented, where the total axial load is normalized with respect to the nominal column axial capacity computed using the ACI 318-08 Standard (P_o) (see Equation 4-2):

$$P_o = 0.85f'_c(A_g - A_s) + A_s f_y \quad \text{Equation 4-2}$$

In this equation, A_g and A_{st} represent the gross column cross-section area and the total area of longitudinal steel, while f'_c and f_y represent the compressive strength of concrete and yield strength of steel, respectively.

When presenting the strain data for the SFRC columns the yield strains for the longitudinal and transverse steel are taken as 2290 $\mu\epsilon$ and 2600 $\mu\epsilon$, respectively. For the UHPFRC columns, the corresponding strains are 2422 $\mu\epsilon$ and 2536 $\mu\epsilon$, respectively.

4.2 Results - SFRC Test Series (Specimens With 30 mm Cover)

This section presents the results for the nine SFRC specimens with 30 mm cover. In addition to presenting load-displacement and progression of failure, the load-strain, normalized load-strain and strain gauge data are presented for each specimen.

4.2.1 Specimen # AR0

Specimen # AR0 was detailed in accordance with the basic provisions of the CSA A23.3 Standard for columns having a ductility-related force modification factor, R_d of 1.5, and had a tie spacing of 220 mm. The column was constructed with plain SCC (0% fibres) and had a 30 mm of cover.

Figure 4.1 and Figure 4.2 show the load versus machine-head displacement response for the column, along with snapshots showing the progress of damage in this specimen. The main events as recorded during testing are summarized below:

- First cracking for this specimen was observed at displacement of 4.14 mm and a corresponding load of 4317 kN;
- The column reached ultimate capacity of 4389 kN at a displacement of 4.37 mm;

- Soon after reaching peak capacity, the load dropped suddenly to 40 % peak capacity, and this drop in capacity was accompanied by cover spalling and buckling of the reinforcement;
- Thereafter the load continued to drop at a more stable rate until reaching 20 % of peak capacity at a displacement of 8mm at which time the test was stopped.

Specimen AR0 had nominal confinement reinforcement and did not contain any fibres and therefore the large and sudden drop in capacity is due to a combination of several factors. First, a drop in strength is expected due to sudden cover spalling (see Figure 4.3 (a)), which occurs due to insufficient concrete tensile capacity at the core-cover interface. The next factor is longitudinal bar buckling which occurred suddenly due the very large tie spacing of 220 mm relative to the 16 mm bar diameter (see Figure 4.3 (b)). Finally, the large tie spacing results in very poor confinement which results in a large area of unconfined concrete within the core region, which crushes and fails at rapid rate.

The post-failure status of the column is presented in Figure 4.2(d) and Figure 4.3. In particular, the spalling of the cover (Figure 4.3(a)), buckling of the reinforcement (Figure 4.3(b)), and crushing of core concrete (Figure 4.3(c)) is evident.

Figure 4.4(a) presents the total load versus axial strain response for the column (also presented in the same figure is the concrete contribution obtained by subtracting the load carried by the longitudinal steel reinforcement). The normalized load versus axial strain response is plotted in Figure 4.4(b). It is noted that the peak capacity is 5 % lower than the nominal capacity P_o that is suggested by the ACI 318-08 Standard.

The strains recorded in the longitudinal reinforcement are presented in Figure 4.4(c). It is observed that except for gauge S6 that buckled before reaching yield, the others gauges reached large compressive strains and reached yield when the column reached its peak axial capacity.

The strains for the transverse reinforcement are presented in Figure 4.4 (d). Gauge S2 which had been placed on the interior crosstie did not reach its yield point. Gauge S1 which was located on the larger perimeter dimension carried large tensile strains, while gauge S3 which

was placed on the smaller perimeter dimension, became dysfunctional slightly after peak. It is noted that none of the ties had reached yield when the column reached peak capacity, although larger strains are observed later in gauge S1 when compared to gauges S2/S3.

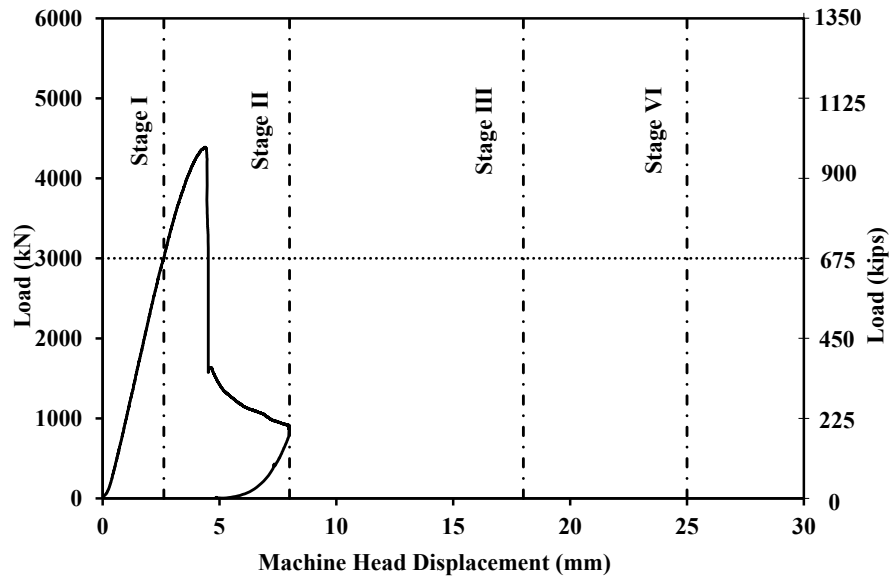


Figure 4.1 Staged Failure Process for Specimen # AR0

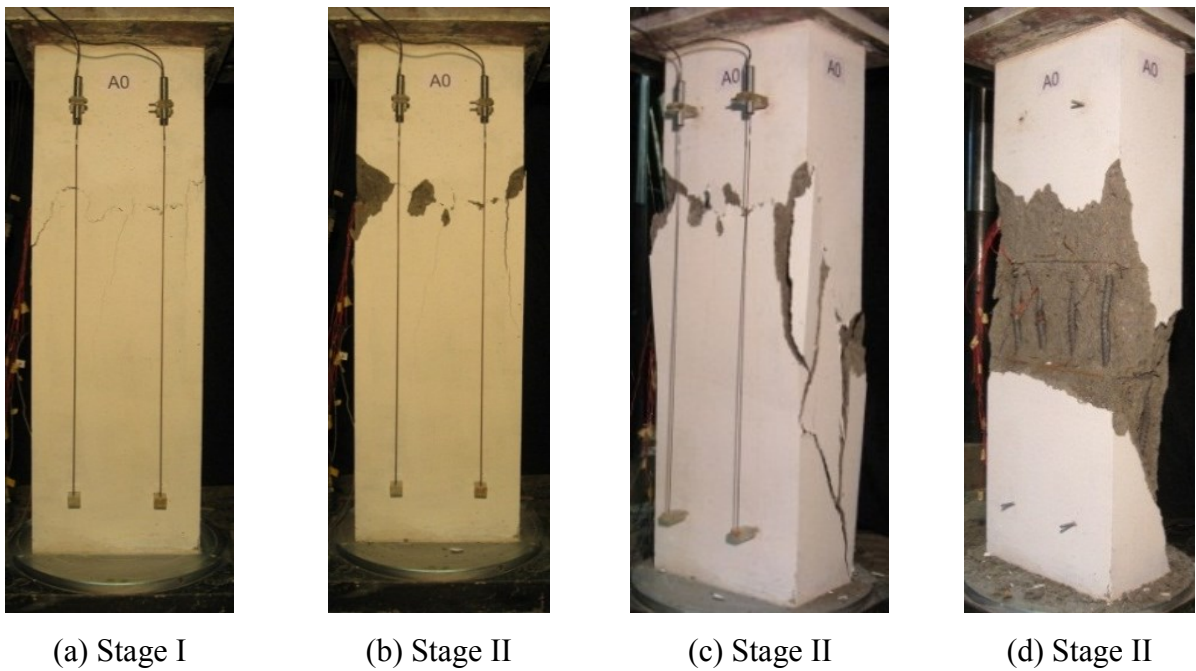


Figure 4.2 Specimen # AR0 Failure Snapshots During the Test



(a)

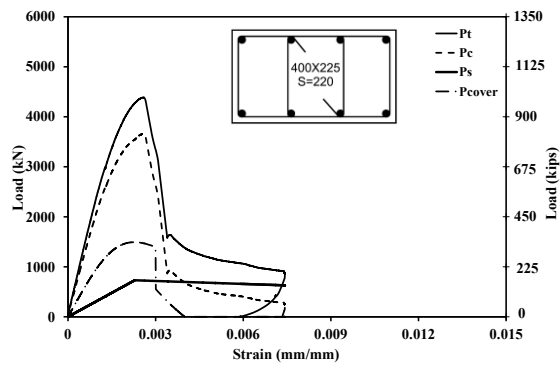


(b)

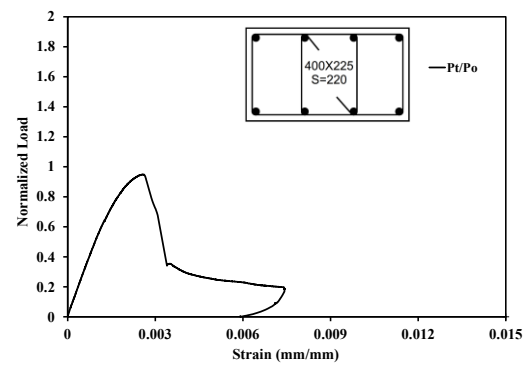


(c)

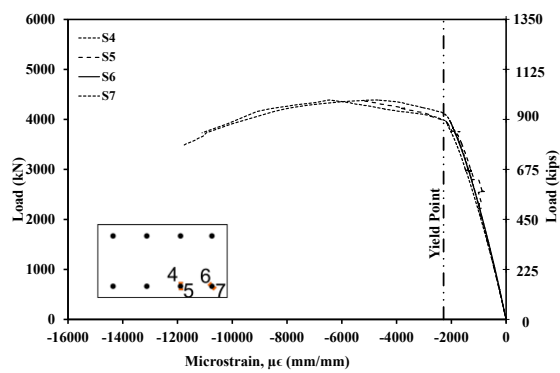
Figure 4.3 Close Look at Specimen # AR0



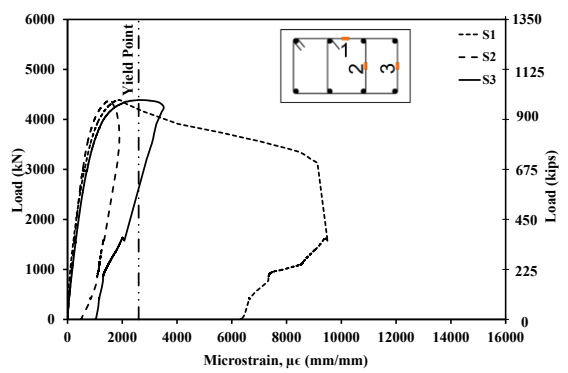
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



c) Longitudinal Reinforcement Behaviour



d) Transverse Reinforcement Behaviour

Figure 4.4 Results for Specimen # AR0

4.2.2 Specimen # AR1

Specimen #AR1 had identical details to column AR0 (tie spacing of 220mm, 30mm cover) with the exception of the provision of 1% steel fibres.

Figure 4.5 shows the load versus displacement response for the column, while Figure 4.6 shows the progress of damage during testing. The main events for this column are summarized below:

- First cracking for this specimen was observed at displacement of 4.37 mm and a corresponding load of 4090 kN;
- The column reached ultimate capacity of 4180 kN at a displacement of 4.88 mm;
- After peak capacity, the load began to drop at a rapid yet more gradual rate than what was observed in specimen AR0;
- During this time cracking was observed in the cover and although spalling was much better controlled, it was clear the bars were pushing against large pieces of SFRC that were still attached to the column;
- The load dropped to almost 65% at the end of stage II, 25% capacity at end of stage III, and at the end of testing (27 mm) the load had dropped to 16 % of peak capacity.

The post-failure status of the column is presented in Figure 4.2(d) and Figure 4.7. In particular, it is noted that large pieces of cover appeared to be attached to column at the end of testing (Figure 4.7 (a)), however further investigation revealed that the cover had effectively spalled and was only hanging due to a few fibres, and subsequently the cover was easily removed by hand. Furthermore, as it is evident from Figure 4.7 (b), buckling of the longitudinal rebar occurred due to the large spacing between the transverse ties in this column. Furthermore as shown in Figure 4.7 (c) the failure and crushing has penetrated deep within the core despite provision of steel fibres.

Figure 4.8 (a) presents the load versus axial strain response, while the normalized response is plotted in Figure 4.8 (b). It is noted that the peak capacity is only slightly above the nominal capacity prescribed by the ACI 318-08 Standard (reaching a value of 1.03 P_o). The strain gauge data in Figure 4.8 (c) shows that longitudinal reinforcement reached yield and high

compressive strain although bar buckling eventually occurred. Examination of the strains in the transverse reinforcement demonstrates that the ties reach yield slightly after peak capacity of the column.

Overall, some improvements in column response were observed when comparing to specimen AR0. The improved response arises due to the ability of the fibres to improve cover and core response which led to a more gradual failure response.

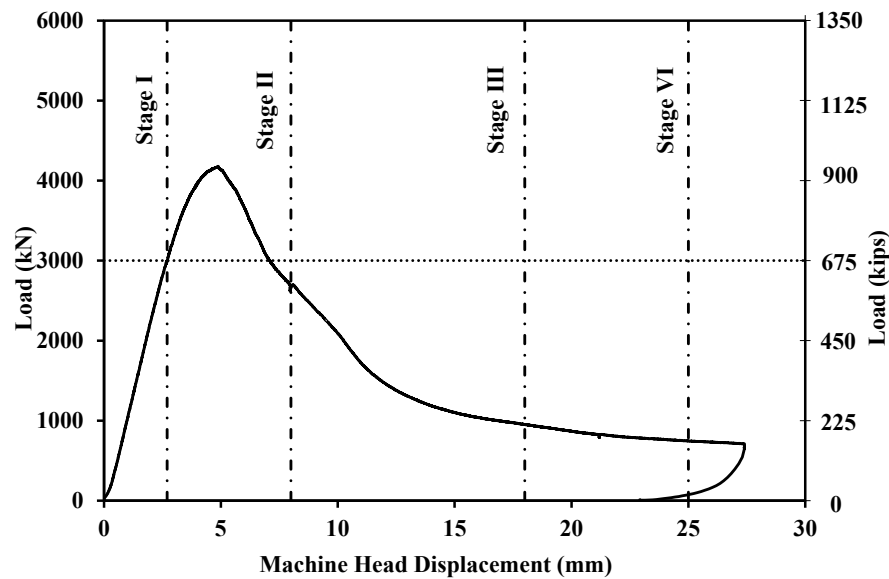


Figure 4.5 Staged Failure Process for Specimen # AR1

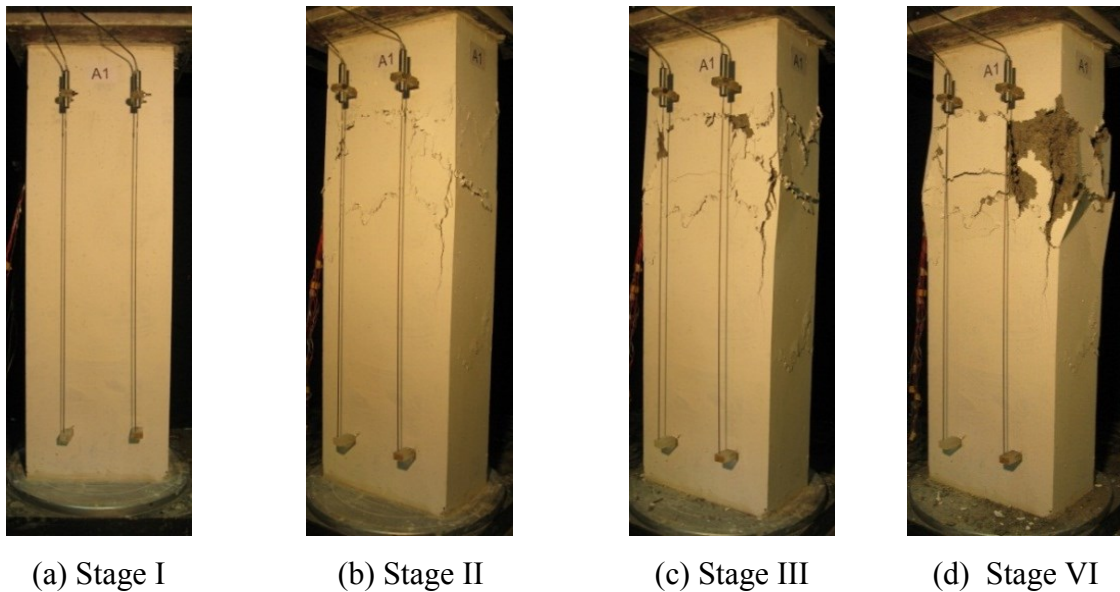


Figure 4.6 Specimen # AR1 Failure Snapshots During the Test



(a)

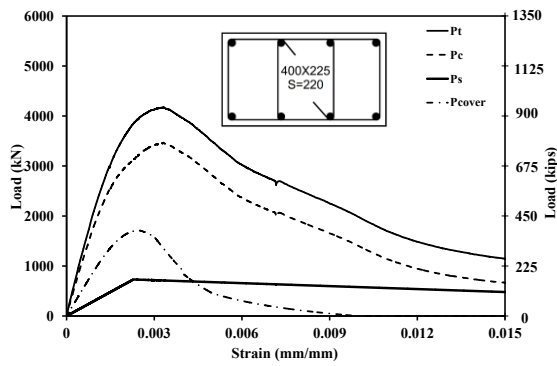


(b)

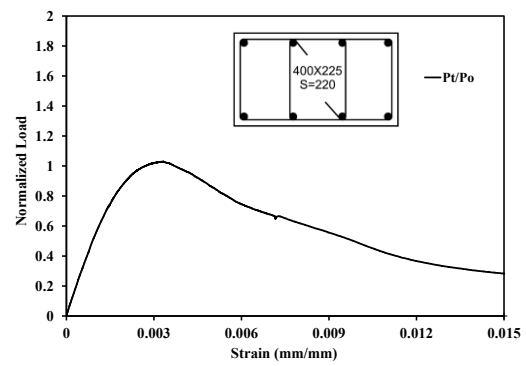


(c)

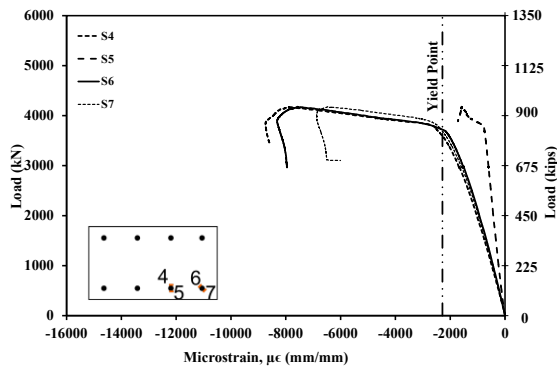
Figure 4.7 Close Look at Specimen # AR1



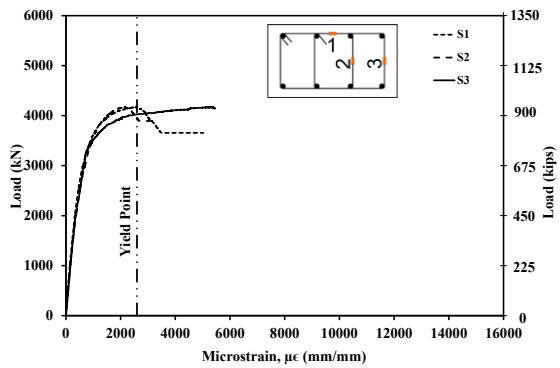
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



c) Longitudinal Reinforcement Behaviour



d) Transverse Reinforcement Behaviour

Figure 4.8 Results for Specimen # AR1

4.2.3 Specimen # AR15

Specimen #AR15 was the third column in the AR series and had same detailing ($s=220$ mm, 30 mm cover) with an increase in volume of fibres to 1.5%.

The column response as recorded by the MTS machine is shown in Figure 4.9 while the progress of damage through load stages I-IV is shown in Figure 4.10. The main events for this column are summarized below:

- First cracking for this specimen occurred 3750 kN at displacement of 3.8 mm;
- Ultimate capacity was 3966 kN at a displacement of 4.5 mm;
- The load then began to drop gradually and at a steady rate, reaching approximately 75% and 30% of peak capacity at the end of stages II and III, respectively, and 20 % of peak capacity at the end of testing (27 mm).

Figure 4.10 (d) and Figure 4.11 show the post-failure photos for the column. Similar to the previous specimen, cover spalling is gradual, rather than sudden, although effective spalling is observed (Figure 4.11 (a)). After the test, a crow-bar was used to remove the cover which was held in place by a number of fibres at the core-cover interface. After the cover was removed it was evident that bar buckling had occurred in both the corner and mid-section longitudinal reinforcement (Figure 4.11 (a)). Similarly, crushing is significant in the core, with arching action evident as shown in Figure 4.11 (c).

The column load and normalized load responses as recorded by the LVDTs are shown in Figure 4.12 (a) and (b) with normalized capacity reaching 10% above the ACI suggested strength, P_o . Strain gauge data in Figure 4.12 (c) shows that yielding of the longitudinal rebar and large strains occur in gauge S5, while lower strains are observed in the remaining gauges which may be due to buckling in proximity of the gauge locations. Similarly strain data from gauge S1 in Figure 4.12 (d) shows that yielding occurs in the transverse steel reinforcement near peak capacity with the gauges in the opposite hoop direction sustaining lower strains before failure.

Overall, the results from this test and the previous specimen demonstrate the effect of fibres in improving response in nominally confined columns. The greatest improvements are in the

cover sapling mechanism as well in the response of the core, whereas buckling of the reinforcement is not prevented. Observations also show that buckling plays an important role in the effective spalling of cover in SFRC columns.

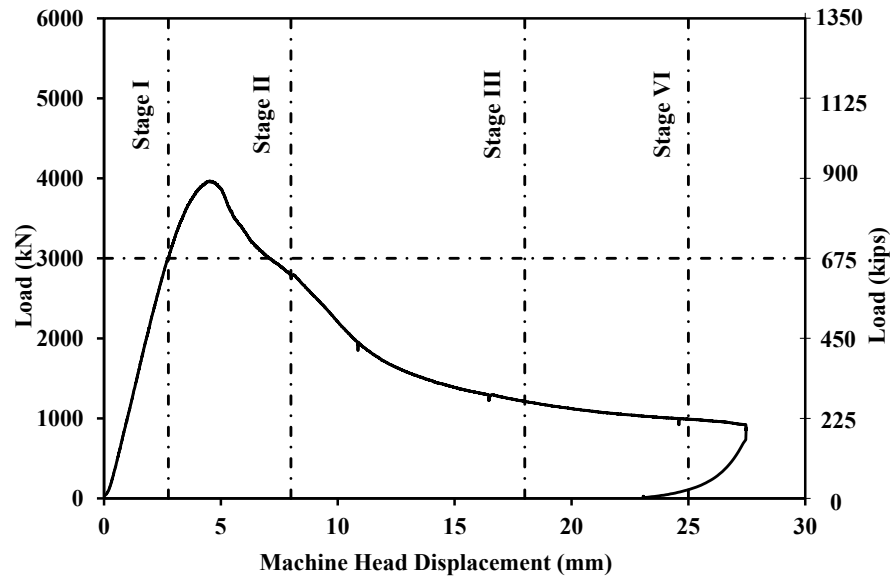


Figure 4.9 Staged Failure Process for Specimen # AR15

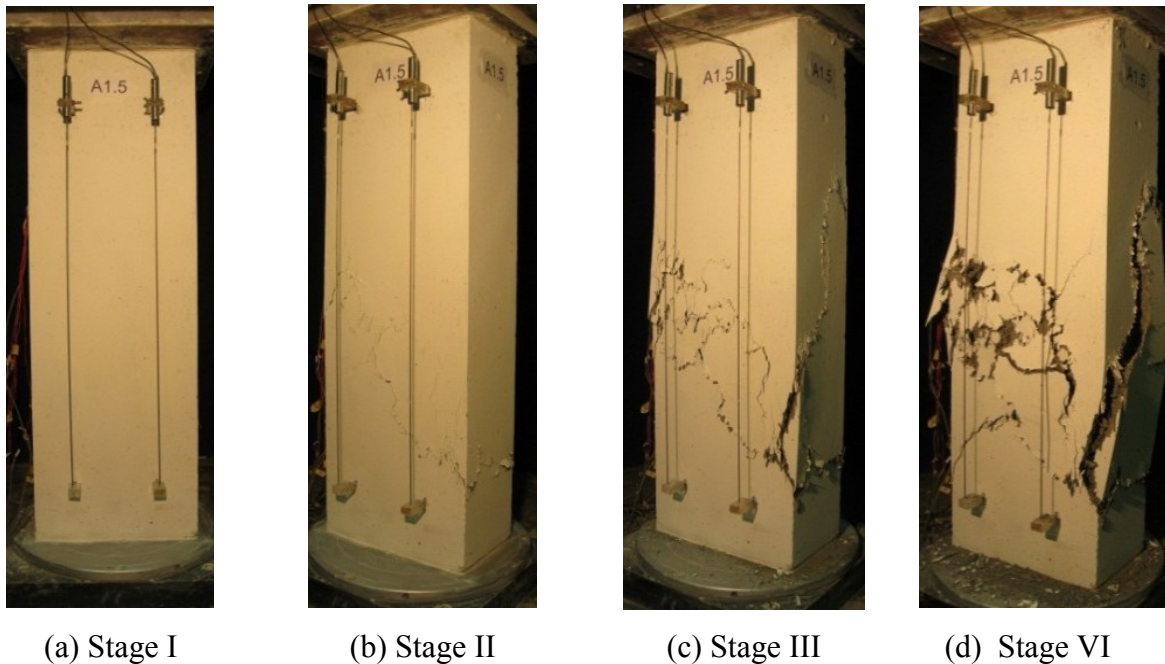


Figure 4.10 Specimen # AR15 Failure Snapshots During the Test



(a)

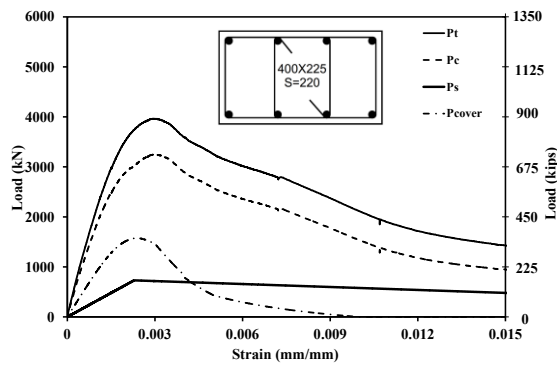


(b)

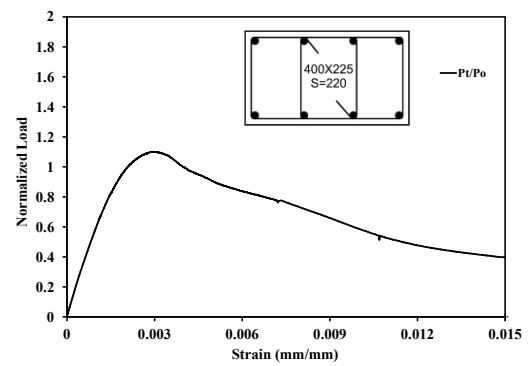


(c)

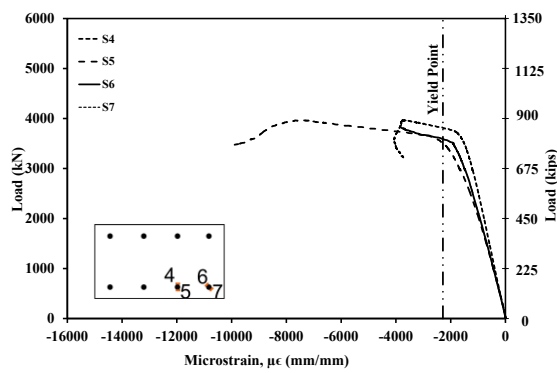
Figure 4.11 Close Look at Specimen # AR15



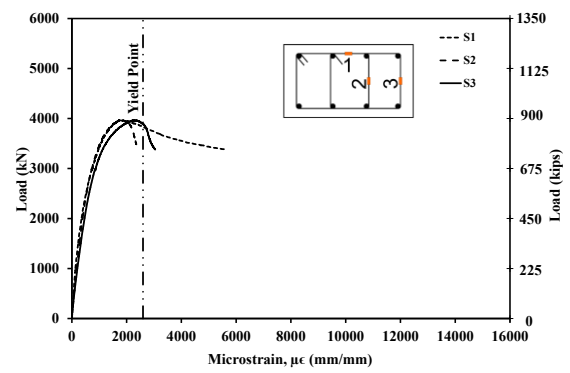
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



c) Longitudinal Reinforcement Behaviour



d) Transverse Reinforcement Behaviour

Figure 4.12 Results for Specimen # AR15

4.2.4 Specimen # BR0

Specimen # BR0 was detailed in accordance with the provisions for moderately ductile columns in the CSA A23.3 Standard and had a tie spacing of 110 mm. The column was constructed with plain SCC (0% fibres) and had a 30 mm of cover.

Figure 4.13 and Figure 4.14 show the load-displacement response for the column, along with the progress of damage in this specimen through stages I-IV. The main observations from this test are as follows:

- First cracking for this specimen was observed at displacement of 4.5 mm and a corresponding load of 4370 kN;
- The column reached ultimate capacity of 4539 kN at a displacement of 4.4 mm;
- Due to sudden cover spalling, the column capacity reduced by 20% soon after reaching peak and the load then began to reduce at a more stable rate, reaching approximately 50% and 30% of peak capacity at the end of stages II and III, respectively;
- At 23 mm the test was stopped, and the load reached 20 % of peak at this stage.

Specimen BR0 had moderate confinement and therefore showed improved response when compared to the brittle failure observed in specimen AR0. Although, spalling occurred in a sudden manner during stage II (as was the case for AR0), failure was more gradual after this point. The post-failure photos of the column in Figure 4.2(d) and Figure 4.15 show spalling of large pieces of concrete cover and buckling of longitudinal steel reinforcement despite the intermediate spacing of the hoops in this column. Nonetheless, it is clear that the reduced spacing has resulted in improved core confinement and the region of crushed concrete within the core is reduced when compared to column AR0, with arching action evident (Figure 4.15 (c)). Another interesting observation is the opening of the 90 degree leg in the intermediate cross-tie due to the transverse pressure from the core concrete.

Figure 4.16 (a) presents the axial load-strain response, while the normalized response is shown in Figure 4.16(b); the result shows that the column only reached 1% above the nominal capacity P_o that is suggested by the ACI 318-08 Standard, despite provision of

moderate confinement. Examination of strain gauge data in Figure 4.16 (c) shows that with the exception of gauge S7, the longitudinal reinforcement record large compressive strains. In addition, examination of Figure 4.16 (d) demonstrates that large tensile strains were recorded in the gauges, although yield is reached after ultimate column capacity is reached.

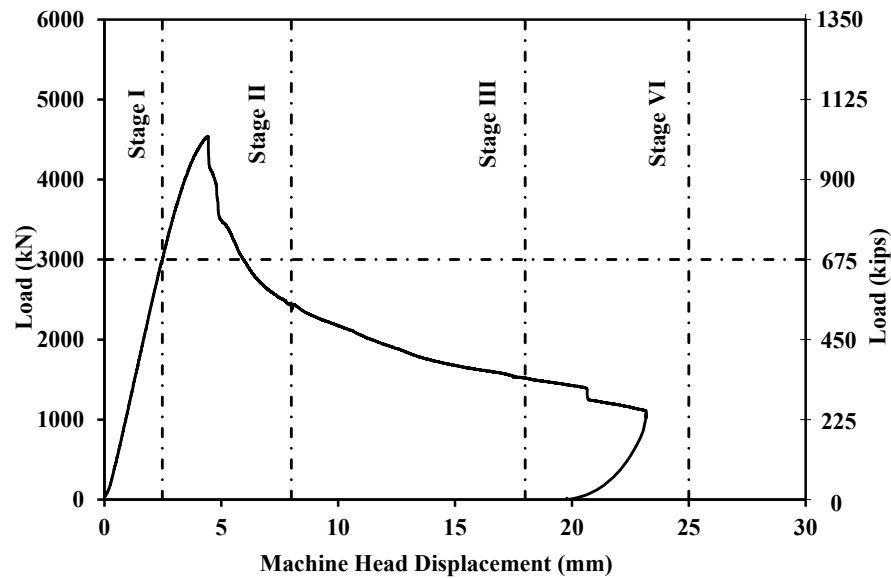


Figure 4.13 Staged Failure Process for Specimen # BR0

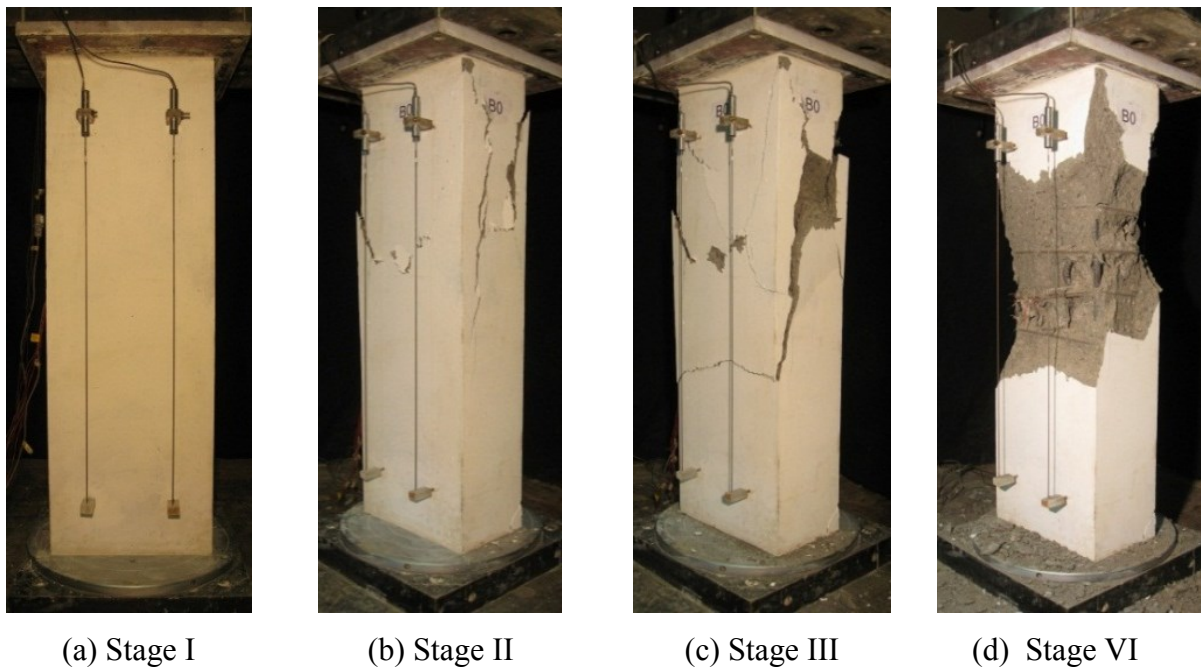


Figure 4.14 Specimen # BR0 Failure Snapshots During the Test



(a)

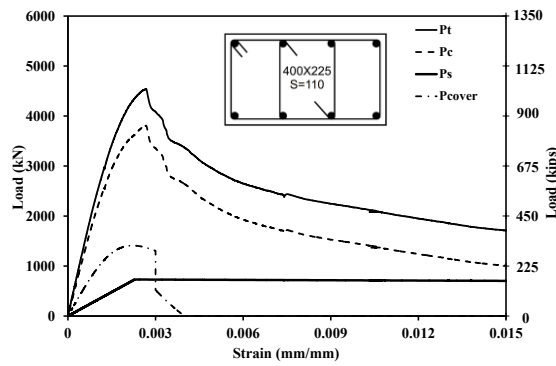


(b)

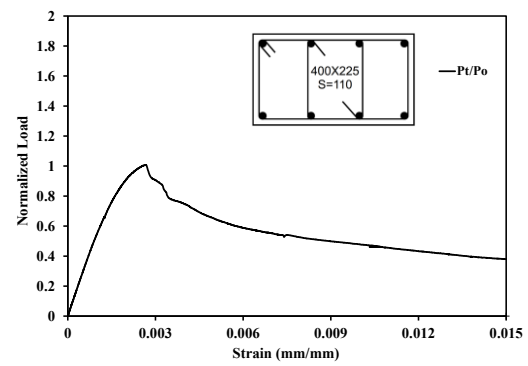


(c)

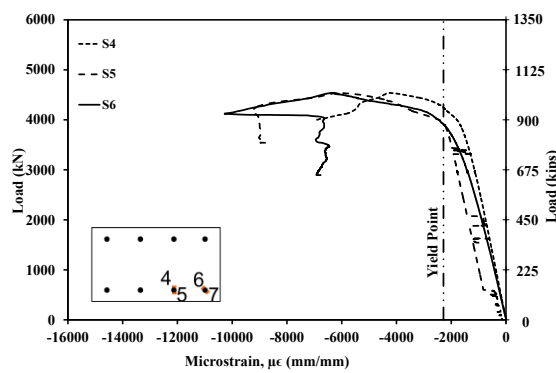
Figure 4.15 Close Look at Specimen # BR0



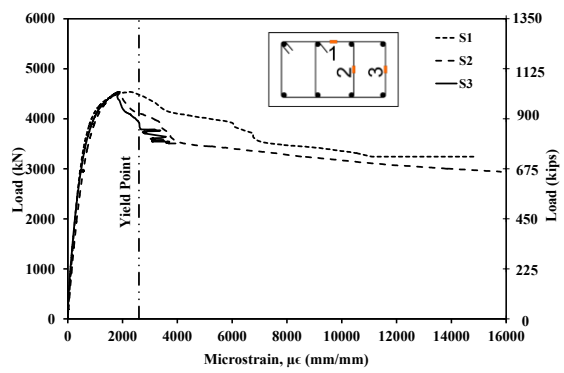
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



(c) Longitudinal Reinforcement Behaviour



(d) Transverse Reinforcement Behaviour

Figure 4.16 Results for Specimen # BR0

4.2.5 Specimen # BR1

Specimen# BR1 had identical details to column BR0 (intermediate tie spacing of 110mm, 30mm cover) but was constructed with SCC reinforced with 1% steel fibres. This test allows for an investigation of the effect of fibres in moderately confined columns.

Figure 4.17 and Figure 4.18 show the load versus displacement response and progress of damage for this column. The main events for this column are summarized below:

- First cracking occurred at a displacement and load of 4.4 mm and 4284 kN;
- Ultimate capacity for this column was 4712 kN at a displacement of 6 mm;
- Sudden cover spalling was prevented, and the column shows a gradual and well controlled post-peak response until the end of testing with the load reaching almost 80% at the end of stage II, 45% capacity at end of stage III, finally reaching 30% capacity at the end of testing (27 mm).

Figure 4.17 (d) and Figure 4.18 present the specimen at the end of testing. The photos show that cover spalling is delayed and large pieces of cover remain attached at the end of testing. Using a crow-bar, the cover in the damaged zone was removed revealing that great core integrity is maintained. Nonetheless bar buckling is not prevented in spite of existence of steel fibres and the intermediate spacing of the hoops.

The response as recorded by the LVDTs and strain gauges is shown in Figure 4.20. Examination of the normalized load-strain response shows that the column only reached peak strength of $1.05P_o$. Examination of the strain gauge data shows that all longitudinal reinforcement reach yield and sustain large compressive strains (in addition the data seems to capture buckling of the reinforcement as observed in the strains recorded by the inner vs. outer gauges on the bars). Examination of the data from the gauges on the transverse reinforcement show that yielding occurs with the exception of gauge S1. In all cases yielding is observed to occur after the ultimate capacity of the column is reached.

This test demonstrates that addition of fibres improves the behaviour of rectangular columns with moderate transverse reinforcement detailing due to improved cover response and increased confinement within the core region.

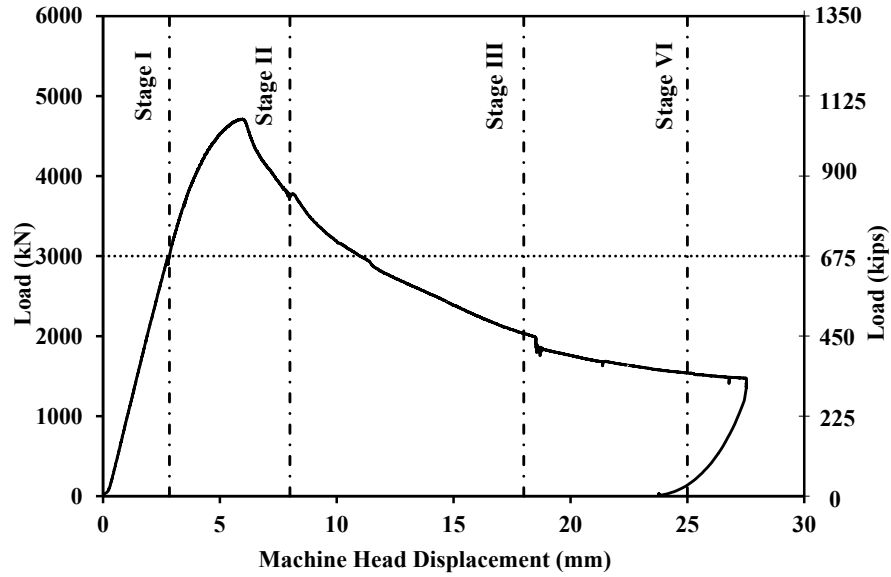


Figure 4.17 Staged Failure Process for Specimen # BR1



(a) Stage I

(b) Stage II

(c) Stage III

(d) Stage VI

Figure 4.18 Specimen # BR1 Failure Snapshots During the Test

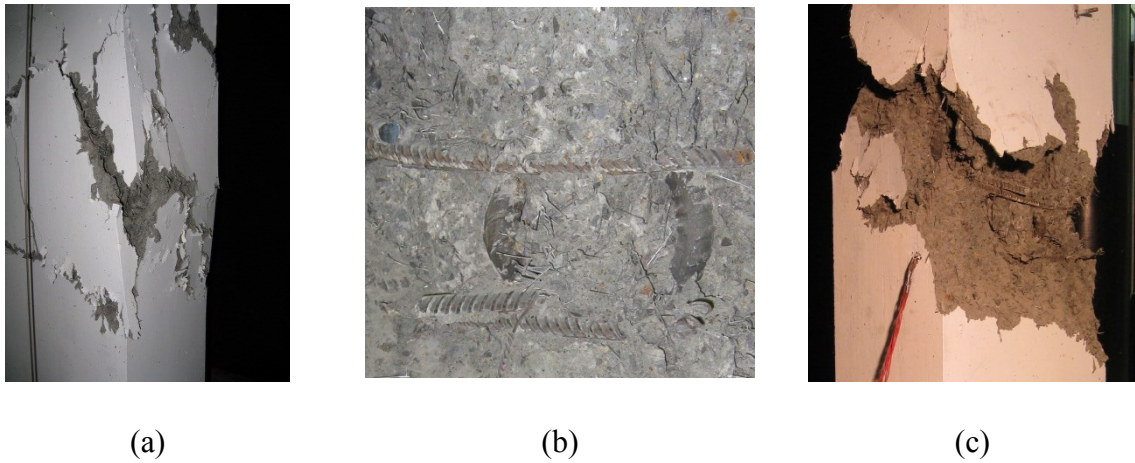


Figure 4.19 Close Look at Specimen # BR1

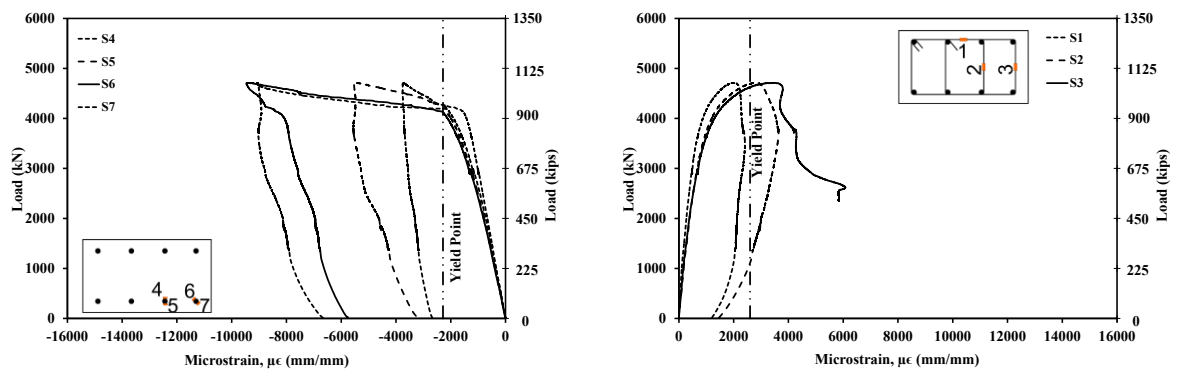
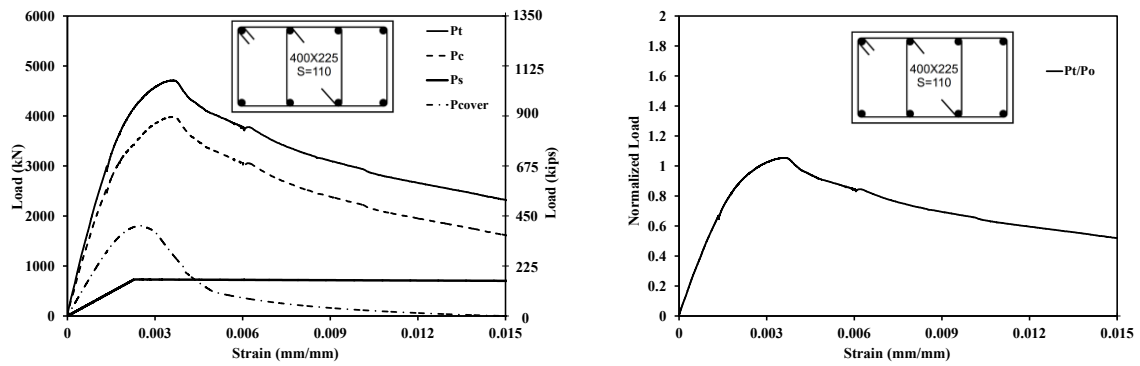


Figure 4.20 Results for Specimen # BR1

4.2.6 Specimen # BR15

Specimen # BR15 was identical to the previous two BR columns ($s = 110$ mm, 30 mm cover), but was reinforced with 1.5% of fibres.

Figure 4.21 and Figure 4.22 show the column response as recorded by the MTS machine as well as the progress of damage through load stages I-IV. The main events for this column are summarized below:

- First cracking for this specimen occurred 4400 kN at displacement of 5.1 mm which is a slightly later than the first cracking observed in previous specimens;
- Ultimate capacity was 4668 kN at a displacement of 5.5 mm;
- The post-peak response of the column was well controlled, staying at around 95% peak capacity at the end of stages II and falling to approximately 60% and 40% capacity at the end of stages III and IV (end of testing - 28 mm).

Figure 4.22 (d) and Figure 4.23 show the post-failure photos for this column. It is noted that spalling was very well controlled in this column, and cover remained attached to the column at the end of testing. By using some force, the cover was removed manually at which point buckling of the reinforcement was observed. However it is clear that the provision of the higher fibre content in this column resulted in greater integrity within the core. Unlike the previous specimens the 90 degree hooks on the intermediate cross-ties did not bulge out which may be due to resistance to lateral expansion provided by the cover.

The column load and normalized load responses as recorded by the LVDTs are shown in Figure 4.24 (a) and (b) with normalized capacity reaching 1.03Po. Strain gauge data in Figure 4.24 (c) shows that yielding and large strains occur in the longitudinal steel bars (exception of gauge S7 which may have been affected by buckling). Similarly all transverse reinforcement gauges with the exception of S4 reach yield at or near the ultimate capacity of the column and sustain great tensile strains.

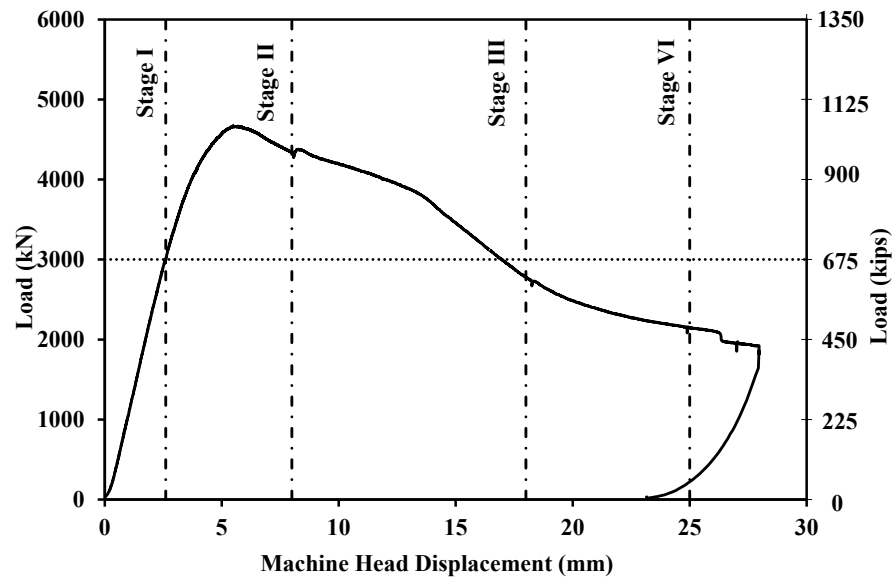


Figure 4.21 Staged Failure Process for Specimen # BR15



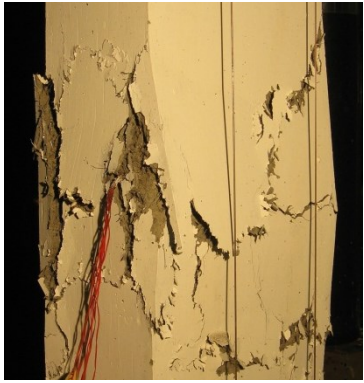
(a) Stage I

(b) Stage II

(c) Stage III

(d) Stage VI

Figure 4.22 Specimen # BR15 Failure Snapshots During the Test



(a)

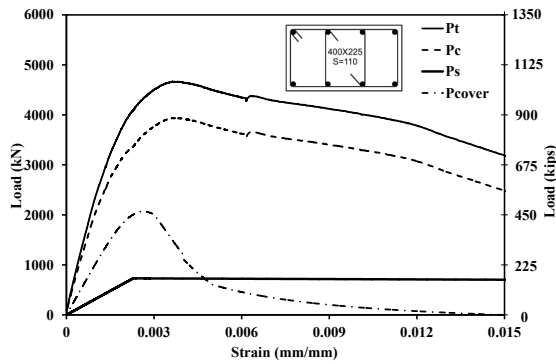


(b)

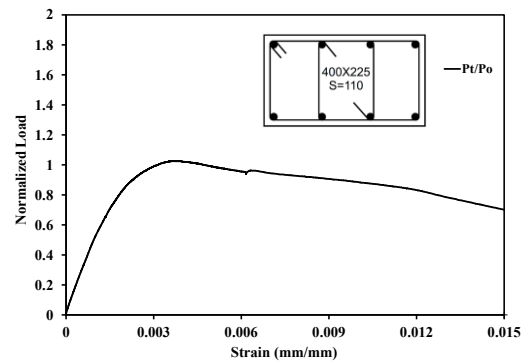


(c)

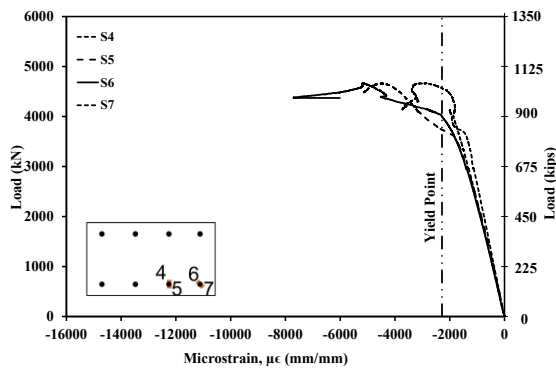
Figure 4.23 Close Look at Specimen # BR15



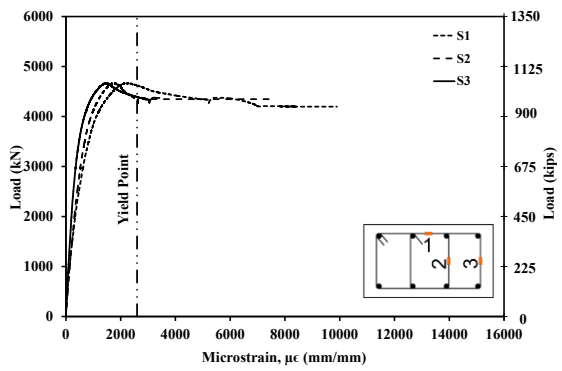
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



c) Longitudinal Reinforcement Behaviour



d) Transverse Reinforcement Behaviour

Figure 4.24 Results for Specimen # BR15

4.2.7 Specimen # CR0

Specimen # CR0 was detailed in accordance with the seismic detailing provisions of the CSA A23.3 Standard and had a tie spacing of 55 mm. The column was constructed with plain SCC (0% fibres) and had a 30 mm of cover. Figure 4.25 and Figure 4.26 show the load versus displacement response along with the progress of damage during testing. The main events are summarized below:

- First cracking for this specimen was observed at displacement of 4.4 mm and a corresponding load of 4646 kN;
- The column reached ultimate capacity of 4790 kN at a displacement of 4.53 mm;
- After reaching peak capacity, a sharp drop to 80% of capacity occurs, after which the column shows excellent post-peak performance due to provision of closely spaced transverse reinforcement; (the sudden load drop can be related to cover spalling, core crushing in between ties or bar buckling.)
- With further loading the capacity reaches 60% peak capacity at the end of stage III, and falls to 50% peak capacity at the end of testing (26 mm, stage IV).

The post-failure status of the column is presented in Figure 4.26 (d) and Figure 4.27. Spalling of the cover is evident and upon close investigation after the test, longitudinal rebar was seen to have buckled over the reduced clear spacing between the ties (Figure 4.27 (c)). Examination of the core concrete shows some crushing however excellent integrity is maintained due to the high confinement in this specimen (Figure 4.27 (a)). Due to lateral expansion of the concrete and ties, the 90 degree hook in the interior cross tie deformed outward (Figure 4.27 (b)).

Data collected from the LVDTs and strain gauges is presented in Figure 4.28. In Figure 4.28 (a) it can be seen that the drop of load is due to cover spalling which initiates shortly after peak and completes at 0.004 strain (it can be seen that the cover spalling model in the figure estimates the spalling response closely - see Ch.6). Also, it can be observed that the main effect of confinement is in post-peak segment of the response where capacity remains almost constant although strain is increasing. It is interesting to note that the capacity only reaches to $0.99 P_o$ despite the high degree of confinement in this column.

The behaviour of longitudinal and the transverse reinforcement can be studied by examining the strain data in Figure 4.28 (c) and (d). It is seen that all gauges on the longitudinal reinforcement yield and sustain large compressive strains. The transverse reinforcement gauges reach yield after ultimate column capacity but they do not show considerable tensile strain after yielding.

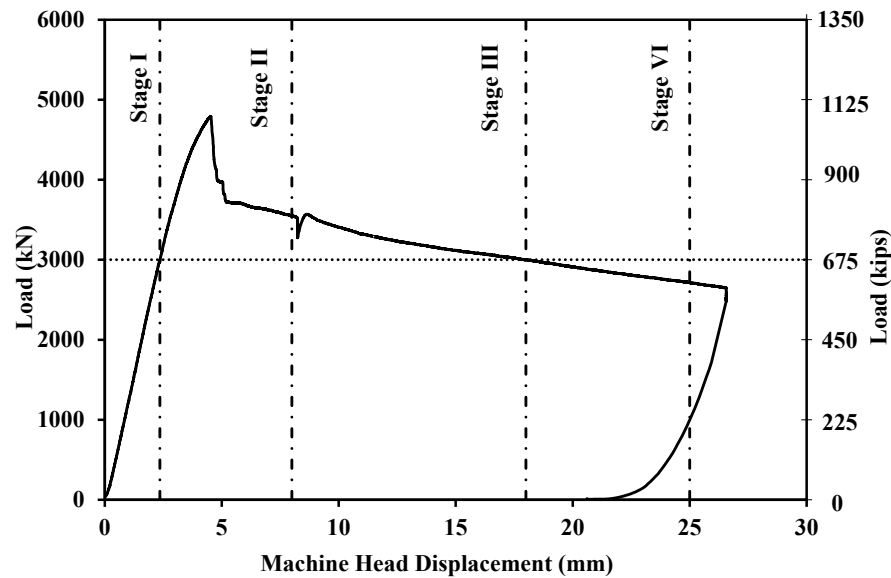


Figure 4.25 Staged Failure Process for Specimen # CR0

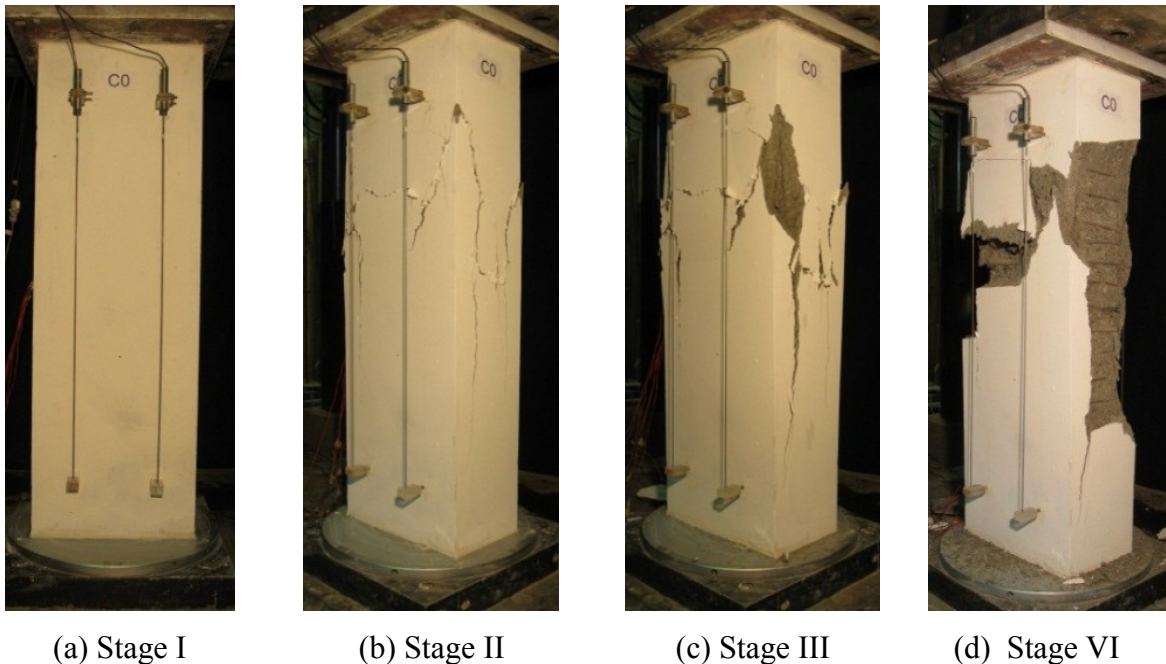


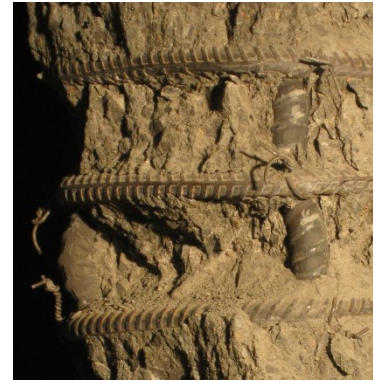
Figure 4.26 Specimen # CR0 Failure Snapshots During the Test



(a)

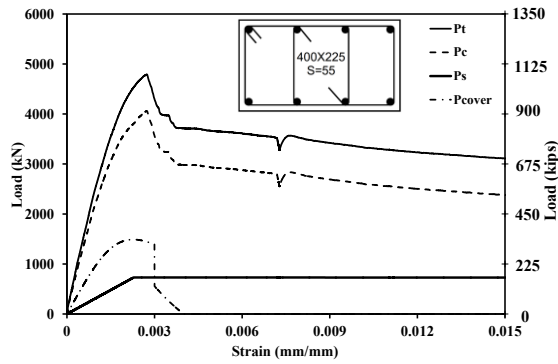


(b)

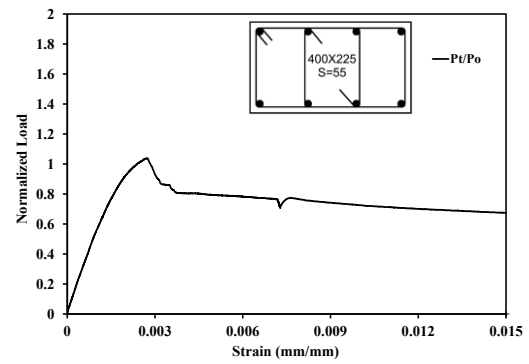


(c)

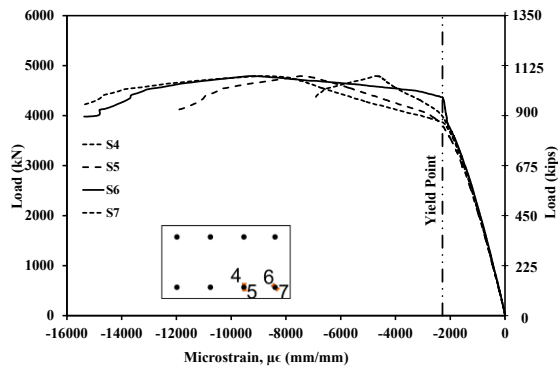
Figure 4.27 Close Look at Specimen # CR0



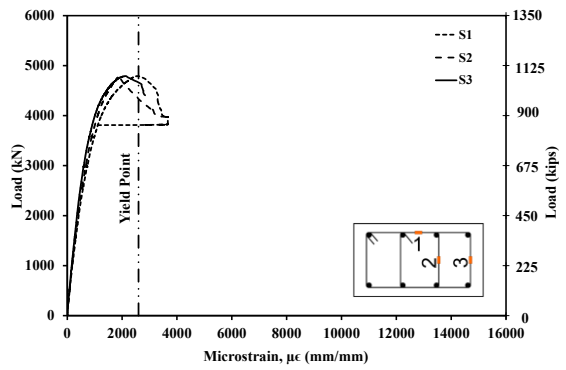
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



c) Longitudinal Reinforcement Behaviour



d) Transverse Reinforcement Behaviour

Figure 4.28 Results for Specimen # CR0

4.2.8 Specimen # CR1

Specimen #CR1 had identical details to column CR0 (tie spacing of 550mm, 30mm cover) but was reinforced with 1% steel fibres. This test allows for an investigation of the effect of moderate fibres in highly confined columns.

Figure 4.29 shows the load versus displacement response for the column, while Figure 4.30 shows the progress of damage during testing. The main events are summarized below:

- First cracking for this specimen was observed at displacement of 4.0 mm and a corresponding load of 3800 kN;
- The column reached ultimate capacity of 4790 kN at a displacement of 4.53 mm;
- After peak capacity, a slight drop in capacity is observed which is then followed by an almost linear gradual descending failure;
- The load dropped to almost 80% capacity at end of stage III, and at the end of testing (28 mm) the load remained stable at 70 % of peak capacity.

The post-failure status of the column is presented in Figure 4.30 (d) and Figure 4.31. Spalling was very well controlled in this specimen and the cover remained attached until the end of the testing (Figure 4.7 (a)). With some manual effort the cover was removed in order to expose the interior core. After inspection, it was clear that macroscopic cracks are more dense in the failure region due to existence of steel fibres, and no core crushing was observed (Figure 4.7 (c)). The 90 degree hooks on the interior cross ties deformed along with the long side of the rectangular hoops due to great expansion of the core concrete (Figure 4.7 (b)). Slight buckling was also observed for one of the corner bars.

The loads-train response for this specimen is shown in Figure 4.32 . In Figure 4.32 (a), it can be seen that cover response as calculated using the spalling factor for SFRC proposed by Aoude (2008) approximates the response of the cover (see Ch. 6). By looking at the normalized load strain curve it is seen that column has reached $1.1P_o$ at peak.

The strain data in Figure 4.32 (c) shows that all gauges sustain significant compressive strain after they pass their yield point, furthermore the strain data in Figure 4.32 (d) shows that the

transverse reinforcement yield eventually, but this occurs after peak column capacity is reached.

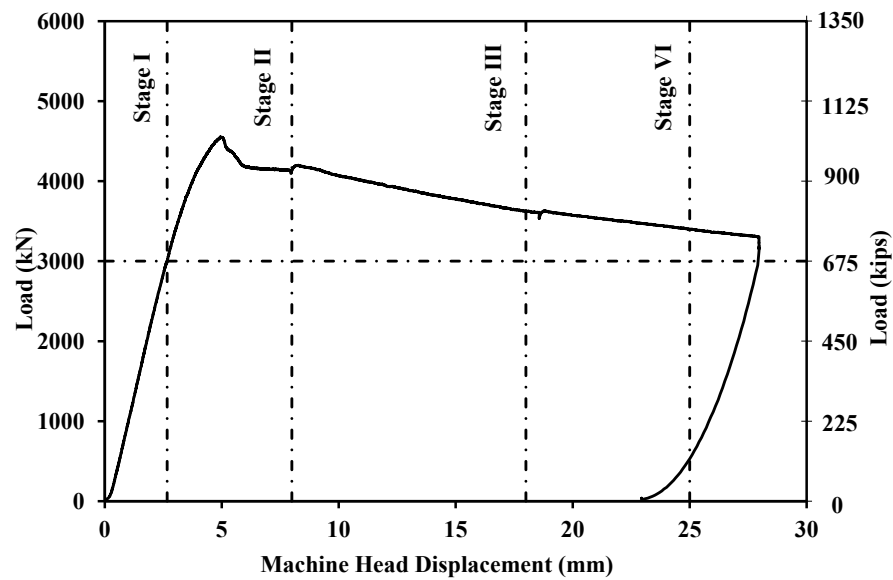


Figure 4.29 Staged Failure Process for Specimen # CR1



(a) Stage I (b) Stage II (c) Stage III (d) Stage VI

Figure 4.30 Specimen # CR1 Failure Snapshots During the Test

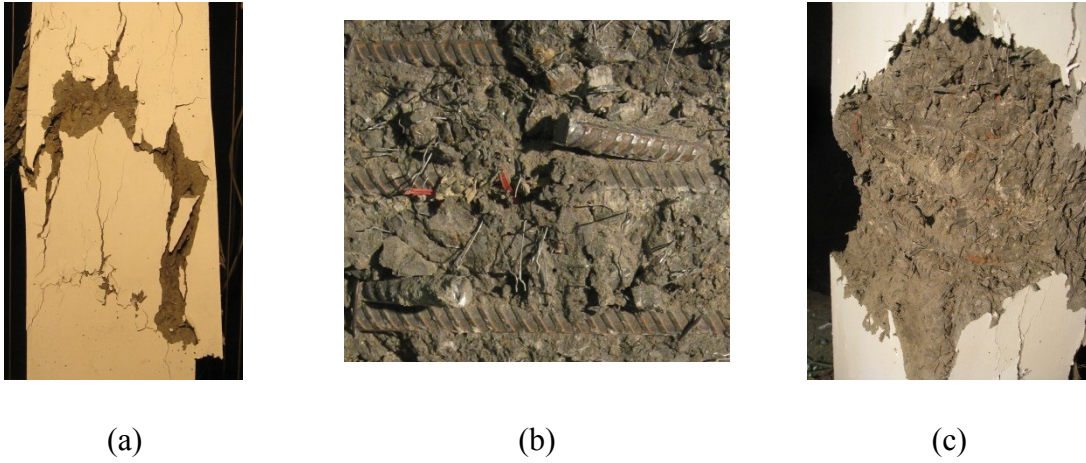


Figure 4.31 Close Look at Specimen # CR1

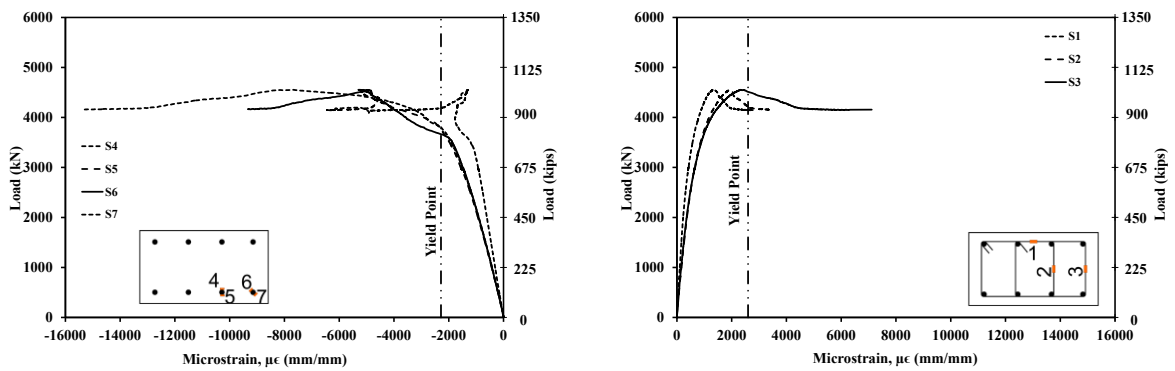
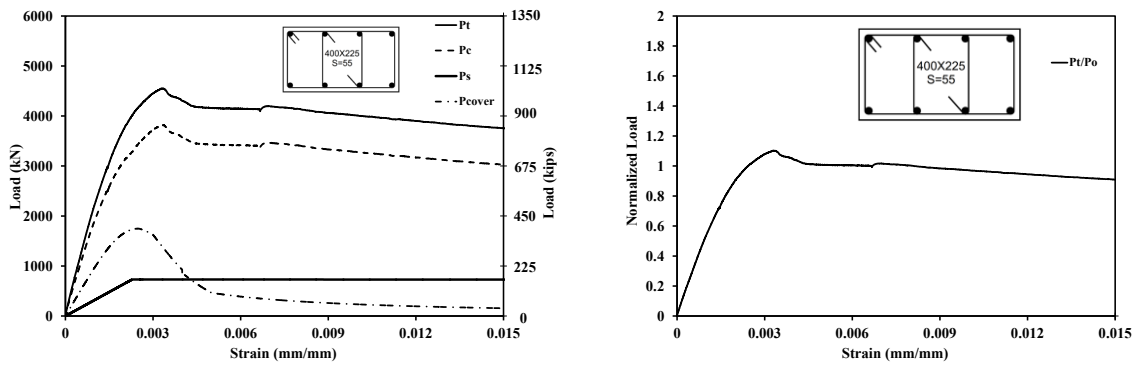


Figure 4.32 Results for Specimen # CR1

4.2.9 Specimen # CR15

Specimen # CR15 had the same details as the other specimens in the CR series ($s = 55$ mm, cover = 30 mm), however 1.5% volume of steel fibres were provided in this column. This test allows for an examination of the combined use of high confinement and high fibre content on column response.

The load-deflection response and progress of damage is shown in Figure 4.33 and Figure 4.34(a) through (d). The main events for this column are summarized below:

- First cracking for this specimen occurred 4500 kN at displacement of 5.0 mm;
- Ultimate capacity was 4967 kN at a displacement of 6 mm;
- After peak the column shows a slight drop in capacity followed by excellent post-peak response until the end of testing: with the load reaching approximately 95% and 85% of peak capacity at the end of stages II and III, respectively. At the end of testing (30 mm), the strength remained above 60 % of ultimate capacity.

Specimen CR15 had high ratio of transverse steel and fibre content, as a result this column showed excellent post-peak behaviour. Figure 4.34 (d) and Figure 4.35 show the post-failure photos for the column. The column showed well controlled and gradual cover spalling, and while cracking is observed the cover remains attached even at the end of testing (Figure 4.34 (d) and Figure 4.35 (a)). As with the other columns in this series excellent integrity in the core was maintained due to high confinement provided by the combined effect of closely spaced transverse reinforcement and provision of fibres. Examination of the core shows little crushing and high damage tolerance (Figure 4.35(b)). Upon close examination no buckling of the longitudinal rebar is observed. The 90 degree hooks on the interior cross-ties deformed slightly outward due to significant expansion of the core concrete (Figure 4.35 (c)).

The column response as recorded by the LVDTs in Figure 4.36 (a) and (b) with normalized capacity reaching 24% above the ACI suggested strength, P_o . Strain gauge data in Figure 4.36 (c) shows that yielding and large strains occur in the longitudinal steel. Similarly strain data in Figure 4.36 (d) shows that yielding occurs in the transverse steel reinforcement and sustain great tensile strain.

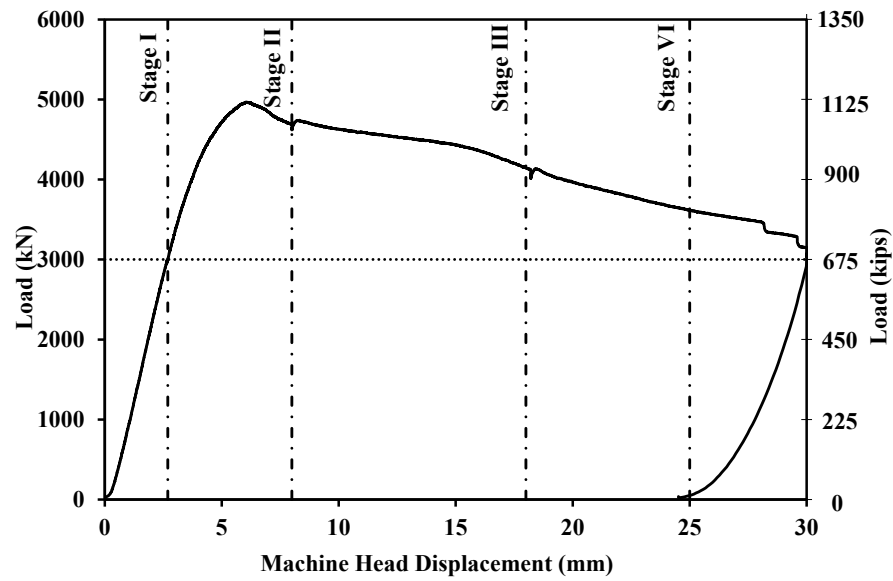


Figure 4.33 Staged Failure Process for Specimen # CR15



(a) Stage I (b) Stage II (c) Stage III (d) Stage VI

Figure 4.34 Specimen # CR15 Failure Snapshots During the Test



(a)

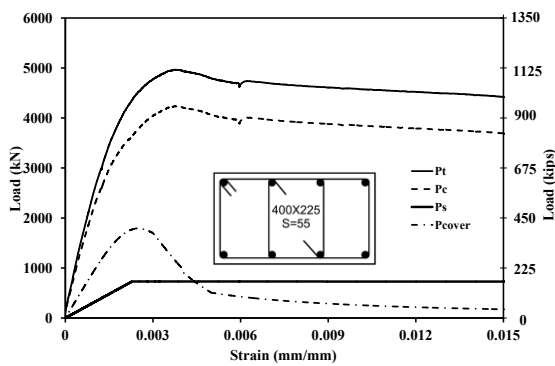


(b)

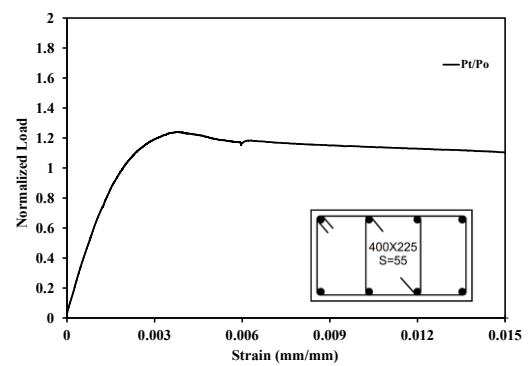


(c)

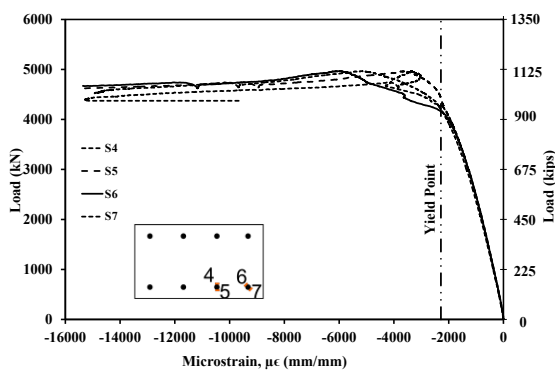
Figure 4.35 Close Look at Specimen # CR15



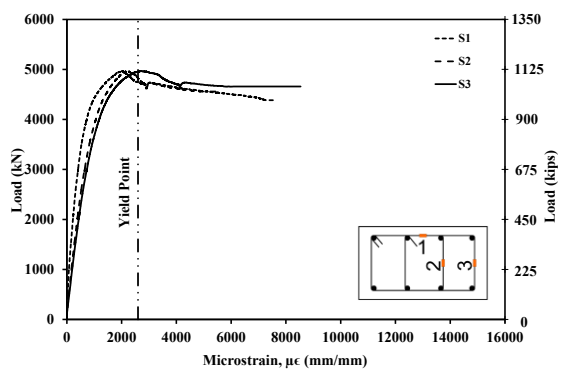
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



c) Longitudinal Reinforcement Behaviour



d) Transverse Reinforcement Behaviour

Figure 4.36 Results for Specimen # CR15

4.3 Results - SFRC Test Series (Specimens Without Cover)

This section presents the results for the seven companion SFRC specimens constructed "without cover". In addition to presenting load-displacement and progression of failure, the load-strain, normalized load-strain response for each column is presented. For brevity, the strain data is presented but not discussed in this section.

4.3.1 Specimen # AR0nc

Specimen #AR0nc had hoop spacing of 220 mm and was constructed with plain SCC (0% fibres) but was constructed without cover (a 5mm cover was provided in order to allow for placement in the in the formwork). It is the companion column to specimen AR0 which had identical details but was constructed with cover.

Figure 4.37 presents the load versus MTS recorded displacement for this specimen and the snapshots of the specimen through the various load stages are shown in Figure 4.38. The main events for this specimen are summarized below:

- First cracking appeared at deflection of 4.3 mm with a corresponding load of 3339 KN;
- The specimen reached peak load of 3503 KN at a deflection of 5.3 mm;
- This was followed by sudden drop in load carrying capacity to 40% of peak load, the drop in load was associated with crushing of core concrete and buckling of the longitudinal reinforcement;
- At the end of stage II the residual strength was 25% of peak, and it was decided to end the test at 14 mm (the strength was 14 % of peak at this point).

Figure 4.39 presents some post-failure photos for this specimen. Bar buckling in all longitudinal bars especially in the corner is clear. In addition, it is observed that a large section of the core concrete has crushed and disintegrated deep in the core (arching action between the ties is also evident). The load-strain, normalized load-strain and strain data for this specimen is shown in Figure 4.40. The poor performance of this column is due to the large hoop spacing which results in low confinement and large unsupported length which

results in instability and buckling of the longitudinal reinforcement. It is noted that peak normalized load reaches $1.03 P_o$ for this column.

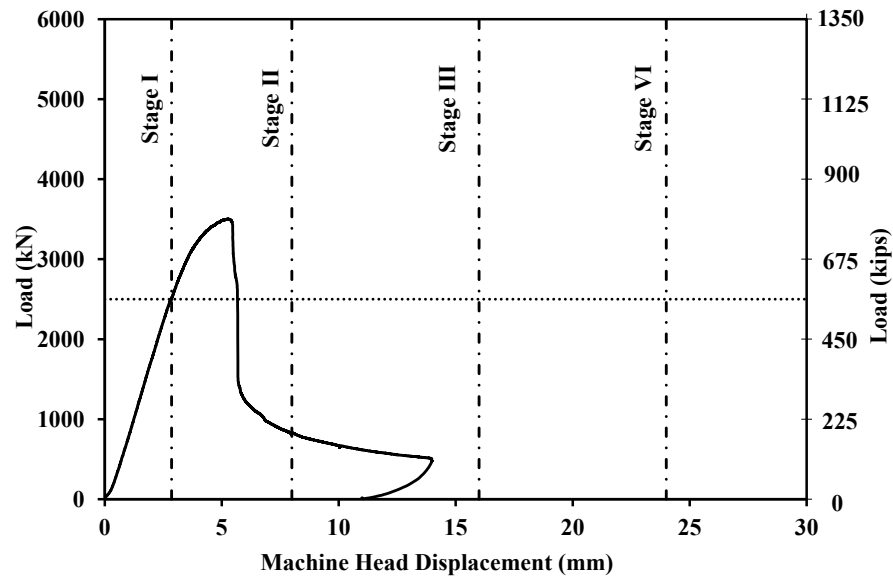


Figure 4.37 Staged Failure Process



(a) Stage I

(b) Stage II

(c) Stage IIIa

(d) Stage IIIb

Figure 4.38 Column Failure Snapshots During the Test

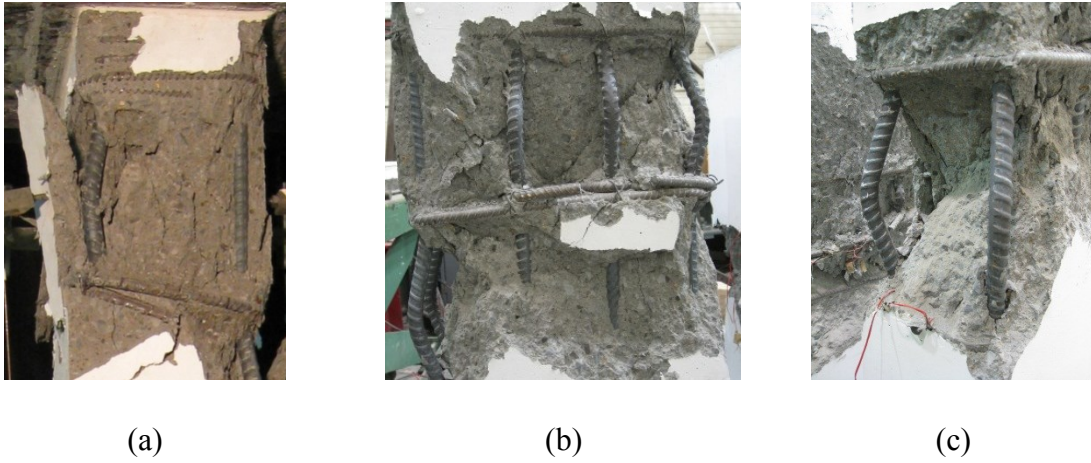
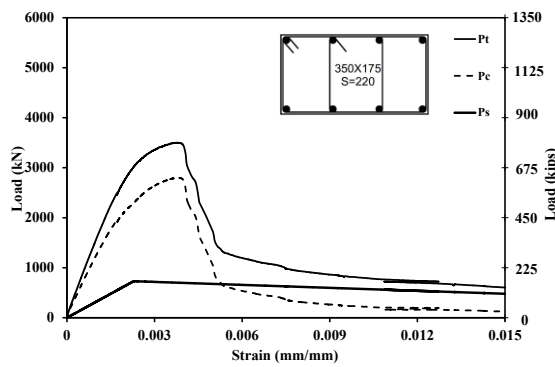
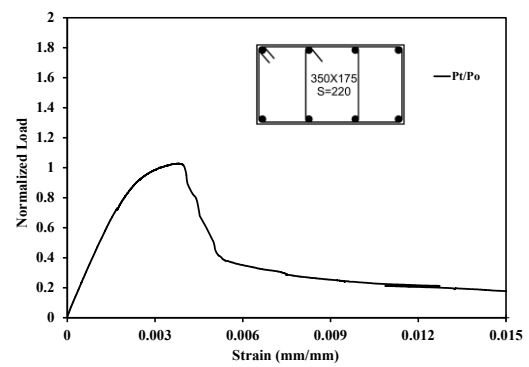


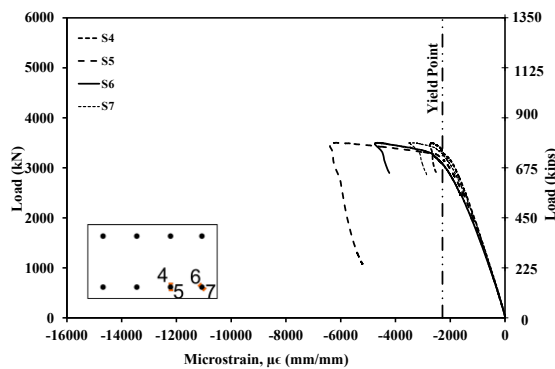
Figure 4.39 Close Look at Specimen # AR0nc



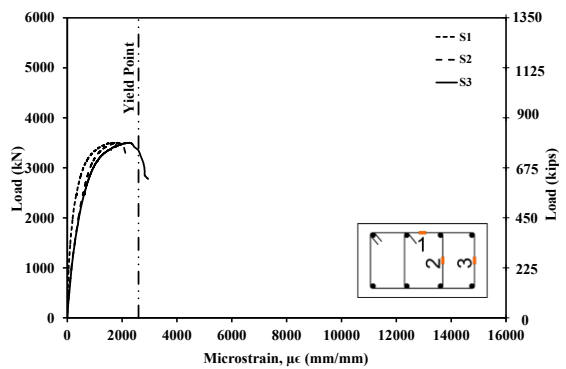
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



(c) Longitudinal Reinforcement Behaviour



(d) Transverse Reinforcement Behaviour

Figure 4.40 Results for Specimen # AR0nc

4.3.2 Specimen # AR1nc

Specimens #AR1nc had identical details to specimen AR0nc ($s = 220$ mm, no cover) but was reinforced with 1% steel fibres. It is a companion column to specimen AR1 (which had a 30 mm cover).

The load-displacement response and failure process for this column is shown in Figure 4.41 and Figure 4.42, respectively. The main events for this column are as follows:

- First cracking appeared at deflection of 4.0 mm with a corresponding load of 2987 KN
- A peak load of 3275 KN at deflection of 5.6 mm was recorded for this specimen;
- After reaching peak, a rapid yet relatively more gradual drop in capacity is observed for this specimen when comparing to specimen AR0nc, with the load reaching 2000 KN (61% of peak) at the end of Stage II;
- After Stage II the failure rate became more gradual and testing continued until 27 mm at which point the strength had dropped to 15% of peak capacity.

Figure 4.42(a) and Figure 4.43 show the specimen at the end of testing. It is noted that the small 5 mm cover (which had been used to allow for construction) remained attached to the specimen. Removal of this thin cover showed significant bar buckling, particularly in the corner bars. Moreover while greater core integrity is maintained due to provision of fibres, significant crushing is observed deep inside the core, with crushing being more severe in between the tie levels (arching action).

The load-strain, normalized, load-strain and reinforcement strain gauge data is presented in Figure 4.44. This column reached a peak capacity of 13 % above ACI nominal strength, P_o .

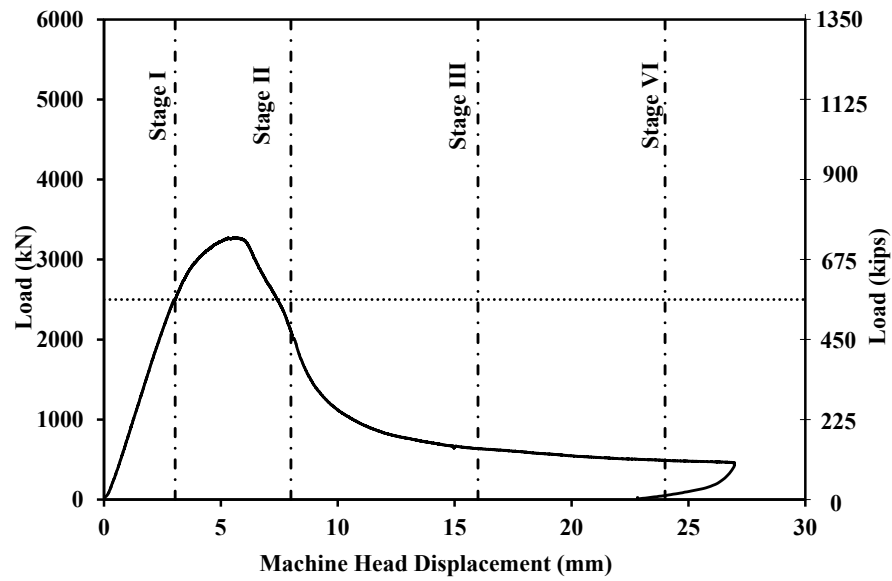
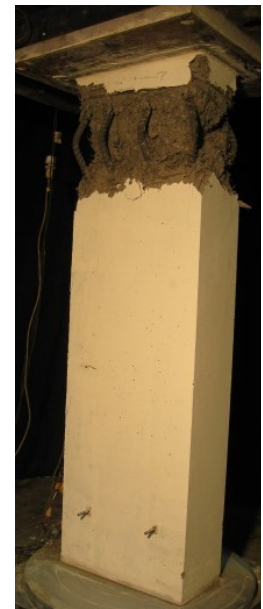
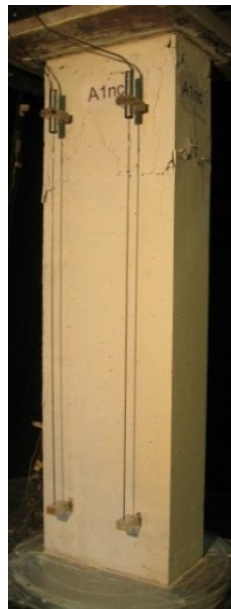
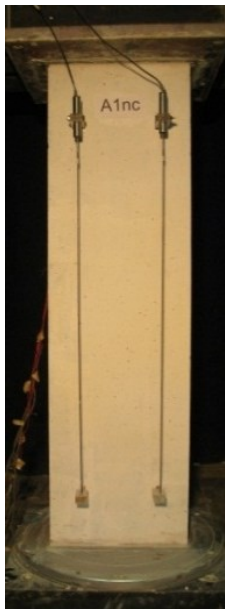


Figure 4.41 Staged Failure Process for Specimen # AR1nc



(a) Stage I

(b) Stage II

(c) Stage III

(d) Stage IV

Figure 4.42 Specimen # AR1nc Failure Snapshots During the Test



(a)

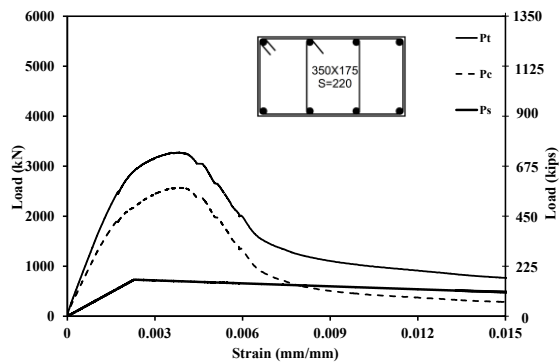


(b)

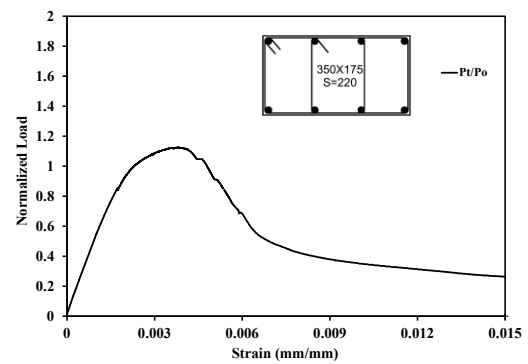


(c)

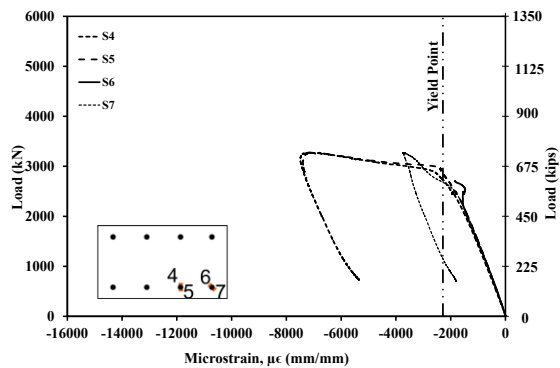
Figure 4.43 Close Look at Specimen # AR1nc



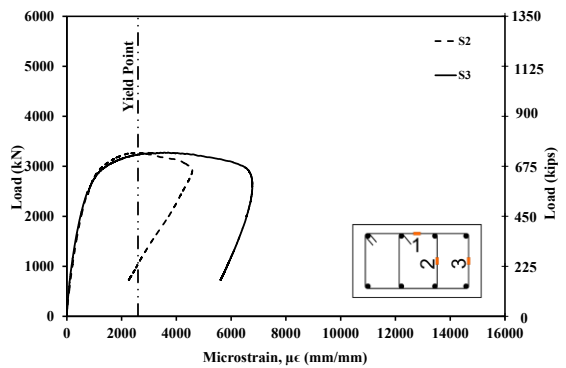
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



(c) Longitudinal Reinforcement Behaviour



(d) Transverse Reinforcement Behaviour

Figure 4.44 Results for Specimen # AR1nc

4.3.3 Specimen # AR15nc

Specimen# AR15nc was the third specimen in the ARnc series ($s = 220$ mm, no cover) and was reinforced with 1.5 % fibres.

Figure 4.45 and Figure 4.46 present the load-displacement and failure process for this specimen. The main events are summarized below:

- First cracking appeared at deflection of 4.7 mm and load of 3100 KN
- The peak load reached 3177 KN at deflection of 5 mm;
- After peak, the load decreased at a steady yet gradual rate reaching 2000 KN at the end of stage II (63% of peak);
- The residual strength at the end of the test (28 mm) was 16 % of peak capacity.

The post-failure photos for this specimen are shown in Figure 4.47. Similar to previous specimen, thin cover layer of cover remains attached to the specimen at the end of the test. Removing the cover reveals severe longitudinal rebar buckling, particularly in the corner bars. Furthermore crushing is observed in the core region.

Figure 4.48 presents the load-strain, normalized load-strain and strain data for this specimen. This column reaches a normalized load of 1.11 P_o .

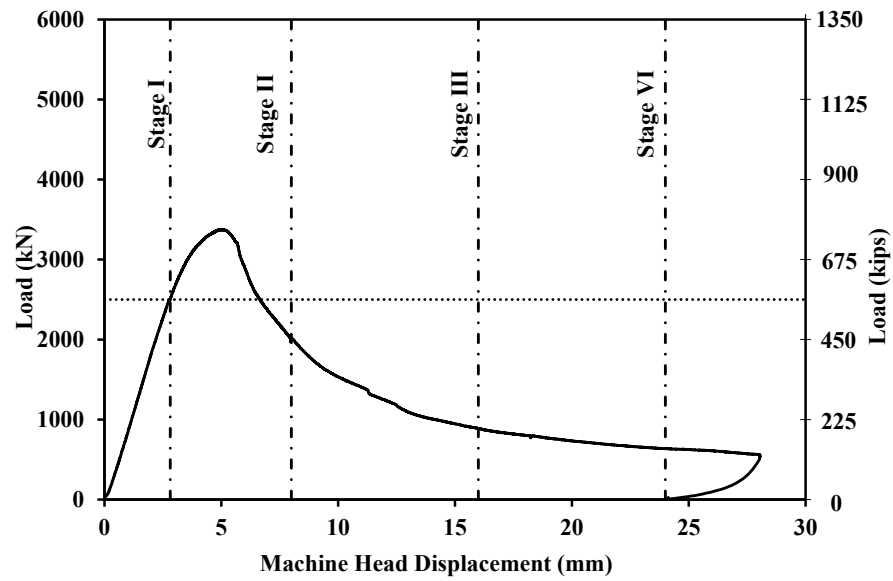


Figure 4.45 Staged Failure Process for Specimen # AR15nc



(a) Stage I

(b) Stage II

(c) Stage III

(d) Stage VI

Figure 4.46 Specimen # AR15nc Failure Snapshots During the Test



(a)

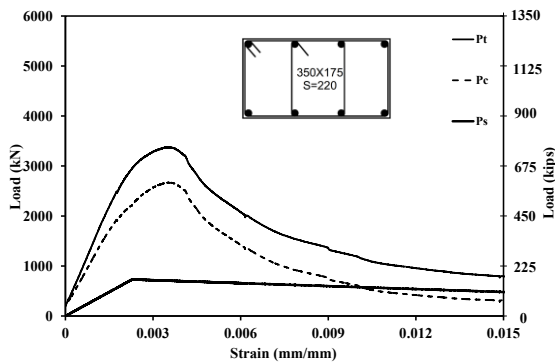


(b)

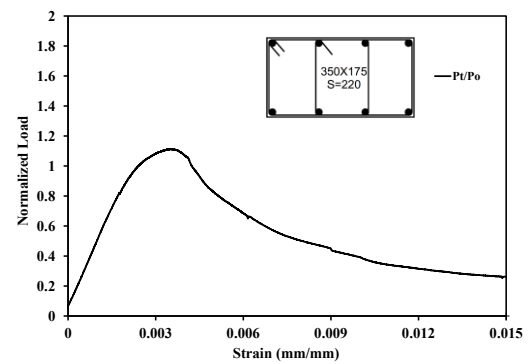


(c)

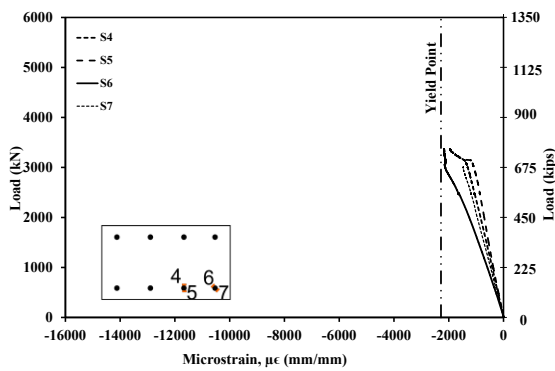
Figure 4.47 Close Look at Specimen # AR15nc



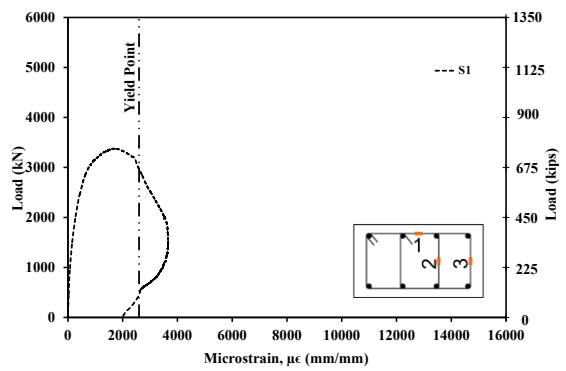
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



c) Longitudinal Reinforcement Behaviour



d) Transverse Reinforcement Behaviour

Figure 4.48 Results for Specimen # AR15nc

4.3.4 Specimen # BR0nc

Specimen # BR0nc was detailed in accordance with the requirements for moderately ductile columns and had tie spacing of 110 mm, 0% of fibres and no cover. It is the companion column to specimen BR0 which had identical details but was constructed with cover.

Figure 4.49 and Figure 4.50 show the load-displacement and failure process for this specimen. The main events for this specimen are summarized below:

- First cracking appeared at deflection of 4.6 mm with a corresponding load of 3325 KN;
- The specimen reached peak load of 3318 KN at a deflection of 4.6 mm;
- This was followed by a more gradual drop in load carrying capacity than what was observed in specimen AR0;
- The residual strength reaches approximately 70% and 30% of peak at the end of stages II and III, respectively, and the strength was 20 % of peak capacity at the end of testing (24mm).

Figure 4.51 shows the post-failure photos for this column. As shown in the figures, the 5 mm cover has spalled off with thick pieces of the core. The 90 degree hooks on the intermediate cross-ties have also deformed outward due to pressure exerted on the ties. Bar buckling of the longitudinal reinforcement and crushing of the core is clear, although core integrity is greater when compared to specimen AR0.

Figure 4.52 shows the load-strain, normalized load-strain and reinforcement strain gauge data for this column. The peak normalized load for this column reaches $0.99 P_o$.

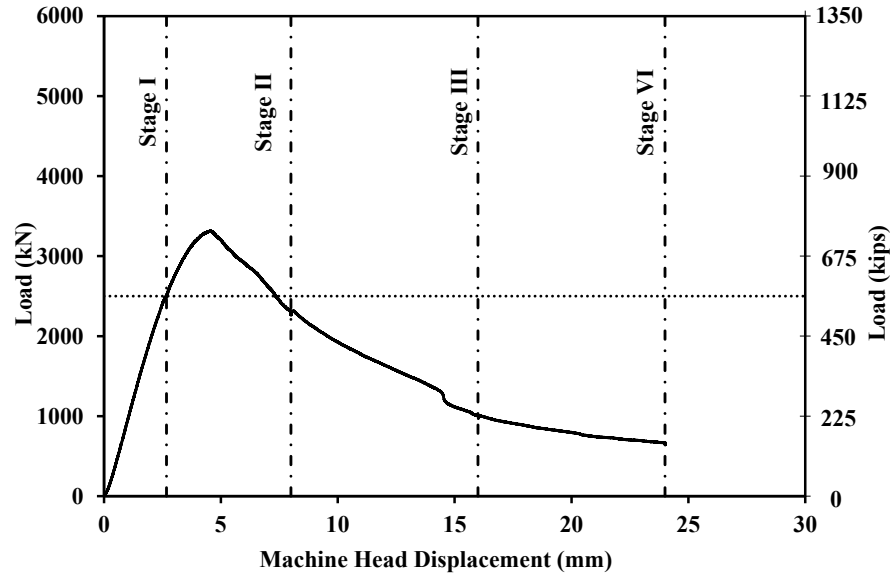


Figure 4.49 Staged Failure Process for Specimen # BR0nc



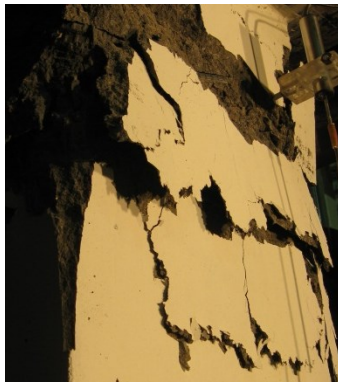
(a) Stage I

(b) Stage II

(c) Stage III

(d) Stage VI

Figure 4.50 Specimen # BR0nc Failure Snapshots During the Test



(a)

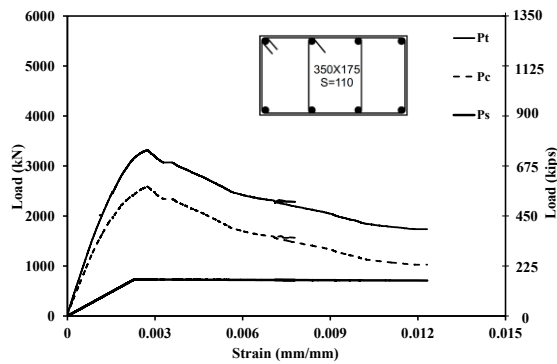


(b)

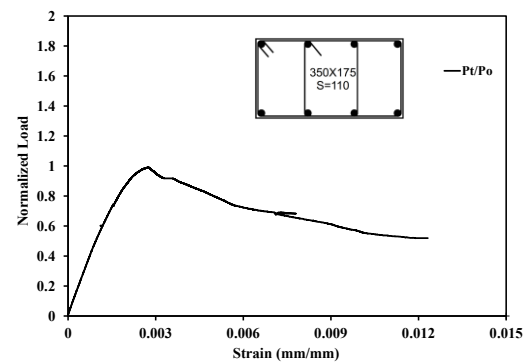


(c)

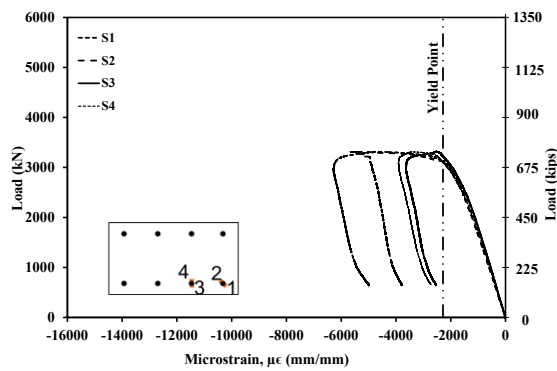
Figure 4.51 Close Look at Specimen # BR0nc



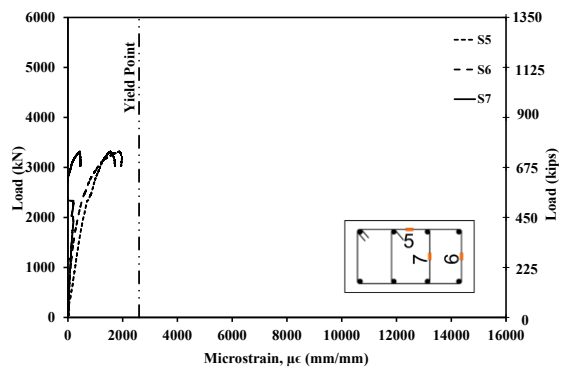
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



c) Longitudinal Reinforcement Behaviour



d) Transverse Reinforcement Behaviour

Figure 4.52 Results for Specimen # BR0nc

4.3.5 Specimen # BR1nc

Specimens# BR1nc had the same details as specimen BR0nc ($s = 110$ mm, no cover) but was reinforced with 1% steel fibres.

Figure 4.53 and Figure 4.54 present the load-displacement and failure process for BR0nc. The main events are summarized below:

- Prior to any visible signs of cracking the specimen reached a peak load of 2630 KN at deflection of 4.6 mm;
- After reaching peak, cracking became visible at 5.3 mm with failure localizing in the top region of the column;
- Despite the low strength, the column shows better than expected post-peak response, with the load remaining at 65 % of the peak strength at the end of the test (27 mm) .

Figure 4.55 shows the post-failure status for this column, where cracking is well distributed in the failure region. Bar buckling with spacing is not prevented and a part of core has crushed and failed. The 90 degree hook on the interior cross-ties has deformed outward due to lateral expansion of the core.

Load-strain response, normalized load-strain response and data from the strain gauges are presented in Figure 4.56. Taking into account the low cylinder strength for this column, the normalized peak total load reaches 16 % above P_o .

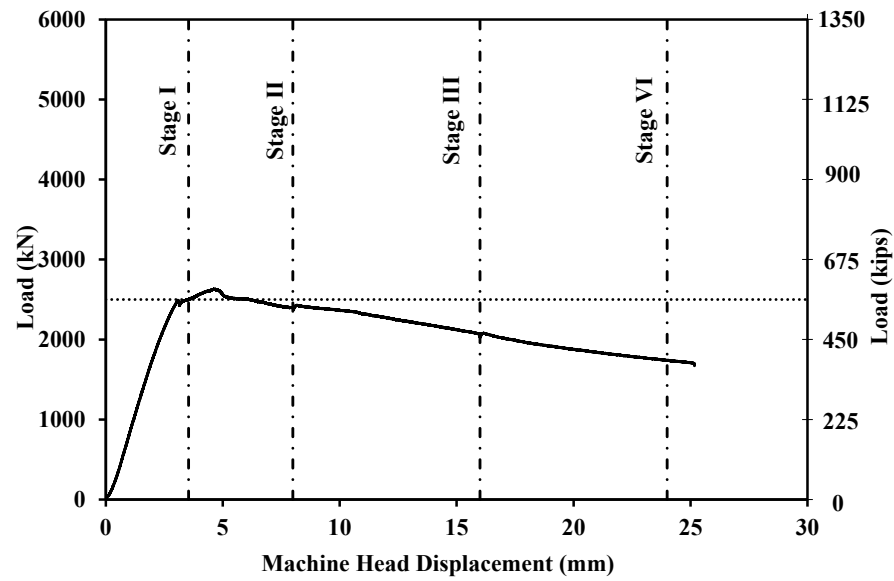


Figure 4.53 Staged Failure Process for Specimen # BR1nc



(a) Stage I

(b) Stage II

(c) Stage III

(d) Stage VI

Figure 4.54 Specimen # BR1nc Failure Snapshots During the Test

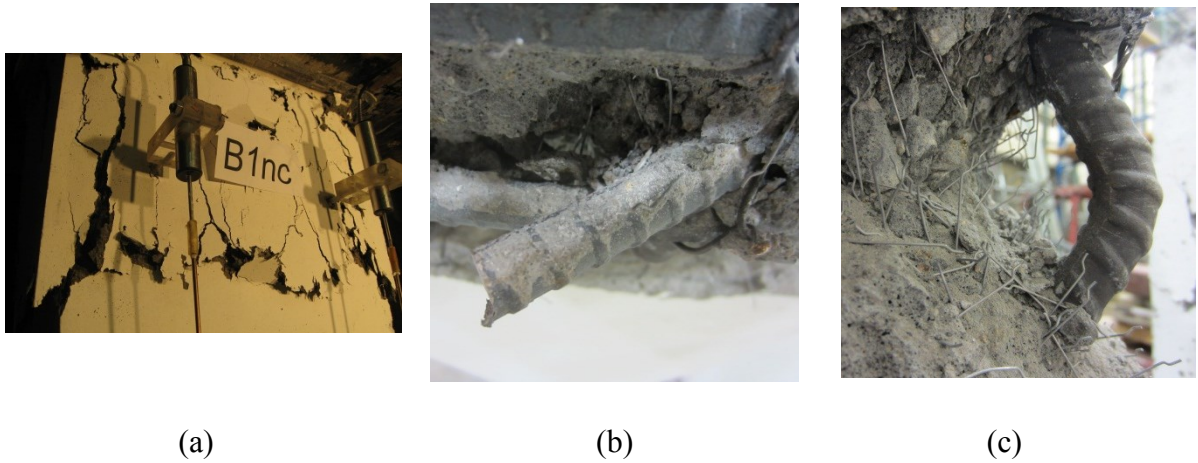
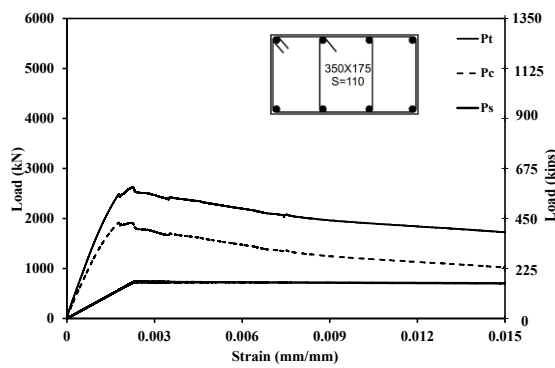
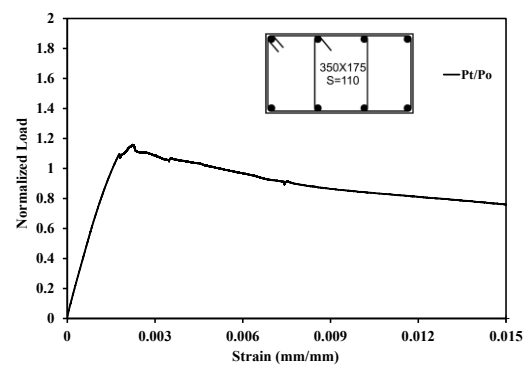


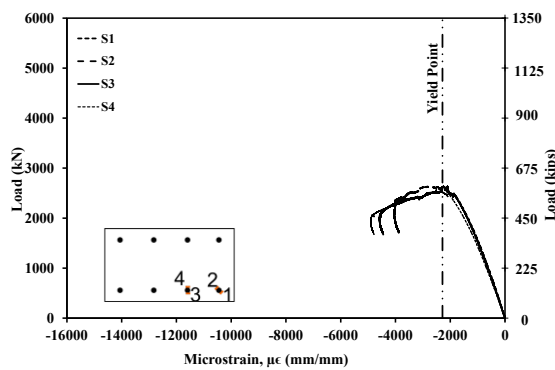
Figure 4.55 Close Look at Specimen # BR1nc



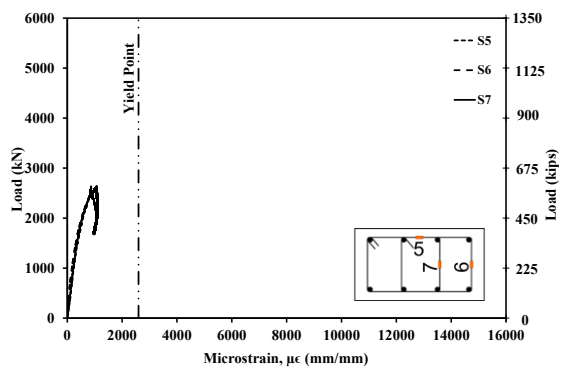
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



c) Longitudinal Reinforcement Behaviour



d) Transverse Reinforcement Behaviour

Figure 4.56 Results for Specimen # BR1nc

4.3.6 Specimen # BR15nc

Specimen # BR15nc had the same detailing as the other BRnc series columns ($s = 110$ mm, no cover) but had a fibre content of 1.5%.

Figure 4.57 and Figure 4.58 present the load-displacement and failure process for this specimen. The main events are summarized below:

- First cracking appeared at deflection of 3.94 mm and load of 2370 KN
- The peak load reached 2493 KN at deflection of 3.42 mm;
- The post-peak response resembles that of specimen BR1nc showing a gradual yet steady drop in capacity with increasing strains;
- The residual strength at the end of the test (25 mm) was 40 % of peak capacity.

The post-failure photos for this specimen are shown in Figure 4.59 and similar observations to those made for specimen BR1nc are seen: spalling of the 5 mm cover is gradual and cracking is well distributed and dense in the failure zone, buckling of the reinforcement occurs, opening of the 90 degree hooks is observed, and controlled crushing occurs in the core region.

Figure 4.60 presents the load-strain, normalized load-strain and strain data for this specimen. Taking into account the low concrete cylinder strength, this column reaches a normalized load of $1.09 P_o$.

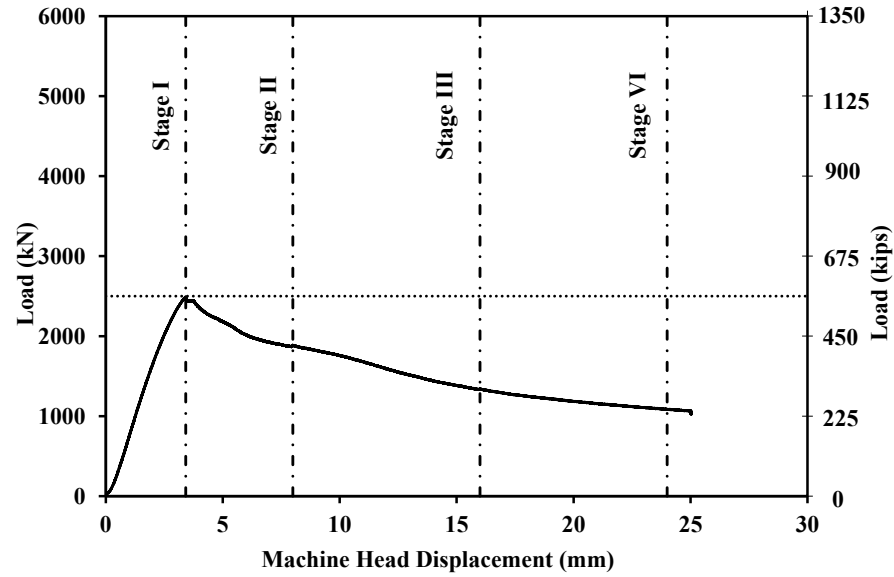


Figure 4.57 Staged Failure Process for Specimen # BR15nc



(a) Stage I (b) Stage II (c) Stage III (d) Stage VI

Figure 4.58 Specimen # BR15nc Failure Snapshots During the Test



(a)

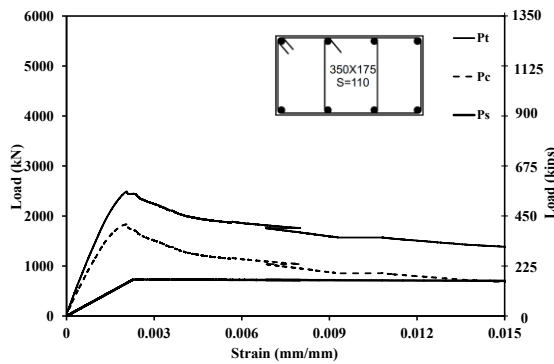


(b)

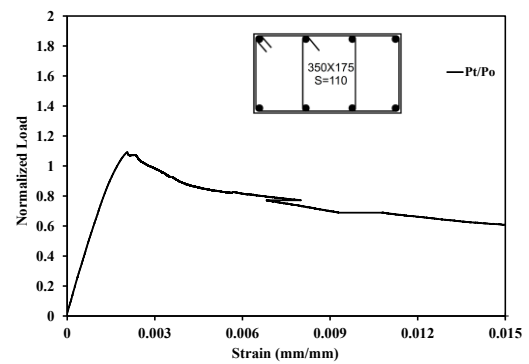


(c)

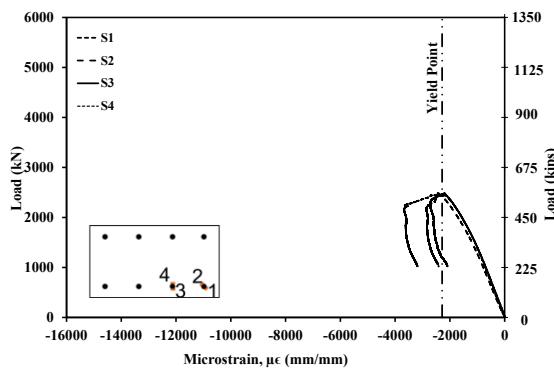
Figure 4.59 Close Look at Specimen # BR15nc



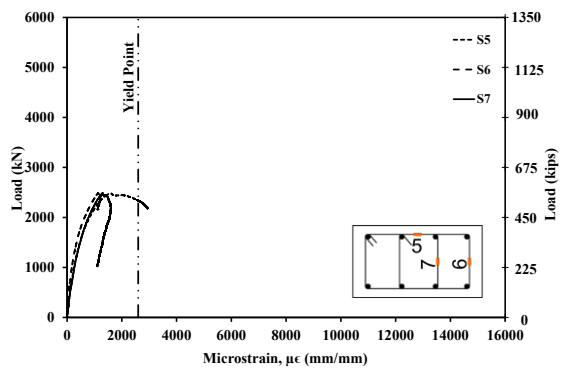
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



c) Longitudinal Reinforcement Behaviour



d) Transverse Reinforcement Behaviour

Figure 4.60 Results for Specimen # BR15nc

4.3.7 Specimen # CR0nc

Specimen # CR0nc was detailed in accordance with the provisions for ductile columns in the CSA A23.3 standard and had a tie 55 mm and was constructed with plain SCC. The column is a companion to specimen CR0, with the exception being that this column was constructed without cover.

Figure 4.61 and Figure 4.62 present load-displacement response and failure progression for this column. The main events are summarized below:

- The column reached ultimate capacity of 3652 kN at a displacement of 5.1 mm;
- After reaching peak capacity the post-peak response shows almost plastic behaviour with the capacity reducing at a slow rate;
- At the end of testing (24 mm) the capacity remains at about 70% of peak capacity.

The post-failure status of the column is shown in Figure 4.63. Spalling of the thin construction cover occurs however buckling of the reinforcement is effectively prevented and core integrity is clear. Figure 4.64 shows the load-strain, normalized load-strain and strain gauge data for this column. The peak normalized load for this column reaches 1.1 P_o .

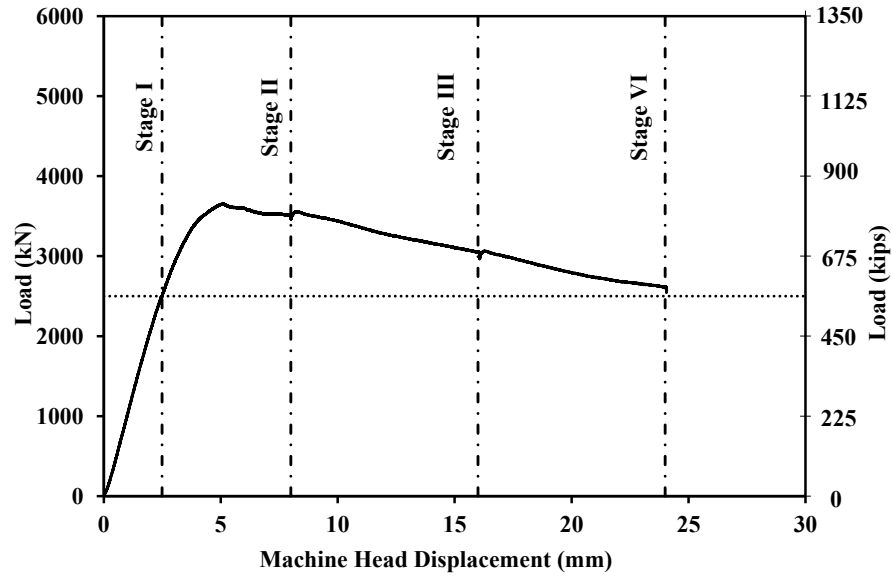
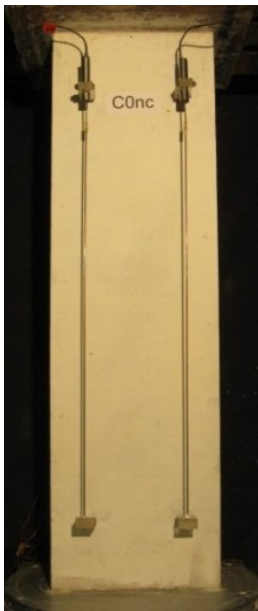


Figure 4.61 Staged Failure Process for Specimen # CR0nc



(a) Stage I

(b) Stage II

(c) Stage III

(d) Stage VI

Figure 4.62 Specimen # CR0nc Failure Snapshots During the Test

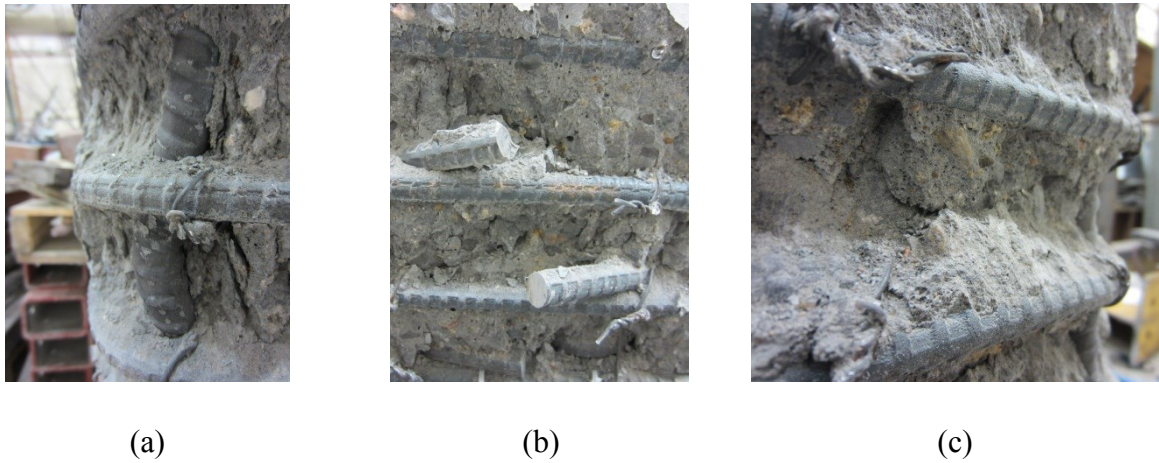


Figure 4.63 Close Look at Specimen # CR0nc

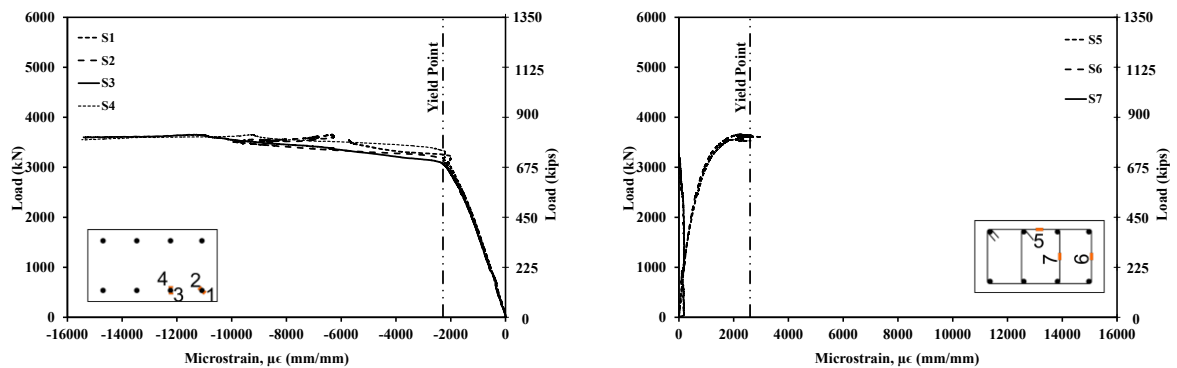
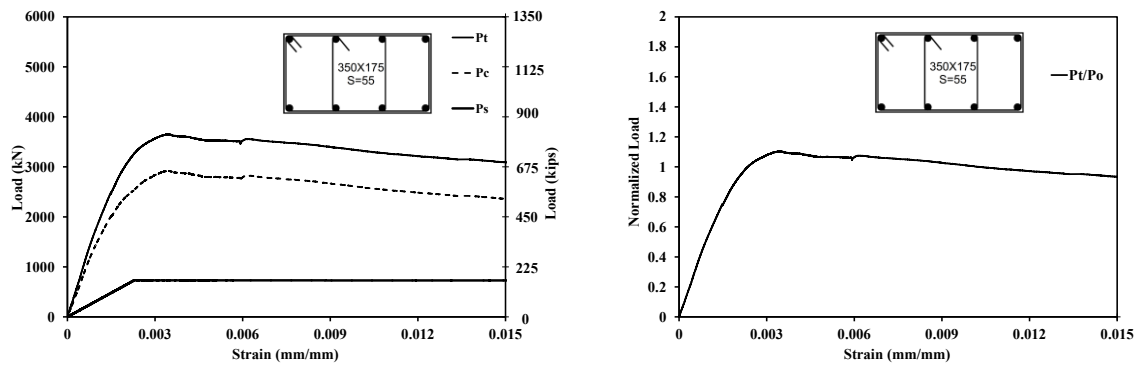


Figure 4.64 Results for Specimen # CR0nc

4.3.8 Specimen # CR1nc

Specimen # CR1nc had tie spacing of 55 mm and no cover, matching the details of CR0nc, but was reinforced with 1% steel fibres.

Figure 4.65 and Figure 4.66 present the load-displacement and failure process for this specimen. The main events are summarized below:

- First cracking appeared at deflection of 5.1 mm and load of 2600 KN
- The peak load reached 2970 KN at deflection of 11 mm, which is later than what was observed in previous specimens;
- The post-peak response shows an almost plastic post-peak behaviour and the residual strength remained above 85% at the end of testing (28 mm).

The post-failure photos for this specimen are shown in Figure 4.67 . Core integrity is maintained even at high strains and buckling of the reinforcement is effectively prevented. Figure 4.60 presents the load-strain, normalized load-strain and strain gauge data for this specimen. This column reaches a normalized load of 1.12 P_o .

It is noted that this specimen had lower than expected concrete cylinder strength (38 MPa). This combined with the high degree in confinement can possibly explain the larger than expected peak strain.

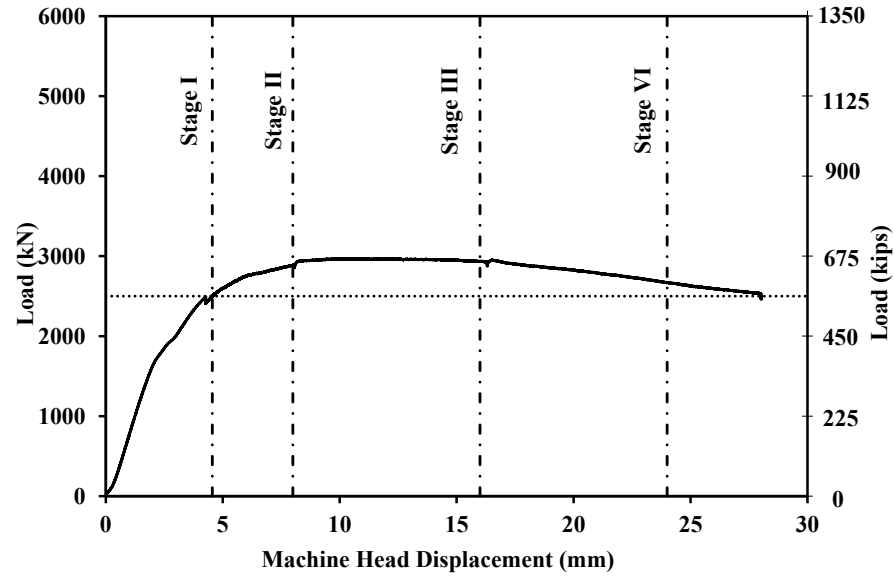


Figure 4.65 Staged Failure Process for Specimen # CR1nc



(a) Stage I

(b) Stage II

(c) Stage III

(d) Stage VI

Figure 4.66 Specimen # CR1nc Failure Snapshots During the Test



(a)

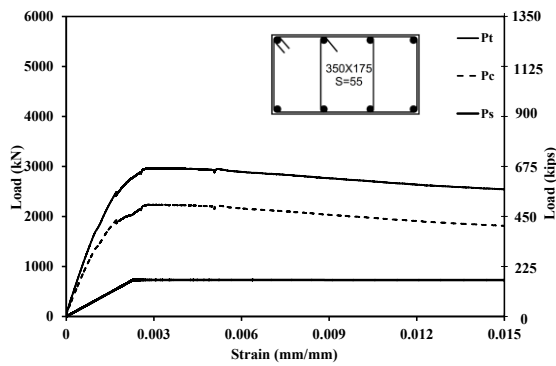


(b)

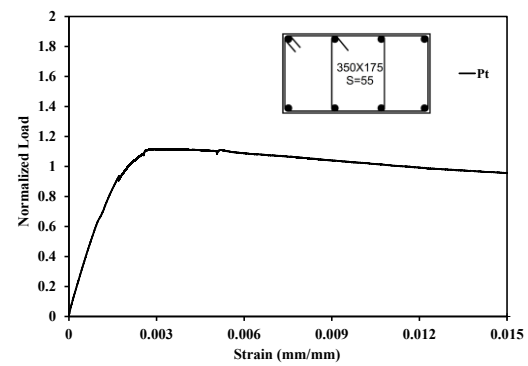


(c)

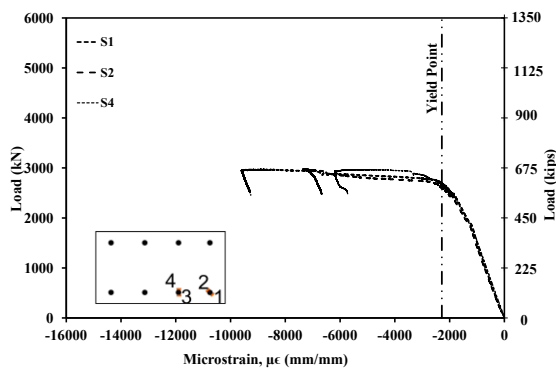
Figure 4.67 Close Look at Specimen # CR1nc



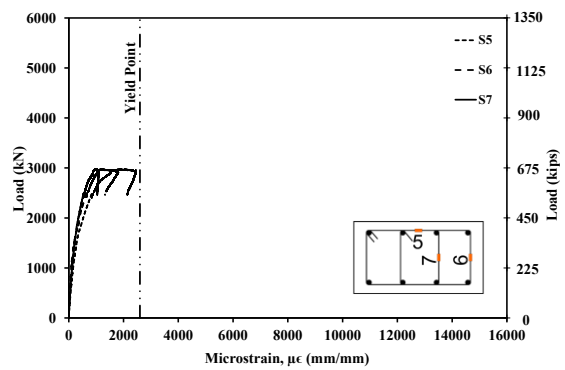
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



c) Longitudinal Reinforcement Behaviour



d) Transverse Reinforcement Behaviour

Figure 4.68 Results for Specimen # CR1nc

4.4 Ultra High Strength Specimens

This section presents the results for the six UHPFRC specimens tested in the experimental program. In addition to presenting load-displacement and progression of failure, the load-strain, normalized load-strain and strain gauge data is presented for each specimen.

4.4.1 Specimen # C3-120

Specimen # C3-120 was the first column tested in the C3 series (8 bar configuration) and was detailed in accordance with the requirements for moderately ductile columns with a tie spacing of 120 mm. As with all columns in the UHPFRC series, the column had cross-section of 250 x 250 mm, 10 mm cover and 2.5 % steel fibres. Figure 4.69 presents the load versus machine head displacement response for the column and the main events as recorded during testing are summarized below:

- First cracking for this specimen was observed at displacement of 4.98 mm and a corresponding load of 8260 kN;
- The column reached ultimate capacity of 9069 kN at a displacement of 6.13 mm;
- After peak capacity, the load decreased rapidly to 4000 KN after which the rate of failure reduced until the end of the test;
- The column reached residual strengths of approximately 65%, 50% and 42% of peak capacity at the end of stages II-IV respectively, eventually reaching almost 38 % of peak at the end of testing (20 mm);

The progression of failure for this specimen is shown in Figure 4.70. When comparing to the normal strength specimens, the failure process is noticeably different. After initial cracking (which occurred along the mid-level tie), micro cracks developed throughout the specimen, first along the corner bars and ties and then all over the column surface. Unlike what would be expected in conventional high-strength concrete, spalling of the 10 mm cover was gradual and well controlled. Although some spalling occurs, the cover remained attached until the end of testing. Post-failure photos are shown in Figure 4.71. After the test the cover was removed manually, and a close investigation revealed the core to have very little sign of crushing, although bar buckling was evident. The 90-degree hook on the cross-tie opened outwards under the large lateral pressure exerted by the core.

Figure 4.72 (a) presents the load versus axial strain response, while the normalized response is plotted in Figure 4.72 (b). Peak capacity reaches 30% above the nominal capacity prescribed by the ACI A23.3 Standard ($1.30 P_o$). The longitudinal strain gauge data in Figure 4.8 (c) shows some fluctuation however yielding is clear with the bars sustaining large compressive strains. Figure 4.8 (d) shows that the ties reach yield before peak, failing slightly after this point.

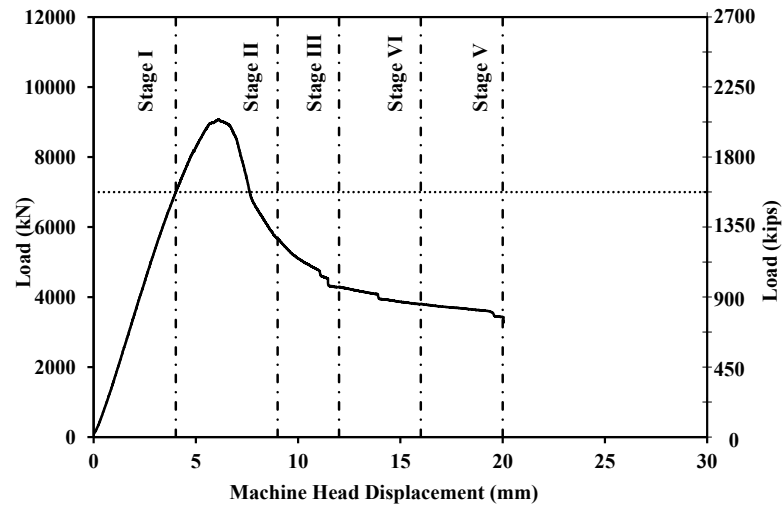


Figure 4.69 Staged Failure Process for Specimen # C3-120

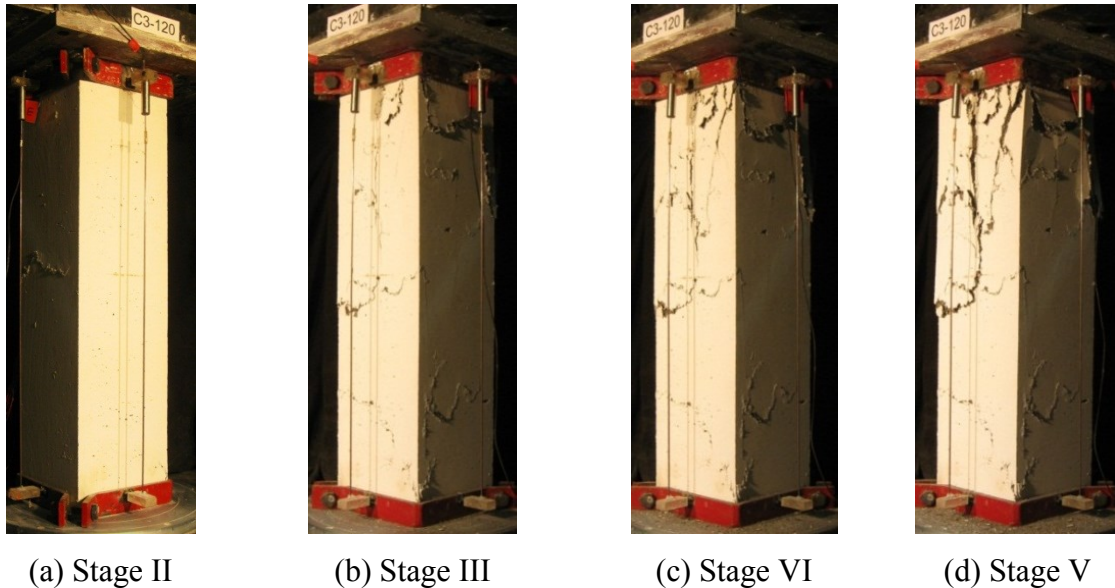


Figure 4.70 Specimen # C3-120 Failure Snapshots During the Test

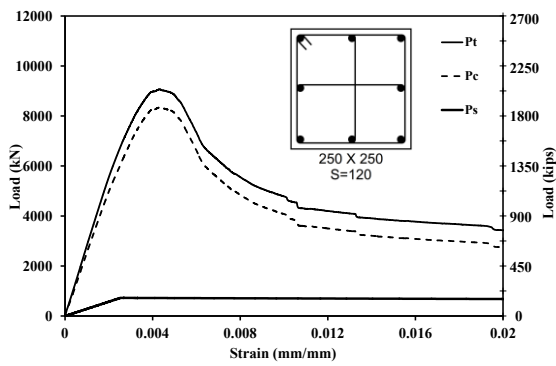


(a)

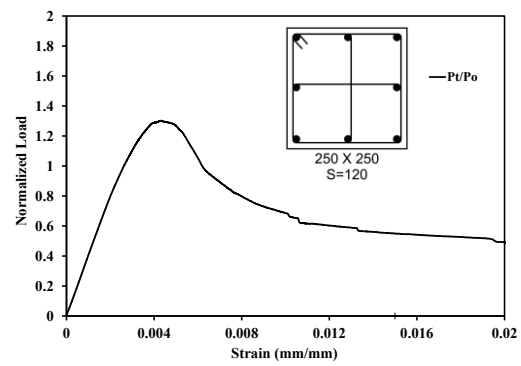
(b)

(c)

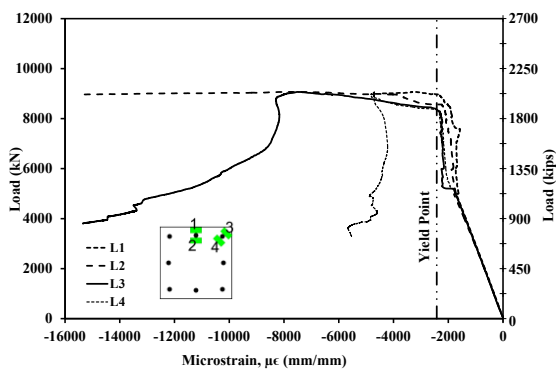
Figure 4.71 Close Look at Specimen # C3-120



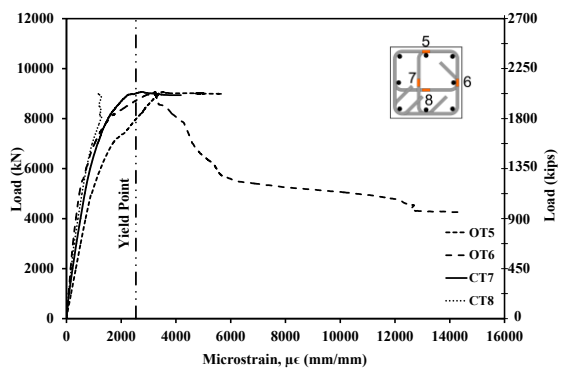
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



c) Longitudinal Reinforcement Behaviour



d) Transverse Reinforcement Behaviour

Figure 4.72 Results for Specimen # C3-120

4.4.2 Specimen # C3-80

Specimen # C3-80 was the second column tested in the C3 series and had similar properties to the previous specimen, but had a tie spacing of 80 mm. This spacing is intermediate between the $R_d=2.5$ and 4.0 requirements of the CSA A23.3 standard, and allows for further examination of the effect of tie spacing on confinement in UHPFRC columns.

The load-displacement response is shown in Figure 4.74 and the main events for this specimen are as follows:

- First cracking appeared at deflection of 6.3 mm with a corresponding load of 9700 KN
- A peak load of 9946 KN at deflection of 7.1 mm was recorded for this specimen;
- After reaching peak, a rapid drop in capacity was observed and this was followed by a more gradual strength decay;
- The column reached residual strengths of, 76% and 50% of peak capacity at the end of stages III and IV respectively, eventually reaching 4200 KN (42 % of peak) at the end of testing (20 mm);

The progression of failure in this column is shown in Figure 4.74. Initial cracking occurred on the side of the column at the tie levels, thereafter cracking increased and distributed throughout the column. The cover failed gradually and remained attached to the column until the end of testing. The post-failure photos for the column are shown in Figure 4.75. After manually removing the cover, bar buckling is observed while the core showed little sign of damage. As with the previous test the 90 degree hook on the intermediate cross-tie deformed outwards.

The load-strain and normalized load-strain plots are shown in Figure 4.76. The results show that the column reached peak strength of 25 % above P_o and stayed above 50 % P_o until the end of testing. The strain data in Figure 4.76 (c) shows yielding and large compressive strains in the longitudinal reinforcement. Similarly Figure 4.76 (d) shows that yielding has occurred in the transverse reinforcements near peak column capacity.

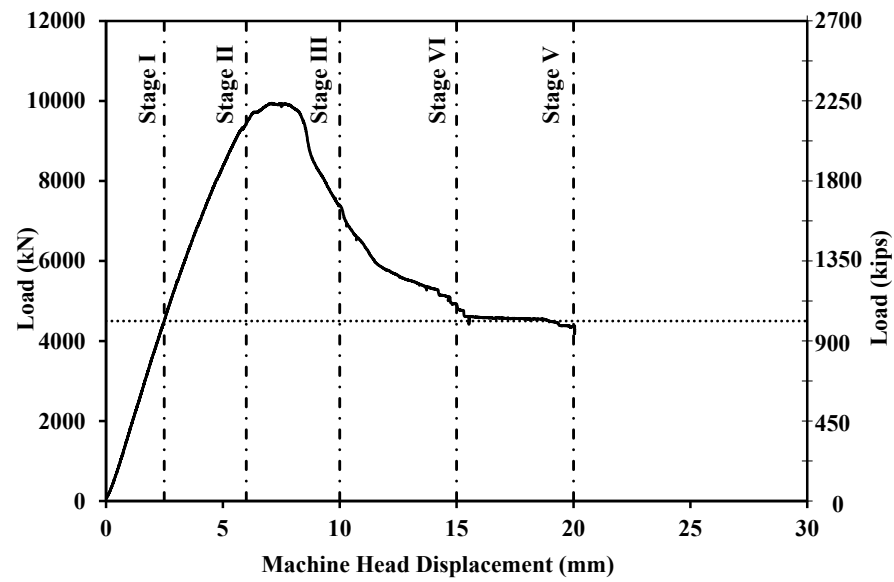


Figure 4.73 Staged Failure Process for Specimen # C3-80



(a) Stage II

(b) Stage III

(c) Stage VI

(d) Stage V

Figure 4.74 Specimen # C3-80 Failure Snapshots During the Test



(a)

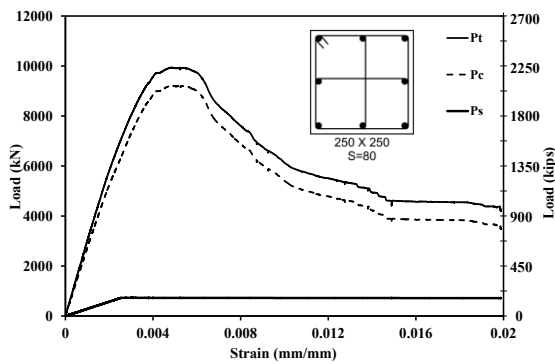


(b)

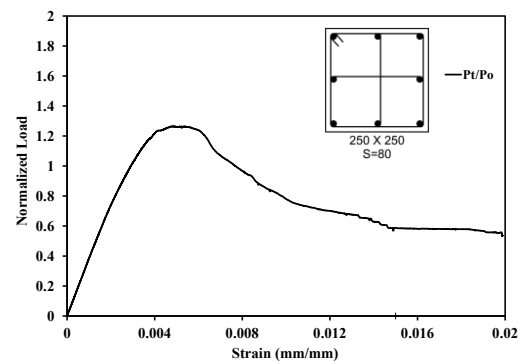


(c)

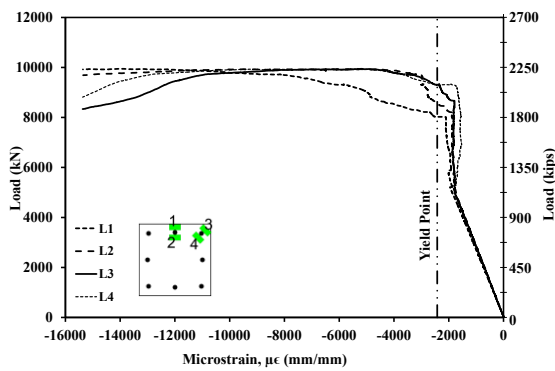
Figure 4.75 Close Look at Specimen # C3-80



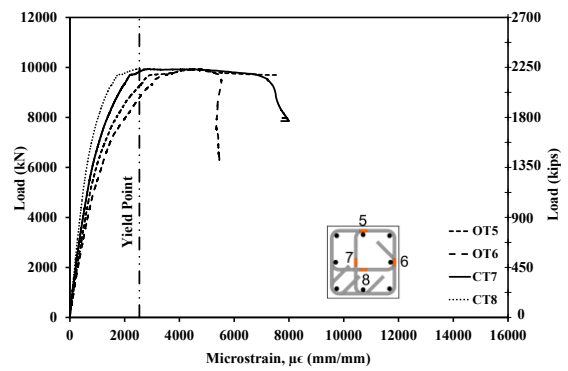
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



c) Longitudinal Reinforcement Behaviour



d) Transverse Reinforcement Behaviour

Figure 4.76 Results for Specimen # C3-80

4.4.3 Specimen # C3-60

Specimen # C3-60 was the third column in the C3 test series (250 mm x 250 mm section, 8 bar configuration, 10 mm cover, and 2.5% fibre content) but was detailed with a tie spacing of 60 mm. In addition to providing further insight into effect of tie spacing in UHPFRC, comparison with the result from specimen C4-60 allows for investigation into effect of tie configuration.

Figure 4.74 shows the load-displacement response for this column and the main events for this specimen are summarized below:

- First cracking occurred at a load of 8980 KN and displacement of 5.64 mm;
- At a displacement of 7.75 mm the column reached peak capacity of 9670 KN;
- In contrast to the previous two specimens, in the ascending branch a small plateau occurs just prior to peak (similar to that observed in the cylinder tests), thereafter the load drops suddenly by about 30%, with the response showing a stable response after this point;
- The column reached residual strength of 5253 KN (54 % of peak) at the end of testing (20 mm).

The progression of failure for this column is presented in Figure 4.78 . The controlled spalling and development of cracks are analogous to previous specimens. Figure 4.79 (a) shows a thin layer of cover that remains attached to the column at the end of testing. After manually removing the cover, and upon close investigation, buckling of the longitudinal reinforcement is observed in the corner regions (Figure 4.79 (b)). Furthermore, examination of the core concrete remains undamaged to the eye, and the 90 degree hook on the crossties deformed outward under the lateral pressure exerted by the core.

The data recorded by the LVDTs and strain gauges is presented in Figure 4.80. The column reaches a normalized peak total load of $1.35P_o$, and load remains at 70 % of P_o , at the end of testing, demonstrating the excellent post-peak behaviour of the column. Strain data shows that longitudinal reinforcement sustain large compressive strain after yield, although the recorded behaviour is not stable for most gauges. Strain data from the transverse

reinforcement shows reach yield point at or near the ultimate capacity of column, but fail slightly after this point.

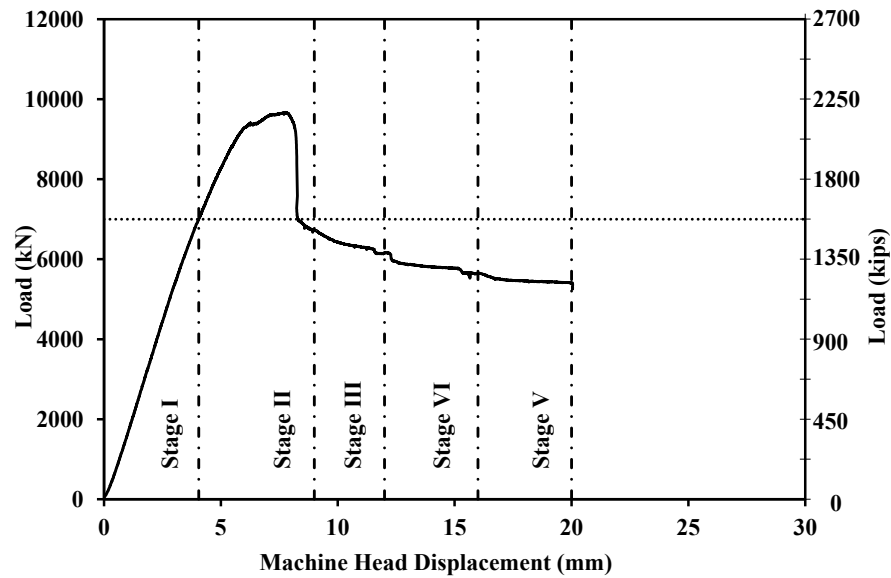


Figure 4.77 Staged Failure Process for Specimen # C3-60



(a) Stage II

(b) Stage III

(c) Stage IV

(d) Stage V

Figure 4.78 Specimen # C3-60 Failure Snapshots During the Test



(a)

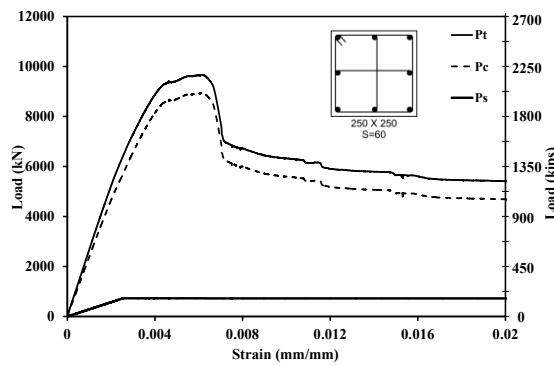


(b)

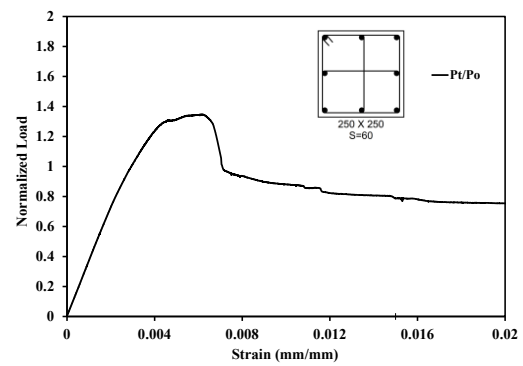


(c)

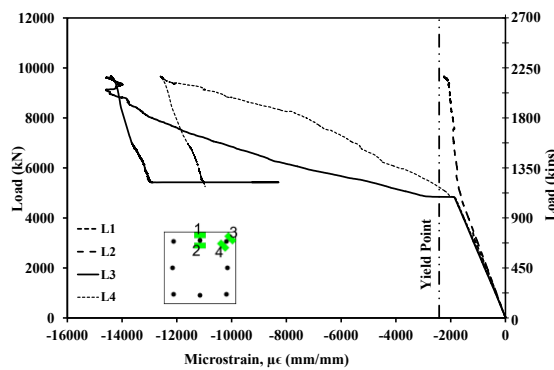
Figure 4.79 Close Look at Specimen # C3-60



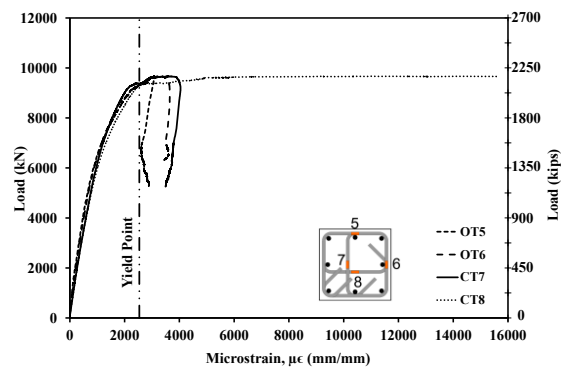
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



c) Longitudinal Reinforcement Behaviour



d) Transverse Reinforcement Behaviour

Figure 4.80 Results for Specimen # C3-60

4.4.4 Specimen # C3-40

Specimen # C3-40 was the last column in the C3-series (250 mm x 250 mm section, 8 bar configuration, 10 mm cover, 2.5% steel fibres) and had a tie spacing of 40 mm to meet the more stringent requirements for ductile columns ($R_d=4.0$) in the CSA A23.3 standard. The detailing also meets the requirements for special moment-frames (SMRF) in the ACI 318 code.

This column had the smallest tie spacing among all columns in the UHPFRC series. Figure 4.81 shows the load versus displacement response for the column. The main events as recorded during testing are summarized below:

- First cracking occurred at displacement of 5.95 mm and load of 9586 kN;
- The column reached an ultimate capacity of 10965 kN at a displacement of 9.42 mm, owing to the high confinement provided by the closely spaced ties in this specimen (it is noted that this is the highest capacity recorded in the test program and this load was very close to the maximum capacity of the MTS machine);
- As with the previous specimen, a small plateau is observed prior to peak capacity, this is followed by rapid drop in capacity to 9000 kN (82 % of peak), followed by a more gradual and controlled post-peak response;
- The column reached residual strengths of 6945 kN and 63% of peak capacity at the end of stages III and IV respectively, and reduced to 64 % of peak at the end of testing (20 mm);

Snapshots showing the progression of failure in the column are presented in Figure 4.82. One noticeable failure characteristics, observed in the previous tests, but more pronounced in this specimen is the great distribution of cracks throughout the column, even beyond the expected failure region. Although cracking was observed in the cover, spalling was effectively prevented. Inspection of the column after failure (see Figure 4.83) and manual removal of the cover reveals neither visible crushing in the core, nor any buckling in the longitudinal reinforcement. As with previous specimens the 90-degree hook on the intermediate crossties deforms outward.

The load-strain, normalized load-strain and strain gauge data are presented in Figure 4.84. This column reaches a normalized peak load which is 65 % higher than expected nominal strength of the code ($1.65 P_o$), and the residual strength remains above $1.05 P_o$ even at the end of the test. Examination of the strain data in Figure 4.84(c) confirms that yielding and high compressive strains occur in the longitudinal reinforcement. Likewise Figure 4.84 (d) that all ties, regardless of their location in the section, reach at strains slightly before the ultimate capacity is reached.

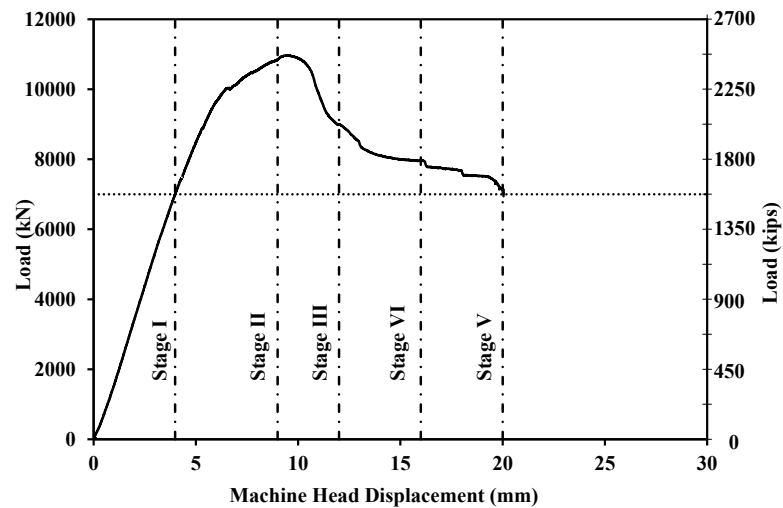


Figure 4.81 Staged Failure Process for Specimen # C3-40

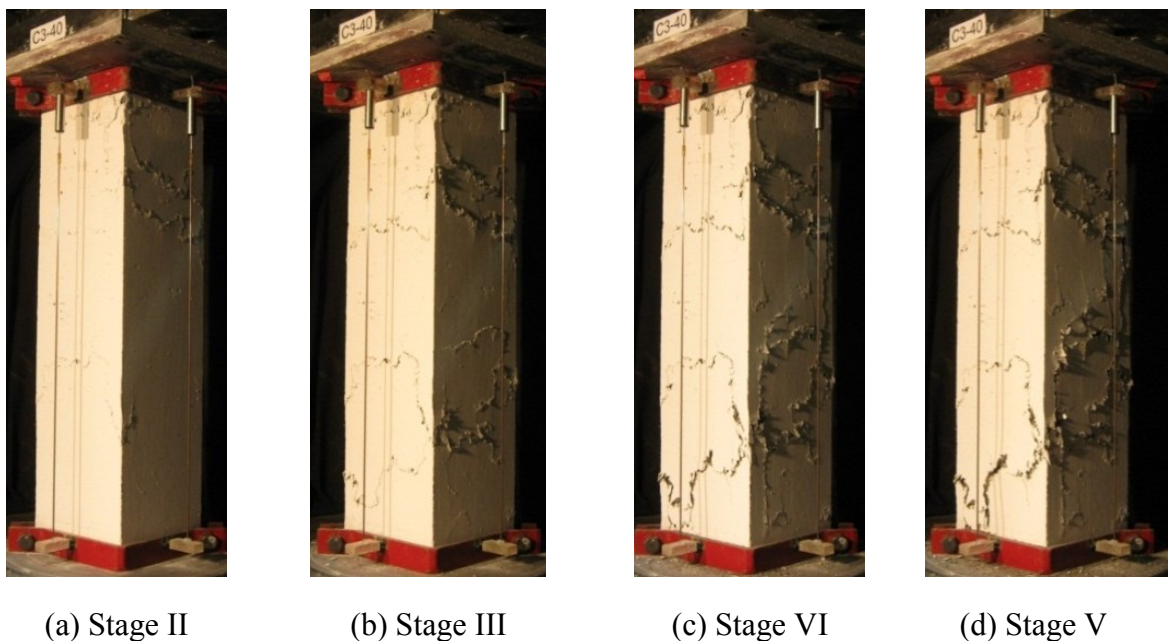


Figure 4.82 Specimen # C3-40 Failure Snapshots During the Test

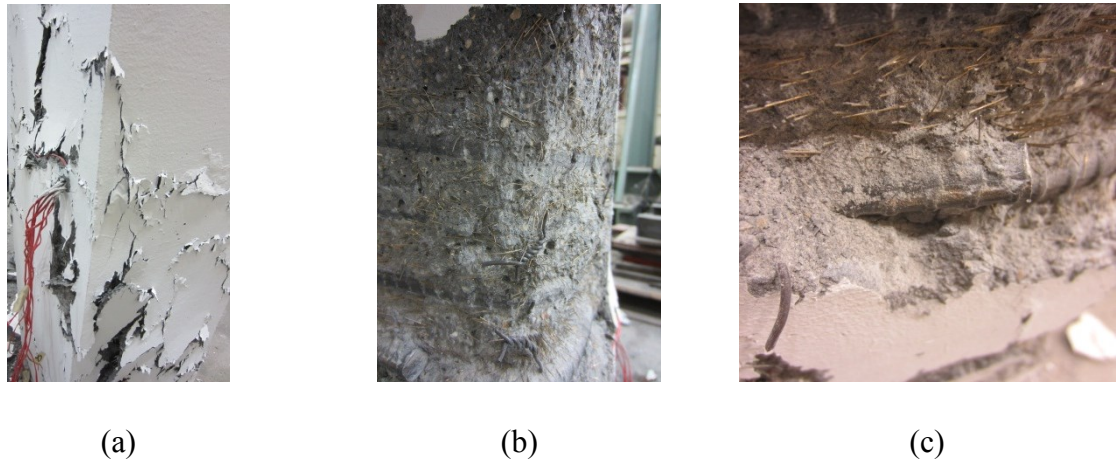
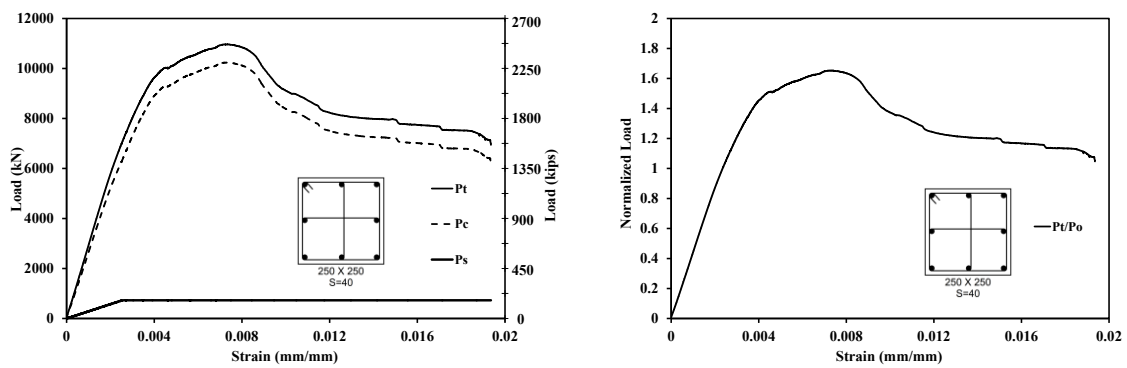
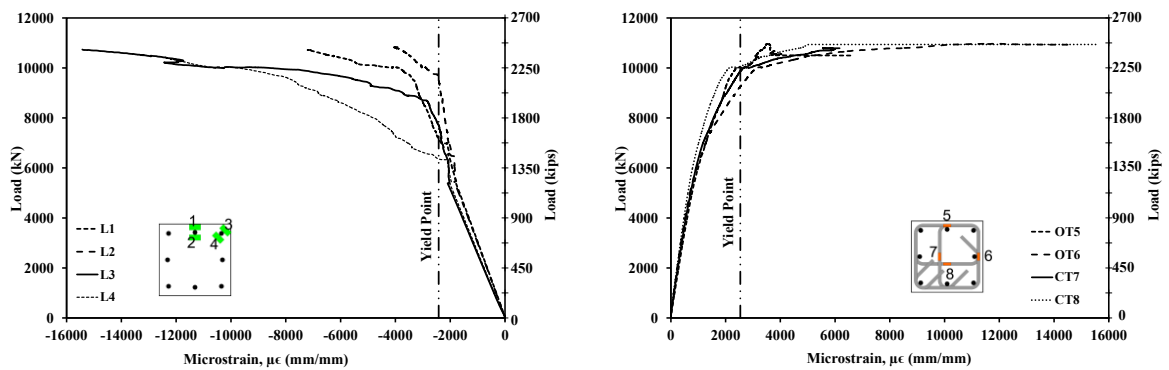


Figure 4.83 Close Look at Specimen #C3-40



a) Column Load-Strain Response

b) Column Normalized Load-Strain Response



c) Longitudinal Reinforcement Behaviour

d) Transverse Reinforcement Behaviour

Figure 4.84 Results for Specimen # C3-40

4.4.5 Specimen # C4-120

Specimen # C4-120 was the first of two columns in the C4 series. The columns in this series have 12-15M longitudinal reinforcing bars, while the transverse reinforcement consists of double-hoops, resulting in increased confinement. This particular column was detailed to meet CSA requirements for moderately ductile columns ($R_d=2.5$) and had tie spacing of 120 mm.

Figure 4.85 presents the load versus machine head displacement response for the column and the main events as recorded during testing are summarized below:

- The column reached ultimate capacity of 9802 kN at a displacement of 6.9 mm;
- After peak capacity, the load decreased rapidly to 6446 kN (66% of peak) with more gradual strength decay thereafter, with the residual strength reaching 60%, 52% and 40% of peak capacity at the end of stages III, IV and the end of testing, respectively.

Progression of failure and post-failure photos are shown in Figure 4.86 and Figure 4.87. Initiation of cracking in this specimen was on mid-level ties. Later these cracks join other cracks which had formed on the west side of the column (not visible in the photos) and diagonal cracking is also observed on the front column face. Consistent with the observations made in the companion C3-120 specimen, cover spalling is gradual and controlled. After the test, removal of the cover reveals buckling in the longitudinal reinforcement, while the core seems undamaged upon visual inspection. Buckling of the rebar may have contributed to the degree of cover spalling.

The data recorded by the LVDTs and strain gauges are plotted in Figure 4.88. The normalized load-strain plot shows that the column reaches normalized peak strength of 1.27 P_o and at the end of testing the capacity the residual strength falls to 0.55 P_o . Yielding and large compressive strains are observed in the longitudinal reinforcement. The transverse steel reaches yield slightly before peak column capacity and large tensile strains are recorded in all the gauges.

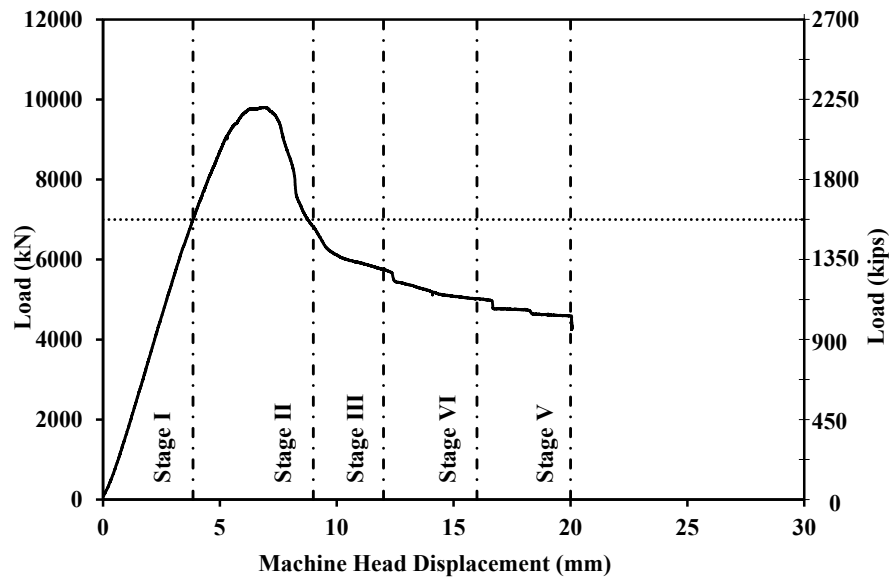


Figure 4.85 Staged Failure Process for Specimen # C4-120

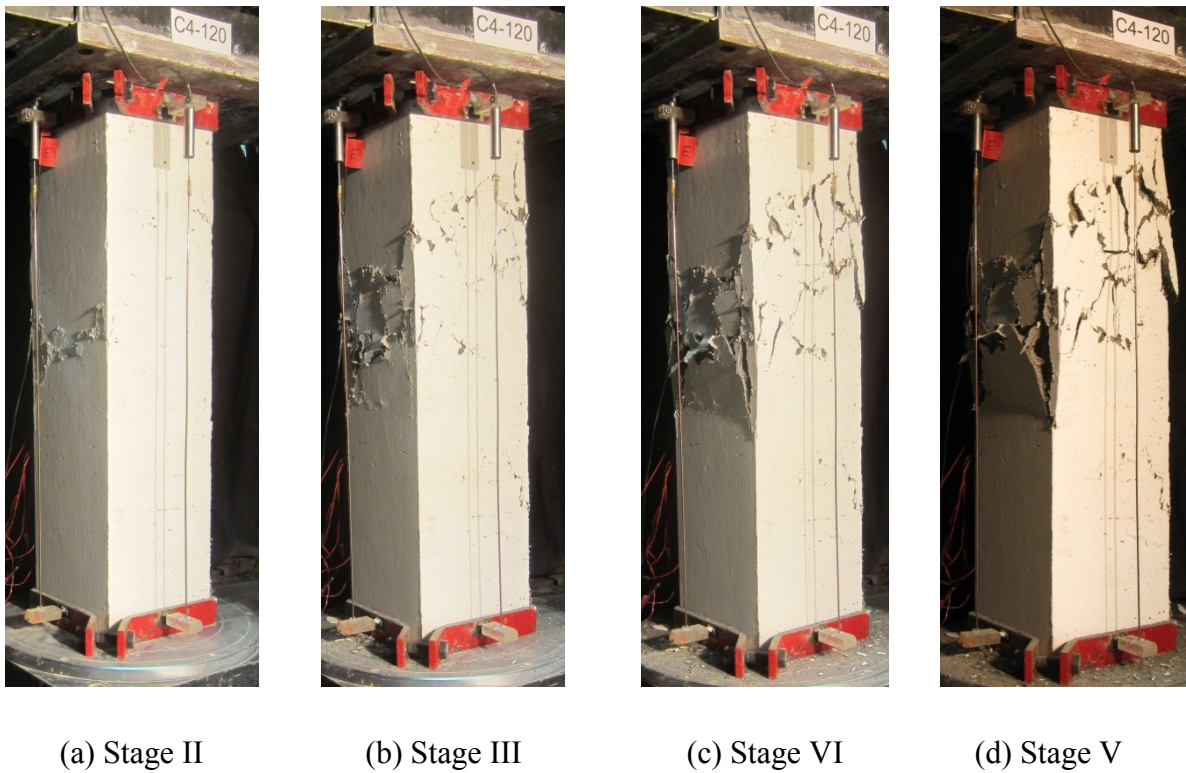


Figure 4.86 Specimen # C4-120 Failure Snapshots During the Test

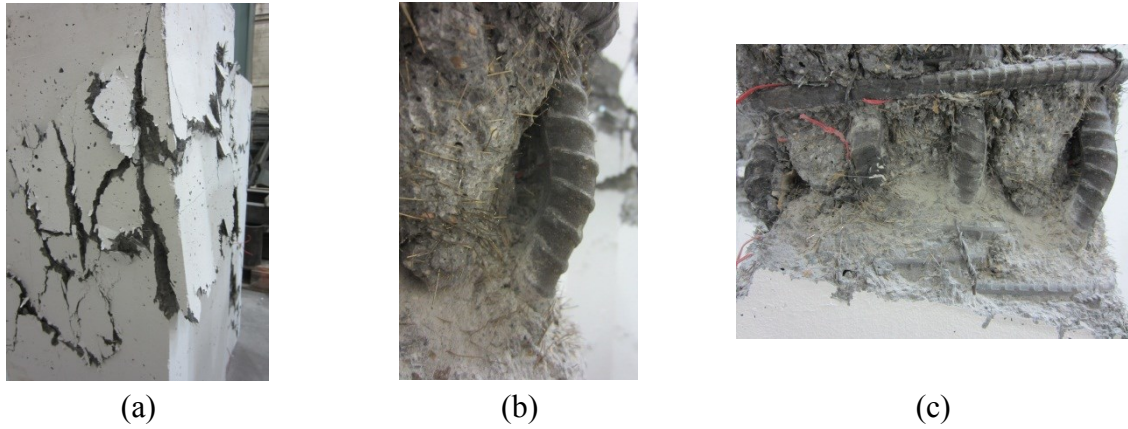


Figure 4.87 Close Look at Specimen #C4-120

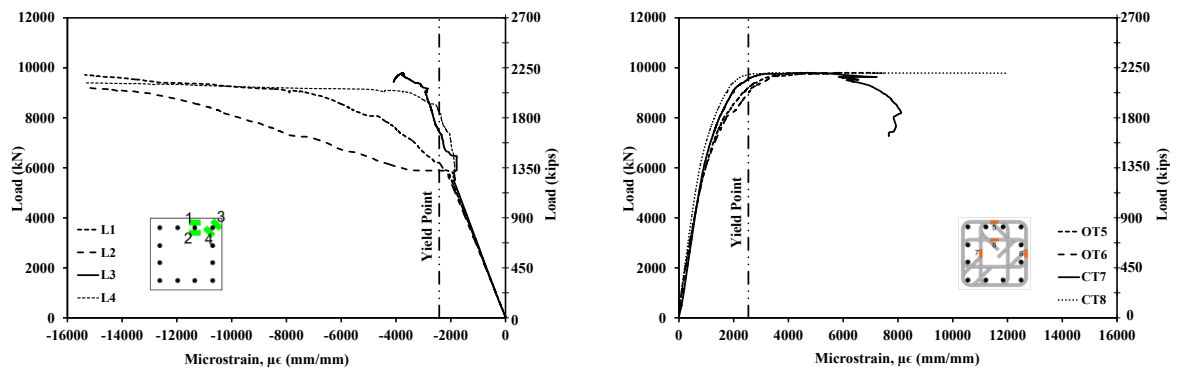
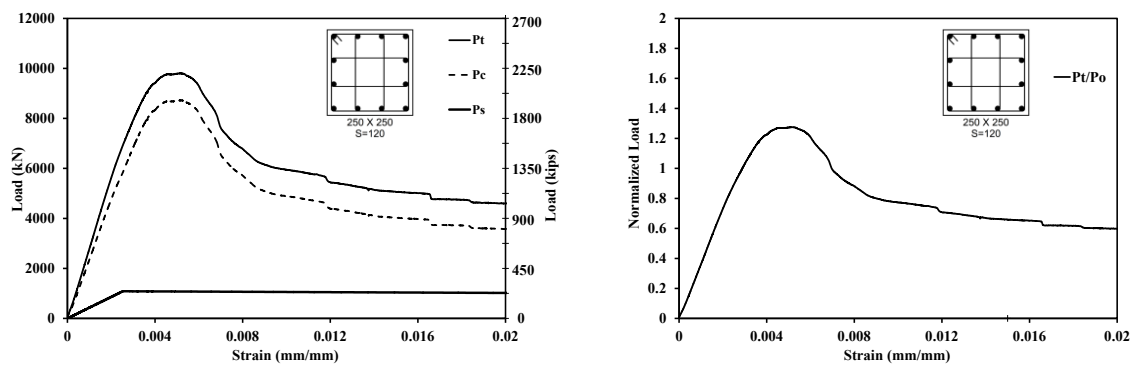


Figure 4.88 Results for Specimen # C4-120

4.4.6 Specimen # C4-60

Specimen C4-60 was the second column in the C4 series but had a tie spacing of 60mm to meet the requirements of ductile columns ($R_d=4.0$) in the CSA A23.3 standard and SMRF in the ACI 318 code.

Figure 4.74 shows the load-displacement response for this column and the main events for this test are listed below

- At a displacement of 8.52 mm the column reached peak capacity of 10246 KN;
- After peak the load gradually drops to 85% of peak capacity, thereafter showing a ductile and stable post-peak response;
- The column reached residual strength of 7798 KN (76 % of peak) at the end of testing.

The progression of failure through the various load stages is shown in Figure 4.90. Cracking initiates along the east and west side ties and as testing continues distribution of cracks and damage progresses throughout the column surface. Cover spalling is prevented, with the cover remaining attached to the column throughout the test. Figure 4.91 shows the post-failure status of the column after removal of the cover. Integrity of the core is maintained with little signs of damage, however upon close inspection bar buckling is visible.

Figure 4.92(a) and (b) report the load-strain and normalized load-strain response for this specimen. The peak normalized load reaches $1.26 P_o$, with the load remaining at P_o at the end of testing. Strain gauge data in Figure 4.92(c) shows development of yield and large compressive strains in the longitudinal bars, the divergence in the strains recorded by different gauges possibly arises due to effect of buckling in the bars. Figure 4.92(d) shows that yielding and large tensile strains are developed in the transverse reinforcement.

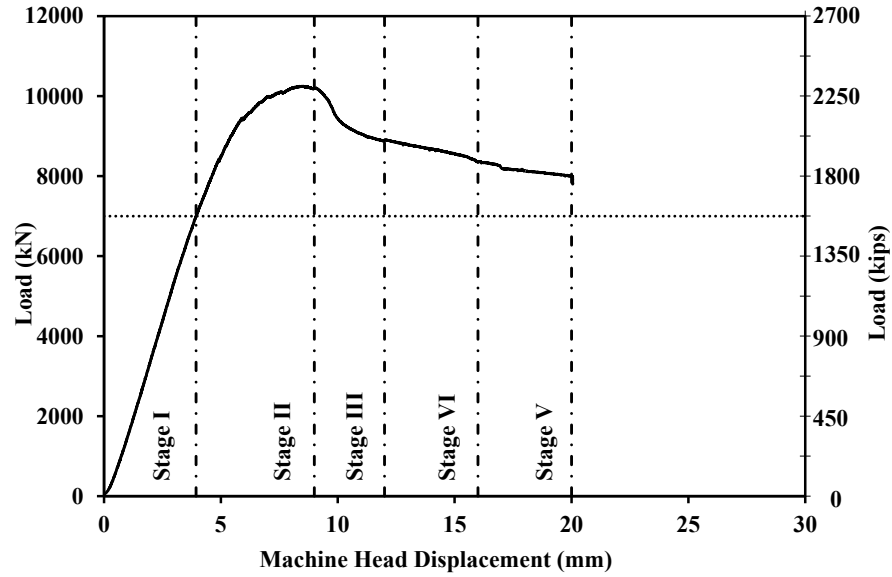


Figure 4.89 Staged Failure Process for Specimen # C4-60



(a) Stage II

(b) Stage III

(c) Stage VI

(d) Stage V

Figure 4.90 Specimen # C4-60 Failure Snapshots During the Test



(a)

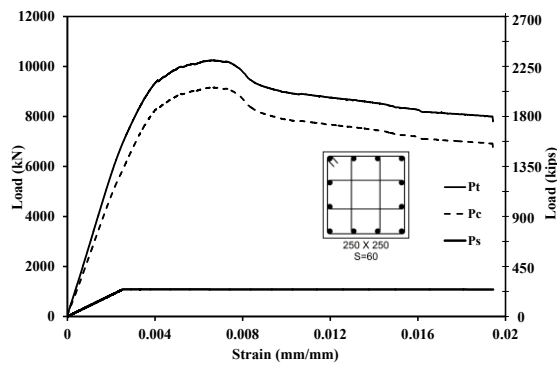


(b)

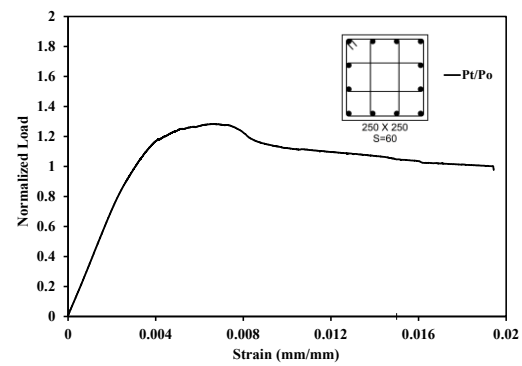


(c)

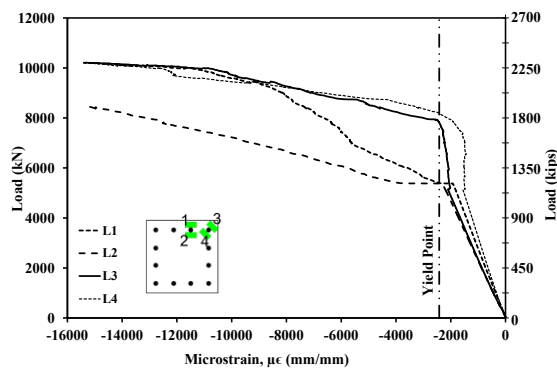
Figure 4.91 Close Look at Specimen # C4-60



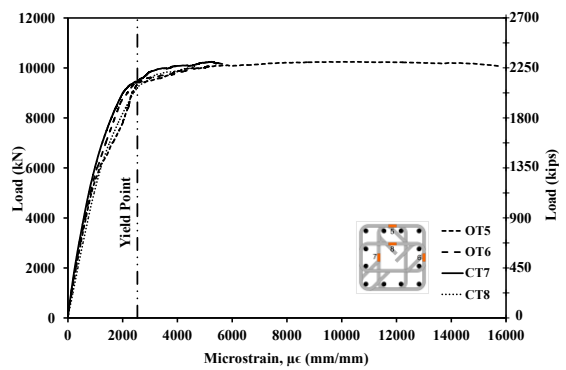
a) Column Load-Strain Response



b) Column Normalized Load-Strain Response



c) Longitudinal Reinforcement Behaviour



d) Transverse Reinforcement Behaviour

Figure 4.92 Results for Specimen # C4-60

85% drop (ϵ_{c85}) and 50% drop in capacity (ϵ_{c50}). The peak loads are compared to the nominal axial strength computed using the ACI 318-08 Standard (P_0) to obtain the ratio $P_{\max}/P_0$. For comparison, strains normalized with respect to peak strain of unconfined concrete (ϵ_{c0}) are also presented (where ϵ_{cu} is assumed to be 0.002).

In addition, in order to study the effect of the test variables on toughness and post-peak ductility, the area under the concrete load-strain curves up to strain of $\epsilon_{Au} = 0.02$ is also presented (A_u). For those specimens that failed prior to reaching this strain, the area is only computed up to a strain corresponding to the end of testing. A toughness index (T.I.), defined as ratio of the area under the curve, A_u and the area of an equivalent rigid material, $A_R = P_c \times \epsilon_{Au}$, is also reported (see Figure 5.2 and Equation 5-1):

$$\text{T.I.} = \frac{A_u}{P_c \times \epsilon_{Au}} \quad \text{Equation 5-1}$$

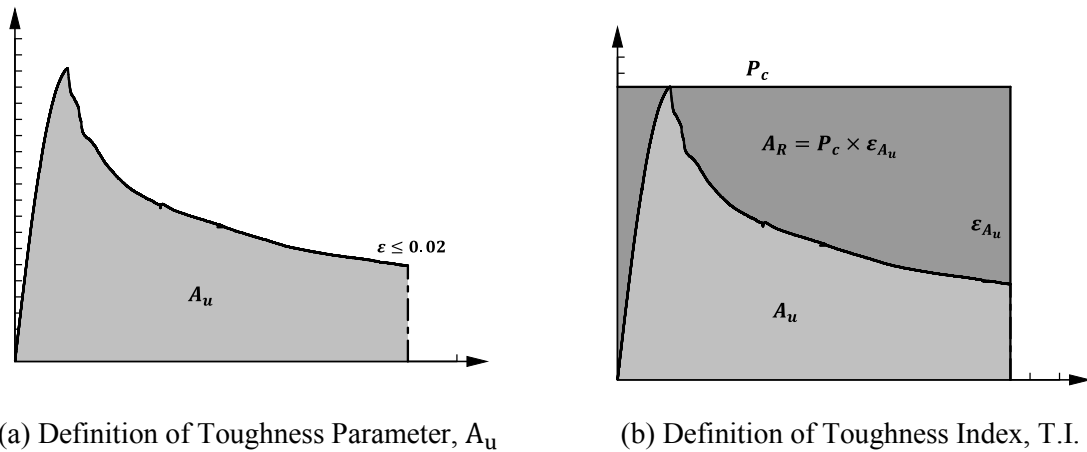


Figure 5.2 Definition of Toughness Parameters

For the specimens without fibres, examination of the results shows that as the degree of confinement increases so does the ratio $P_{\max}/P_0$, with values of 0.95, 1.01 and 1.04 for columns AR0, BR0 and CR0, respectively. Examination of the ϵ_{c50} strains demonstrates the effect of increasing confinement on post-peak ductility, with this strain value doubling in the case of column BR0 ($\epsilon_{c50} = 0.0061$) when compared to column AR0 ($\epsilon_{c50} = 0.0032$). Similarly, the use of ductile detailing in column CR0 results in a stable post-peak response.

By comparison of parameters A_u and T.I. it is found that toughness increases significantly when comparing column AR0 ($A_u = 10$ KN, T.I. = 0.37) and column CR0 ($A_u = 52$ KN, T.I. = 0.64). Similar trends are observed in the case of the columns constructed without cover, with post-peak ductility and toughness improving as core confinement increases.

In the case of specimens with fibres, examination of the data shows that addition of fibers results in increases in column capacity if one considers the ratio P_{max}/P_0 . For example, columns AR0, AR1 and AR1.5 have ratios of 0.95, 1.03 and 1.1, respectively, while ratios of 1.01, 1.05 and 1.03 are obtained for columns BR0, BR1 and BR1.5. For the CR-series specimens, ratios of 1.04, 1.1 and 1.24 are obtained for columns CR0, CR1 and CR1.5, respectively. The results show that in the case of rectangular columns the effect of fibres in increasing peak strength is modest for columns with low confinement (AR-series) and improves slightly as the degree of confinement provided by transverse reinforcement increases (CR-series). In general, examination of the test results also shows that the addition of fibres results in increases in peak strain, ϵ_{max} . In terms of the effect of increasing fiber content (1% vs. 1.5%) on capacity, negligible increases are observed in the case of columns with low confinement (AR-series) and a more noticeable effect is observed in the case of columns with larger amount of transverse reinforcement (BR and CR series). Despite the above findings and taking into account the limited data set, no reliable trend emerges regarding the effect of fibres on the ascending branch of the load-strain response for rectangular columns (this is in contrast to the observations made by Aoude et al. (2009) and others for square columns). A clearer trend emerges by examining the effect of fibres on the post-peak ductility. Examination of the ϵ_{c50} strains reveals that the addition of fibres results in important improvements. For example in the AR-series, increases of 271% and 300% are observed for columns AR1 ($\epsilon_{c50} = 0.0087$) and AR1.5 ($\epsilon_{c50} = 0.0096$), when compared to the control column AR0 ($\epsilon_{c50} = 0.0032$). In the case of the BR-series, increases of 190% and 297% are observed for columns BR1 ($\epsilon_{c50} = 0.0116$) and BR1.5 ($\epsilon_{c50} = 0.0181$) when compared to the control column BR0 ($\epsilon_{c50} = 0.0061$). Comparison of the A_u and T.I. results, demonstrates that while provision of fibres improves toughness, the relative improvement in toughness reduces as confinement increases. For example column AR0 has T.I. of 0.14 ($A_u = 10.26$), and addition of 1% fibres in column AR1 results in T.I. of 0.44

($A_u = 31$), a relative increase of 314%. For the highly confined CR-series columns, T.I. increases from 0.64 ($A_u = 52$) to 0.8 ($A_u = 61$) in columns CR0 and CR1, a more modest increase of 25%. This trend becomes more evident when comparing the results from the “nc”-series. For the nominally confined ARnc series, specimens AR0nc and AR1nc have T.I. values of 0.24 and 0.32, respectively. In contrast, the improvement in toughness becomes less important in the case of the highly confined CRnc columns, with specimens CR0nc and CR1nc having T.I. values of 0.82 and 0.87, respectively.

To further study post-peak response, Table 5.1 also reports two softening rates (see Figure 5.3). The parameter K_{sd} is the slope of a line between peak strain and strain equal to $\varepsilon = 0.004$ and gives an indication of the rate of cover spalling, bar buckling and early failure of unconfined core concrete, while the parameter K_{pd} is the slope of a line between strain values of $\varepsilon = 0.004$ and $\varepsilon = 0.01$, and gives an indication of post-peak strength decay. Comparison of the K_{sd} values shows that the sudden cover spalling observed in the SCC specimens is transformed into gradual and controlled spalling in the SFRC columns. Examination of the K_{pd} values shows that fibers have an important effect on reducing the post-peak softening rate in the case of columns with low confinement (AR-series), while a more modest effect is observed as the confinement in the columns increases (BR-series). The effect becomes negligible in the case of the highly confined columns in the CR-series.

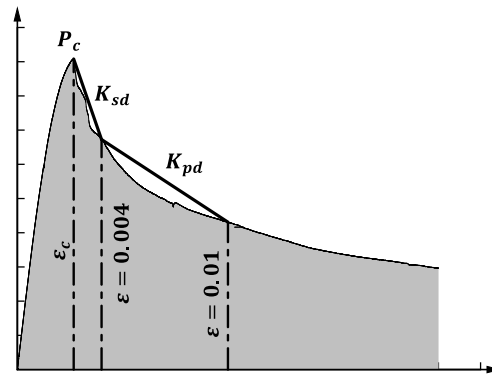


Figure 5.3 Definition of Softening Rates K_{sd} and K_{pd}

Table 5.1 Summary of Results for SCC and SCFRC Specimens

	P_t (KN)	ϵ_t	P_c (KN)	ϵ_c	ϵ_{c85}	ϵ_{c50}	P_t/P_o	$\frac{\epsilon_t}{\epsilon_{co}}$	$\frac{\epsilon_{c85}}{\epsilon_{co}}$	$\frac{\epsilon_{c50}}{\epsilon_{co}}$	K_{sd} (10^3)	K_{pd} (10^3)	T. I.	A_u (KN)
AR0	4389	0.0026	3664	0.0026	0.0028	0.0032	0.95	1.28	1.39	1.60	-2183	-222	0.37* (0.14)	10* (10.26)
AR1	4180	0.0033	3469	0.0033	0.0047	0.0087	1.03	1.66	2.36	4.33	-479	-332	0.44	31
AR15	3966	0.0029	3248	0.0030	0.0044	0.0096	1.10	1.47	2.20	4.81	-352	-254	0.50	33
BR0	4539	0.0027	3808	0.0027	0.0032	0.0061	1.01	1.34	1.59	3.06	-849	-199	0.42	32
BR1	4713	0.0036	3984	0.0036	0.0047	0.0116	1.05	1.82	2.37	5.82	-444	-244	0.56	45
BR15	4669	0.0036	3940	0.0038	0.0096	0.0181	1.03	1.81	4.81	9.04	-227	-105	0.73	58
CR0	4791	0.0027	4060	0.0027	0.0031	0.0249	1.04	1.37	1.53	12.46	-683	-74	0.64	52
CR1	4553	0.0033	3822	0.0033	0.0106	---	1.10	1.65	5.30	---	-312	-48	0.80	61
CR15	4967	0.0038	4236	0.0038	0.0164	---	1.24	1.89	8.21	---	-166	-59	0.85	72
AR0nc	3503	0.0038	2802	0.0038	0.0041	0.0048	1.03	1.90	2.05	2.38	-993	-417	0.24* (13.28)	13* (13.28)
AR1nc	3275	0.0038	2575	0.0038	0.0048	0.0061	1.13	1.89	2.42	3.03	-104	-373	0.32	17
AR15nc	3377	0.0035	2669	0.0035	0.0043	0.0062	1.11	1.75	2.16	3.08	-448	-330	0.34	18
BR0nc	3318	0.0027	2587	0.0027	0.0041	0.0092	0.99	1.37	2.04	4.58	-305	-169	0.62*	20*
BR1nc	2631	0.0022	1923	0.0018	0.0043	0.0154	1.16	1.12	2.16	7.70	-143	-76	0.68*	20*
BR15nc	2493	0.0021	1835	0.0021	0.0028	0.0085	1.09	1.03	1.38	4.23	-240	-74	0.50	18
CR0nc	3652	0.0034	2921	0.0034	0.0121	---	1.10	1.72	6.03	---	-98	-41	0.82	48
CR1nc	2970	0.0031	2239	0.0031	0.0155	---	1.12	1.56	7.77	---	-8	-24	0.87	38

* Testing was stopped at a strain $\epsilon < 0.02$; however data is extrapolated up to $\epsilon = 0.02$ and the corresponding toughness is reported in parentheses.

**Specimen residual post-peak strength remained above 50% of ultimate capacity.

5.3 Comparison of Load-Strain Curves – SFRC Series

In this section, the overall performance of the columns in the SFRC test series and the effect of transverse reinforcement and fibres are studied by comparing the normalized load-strain responses of the specimens. The normalized load is obtained by normalizing the axial load carried by the column P_t with respect to the nominal column capacity, P_o :

$$P_n = \frac{P_t}{P_o} = \frac{P_t}{0.85f'_c(A_g - A_s) + A_s f_y} \quad \text{Equation 5-2}$$

5.3.1 Effect of Transverse Reinforcement Spacing

To study the effect of transverse reinforcement spacing (confinement), the normalized load-strain responses of the columns in the AR, BR and CR series are plotted together for each fibre content (0%, 1% and 1.5%) as summarized in Table 5.2.

Table 5.2 Comparison of specimens for effect of transverse reinforcement ratio

0%	with cover	0% Fibres (AR0, BR0, CR0)	Figure 5.4
	without cover	0% Fibres (AR0nc, BR0nc, CR0nc)	Figure 5.5
1%	with cover	1% Fibres (AR1, BR1, CR1)	Figure 5.6
	without cover	1% Fibres (AR1nc, BR1nc, CR1nc)	Figure 5.7
1.5%	with cover	1.5% Fibres (AR15, BR15 and CR15)	Figure 5.8
	without cover	1.5% Fibres (AR15nc, BR15nc and CR15nc)	Figure 5.9

5.3.1.1 SCC Columns (0% fibres)

Specimens AR0, BR0 and CR0, did not contain fibres but were detailed with transverse reinforcement corresponding to the three levels of ductility (Rd of 1.5, 2.5 and 4) prescribed by the CSA A23.3 standard. Comparison of their behaviour is valuable to see the effect of confinement in SCC columns. A comparison of the normalized load versus strain responses for the three columns is shown in Figure 5.4. The main observation is in the post-peak segment where by increasing confinement, the failure tends to become more ductile. As expected, column AR0, which had a large hoop spacing ($s = 220$ mm) showed a poor

response with a sudden loss in load carrying capacity soon after the peak resistance was reached. In contrast, column CR0 exhibited an excellent response due to the provision of closely spaced and well-detailed transverse reinforcement ($s = 55$ mm). The drop of almost 70 % in AR0 is reduced in CR0, which shows an initial drop of 20% in capacity due to cover spalling, followed by a well-controlled and stable post-peak response. Due to the intermediate spacing of 110 mm, specimen BR0 showed an improved post-peak response when compared to specimen AR0. The results confirm that an increase in transverse reinforcement ratio in SCC columns results in improved in core confinement and column response.

The response of the three companion specimens constructed without cover is compared in Figure 5.5. The same trend is observed, with post-peak ductility improving as the transverse reinforcement ratio and confinement in the columns increases. After reaching peak, specimen AR0nc shows a sudden 60% drop in capacity due to bar buckling and failure of poorly confined core concrete. Due to the absence of concrete cover and excellent core confinement, specimen CR0nc shows an almost plastic response, with very little drop in strength after peak.

5.3.1.2 SCC Columns (1% and 1.5% fibres)

Specimens AR1, BR1 and CR1, had a constant fibre content of 1% and had transverse reinforcement spacing, s of 220 mm, 110 mm and 55 mm. The companion specimens AR1nc, BR1nc and CR1nc, had similar details but were constructed without cover. The response of the column in these two series is shown in Figure 5.6 and Figure 5.7. The results from the specimens with cover demonstrate that when fibre content is held constant at 1%, normalized peak strength and post-peak ductility improve as transverse reinforcement ratio increases. When examining the response of the specimens without cover a similar trend is observed, demonstrating that fibres and transverse reinforcement play a hybrid role in confining core concrete in SCC columns.

Similar observations are made when comparing the response curves for the 1.5% series columns (see Figure 5.8 and Figure 5.9 for the response of the specimens with and without cover, respectively).

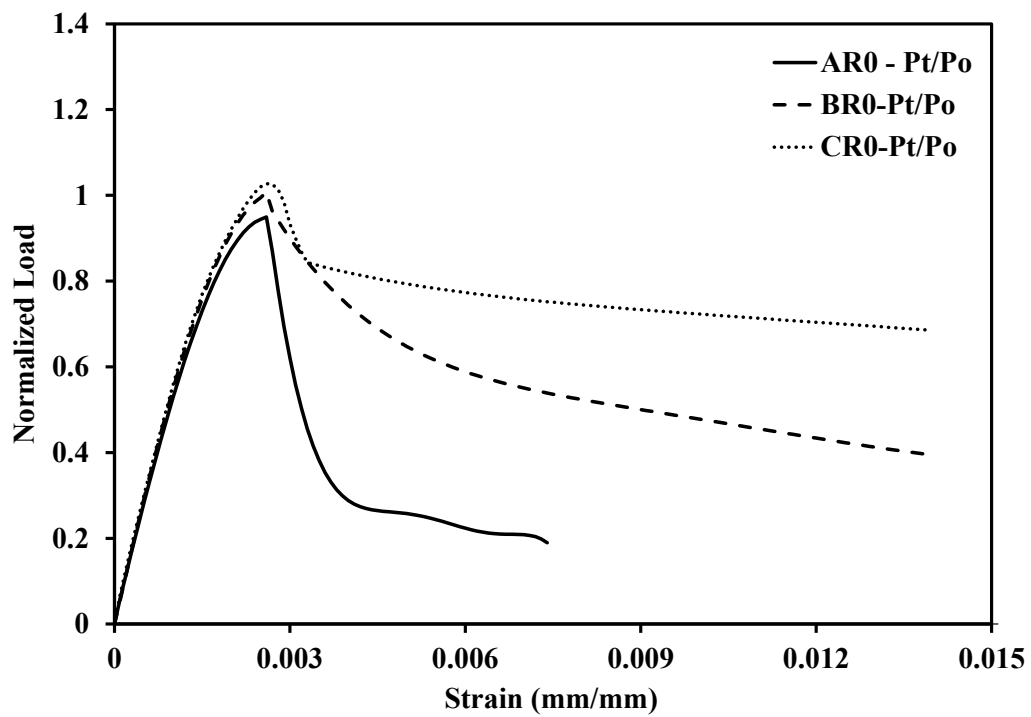


Figure 5.4 Comparison of 0% Series for Specimens with Cover

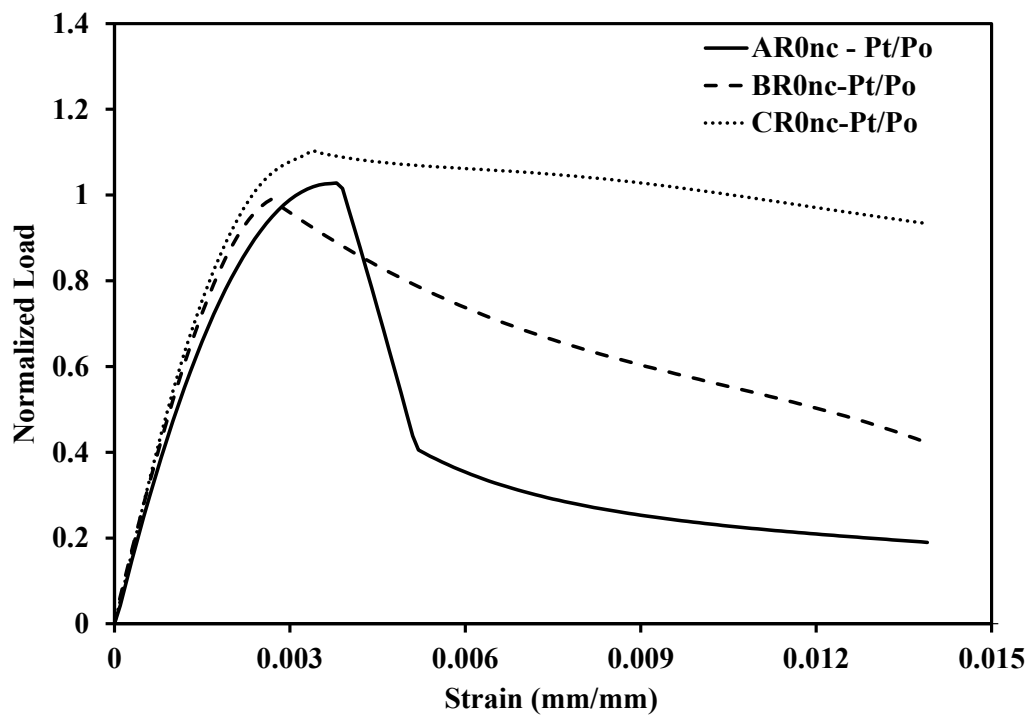


Figure 5.5 Comparison of 0% Series for Specimens without Cover

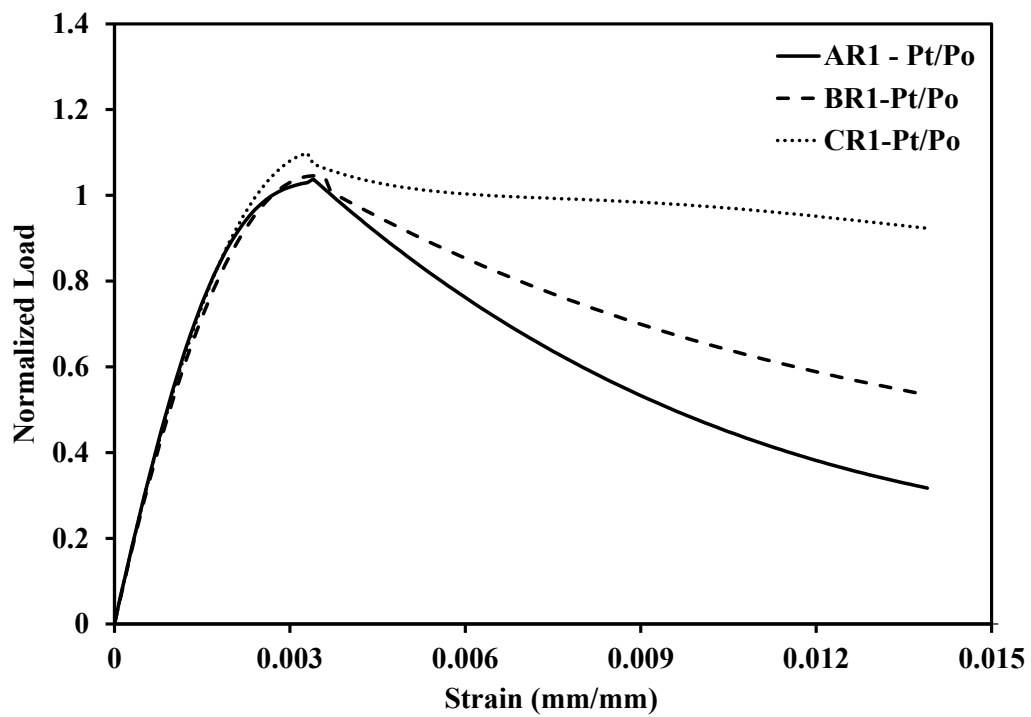


Figure 5.6 Comparison of 1% Series for Specimens with Cover

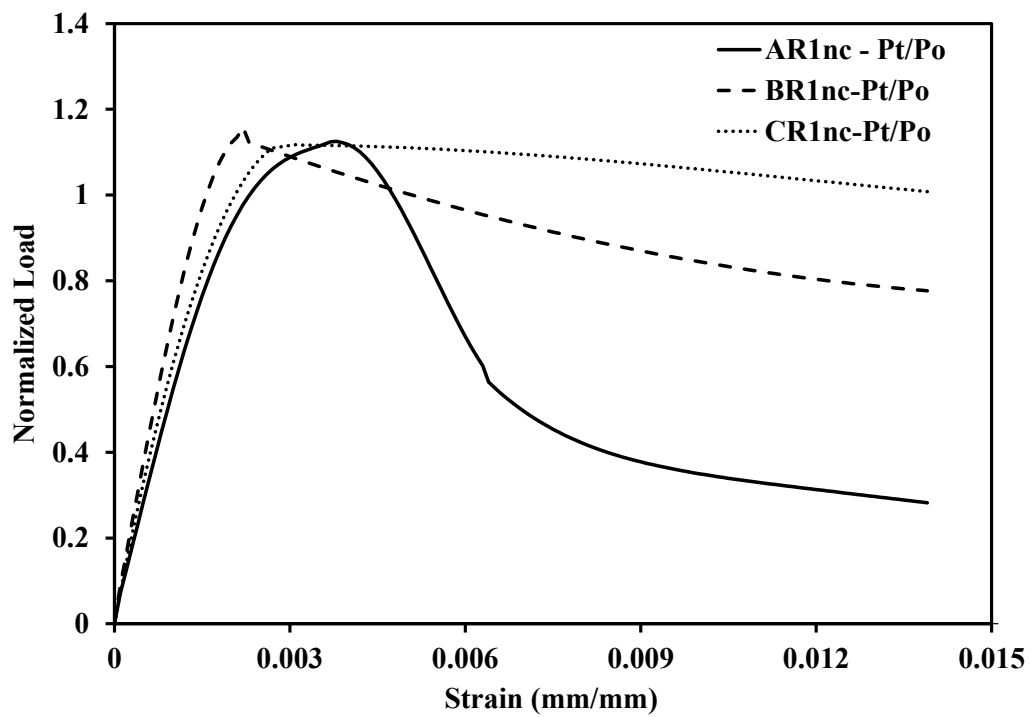


Figure 5.7 Comparison of 1% Series for Specimens without Cover

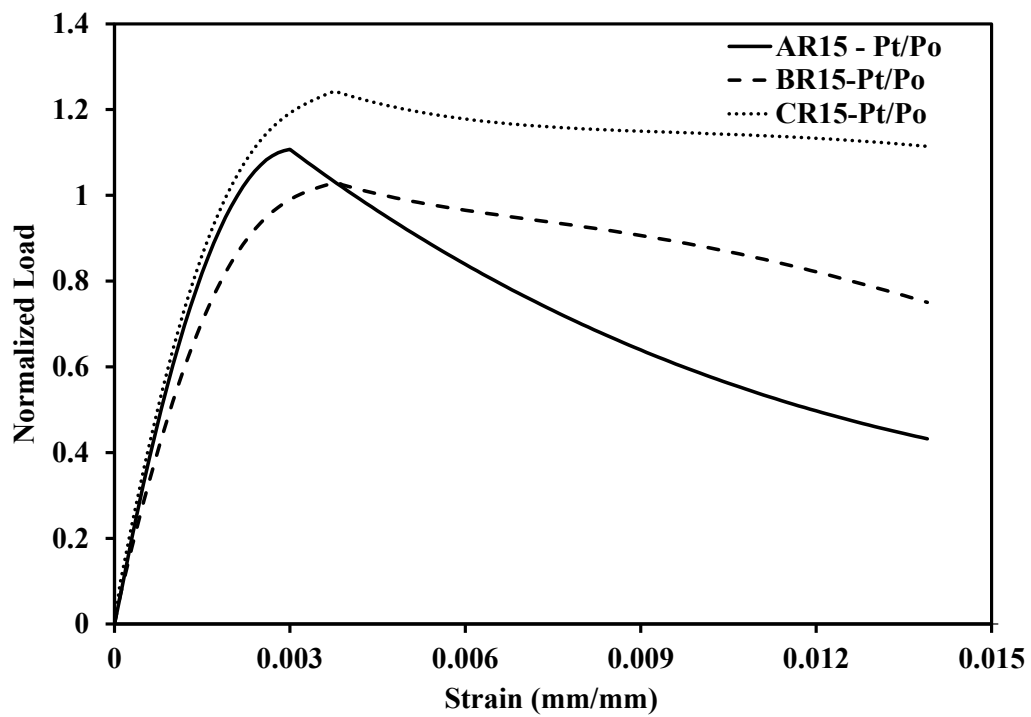


Figure 5.8 Comparison of 1.5% Series for Specimens with Cover

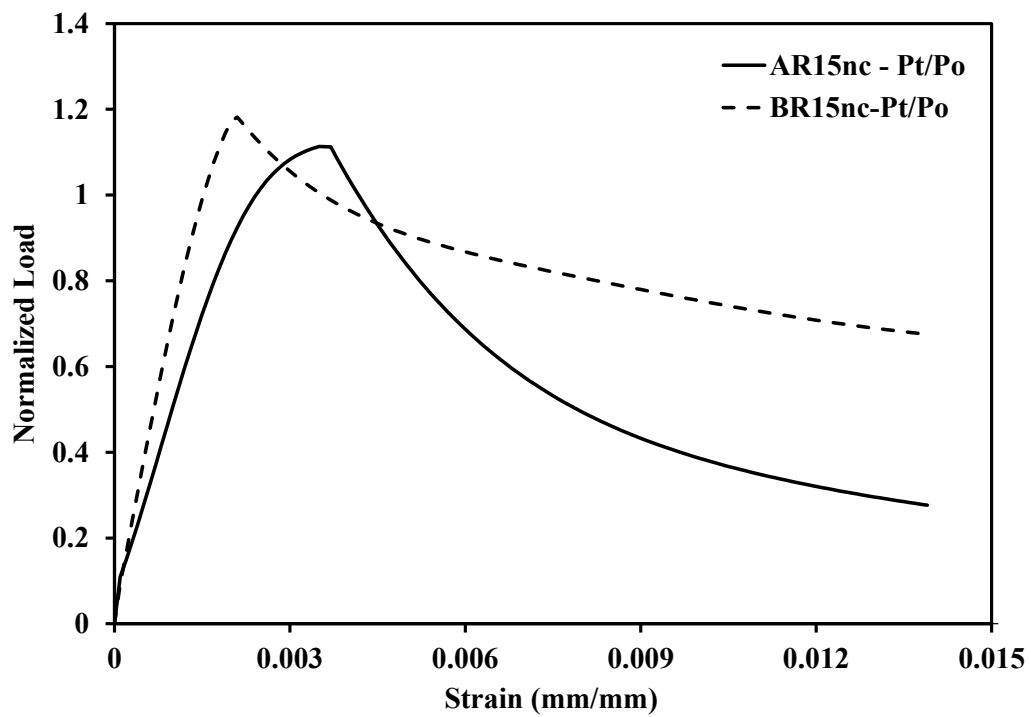


Figure 5.9 Comparison of 1.5% Series for Specimens without Cover

5.3.2 Effect of Fibres on Column Performance

In order to study the effect of increasing fibre content on column performance, the columns within each series having fibre contents of 0%, 1% and 1.5 % are compared.

5.3.2.1 Effect of Fibres in Columns with Low Confinement (AR-series)

The AR-series columns had a minimum amount of transverse reinforcement ($R_d = 1.5$; $s = 220$ mm), and therefore the comparison of the response of the columns in this series allows for a study of the effect of fibres in columns with low confinement. As shown in Figure 5.10, the addition of 1% fibres in column AR1 resulted in a significant improvement in the post-peak response when compared to the companion specimen without fibres (AR0), and further improvements are observed in specimen AR15 which had 1.5 % fibres. The improvements result from the ability of fibres to delay cover spalling and improvements in the response of the core concrete. In particular, the low confinement in this series results in a large area of "ineffectively confined" concrete within the core region and it is in this region that fibres can play an important role in improving the post-peak response. This will be further discussed in sections 5.3.4 and 5.3.5 .

5.3.2.2 Effect of Fibres in Columns with Moderate Confinement (BR-series)

The BR-series, had an intermediate amount of transverse reinforcement ($R_d = 2.5$; $s = 110$ mm). Increase of normalized peak load, as fibre content increases is not considerable. However, when compared to the control column without fibres (BR0), the addition of 1% and 1.5% fibres in specimens BR1 and BR1.5 resulted in improvement in overall toughness and ductility, with the rate of post-peak failure decreasing (see Figure 5.11). The improvements result from the enhanced cover response and improvement in core confinement.

5.3.2.3 Effect of Fibres in Columns with High Confinement (CR-series)

Specimens CR0, CR1, and CR15 were detailed in accordance with the more stringent confinement provisions of the CSA A23.3 standard ($R_d = 4.0$, $s = 55$ mm). The response of the columns in this series is shown in Figure 5.12. In terms, of post-peak behaviour, Specimen CR0 had an exceptionally well controlled response with the only drop in capacity occurring after cover spalling. This can be attributed to the excellent degree of confinement

in this column. The columns containing fibres displayed further improvements in post-peak behaviour which can be attributed to the influence of the fibres in delaying and minimizing the effects of cover spalling (notice the lack of sudden drop in capacity after peak), and to a much lesser extent to the improved response of the core region. It is noted that although overall toughness improves, post-peak softening rate remains essentially unchanged as fibres were added. The result demonstrates that as the amount of confinement provided by transverse reinforcement increases, the effect of fibres on post-peak ductility becomes less important. Further discussions in sections 5.3.4 and 5.3.5 will clarify this point.

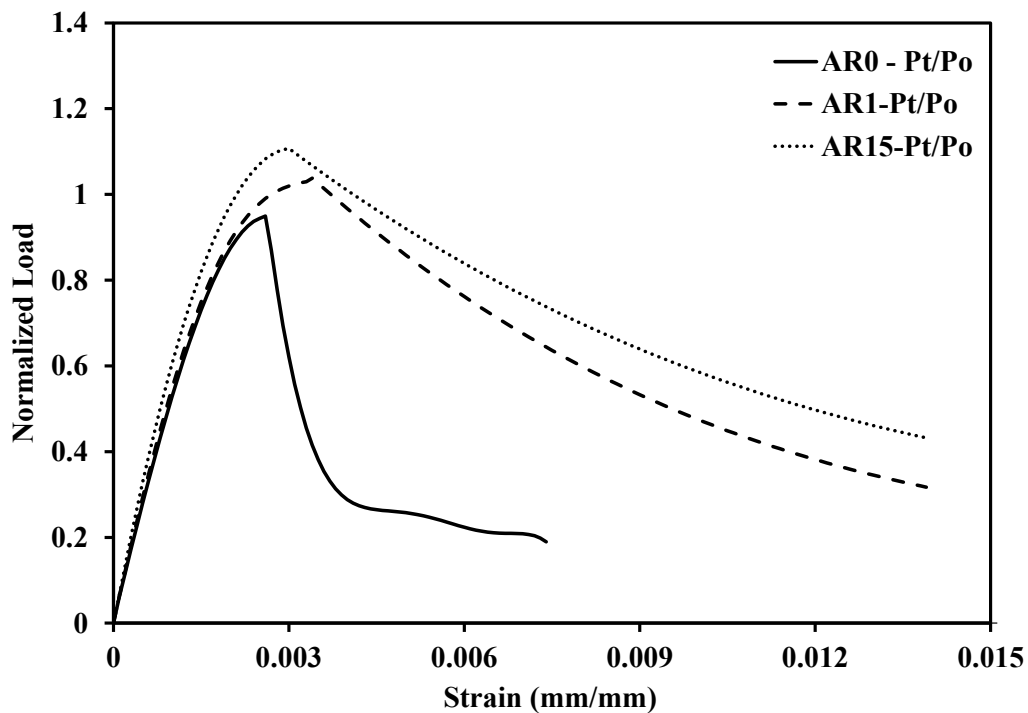


Figure 5.10 Comparison of Response for the AR-Series Specimens with Cover

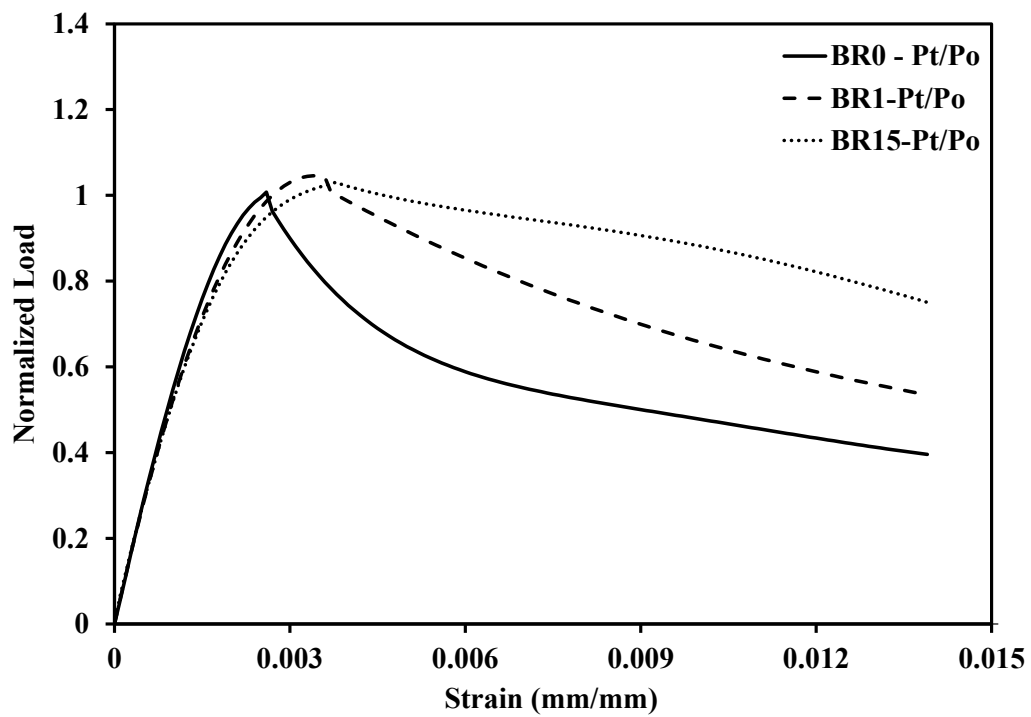


Figure 5.11 Comparison of Response for the BR-Series Specimens with Cover

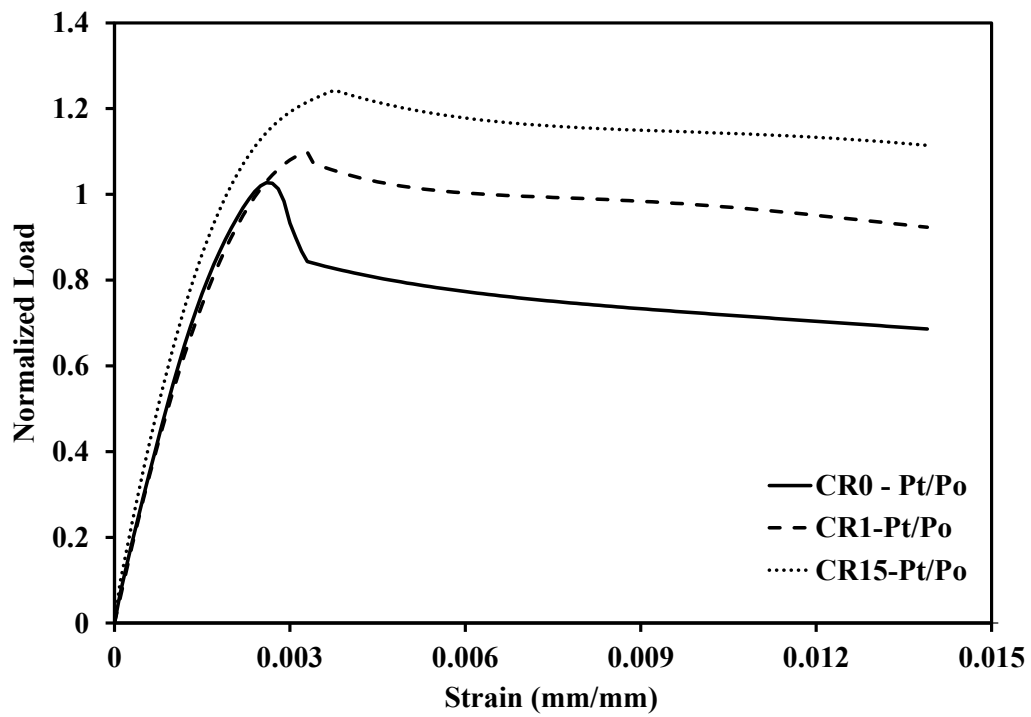


Figure 5.12 Comparison of Response for the CR-Series Specimens with Cover

5.3.3 Ability of Fibres to Partially Substitute For Transverse Reinforcement

Figure 5.13 compares the normalized load-strain responses of columns AR15, AR1 and BR0. Column BR0 had a two-fold increase in the amount of transverse reinforcement when compared to the AR-series columns. Examination of the response of column AR15 shows that the addition of 1.5% steel fibres in a column with minimum confinement reinforcement results in performance that surpasses that of Specimen BR0. Specimen AR1, which contained 1% fibres showed a response that equalled that of Specimen BR0.

A comparison of the normalized load-strain responses of Specimens BR15, BR1 and CR0 is shown in Figure 5.14. Specimen BR15, which had a tie spacing of 110 mm and 1.5% fibres, was able to maintain a higher normalized load than that of Specimen CR0 ($R_d = 4.0$) up-to a strain of 0.015 (after which its capacity dropped below that of Specimen CR0 which continued to maintain its strength even at very high strains).

This comparison shows that fibres can partially substitute for confinement reinforcement in "moderately ductile" columns, and to a lesser extent in "ductile" columns.

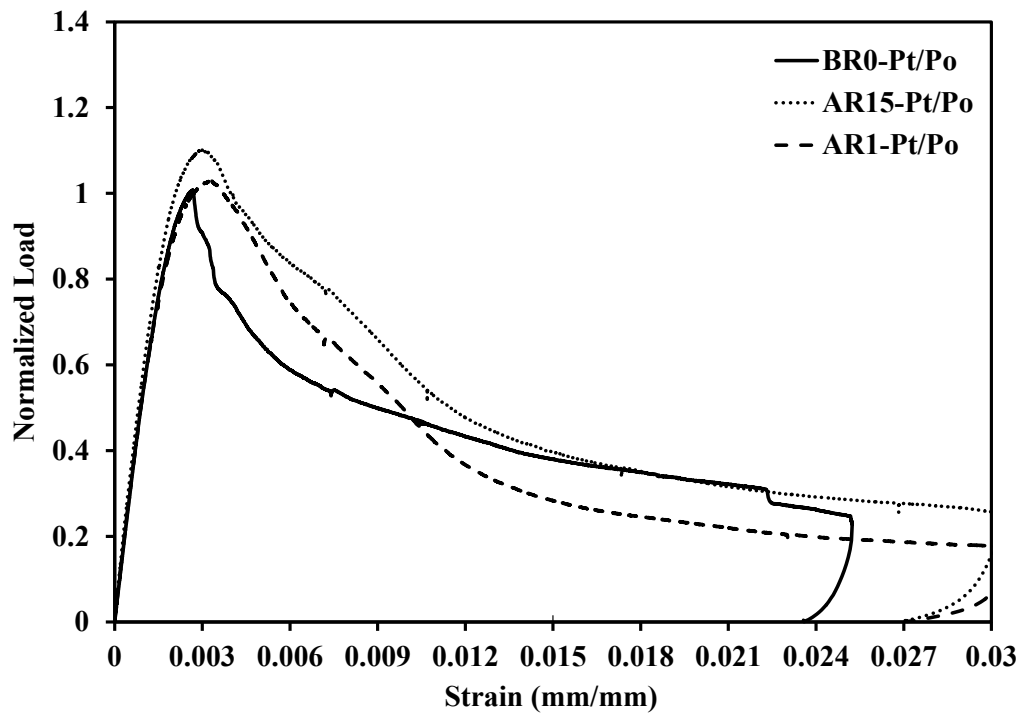


Figure 5.13 Ability of Fibres to Substitute Transverse Reinforcement in Moderately Ductile Columns

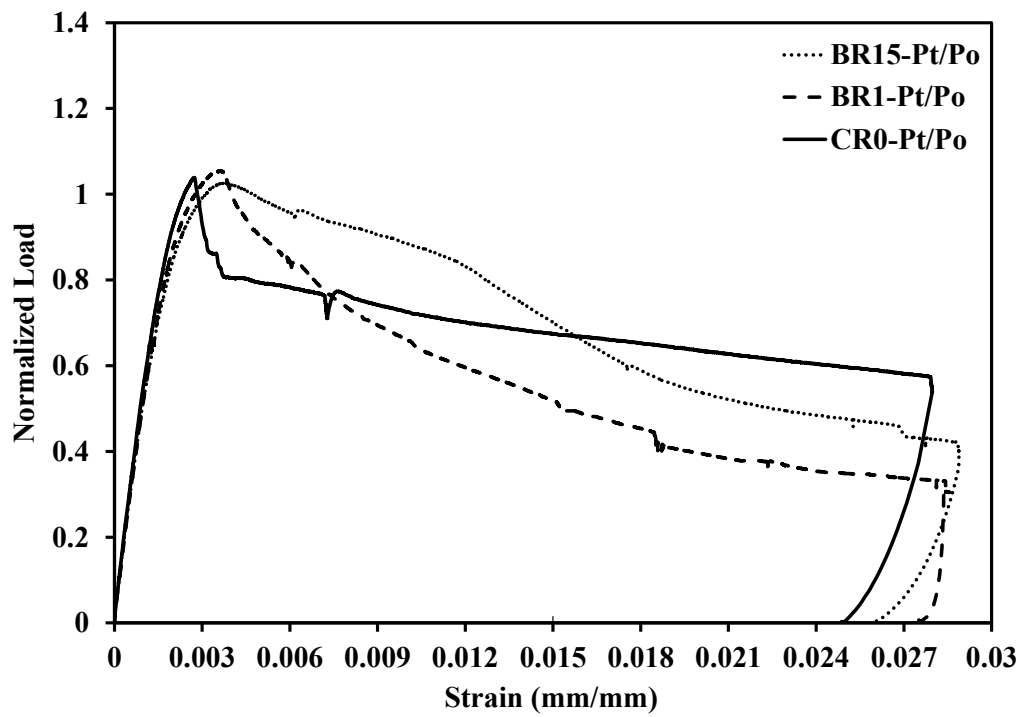


Figure 5.14 Ability of Fibres to Substitute Transverse Reinforcement in Ductile Columns

5.3.4 Effect of Fibres on Core Behaviour

In this section, the load-strain response of the nc-series columns is compared in order to study the effect of fibres on core behaviour. The normalized concrete load-strain response is also compared, where the load carried by concrete (P_c) is normalized with respect to the nominal concrete core strength (P_{occ}) as defined in Equation 5-3 (where A_{ch} represents the net concrete cross-section area in the columns). The contribution of the fibres to core response is also studied by isolating the “core improvement” using Equation 5-4, where the response of the plain SCC columns is subtracted from the response of the companion SFRC columns:

$$\frac{P_c}{P_{occ}} = \frac{P_c}{0.85f'_c A_{ch}} \quad \text{Equation 5-3}$$

$$\text{Core Improvement} = \left[\left(\frac{P_c}{P_{occ}} \right)_{\text{with fibers}} - \left(\frac{P_c}{P_{occ}} \right)_{SCC} \right]_{nc \text{ series}} \quad \text{Equation 5-4}$$

5.3.4.1 Effect of Fibres on Core Confinement in ARnc Series Columns

Specimens AR0nc, AR1nc and AR15nc had nominal transverse reinforcement ($R_d = 1.5$, $s = 220$ mm) and were constructed without cover. The effect of fibres on core behaviour is examined by comparing the response of these specimens in Figure 5.15. Column AR0nc shows a sudden drop in capacity after peak due to buckling of reinforcement and crushing of poorly confined concrete within the core. Specimens AR1nc and AR15nc show improved post-peak response, and since buckling occurred in these specimens, the improvement can be linked to the contribution of the fibres to improving the response of the concrete within the core. The large spacing of transverse reinforcement results in a large area of ineffectively confined concrete within the core. The fibres transform the response of this (“unconfined”) concrete by arresting and delaying the development of macroscopic cracks in this zone. Figure 5.16 shows the normalized concrete load-strain responses of specimens AR1nc and AR0nc as well as the isolated “core improvement” (obtained by subtracting the two responses as defined in Equation 5-4). It is observed that fibres play a more important role

after peak, where a rapid increase in core improvement is observed. After the core improvement peaks, a gradual decrease is observed. It is interesting to note that the shape of the core improvement curve matches the general shape of the pull-out response of hooked-end steel fibres. Prior to peak, micro-cracking occurs within the core, the fibres control these cracks and this result in a moderate increase in peak strength. After peak, macroscopic cracking begins within the large area of ineffectively confined concrete outside the inner core. The fibres within this “unconfined” region arrest and control these cracks, contributing to the observed strength increase in the core until they eventually pull-out.

5.3.4.2 Effect of Fibres on Core Confinement in CRnc Series Columns

Specimens CR0nc and CR1nc were detailed with a large amount of transverse reinforcement ($R_d = 4.0$, $s=55\text{mm}$), and were constructed without cover. The effect of fibres on core behaviour is studied by comparing normalized load-strain response of the specimens in Figure 5.17. In specimen CR0nc, high confinement (and lack of cover) results in excellent post-peak behaviour. While specimen CR1nc shows an improvement in normalized peak strength, the post-peak behaviour remains essentially unchanged. Figure 5.18 plots the normalized concrete load-strain responses of the specimens along with the isolated “core improvement” (as defined in Equation 5-4). The core improvement is not significant due to the negligible contribution of the fibres in improving the response of the well-confined core concrete. The closely spaced transverse reinforcement results in high confinement, and thus the amount of ineffectively confined (i.e. unconfined) concrete outside the inner core is quite low. As a result, while fibres contribute during the micro-cracking stage, as macro-cracking begins in the small region of unconfined concrete in the outer core, minimal pull-out occurs and therefore the contribution of the fibres to core improvement is not significant.

5.3.4.3 Effect of Fibres on Core Confinement in BRnc Series Columns

The BRnc series columns were detailed with an intermediate amount of transverse reinforcement ($R_d = 2.5$, $s = 110 \text{ mm}$) when compared to the ARnc and CRnc series columns. As a note, specimen BR1nc had low concrete strength when compared to specimen BR0nc; nonetheless the normalized load-strain response of the columns is presented in Figure 5.19. When considering normalized response, BR1nc shows an improved post-peak response when compared to BR0nc. Since bar buckling was observed in both columns, the

improvement in response can be linked to the effect of fibres in improving the response of the core concrete. Figure 5.20 shows the normalized concrete load-strain responses for the columns, along with the isolated “core improvement”. The improvement in core response (after peak) is moderate. This moderate confinement in this column results in a “moderate” region of ineffectively confined (i.e. unconfined) concrete outside the inner core. Prior to peak the fibres within the core contribute to the increase in “normalized” peak strength, and after peak, fibre pull-out is limited to the unconfined region outside the inner confined core, which results in the moderate improvement in normalized response.

5.3.4.4 Summary

Figure 5.21 plots the core improvement for the ARnc, BRnc and CRnc series columns. The trend shows that the relative improvement in core response is related to the level of confinement in the columns. While important enhancements in core response are expected in columns with nominal confinement, the improvement in core response may become negligible in highly confined columns.

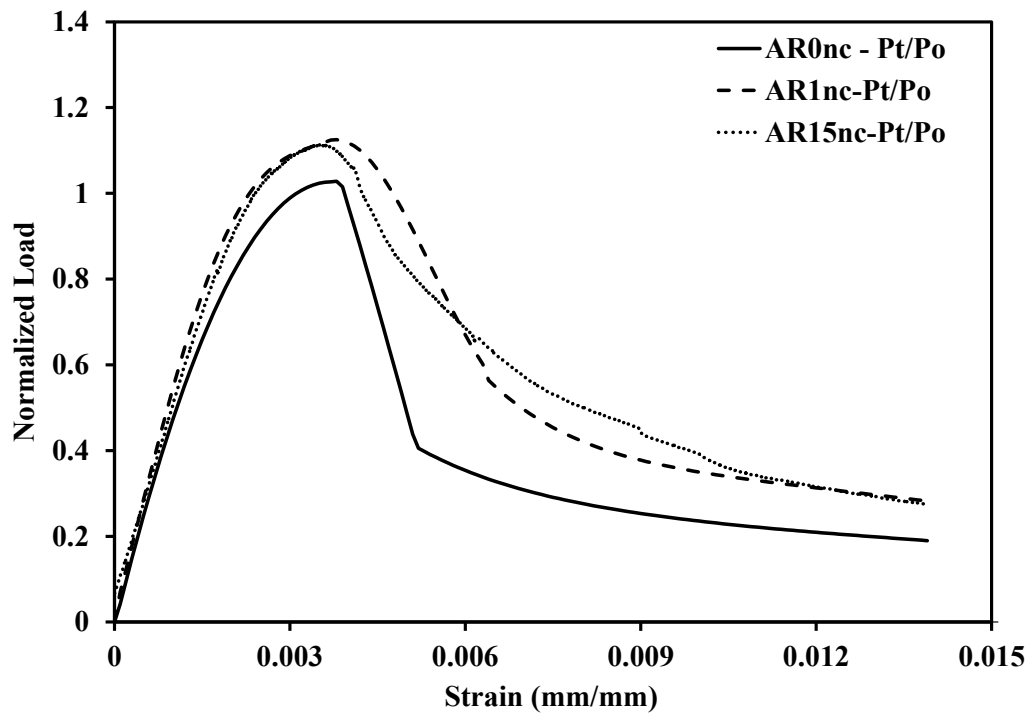


Figure 5.15 Comparison of Response for the AR-Series Specimens without Cover

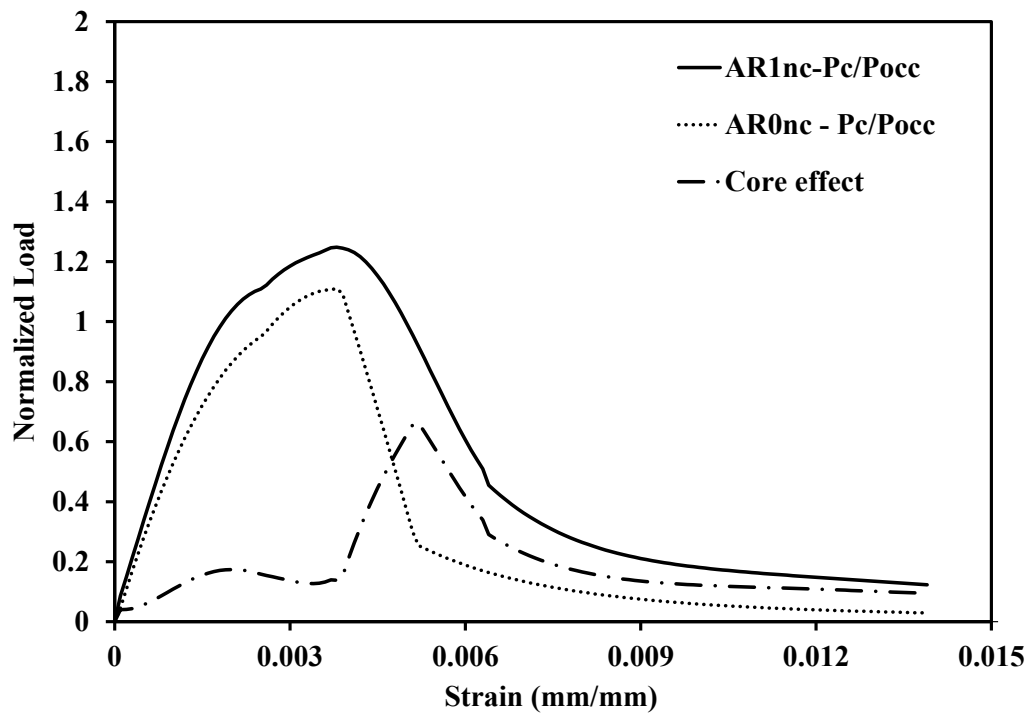


Figure 5.16 Effect of Fibres on Core Improvement in Columns with Nominal Confinement

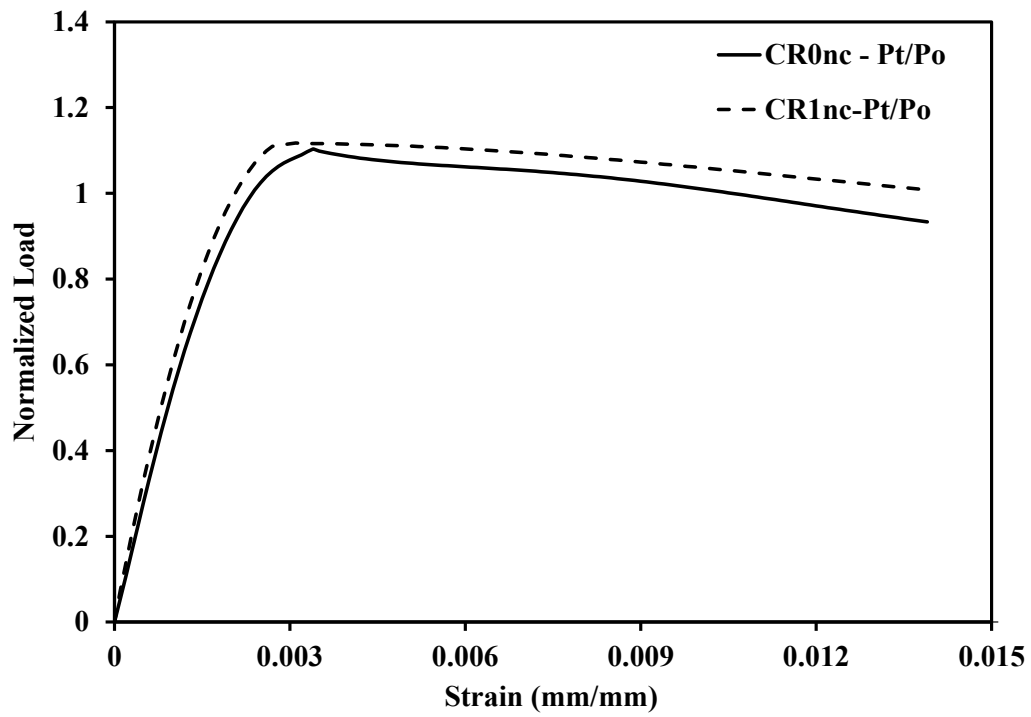


Figure 5.17 Comparison of Response for the CR-Series Specimens without Cover

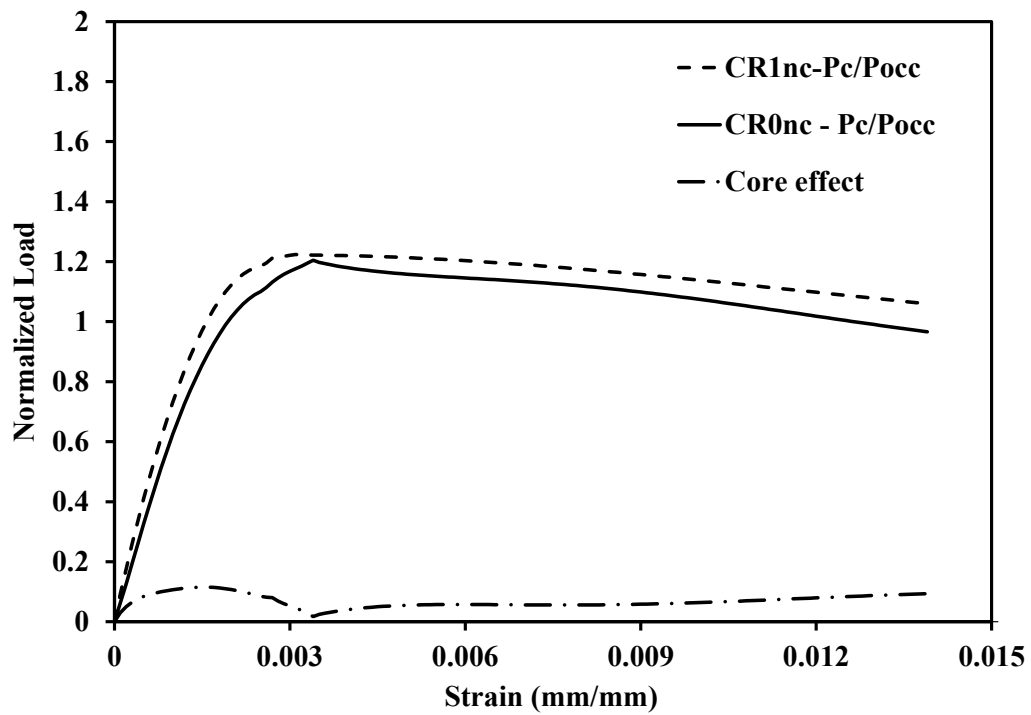


Figure 5.18 Effect of Fibres on Core Improvement in Highly-Confined Columns

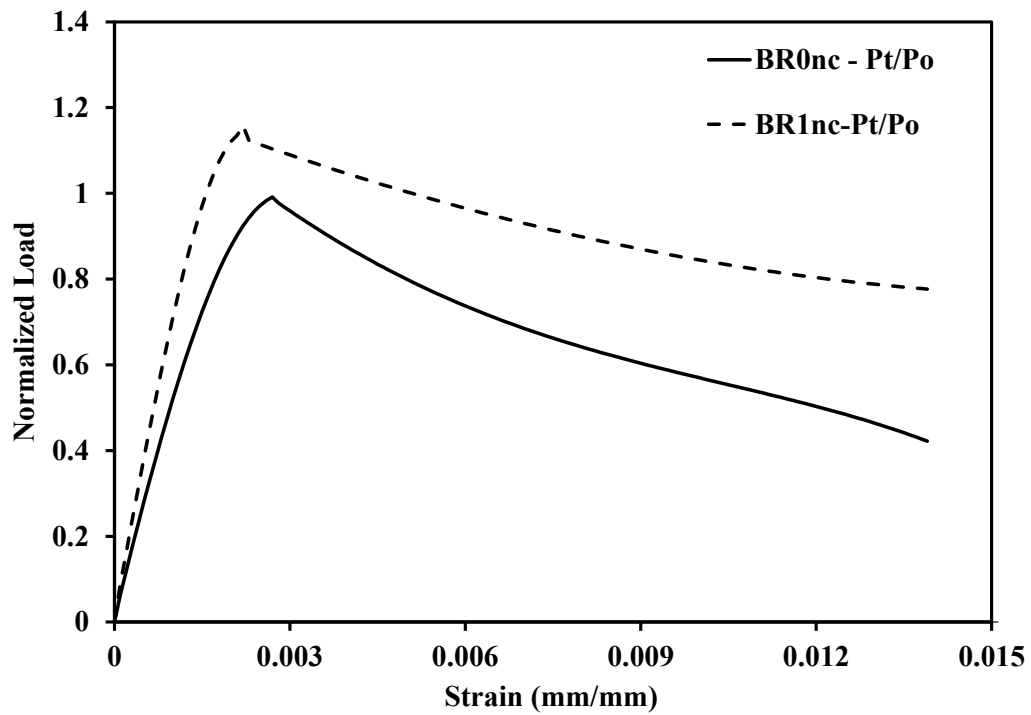


Figure 5.19 Comparison of Response for the BR-Series Specimens without Cover

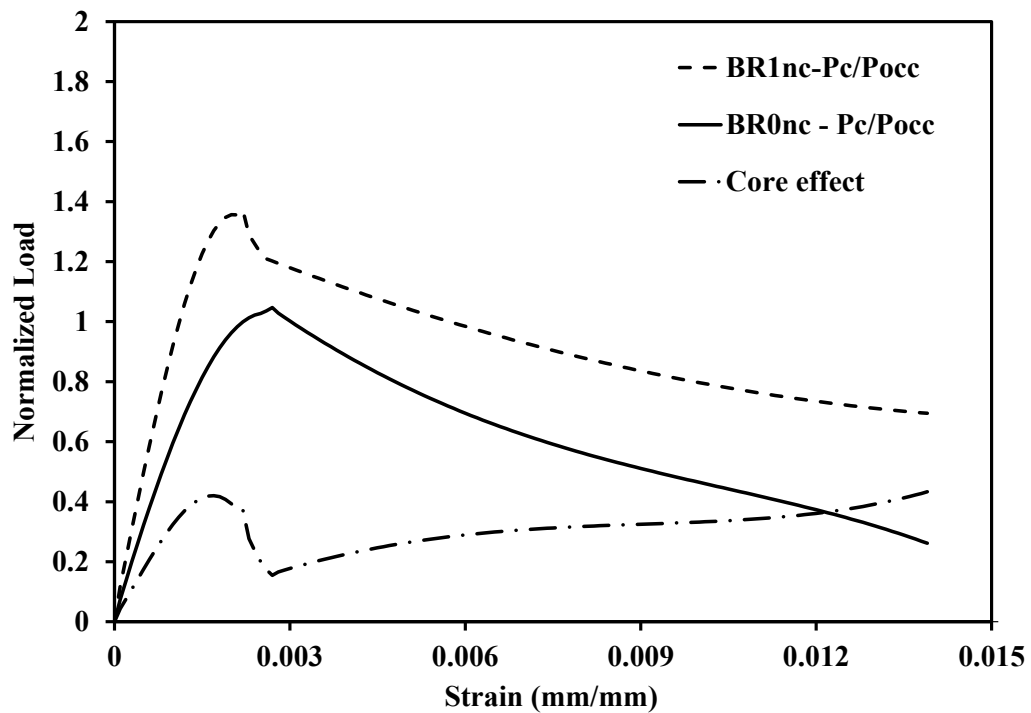


Figure 5.20 Effect of Fibres on Core Improvement in Moderately-Confined Columns

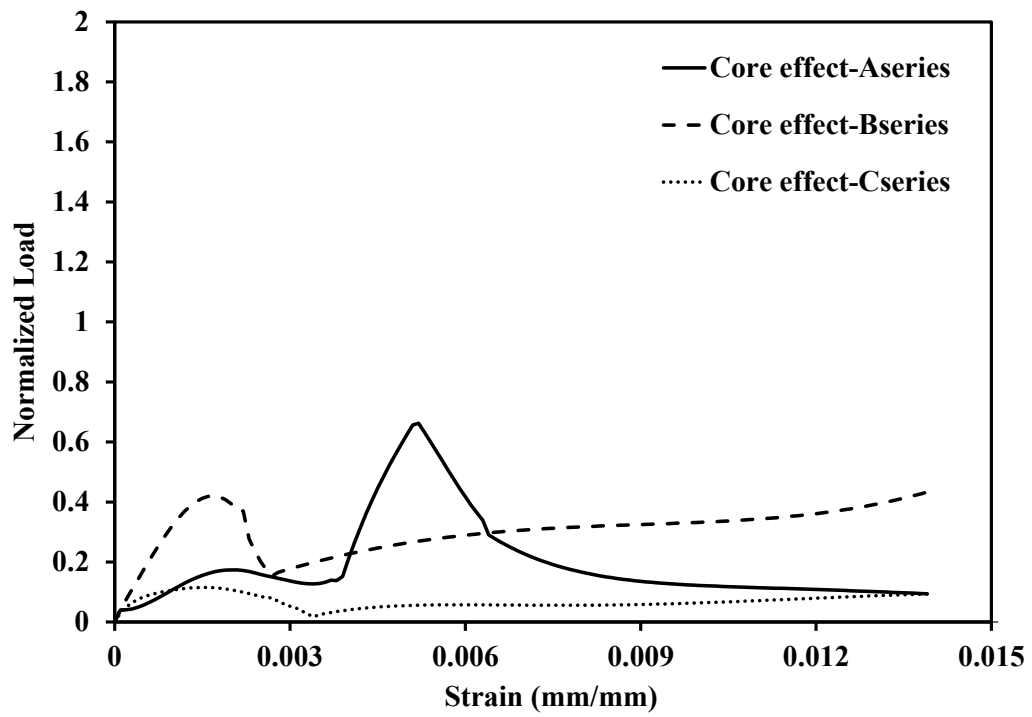


Figure 5.21 Comparison of the Core-Effect in the AR, BR and CR Series Columns

5.3.5 Effect of Fibres on Cover Behaviour

In this section, the effect of fibres in improving cover response is examined. The normalized cover contribution ($P_{n,cover}$) can be obtained by subtracting the response of the columns constructed without cover, from the response of the companion columns constructed with 30 mm cover, as defined in Equation 5-5:

$$P_{n,cover} = \left[\frac{\left(\frac{P_c}{0.85f'_c} \right)_{with\ cover} - \left(\frac{P_c}{0.85f'_c} \right)_{no\ cover}}{A_{ch}} \right] \times \left(\frac{A_{ch}}{A_{co}} \right) \quad \text{Equation 5-5}$$

The parameter $\left(\frac{P_c}{0.85f'_c} \right)_{with\ cover}$ represents the concrete load in the specimens with cover normalized with respect to concrete cylinder strength, $\left(\frac{P_c}{0.85f'_c} \right)_{no\ cover}$ represents the concrete load in the specimens without cover normalized with respect to concrete cylinder strength, and the parameters A_{ch} and A_{co} represent the net core concrete area, and the 30 mm concrete cover area, respectively.

5.3.5.1 Effect of Cover in RC and SFRC Columns

In RC columns, the cover is assumed to spall away at a relatively small strain. Foster (2001) has shown that cover spalling in reinforced concrete columns is initiated due to the triaxial stress condition that occurs between the confined core, the tie reinforcement and the cover shell which causes cracking to initiate at the core-cover interface. With the inclusion of fibres, cover spalling can be delayed due to the resulting improvement in tensile resistance at this interface.

Figure 5.22 plots the normalized cover contributions in specimens AR0 and CR0. The cover in both concrete specimens rapidly spalls shortly after peak, due to the poor tensile resistance at core-cover interface. The cover response in both specimens is essentially the same and thus it can be understood that buckling plays a minor role in the spalling mechanism of the plain concrete cover in these columns.

Figure 5.23 and Figure 5.24 show the normalized cover contributions in the AR-series and CR-series specimens. The cover in the SFRC specimens carry significantly more load when compared to the companion columns constructed without fibres. In particular, the cover continues to contribute after peak, demonstrating that addition of fibres significantly improves cover contribution to post-peak resistance in SFRC columns.

In contrast to the plain concrete columns, differences are observed when comparing the SFRC cover contributions of the AR vs. CR series columns. In the AR-series, an initial drop occurs shortly after peak, followed by a secondary peak, after which the cover contribution gradually decays. In the CR-series, a single peak is observed, and after a slight drop, the cover response stabilizes, even as very large axial strains are applied. The initial drop in cover contribution can be related to the initiation of bar buckling. The second peak occurs due to pull-out of fibres within the core, as the cover and ineffective region within the core pull away together from the inner core. As buckling continues in the AR-series columns, the bars push against large pieces of SFRC cover that are still carrying load, causing “effective spalling” and a gradual drop in cover contribution with increasing axial strains. In the case of the CR-series, the ineffective core region is minimal and therefore the secondary peak is not observed. Furthermore, buckling is minimal, and therefore the cover contribution remains stable until the end of testing.

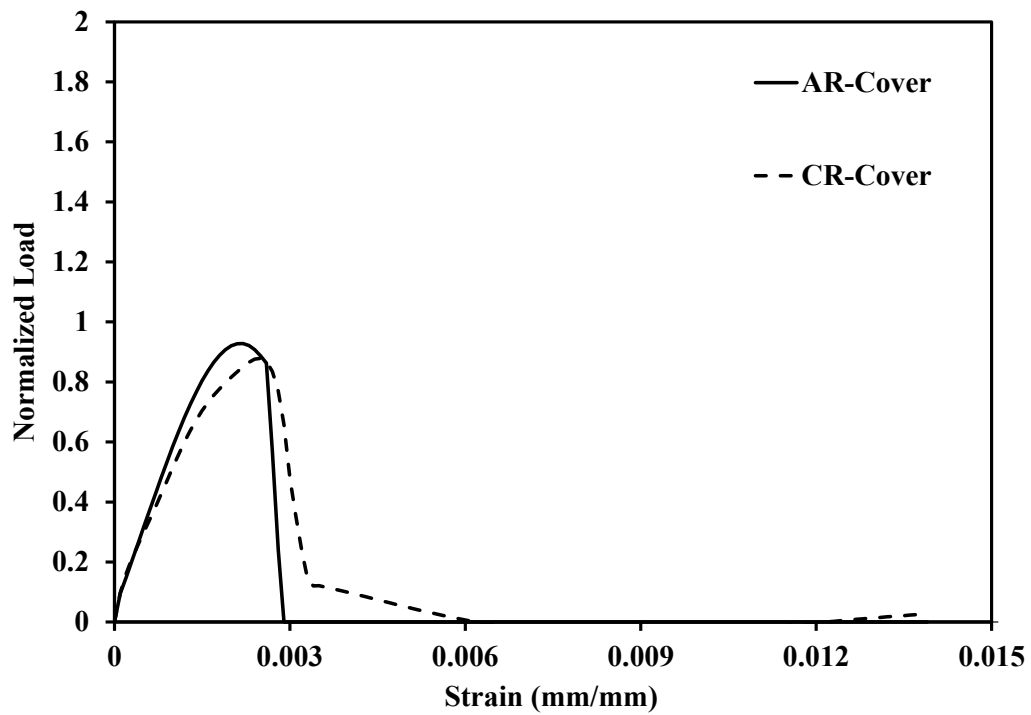


Figure 5.22 Comparison of Cover Response for 0% Fibres

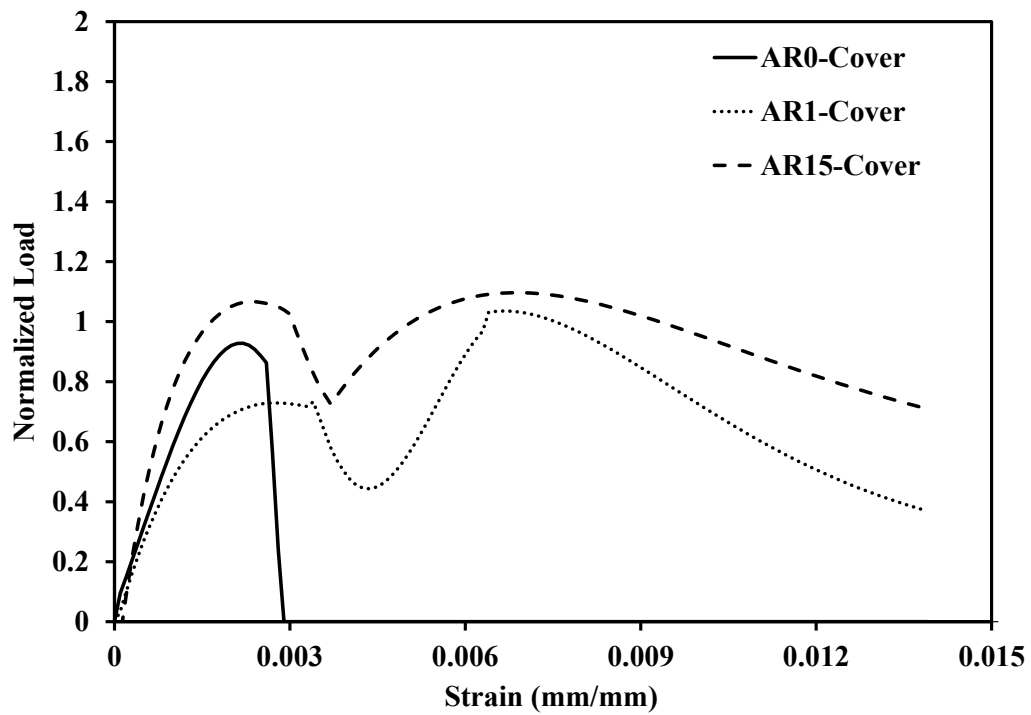


Figure 5.23 Comparison of Cover Response in the AR-Series Columns

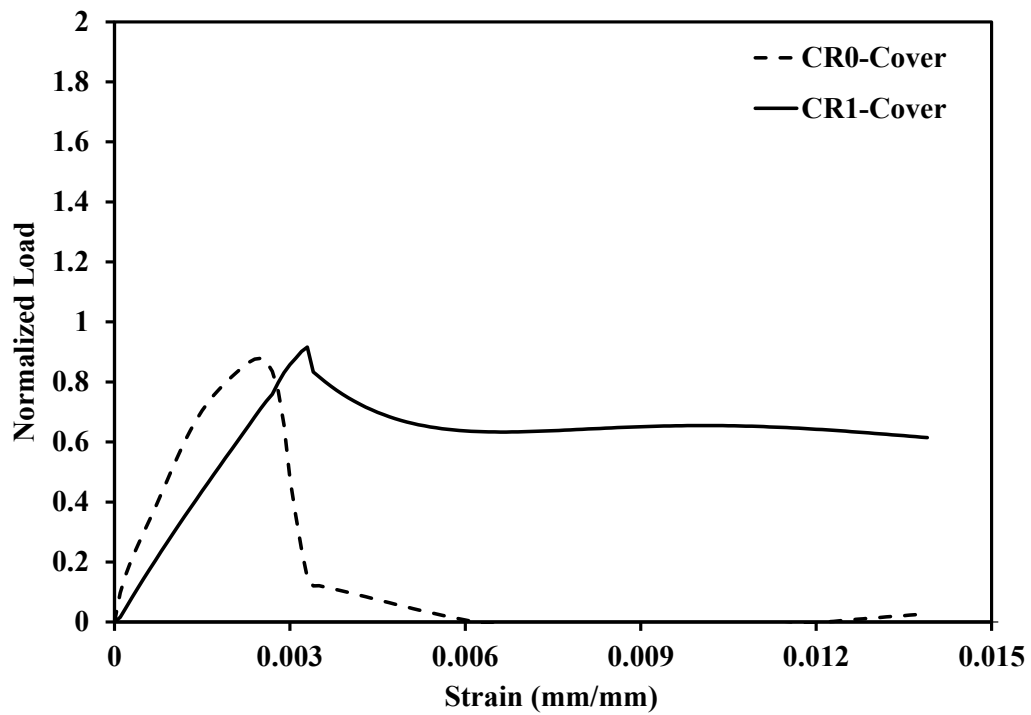


Figure 5.24 Comparison of Cover Response in the CR-Series Columns

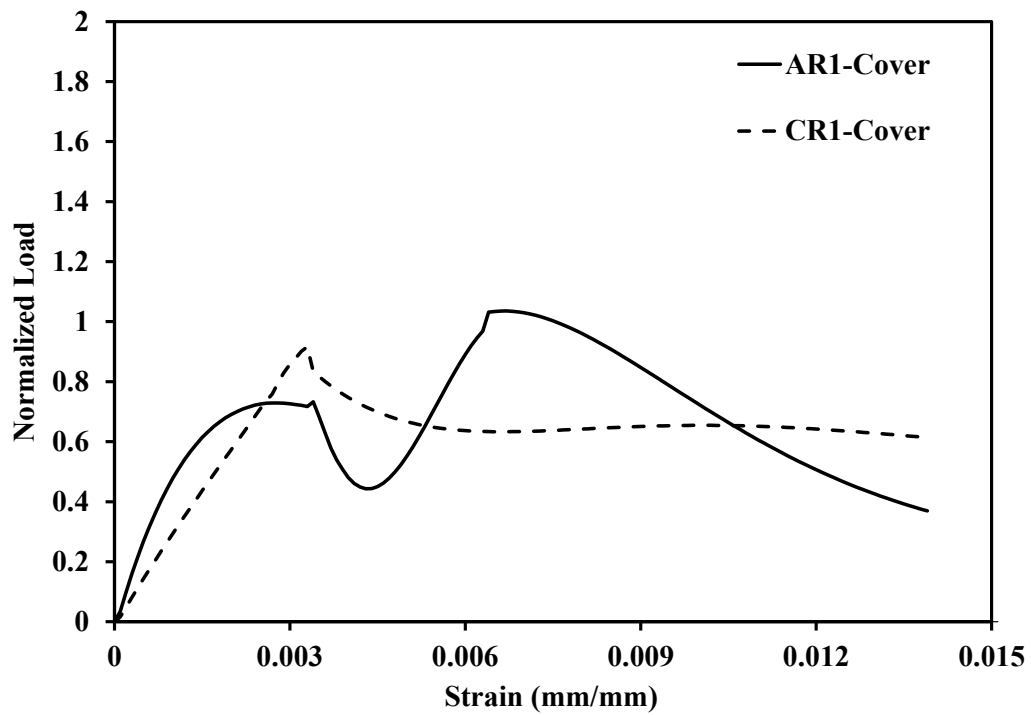


Figure 5.25 Comparison of Cover Response for 1% Fibres

5.3.6 Summary of Effect of Fibres on Core and Cover Concrete

After reviewing the effect of fibres on core and cover individually, it is possible to see the superposition of the combined effects. This will show that, fibres enhance the column behaviour step by step.

Figure 5.26 plots the normalized response of specimens AR0nc, AR1nc and AR1. Also plotted is the “core improvement”, “cover contribution” and “total improvement” (core + cover) for 1% fibres. If one starts with the response of AR0nc and adds the “core contribution”, the improved response of AR1nc results. Next, if one adds the “cover contribution”, the total improvement in AR1 is obtained. Similarly, Figure 5.27 plots the normalized response of specimens CR0nc, CR1nc and CR1, along with the “core improvement”, “cover contribution” and “total improvement” (core + cover) for 1% fibres. If one starts with the response of CR0nc and adds the “core contribution”, the slightly improved response of CR1nc results. Next, if the “cover contribution” is added, the total improvement results in CR1.

Thus, in nominally confined columns (e.g. AR-series), fibres contribute to improving core and cover response. In the case of highly confined columns (e.g. CR-series), fibres contribute primarily in the cover, while the effect in the core is less important.

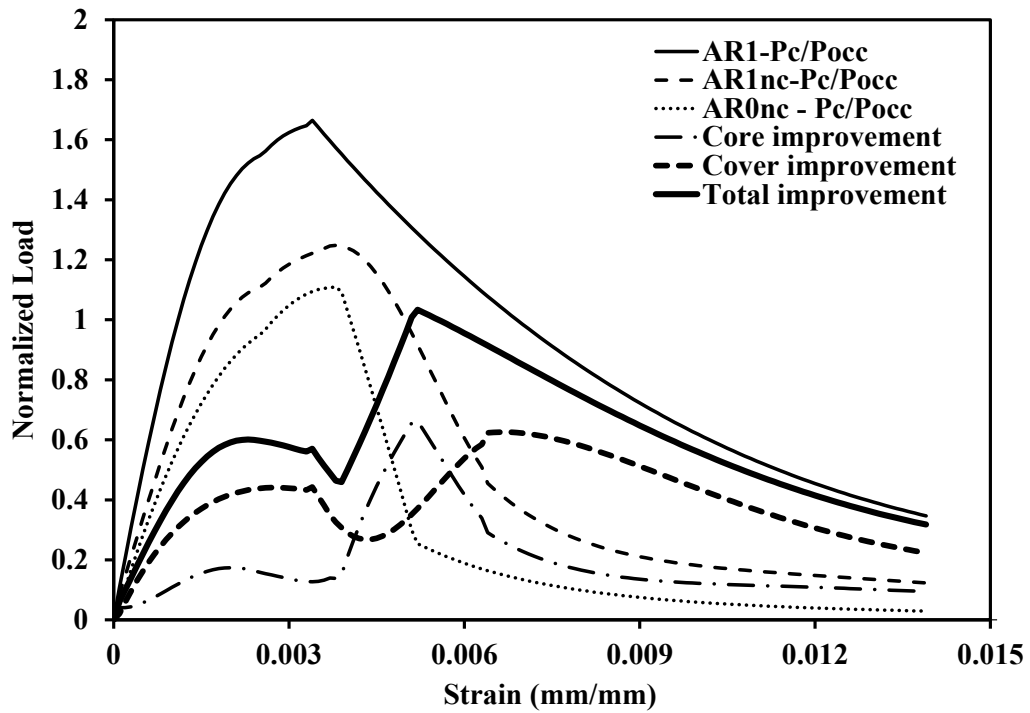


Figure 5.26 Fibre Effect on Columns with Low Confinement (Individual Improvements)

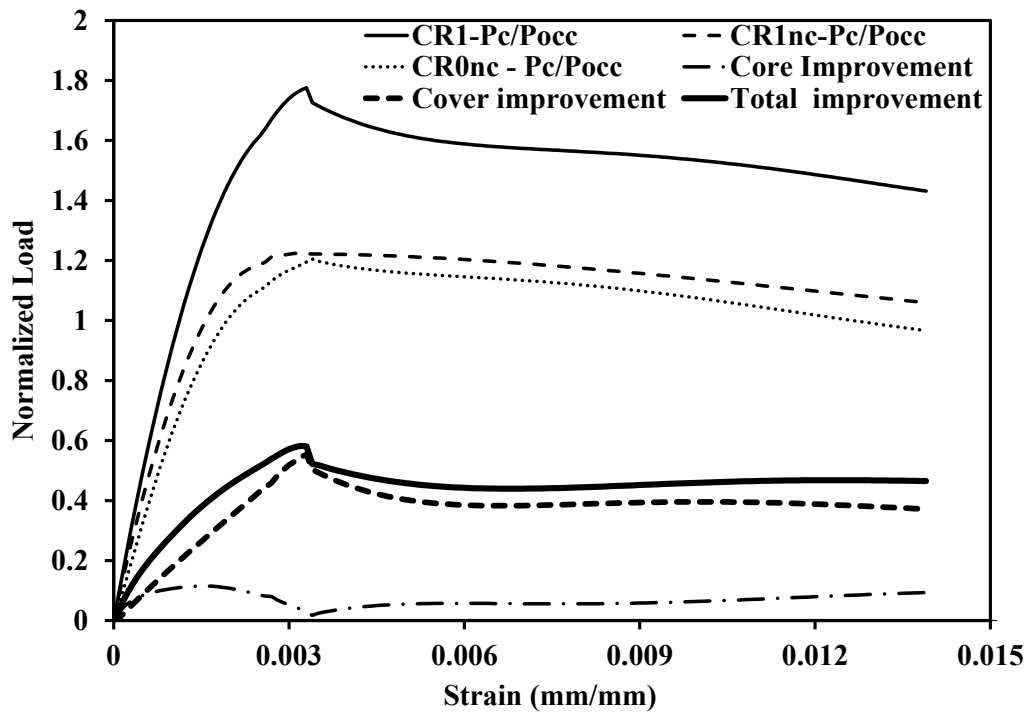


Figure 5.27 Fibre Effect on Columns with High Confinement (Individual Improvements)

5.4 Discussions of Failure Mechanisms – SFRC Series

Figure 5.28 and Figure 5.29 show the observed stages in the failure of specimens AR0-BR0-CR0 and specimens AR15-BR15-CR15 and shows the differences in failure process in the SCC and SFRC columns.

5.4.1 Effect of Fibres on Crack Pattern, Cover Spalling and Crushing of the Core

For the plain SCC columns (0% fibres) spalling is observed to occur rapidly soon after peak load is reached in the columns. After initiation of cracking at the core-cover interface, large pieces of cover buckle and detach suddenly from the specimens. It is also observed that the cracks on the column surface are mainly straight without significant secondary cracking. Addition of fibres significantly changes the spalling and crushing mechanisms in these columns. Spalling of the cover is well controlled and occurs gradually. The provision of fibres ensures that the tensile load path at the core-cover interface is kept functional and thus more dispersed crack pattern with development of many secondary cracks is observed. This can be further seen by comparing the staged failure process and the crack pattern on the cover surface in Figure 5.28 and Figure 5.29. Nonetheless, bar buckling did play a secondary role in “effective spalling” of SFRC cover (see next section).

Core crushing also was observed to be affected by introduction of fibres and different amount of confinement. For the plain SCC columns, the concrete in the ineffective core region crushes and disintegrates at a rapid rate in specimen AR0, while crushing of the core is better controlled in column BR0. In the case of CR0, crushing of core concrete is all but eliminated due to the provision of closely spaced transverse reinforcement. With provision of fibres, core crushing is better controlled and it is observed that as amount of confinement is increased, the intactness of core concrete becomes more obvious (see Figure 5.30).

5.4.2 Effect of Fibres on Bar Buckling

Despite the improvements in cover and core mechanisms, the results from the experimental program demonstrate that bar buckling is not prevented when fibres are added to concrete. Figure 5.30 shows the different specimens after testing. It can be observed that even though specimens AR1, AR1.5, BR1 and BR1.5 contain fibres the longitudinal bars buckled between the hoops (both in middle of the section and especially at the corner). In the CR-

series, due to the reduced tie spacing, little or no bar buckling is observed. As testing continued beyond the peak capacity, the bars began pushing against the SFRC cover causing the cover to gradually spall (see Figure 5.31). Eventually the cover became ineffective, although weakly held in place with a few fibres. This rate of spalling was more rapid in the case of columns with large hoop spacing.

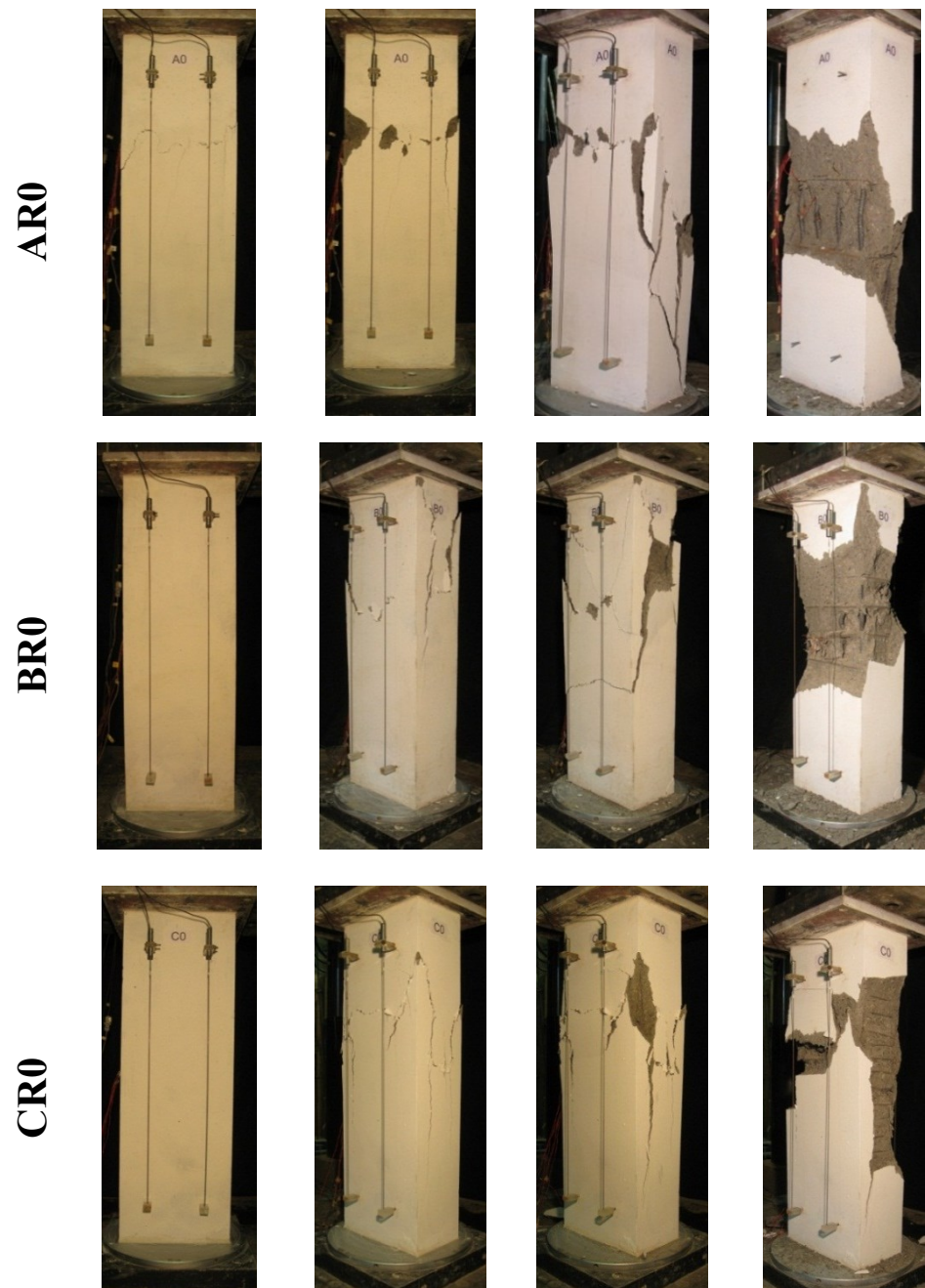
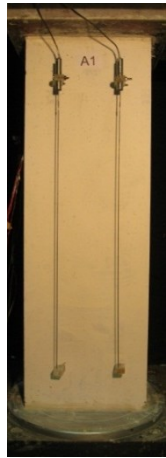
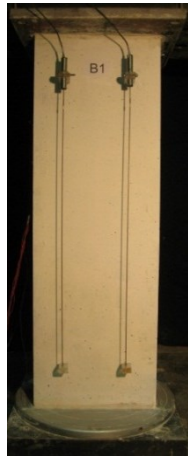


Figure 5.28 Staged Failure Process for Columns with 0%-Fibres

AR1



BR1



CR1

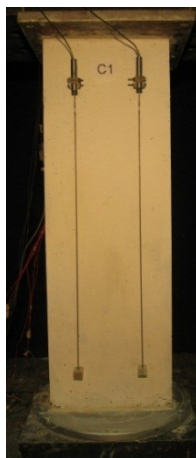


Figure 5.29 Staged Failure Process for Columns with 1%-Fibres



(a)AR0



(b)AR1
AR-Series



(c)AR15



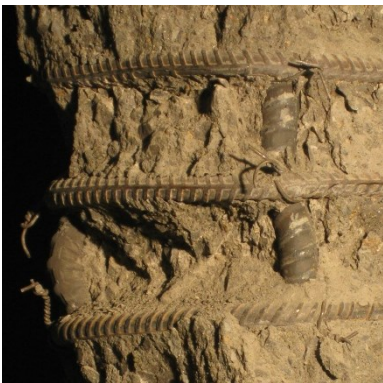
(d)BR0



(e)BR1
BR-Series



(f)BR15



(g)CR0



(h)CR1
CR-Series



(i)CR15

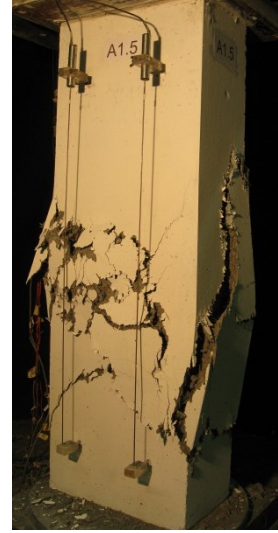
Figure 5.30 Visual Comparison of Core Crushing and Bar Buckling



(a) AR1



(b) AR15



(c) BR1



(d) BR15-side



(e) BR15-front

Figure 5.31 Examples of Effective Cover Spalling in SFRC Columns

5.4.3 Effect of Fibres on Development of Failure Zone

Paultre et al. (2010) observed that the appearance of the concrete columns after the test changes when fibres are added. In the non-fibrous specimens, a large cover region buckles and thus a large amount of core surface is evident at the end of testing, whereas in the fibre reinforced concrete columns, cover failure occurs when the crushing strength is reached or exceeded in the cover, and therefore spalling is limited to the failure zone. In the current study this fact is also observed when examining the failure photos of the specimens (see Figure 5.32).



(a) AR0



(b) BR0



(c) CR0



(d) AR1



(e) AR15



(f) BR1



(g) BR15



(h) CR1



(i) CR15

Figure 5.32 Effect of Fibre in Development of Failure Zone

5.4.4 Hook Deformation

Finally, examination of the columns at the end of testing showed that the 90 degree hooks on the cross-ties in the columns had a tendency to open up for most specimens tested in the experimental program (Figure 5.33). It is noted that the two intermediate cross-ties had a 135 degree seismic hook at one end and a 90 degree hook on the opposite end, with the 90 degree hooks of two successive crossties alternated from end to end. It is believed that deformation of 90 degree hook is one of the factors for initiation of cover spalling and further instability of the cover.



(a) BR0nc



(b) BR1nc



(c) CR0nc



(d) CR1nc

Figure 5.33 Hook Deformation

5.5 Summary of Results - UHPFRC Series

Six UHPFRC columns were tested in the experimental program. All columns had 250 mm x 250 mm square section, 10 mm cover and were reinforced with 2.5% steel fibres. The main variables were the configuration of transverse reinforcement (C3 and C4) and spacing of the transverse reinforcement ($s=120 - 40$ mm). As a reminder, Figure 5.34 shows the two configurations studied in the experimental program.

In order to compare the results Table 5.3 lists several parameters, including: peak load (P_t) and strain (ϵ_t), peak load carried by concrete (P_c) and corresponding strain (ϵ_c) and strains corresponding to 85% drop (ϵ_{c85}) and 50% drop of capacity (ϵ_{c50}). Furthermore, normalized load (P_t/P_0) and strains (ϵ_t/ϵ_{cu} , $\epsilon_{c85}/\epsilon_{cu}$, $\epsilon_{c50}/\epsilon_{cu}$), are also listed (where the unconfined strain ϵ_{cu} is assumed to be 0.0028 for UHPFRC).

In order to study the effect of the test variables on toughness and post-peak ductility, the area under the concrete load-strain curves up to strain of $\epsilon_{Au} = 0.02$ is also presented (A_u) along with the toughness index (T.I.), defined previously.

In addition, to give an indication of confinement, the "effective confinement index" proposed by Mander et al. (1988) (see Equation 5-6) is reported for each specimen.

$$K_e = \frac{\left(1 - \frac{\sum w_i^2}{6c_x c_y}\right) \left(1 - \frac{s'}{2c_x}\right) \left(1 - \frac{s'}{2c_y}\right)}{1 - \rho_c} \quad \text{Equation 5-6}$$

Where K_e is the confinement effectiveness coefficient and the parameters s' and c represent the clear spacing between ties and the width of the concrete core region. The quantities $\sum w_i^2$ and ρ_c represent the sum of the squares of the clear spacing between adjacent longitudinal bars and the ratio of longitudinal reinforcement in the core region, respectively.

Table 5.3 Summary of Results for UHPFRC Specimens

	f'_c (MPa)	P_t (KN)	ϵ_t	P_c (KN)	ϵ_c	ϵ_{c85}	ϵ_{c50}	$\frac{P_t}{P_o}$	$\frac{\epsilon_t}{\epsilon_{co}}$	$\frac{\epsilon_{c85}}{\epsilon_{co}}$	$\frac{\epsilon_{c50}}{\epsilon_{co}}$	T. I.	A_u (KN)	K_e^*
C3-40	114.1	10965	0.0072	10236	0.0072	0.0095	0.0340	1.65	2.61	3.42	12.23	0.71	145	0.74
C3-60	124.6	9670	0.0061	8943	0.0061	0.0069	0.0202	1.35	2.2	2.47	7.25	0.63	112	0.67
C3-80	137.8	9946	0.0049	9220	0.0049	0.0068	0.0130	1.27	1.75	2.46	4.66	0.58	108	0.60
C3-120	120.5	9070	0.0043	8349	0.0043	0.0057	0.0097	1.30	1.56	2.06	3.47	0.52	87	0.48
C4-60	134.9	10246	0.0067	9161	0.0067	0.0109	0.0396	1.28	2.41	3.94	14.24	0.76	139	0.75
C4-120	129.1	9803	0.0052	8726	0.0052	0.0066	0.0123	1.28	1.86	2.37	4.43	0.58	101	0.54

*Effective Confinement index calculated based on Mander et al 1988



Figure 5.34 UHPFRC Column Configurations

5.5.1 Effect of Tie Spacing

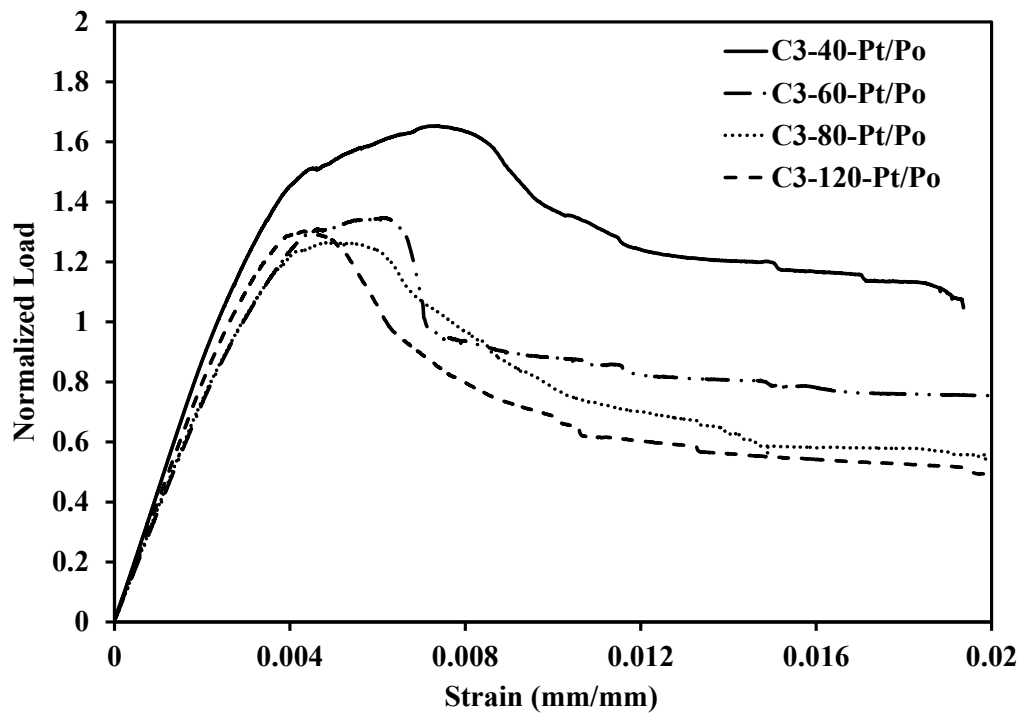
The UHPFRC columns were detailed with various tie spacing ranging from 40 mm to 120 mm.

For configuration C3, the comparison of all specimens is plotted in Figure 5.35 both in terms of load-strain and normalized load-strain response. It is observed that by reducing the spacing from 120 mm to 40mm, and increasing the amount of confinement by 54 % (based on the effective confinement index, K_e), the strength and ductility parameters are improved.

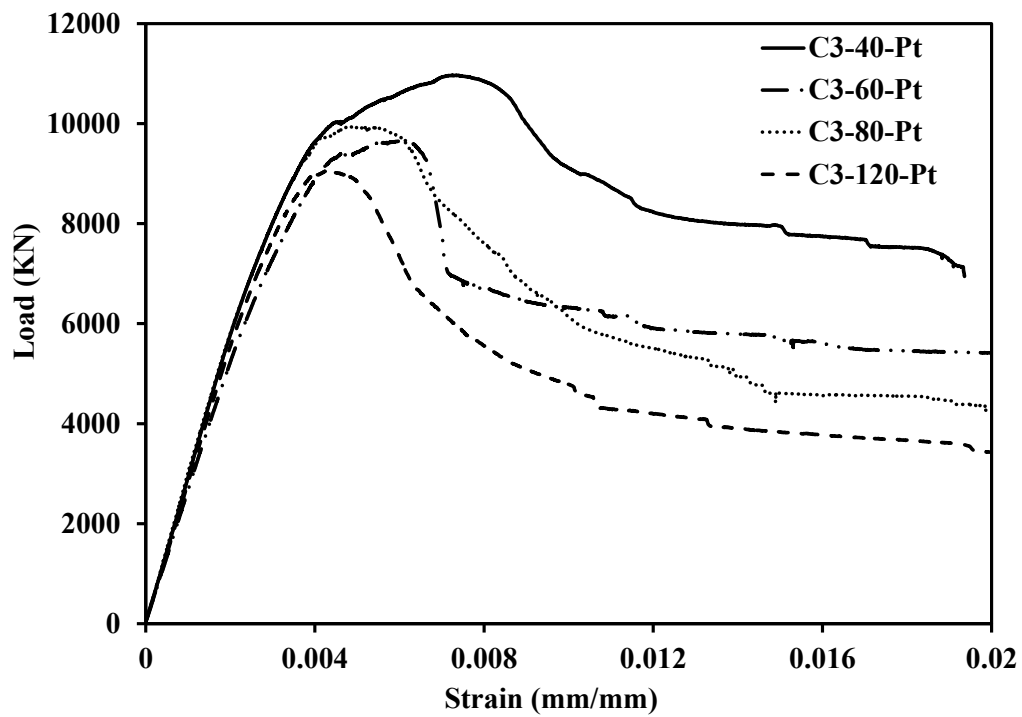
Examining Figure 5.35 and Table 5.3, it is observed that among all columns the ratio P_t/P_o reaches a maximum value of 1.65 for column C3-40, while ϵ_t/ϵ_{co} reaches a maximum value of 2.61 for the same specimen. The trend of $\epsilon_{c85}/\epsilon_{co}$ and $\epsilon_{c50}/\epsilon_{co}$ shows that by reducing spacing from 120 mm to 40 mm, ductility improves by 66% and 352% for these two parameters, respectively. Moreover, by considering the improvement of area under the curve and toughness index, an improvement of 67 % and 37 % can be obtained, respectively.

By the same method, for configuration C4 the results for specimens C4-60 and C4-120 can be compared in Figure 5.36. It is clear that by reducing the spacing (39% increase in confinement index) although ultimate strength is increased slightly, improvement of the post-peak ductility is significant. Examining Table 5.3, it is evident that P_t/P_o remains constant at 1.28 while ϵ_t/ϵ_{co} increases by 30 %. The ductility strain values $\epsilon_{c85}/\epsilon_{co}$ and $\epsilon_{c50}/\epsilon_{co}$ show improvements of 48% and 321%, respectively. In addition area under the curve and toughness index increases from 101 KN and 0.58 to 139 KN and 0.76 (increases of 38% and 31 %, respectively).

From both configurations, the results show that increasing the amount of confinement through reduction of tie spacing improves strength and ductility in UHPFRC columns.

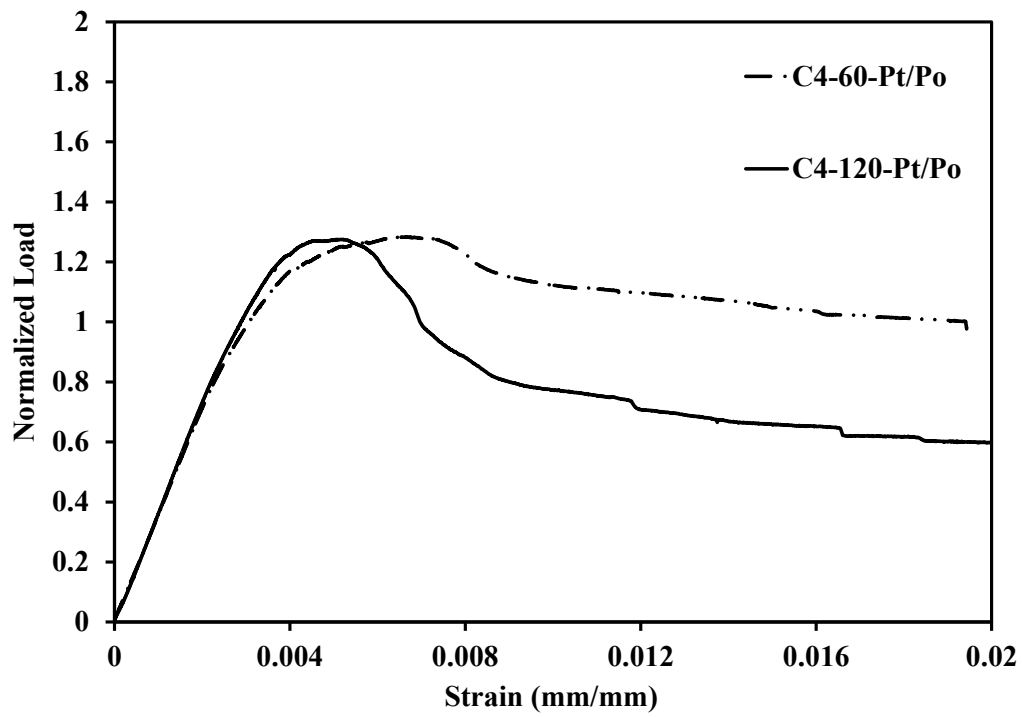


(a) Normalized Load versus Strain

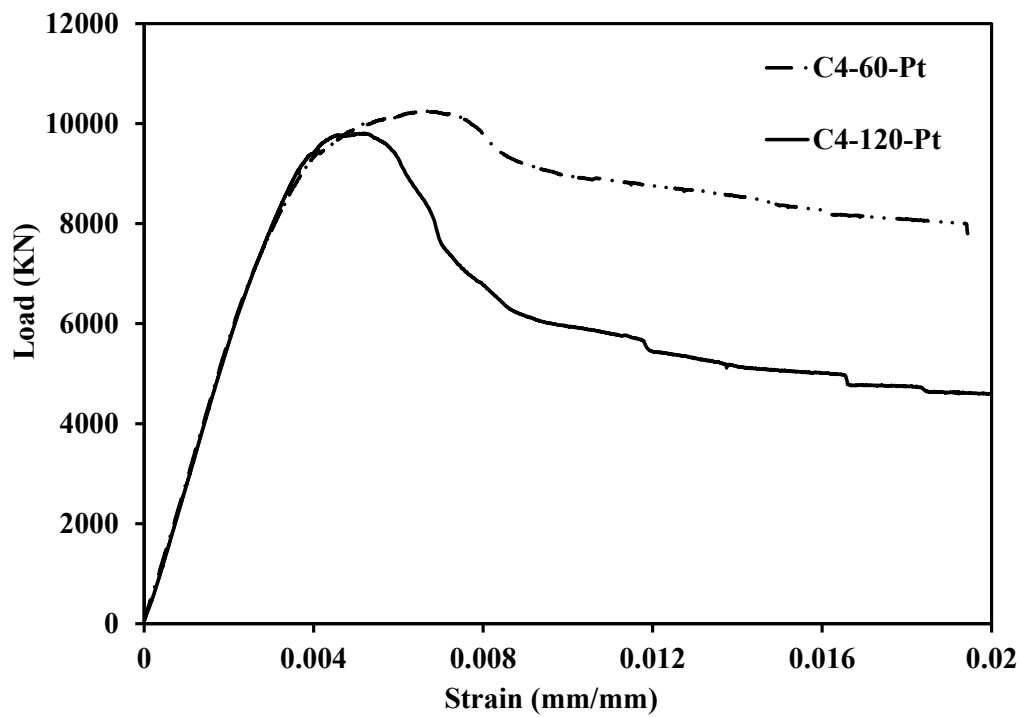


(b) Load versus Strain

Figure 5.35 Comparison of UHPFRC Responses for Specimens with Configuration C3



(a) Normalized Load versus Strain



(b) Load versus Strain

Figure 5.36 Comparison of UHPFRC Responses for Specimens with Configuration C4

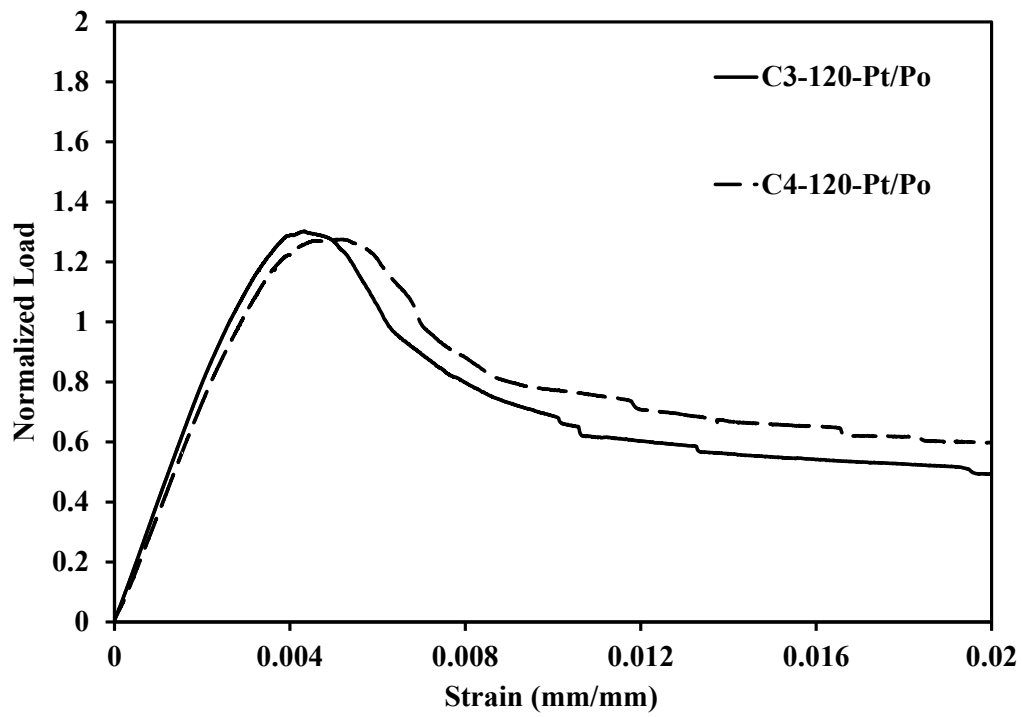
5.5.2 Effect of Tie Configuration

Since columns C3-60 and C4-60 ($s = 60$ mm) and C3-120 and C4-120 ($s = 120$ mm) had the same tie spacing but had different configurations, the behaviour of these columns can allow for an investigation into the effect of transverse reinforcement arrangement. To recall, configuration C4 had an extra crosstie and extra longitudinal bar in each direction (Figure 5.34).

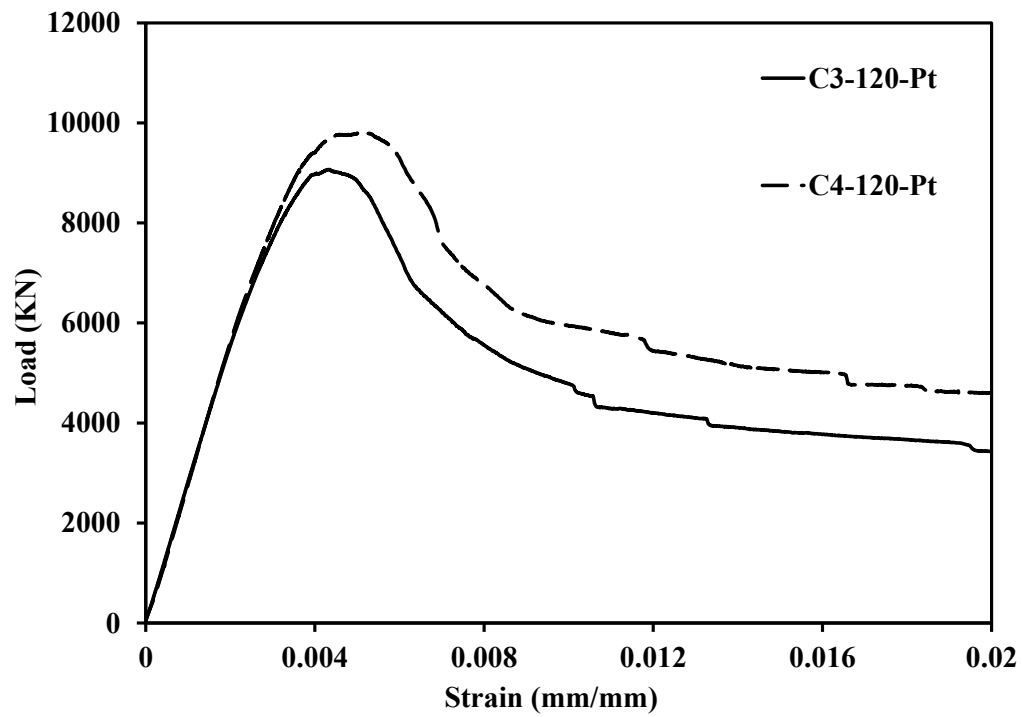
For the columns with spacing of 120 mm, examination of Figure 5.37 and Table 5.3, shows that by increasing the confinement from C3 towards C4 (12 % increase in effective confinement index), while no improvement in the ratio P_t/P_o is observed, the value of ϵ_t/ϵ_{co} is increased by 19%. The ductility strains $\epsilon_{c85}/\epsilon_{co}$ and $\epsilon_{c50}/\epsilon_{co}$ are also increased by 31% and 28 % respectively. Furthermore, the toughness parameters A_u and T.I. increase by 12% and 16 %, respectively.

Considering the columns with 60 mm spacing, which have improved level of confinement due to reduced spacing, by further addition of confinement through changing of the configuration from C3 to C4 (21 % increase in effective confinement index), while no improvement of P_t/P_o is observed, , the value of ϵ_t/ϵ_{co} is increased by 9.3%. However, the ductility strains $\epsilon_{c85}/\epsilon_{co}$ and $\epsilon_{c50}/\epsilon_{co}$ are increased significantly by 60% and 96%. In addition, the parameters A_u and T.I. increase by 24% and 21 %, respectively.

In summary, by increasing the confinement through improvement of cross-section detailing, no improvement in strength is observed while post-peak ductility is affected. The effect is more significant in columns with reduced tie spacing (e.g. 60 mm spacing) than in columns having large tie spacing (e.g. 120 mm spacing) where only slight change is seen in post-peak ductility.

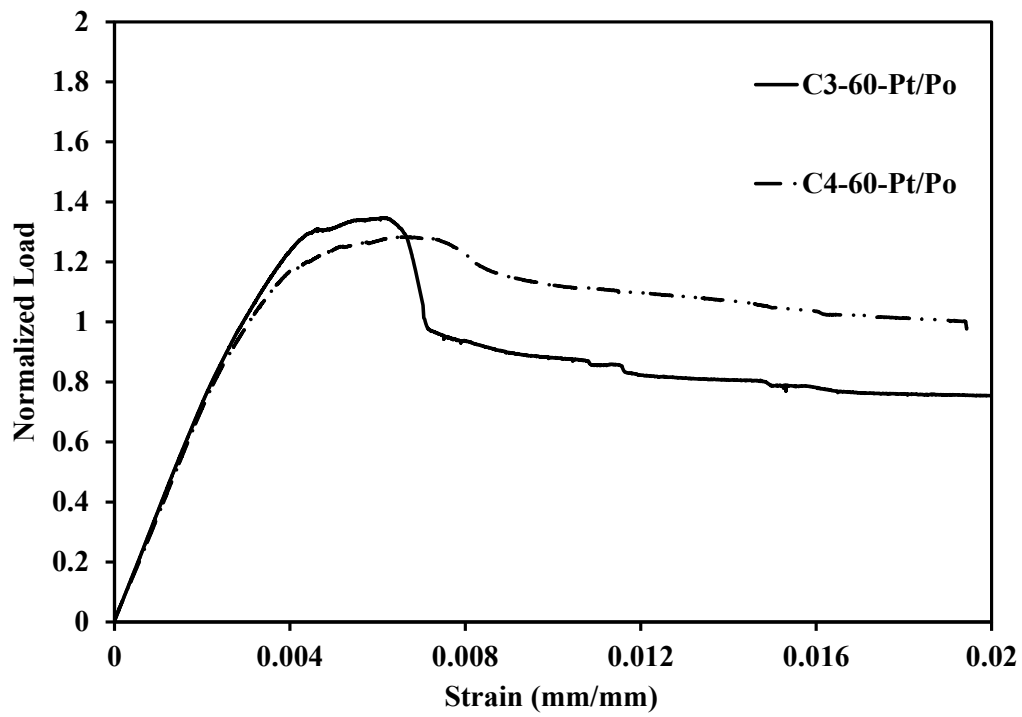


(a) Normalized Load versus Strain

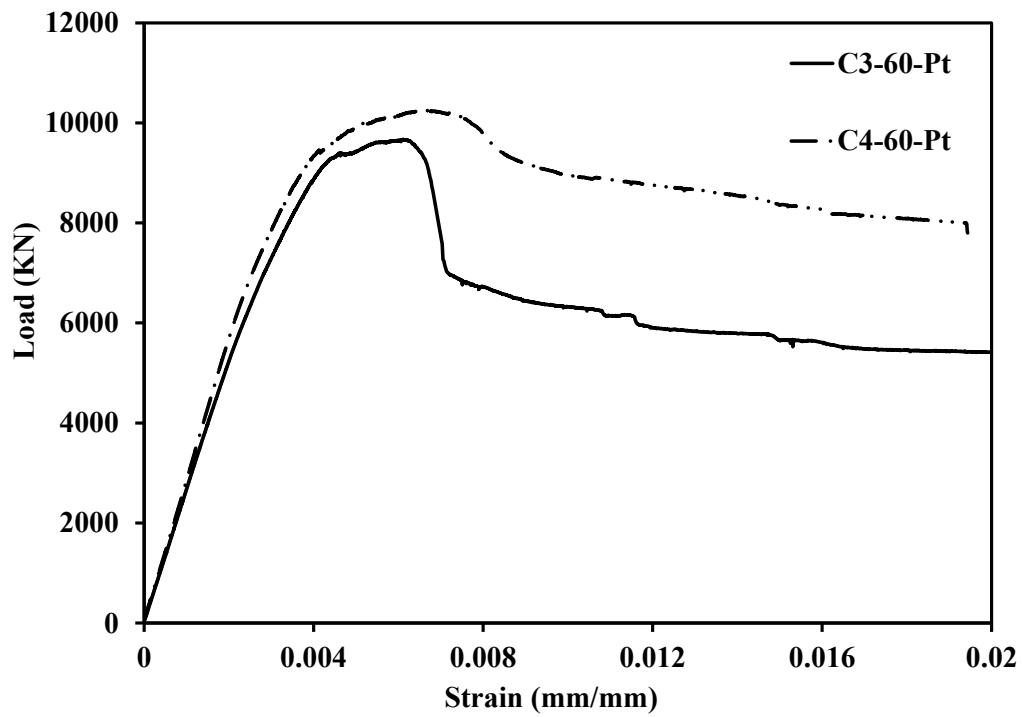


(b) Load versus Strain

Figure 5.37 Comparison of UHPFRC Responses for Specimens with Spacing of 120 mm



(a) Normalized Load versus Strain



(b) Load versus Strain

Figure 5.38 Comparison of UHPFRC Responses for Specimens with Spacing of 60 mm

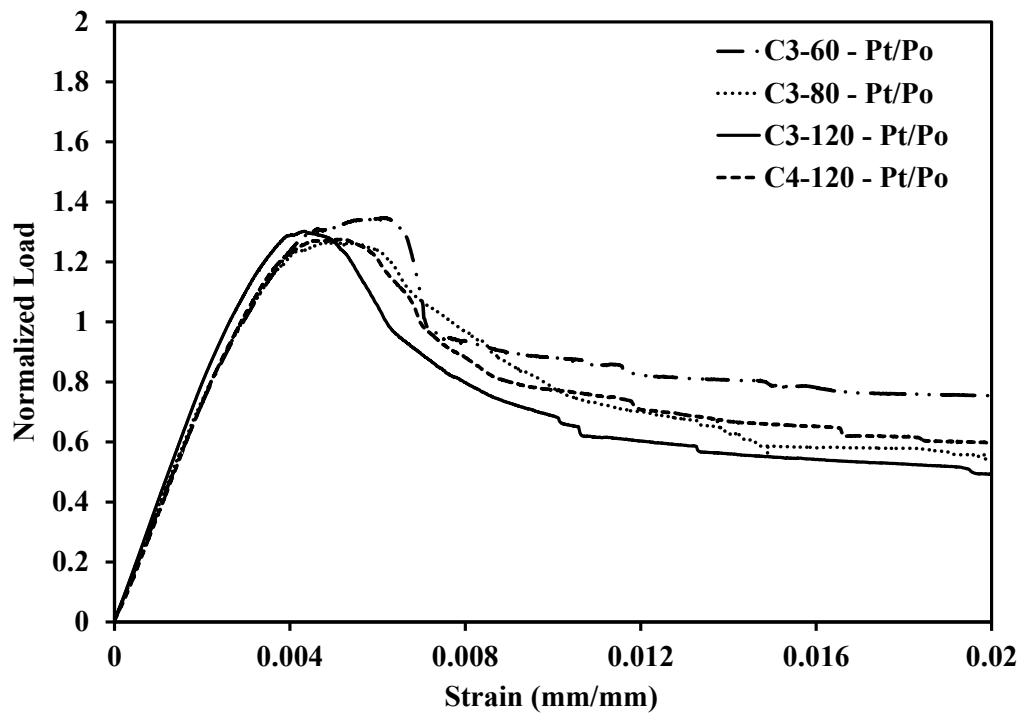
5.5.3 Further Discussion

5.5.3.1 Effect of Confinement in Moderately Ductile Columns ($R_d=2.5$)

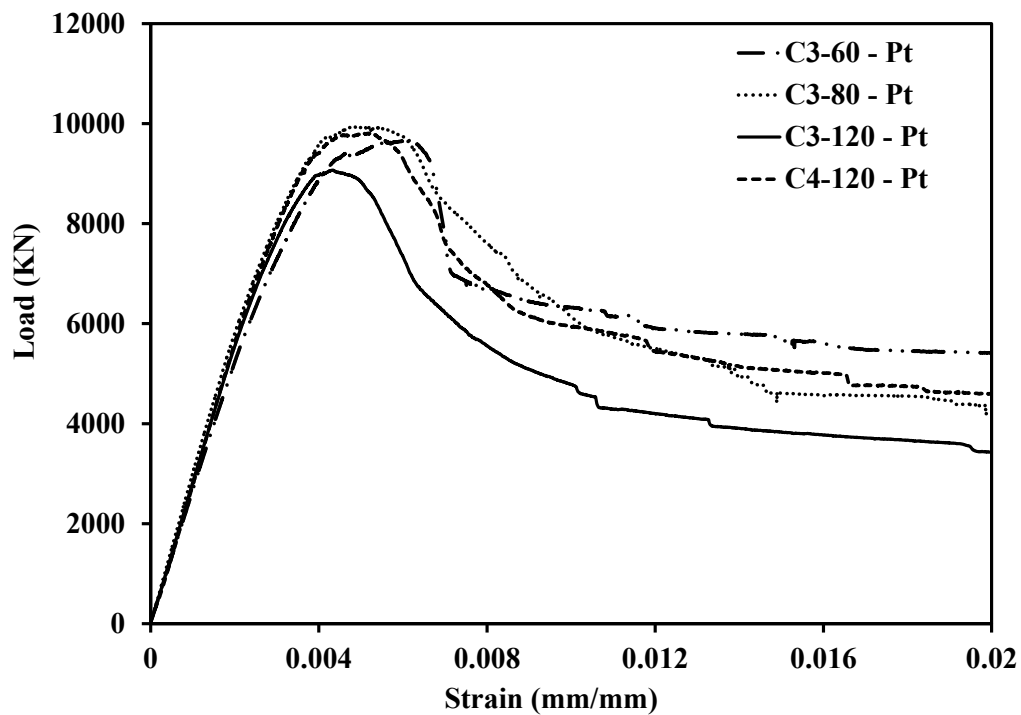
The response of the columns meeting the minimum requirements for moderately ductile columns in the CSA A23.3 standard is compared in Figure 5.39 (specimens C3-120, C4-120, C3-80 and C3-60). Considering these specimens the effective confinement index ranges from 0.48 up to 0.67 (with a 40 % increase when comparing columns C3-120 and C3-60). It is evident from Figure 5.39 that no significant improvement of P_t/P_o , nor ϵ_t/ϵ_{co} is observed, while area under the curve (A_u) and toughness index (T.I.) are increased as hoop spacing reduces. The toughness index has the same value of 0.58 for columns C4-120 and C3-80 while ductility strains $\epsilon_{c85}/\epsilon_{cu}$ and $\epsilon_{c50}/\epsilon_{co}$ are slightly higher for C3-80 (4% and 5% respectively). Generally it is seen that behaviour of all specimens in this group is similar, although some slight improvement is observed when the configuration improves.

5.5.3.2 Effect of Confinement in Highly Ductile Columns ($R_d=4.0$)

The response of the columns meeting the minimum requirements for ductile columns in the CSA A23.3 standard is compared in Figure 5.40 (specimens C3-40 and C4-60). Consideration of the P_t/P_o ratios shows a higher normalized capacity for column C3-40. However, this specimen had lower than expected cylinder strength, and it is uncertain if the data obtained from the cylinder testing is reliable. It is noted that the P_t values are similar (only 7% higher for C3-40). It is observed that although ϵ_t/ϵ_{co} is slightly higher for C3-40 (increase of 8.4%), ductility strains $\epsilon_{c85}/\epsilon_{co}$ and $\epsilon_{c50}/\epsilon_{co}$ are improved for C4-60. Similarly the toughness index (T.I.) is 6.5% higher for C4-60. In summary, the behaviour of both columns that are detailed based on the same level of ductility, regardless of configuration is similar.

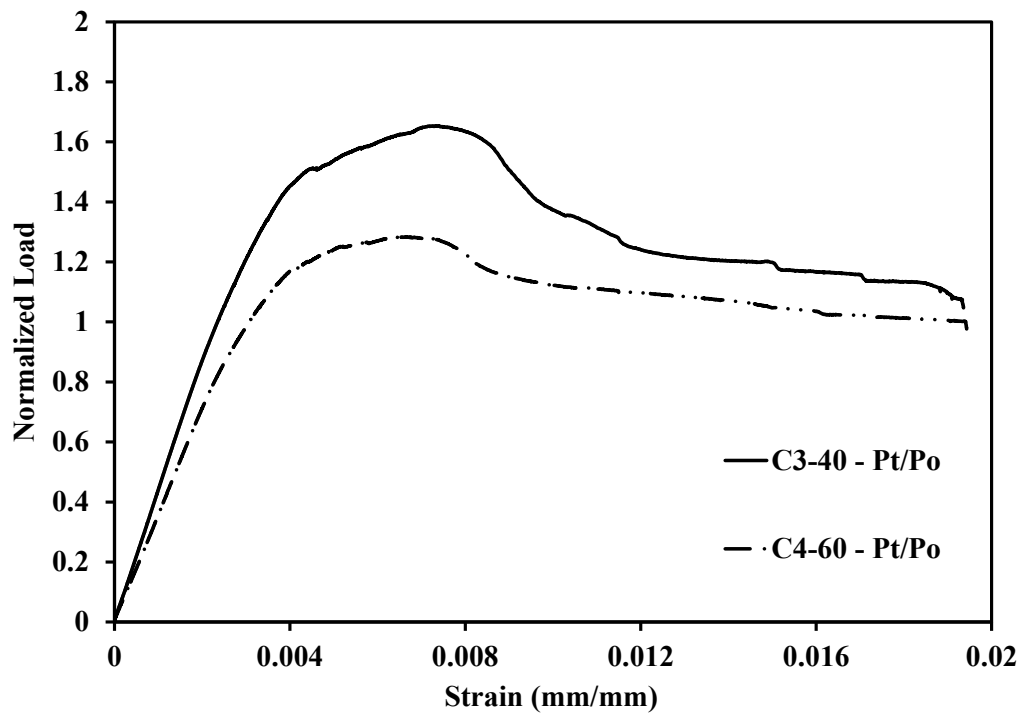


(a) Normalized Load versus Strain

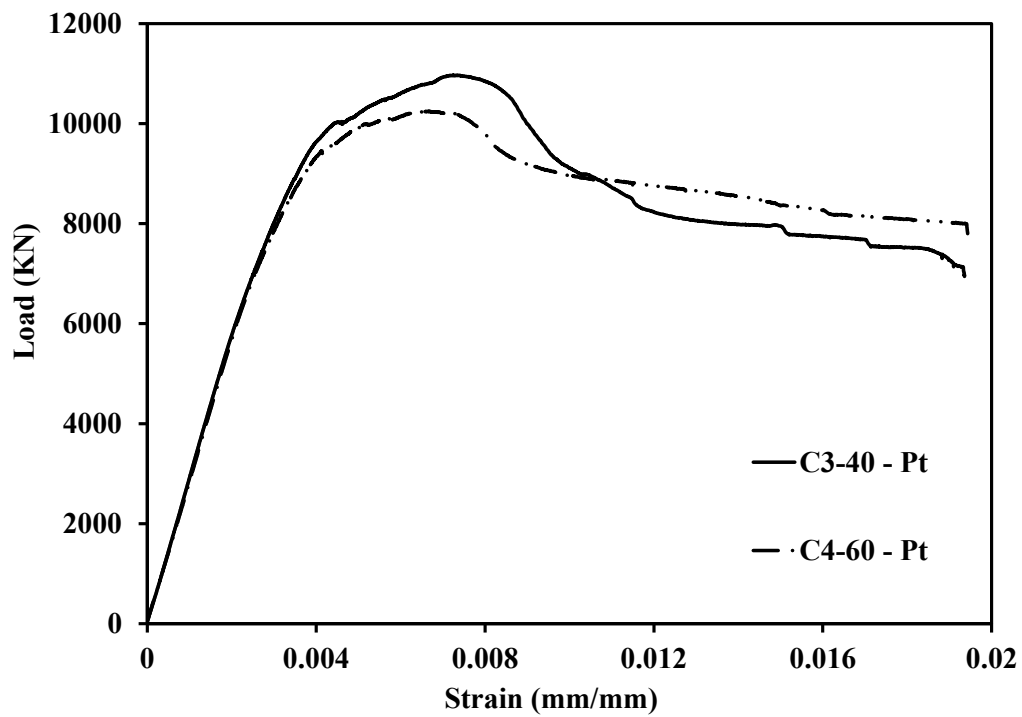


(b) Load versus Strain

Figure 5.39 Comparison of UHPFRC Responses for Specimens with Moderate Ductility



(a) Normalized Load versus Strain



(b) Load versus Strain

Figure 5.40 Comparison of UHPFRC Responses for Specimens with High Ductility

5.6 Discussions of Failure Process – UHPFRC Columns

Figure 5.41 presents the staged failure for two UHPFRC specimens (columns C3-40 and C4-60). Comparing to the SCC and SFRC specimens, the mode of failure in the UHPFRC columns is different with damage distributed throughout the column. Due to existence of fibres, cracking is well distributed on the cover surface with the formation of primary cracks and a network of secondary cracks. In addition it is evident that although cover spalling initiates, cover remains attached to the specimens throughout the test (Figure 5.42).

Core integrity and bar buckling is investigated in Figure 5.43. It is observed that buckling of the bars occurs in all specimens, except for column C3-40 which showed great core integrity and therefore bar buckling is not clear. Since the concrete was the same for all specimens, the amount of core integrity can be compared relative to amount of column confinement. It is seen that regardless of confinement level, great core integrity is observed in all specimens, even in the middle region between the ties where concrete is expected to be unconfined. Only slight crushing is observed in C3-120, C4-120 and C3-80 which have low level of confinement relative to the other specimens.

C3-40



C4-60



(a) Stage II

(b) Stage III

(c) Stage VI

(d) Stage V

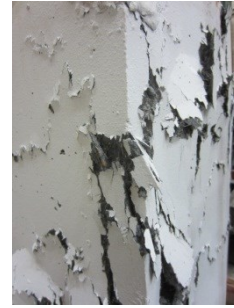
Figure 5.41 Staged Failure and Damage Tolerance in the UHPFRC Columns



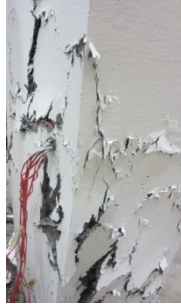
(a) C3-120



(b) C3-80



(c) C3-60



(d) C3-40



(e) C4-120



(f) C4-60

Figure 5.42 Cover Spalling in the UHPFRC Columns



(a) C3-120



(b) C3-80



(c) C3-60



(d) C3-40



(e) C4-120



(g) C4-60

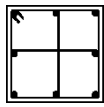
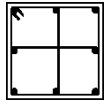
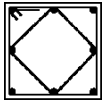
Figure 5.43 Bar Buckling and Core Integrity in the UHPFRC Columns

6 Analytical Evaluation

6.1 Introduction

This chapter presents an analytical study aimed at examining the suitability of using existing models in the literature to predict the axial load response of SCC, SFRC and UHPFRC columns tested in this research program. The chapter begins by clarifying the analysis methodology used to predict the response of the columns (section 6.2). Thereafter, various models that have been proposed in the literature are reviewed in section 6.3. The models are then used to predict the response of the SCC, SFRC and UHPFRC columns in section 6.4. Table 6.1 summarizes the properties of the columns included in the analytical evaluation. For the SCC columns, six rectangular columns from the current research program and six square-section columns tested by Aoode (2008) are considered. For the SFRC columns, the analytical evaluation includes eleven rectangular columns from the current research program and ten square-section columns tested by Aoode (2008). Finally the six UHPFRC columns tested in this research program are also considered in the analytical evaluation.

Table 6.1 Summary of Columns Considered in the Analytical Evaluation

Concrete Type	Study	# specimens*	Cross-section (mm x mm)	Cover (mm)	f'_c (MPa)	V_f (%)	Tie Spacing (mm)	Configuration
SCC	Aoode (2008)	3 <i>c</i> 3 <i>nc</i>	300 x 300 250 x 250	30 5	44~50	--	(C1) 120-240 (C2) 65	  C1 C2
	Current Study	3 <i>c</i> 3 <i>nc</i>	400 x 225 350 x 175	30 5	49~53	--	55-220	 CR
SFRC	Aoode (2008)	6 <i>c</i> 4 <i>nc</i>	300 x 300 250 x 250	30 5	42~48	1-1.5	(C1) 120-240 (C2) 65	  C1 C2
	Current Study	6 <i>c</i> 5 <i>nc</i>	400 x 225 350 x 175	30 5	27~51	1-1.5	55-220	 CR
UHPFRC	Current Study	6	250 x 250	10	114~138	2.5	(C3) 40-120 (C4) 60-120	  C3 C4

*Note: *c* indicates columns were constructed with cover, *nc* indicates columns were constructed "without cover"

6.2 Analysis Methodology

Various methods can be used to predict the axial load response of reinforced concrete columns depending on the modelling purpose and required level of precision. For the purpose of the current analytical evaluation a simplified approach is used, where the response of a column under concentric axial load is considered to be a combination of contributions from three parts:

1. Core concrete;
2. Cover Concrete;
3. Longitudinal bars in compression.

When considering a typical column cross-section, the cover concrete is assumed to extend from the outside column surface to the centerline of the transverse reinforcement. Core concrete is obtained by subtracting the cover concrete thickness from each column dimension (see Figure 6.1). Therefore, it is assumed that the concrete inside the centerline of the transverse reinforcement is confined and the remaining concrete is unconfined.

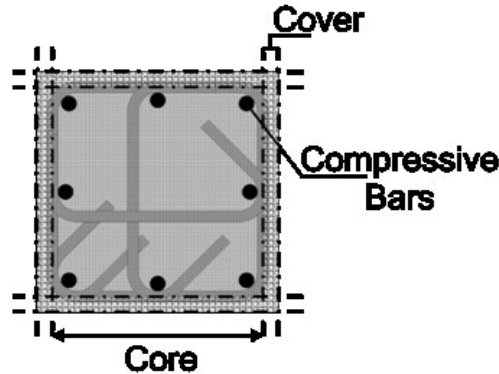


Figure 6.1 Definition of Cover, Core and Reinforcement for Analysis

To model the contribution of the cover, stress-strain models for unconfined concrete and SFRC will be used. As it was observed in the experimental program, addition of steel fibres modifies the mechanism of cover spalling. Thus, for both plain concrete and fibre reinforced concrete, models which account for cover spalling are considered. In the case of the core concrete, stress-strain models for confined concrete and SFRC will be used. Since the longitudinal bars in the columns are in compression, and in order to consider buckling in

specimens with large tie spacing, various models for longitudinal steel reinforcement in compression are considered. The axial load carried by a column at any given strain is given below where the contributions of the core concrete, cover concrete and longitudinal reinforcement in compression are combined:

$$P_{\text{Total}} = A_{\text{ch}}f_{\text{cc}} + \phi_{\text{sp}}A_{\text{co}}f_{\text{cu}} + A_{\text{st}}f_{\text{s}} \quad \text{Equation 6-1}$$

In this equation, A_{ch} , A_{co} and A_{st} represent the core concrete, cover concrete and longitudinal steel area in the column cross-section, respectively; f_{cc} , f_{cu} and f_{s} represent the stress in confined concrete, unconfined concrete and steel in compression at any given axial strain, respectively, and ϕ_{sp} represents the spalling factor proposed by Aoude (2008).

6.3 Review of Existing Models

Over the years models have been proposed for unconfined and confined concrete, as well as models that account for the buckling of longitudinal reinforcement in columns. A limited number of models have also been proposed for unconfined and confined SFRC. In order to assess the suitability of these models for predicting the axial load response of the SCC, SFRC and UHPFRC columns various models are reviewed in this section (see Table 6.2).

Table 6.2 Constitutive Models Reviewed in Section 6.3

Unconfined Concrete		Confined Concrete	Longitudinal Bars in Compression
Plain Concrete	Hognestad 1951	Sheikh & Uzumeri 1982	Dhakar & Maekawa 2002 Yalcin & Saatcioglu 2000 Bae et al. 2005
	Popovics 1973	Park et al 1982	
		Mander et al 1988	
		Cusson & Paultre 1995	
		Razvi & Saatcioglu 1999	
		Légeron & Paultre 2003	
SFRC	Fanella & Naaman 1985		
	Soroushian & Lee 1989		
	Ezeldin & Balaguru 1992	Campione 2002	
	Nataraja et al. 1999	Aoude 2008	
	Bhargava et al. 2006	Paultre et al 2010	
	Aoude 2008		
	Ou et al 2012		

6.3.1 Models for Plain Unconfined Concrete

Several researchers have proposed models for unconfined plain concrete including Hognestad (1951), Popovics (1973), Sargin et al. (1971) , Carriera and Chu (1985), and others. Table 6.3 summarizes the equation form for some of these models. According to Lu and Zhao (2010), the Hognestad (1951) model is mainly used for normal strength concrete (NSC, where $f'_c < 60$ MPa). It consists of a second degree parabola and a linear descending branch up to strain of 0.0038, where residual stress drops to 85% of peak. For high strength concrete (HSC), models can be divided into two groups. In the first group models are based on the equation form proposed by Popovics (1973), while the other group consists of models based on the Sargin et al. (1971) model. The model by Popovics 1973 uses one equation for both ascending and descending segments.

6.3.2 Models for Unconfined Steel Fibre Reinforced Concrete

The use of SFRC in design practice requires accurate stress-strain relationships to relate material behaviour to structural response. While reliable stress-strain relationships are available for plain unconfined concrete in compression, there is debate over the reliability of SFRC models proposed in the literature. Bencardino et al. (2008) provided a detailed review of models proposed in the literature for the compressive stress-strain behaviour of unconfined SFRC. In the first phase of the study cubes and cylinders made from SFRC having fibre contents, V_f , of 1%, 1.6% and 3% were tested under compression. Various models in the literature were then used to predict the results from the test series as well as from eleven other published studies. The results showed that while many of the models showed good agreement with test results from which they were derived, there was wide scatter when the models were applied to other published test data. Table 6.6 summarizes some of the relationships for peak stress and strain from some of these models. It can be seen that the majority of the equations are empirical and account for the contribution of the fibres through the so-called "reinforcing-index", RI:

$$RI = \frac{V_f l_f}{d_f} \quad \text{Equation 6-2}$$

Where V_f , l_f and d_f , represent the fibre content, fibre length and fibre diameter, respectively. Figure 6.2 shows the stress-strain curves as predicted from the various models for a SFRC with compressive strength of 50 MPa and 1% fibre content; it is noted that the results show wide scatter.

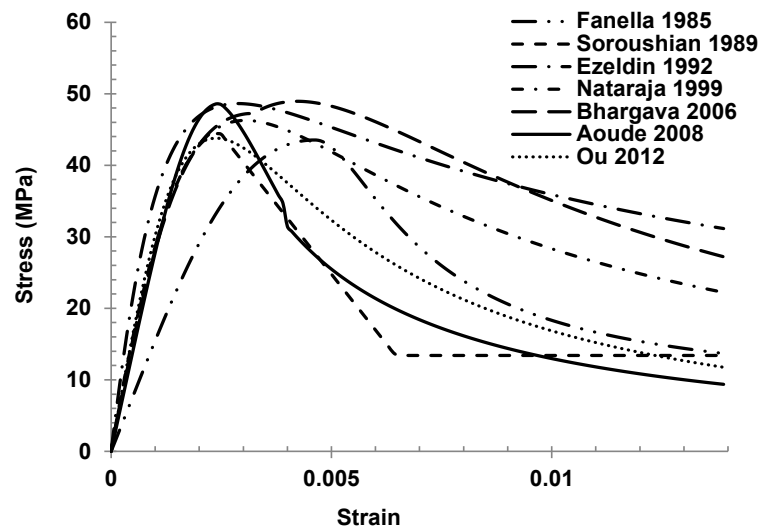


Figure 6.2 Comparison of Predicted Stress-Strain Response from Various Unconfined SFRC Models Proposed in the Literature (for $f_c = 50$ MPa, $V_f = 1\%$)

6.3.3 Models for Plain Confined Concrete

Over the past few decades, there has been extensive research on the axial load behaviour of traditional reinforced concrete columns. Research has included tests on circular, square and rectangular columns constructed with normal-strength and high-strength concrete. Based on this research data, several researchers have proposed models that can predict the complete stress-strain curve of confined concrete.

Kent and Park (1971) developed a model for concrete that is confined by rectangular transverse reinforcement. The ascending branch is a second degree parabola with a linear descending segment. The model neglects the increase in strength due to confinement but accounts for the increased ductility that results due to the presence of rectangular steel ties. This model was later modified by Park et al. (1982) to take into consideration the potential strength gains at peak resistance. Sheikh and Uzumeri (1982) proposed the concept of "effective confinement" and related the additional strength gain and ductility in reinforced concrete columns to the spacing and arrangement of transverse reinforcement. The authors of this study suggested that concrete specimens that are confined by passive pressure from rectangular tie reinforcement are confined by a pressure that is not uniformly applied throughout the volume of the concrete core, suggesting that at high strains, part of the core region becomes ineffective. The authors proposed a model in which the enhanced behaviour of columns confined by rectangular ties is related to an "effectively confined" core area determined using the tie configuration and the principle of arching action in the concrete section. Mander et al. (1988) later used the concept of arching action (based on vertical spacing and sectional arrangement of transverse steel) and proposes a coefficient for effective confinement area that has been widely adopted by researchers (see Equation 6-3). It is noted that the Mander et al. (1988) model uses one continuous relationship to predict both ascending and descending segments of the confined stress-strain curve.

$$K_e = \frac{\left(1 - \frac{\sum w_i^2}{6c_x c_y}\right) \left(1 - \frac{s'}{2c_x}\right) \left(1 - \frac{s'}{2c_y}\right)}{1 - \rho_c} \quad \text{Equation 6-3}$$

Where K_e is the confinement effectiveness coefficient and the parameters s' and c represent the clear spacing between ties and the width of the concrete core region. The quantities $\sum w_i^2$ and ρ_c represent the sum of the squares of the clear spacing between adjacent longitudinal bars and the ratio of longitudinal reinforcement in the core region, respectively.

The aforementioned models were calibrated based on extensive research on normal strength concrete (NSC) columns and thus are not applicable to high-strength concrete (HSC) columns which typically show reduced ductility. In addition, the models typically use the yield strength of the transverse ties to compute lateral confinement pressure, which can lead to over-estimation of lateral pressure (particularly in the case of HSC columns).

Based on a large database of tests on HSC columns, Cusson and Paultre (1995) developed a confinement model which not only accounts for arrangement and spacing of transverse reinforcement, but also accounts for expected stress in the transverse steel at peak stress of confined concrete. They also proposed an "Effective Confinement Index", I'_E which accounts for yield strength, configuration and spacing of transverse reinforcement:

$$I'_E = \frac{f_{le}}{f'_{cu}} \quad \text{Equation 6-4}$$

where f'_{cu} is the peak strength of unconfined concrete and f_{le} is the effective confining pressure that acts on the core, taken as (for square columns):

$$f_{le} = \frac{K_e f_{hcc} A_{sh}}{s_c} \quad \text{Equation 6-5}$$

Where A_{sh} and f_{hcc} refer to the total cross-sectional area of transverse reinforcement perpendicular to one direction, and the stress in the transverse steel reinforcement at maximum strength of confined concrete.

Saatcioglu and Razvi (1992) proposed a model with parabolic ascending and linear descending segments that can predict the stress-strain behaviour of confined concrete in NSC columns. Later, Razvi and Saatcioglu (1999) modified the ascending branch of the

stress-strain curve and proposed relationships that can predict the stress-strain behaviour of confined concrete in both NSC and HSC columns. The model is based on an "equivalent lateral pressure" concept and includes a relationship to consider expected stress in the transverse ties in HSC columns.

Légeron and Paultre (2003) modified and recalibrated the model previously proposed by Cusson and Paultre (1995) based on a large database of test results including columns having concrete strength ranging from 20-140 MPa and transverse steel strength ranging from 300-1400 MPa. One major improvement is that an explicit relationship for computing stress in transverse steel was proposed, replacing the iterative procedure in the previous model. The model is applicable to columns having both square and circular cross-section and having a wide range of concrete and transverse steel properties.

According to Lokuge et al. (2005) the various confinement models can be grouped into two categories based on the relationships used to model the stress-strain behaviour of confined concrete. The first group of models follow the same structure as the early model proposed by Kent and Park (1971). Models in this category include those proposed by Park et al. (1982), Sheikh and Uzumeri (1982), Saatcioglu and Razvi (1992) and Razvi and Saatcioglu (1999) model post-peak behaviour using a linear descending branch. The second group of models have the same mathematical curve as proposed by Popovics (1973). Models in this category include those proposed by Mander et al. (1988), Cusson and Paultre (1995) and Légeron and Paultre (2003).

Table 6.5 summarizes the relationships for the peak confined stress and strain for some of the models discussed above. Table 6.4 lists the form of the relationships used to model the ascending and descending branches of the stress-strain curves in accordance with the two groupings discussed by Lokuge et al. (2005).

6.3.4 Models for Confined SFRC

While several researchers have proposed models for the compressive stress-strain response of unconfined SFRC, only a limited number of models have been proposed to account for confinement in SFRC. These include models proposed by Campione (2002), Aoude (2008) and Paultre et al. (2010).

Campione (2002) developed an empirical model to express the stress-strain relationships of confined fibre reinforced concrete (FRC) in compression for both normal and high-strength concrete columns. The author of this study proposed that the contribution of FRC to confinement can be taken into account by replacing the parameter s' in Mander's expression for effective confinement coefficient, K_e (see Eq. 1) with a fictitious parameter s'_1 :

$$s'_1 = s' - 10 \times RI \quad \text{Equation 6-6}$$

The revised confinement coefficient is then used to modify the expressions for peak confined stress and strain in the model of Cusson and Paultre (1995) (see Table 6.8).

Based on an experimental program on HSC columns constructed with steel fibres, Paultre et al. (2010) proposed a model for predicting the stress-strain behaviour of confined high-strength SFRC. In the model, the effective confinement index and expressions for peak confined stress and strain proposed by Légeron and Paultre (2003) are modified for high-strength SFRC based on additional confinement provided by fibres. Using a similar approach, Aoude (2008) proposed a model for normal-strength SFRC which modifies the well accepted models of Cusson and Paultre (1995) and Légeron and Paultre (2003) for traditional reinforced concrete. Confinement in SFRC columns is accounted for with an additional fibre confining pressure, f_{fib} , that is function of fibre pull-out strength, which is obtained by multiplying the expected number of fibres per unit area by the pull-out strength per fibre as proposed by Foster (2001):

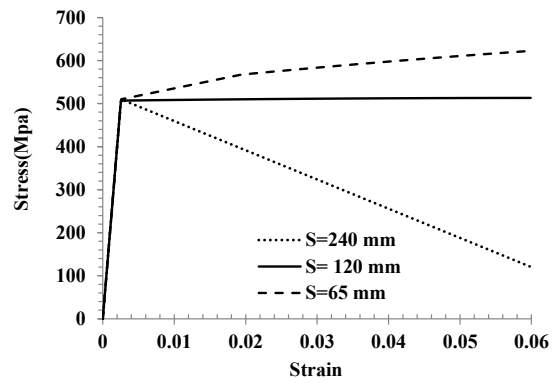
$$f_{\text{fib}} = \alpha \times RI \times [0.6 \times (f'_c)^{2/3}] \quad \text{Equation 6-7}$$

The parameter α is the fiber orientation factor (typically taken as 3/8), while $0.6 \times (f_c')^{2/3}$ is used to estimate the bond shear strength. This fibre confining pressure is scaled by a factor of 4.1 to determine the additional increase in peak confined stress in SFRC (see Table 6.8). The confining pressure and the ratio of effective to ineffective core area are then used to adjust the expressions for the descending branch of the post-peak stress-strain response.

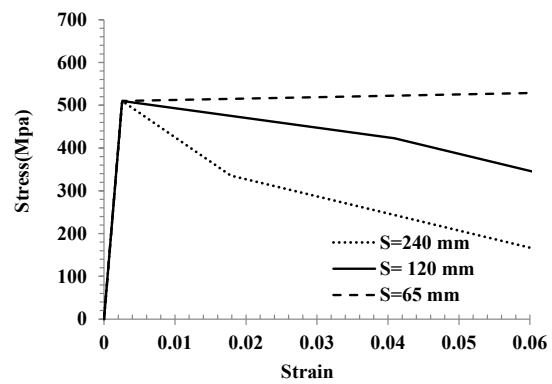
6.3.5 Longitudinal Reinforcement in Compression

In order to accurately model the axial load response of reinforced concrete columns it is important to account for buckling of longitudinal reinforcement, particularly in cases of columns with large hoop spacing. Several models for the inelastic buckling behaviour of reinforcing bars under compressive loading have been proposed in the literature. These include models proposed by Yalcin and Saatcioglu (1999), Dhakal and Maekawa (2002) and Bae et al. (2005) which were developed based on extensive experimental data which indicates that the inelastic buckling behaviour of longitudinal reinforcing bars is very sensitive to the aspect ratio (L/d_b), which is defined as the ratio of unsupported length, L , and diameter of the longitudinal reinforcement, d_b . In the model proposed by Yalcin and Saatcioglu (2000), the stress-strain response is assumed to show stable response and is not affected by buckling at aspect ratios of less than 4.5. At aspect ratios less than 8.0 the stress-strain behaviour shows limited stability while for aspect ratios greater than 8.0 the model assumes stability of the reinforcing bar is lost upon reaching yield. In the model proposed by Bae et al. (2005) effect of buckling is taken into account by modifying relationships for post-yield stress and strain based on transverse displacement caused by buckling (which is function of the L/d_b ratio). Similarly, Dhakal and Maekawa (2002) developed expressions for stress-strain response of reinforcement in compression which are function of the square root of the yield strength and aspect ratio of the longitudinal reinforcement. Figure 6.3 shows the compressive stress-strain behaviour predicted by the models proposed by Yalcin and Saatcioglu (1999), Dhakal and Maekawa (2002) and Bae et al. (2005) for reinforcement having d_b of 16 mm and L of 240, 120 and 65 mm (corresponding to the hoop spacing in the columns tested by Aoude et al. (2009)). It can be seen that all three models predict the instability in the post-yield branch of the stress-strain response in the case of bars with large unsupported length. It is noted that observations by Aoude et al. (2009) and the current

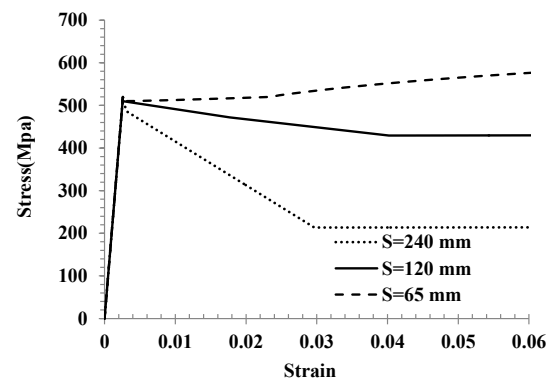
experimental program indicates that SFRC does not have a significant effect on delaying buckling of longitudinal reinforcement in columns.



(a) Yalcin and Saatcioglu (1999)



(b) Dhakal and Maekawa (2002)



(c) Bae et al. (2005)

Figure 6.3 Model Predictions for Post Peak Buckling of Longitudinal Reinforcement

6.3.6 Accounting for Cover Spalling

In RC columns, the cover is assumed to spall away abruptly at a very early strain. Foster (2001) suggested that cover spalling in RC columns is initiated due to the tri-axial stress condition that occurs between the confined core, the tie reinforcement and the cover shell which causes cracking to initiate at the core-cover interface. As lateral confinement is provided to the core, tension stresses are initiated at the core-cover interface, and the inevitable consequence of this triaxial stress state is the cover spalling mechanism. Although cover spalling cannot occur before this crack initiation, several “driving force mechanisms” (such as buckling of the longitudinal bars on the surrounding concrete) are required to cause the cover shell to buckle. However in all cases, due to the weakness of the concrete in tension after cracking, the spalling mechanism occurs rather suddenly. It is well established that the use of SFRC has an important effect on delaying cover spalling. In traditional RC columns, buckling of reinforcement may play a secondary role, but since SFRC can carry tension across the core-cover interface after the initiation of cracking, the amount of buckling that occurs in the reinforcement can have a significant effect on the acceleration of the spalling mechanism. Aoude (2008) has proposed a "spalling factor" which can be used to scale the unconfined stress-strain response of SFRC to account for spalling:

$$f_{\text{cuf}_{\text{cover}}} = f_{\text{cuf}} \times \varphi_{\text{sp}} \quad \text{Equation 6-8}$$

Where f_{cuf} is the stress at a given strain in unconfined SFRC, φ_{sp} is a spalling factor and $f_{\text{cuf}_{\text{cover}}}$ is the stress in SFRC taking into account spalling. A graphical representation of the spalling-factor is shown in Figure 6.4. (Shown are the factors corresponding to unsupported lengths of 240 mm, 120 mm, and 65 mm - corresponding to the tie spacing in the Aoude (2008) test series). As shown in the figure, the factor is taken as 1.0 up to a strain of 0.003 (when spalling is assumed to initiate) and then reduces gradually to 0.5 at a strain of 0.005. Thereafter the factor reduces due to account for effective spalling caused by buckling of the longitudinal reinforcement. It can be seen that the factor decreases rapidly when the reinforcing bars have large unsupported length, and remains stable in the case of small

unsupported length. A typical stress-strain response for unconfined SFRC with and without spalling is shown in Figure 6.5.

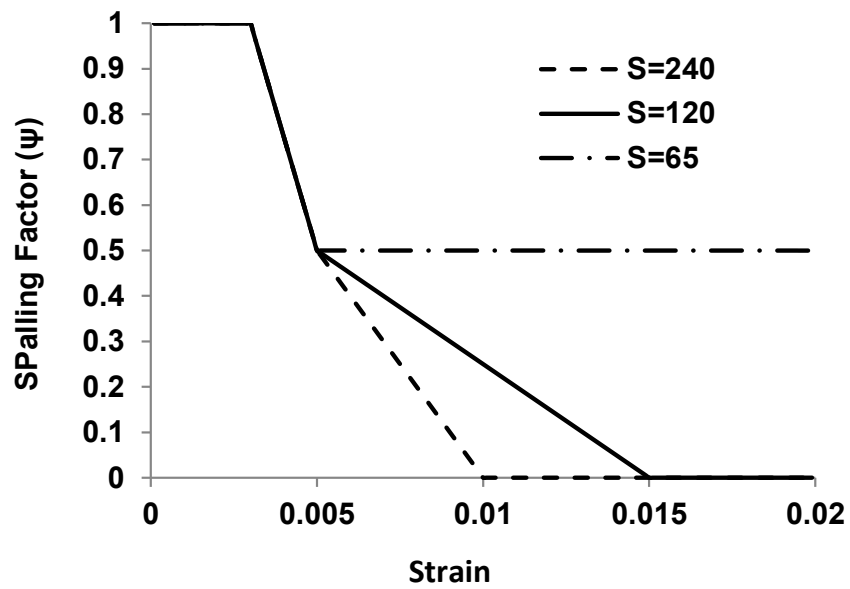


Figure 6.4 Spalling Factor Proposed by Aoude 2008

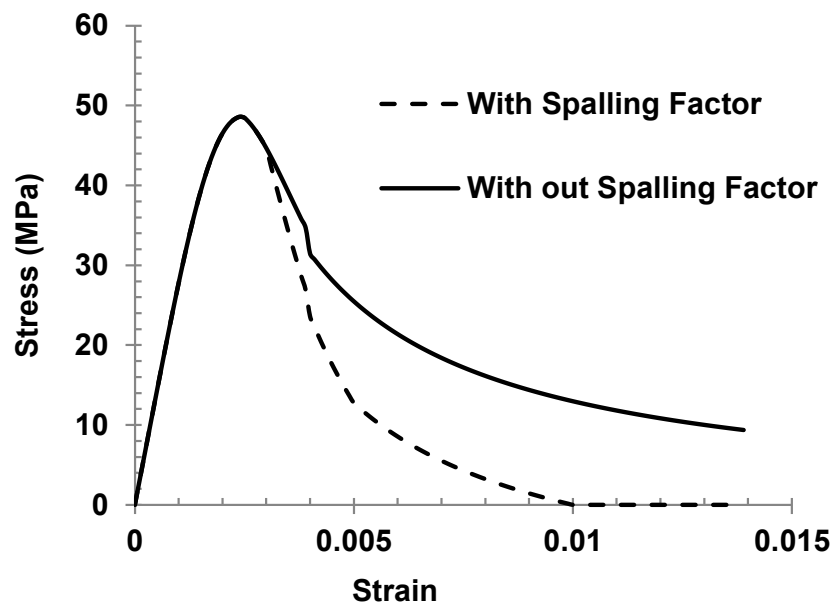


Figure 6.5 Typical Stress-Strain Curves with and without Spalling Factor

Table 6.3 Equation Forms for Unconfined Plain Concrete (Stress-Strain Curve)

	Ascending	Descending
Hognestad 1951	$Y = 2X - X^2$	$1 - 0.15 \left[\frac{X-1}{a-1} \right], a = 0.0038/\varepsilon_{co}$
Sargin et al 1971 *		$Y = \frac{[AX + (B-1)X^2]}{[1 + (A-2)X + DX^2]}$
Popovics 1973		$Y = X \left[\frac{n}{n-1+X^n} \right], n = \frac{E_{it}}{E_{it}-E_{sec}}$
Carriera and Chu 1985		$Y = X \left[\frac{\beta}{\beta-1+X^\beta} \right], \beta = \frac{1}{1-f'_c/\varepsilon_{co}E_{it}}$

$1-Y = f_c/f'_c, X = \varepsilon_c/\varepsilon_{co}$

*for coefficients, please refer to the main model reference.

Table 6.4 Equation Forms for Plain Concrete Confinement Models (Stress-Strain Curve)

		Ascending	Descending	Parameter
Group I	Kent and Park 1971	$f_{cc}[2(\varepsilon_c/0.002) - (\varepsilon_c/0.002)^2]$	$f_c [1 - Z(\varepsilon_c - 0.002)]$	$Z = \frac{0.5}{\varepsilon_{50u} + \varepsilon_{50h} - 0.002}$
	Park et al 1982	$Kf_c[2(\varepsilon_c/0.002K) - (\varepsilon_c/0.002K)^2]$	$Kf_c[1 - Z(\varepsilon_c - 0.002K)]$	$Z = \frac{0.5}{\left(\frac{3 + 0.29f'_c}{145f'_c - 1000}\right) + \frac{3}{4}\rho_{sh}\sqrt{\frac{b_c}{s}} - \varepsilon_{cc}}$
	Sheikh and Uzumeri 1982	$Kf_c[2(\varepsilon_c/\varepsilon_{cc}) - (\varepsilon_c/\varepsilon_{cc})^2]$	$f_{cc}[1 - Z(\varepsilon_c - \varepsilon_{cc})]$	$Z = \left[\frac{0.5}{0.75\rho_{sh}\sqrt{\frac{B}{s}}} \right]$
	Saatcioglu and Razvi 1992	$f_{cc}[2(\varepsilon_c/0.002) - (\varepsilon_c/0.002)^2]^{1/(1+2K)}$	$f_{cc}[1 - Z(\varepsilon_c - \varepsilon_{cc})]$	$Z = \frac{0.15f_{cc}}{\varepsilon_{85} - \varepsilon_{cc}}$
	Razvi and Saatcioglu 1999	$f_{cc}\{[r(\varepsilon_c/\varepsilon_{cc})]/[r - 1 - (\varepsilon_c/\varepsilon_{cc})^r]\}$	$f_{cc}[1 - Z(\varepsilon_c - \varepsilon_{cc})]$	
Group II	Mander et al 1988	$f_{cc}\{[r(\varepsilon_c/\varepsilon_{cc})]/[r - 1 - (\varepsilon_c/\varepsilon_{cc})^r]\}$		$r = \frac{5000\sqrt{f'_c}}{5000\sqrt{f'_c} - E_{sec}}$
	Cusson and Paultre 1995	$f_{cc}\{[r(\varepsilon_c/\varepsilon_{cc})]/[r - 1 - (\varepsilon_c/\varepsilon_{cc})^r]\}$	$f_{cc}e^{-k_1(\varepsilon_c - \varepsilon_{cc})^{k_2}}$	$r = \frac{E_{ct}}{E_{ct} - E_{sec}}$
	Legeron and Paultre 2003	$f_{cc}\{[r(\varepsilon_c/\varepsilon_{cc})]/[r - 1 - (\varepsilon_c/\varepsilon_{cc})^r]\}$	$f_{cc}e^{-k_1(\varepsilon_c - \varepsilon_{cc})^{k_2}}$	

Table 6.5 Relationships for Peak Confined Stress and Strain in RC Confinement Models

	f_{cc}/f_{co}	$\varepsilon_{cc}/\varepsilon_{co}$
Sheikh & Uzumeri 1982	$1 + (b_c^2/140P_{OCC})[(1 - nC^2/5.5b_c^2)(1 - s/2b_c)^2]$	$1 + 248/C [1 - 5.0(s/b_c)^2](\rho_{sh} f'_s/\sqrt{f_{co}})$
Park et al. 1982	$(1/0.85)(1 + \rho_{sh} f_{yh}/f'_c)$	$(1/0.85)(1 + \rho_{sh} f_{yh}/f'_c)$
Mander et al. 1988	$-1.254 + 2.254 \sqrt{1 + 7.94 f'_l/f_{co} - 2 f'_l/f_{co}}$	$1 + 5[f_{cc}/f_{co} - 1]$
Cusson & Paultre 1995	$1 + 2.1 (f_{le}/f_{co})^{0.7}$	$1 + (0.21/\varepsilon_{co})(f_{le}/f_{co})^{1.7}$
Razvi & Saatcioglu 1999	$1 + k_1 f_{le}/f_{co}$	$1 + 5k_1 f_{le}/f_{co}$
Légeron & Paultre 2003	$1 + 2.4 (f_{le}/f_{co})^{0.7}$	$1 + 35(f_{le}/f_{co})^{1.2}$

Table 6.6 Relationships for Peak Unconfined Stress and Strain in SFRC Models

	f_{cf}/f_{co}	$\varepsilon_{cf}/\varepsilon_{co}$
Fanella & Naaman 1985	$1 + (275/f_{co})RI \text{ (psi)}$	$(1/\varepsilon_{co})(0.00079RI + 0.0041 f_{cf}/f_{co}) \text{ (psi)}$
Soroushian & Lee 1989	$1 + (3.6/f_{co})RI$	$(1/\varepsilon_{co})(0.0007RI + 0.0021)$
Ezeldin & Balaguru 1992	$1 + (3.51/f_{co})(RI)$	$1 + 446 \times 10^{-6}(RI/\varepsilon_{co})$
Nataraja et al. 1999	$1 + (2.1604/f_{co})(RI)$	$1 + 0.0006(RI/\varepsilon_{co})$
Mansur et al. 1999 *	1.05	$(0.0005 + 0.00000072(RI))(1/\varepsilon_{co})(f_0)^{0.35}$
Bhargava & Sharma 2006	$1 + (0.45/f_{co}) + (8.89/f_{co})(RI)_s + (2.47/f_{co}) (RI)_l$	$1 - 0.00026/\varepsilon_{co} + 0.001214(RI/\varepsilon_{co})_s + 0.00086(RI/\varepsilon_{co})_l$
Aoude 2008	1.0	$0.001684/\varepsilon_{co} + 0.000016(f'_{cf}/\varepsilon_{co})$
Ou et al. 2012	$1 + (2.35/f_{co})(RI_v) \text{ Mpa}$	$1 + 0.0007(RI/\varepsilon_{co})$

*-different values depend on horizontal or vertically casted members

Table 6.7 Equation Forms for Unconfined SFRC (Stress-Strain Curve)

	Ascending	Descending
Fanella & Naaman 1985	$\frac{\sigma}{f_{cf}} = \frac{\left[A \left(\frac{\varepsilon}{\varepsilon_{cf}} \right) + (B - 1) \left(\frac{\varepsilon}{\varepsilon_{cf}} \right)^2 \right]}{\left[1 + C \left(\frac{\varepsilon}{\varepsilon_{cf}} \right) + D \left(\frac{\varepsilon}{\varepsilon_{cf}} \right)^2 \right]}$	$\frac{\sigma}{f_{cf}} = \frac{\left[A' \left(\frac{\varepsilon}{\varepsilon_{cf}} \right) + (B' - 1) \left(\frac{\varepsilon}{\varepsilon_{cf}} \right)^2 \right]}{\left[1 + C' \left(\frac{\varepsilon}{\varepsilon_{cf}} \right) + D' \left(\frac{\varepsilon}{\varepsilon_{cf}} \right)^2 \right]}$
Soroushian & Lee 1989	$\frac{\sigma}{f_{cf}} = 2 \left(\frac{\varepsilon}{\varepsilon_{cf}} \right) - \left(\frac{\varepsilon}{\varepsilon_{cf}} \right)^2$	$\sigma = z(\varepsilon - \varepsilon_{cf}) + f_{cf}$ $\geq 0.12 f_{cf} + 14.8 RI$
Ezeldin & Balaguru 1992	$\frac{\sigma}{f_{cf}} = \frac{\beta \left(\frac{\varepsilon}{\varepsilon_{cf}} \right)}{\beta - 1 + \left(\frac{\varepsilon}{\varepsilon_{cf}} \right)^\beta}, \beta = 1.093 + 0.7132 RI^{-0.926}$	
Nataraja et al. 1999	$\frac{\sigma}{f_{cf}} = \frac{\beta \left(\frac{\varepsilon}{\varepsilon_{cf}} \right)}{\beta - 1 + \left(\frac{\varepsilon}{\varepsilon_{cf}} \right)^\beta}, \beta = 0.5811 + 1.93 RI^{-0.7406},$	
Mansur et al. 1999 ^{a,b}	$\frac{\sigma}{f_{cf}} = \frac{K_1 \beta \left(\frac{\varepsilon}{\varepsilon_{cf}} \right)}{K_1 \beta - 1 + \left(\frac{\varepsilon}{\varepsilon_{cf}} \right)^{K_2 \beta}}, \beta = \frac{E_{it}}{E_{it} - E_{sec}}$	
Bhargava & Sharma 2006 ^a	$\frac{\sigma}{f_{cf}} = \frac{K_1 \beta \left(\frac{\varepsilon}{\varepsilon_{cf}} \right)}{K_1 \beta - 1 + \left(\frac{\varepsilon}{\varepsilon_{cf}} \right)^{K_2 \beta}}, \beta = \left[\frac{f_{cf}}{A} \right]^3 + C$	
Aoude 2008 ^a	$\frac{\sigma}{f_{cf}} = \frac{K_1 \beta \left(\frac{\varepsilon}{\varepsilon_{cf}} \right)}{K_1 \beta - 1 + \left(\frac{\varepsilon}{\varepsilon_{cf}} \right)^{K_2 \beta}}, \beta = \frac{E_c}{E_c - E_{sec}},$	
Ou et al. 2012	$\frac{\sigma}{f_{cf}} = \frac{\beta \left(\frac{\varepsilon}{\varepsilon_{cf}} \right)}{\beta - 1 + \left(\frac{\varepsilon}{\varepsilon_{cf}} \right)^\beta}, \beta = 0.71 (RI)^2 - 2(RI.) + 3.05$	

a: K_1 and K_2 varies for ascending and descending as a function of f_{cf} , and R.I.

b: K_1 and K_2 depend on direction of casting

Table 6.8 Relationships for Peak Confined Stress and Strain in SFRC Models

Authors	f_{ccf}/f_{co}	$\varepsilon_{ccf}/\varepsilon_{co}$
Campione 2002	$1 + 4.1 (K_e' f_l/f_{co})$	$1 + 20.5(K_e' f_l/f_{co})^{0.7}$
Aoude 2008	$1 + 2.4 (f_{le}/f_{co})^{0.7} + 4.1 (f_{fib}/f_{co})$	$1 + 0.21/\varepsilon_{co} (f_{le}/f_{co})^{1.7}$
Paultre et al 2010	$1 + 2.4 (f_{le}/f_{co} + f_{fib}/f_{co})^{0.7}$	$1 + 0.21/\varepsilon_{co} (f_{le}/f_{co} + f_{fib}/f_{co})^{1.7}$

Table 6.9 Equation Forms for SFRC Confinement Models (Stress-Strain Curve)

Authors	Ascending	Descending
Campione 2002 ^a	$\frac{\sigma}{f_{ccf}} = \frac{\beta \left(\frac{\varepsilon}{\varepsilon_{ccf}} \right)}{\beta - 1 + \left(\frac{\varepsilon}{\varepsilon_{ccf}} \right)^\beta}, \beta = 0.5811 + 1.93 R.I.^{-0.7406}$	
Aoude 2008	$\frac{\sigma}{f_{ccf}} = \frac{\beta \left(\frac{\varepsilon}{\varepsilon_{ccf}} \right)}{\beta - 1 + \left(\frac{\varepsilon}{\varepsilon_{ccf}} \right)^\beta}, \beta = \frac{E_c}{E_c - E_{sec}},$	$f_{ccf} e^{-k_1(\varepsilon_c - \varepsilon_{ccf})^{k_2}}$
Paultre et al 2010	$\frac{\sigma}{f_{ccf}} = \frac{\beta \left(\frac{\varepsilon}{\varepsilon_{ccf}} \right)}{\beta - 1 + \left(\frac{\varepsilon}{\varepsilon_{ccf}} \right)^\beta}, \beta = \frac{E_{ct}}{E_{ct} - E_{sec}},$	$f_{ccf} e^{k_1(\varepsilon_c - \varepsilon_{ccf})^{k_2}}$

a: the model varies for NSC and HSC , here the equation form for NSC is shown.

6.4 Analytical Comparisons

6.4.1 Overview

In this section the experimental load-strain curves for the SCC, SFRC and UHPFRC columns are predicted using various models proposed in the literature. Table 6.10 lists the models considered for each column series.

Table 6.10 Constitutive Models Considered for Each Column Series

	Unconfined Concrete	Confined Concrete	Longitudinal Bars in Compression
SCC	Popovics 1973	Sheikh & Uzumeri 1982	Dhakal & Maekawa 2002
		Park et al 1982	
		Mander et al 1988	
		Cusson & Paultre 1995	
		Razvi & Saatcioglu 1999	
		Légeron & Paultre 2003	
SFRC	Aoude 2008	Campione 2002	Dhakal & Maekawa 2002
		Aoude 2008	
UHPFRC	Popovics 1973	Razvi & Saatcioglu 1999	Dhakal & Maekawa 2002
		Légeron & Paultre 2003	
		Paultre et al 2010	

In addition to comparing the analytical and experimental load-strain responses of the columns, several experimental parameters presented in Figure 6.6 and Figure 6.7 are also predicted analytically. These parameters include ultimate column capacity (P_{max}) and associated strain (ε_{max}); area under the curves (A_u) up to strain of 0.01; the slope of a line between peak strain and strain equal to $\varepsilon = 0.004$ (K_{sd}) which gives an indication of the rate of cover spalling (only considered for columns with 30 mm cover); and the slope of a line between strain values of $\varepsilon = 0.004$ and $\varepsilon = 0.01$ (K_{pd}), which shows the approximate rate of post peak failure. Researchers have proposed different methods to quantify post-peak ductility in columns, including the rate of strength decay to 85% and 50% P_{max} (Razvi &

Saatcioglu 1999; Cusson & Paultre 1995). For the columns tested in this research post-peak ductility was evaluated by comparing experimental and analytical strength at a constant strain of 0.01 ($P_{\varepsilon=0.01}$) for the SCC/SFRC columns, and constant strain of 0.02 ($P_{\varepsilon=0.02}$) for the UHPFRC columns.

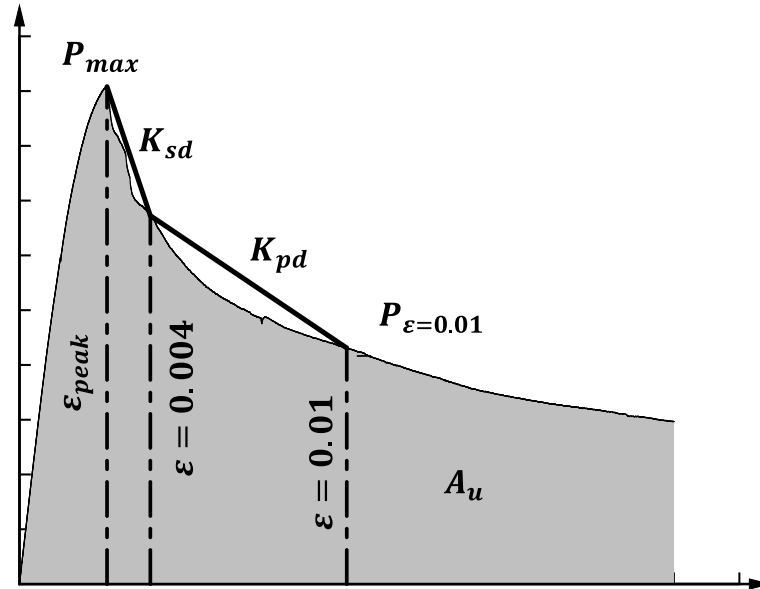


Figure 6.6 Parameters for Evaluation of SCC & SFRC Models

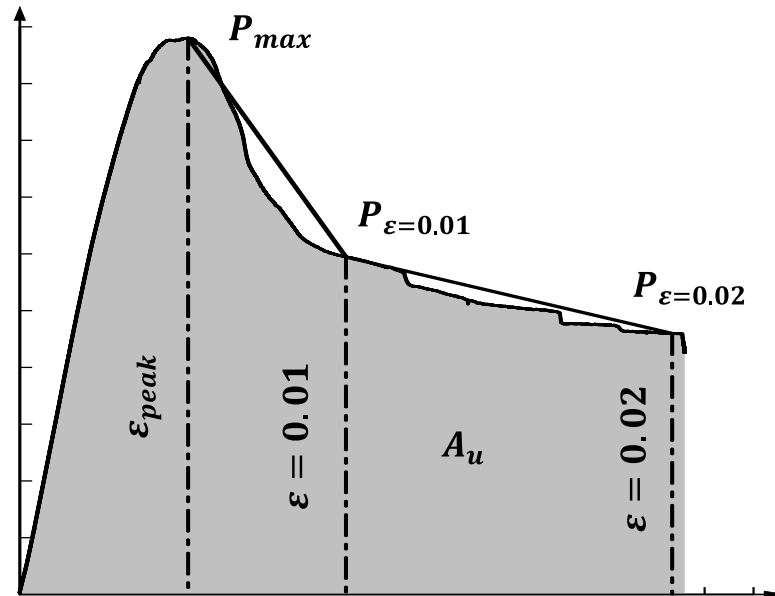


Figure 6.7 Parameters for Evaluation of UHPFRC Models

6.4.2 Analysis - SCC Columns (0% Fibres)

Over the years several researchers have proposed confinement models that have been calibrated based on experimental tests on columns constructed with traditional concrete. There is a need to verify the suitability of these models for SCC columns. In this section the experimental load-strain curves for the SCC columns from this study and from the Aoude (2008) study are predicted using various confinement models proposed in the literature. Confinement was taken into account using the models proposed by Sheik and Uzumeri (1982), Kent and Park (1982), Mander et al. (1988), Cusson and Paultre (1995), Razvi and Saatcioglu (1999) and Légeron and Paultre (2003). Cover response was modeled using the unconfined concrete model proposed by Popovics (1973). Spalling was assumed to initiate at a strain of 0.003 and a complete loss of cover was assumed at a strain of 0.004. The response of the longitudinal steel in compression was taken into account using the model proposed by Dhakal and Maekawa (2002).

6.4.2.1 Square-section Columns (Aoude 2008)

For the square-section series, columns A0, B0 and C0 which were constructed with plain SCC and had detailing corresponding to low, moderate and high confinement, are included in the analysis. The columns had a 300 x 300 mm cross-section, a 30 mm cover and tie spacing of 240, 120 and 65 mm, respectively. Also included are companion columns A0nc, B0nc and C0nc which had exact details but were constructed without cover.

Figure 6.8 to Figure 6.13 present a comparison of the experimental versus analytical load-strain curves. In general, the results demonstrate that the RC confinement models can be used to predict the axial load response of SCC columns having square cross-section. The best results are obtained using the Légeron and Paultre (2003) and Cusson and Paultre (1995) confinement models. Reliable results are also obtained using the Razvi and Saatcioglu (1999) model, while the remaining models provide results with reduced reliability.

Table 6.11 presents the analytical-to-experimental ratios for parameters P_{max} , ε_{max} , A_u , K_{sd} , K_{pd} and $P_{\varepsilon=0.01}$. Furthermore the average and standard deviation values are also reported in Table 6.12.

By considering average values of P_{max} , it is evident that the models generally under-predict peak capacity in SCC columns regardless of the confinement model and some scatter is seen for most of the models. By comparing the ratios of P_{max} , under-predictions range from 1% to 15 % in group I, and 0%~4% in group II. By considering standard deviation, a minimum of 0.05 is found for the Razvi and Saatcioglu (1999) model in group I, while the lowest standard deviation of 0.07 is found for the Cusson and Paultre (1995) model in group II.

Comparing the predictions for ε_{max} , a range of 16% under prediction (Park et al, 1982) to 11 % over prediction (Razvi and Saatcioglu 1999) in group I, while only over predictions are observed in Group II (15 % for Mander et al 1988). The lowest standard deviations in group I and II are found for the Park et al (1982) and Mander et al (1988) models, respectively.

For the ratios of A_u and $P_{\varepsilon=0.01}$, the best standard deviation was found for the Légeron and Paultre (2003) model with values of 0.09 and 0.1 while average over predictions of 12 % and 19% are observed.

Finally, the ratios for the slopes K_{sd} and K_{pd} show wide scatter for the various models. K_{sd} is predicted with low standard deviation of 0.04 for Légeron and Paultre (2003) but with a 29 % average under prediction. For K_{pd} , models in group I generally show smaller standard deviations. It is noted that the negative sign seen in some of the prediction ratios can be explained by the fact that in some columns softening is observed in the experimental load-strain curve, whereas the models predict significant strain hardening (e.g, C0 and C0nc).

6.4.2.2 Rectangular-Section Columns

For the rectangular-section series, the SCC columns tested in the current study are included in the analysis, including columns AR0, BR0 and CR0 which had low, moderate and high confinement, and columns A0nc, B0nc and C0nc which did not have cover.

Figure 6.14 to Figure 6.19 present a comparison of the experimental versus analytical load-strain curves. In general, the results demonstrate that the RC confinement models can be used to predict the axial load response of SCC columns having rectangular cross-section. As with the square-section columns, the best results are obtained using the Légeron and Paultre

(2003), Cusson and Paultre (1995) and Razvi and Saatcioglu (1999) confinement models, while reduced reliability is obtained using the remaining confinement models.

Table 6.13 presents the analytical-to-experimental ratios for parameters P_{max} , ε_{max} , A_u , K_{sd} , K_{pd} and $P_{\varepsilon=0.01}$. Furthermore the average and standard deviation values are also reported in Table 6.14.

By Comparing the ratios of P_{max} , except for Sheikh and Uzumeri (1982) with average under prediction of 10 % (Standard deviation = 0.04), other models generally show average over prediction. The best results are observed for the Cusson and Paultre (1995) and Legeron and Paultre (2003) models.

Comparing the ratios of ε_{max} in group I, predictions vary by model (average values range from -14% for the Park et al 1982 model up to +39 % for the Razvi and Saatcioglu 1999 model) with the best results in terms of average predictions and standard deviations for the Mander et al. (1988) model. In group II, only average over prediction is observed and the strains are generally poorly predicted.

For the ratios of A_u and $P_{\varepsilon=0.01}$, the best standard deviation was found for Legeron and Paultre (2003) model with values of 0.07 and 0.14, although 21 % and 39 % over prediction is observed when considering average values.

Finally, the ratios for the slopes K_{sd} and K_{pd} show wide scatter for the various models. Values of K_{sd} are found to be much more under predicted in AR0 and over predicted in CR0 for most models. As previously discussed, the negative K_{pd} ratios for columns C0 and C0nc can be explained by the fact that post-peak softening was observed experimentally, whereas analytical models predict strain hardening for these columns.

Table 6.11 Ratio of Analytical /Experimental Results – 0% Square Sections

	Group I				Group II		
	#	Shk82	Park82	Raz99	Man88	Cus95	Leg03
P_{max}	A0	0.97	1.06	0.99	1.06	1.06	1.06
	B0	0.88	0.97	0.95	0.97	0.98	0.97
	C0	0.83	1.05	0.95	1.00	0.90	0.93
	A0nc	0.88	0.94	0.85	0.94	0.92	0.93
	B0nc	0.79	0.88	0.87	0.90	0.88	0.88
	C0nc	0.76	1.01	0.94	1.15	1.03	1.07
ϵ_{peak}	A0	1.08	0.88	1.00	1.00	0.92	0.96
	B0	0.96	0.85	0.96	0.96	0.93	0.93
	C0	0.87	0.87	0.97	0.97	0.97	0.97
	A0nc	1.16	0.96	1.04	0.96	0.88	0.96
	B0nc	1.07	0.89	1.07	1.07	0.89	1.00
	C0nc	0.59	0.57	1.63	1.92	1.96	1.98
A_u	A0	1.55	1.49	1.64	1.48	1.22	1.24
	B0	1.14	1.23	1.30	1.35	0.98	1.12
	C0	0.77	1.01	1.03	1.13	1.01	1.05
	A0nc	1.43	1.44	1.60	1.51	1.16	1.20
	B0nc	1.09	1.21	1.28	1.35	0.95	1.11
	C0nc	0.72	0.94	0.88	1.07	0.94	0.99
$P_{\epsilon=0.01}$	A0	2.00	1.63	2.29	1.67	1.30	1.16
	B0	1.33	1.39	1.53	1.58	0.93	1.18
	C0	0.74	0.97	1.06	1.20	1.07	1.11
	A0nc	1.74	1.65	2.32	1.87	1.33	1.22
	B0nc	1.42	1.55	1.74	1.83	1.08	1.36
	C0nc	0.74	0.95	0.96	1.18	1.06	1.10
K_{sd}	A0	0.43	0.53	0.54	0.63	0.82	0.74
	B0	0.58	0.57	0.58	0.54	0.87	0.67
	C0	0.85	0.93	0.74	0.69	0.74	0.71
	A0nc	---	---	---	---	---	---
	B0nc	---	---	---	---	---	---
	C0nc	---	---	---	---	---	---
K_{pd}	A0	1.58	1.69	0.87	1.65	1.15	1.75
	B0	0.43	0.54	0.41	0.47	1.00	0.92
	C0	0.55	2.66	-5.00	-9.88	-9.79	-10.42
	A0nc	2.59	2.52	1.24	2.28	1.70	2.55
	B0nc	0.35	0.40	0.27	0.26	0.71	0.62
	C0nc	1.02	2.86	-4.96	-8.76	-8.33	-9.33

Table 6.12 Average Ratio of Analytical /Experimental Results – 0% Square

Sections

	Group I			Group II		
	Shk82	Park82	Raz99	Man88	Cus95	Leg03
P_{max}	0.85	0.99	0.93	1.00	0.96	0.97
[St.d.]	[0.07]	[0.07]	[0.05]	[0.09]	[0.07]	[0.08]
ϵ_{peak}	0.96	0.84	1.11	1.15	1.09	1.13
[St.d.]	[0.21]	[0.14]	[0.26]	[0.38]	[0.43]	[0.42]
A_u	1.12	1.22	1.29	1.31	1.05	1.12
[St.d.]	[0.34]	[0.22]	[0.30]	[0.18]	[0.12]	[0.09]
$P_{\epsilon=0.01}$	1.33	1.36	1.65	1.55	1.13	1.19
[St.d.]	[0.51]	[0.32]	[0.58]	[0.30]	[0.16]	[0.10]
K_{sd}	0.90	0.95	0.77	0.94	0.75	0.83
[St.d.]	[0.43]	[0.49]	[0.26]	[0.37]	[0.29]	[0.27]
K_{pd}	1.09	1.78	-1.20	-2.33	-2.26	-2.32
[St.d.]	[0.87]	[1.09]	[2.95]	[5.48]	[5.30]	[5.90]

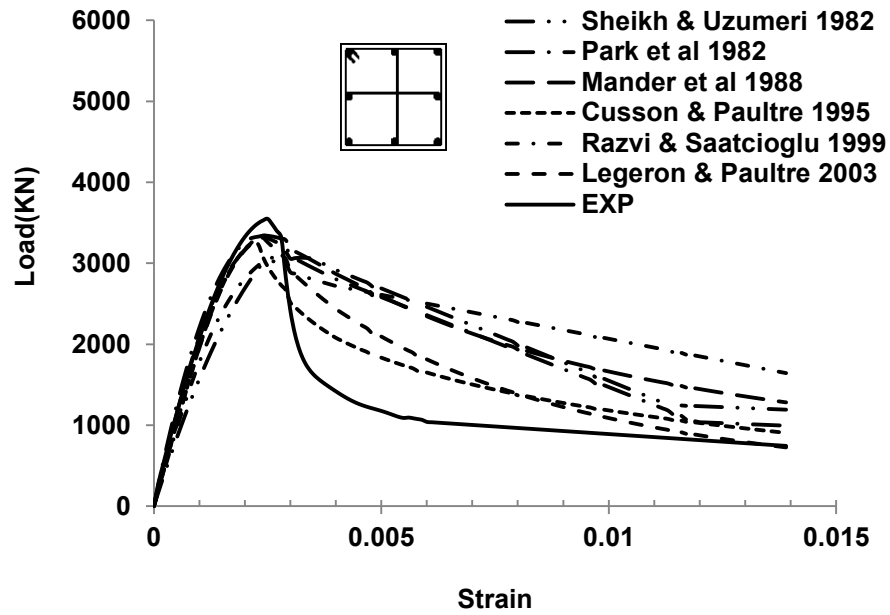


Figure 6.8 Comparison of Existing Models with Experimental Results – A0nc

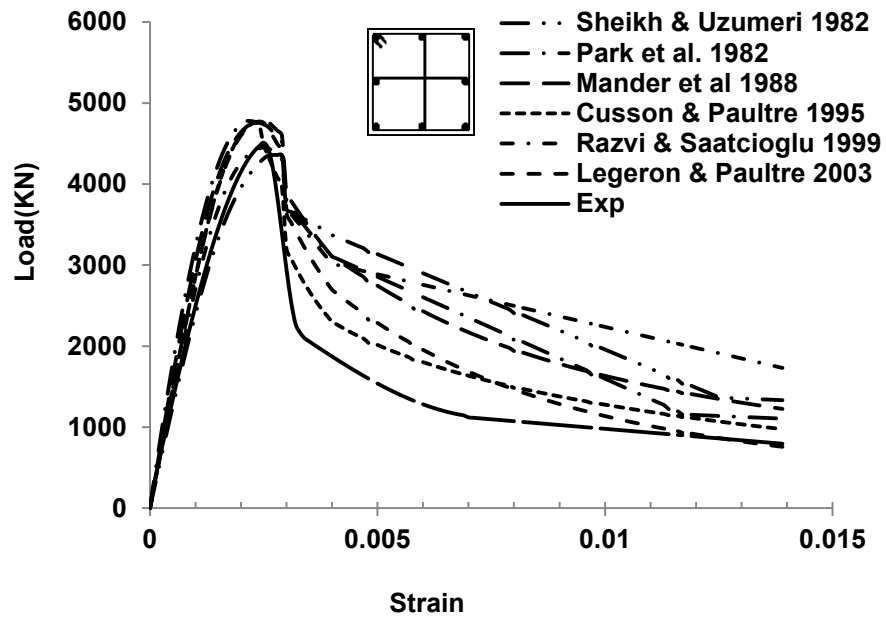


Figure 6.9 Comparison of Existing Models with Experimental Results – A0

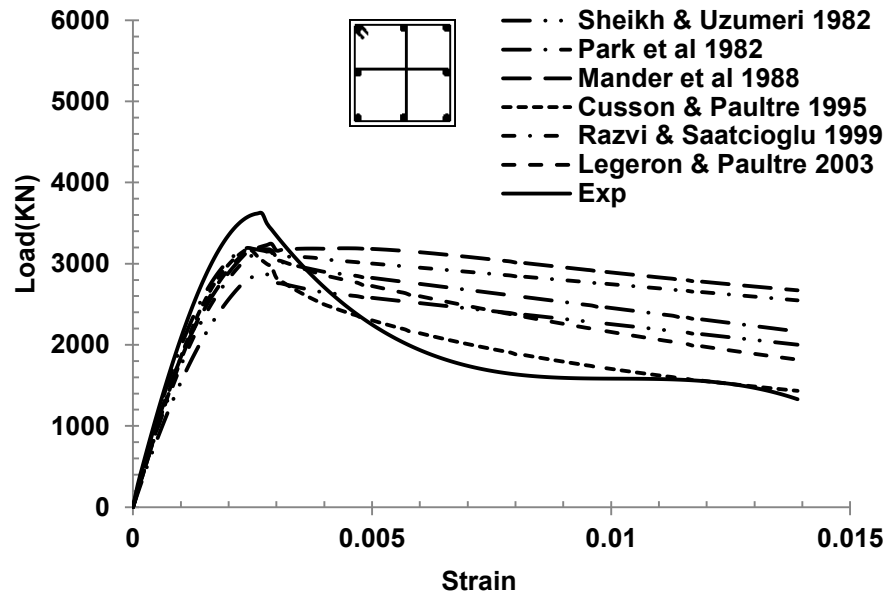


Figure 6.10 Comparison of Existing Models with Experimental Results – B0nc

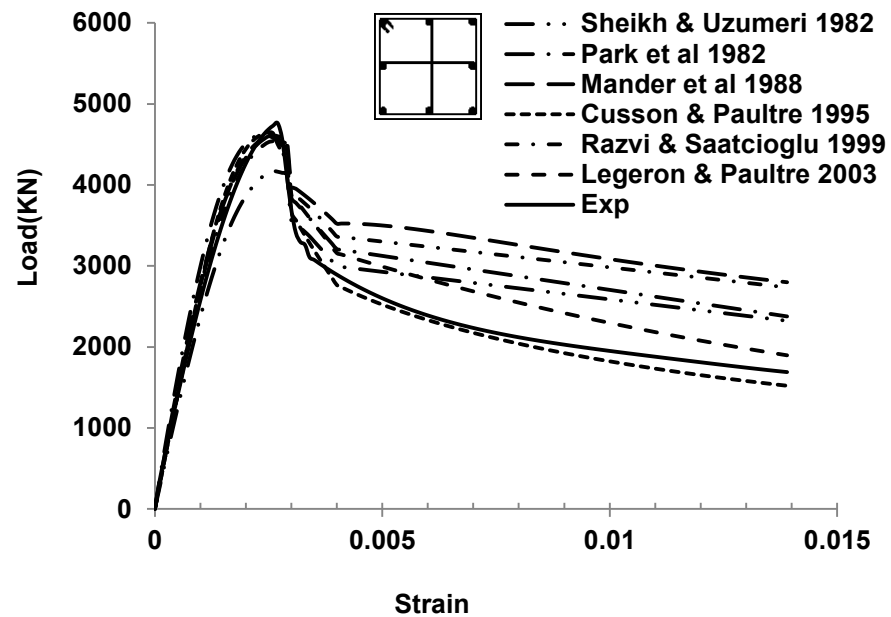


Figure 6.11 Comparison of Existing Models with Experimental Results – B0

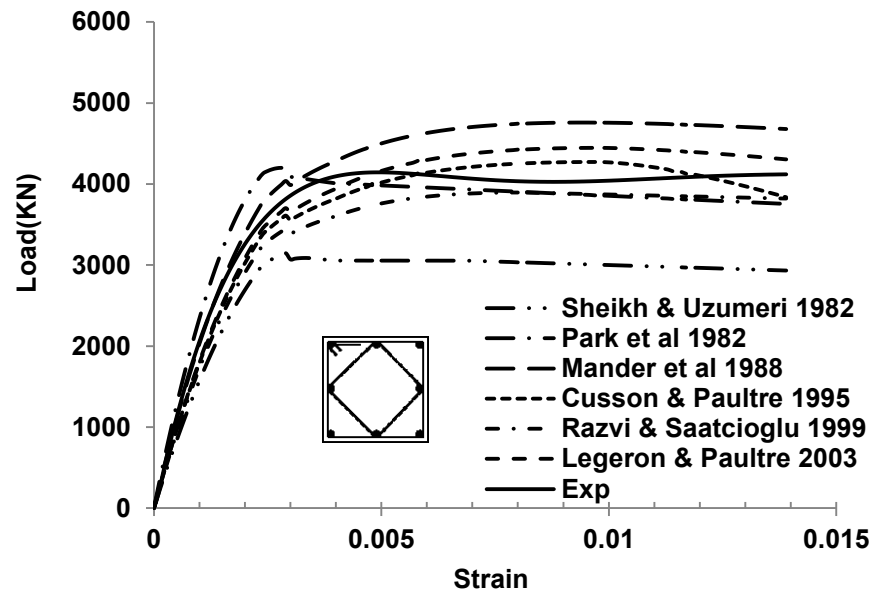


Figure 6.12 Comparison of Existing Models with Experimental Results – C0nc

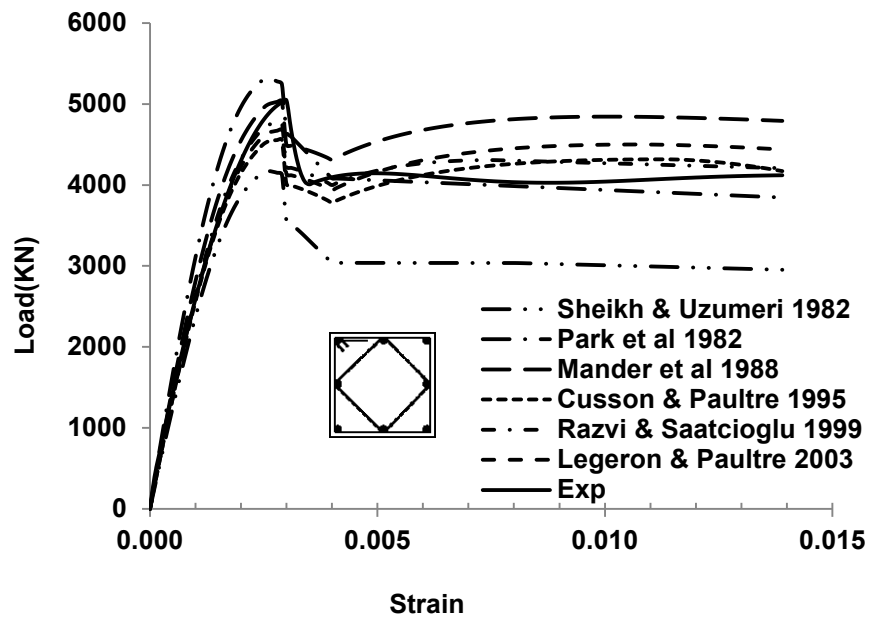


Figure 6.13 Comparison of Existing Models with Experimental Results – C0

Table 6.13 Ratio of Analytical /Experimental Results – 0% Rectangular Sections

	Group I				Group II		
	#	Shk82	Park82	Raz99	Man88	Cus95	Leg03
P_{max}	AR0	0.95	1.03	1.02	1.01	1.00	1.00
	BR0	0.92	1.05	1.01	1.02	1.04	1.02
	CR0	0.92	1.16	1.06	1.10	1.02	1.03
	AR0nc	0.88	1.00	0.98	0.96	0.95	0.95
	BR0nc	0.92	1.13	1.10	1.14	1.08	1.07
	CR0nc	0.83	1.17	1.25	1.32	1.17	1.22
ε_{peak}	AR0	1.00	0.88	0.96	0.96	0.85	0.88
	BR0	1.00	0.92	1.00	1.00	1.00	1.00
	CR0	1.00	0.96	1.04	1.04	1.00	1.04
	AR0nc	0.76	0.61	0.76	0.66	0.61	0.61
	BR0nc	1.07	0.93	1.85	2.11	1.04	1.07
	CR0nc	0.85	0.85	2.74	3.18	3.85	3.44
A_u	AR0	1.46	1.57	1.60	1.51	1.24	1.34
	BR0	1.09	1.25	1.31	1.33	1.00	1.15
	CR0	0.92	1.17	1.24	1.31	1.16	1.22
	AR0nc	1.48	1.59	1.75	1.50	1.10	1.18
	BR0nc	1.13	1.36	1.42	1.48	1.07	1.24
	CR0nc	0.82	1.13	1.20	1.27	1.11	1.15
$P_{\varepsilon=0.01}$	AR0						
	BR0	1.26	1.41	1.55	1.57	0.96	1.24
	CR0	0.94	1.18	1.34	1.43	1.27	1.34
	AR0nc	2.62	2.66	3.21	2.39	1.54	1.60
	BR0nc	1.40	1.64	1.81	1.89	1.13	1.46
	CR0nc	0.86	1.18	1.37	1.45	1.27	1.32
K_{sd}	AR0	0.42	0.42	0.42	0.47	0.58	0.52
	BR0	1.03	1.04	0.89	0.89	1.31	1.02
	CR0	1.24	1.39	0.93	0.95	0.92	0.95
	AR0nc	---	---	---	---	---	---
	BR0nc	---	---	---	---	---	---
	CR0nc	---	---	---	---	---	---
K_{pd}	AR0	0.55	0.64	0.43	0.79	0.64	0.93
	BR0	0.19	0.26	0.15	0.17	0.89	0.63
	CR0	0.08	0.31	-0.66	-0.97	-0.68	-0.97
	AR0nc	0.32	0.36	0.25	0.43	0.36	0.51
	BR0nc	0.25	0.33	0.14	0.09	0.94	0.62
	CR0nc	0.11	0.56	-2.14	-2.44	-2.43	-2.51

Table 6.14 Average Ratio of Analytical /Experimental Results – 0% Rectangular Sections

	Group I			Group II		
	Shk82	Park82	Raz99	Man88	Cus95	Leg03
P_{max}	0.90	1.09	1.07	1.09	1.04	1.05
[St.d.]	[0.04]	[0.07]	[0.10]	[0.13]	[0.07]	[0.09]
ϵ_{peak}	0.95	0.86	1.39	1.49	1.39	1.34
[St.d.]	[0.12]	[0.13]	[0.76]	[0.96]	[1.22]	[1.04]
A_u	1.15	1.35	1.42	1.40	1.11	1.21
[St.d.]	[0.27]	[0.20]	[0.22]	[0.11]	[0.08]	[0.07]
$P_{\epsilon=0.01}$	1.42	1.62	1.86	1.75	1.23	1.39
[St.d.]	[0.71]	[0.61]	[0.78]	[0.40]	[0.21]	[0.14]
K_{sd}	0.74	0.92	0.21	0.25	0.68	0.43
[St.d.]	[0.46]	[0.58]	[0.74]	[0.70]	[0.74]	[0.67]
K_{pd}	0.25	0.41	-0.31	-0.32	-0.05	-0.13
[St.d.]	[0.17]	[0.15]	[0.98]	[1.19]	[1.31]	[1.34]

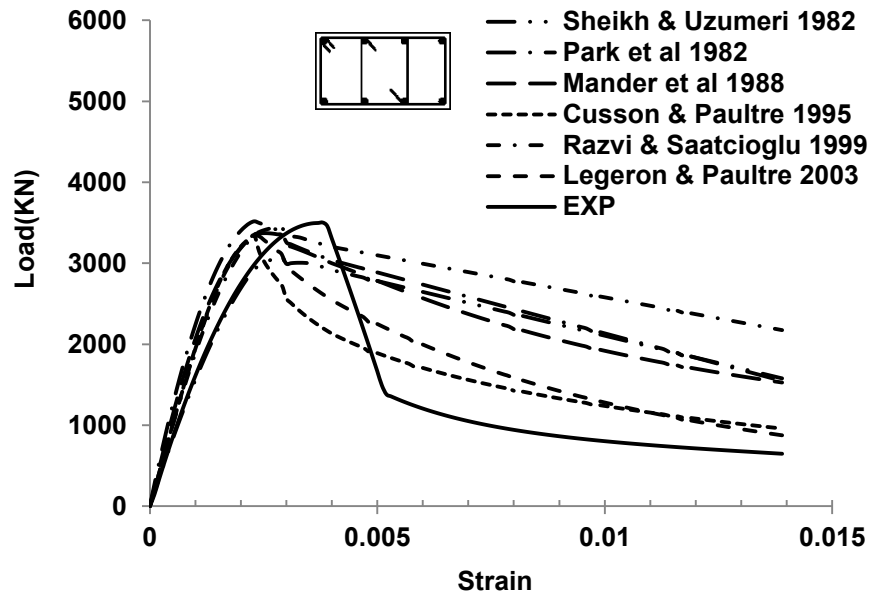


Figure 6.14 Comparison of Existing Models with Experimental Results – AR0nc

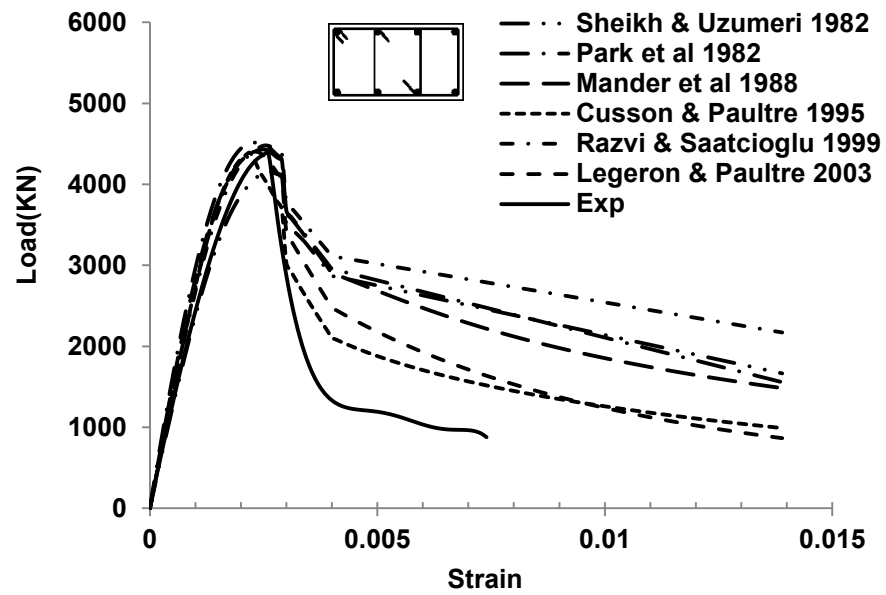


Figure 6.15 Comparison of Existing Models with Experimental Results – AR0

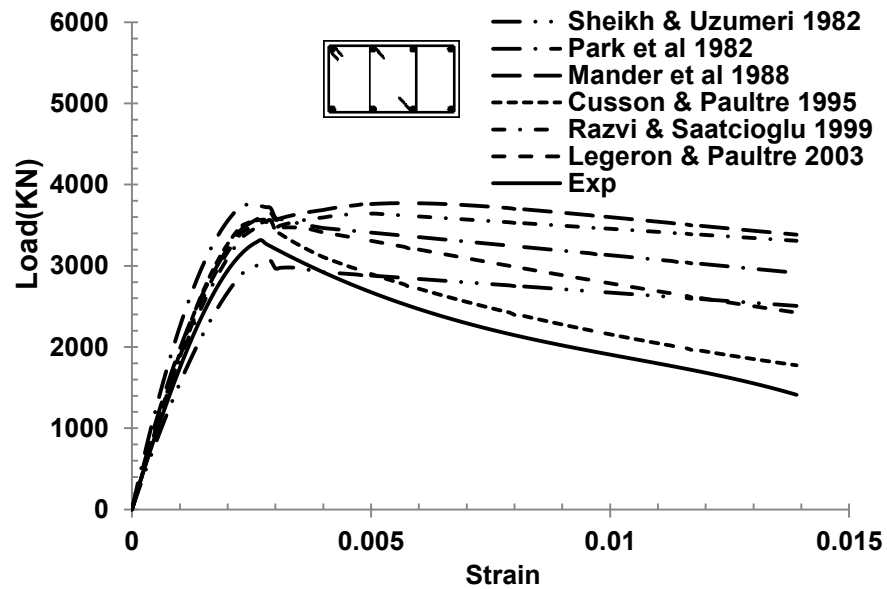


Figure 6.16 Comparison of Existing Models with Experimental Results – BR0nc

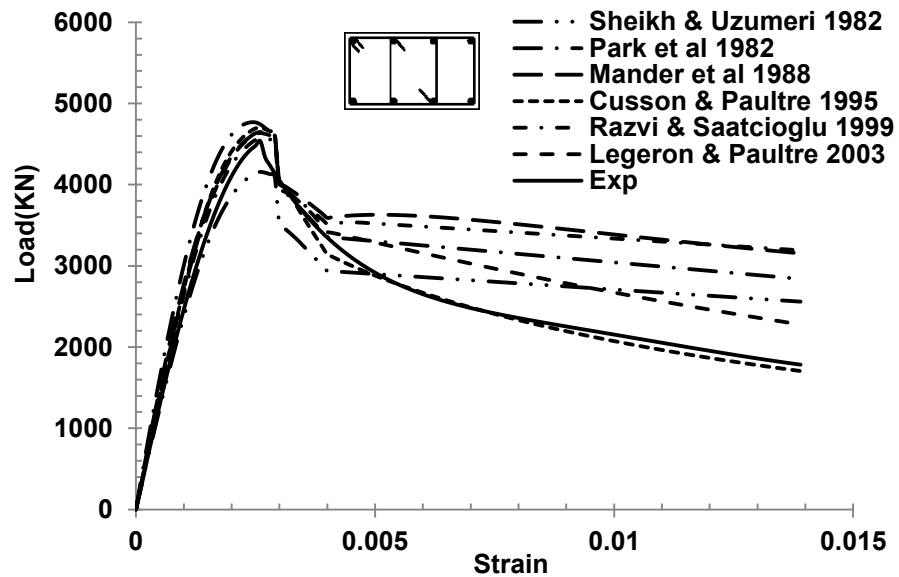


Figure 6.17 Comparison of Existing Models with Experimental Results – BR0

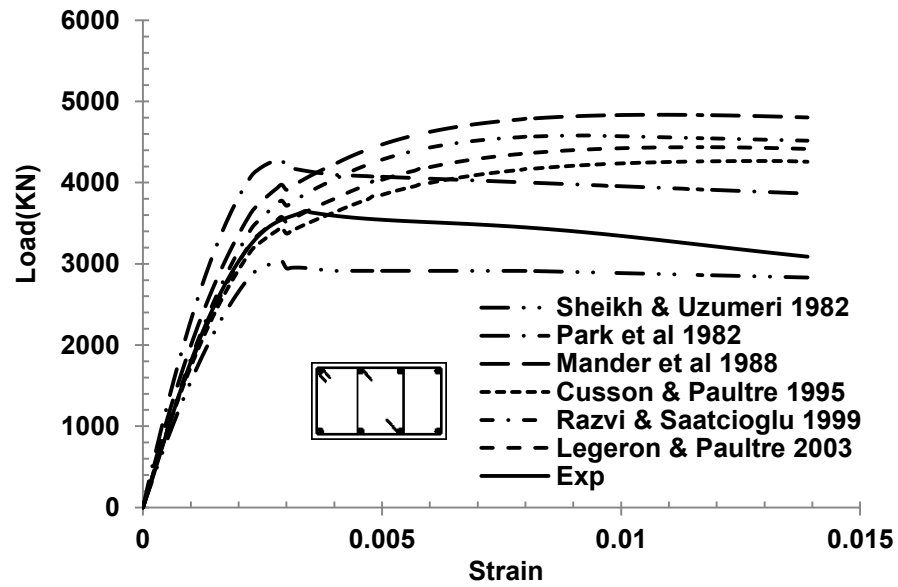


Figure 6.18 Comparison of Existing Models with Experimental Results – CR0nc

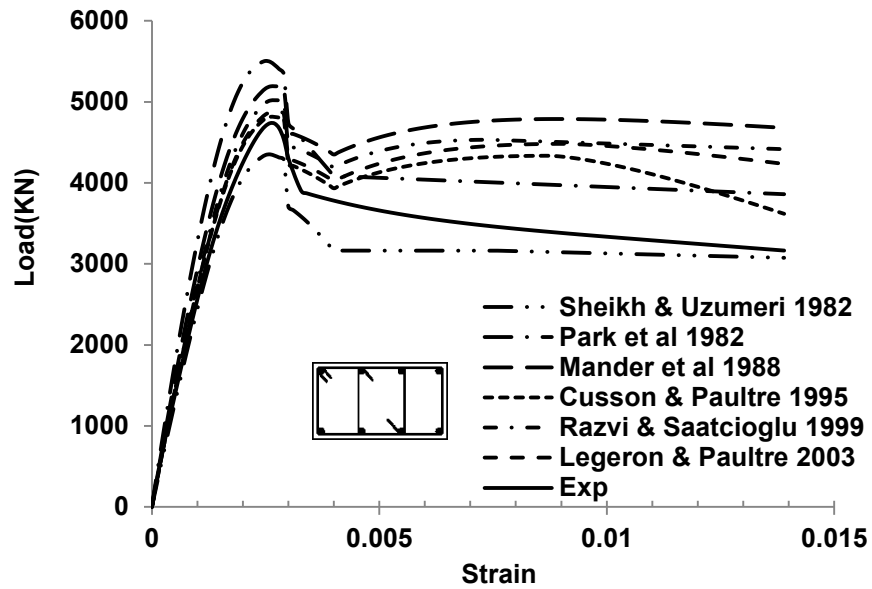


Figure 6.19 Comparison of Existing Models with Experimental Results – CR0

6.4.3 Analysis - SFRC Columns (1% and 1.5% Fibres)

In this section the experimental load-strain curves from the SFRC columns from the current research program and from the Aoude (2008) test series are predicted. For the SFRC columns the confinement models proposed by Campione (2002) and Aoude (2008) are investigated, while the cover response was modeled using the unconfined SFRC model and spalling factor proposed by Aoude (2008). The response of the longitudinal steel in compression and buckling was taken into account using the model proposed by Dhakal and Maekawa (2002).

6.4.3.1 Square-Section Columns (Aoude 2008)

For the square-section series, columns A1, A15, B1, B15 and C1, C15 are included in the analysis. The columns were constructed with SCC and steel fibres, had 300 x 300 mm section, 30 mm cover, and tie spacing and fibre contents of 240-120-65 mm and 1-1.5%, respectively. Also considered are a series of companion columns with identical detailing but constructed without cover, including specimens A1nc/ A15nc and B1nc/ B15nc.

Figure 6.20 to Figure 6.24 present the load-strain comparisons for the specimens with 1% fibres and the results for the specimens with 1.5% fibres are presented in Figure 6.25 to Figure 6.29. The results demonstrate that the confinement model proposed by Aoude (2008) provides good predictions of the axial load-strain responses for the SFRC columns. The model proposed by Campione (2002) generally under-predicts the responses in most cases.

Analytical over experimental ratios for the response parameters defined previously are provided in Table 6.15, along with average and standard deviation values in Table 6.16.

By comparing the ratios for peak load, P_{max} show greater accuracy (7 % and 12 % average under predictions for fibre content of 1% and 1.5%, respectively).

Comparing the ratios for peak strain, ϵ_{max} , poor results are observed for the Campione (2002) model (average ratio of 1.75 and 1.31 with standard deviation of 1.37 and 0.97 for fibre contents of 1% and 1.5% respectively). Predictions from the Aoude (2008) model are generally improved with average under prediction (ratio of 0.94 and 0.78 with standard deviation of 0.08 and 0.05 for fibre contents of 1% and 1.5% respectively).

By Comparing the ratios for the parameters, A_u and $P_{\varepsilon=0.01}$, for 1% fibre content, the Aoude (2008) model shows better prediction of A_u (ratio of 0.93 with standard deviation of 0.05) while the Campione (2002) model shows better prediction of $P_{\varepsilon=0.01}$. On the other hand, when fibre content is 1.5%, Aoude (2008) model shows better prediction for both A_u and $P_{\varepsilon=0.01}$ (ratio of 0.88 and 1.02 with standard deviation of 0.04 and 0.15 respectively).

For the ratios of K_{sd} , while it is observed that Aoude (2008) model shows lesser standard deviations, the prediction less accurate as fibre content increase to 1.5 %.

Finally for ratios of K_{pd} , for specimens C1 and C15, while experiments show small rate of softening, models predict strain hardening which explains the negative signs in the ratios.

6.4.3.2 Rectangular-section Columns

For the rectangular series, columns AR1, AR15, BR1, BR15 and CR1, CR15, which were tested in the current research program, are included in the analysis. Also considered are the companion columns constructed without cover: AR1nc/ AR15nc, BR1nc/ BR15nc and CR1nc.

Figure 6.30 to Figure 6.40 present the load-strain comparisons for the specimens with 1% and 1.5% fibres. Comparing the load-strain curves, it is evident that Aoude (2008) model generally shows closer behaviour to the experimental results, specifically for the columns with 1% fibre content.

Analytical over experimental ratios for the response parameters are provided in Table 6.17, along with average and standard deviation values are provided in Table 6.18.

By considering average values of P_{max} , it is evident that while the Aoude (2008) model over-predicts, Campione (2002) under-predicts peak capacity in SFRC columns (when considering average as well as the majority of individual specimen responses). The Campione (2002) model shows lesser standard deviation (0.13 and 0.09 for fibre content of 1% and 1.5%) and closer average values to experimental results (0.96 and 0.91 for fibre content of 1% and 1.5%) than the Aoude(2008) model.

Comparing the predictions for ε_{max} , the Aoude (2008) model shows lesser standard deviation (1.55 and 0.84 for fibre content of 1% and 1.5%) and closer average values to the experimental results (1.65 and 1.13 for fibre content of 1% and 1.5%) when comparing to the Campione(2002) model.

For the ratios of A_u and $P_{\varepsilon=0.01}$, the Campione(2002) model shows both lesser standard deviation and closer values to experimental results than Aoude(2008) model .

By comparing the ratios for the slopes K_{sd} and K_{pd} , it is found that while Aoude (2008) shows better prediction for fibre content of 1% (ratio of 1.24 with standard deviation of 0.28), but Campione(2002) model, shows better prediction for fibre content of 1.5% (ratio of 1.85 with standard deviation of 0.42).

Finally by comparing the ratios for the slopes K_{pd} , it is found that while for fibre content of 1% , both models predict hardening (softening occurs in experiment), for fibre content of 1.5% models predict softening correctly .

Table 6.15 Ratio of Analytical /Experimental Results – 1% & 1.5% Square
Sections

	#	Cam02	Aou08	#	Cam02	Aou08
P_{max}	A1	0.84	0.91	A15	0.82	0.94
	B1	0.82	0.96	B15	0.79	0.93
	C1	0.87	0.99	C15	0.72	0.87
	A1nc	0.78	0.87	A15nc	0.71	0.91
	B1nc	0.78	0.92	B15nc	0.71	0.97
ε_{peak}	A1	0.96	0.88	A15	0.81	0.78
	B1	1.07	0.93	B15	0.82	0.76
	C1	4.14	1.07	C15	3.03	2.69
	A1nc	0.92	0.88	A15nc	0.73	0.79
	B1nc	1.67	0.93	B15nc	1.18	0.79
A_u	A1	0.84	0.91	A15	0.82	0.94
	B1	0.82	0.96	B15	0.79	0.93
	C1	0.87	0.99	C15	0.72	0.87
	A1nc	0.78	0.87	A15nc	0.71	0.91
	B1nc	0.78	0.92	B15nc	0.71	0.97
$P_{\varepsilon=0.01}$	A1	0.96	1.12	A15	0.77	0.98
	B1	0.98	0.98	B15	1.03	1.02
	C1	0.92	1.05	C15	0.79	0.97
	A1nc	1.35	1.58	A15nc	1.11	1.51
	B1nc	1.12	1.12	B15nc	1.05	1.23
K_{sd}	A1	0.75	0.80	A15	1.25	1.26
	B1	0.50	0.80	B15	1.35	1.99
	C1	-0.43	1.11	C15	-0.13	-0.10
	A1nc	---	---	A15nc	---	---
	B1nc	---	---	B15nc	---	---
K_{pd}	A1	0.76	0.69	A15	0.64	0.64
	B1	0.77	0.97	B15	0.37	0.53
	C1	-23.87	-8.14	C15	-0.86	-0.56
	A1nc	0.76	0.64	A15nc	0.38	0.41
	B1nc	0.38	0.59	B15nc	0.18	0.43

Table 6.16 Average Ratio of Analytical /Experimental Results – 1% & 1.5 % Square Sections

		Cam02	Aou08		Cam02	Aou08
P_{max}	1% Fibres	0.82	0.93	1.5% Fibres	0.75	0.88
[St.d.]		[0.04]	[0.05]		[0.05]	[0.04]
ϵ_{peak}		1.75	0.94		1.31	0.78
[St.d.]		[1.37]	[0.08]		[0.97]	[0.05]
A_u		0.82	0.93		0.75	0.88
[St.d.]		[0.04]	[0.05]		[0.05]	[0.04]
$P_{\epsilon=0.01}$		1.06	1.17		0.95	1.02
[St.d.]		[0.17]	[0.24]		[0.16]	[0.15]
K_{sd}		0.56	0.90		0.82	1.05
[St.d.]		[0.17]	[0.18]		[0.82]	[1.06]
K_{pd}		-4.24	-1.05		0.14	0.32
[St.d.]		[10.97]	[3.97]		[0.58]	[0.43]

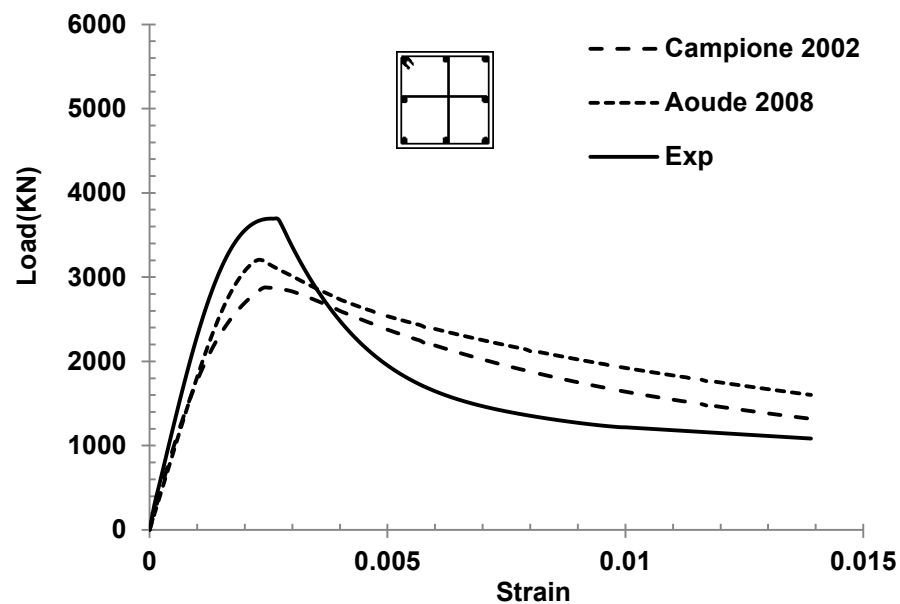


Figure 6.20 Comparison of Existing Models with Experimental Results – A1nc

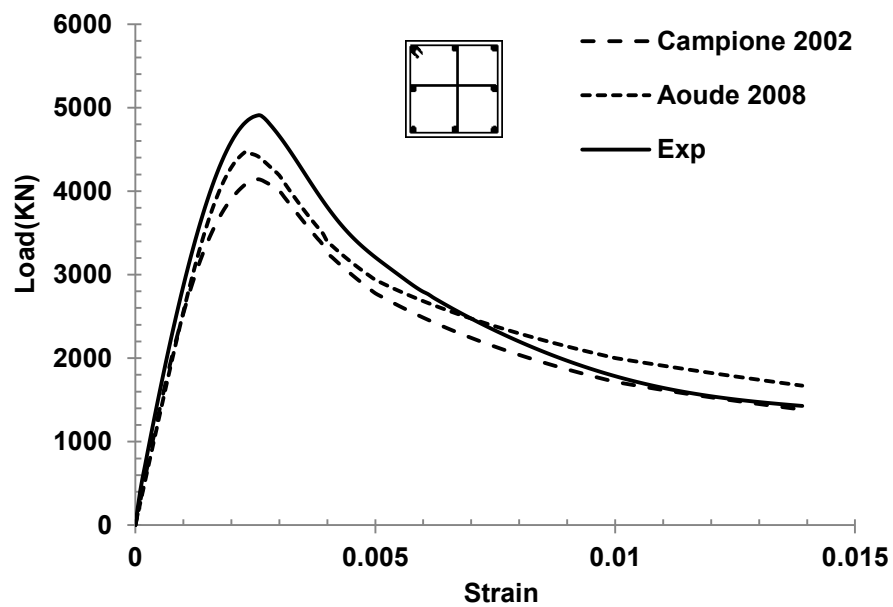


Figure 6.21 Comparison of Existing Models with Experimental Results – A1

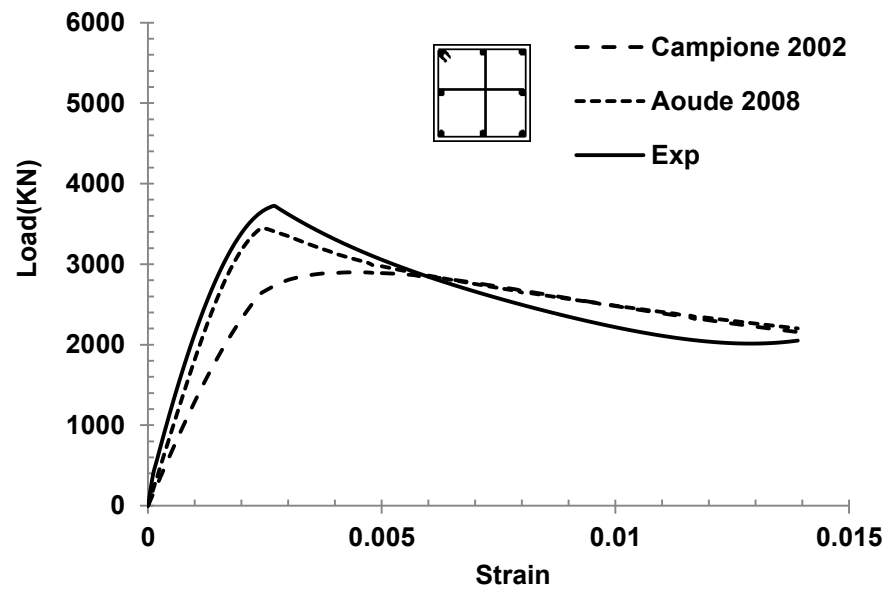


Figure 6.22 Comparison of Existing Models with Experimental Results – B1nc

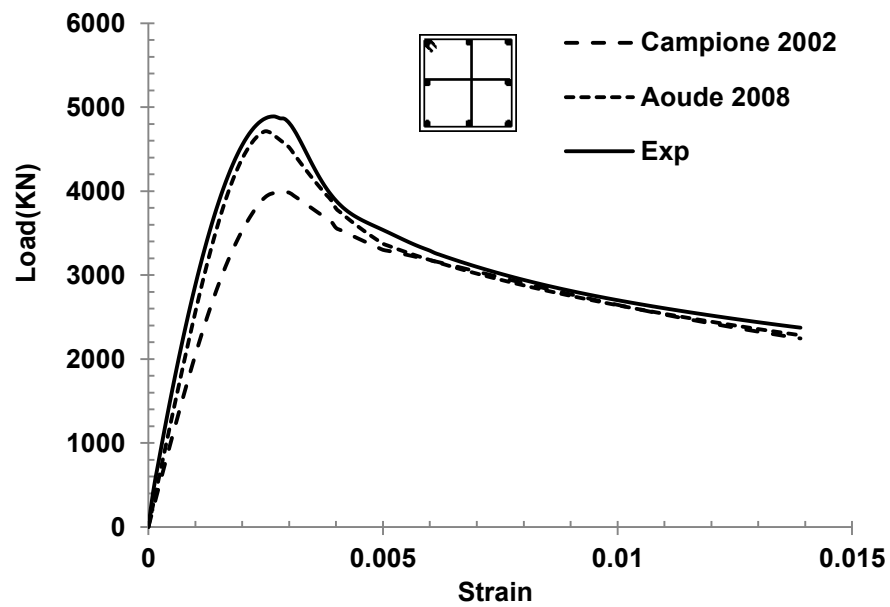


Figure 6.23 Comparison of Existing Models with Experimental Results – B1

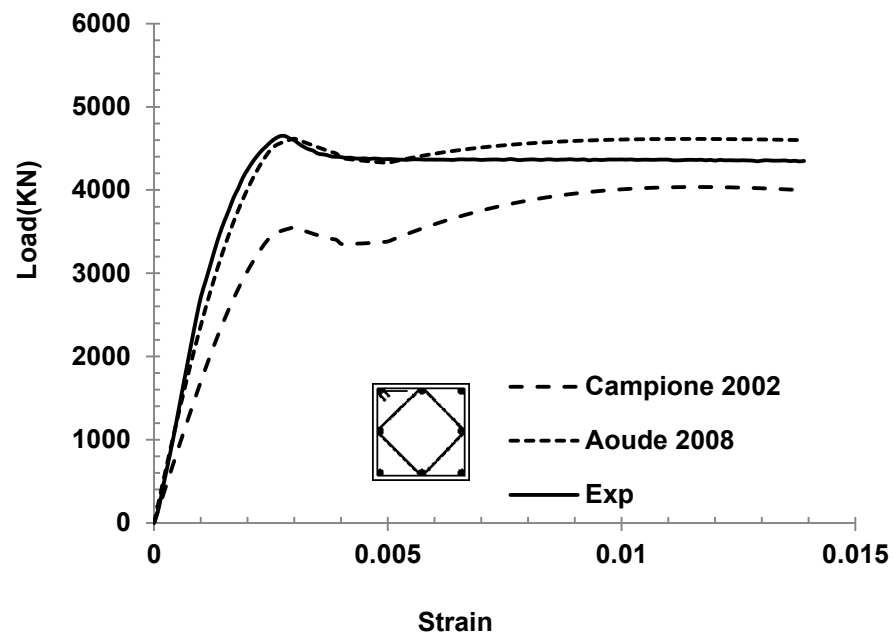


Figure 6.24 Comparison of Existing Models with Experimental Results – C1

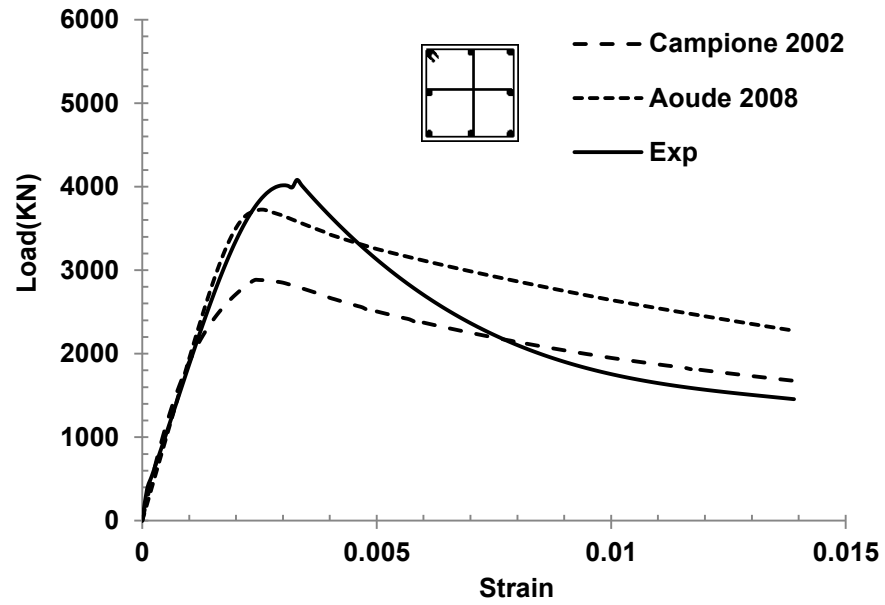


Figure 6.25 Comparison of Existing Models with Experimental Results – A15nc

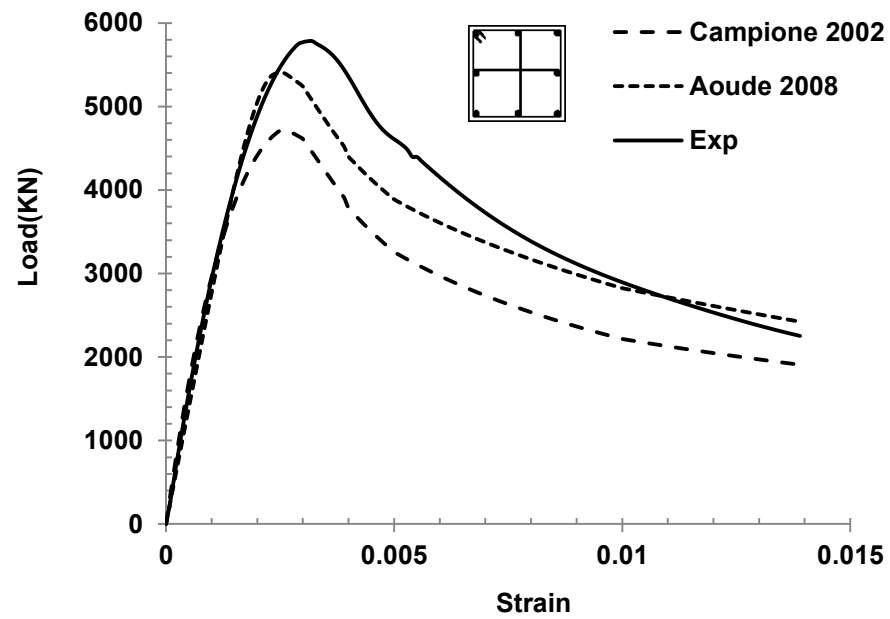


Figure 6.26 Comparison of Existing Models with Experimental Results – A15

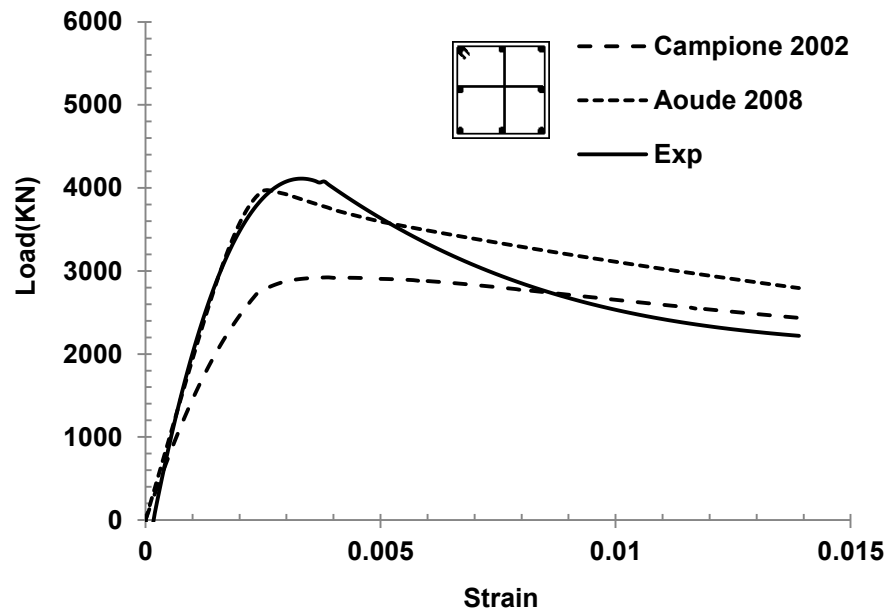


Figure 6.27 Comparison of Existing Models with Experimental Results – B15nc

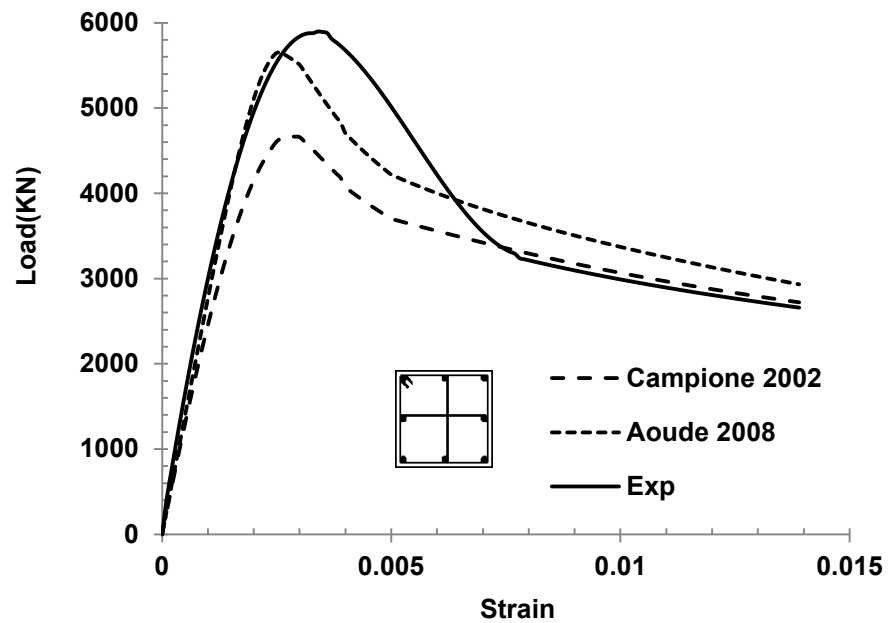


Figure 6.28 Comparison of Existing Models with Experimental Results – B15

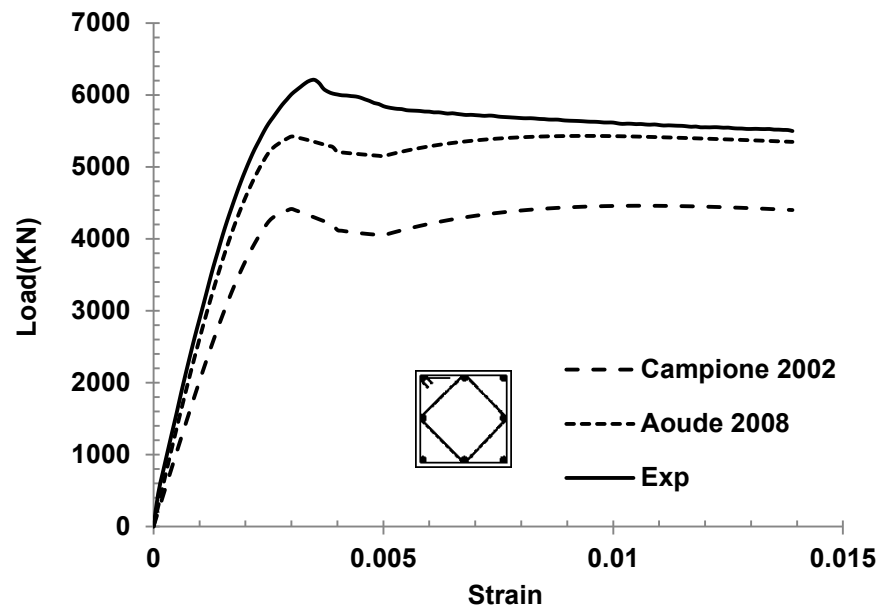


Figure 6.29 Comparison of Existing Models with Experimental Results – C15

Table 6.17 Ratio of Analytical /Experimental Results – 1% & 1.5% Rectangular Sections

	#	Cam02	Aou08	#	Cam02	Aou08
P_{max}	AR1	0.99	1.07	AR15	0.95	1.06
	BR1	0.94	1.14	BR15	1.00	1.21
	CR1	0.86	1.04	CR15	0.79	0.96
	AR1nc	0.84	0.95	AR15nc	0.85	1.00
	BR1nc	0.91	1.13	BR15nc	0.97	1.24
	CR1nc	1.20	1.35			
ε_{peak}	AR1	0.74	0.68	AR15	0.87	0.83
	BR1	0.88	0.79	BR15	0.76	0.74
	CR1	3.18	0.91	CR15	0.79	0.79
	AR1nc	0.76	0.61	AR15nc	0.86	0.66
	BR1nc	3.18	2.45	BR15nc	3.29	2.62
	CR1nc	4.48	4.48			
A_u	AR1	0.99	1.07	AR15	0.95	1.06
	BR1	0.94	1.14	BR15	1.00	1.21
	CR1	0.86	1.04	CR15	0.79	0.96
	AR1nc	0.84	0.95	AR15nc	0.85	1.00
	BR1nc	0.91	1.13	BR15nc	0.97	1.24
	CR1nc	1.20	1.35			
$P_{\varepsilon=0.01}$	AR1	0.90	1.06	AR15	0.89	1.02
	BR1	1.01	1.07	BR15	0.80	0.85
	CR1	0.97	1.12	CR15	0.86	1.00
	AR1nc	1.63	1.94	AR15nc	1.74	1.97
	BR1nc	1.20	1.48	BR15nc	1.49	1.86
	CR1nc	1.21	1.38			
K_{sd}	AR1	1.24	1.30	AR15	1.49	1.61
	BR1	1.07	1.49	BR15	2.32	3.22
	CR1	-0.23	0.93	CR15	1.75	1.77
	AR1nc	---	---	AR15nc	---	---
	BR1nc	---	---	BR15nc	---	---
	CR1nc	---	---			
K_{pd}	AR1	0.77	0.70	AR15	0.78	0.81
	BR1	0.63	0.89	BR15	1.43	2.18
	CR1	-1.63	-0.26	CR15	-0.79	-0.24
	AR1nc	0.43	0.36	AR15nc	0.37	0.41
	BR1nc	-0.17	0.22	BR15nc	-0.13	0.17
	CR1nc	-6.90	-3.68			

Table 6.18 Average Ratio of Analytical /Experimental Results – 1% & 1.5 %
Rectangular Sections

	Cam02	Aou08	Cam02	Aou08
P_{max}	0.96	1.11	0.91	1.09
[St.d.]	[0.13]	[0.13]	[0.09]	[0.12]
ϵ_{peak}	2.20	1.65	1.31	1.13
[St.d.]	[1.62]	[1.55]	[1.10]	[0.84]
A_u	0.96	1.11	0.91	1.09
[St.d.]	[0.13]	[0.13]	[0.09]	[0.12]
$P_{\epsilon=0.01}$	1.15	1.34	1.16	1.34
[St.d.]	[0.27]	[0.34]	[0.43]	[0.53]
K_{sd}	0.85	1.24	1.85	2.20
[St.d.]	[0.54]	[0.28]	[0.42]	[0.89]
K_{pd}	-1.14	-0.30	0.33	0.67
[St.d.]	[2.95]	[1.71]	[0.85]	[0.93]

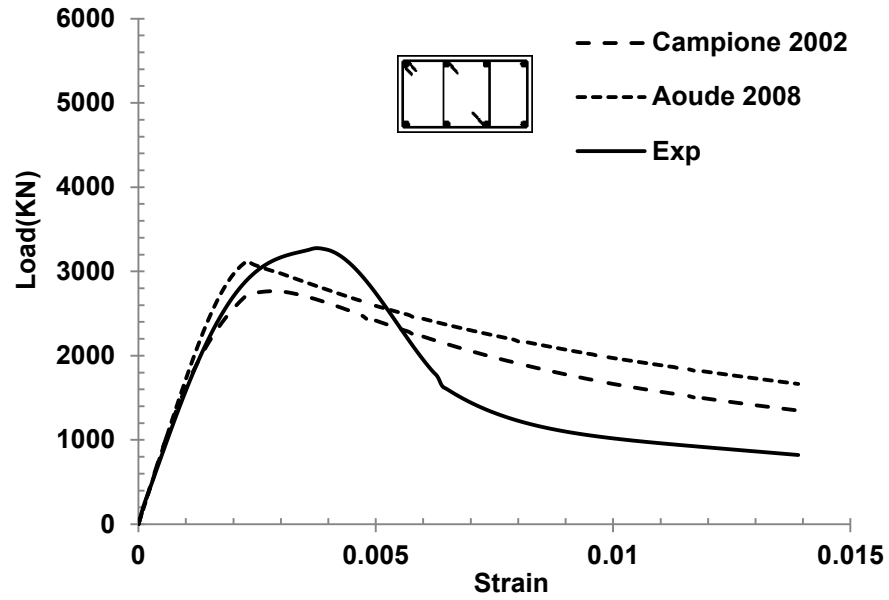


Figure 6.30 Comparison of Existing Models with Experimental Results – AR1nc

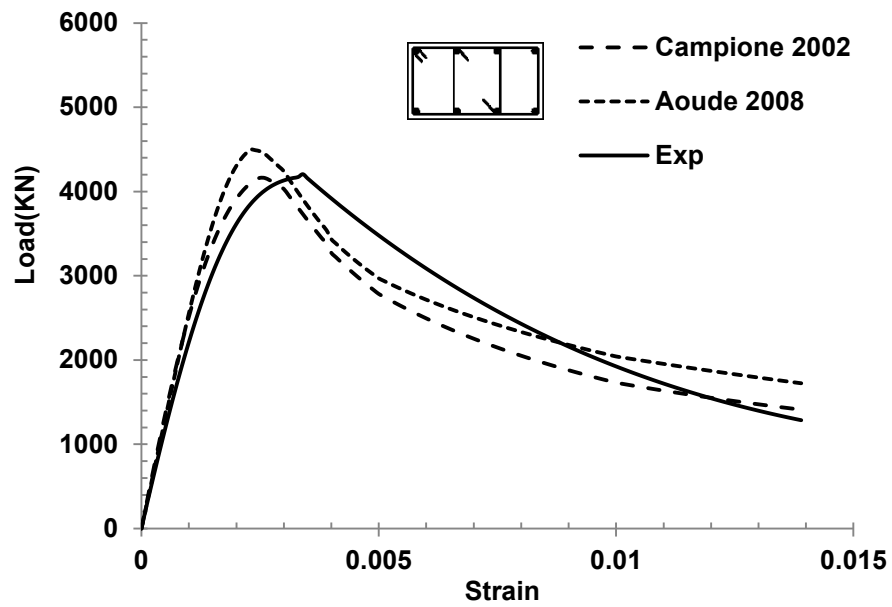


Figure 6.31 Comparison of Existing Models with Experimental Results – AR1

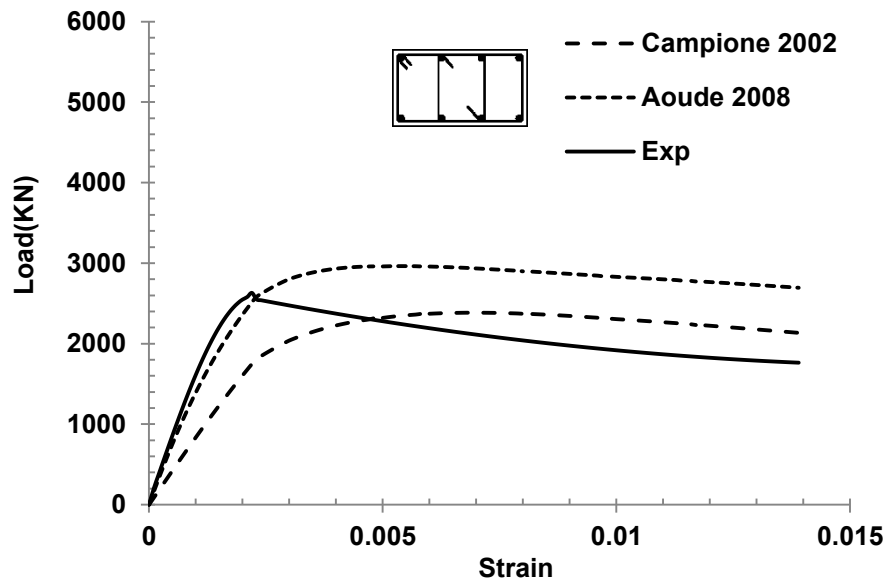


Figure 6.32 Comparison of Existing Models with Experimental Results – BR1nc

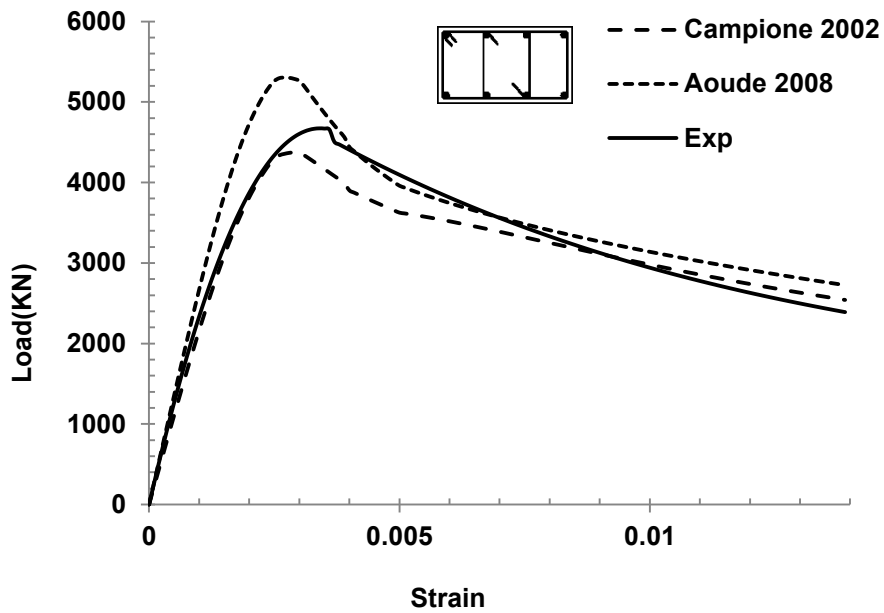


Figure 6.33 Comparison of Existing Models with Experimental Results – BR1

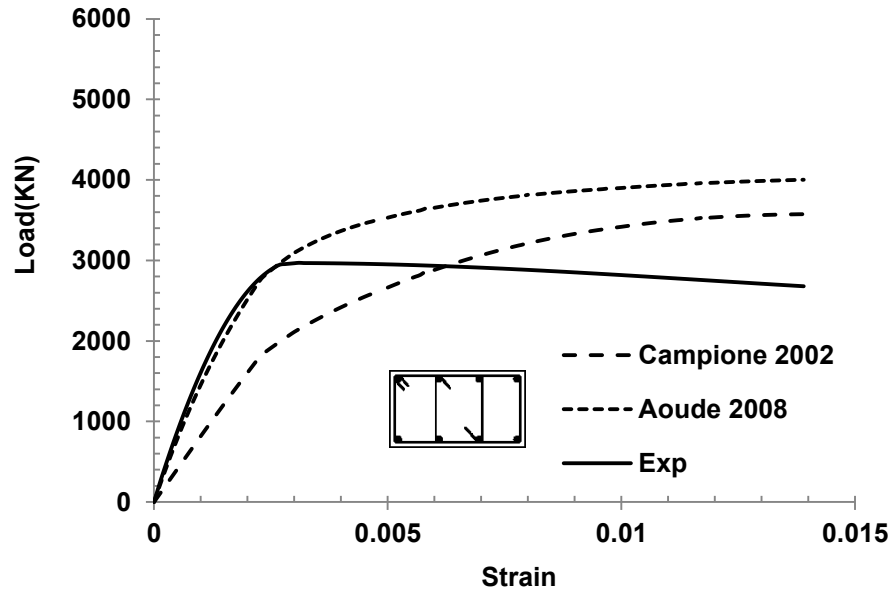


Figure 6.34 Comparison of Existing Models with Experimental Results – CR1nc

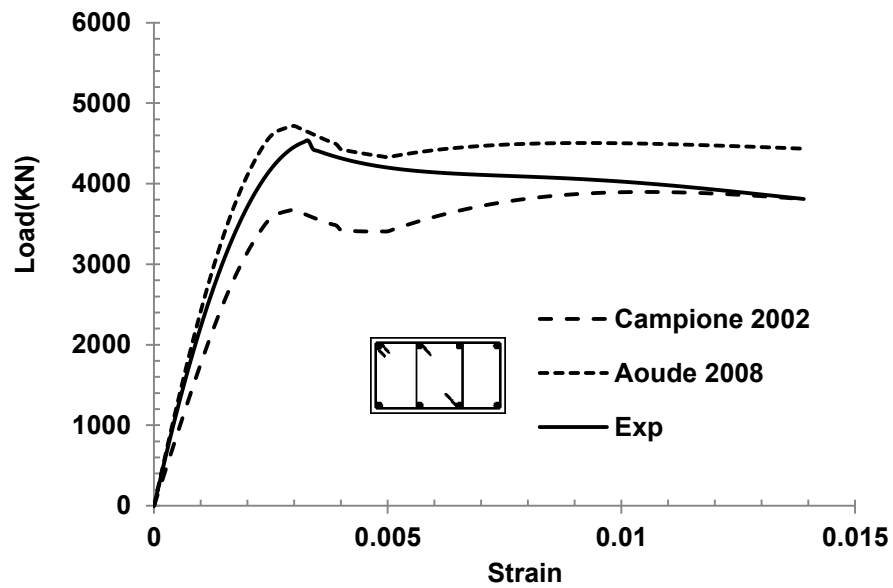


Figure 6.35 Comparison of Existing Models with Experimental Results – CR1

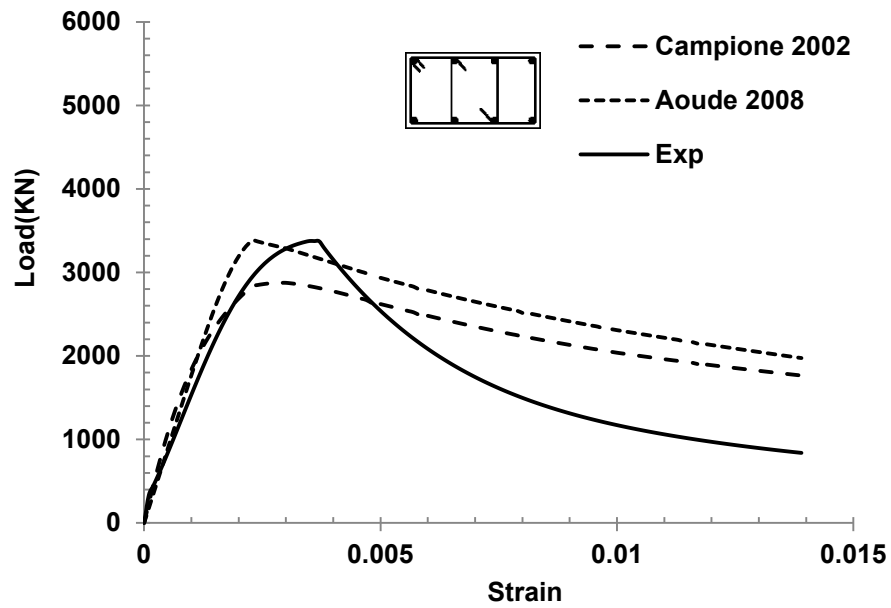


Figure 6.36 Comparison of Existing Models with Experimental Results – AR15nc

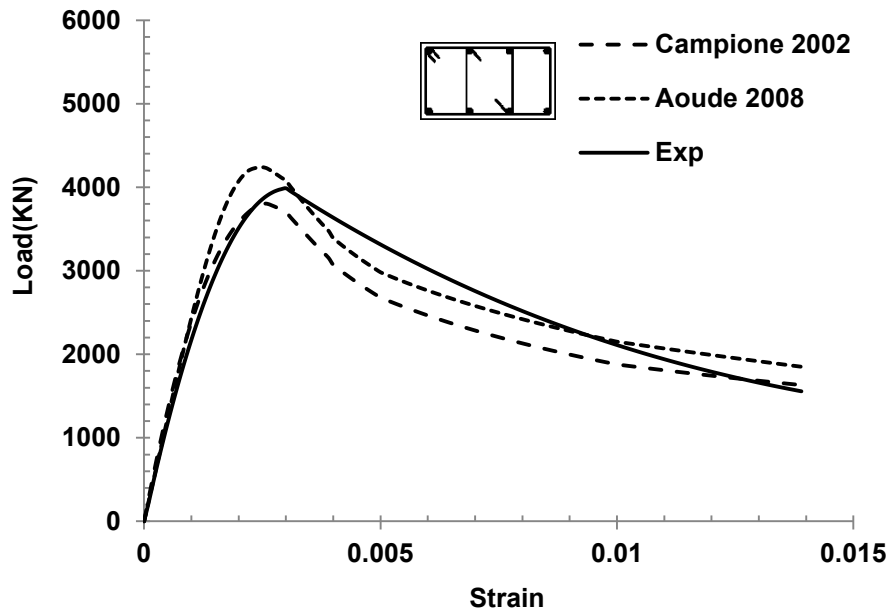


Figure 6.37 Comparison of Existing Models with Experimental Results – AR15

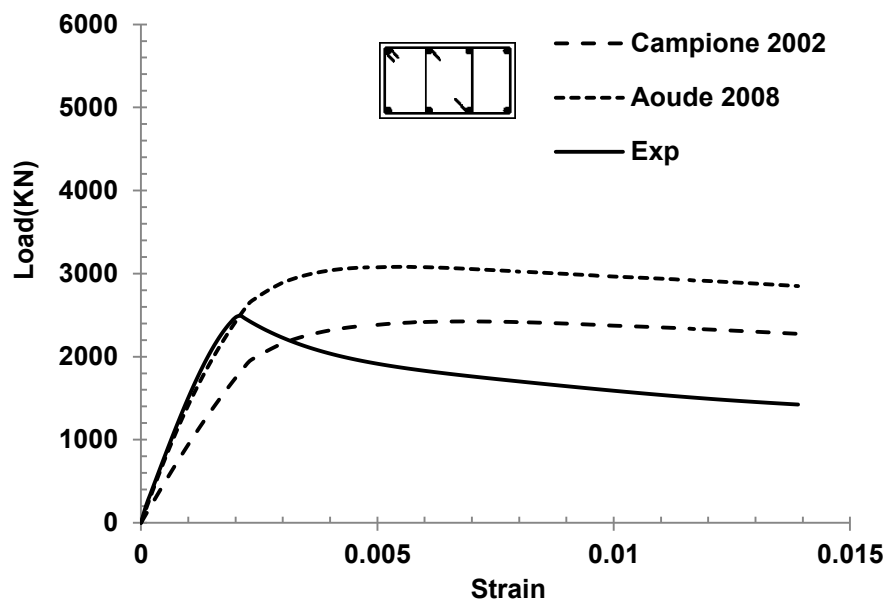


Figure 6.38 Comparison of Existing Models with Experimental Results – BR15nc

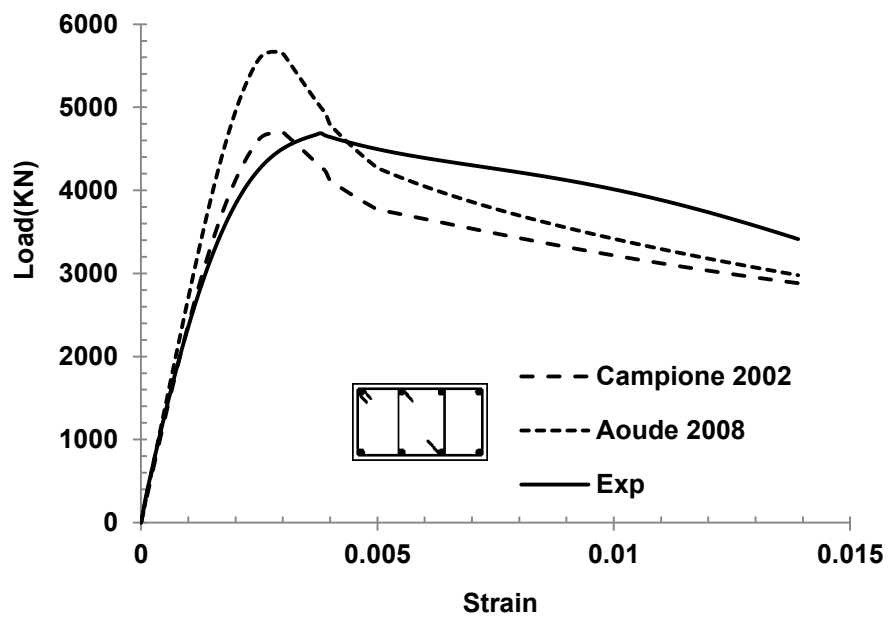


Figure 6.39 Comparison of Existing Models with Experimental Results – BR15

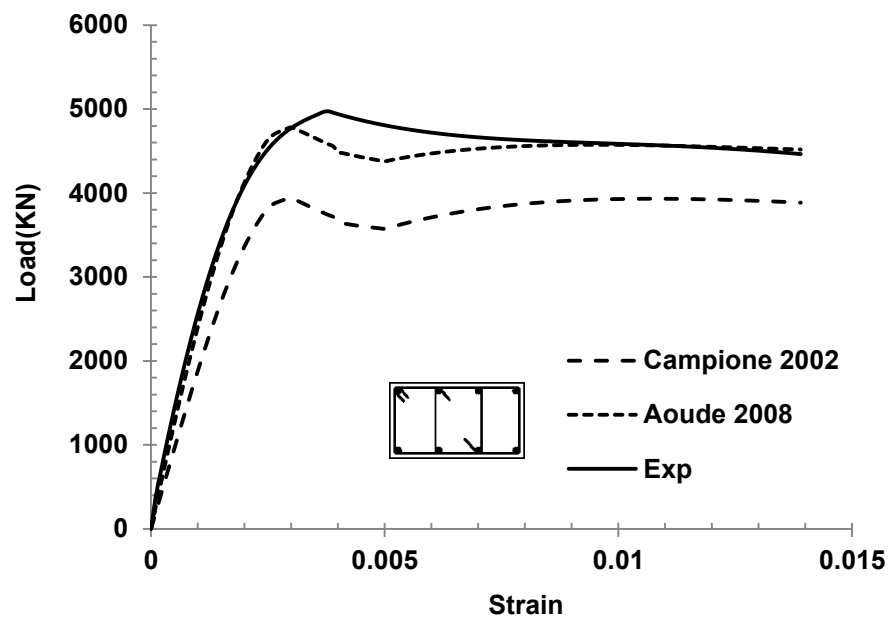


Figure 6.40 Comparison of Existing Models with Experimental Results – CR15

6.4.4 Analysis - UHPFRC Columns

While limited research exists on the axial load performance of UHPFRC columns, there is lack of confinement models in the literature for UHPFRC. In the current analytical study, the suitability of using high-strength RC and high-strength SFRC confinement models to predict the response of the six UHPFRC columns is investigated. Confinement was taken into account using the high-strength concrete models proposed by Razvi and Saatcioglu (1999) and Légeron and Paultre (2003). In addition the Paultre et al. (2010) high-strength SFRC model is also considered. Cover response was modeled using the unconfined concrete model proposed by Popovics (1973) and the spalling factor proposed by Aoude (2008). The response of the longitudinal steel in compression and buckling was taken into account using the model proposed by Dhakal and Maekawa (2002).

The experimental and analytical load-strain curves are presented in Figure 6.41 to Figure 6.46. Table 6.19 and Table 6.20 list the analytical over experimental ratios for the response parameters introduced in Figure 6.7. Comparing the load-strain curves it is observed that, in the specimens with low confinement (e.g. columns C3-120, C3-80 and C4-120) models generally provide good predictions of column response, except for residual strength at large strains. On the other hand, for the specimens that are highly confined, prediction of behaviour is less accurate especially in the post-peak stage, although residual strength at high strains is predicted with reasonable accuracy by the Razvi and Saatcioglu (1999) model.

By Comparing the ratios for peak load, P_{max} , average predictions are reasonable, for example the Legeron and Paultre (2003) model shows slight average over prediction of 1.06 with standard deviation of 0.06. For peak strain, ϵ_{max} , the Paultre et al (2010) shows the best predictions average ratio of 0.86 and standard deviation 0.08. Examining the ratios for the parameters $P_{\epsilon=0.01}$ and $P_{\epsilon=0.02}$, the best results are for the Razvi and Saatcioglu (1999) model (ratios of 0.99 and 0.83 with standard deviation of 0.11 and 0.17, respectively). Finally, for A_u , all models show reasonable predictions when considering average ratios with the Legeron and Paultre (2003) model providing predictions which are closer to the experimental results (ratio of 0.98 with standard deviation of 0.13).

The analysis demonstrates the need for UHPFRC specific models, In summary while existing models for high-strength concrete and SFRC show reasonable predictions there is a need to develop confinement models which are specific to UHPFRC.

Table 6.19 Ratio of Analytical /Experimental Results – UHPFRC Columns

	#	Raz99 ^a	Leg03	Pau2010
P_{max}	C3-120	0.93	1.04	1.10
	C3-80	---	1.09	1.15
	C3-60	0.97	1.09	1.11
	C3-40	0.85	0.97	0.96
	C4-120	---	1.03	1.10
	C4-60	1.03	1.16	1.16
ϵ_{peak}	C3-120	0.67	0.91	0.85
	C3-80	---	1.06	0.87
	C3-60	0.55	1.10	0.78
	C3-40	0.55	1.35	0.99
	C4-120	---	0.85	0.77
	C4-60	0.55	1.28	0.90
$P_{\epsilon=0.01}$	C3-120	1.01	0.77	0.89
	C3-80		1.02	0.91
	C3-60	1.13	1.34	1.07
	C3-40	0.87	1.17	1.05
	C4-120	---	0.86	0.83
	C4-60	0.95	1.25	1.01
$P_{\epsilon=0.02}$	C3-120	0.59	0.35	0.60
	C3-80	---	0.54	0.62
	C3-60	0.94	0.73	0.58
	C3-40	0.95	1.05	0.74
	C4-120	---	0.39	0.51
	C4-60	0.84	0.82	0.55
A_u	C3-120	0.89	0.81	0.89
	C3-80	---	0.96	0.91
	C3-60	1.05	1.13	0.99
	C3-40	0.89	1.05	0.95
	C4-120	---	0.83	0.83
	C4-60	0.94	1.09	0.94

a: Model did not work for the specimen C3-80 and C4-120.

Table 6.20 Average Ratio of Analytical /Experimental Results – UHPFRC Columns

	Raz99	Leg03	Pau10
P_{max}	0.94	1.06	1.10
[St.d.]	[0.08]	[0.06]	[0.07]
ϵ_{peak}	0.58	1.09	0.86
[St.d.]	[0.06]	[0.20]	[0.08]
$P_{\epsilon=0.01}$	0.99	1.07	0.96
[St.d.]	[0.11]	[0.22]	[0.10]
$P_{\epsilon=0.02}$	0.83	0.65	0.60
[St.d.]	[0.17]	[0.27]	[0.08]
A_u	0.94	0.98	0.92
[St.d.]	[0.08]	[0.13]	[0.05]

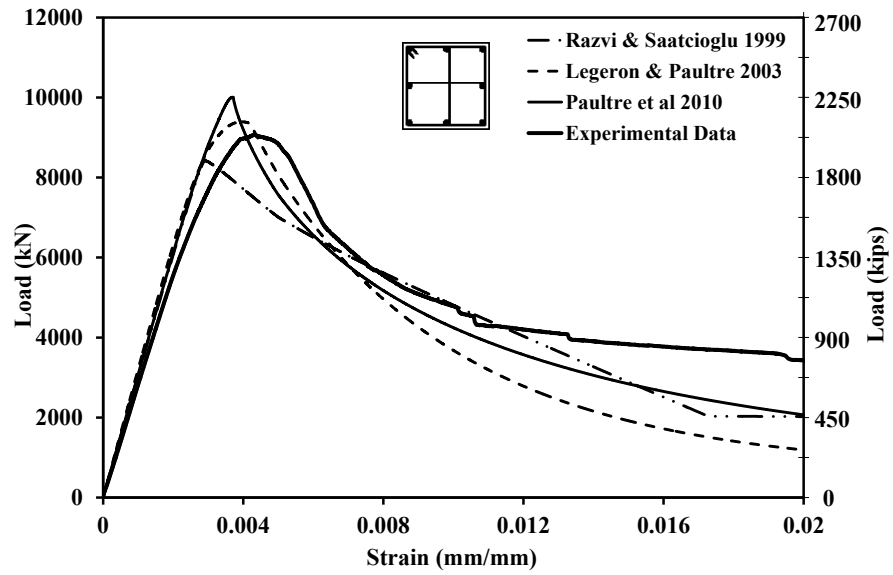


Figure 6.41 Comparison of Existing Models with Experimental Results –C3-120

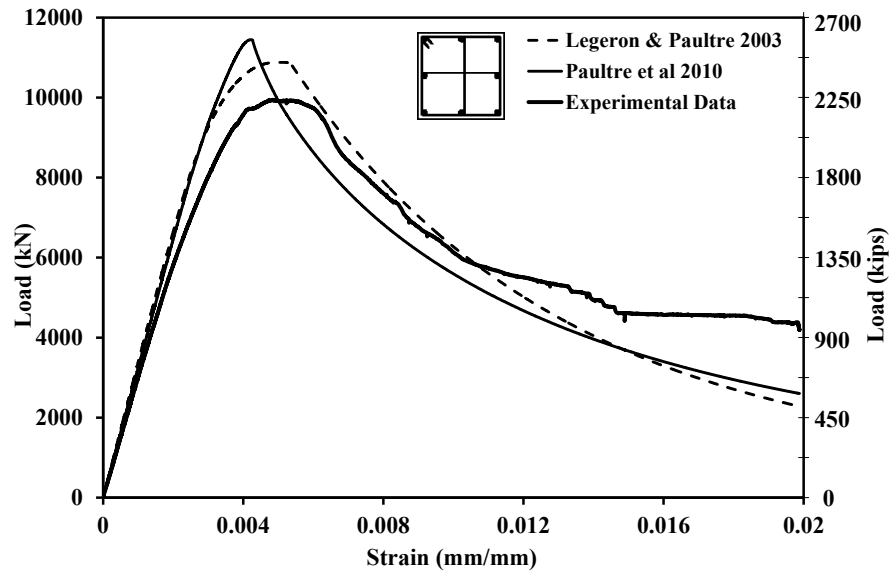


Figure 6.42 Comparison of Existing Models with Experimental Results –C3-80

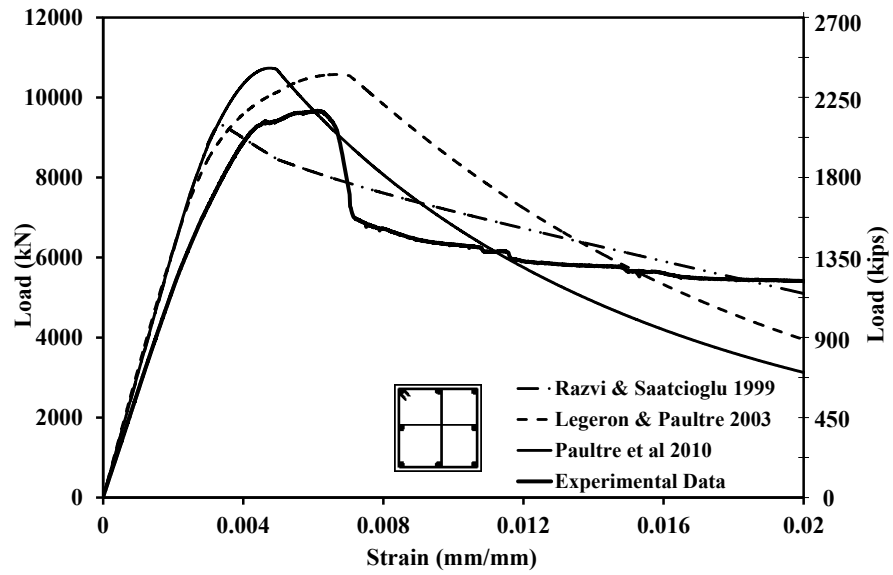


Figure 6.43 Comparison of Existing Models with Experimental Results –C3-60

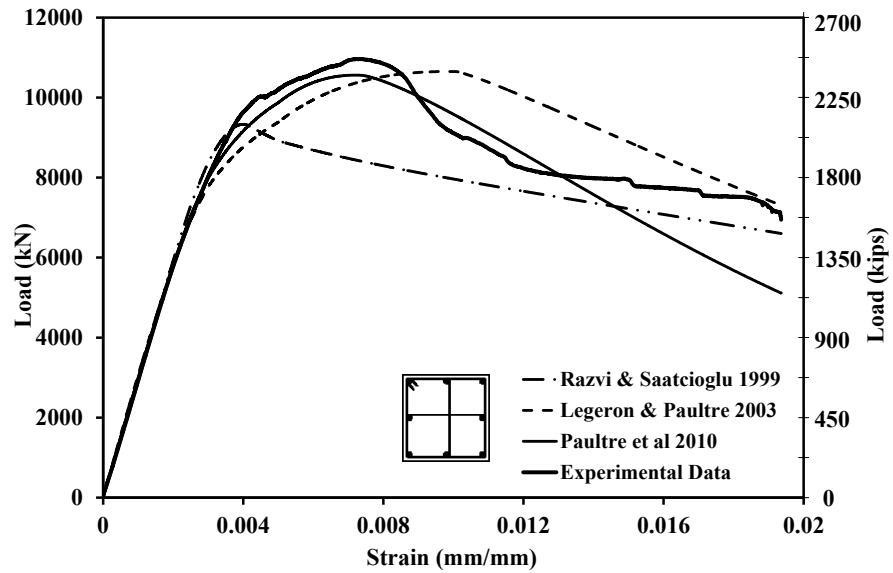


Figure 6.44 Comparison of Existing Models with Experimental Results –C3-40

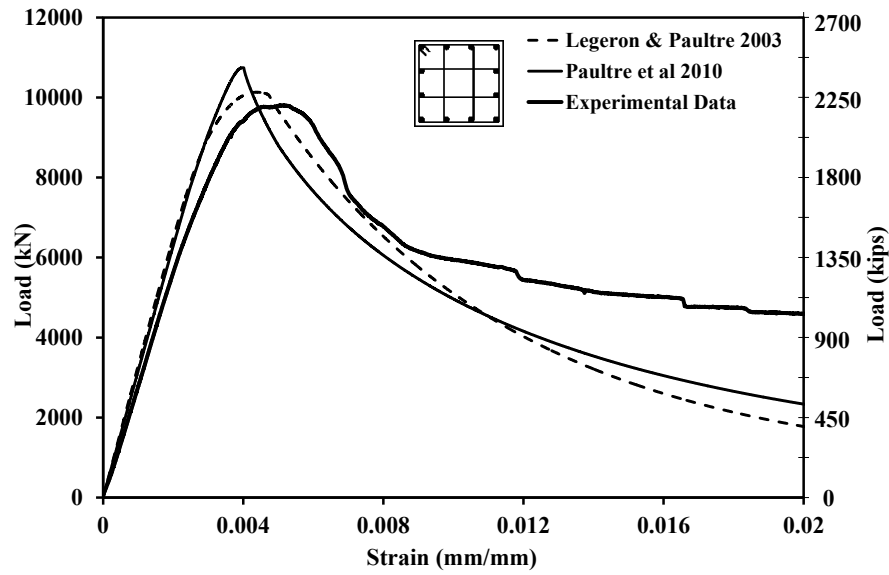


Figure 6.45 Comparison of Existing Models with Experimental Results – C4-120

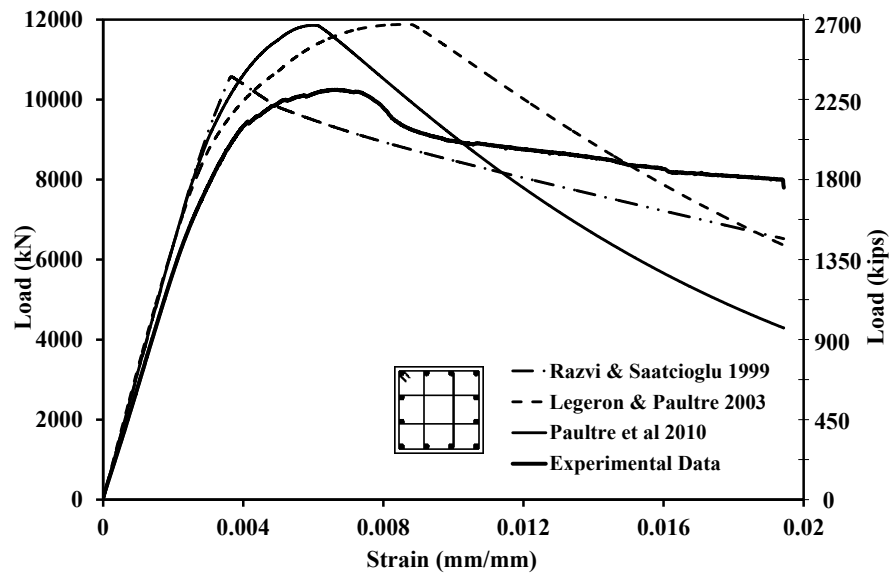


Figure 6.46 Comparison of Existing Models with Experimental Results – C4-60

7 Conclusions and Recommendations

7.1 Conclusions

This thesis presented the results from a research program studying the axial load behaviour of SCC, SFRC and UHPFRC columns.

In the case of the SFRC series, seventeen columns having rectangular section and constructed with SCC and steel fibres were tested and the following conclusions can be drawn from the experimental and analytical research program:

- The use of SCC in heavily confined columns allows for ease of construction and eliminates the need for vibration. The combined use of SCC and moderate amounts of fibres improves workability SFRC, however at higher fibre contents self-compacting properties can be lost and vibration may be required;
- The use of closely spaced and well detailed transverse reinforcement in rectangular SCC columns results in improved confinement which leads to increased strength and ductility;
- In the case of rectangular columns, the use of SFRC leads to significant improvements in the post-peak ductility, without significant increases in the maximum axial capacity;
- Steel fibres can potentially be used to partially substitute for the transverse reinforcement in columns and hence could result in improved constructability;
- Addition of fibres results in increased core integrity, however when considering core concrete improvement, effect of fibres is more significant in columns with low confinement when compared to columns with high confinement;
- Sudden cover spalling occurs in plain SCC columns regardless of the amount of confinement. This behaviour is improved by introduction of fibres allowing the cover to contribute significantly to column axial capacity. The cover contribution remains stable for highly confined columns but gradually reduces in the case of columns with low confinement;

- Addition of fibres improves tensile capacity of concrete and therefore results in a dispersed crack pattern with development of many secondary cracks on the column surface;
- In SCC columns large pieces of cover buckle due to instability, the improved tensile capacity of SFRC in the core-cover interface transforms this mechanism of failure and therefore failure zone becomes more localized;
- Bar buckling is affected by hoop spacing in SCC columns, however it is not prevented by addition of fibres. Both bar buckling and deformation of 90 degree hooks in cross-ties can initiate cover spalling in SFRC columns;
- The analytical results demonstrate that well-established confinement models for reinforced concrete are applicable for SCC. Further research is recommended to study confinement in rectangular SCC columns. In particular the model proposed by Légeron and Paultre (2003) provides good prediction of SCC column response under axial loading if cover spalling and buckling models are included in the analysis;
- In the case of square SFRC columns, good agreement is obtained when using the confinement model proposed by Aoude (2008). Additional research and experimental data is needed to develop accurate confinement models for rectangular SFRC columns.

In the case of the UHPFRC series, six columns having square section were tested and the following conclusions can be drawn from the experimental program:

- The use of closely spaced and well detailed transverse reinforcement in UHPFRC columns results in improved confinement of the material which prevents its brittle failure and leads to increased ductility;
- For a given column configuration (i.e. tie arrangement) by reducing tie spacing confinement improves and results in enhancement of post-peak ductility;
- For a given tie spacing, in low confined columns, tie configuration does not have a significant effect, whereas in highly confined columns more efficient configuration results in more enhancements of column response;
- The provision of fibres in UHPFRC prevents cover spalling;

- Regardless of confinement level, great core integrity is observed in UHPFRC columns;
- The mode of failure in UHPFRC columns is different with damage distributed throughout the column. Due to existence of fibres and tensile strength is well distributed on the cover surface which results in primary cracking and a large amount of secondary cracks on the column surface;
- The analytical results demonstrate that while existing models for high-strength concrete and SFRC show reasonable predictions there is a need to develop confinement models which are specific to UHPFRC.

7.2 Recommendations for Future Research

The following is recommended for future research:

- Further experimental research is required to validate the potential benefits of using SFRC in columns under axial loads, and to assess the effect of different variables such as cross-section detailing and shape, maximum amount of fibres and effect of fibre properties (strength, aspect ratio, type);
- Further research is recommended to study the effect of eccentricity on axial behaviour of SCC and SFRC columns;
- Research on the behaviour of SFRC columns under reverse-cyclic loading is recommended to validate the benefits of using SFRC in seismic design;
- Research on the potential of using SFRC to retrofit seismically-deficient columns is recommended (axial tests and reverse-cyclic tests);
- Further experimental research on the behaviour of UHPFRC columns under axial loads is recommended to study effect of:
 - high-strength transverse steel;
 - cross-section type and configuration;
 - fibre type and maximum fibre content;
- Research on the behaviour of UHPFRC columns under different loading conditions - including: reverse-cyclic loading, under eccentricity, biaxial loading, etc.

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9 Appendix

Appendix A – Detailed Tables of Database for SCC

		Dimension	Cover	Spacing	Bar-Long	Tie	Fyb	Fyh	f'c
		(mm)	(mm)				(Mpa)	(Mpa)	(Mpa)
Paultre et al. 2005	10BSCC	235x235	20	50	4No.20+4No.15	φ 9.5 (Dm)	431.5	820	39.1
	10BSCCO	235x235	20	50	4No.20+4No.15	φ 9.5 (Dm)	431.5	410	41
	10DSCC	235x235	20	50	4No.20+8No.10	φ 7.9 (Sq+)	468.5	801	39.1
	10DSCCO	235x235	20	50	4No.20+8No.10	φ 7.9 (Sq+)	468.5	440	41
	8BSCC	235x235	20	50	4No.20+4No.15	φ 9.5 (Dm)	431.5	820	60
	8BSCCO	235x235	20	50	4No.20+4No.15	φ 9.5 (Dm)	431.5	411	60
	8DSCC	235x235	20	50	4No.20+8No.10	φ 7.9 (Sq+)	429	769	60
	8DSCCO	235x235	20	50	4No.20+8No.10	φ 7.9 (Sq+)	468.5	440	61.6
	7BSCC	235x235	20	50	4No.20+4No.15	φ 9.5 (Dm)	439.5	891	80.5
	7BSCCO	235x235	20	50	4No.20+4No.15	φ 9.5 (Dm)	436	410	87.2
	7DSCC	235x235	20	50	4No.20+8No.10	φ 7.9 (Sq+)	429	769	80.5
	7DSCCO	235x235	20	50	4No.20+8No.10	φ 7.9 (Sq+)	468.5	440	87.2

Dm=Diamond, Sq+ = Square with additional ties, Sq = Square

		Dimension (mm)	Cover (mm)	Spacing	Bar-Long	Tie	Fyb (Mpa)	Fyh (Mpa)	f'c (Mpa)
Lin et al . 2008	S4	300x300	30	90	8 ϕ 19	ϕ 10 (Sq)	552	447.2	30.2
	S5	300x300	30	90	8 ϕ 19	ϕ 10 (Sq)	552	447.2	41.9
	S6	300x300	30	90	8 ϕ 19	ϕ 10 (Sq)	552	447.2	53.2
	S7	300x300	30	90	4 ϕ 22	ϕ 10 (Sq)	552	447.2	40.6
	S8	300x300	30	90	8 ϕ 22	ϕ 10 (Sq)	552	447.2	39.2
	S9	300x300	30	90	8 ϕ 19	ϕ 10 (Sq)	552	560	41.2
	S10	300x300	30	90	8 ϕ 19	ϕ 10 (Sq)	552	339.4	43.1
	S11	300x300	30	150	8 ϕ 19	ϕ 10 (Sq)	552	447.2	41.8
	S12	300x300	30	60	8 ϕ 19	ϕ 10 (Sq)	552	447.2	42.5
	S13	300x300	30	60	8 ϕ 19	ϕ 10 (Sq+)	552	447.2	42.2
	S14	300x300	30	68.4	8 ϕ 19	ϕ 10 (Sq)	552	339.4	43.7
	S15	300x300	30	113	8 ϕ 19	ϕ 10 (Sq)	552	560	42
	S16	300x300	30	135	8 ϕ 19	ϕ 10 (Sq+)	552	447.2	42.7

Dm=Diamond, Sq+ = Square with additional ties, Sq = Square

		Dimension (mm)	Cover (mm)	Spacing	Bar-Long	Tie	Fyb (Mpa)	Fyh (Mpa)	f'c (Mpa)
Bullo et al. 2008	SCCA	220x220	---	150	4 ϕ 20	ϕ 10 (Sq)	520	---	40.5
	SCCB	220x220	---	100	8 ϕ 14	ϕ 10 (Dm)	533	---	40.5
	SCCC	220x220	---	50	8 ϕ 14	ϕ 10 (Dm)	533	---	40.5

Dm=Diamond, Sq+ = Square with additional ties, Sq = Square

		Dimension (mm)	Cover (mm)	Spacing	Bar-Long	Tie	Fyb (Mpa)	Fyh (Mpa)	f'c (Mpa)
Aoude et al. 2009	A0	300x300	30	240	8 No. 15	No. 10(Sq)	515	409	49.5
	B0	300x300	30	120	8 No. 15	No. 10(Sq)	515	409	49.5
	C0	300x300	30	65	8 No. 15	No. 10(Sq)	515	409	49.5

Dm=Diamond, Sq+ = Square with additional ties , Sq = Square

Appendix B – Detailed Tables of Database for SFRC

Part I – Square Specimens

		Dimension (mm)	Cover (mm)	Spacing	Bar-Long	Tie	Fyb (Mpa)	Fyh (Mpa)	f'c (Mpa)	Vf (%)	L/φ
Ganesan 1990	R1	200	25	240	8 φ 12	φ 6 (Dm)	468.59	468.59	19.7	0	0
	R2	200	25	180	8 φ 12	φ 6 (Dm)	468.59	468.59	19.8	0	0
	R3	200	25	90	8 φ 12	φ 6 (Dm)	468.59	468.59	20.0	0	0
	R4	200	25	60	8 φ 12	φ 6 (Dm)	468.59	468.59	19.9	0	0
	F1	200	25	240	8 φ 12	φ 6 (Dm)	468.59	468.59	20.6	1.5	70
	F2	200	25	180	8 φ 12	φ 6 (Dm)	468.59	468.59	20.5	1.5	70
	F3	200	25	90	8 φ 12	φ 6 (Dm)	468.59	468.59	20.5	1.5	70
	F4	200	25	60	8 φ 12	φ 6 (Dm)	468.59	468.59	20.6	1.5	70

Dm=Diamond, Sq+ = Square with additional ties, Sq = Square

		Dimension (mm)	Cover (mm)	Spacing	Bar-Long	Tie	Fyb (Mpa)	Fyh (Mpa)	f'c (Mpa)	Vf(%)	L/φ
Masicotte 1999	C1	170	0	250	4-15M	10M (Sq)	525	475	40.5	0	0
	C2	170	0	125	4-15M	10M (Sq)	525	475	40.5	0	0
	C3	170	0	62.5	4-15M	10M (Sq)	525	475	40.5	0	0
	C4	170	0	250	4-15M	10M (Sq)	525	475	43.3	0.5	60
	C5	170	0	125	4-15M	10M (Sq)	525	475	43.3	0.5	60
	C6	170	0	62.5	4-15M	10M (Sq)	525	475	43.3	0.5	60
	C7	170	0	250	4-15M	10M (Sq)	525	475	35.5	1	60
	C8	170	0	125	4-15M	10M (Sq)	525	475	35.5	1	60
	C9	170	0	62.5	4-15M	10M (Sq)	525	475	35.5	1	60
	C10	250	40	250	4-15M	10M (Sq)	525	475	43.3	0.5	60
	C11	250	40	125	4-15M	10M (Sq)	525	475	43.3	0.5	60
	C12	250	40	62.5	4-15M	10M (Sq)	525	475	43.3	0.5	60
	C13	250	40	250	4-15M	10M (Sq)	525	475	35.5	1	60
	C14	250	40	125	4-15M	10M (Sq)	525	475	35.5	1	60
	C15	250	40	62.5	4-15M	10M (Sq)	525	475	35.5	1	60

Dm=Diamond, Sq+ = Square with additional ties, Sq = Square

		Dimension (mm)	Cover (mm)	Spacing	Bar-Long	Tie	Fyb (Mpa)	Fyh (Mpa)	f'c (Mpa)	Vf (%)	L/φ
Foster & Attard 2001	4HF0-30A	155	27.5	30	8 φ 12.2	φ 4(Sq)	450	600	90	2	83.72
	4HF0-60A	155	27.5	60	8 φ 12.2	φ 4(Sq)	450	600	90	2	83.72
	2MF0-50G	200	30	50	8 φ 12.2	φ 6.1(Dm)	450	470	70	2	83.72
	2MF0- 100G	200	30	100	8 φ 12.2	φ 6.1(Dm)	450	470	70	2	83.72
	2MF0-50S	200	30	50	8 φ 12.2	φ 6.1(Dm)	450	470	70	2	83.72
	2MF0-100S	200	30	100	8 φ 12.2	φ 6.1(Dm)	450	470	70	2	83.72

Dm=Diamond, Sq+ = Square with additional ties, Sq = Square

		Dimension (mm)	Cover (mm)	Spacing	Bar-Long	Tie	Fyb (Mpa)	Fyh (Mpa)	f'c (Mpa)	Vf (%)	L/φ
Lima 2003	P168150	150x150	20	150	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	68	0	80
	P268150	150x150	20	150	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	68	0	80
	P16850	150x150	20	50	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	68	0	80
	P26850	150x150	20	50	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	68	0	80
	P1681505	150x150	20	150	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	68	0.5	80
	P2681505	150x150	20	150	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	68	0.5	80
	P168505	150x150	20	50	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	68	0.5	80
	P268505	150x150	20	50	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	68	0.5	80
	P168151	150x150	20	150	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	68	1	80
	P268151	150x150	20	150	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	68	1	80
	P16851	150x150	20	50	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	68	1	80
	P26851	150x150	20	50	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	68	1	80
	P191150	150x150	20	150	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	91	0	80
	P291150	150x150	20	150	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	91	0	80
	P19150	150x150	20	50	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	91	0	80
	P29150	150x150	20	50	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	91	0	80
	P1911505	150x150	20	150	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	91	0.5	80
	P2911505	150x150	20	150	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	91	0.5	80
	P191505	150x150	20	50	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	91	0.5	80
	P291505	150x150	20	50	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	91	0.5	80
	P191151	150x150	20	150	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	91	1	80
	P291151	150x150	20	150	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	91	1	80
	P19151	150x150	20	50	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	91	1	80
	P29151	150x150	20	50	4 φ 12.3	φ 6.3 (Sq)	597.5	655.9	91	1	80

Dm=Diamond, Sq+ = Square with additional ties, Sq = Square

		Dimension (mm)	Cover (mm)	Spacing	Bar-Long	Tie	Fyb (Mpa)	Fyh (Mpa)	f'c (Mpa)	Vf (%)	L/φ
Zaina & Foster 2005	4HF0-50R6r	190	15	50	8 φ 16	φ 6 (Dm)	576	660	86	2	83.72
	4HF0-100R6r	190	15	100	8 φ 16	φ 6 (Dm)	576	660	86	2	83.72
	2HF0-150Z4r	190	15	150	8 φ 12	φ 4 (Dm)	560	660	86	2	83.72
	2HF0-150Z4	200	20	150	8 φ 12	φ 4 (Dm)	560	660	86	2	83.72
	2HF0-150Z6r	190	15	150	8 φ 12	φ 6 (Dm)	560	660	86	2	83.72
	2HF0-72M6r	190	15	72	8 φ 12	φ 6 (Dm)	560	660	90	2.6	83.72
	2HF0-72M6	200	20	72	8 φ 12	φ 6 (Dm)	560	660	90	2.6	83.72
	2HF0-100M6r	190	15	100	8 φ 12	φ 6 (Dm)	560	660	90	2.6	83.72
	2HF0-100M6	200	20	100	8 φ 12	φ 6 (Dm)	560	660	90	2.6	83.72
	2HF0-150M6r	190	15	150	8 φ 12	φ 6 (Dm)	560	660	90	2.6	83.72
	2HF0-150M6-1	200	20	150	8 φ 12	φ 6 (Dm)	560	660	90	2.6	83.72
	2HF0-150M6-2	200	20	150	8 φ 12	φ 6 (Dm)	560	660	90	2.6	83.72
	4HF0-100A6	200	20	100	8 φ 16	φ 6 (Dm)	576	660	91	1	83.72
	4HF0-150A6	200	20	150	8 φ 16	φ 6 (Dm)	576	660	91	1	83.72

Dm=Diamond, Sq+ = Square with additional ties, Sq = Square

		Dimension (mm)	Cover (mm)	Spacing	Bar-Long	Tie	Fyb (Mpa)	Fyh (Mpa)	f _c (Mpa)	V _f (%)	L/φ
Sharma 2007	SA1	150	10	50	4 φ 12	φ 8 (Sq)	395	412	62.2	0	0
	SA2	150	10	50	4 φ 12	φ 8 (Sq)	395	412	62.2	1.5	25
	SA3	150	10	50	4 φ 12	φ 8 (Sq)	395	412	62.2	1.5	37.5
	SA4	150	10	50	4 φ 12	φ 8 (Sq)	395	412	62.2	1.5	50
	SB1	150	10	75	4 φ 12	φ 8 (Sq)	395	412	62.8	0	0
	SB2	150	10	75	4 φ 12	φ 8 (Sq)	395	412	62.8	1.5	25
	SB3	150	10	75	4 φ 12	φ 8 (Sq)	395	412	62.8	1.5	50
	SC1	150	10	50	4 φ 12	φ 8 (Sq)	395	520	61.85	0	0
	SC2	150	10	50	4 φ 12	φ 8 (Sq)	395	520	61.85	1.5	25
	SC3	150	10	50	4 φ 12	φ 8 (Sq)	395	520	61.85	1.5	37.5
	SC4	150	10	50	4 φ 12	φ 8 (Sq)	395	520	61.85	1.5	50
	SD1	150	10	50	8 φ 12	φ 8 (Dm)	395	412	63.35	0	0
	SD2	150	10	50	8 φ 12	φ 8 (Dm)	395	412	63.35	1.5	25
	SD3	150	10	50	8 φ 12	φ 8 (Dm)	395	412	63.35	1.5	50
	SE1	150	10	30	4 φ 12	φ 8 (Sq)	395	412	82.5	0	0
	SE2	150	10	30	4 φ 12	φ 8 (Sq)	395	412	82.5	1.5	25
	SE3	150	10	30	4 φ 12	φ 8 (Sq)	395	412	82.5	1.5	50
	SF1	150	10	50	4 φ 12	φ 8 (Sq)	395	412	81.75	0	0
	SF2	150	10	50	4 φ 12	φ 8 (Sq)	395	412	81.75	1.5	25
	SF3	150	10	50	4 φ 12	φ 8 (Sq)	395	412	81.75	1.5	37.5
	SF4	150	10	50	4 φ 12	φ 8 (Sq)	395	412	81.75	1.5	50
	SF5	150	10	50	4 φ 12	φ 8 (Sq)	395	412	81.75	1	25
	SF6	150	10	50	4 φ 12	φ 8 (Sq)	395	412	81.75	1	50
	SF7	150	10	50	4 φ 12	φ 8 (Sq)	395	412	81.75	2	25

Dm=Diamond, Sq+ = Square with additional ties, Sq = Square

		Dimension (mm)	Cover (mm)	Spacing	Bar-Long	Tie	Fyb (Mpa)	Fyh (Mpa)	f'c (Mpa)	Vf (%)	L/φ
Sharma 2007	SG1	150	10	75	4 φ 12	φ 8 (Sq)	395	412	83.15	0	0
	SG2	150	10	75	4 φ 12	φ 8 (Sq)	395	412	83.15	1.5	25
	SG3	150	10	75	4 φ 12	φ 8 (Sq)	395	412	83.15	1.5	50
	SH1	150	10	50	4 φ 12	φ 8 (Sq)	395	520	81.8	0	0
	SH2	150	10	50	4 φ 12	φ 8 (Sq)	395	520	81.8	1.5	25
	SH3	150	10	50	4 φ 12	φ 8 (Sq)	395	520	81.8	1.5	50
	SI1	150	10	50	8 φ 12	φ 8 (Dm)	395	412	82.55	0	0
	SI2	150	10	50	8 φ 12	φ 8 (Dm)	395	412	82.55	1.5	25
	SI3	150	10	50	8 φ 12	φ 8 (Dm)	395	412	82.55	1.5	37.5
	SI4	150	10	50	8 φ 12	φ 8 (Dm)	395	412	82.55	1.5	50
	SI5	150	10	50	8 φ 12	φ 8 (Dm)	395	412	82.55	1	25
	SI6	150	10	50	8 φ 12	φ 8 (Dm)	395	412	82.55	1	50

Dm=Diamond, Sq+ = Square with additional ties, Sq = Square

		Dimension (mm)	Cover (mm)	Spacing	Bar-Long	Tie	Fyb (Mpa)	Fyh (Mpa)	f'c (Mpa)	Vf(%)	L/φ
Paultre 2010	3B	235	20	100	4 No.15 + 4 No. 10	φ 9.5 (Dm)	450	410	98.1	0	50
	3BF0.50-2	235	20	100	4 No.15 + 4 No. 10	φ 9.5 (Dm)	419.5	409	88.4	0.5	50
	3BF1.0	235	20	100	4 No.15 + 4 No. 10	φ 9.5 (Dm)	460.5	410	88.4	1	50
	4B	235	20	50	4 No.20 + 4 No. 15	φ 7.9 (Dm)	428	392	93.1	0	50
	4BF0.25	235	20	50	4 No.20 + 4 No. 15	φ 7.9 (Dm)	512.5	428	88.4	0.25	50
	4BF0.50	235	20	50	4 No.20 + 4 No. 15	φ 7.9 (Dm)	512.5	428	88.4	0.5	50
	4BF0.75	235	20	50	4 No.20 + 4 No. 15	φ 7.9 (Dm)	433	442	88.4	0.75	50
	4BF1.0	235	20	50	4 No.20 + 4 No. 15	φ 7.9 (Dm)	470.5	428	88.4	1	50
	5B	235	20	50	4 No.20 + 4 No. 15	φ 7.9 (Dm)	428	770	99.9	0	50
	5BF0.25	235	20	50	4 No.20 + 4 No. 15	φ 7.9 (Dm)	512.5	745	88.4	0.25	50
	5BF0.50	235	20	50	4 No.20 + 4 No. 15	φ 7.9 (Dm)	512.5	745	88.4	0.5	50
	5BF0.75	235	20	50	4 No.20 + 4 No. 15	φ 7.9 (Dm)	433	856	88.4	0.75	50
	5BF1.0	235	20	50	4 No.20 + 4 No. 15	φ 7.9 (Dm)	470.5	745	88.4	1	50
	9BF0.50-2	235	20	75	4 No.20 + 4 No. 15	φ 9.5 (Dm)	464.5	409	88.4	0.5	50
	9BF1.0	235	20	75	4 No.20 + 4 No. 15	φ 9.5 (Dm)	435.5	410	88.4	1	50

Dm=Diamond, Sq+ = Square with additional ties, Sq = Square

Part II – Circular Sections

		Diameter (mm)	Cover (mm)	Spacing	ϕ_b	ϕ_h	Fyb (Mpa)	Fyh (Mpa)	f'c (Mpa)	Vf(%)	L/ ϕ
Sharma et al. 2006	CE1	150	10	30	8	8	412	412	82.5	0	0
	CE2	150	10	30	8	8	412	412	82.5	1.5	20.16129
	CE3	150	10	30	8	8	412	412	82.5	1.5	40.32258
	CF1	150	10	50	8	8	412	412	81.75	0	0
	CF2	150	10	50	8	8	412	412	81.75	1.5	20.16129
	CF3	150	10	50	8	8	412	412	81.75	1.5	30.24
	CF4	150	10	50	8	8	412	412	81.75	1.5	40.32258
	CF5	150	10	50	8	8	412	412	81.75	1	20.16129
	CF6	150	10	50	8	8	412	412	81.75	1	40.32258
	CF7	150	10	50	8	8	412	412	81.75	2	20.16129
	CG1	150	10	75	8	8	412	412	83.15	0	0
	CG2	150	10	75	8	8	412	412	83.15	1.5	20.16129
	CG3	150	10	75	8	8	412	412	83.15	1.5	40.32258

		Diameter (mm)	Cover (mm)	Spacing	ϕ_b	ϕ_h	Fyb (Mpa)	Fyh (Mpa)	f'c (Mpa)	Vf(%)	L/ ϕ
Djumbong et al. 2008	C50-S50	300	25	50	19.5	11.3	494	459	82.50	0	198*
	C50-S50-FP0.5	300	25	50	19.5	11.3	494	459	82.50	0.5	198*
	C50-S50-FP1.0	300	25	50	19.5	11.3	494	459	82.50	1	198*
	C50-S100	300	25	100	19.5	11.3	494	459	81.75	0	198*
	C50-S100-FP0.5	300	25	100	19.5	11.3	494	459	81.75	0.5	198*
	C50-S100-FP1.0	300	25	100	19.5	11.3	494	459	81.75	1	198*
	C80-S50	300	25	50	19.5	11.3	494	459	81.75	0	198*
	C80-S50-FP0.5	300	25	50	19.5	11.3	494	459	81.75	0.5	198*
	C80-S50-FP1.0	300	25	50	19.5	11.3	494	459	81.75	1	198*
	C80-S100	300	25	100	19.5	11.3	494	459	81.75	0	198*
	C80-S100-FP0.5	300	25	100	19.5	11.3	494	459	83.15	0.5	198*
	C80-S100-FP1.0	300	25	100	19.5	11.3	494	459	83.15	1	198*

*-Rectangular section fibre which is assumed to have equivalent circular section .the original dimension of section is 0.05 mm by 1 mm