

Blast Retrofit of Reinforced Concrete Columns

Alan Eric Walker Lloyd

Thesis submitted to the
Faculty of Graduate and Postdoctoral Studies
in partial fulfillment of the requirements
for the Doctorate in Philosophy degree in Civil Engineering

Department of Civil Engineering
Faculty of Engineering
University of Ottawa



uOttawa

Abstract

Explosives place large demands on the lateral load carrying capacity of structures. If these loads are applied on columns, the high pressure transient loads from explosives can result in significant damage to the primary gravity load carrying elements. The loss of these elements, which are responsible from overall strength and stability of the structure, may cause collapse of all or parts of the structure. Therefore, it is important to mitigate the blast loads effects on columns. A comprehensive research study into the design, application, and use of different retrofit systems to mitigate damage to columns under blast loads has been undertaken. This research program, consisting of experimental testing and analytical investigation, sought out retrofits that address the strength of columns as well as those that enhance ductility are explored. Different materials and resistance mechanisms are used to increase column capacity.

An experimental testing program was conducted using a shock tube to test the capacity of columns under blast loads. For this program, a total of sixteen reinforced concrete columns were constructed and the data from a further two columns from a previous study was compiled. Of these columns, a total of thirteen were retrofitted to mitigate the effects of blast. Carbon fibre reinforced polymer (CFRP) was applied to eight of the columns in the form of jacketing, longitudinal reinforcement, or the combination of the two. The other retrofits included steel prestressed confinement applied to one column, steel bracing acting as compression members applied to one column, and steel bracing acting as tension members applied to three columns. The columns were tested under incrementally increasing shock tube induced shock wave loading up to failure of the specimen or capacity of the shock tube. The performance of the retrofitted columns was compared with the control columns and against other retrofits. Quantitative comparisons of displacements and strains were made along with qualitative assessments of damage. The results indicated that all the retrofits increased capacity to the column, however, certain retrofits out performed others. The best FRP retrofit technique was found to be the combination of longitudinal and transverse FRP. The prestressed steel jacketing proved to be effective at increasing ductility capacity of the column. The compression brace retrofit was found to be effective in significantly increasing capacity of the column. The tension brace retrofits had the best performance over all the retrofits including the compression brace retrofit.

The experimental data was used to validate analysis techniques to model the behaviour of the specimens. This technique reduced the columns to an equivalent single-degree-of-freedom (SDOF)

system for dynamic analysis purposes. The reduction to the SDOF system was achieved by computing a resistance to lateral load and lateral displacement relationship. Each retrofit was carefully considered in this analysis including the retrofit's possible effect on material and sectional properties as well as any force resistance mechanism that the retrofit introduces. The results of the modeling and experimental program were used to develop retrofit design guidelines. These guidelines are presented in detail in this thesis.

Acknowledgements

My wife, Réne Beaubien, has always been a great supporter of my studies. Her support, and the motivation I received from my two daughters, Sidney and Erin, allowed me to finish this endeavour. First and foremost I thank them.

My two supervisors, Dr. Murat Saatcioglu and Dr. Timo Tikka, have been guiding me for many years. The knowledge, advice, inspiration, and motivation they have provided was exceptional. I thank them both for providing the guidance I needed for this research and many other projects.

I would also like to thank my fellow graduate student, Mr. Eric Jacques, for his laboratory assistance over the course of this project.

Finally, I would like to acknowledge the financial support from NSERC and the University of Ottawa that allowed me to focus my time on this research.

Table of Contents

Chapter 1.	<i>Introduction</i>	<i>1</i>
1.1.	<i>General</i>	<i>1</i>
1.2.	<i>Objectives</i>	<i>2</i>
1.3.	<i>Scope</i>	<i>2</i>
1.4.	<i>Brief Description of Contents</i>	<i>3</i>
1.5.	<i>Research Significance</i>	<i>4</i>
Chapter 2.	<i>Background and Literature Review</i>	<i>6</i>
2.1.	<i>General</i>	<i>6</i>
2.2.	<i>Explosions and Explosive (Blast) Loads on Structures.....</i>	<i>6</i>
2.3.	<i>History of Accidental and Malicious Explosions.....</i>	<i>7</i>
2.3.1.	<i>The Alfred P. Murrah Federal Building - 1995.....</i>	<i>7</i>
2.3.2.	<i>The World Trade Center - 1992.....</i>	<i>8</i>
2.3.3.	<i>Jewish Community Centre, Buenos Aires - 1994</i>	<i>8</i>
2.3.4.	<i>St. Mary Axe, London - 1992.....</i>	<i>8</i>
2.3.5.	<i>Bishopgate, London - 1992</i>	<i>8</i>
2.3.6.	<i>Docklands, London - 1996.....</i>	<i>8</i>
2.3.7.	<i>Al-Khobar, Saudi Arabia - 1996.....</i>	<i>8</i>
2.4.	<i>Experimental Case Studies of Explosions on Reinforced Concrete Columns and Flexure Members</i>	<i>9</i>
2.4.1.	<i>S.C. Woodson and J.T. Baylot.....</i>	<i>9</i>
2.4.2.	<i>J. Magnusson</i>	<i>11</i>
2.4.3.	<i>L.C Muszynski and M.R. Purcell.....</i>	<i>12</i>
2.4.4.	<i>A. Schenker et al.....</i>	<i>13</i>
2.4.5.	<i>F. Toutlemonde et al.</i>	<i>13</i>
2.4.6.	<i>A.G. Razaqpur, A. Tolba and E. Contestabile</i>	<i>13</i>
2.4.7.	<i>E. Jacques.....</i>	<i>14</i>
2.4.8.	<i>A. Lloyd.....</i>	<i>15</i>
2.4.9.	<i>R. Burrell.....</i>	<i>16</i>
2.4.10.	<i>P.F Silva and B. Lu</i>	<i>16</i>
2.4.11.	<i>C.A. Ross, M.R. Purcell, and E.L. Jerome,</i>	<i>17</i>
2.4.12.	<i>M. Carriere et al.</i>	<i>18</i>
2.4.13.	<i>J.O. Berger, P.J. Heffernan, and R.G. Wight</i>	<i>18</i>
2.4.14.	<i>C. Wu et al.....</i>	<i>19</i>
2.4.15.	<i>T. Rodriguez-Nikl et al.</i>	<i>20</i>
2.4.16.	<i>C.P. Pantelides et al.....</i>	<i>20</i>
2.4.17.	<i>C.E. Davis et al.....</i>	<i>21</i>
2.4.18.	<i>P.A. Buchan and J.F. Chen</i>	<i>22</i>

2.4.19.	<i>J. E. Crawford et al.</i>	22
2.5.	<i>Previous Research on FRP Strengthening Retrofits for Flexure</i>	23
2.5.1.	<i>W.Seim et al.</i>	24
2.5.2.	<i>M. Maalej and K.S. Leong</i>	25
2.5.3.	<i>R. Kotynia et al.</i>	26
2.5.4.	<i>H.B. Pham and R. Al-Mahaidi</i>	27
2.5.5.	<i>H. Niu, A. Vasques, and V.M. Karbhari</i>	27
2.5.6.	<i>S. Lee</i>	28
2.5.7.	<i>S.P. Chiew, Q. Sun, and Y. Yu</i>	28
2.5.8.	<i>E. Ahmed, H.R. Sobuz, and N.M. Sutan</i>	29
2.5.9.	<i>M. Arduini and A. Nanni</i>	29
2.5.10.	<i>G. Spadea, F. Bencardino, and R.N. Swamy</i>	30
2.5.11.	<i>J.F. Bonacci and M. Maalej</i>	30
2.5.12.	<i>M.J. Chajes et al.</i>	30
2.5.13.	<i>J. Pan, T.C.F. Chung, and C.K.Y. Leung</i>	31
2.5.14.	<i>J. Pan, C.K.Y. Leung, and M. Luo</i>	31
2.6.	<i>Previous Research on Prestress Jacketing for Confinement</i>	32
2.7.	<i>Conclusions from Previous Research</i>	32
Chapter 3.	<i>Experimental Program</i>	33
3.1.	<i>General</i>	33
3.2.	<i>Description of Test Specimens</i>	33
3.3.	<i>Description of Column Groups</i>	34
3.3.1.	<i>Group 1 Columns</i>	34
3.3.2.	<i>Group 2 Columns</i>	35
3.3.3.	<i>Group 3 Columns</i>	36
3.4.	<i>Description of Test Setup</i>	37
3.4.1.	<i>Shock Tube Operation</i>	37
3.4.2.	<i>Shock Tube Capabilities</i>	38
3.4.3.	<i>Axial Load Mechanism</i>	39
3.4.4.	<i>Lateral Load Transferring Devices</i>	39
3.4.4.1.	<i>LTD A – Group 1 Columns</i>	39
3.4.4.2.	<i>LTD B – Group 2 and Group 3 Columns</i>	39
3.4.5.	<i>Support Conditions</i>	40
3.4.5.1.	<i>Support A – Group 1 Columns</i>	40
3.4.5.2.	<i>Support B – Group 2 and Group 3 Columns</i>	40
3.5.	<i>Construction of Specimens</i>	41
3.6.	<i>Material Properties</i>	41

3.6.1.	Concrete	41
3.6.2.	Reinforcing Steel	42
3.6.3.	CFRP	42
3.6.4.	Confinement Prestress Material	42
3.6.4.1.	Concrete	42
3.6.4.2.	Steel.....	43
3.6.5.	Compression Brace Material.....	43
3.6.6.	Tension Brace Material	43
3.6.6.1.	Threaded Brace Rods.....	43
3.6.6.2.	Connector Elements and Bearing Plates	43
3.6.6.3.	Shear Pins.....	43
3.7.	Retrofit Procedures	44
3.7.1.	Externally Bonded CFRP	44
3.7.2.	External Steel Prestressing.....	44
3.7.3.	Compression Brace.....	46
3.7.4.	Tension Brace.....	46
3.8.	Instrumentation and Data Acquisition	48
3.9.	Test Procedure and Loading Program	48
Chapter 4.	Experimental Results and Discussion	88
4.1.	General	88
4.1.1.	Load Transferring Device	88
4.1.2.	Supports.....	88
4.1.3.	Axial Load.....	89
4.2.	Experimental Results	89
4.2.1.	Column 1NS-C	90
4.2.1.1.	1NS-C-1.....	90
4.2.1.2.	1NS-C-2.....	91
4.2.1.3.	1NS-C-3.....	91
4.2.2.	Column 1S-C.....	92
4.2.2.1.	1S-C-1	92
4.2.2.2.	1S-C-2	93
4.2.2.3.	1S-C-3	93
4.2.3.	Column 1NS-J	94
4.2.3.1.	1NS-J-1	94
4.2.3.2.	1NS-J-2	95
4.2.3.3.	1NS-J-3	96
4.2.4.	Column 1S-J.....	97

4.2.4.1.	1S-J-1	97
4.2.4.2.	1S-J-2	98
4.2.4.3.	1S-J-3	98
4.2.5.	Column 2NS-C	99
4.2.5.1.	2NS-C-1.....	100
4.2.5.2.	2NS-C-2.....	100
4.2.5.3.	2NS-C-3.....	101
4.2.6.	Column 2S-C.....	102
4.2.6.1.	2S-C-1	102
4.2.6.2.	2S-C-2	103
4.2.6.3.	2S-C-3	104
4.2.7.	Column 2NS-J	105
4.2.7.1.	2NS-J-1	105
4.2.7.2.	2NS-J-2	106
4.2.7.3.	2NS-J-3	106
4.2.7.4.	2NS-J-4	107
4.2.8.	Column 2S-J.....	107
4.2.8.1.	2S-J-1	108
4.2.8.2.	2S-J-2	108
4.2.8.3.	2S-J-3	109
4.2.8.4.	2S-J-4	110
4.2.9.	Column 2NS-R	111
4.2.9.1.	2NS-R-1.....	111
4.2.9.2.	2NS-R-2.....	112
4.2.9.3.	2NS-R-3.....	112
4.2.10.	Column 2S-R.....	113
4.2.10.1.	2S-R-1	114
4.2.10.2.	2S-R-2	114
4.2.10.3.	2S-R-3	115
4.2.11.	Column 2NS-JR	116
4.2.11.1.	2NS-JR-1	117
4.2.11.2.	2NS-JR-2	117
4.2.11.3.	2NS-JR-3	118
4.2.12.	Column 2S-JR.....	119
4.2.12.1.	2S-JR-1.....	119
4.2.12.2.	2S-JR-2.....	120
4.2.12.3.	2S-JR-3.....	121

4.2.13.	Column 3NS-C	122
4.2.13.1.	3NS-C-1.....	122
4.2.13.2.	3NS-C-2.....	123
4.2.13.3.	3NS-C-3.....	124
4.2.14.	Column 3NS-W.....	125
4.2.14.1.	3NS-W-1	125
4.2.14.2.	3NS-W-2	126
4.2.14.3.	3NS-W-3	127
4.2.14.4.	3NS-W-4	129
4.2.15.	Column 3NS-CB	130
4.2.15.1.	3NS-CB-1	131
4.2.15.2.	3NS-CB-2	132
4.2.15.3.	3NS-CB-3	133
4.2.15.4.	3NS-CB-4	134
4.2.15.5.	3NS-CB-5	135
4.2.16.	Column 3NS-TB1	136
4.2.16.1.	3NS-TB1-1.....	137
4.2.16.2.	3NS-TB1-2.....	137
4.2.16.3.	3NS-TB1-3.....	138
4.2.16.4.	3NS-TB1-4.....	139
4.2.16.5.	3NS-TB1-5.....	140
4.2.17.	Column 3NS-TB2	141
4.2.17.1.	3NS-TB2-1.....	142
4.2.17.2.	3NS-TB2-2.....	143
4.2.17.3.	3NS-TB2-3.....	143
4.2.17.4.	3NS-TB2-4.....	144
4.2.17.5.	3NS-TB2-5.....	145
4.2.18.	Column 3NS-TB3	146
4.2.18.1.	3NS-TB3-1.....	147
4.2.18.2.	3NS-TB3-2.....	147
4.2.18.3.	3NS-TB3-3.....	148
4.2.18.4.	3NS-TB3-4.....	149
4.2.18.5.	3NS-TB3-5.....	150
4.3.	Repeatability of Pressure and Impulse	151
4.4.	Axial Load Variation.....	152
4.5.	Comparison of Column Response	153
4.5.1.	Group 1 Columns.....	153

4.5.2.	Group 2 Columns.....	155
4.5.2.1.	Non-Seismic Columns	155
4.5.2.2.	Seismic Columns.....	156
4.5.2.3.	Control Columns.....	158
4.5.2.4.	Jacketed Columns.....	158
4.5.2.5.	FRP Reinforced Columns.....	159
4.5.2.6.	FRP Reinforced and Jacketed Columns.....	160
4.5.3.	Jacketed and Hoop Prestressed Columns.....	161
4.5.4.	Braced Columns	163
4.6.	Support Rigidity	166
Chapter 5.	Single Degree of Freedom Modeling.....	241
5.1.	General	241
5.2.	Single-Degree-of-Freedom Analysis.....	241
5.2.1.	Numerical Integration of the Equation of Motion	243
5.2.2.	Loaded Area and System Mass	243
5.2.3.	Hysteretic Model.....	243
5.3.	Resistance-Displacement Relationships.....	244
5.3.1.	Material Models.....	244
5.3.1.1.	Unconfined Concrete.....	244
5.3.1.2.	Steel Hoop Confined Concrete.....	245
5.3.1.3.	FRP Confinement Pressure Model Modification.....	246
5.3.1.4.	Confinement Pressure Model Modification for Prestress Steel	246
5.3.1.5.	Reinforcing Steel.....	247
5.3.1.5.1.	Steel in Tension	247
5.3.1.5.2.	Steel in Compression.....	248
5.3.1.6.	Bracing Steel.....	248
5.3.1.6.1.	Threaded Rod for Tension Brace.....	249
5.3.1.6.2.	Mild Steel for Compression Brace	249
5.3.1.7.	FRP	249
5.3.2.	Dynamic Increase Factors	249
5.3.3.	Sectional Analysis for Moment-Curvature	250
5.3.4.	Resistance-Displacement Curve Development.....	252
5.3.4.1.	Plasticity	252
5.3.4.2.	Development of Resistance Curves for Non-Braced Columns	253
5.3.4.2.1.	Variable Axial Load Resistance Curves for Non-Braced Columns.....	254
5.3.5.	Development of Resistance Curve for Column 3NS-CB (Compression Brace)	257
5.3.6.	Development of Resistance Curves for Columns 3NS-TB1 through 3NS-TB3 (Tension Brace) 259	

5.3.7.	<i>Composite Resistance Curves for Braced Columns</i>	261
5.4.	<i>Displacement Predictions for Columns</i>	264
5.4.1.	<i>Displacement Predictions based on Magnitude of Displacement</i>	264
5.4.2.	<i>Group 1 Column Displacement Predictions</i>	264
5.4.3.	<i>Group 2 Column Displacement Predictions</i>	265
5.4.4.	<i>Control Column Predictions</i>	265
5.4.5.	<i>Group 2 Seismic and Non-Seismic Columns</i>	265
5.4.6.	<i>Tension Braced Columns</i>	265
5.4.7.	<i>Compression Braced Columns</i>	266
5.4.8.	<i>Columns Retrofitted for Confinement</i>	266
5.4.9.	<i>FRP Strengthened Columns</i>	267
5.4.10.	<i>Validation of Composite Curves</i>	267
5.5.	<i>Sources of Error</i>	268
5.5.1.	<i>Experimental Error</i>	268
5.5.1.1.	<i>Movement of the Shock Tube</i>	268
5.5.1.2.	<i>Loose or Flexible Displacement Gauge Mounts</i>	269
5.5.1.3.	<i>Gap between the Load Transfer Device (LTD) and Column Surface</i>	269
5.5.2.	<i>Analytical Error</i>	270
5.5.2.1.	<i>Rebound into Negative Displacements</i>	270
5.5.2.2.	<i>Load transfer device separation on rebound</i>	270
5.5.2.3.	<i>Hysteresis and Yield Points</i>	270
5.6.	<i>Summary</i>	270
Chapter 6.	<i>Design Procedures and Recommendations</i>	314
6.1.	<i>General</i>	314
6.2.	<i>Consideration of Axial Load Variation</i>	314
6.3.	<i>Design Procedures for FRP Retrofits</i>	315
6.3.1.	<i>General FRP Retrofit Guidelines</i>	315
6.3.2.	<i>Strengthening with Longitudinal FRP</i>	316
6.3.3.	<i>Confinement with Transverse FRP</i>	316
6.3.3.1.	<i>FRP Jacket Design</i>	318
6.3.4.	<i>Strengthening and Confinement with Longitudinal and Transverse FRP</i>	321
6.4.	<i>Design Guidelines for Prestress Confinement</i>	322
6.5.	<i>Design Guidelines for Bracing</i>	323
6.5.1.	<i>Tension Brace Retrofit Guidelines</i>	324
6.5.1.1.	<i>Tension Brace Retrofit Design Assumptions</i>	325
6.5.1.2.	<i>Simplified Design Procedure</i>	326
6.5.2.	<i>Compression brace guidelines</i>	331

6.5.2.1.	<i>Simplified Design Procedure</i>	331
Chapter 7.	<i>Conclusions</i>	338
7.1.	<i>General</i>	338
7.2.	<i>Summary</i>	338
7.3.	<i>Conclusions</i>	339
7.4.	<i>Recommendations for Future Research</i>	345
	<i>References</i>	347
Appendix A -	<i>Pressure and Displacement Time Histories</i>	353
Appendix B -	<i>Strain Time Histories</i>	394
Appendix C -	<i>SDOF Displacement Predictions</i>	422

List of Tables

<i>Table 3-1: Column properties</i>	<i>50</i>
<i>Table 3-2: Maximum Shock Tube Capacities.....</i>	<i>52</i>
<i>Table 4-1: Pressure, Impulse, Maximum Displacement, and Time to Maximum Displacement Values.....</i>	<i>168</i>
<i>Table 4-2: Strain Values for 1NS-C</i>	<i>171</i>
<i>Table 4-3: Strain Values for 1S-C.....</i>	<i>171</i>
<i>Table 4-4: Strain Values for 1NS-J</i>	<i>172</i>
<i>Table 4-5: Strain Values for 1S-J.....</i>	<i>172</i>
<i>Table 4-6: Strain Values for 2NS-C</i>	<i>173</i>
<i>Table 4-7: Strain Values for 2S-C.....</i>	<i>173</i>
<i>Table 4-8: Strain Values for 2NS-J</i>	<i>174</i>
<i>Table 4-9: Strain Values for 2S-J.....</i>	<i>174</i>
<i>Table 4-10: Strain Values for 2NS-R</i>	<i>175</i>
<i>Table 4-11: Strain Values for 2S-R.....</i>	<i>175</i>
<i>Table 4-12: Strain Values for 2NS-JR</i>	<i>176</i>
<i>Table 4-13: Strain Values for 2S-JR.....</i>	<i>176</i>
<i>Table 4-14: Strain Values for 3NS-C</i>	<i>177</i>
<i>Table 4-15: Strain Values for 3NS-W.....</i>	<i>178</i>
<i>Table 4-16: Strain Values for 3NS-CB</i>	<i>179</i>
<i>Table 4-17: Strain Values for 3NS-TB1</i>	<i>180</i>
<i>Table 4-18: Strain Values for 3NS-TB2</i>	<i>181</i>
<i>Table 4-19: Strain Values for 3NS-TB3</i>	<i>182</i>
<i>Table 5-1: Mass and Loaded Area of Columns for SDOF Analysis</i>	<i>271</i>
<i>Table 5-2: Shock Wave Properties, Experimental Displacements and Time to Maximum Displacements, and SDOF Predicted Maximum Displacements and Time to Maximum Displacements</i>	<i>272</i>
<i>Table 5-3: Summary of Experimental and SDOF Predicted Responses for Various Column Groups.....</i>	<i>275</i>

List of Figures

<i>Figure 3-1: Details of Group 1 Non-Seismic Columns</i>	<i>53</i>
<i>Figure 3-2: Details of Group 1 Seismic Columns</i>	<i>54</i>
<i>Figure 3-3: Details of Group 2 Non-Seismic Columns</i>	<i>55</i>
<i>Figure 3-4: Details of Group 2 Seismic Columns</i>	<i>56</i>
<i>Figure 3-5: Details of Column 3-NS-C</i>	<i>57</i>
<i>Figure 3-6: Details of Column 3-NS-W</i>	<i>58</i>
<i>Figure 3-7: Details of Column 3-NS-CB</i>	<i>59</i>
<i>Figure 3-8: Details of Columns 3-NS-TB1 and 3NS-TB3</i>	<i>60</i>
<i>Figure 3-9: Details of Column 3-NS-TB2</i>	<i>61</i>
<i>Figure 3-10: Driver Section, Diaphragms, and Expansion Section of Shock Tube</i>	<i>62</i>
<i>Figure 3-11: Shock Tube Opening, Test Frame and Expansion system</i>	<i>62</i>
<i>Figure 3-12: Diaphragm Section and Aluminum Diaphragms</i>	<i>63</i>
<i>Figure 3-13: Range of Shock Tube Capacities</i>	<i>63</i>
<i>Figure 3-14: Relationship between Driver Pressure and Reflected Pressure</i>	<i>64</i>
<i>Figure 3-15: Relationship between Driver Pressure and Reflected Impulse</i>	<i>64</i>
<i>Figure 3-16: Axial Loading Mechanism</i>	<i>65</i>
<i>Figure 3-17: Load Transfer Device for Group 1 Columns – LTD A</i>	<i>66</i>
<i>Figure 3-18: Load Transfer Device for Group 2 and Group 3 Columns – LTD B</i>	<i>67</i>
<i>Figure 3-19: Column End Support for Group 1 Columns – Support A</i>	<i>68</i>
<i>Figure 3-20: Group 1 Column Test Setup showing Supports A, Column, Load Transfer Device A, Shock Tube and Installed Column and Assembled Test Apparatus</i>	<i>69</i>
<i>Figure 3-21: Column End Supports for Group 2 and Group 3 Columns – Support B</i>	<i>70</i>
<i>Figure 3-22: Group 2 Column Test Setup showing Supports B, Column, Load Transfer Device B, Shock Tube and Installed Column and Assembled Test Apparatus</i>	<i>71</i>
<i>Figure 3-23: Group 3 Column Test Setup showing Supports B, Column, Load Transfer Device B, Shock Tube and Installed Column and Assembled Test Apparatus</i>	<i>72</i>
<i>Figure 3-24: Reinforcing Steel Material Properties</i>	<i>73</i>
<i>Figure 3-25: CFRP Material Properties</i>	<i>73</i>
<i>Figure 3-26: Prestress Strap Material Properties</i>	<i>74</i>
<i>Figure 3-27: Tension Brace Threaded Rod Material Properties</i>	<i>74</i>
<i>Figure 3-28: Construction of Fibre Reinforced Concrete Wedges</i>	<i>75</i>
<i>Figure 3-29: Prestress Straps Prior to Installation</i>	<i>76</i>
<i>Figure 3-30: Installation of Fibre Reinforced Concrete Wedges and Prestressed Straps</i>	<i>77</i>
<i>Figure 3-31: Two Views of Compression Brace Components showing Diagonal Components, Vertical Components and Bearing Plates</i>	<i>78</i>
<i>Figure 3-32: Compression Brace Components Front View</i>	<i>79</i>
<i>Figure 3-33: Compression Brace Components Side View</i>	<i>80</i>
<i>Figure 3-34: Installation of Compression Braces and Fibre Reinforced Concrete Wedges</i>	<i>81</i>
<i>Figure 3-35: Construction of Tension Brace Components</i>	<i>82</i>
<i>Figure 3-36: Interior Tension Brace Hinge Components and Bearing Plate for Columns 3-NS-TB1 and 3NSTB3</i>	<i>83</i>
<i>Figure 3-37: Interior Tension Brace Hinge Components and Bearing Plate for Columns 3-NS-TB2</i>	<i>84</i>
<i>Figure 3-38: End Tension Brace Hinge and Component and Bearing Connector</i>	<i>85</i>
<i>Figure 3-39: Tension Brace Turnbuckle</i>	<i>86</i>

<i>Figure 3-40: Installation of Tension Braces</i>	<i>87</i>
<i>Figure 4-1: Column 1NS-C After Shot 1, Shot 2, and Shot 3</i>	<i>183</i>
<i>Figure 4-2: Column 1S-C After Shot 1, Shot 2, and Shot 3</i>	<i>184</i>
<i>Figure 4-3: Column 1NS-J Before Testing and After Shot 1 and Shot 2</i>	<i>185</i>
<i>Figure 4-4: Column 1NS-J After Shot 3</i>	<i>186</i>
<i>Figure 4-5: Column 1S-J Before Testing and After Shot 1 and Shot 2</i>	<i>187</i>
<i>Figure 4-6: Column 1S-J After Shot 3</i>	<i>188</i>
<i>Figure 4-7: Column 2NS-C Before Testing and After Shot 1 and Shot 2</i>	<i>189</i>
<i>Figure 4-8: Column 2NS-C After Shot 3</i>	<i>190</i>
<i>Figure 4-9: Column 2S-C Before Testing and After Shot 1 and Shot 2</i>	<i>191</i>
<i>Figure 4-10: Column 2S-C After Shot 3</i>	<i>192</i>
<i>Figure 4-11: Column 2NS-J Before Testing and After Shot 1 and Shot 2</i>	<i>193</i>
<i>Figure 4-12: Column 2NS-J After Shot 3</i>	<i>194</i>
<i>Figure 4-13: Column 2NS-J After Shot 4</i>	<i>195</i>
<i>Figure 4-14: Column 2S-J Before Testing and After Shot 1 and Shot 2</i>	<i>196</i>
<i>Figure 4-15: Column 2S-J After Shot 3</i>	<i>197</i>
<i>Figure 4-16: Column 2S-J After Shot 4</i>	<i>198</i>
<i>Figure 4-17: Column 2NS-R Before Testing and After Shot 1 and Shot 2</i>	<i>199</i>
<i>Figure 4-18: Column 2NS-R After Shot 3</i>	<i>200</i>
<i>Figure 4-19: Column 2S-R Before Testing and After Shot 1 and Shot 2</i>	<i>201</i>
<i>Figure 4-20: Column 2S-R After Shot 3</i>	<i>202</i>
<i>Figure 4-21: Column 2NS-JR Before Testing and After Shot 1 and Shot 2</i>	<i>203</i>
<i>Figure 4-22: Column 2NS-JR After Shot 3</i>	<i>204</i>
<i>Figure 4-23: Column 2S-JR Before Testing and After Shot 1 and Shot 2</i>	<i>205</i>
<i>Figure 4-24: Column 2S-JR After Shot 3</i>	<i>206</i>
<i>Figure 4-25: Column 3NS-C Before Testing and After Shot 1 and Shot 2</i>	<i>207</i>
<i>Figure 4-26: Column 3NS-C After Shot 3</i>	<i>208</i>
<i>Figure 4-27: Column 3NS-W Before Testing and After Shot 1 and Shot 2</i>	<i>209</i>
<i>Figure 4-28: Column 3NS-W After Shot 3</i>	<i>210</i>
<i>Figure 4-29: Column 3NS-W After Shot 4</i>	<i>211</i>
<i>Figure 4-30: Column 3NS-CB Before Testing and After Shot 1 and Shot 2</i>	<i>212</i>
<i>Figure 4-31: Column 3NS-CB After Shot 3 and Shot 4</i>	<i>213</i>
<i>Figure 4-32: Column 3NS-CB After Shot 5</i>	<i>214</i>
<i>Figure 4-33: Column 3NS-TB1 Before Testing and After Shot 1 and Shot 2</i>	<i>215</i>
<i>Figure 4-34: Column 3NS-TB1 After Shot 3 and Shot 4</i>	<i>216</i>
<i>Figure 4-35: Column 3NS-TB1 After Shot 5</i>	<i>217</i>
<i>Figure 4-36: Column 3NS-TB2 Before Testing and After Shot 1 and Shot 2</i>	<i>218</i>
<i>Figure 4-37: Column 3NS-TB2 After Shot 3 and Shot 4</i>	<i>219</i>
<i>Figure 4-38: Column 3NS-TB2 After Shot 5</i>	<i>220</i>
<i>Figure 4-39: Column 3NS-TB3 Before Testing and After Shot 1 and Shot 2</i>	<i>221</i>
<i>Figure 4-40: Column 3NS-TB3 After Shot 3 and Shot 4</i>	<i>222</i>
<i>Figure 4-41: Column 3NS-TB3 After Shot 5</i>	<i>223</i>
<i>Figure 4-42: Repeatability of Reflected Pressure</i>	<i>224</i>
<i>Figure 4-43: Repeatability of Reflected Impulse</i>	<i>224</i>
<i>Figure 4-44: Axial Displacements and Mid-Height Displacements for Column 2NS-C-2</i>	<i>225</i>
<i>Figure 4-45: Axial Load Variation versus Mid-Height Displacement for Column 2NS-C-2</i>	<i>225</i>

Figure 4-46: Comparison of Displacements - Group 1 Shot 1.....	226
Figure 4-47: Comparison of Displacements - Group 1 Shot 2.....	226
Figure 4-48: Comparison of Displacements - Group 1 Shot 3.....	227
Figure 4-49: Comparison of Maximum Displacements - Group 1	227
Figure 4-50: Comparison of Residual Displacements - Group 1	228
Figure 4-51: Comparison of Displacements - Group 2 Non-Seismic Shot 1	228
Figure 4-52: Comparison of Displacements - Group 2 Non-Seismic Shot 2	229
Figure 4-53: Comparison of Displacements - Group 2 Non-Seismic Shot 3	229
Figure 4-54: Comparison of Displacements - Group 2 Seismic Shot 1	230
Figure 4-55: Comparison of Displacements - Group 2 Seismic Shot 2	230
Figure 4-56: Comparison of Displacements - Group 2 Seismic Shot 3	231
Figure 4-57: Comparison of Displacements - Group 2 Control Columns Seismic vs Non-Seismic.....	231
Figure 4-58: Comparison of Displacements - Group 2 Jacketed Columns Seismic vs Non-Seismic.....	232
Figure 4-59: Comparison of Displacements - Group 2 FRP Reinforced Columns Seismic vs Non-Seismic	232
Figure 4-60: Comparison of Displacements - Group 2 FRP Reinforced and Jacketed Columns Seismic vs Non-Seismic.....	233
Figure 4-61: Comparison of Displacements – FRP Jacketed Columns and Wedge Prestressed Column Shot 1	233
Figure 4-62: Comparison of Displacements – FRP Jacketed Columns and Wedge Prestressed Column Shot 2	234
Figure 4-63: Comparison of Displacements – FRP Jacketed Columns and Wedge Prestressed Column Shot 3	234
Figure 4-64: Comparison of Displacements – FRP Jacketed Columns and Wedge Prestressed Column Shot 4	235
Figure 4-65: Comparison of Maximum Displacements – Jacketed Columns.....	235
Figure 4-66: Comparison of Residual Displacements – Jacketed Columns.....	236
Figure 4-67: Comparison of Displacements – Braced Columns Shot 1.....	236
Figure 4-68: Comparison of Displacements – Braced Columns Shot 2	237
Figure 4-69: Comparison of Displacements – Braced Columns Shot 3.....	237
Figure 4-70: Comparison of Displacements – Braced Columns Shot 4.....	238
Figure 4-71: Comparison of Displacements – Braced Columns Shot 5.....	238
Figure 4-72: Comparison of Maximum Displacements - Braced Columns	239
Figure 4-73: Comparison of Residual Displacements - Braced Columns	239
Figure 4-74: Curvatures and Curvature Ratios for Group 1 Support Stiffness	240
Figure 4-75: Rotational Support Stiffness for Groups 2 and 3.....	240
Figure 5-1: Confined and Unconfined Concrete Material Model	276
Figure 5-2: Compression and Tension Reinforcing Steel Material Model	276
Figure 5-3: Locations of Confined and Unconfined Concrete in Different Sections	277
Figure 5-4: Geometry of Confinement of Column with Prestressed Steel and Wedges for Development of Equivalent Confinement Pressure	278
Figure 5-5: Moment-Curvature Relationship for Column 2NSC under Various Axial Loads	279
Figure 5-6: Moment-Curvature Relationship for Column 2SJ under Various Axial Loads.....	280
Figure 5-7: Construction of Composite Resistance Curve for Column 2S-J using Variation in Axial Loads.....	281
Figure 5-8: Curvature Ductilities for End and Mid-Span of Column 2S-J Plotted with Composite Resistance Curve and First Yield Point Identified along with Identification Several Points of Interest for Mode Shape Plots. (Mode Shapes Shown in Figure 5-9)	282
Figure 5-9: Mode Shapes for Four Points of Interest along Composite Resistance Curve for Column 2S-J. (Points of Interest Shown in Figure 5-8)	283
Figure 5-10: Column 1NS-C Resistance-Displacement and Load-Mass Factor Curves	284
Figure 5-11: Column 1S-C Resistance-Displacement and Load-Mass Factor Curves.....	285

Figure 5-12: Column 1NS-J Resistance-Displacement and Load-Mass Factor Curves	286
Figure 5-13: Column 1S-J Resistance-Displacement and Load-Mass Factor Curves.....	287
Figure 5-14: Column 2NS-C Resistance-Displacement and Load-Mass Factor Curves	288
Figure 5-15: Column 2S-C Resistance-Displacement and Load-Mass Factor Curves.....	289
Figure 5-16: Column 2NS-J Resistance-Displacement and Load-Mass Factor Curves	290
Figure 5-17: Column 2S-J Resistance-Displacement and Load-Mass Factor Curves.....	291
Figure 5-18: Column 2NS-R Resistance-Displacement and Load-Mass Factor Curves	292
Figure 5-19: Column 2S-R Resistance-Displacement and Load-Mass Factor Curves.....	293
Figure 5-20: Column 2NS-JR Resistance-Displacement and Load-Mass Factor Curves.....	294
Figure 5-21: Column 2S-JR Resistance-Displacement and Load-Mass Factor Curves.....	295
Figure 5-22: Column 3NS-C Resistance-Displacement and Load-Mass Factor Curves	296
Figure 5-23: Column 3NS-W Resistance-Displacement and Load-Mass Factor Curves.....	297
Figure 5-24: Column 3NS-CB Resistance-Displacement and Load-Mass Factor Curves	298
Figure 5-25: Forces and Dimensions Causing M_{BSUP} in Tension Braced Columns	299
Figure 5-26: Moment-Curvature Relationship for Columns 3NS-TB1 through 3NS-TB3 under Various Axial Loads	299
Figure 5-27: Axial Load-Bending Moment Interaction Diagram for Column 3NS-TB1	300
Figure 5-28: Axial Load-Axial Strain and Axial Deformation Relationship for Column 3NS-TB1	300
Figure 5-29: Construction of Composite Resistance Curve for Column 3NS-TB1 using Variation in Axial Loads	301
Figure 5-30: Curvature Distribution for Various Ductility Ratios Found During Development of Column 3NS-TB1 Resistance Curve	302
Figure 5-31: Distribution of Moment Components for Elastic Response Found During Development of Column 3NS-TB1 Resistance Curve.....	302
Figure 5-32: Distribution of Moment Components Found During Development of Column 3NS-TB1 Resistance Curve for Early Post Hinging of the Mid-Height	303
Figure 5-33: Distribution of Moment Components Found During Development of Column 3NS-TB1 Resistance Curve for Highly Plastic Response Region	303
Figure 5-34: Distribution of Rotations Calculated During Development of Column 3NS-TB1 Resistance Curve for Various Response Regions.....	304
Figure 5-35: Displacements along the Length Observed During Development of Column 3NS-TB1 Resistance Curve for Various Response Regions	304
Figure 5-36: Elastic Mode Shape Calculated During Construction of Resistance Curve for Column 3NS-TB1 Compared with Elastic Mode Shape for Column 3NS-C	305
Figure 5-37: Column 3NS-TB1 Resistance-Displacement and Load-Mass Factor Curves	306
Figure 5-38: Column 3NS-TB2 Resistance-Displacement and Load-Mass Factor Curves	307
Figure 5-39: Column 3NS-TB3 Resistance-Displacement and Load-Mass Factor Curves	308
Figure 5-40: Maximum Mid-Height Displacements Compared with SDOF Predicted Maximum Displacements for All Tests.....	309
Figure 5-41: Maximum Mid-Height Displacements Compared with SDOF Predicted Maximum Displacements for All Tests – Values below 10mm	309
Figure 5-42: Maximum Mid-Height Displacements Compared with SDOF Predicted Maximum Displacements for All Tests – Values Between 10mm and 40mm.....	310
Figure 5-43: Maximum Mid-Height Displacements Compared with SDOF Predicted Maximum Displacements for All Tests – Values Between 40mm and 80mm.....	310
Figure 5-44: Composite Resistance Curve and Resistance Curve for Full Axial Load for Column 2S-J.....	311

Figure 5-45: Experimental and SDOF Predicted Displacements for Shots 1 through 4 of Column 2S-J using Composite Resistance Curves and Resistance Curve for Full Axial Load.	311
Figure 5-46: Displacement of Shock Tube Prior To and During Shock Wave Loading for Three Different Shock Tube Tests	312
Figure 5-47: Example of Errors in Measurement of Displacement Response: Shock Tube Movement and Loose Displacement Gauge Mounts on Column 3NS-W Shot 3.....	312
Figure 5-48: Example of Errors in Testing Effecting Displacement Response: Initial Gap between LTD and Column 3NS-TB1 Shot 1.....	313
Figure 6-1: Simplified Displaced Shapes and Curvature Distributions for Columns under Blast Loads.	334
Figure 6-2: Dynamic Load Factor for Elastic Systems (Department of Defense, 2008).	335
Figure 6-3: Typical Tension Brace Configuration on Column showing Distance to Brace Point and Brace Eccentricity.....	336
Figure 6-4: Typical Compression Brace Configuration on Column showing Distance to Brace Point and Brace Eccentricity.....	336
Figure 6-5: Resistance Curve for Compression Brace Retrofitted Column with Brace Yielding prior to Column Yielding.....	337
Figure 6-6: Resistance Curve for Compression Brace Retrofitted Column with Brace Yielding post Column Yielding.....	337
Figure A-1: Pressure, Impulse, and Displacement Time History for Column 1NS-C Shot 1 and Shot 2.....	354
Figure A-2: Pressure, Impulse, and Displacement Time History for Column 1NS-C Shot 3.....	355
Figure A-3: Pressure, Impulse, and Displacement Time History for Column 1S-C Shot 1 and Shot 2	356
Figure A-4: Pressure, Impulse, and Displacement Time History for Column 1S-C Shot 3	357
Figure A-5: Pressure, Impulse, and Displacement Time History for Column 1NS-J Shot 1 and Shot 2.....	358
Figure A-6: Pressure, Impulse, and Displacement Time History for Column 1NS-J Shot 3.....	359
Figure A-7: Pressure, Impulse, and Displacement Time History for Column 1S-J Shot 1 and Shot 2	360
Figure A-8: Pressure, Impulse, and Displacement Time History for Column 1S-J Shot 3.....	361
Figure A-9: Pressure, Impulse, and Displacement Time History for Column 2NS-C Shot 1 and Shot 2.....	362
Figure A-10: Pressure, Impulse, and Displacement Time History for Column 2NS-C Shot 3.....	363
Figure A-11: Pressure, Impulse, and Displacement Time History for Column 2S-C Shot 1 and Shot 2	364
Figure A-12: Pressure, Impulse, and Displacement Time History for Column 2S-C Shot 3	365
Figure A-13: Pressure, Impulse, and Displacement Time History for Column 2NS-J Shot 1 and Shot 2	366
Figure A-14 : Pressure, Impulse, and Displacement Time History for Column 2NS-J Shot 3.....	367
Figure A-15: Pressure, Impulse, and Displacement Time History for Column 2S-J Shot 1 and Shot 2	368
Figure A-16: Pressure, Impulse, and Displacement Time History for Column 2S-J Shot 3 and Shot 4	369
Figure A-17: Pressure, Impulse, and Displacement Time History for Column 2NS-R Shot 1 and Shot 2.....	370
Figure A-18 : Pressure, Impulse, and Displacement Time History for Column 2NS-R Shot 3.....	371
Figure A-19: Pressure, Impulse, and Displacement Time History for Column 2S-R Shot 1 and Shot 2	372
Figure A-20 : Pressure, Impulse, and Displacement Time History for Column 2S-R Shot 3	373
Figure A-21: Pressure, Impulse, and Displacement Time History for Column 2NS-JR Shot 1 and Shot 2	374
Figure A-22 : Pressure, Impulse, and Displacement Time History for Column 2NS-JR Shot 3	375
Figure A-23: Pressure, Impulse, and Displacement Time History for Column 2S-JR Shot 1 and Shot 2	376
Figure A-24 : Pressure, Impulse, and Displacement Time History for Column 2S-JR Shot 3	377
Figure A-25: Pressure, Impulse, and Displacement Time History for Column 3NS-C Shot 1 and Shot 2.....	378
Figure A-26 : Pressure, Impulse, and Displacement Time History for Column 3NS-C Shot 3.....	379
Figure A-27: Pressure, Impulse, and Displacement Time History for Column 3NS-W Shot 1 and Shot 2	380

Figure A-28: Pressure, Impulse, and Displacement Time History for Column 3NS-W Shot 3 and Shot 4	381
Figure A-29: Pressure, Impulse, and Displacement Time History for Column 3NS-CB Shot 1 and Shot 2.....	382
Figure A-30: Pressure, Impulse, and Displacement Time History for Column 3NS-CB Shot 3 and Shot 4.....	383
Figure A-31: Pressure, Impulse, and Displacement Time History for Column 3NS-CB Shot 5.....	384
Figure A-32: Pressure, Impulse, and Displacement Time History for Column 3NS-TB1 Shot 1 and Shot 2.....	385
Figure A-33: Pressure, Impulse, and Displacement Time History for Column 3NS-TB1 Shot 3 and Shot 4.....	386
Figure A-34: Pressure, Impulse, and Displacement Time History for Column 3NS-TB1 Shot 5.....	387
Figure A-35: Pressure, Impulse, and Displacement Time History for Column 3NS-TB2 Shot 1 and Shot 2.....	388
Figure A-36: Pressure, Impulse, and Displacement Time History for Column 3NS-TB2 Shot 3 and Shot 4.....	389
Figure A-37: Pressure, Impulse, and Displacement Time History for Column 3NS-TB2 Shot 5.....	390
Figure A-38: Pressure, Impulse, and Displacement Time History for Column 3NS-TB3 Shot 1 and Shot 2.....	391
Figure A-39: Pressure, Impulse, and Displacement Time History for Column 3NS-TB3 Shot 3 and Shot 4.....	392
Figure A-40: Pressure, Impulse, and Displacement Time History for Column 3NS-TB3 Shot 5.....	393
Figure B-1: Column 1NS-C Strain-Time Histories	395
Figure B-2: Column 1S-C Strain-Time Histories	396
Figure B-3: Column 1NS-J Strain-Time Histories.....	397
Figure B-4: Column 1S-J Strain-Time Histories	398
Figure B-5: Column 2NS-C Strain-Time Histories.....	399
Figure B-6: Column 2S-C Strain-Time Histories	400
Figure B-7: Column 2NS-J Strain-Time Histories.....	401
Figure B-8: Column 2S-J Strain-Time Histories	402
Figure B-9: Column 2NS-R Strain-Time Histories.....	403
Figure B-10: Column 2S-R Strain-Time Histories	404
Figure B-11: Column 2NS-JR Strain-Time Histories.....	405
Figure B-12: Column 2S-JR Strain-Time Histories	406
Figure B-13: Column 3NS-C Strain-Time Histories Shot 1 and Shot 2	407
Figure B-14: Column 3NS-C Strain-Time Histories Shot 3	408
Figure B-15: Column 3NS-W Strain-Time Histories Shot 1 and Shot 2	409
Figure B-16: Column 3NS-W Strain-Time Histories Shot 3 and Shot 4	410
Figure B-17: Column 3NS-CB Strain-Time Histories Shot 1 and Shot 2.....	411
Figure B-18: Column 3NS-CB Strain-Time Histories Shot 3 and Shot 4.....	412
Figure B-19: Column 3NS-CB Strain-Time Histories Shot 5.....	413
Figure B-20: Column 3NS-TB1 Strain-Time Histories Shot 1 and Shot 2.....	414
Figure B-21: Column 3NS-TB1 Strain-Time Histories Shot 3 and Shot 4.....	415
Figure B-22: Column 3NS-TB1 Strain-Time Histories Shot 5.....	416
Figure B-23: Column 3NS-TB2 Strain-Time Histories Shot 1, Shot 2, Shot 3 and Shot 4.....	417
Figure B-24: Column 3NS-TB2 Strain-Time Histories Shot 5.....	418
Figure B-25: Column 3NS-TB3 Strain-Time Histories Shot 1 and Shot 2.....	419
Figure B-26: Column 3NS-TB3 Strain-Time Histories Shot 3 and Shot 4.....	420
Figure B-27: Column 3NS-TB3 Strain-Time Histories Shot 5.....	421
Figure C-1: Column 1NS-C Experimental and Predicted Displacement Time Histories.....	423
Figure C-2: Column 1S-C Experimental and Predicted Displacement Time Histories	424
Figure C-3: Column 1NS-J Experimental and Predicted Displacement Time Histories.....	425
Figure C-4: Column 1S-J Experimental and Predicted Displacement Time Histories	426

<i>Figure C-5: Column 2NS-C Experimental and Predicted Displacement Time Histories.....</i>	<i>427</i>
<i>Figure C-6: Column 2S-C Experimental and Predicted Displacement Time Histories</i>	<i>428</i>
<i>Figure C-7: Column 2NS-J Experimental and Predicted Displacement Time Histories.....</i>	<i>429</i>
<i>Figure C-8: Column 2S-J Experimental and Predicted Displacement Time Histories</i>	<i>430</i>
<i>Figure C-9: Column 2NS-R Experimental and Predicted Displacement Time Histories.....</i>	<i>432</i>
<i>Figure C-10: Column 2S-R Experimental and Predicted Displacement Time Histories</i>	<i>433</i>
<i>Figure C-11: Column 2NS-JR Experimental and Predicted Displacement Time Histories</i>	<i>434</i>
<i>Figure C-12: Column 2S-JR Experimental and Predicted Displacement Time Histories</i>	<i>435</i>
<i>Figure C-13: Column 3NS-C Experimental and Predicted Displacement Time Histories.....</i>	<i>436</i>
<i>Figure C-14: Column 3NS-W Experimental and Predicted Displacement Time Histories</i>	<i>437</i>
<i>Figure C-15: Column 3NS-CB Experimental and Predicted Displacement Time Histories.....</i>	<i>439</i>
<i>Figure C-16: Column 3NS-TB1 Experimental and Predicted Displacement Time Histories.....</i>	<i>441</i>
<i>Figure C-17: Column 3NS-TB2 Experimental and Predicted Displacement Time Histories.....</i>	<i>443</i>
<i>Figure C-18: Column 3NS-TB3 Experimental and Predicted Displacement Time Histories.....</i>	<i>445</i>

Chapter 1. Introduction

1.1. General

Explosives place large demands on the lateral load carrying capacity of structures. If these loads are applied on columns, the high pressure transient loads from explosives can result in significant damage to the primary gravity load carrying elements. The loss of these elements, which are responsible from overall strength and stability of the structure, may cause collapse of all or parts of the structure. Therefore, it is important to mitigate the blast loads effects on columns. This mitigation may be done by various means including:

- Modification of the area surrounding the structure to limit access and increase standoff distance between the explosive location and the structure. This site control may be achieved through different mechanisms including site security, fencing, and bollards. This primary level of defense does not always sufficiently limit the effects of an explosion, as limiting access is not always possible depending on the function of the structure and the characteristics of the site, resulting in small standoff distances. Therefore, although increasing range to the structure is of primary significance to mitigation of explosion effects, in many cases it is not sufficient.
- Modification of the structure to limit the exposure of columns to blast loads. The use of blast resistant panels or other explosive resistant façade that transfers loads to the high mass, high out-of-plane stiffness floor systems may be effective in reducing strength and deformation demands on columns. These systems may not be appropriate for all structures due to a number of reasons including cost and architectural design considerations.
- Modification of the columns themselves such that they resist the blast loads and retain the ability to resist the gravity loads of the structure and, therefore, prevent the possibility of collapse. This approach may be taken along with the other approaches to ensure maximum resilience. Due to the possibility of extremely large lateral loads from blast, the retrofit of these columns must provide high capacity. Furthermore, the retrofit should allow for some amount of ductility capacity in the event that the blast load is higher than the design value. The focus of this study is on this approach.

1.2. Objectives

The objective of this study is to design, test and analyze different retrofit systems for reinforced concrete columns under simulated blast loads and use the results to develop design recommendations and procedures. As part of the study, a series of columns were constructed and tested under simulated blast loads. These columns vary in geometry, reinforcement, and retrofit mechanism. The retrofits used are intended to be effective and practical in strengthening the columns as well as providing robustness through increased ductility capacity. At the conclusion of the experimental and analytical research program, procedures and recommendations for design of retrofits are presented.

1.3. Scope

The scope of this research includes the following:

- Construction of 15 reinforced concrete columns and data collection from 2 other columns from a previous study.
- Retrofit of a group of the columns with surface bonded carbon fibre reinforced polymers (CFRP) to enhance strength, ductility, or the combination of the two.
- Retrofit of one of the columns with a prestressed transverse reinforcement and concrete wedge system to increase mass and ductility.
- Retrofit of one of the columns with a bracing system that is designed to act as compression members and provide a lateral restoring force to the column.
- Retrofit of three of the columns with a bracing system that is designed to act as tension members and provide lateral restoring forces to the columns.
- Construction of support systems for connection of the columns to the shock tube testing facility to provide a high degree of rotational and translational restraint.
- Construction of a load-transferring device (LTD) that collects the pressure from the shock tube and transfers it as a series of point loads along the length of the column that approximate a uniformly distributed lateral load.
- Experimental testing of columns under incrementally increasing pressure and impulse combinations using the shock tube testing facility.
- Comparison of the results of the columns across different retrofit groups and with their control specimens.

- Development of an analytical model to verify the test data.
- Development of design procedures and recommendations for retrofit of reinforced concrete columns to mitigate the effects of blast loads.

1.4. Brief Description of Contents

This was an experimental and analytical study. As such, a large portion of this thesis are dedicated to the setup of the experimental program and the reporting and discussion of the experimental results. In addition to the main body of the document, several appendices are included that provide comprehensive sets of plots for experimentally recorded data including loading, displacement, and strain time histories. In the main body of the thesis, there are seven chapters of which this is one. The following is a brief description of the contents of each chapter of this study:

Chapter 2: Background and Literature Review

The state of the art of experimental research into flexure dominate reinforced concrete members is presented in this chapter along with a selection of past research into similar retrofit technologies as those used in this research.

Chapter 3: Experimental Program

The detailed description of the geometry and material properties of the 17 columns investigated in this study are presented. The experimental setup and test program are described in detail including all supports, load transfer devices, instrumentation and data acquisition systems. The construction and application of the various types of retrofits used are detailed.

Chapter 4: Experimental Results and Discussion

A detailed description of all tests on all columns outlining deflections, pressures and impulses, strains and qualitative damage assessment is provided in this section. Pictures and plots of data for each test along with summary tables and figures are presented. A discussion on the behaviour of columns relative to each other is provided with a focus on grouping similarities including: seismically detailed and non-seismic columns, FRP strengthened retrofitted columns, braced columns, and columns retrofitted with transverse confinement.

Chapter 5: Single Degree of Freedom Modeling

A procedure for developing the resistance vs. displacement curves for the various columns along with the conversion into equivalent single degree of freedom and the numerical integration of the time history for dynamic analysis is described. Resistance curves are developed with consideration for various effects including: axial load and its variation with displacement, confinement effects on concrete, strain hardening, dynamic material properties, plastic response over hinged regions, and the resistance mechanisms of bracing. The analytical and experimental results are compared and conclusions are drawn.

Chapter 6: Design Procedure and Recommendations

The experimental results and conclusions from the analysis of results are used to validate current design procedures and develop new procedures and recommendations. The design recommendations include comments on FRP bond, design of transverse FRP for ductility requirements of current Canadian blast standards, design of prestressed transverse reinforcement, and design of braces that act in compression and braces that act in tension.

Chapter 7: Conclusions

The study is summarized and several conclusions are provided. Recommendations on future research relating to this study are presented.

1.5. Research Significance

This research expands on current knowledge with the use of FRP as a retrofit technology for both flexural strength enhancement and improvements in mechanisms associated with the use of transverse reinforcement (concrete confinement and strengthening against diagonal tension), and also introduces two new systems of retrofit for columns involving compression and tension braces. The experimental program provides extensive test data and invaluable experimental evidence on numerous physical phenomena that play important roles on blast resistance of reinforced concrete columns. The experimental and analytical testing of columns with various layups of FRP under blast loads allows conclusions to be drawn to the validity of current design standards for bond, dynamic material properties, and confinement efficiency under high strain rate response with a blast mode shape. This data is currently limited in the public domain and proper design of FRP retrofits requires a high degree of confidence in ultimate material response.

The use of prestressed confinement has been studied in seismic applications and this study validates its efficiency for blast retrofits. The change in mode shape between blast and seismic along with variations of axial load in blast due to high velocity response make the confinement efficiency of the prestressed transverse steel questionable under blast loads. This study addresses this issue. Furthermore, the prestressed confinement steel system is modified and improved to use precast concrete spacers with a more readily available and simpler application of prestressing.

Both the compression and tension brace systems presented in this study are novel approaches to retrofit columns under blast loads. They are designed to be simple to install, simple to manufacture, cost effective, non-obtrusive in the structure, and highly effective in resisting blast loads. The new retrofit allows for significant strength gains with limited additional material.

The design procedures and recommendations that are provided provide guidance to designers and researchers exploring blast resistance of structures. This information is gathered by either the direct results of the experimental and analytical research from this project or by modifying seismic design principles (for FRP and prestressed confinement) to suit blast response of columns.

This research also contributes to the experimental setup of subsequent research projects using the same shock tube facility. The load transfer device, supports, axial load application setup, instrumentation and monitoring procedures, and shock tube operation procedures have all been implemented in other projects including projects by other researchers.

Chapter 2. Background and Literature Review

2.1. General

Blast resistant structures, and specifically retrofit techniques for reinforced concrete columns, is a seldom researched area. Some high explosive field testing and other shock tube testing projects have explored column resistance to blast loads and some retrofit techniques limited to the use of fibre reinforced polymers (FRP) for strength and ductility. This section presents the previous blast resistant column research along with related research done on flexural dominate reinforced concrete members under blast loads, FRP under flexure tested to ultimate capacity for static load cases, and prestressed jacketing of columns for seismic resistance. These further topics relate to the retrofit systems used in this study. Prior to those research summaries, a brief introduction to blasts and the resulting loads on structures is given and a summary of some historical explosions effecting structures is provided.

2.2. Explosions and Explosive (Blast) Loads on Structures

A high explosive, such as TNT (the standard unit explosive in blast engineering), releases a large amount of high temperature and high pressure explosive gas when detonated. If this detonation is in air, as is the common concern for malicious or accidental explosions on structures, the explosive gas pressurizes the surrounding atmosphere. The volume of pressurized atmosphere expands as the pressurized air interacts with the ambient air. This expansion is the propagation of a shock wave and wave front. The expansion is spherical (in a homogeneous atmosphere) and moves at speeds greater than the speed of sound in the atmosphere. The barrier of the expanding sphere is the shock front. On one side of this distinct barrier, the pressure is at its maximum and on the other side, the pressure is at normal ambient atmosphere conditions (Smith and Hetherington, 1994).

If this shock wave expands until it interacts with a structure, it will reflect off that structure surface back into the expanding wave. This will have the effect of increasing the pressure at the structure location (known as the reflected pressure). The structure will face pressures from the shock wave that have instantaneous rise to maximum values and decay exponentially over a very short duration. The magnitude of the pressure depends on the explosive size, distance from the structure, structure geometry, and other factors including atmospheric conditions, and any reflections off other surfaces including the ground. Typical design loads could range from 10kPa to 10MPa depending on these

factors. The duration of these loads is also dependent on the same factors with durations ranging from 1ms to 100ms.

The shock wave time dependent pressure at a point a distance away from the source (standoff distance), is commonly described by the Friedlander equation (Equation [2-1]). The impulse, I , is the integral of the pressure-time curve (Equation [2-2]) and is used to relate the duration of the load to the pressure (Smith and Hetherington, 1994).

$$P(t) = P_0 \left(1 - \frac{t}{t_d}\right) e^{-\alpha t/t_d} \quad [2-1]$$

$$I = \int_0^{t_d} P(t) dt \quad [2-2]$$

where P_0 is the maximum overpressure, t_d is the duration of the positive phase, and α is a dimensionless decay parameter.

The interaction of a structure with the blast wave results in an increase in pressure and impulse due to reflection. The amount of this increase varies depending on the size and geometry of the structure. The increased blast parameters are referred to as the reflected pressure and reflected impulse. These values are what loads the structure.

2.3. History of Accidental and Malicious Explosions

Bangash and Bangash present a case study of recent buildings exposed to explosive attacks. In this section, seven of these cases are summarized (Bangash & Bangash, 2006).

2.3.1. The Alfred P. Murrah Federal Building - 1995

The Alfred P. Murrah Federal Building, located in Oklahoma City, Oklahoma, USA was the target of an explosive attack in April 1995. It is estimated that 1800kg of ANFO high explosives was detonated in the form of a car bomb located between three and five meters away from the structure. The TNT equivalent of this explosive was approximately 1905kg which would produce peak incident pressures of 13,550kPa (3m) and 5849kPa (5m) and positive phase incident impulses between 3440kPa-ms (3m) and 2162kPa-ms (5m) when detonated as a hemispherical explosion (Hyde, 1992).

The nine-storey reinforced concrete structure lost a total of four main storey columns due to the blast loading which lead to the progressive collapse of a significant amount of the structure. A total

of 168 fatalities and numerous injuries were caused by the combination of the explosion and progressive collapse of the structure.

2.3.2. The World Trade Center - 1992

On 26 February 1992, the north tower of the World Trade Center located in New York City, New York, USA was attacked with approximately 816kg of high explosive ANFO in the form of a vehicle bomb located in an underground parking facility two levels below the ground floor. The explosion caused two storeys of reinforced concrete slabs to be blown out and extensive damage to the underground parking facility including large cratering. The TNT equivalent of the ANFO explosive was 572.6kg.

2.3.3. Jewish Community Centre, Buenos Aires - 1994

On 18 July 1994 approximately 275kg of high explosives was detonated between three and five meters away from the Jewish Community Centre in Buenos Aires, Argentina. This explosion led to the progressive collapse of the five storey brick masonry structure.

2.3.4. St. Mary Axe, London - 1992

On 11 April 1992 a vehicle bomb containing 350kg of TNT exploded in the neighbourhood of St. Mary Axe, London, England. This explosion caused significant damage to many buildings including the ten-storey reinforced concrete with masonry walls European Development Bank.

2.3.5. Bishopgate, London - 1992

The neighbourhood of Bishopgate, London, England experienced a high explosive vehicle bomb blast on 23 April 1992. This blast destroyed several buildings and caused over forty casualties.

2.3.6. Docklands, London - 1996

On 9 February 1996 a vehicle bomb carrying the approximate equivalent of 500kg TNT in the form of a homemade high explosive detonated in the Docklands neighbourhood, London, England. This blast caused extensive damage to non-structural elements in several buildings.

2.3.7. Al-Khobar, Saudi Arabia - 1996

The Al-Khobar Building in Saudi Arabia was subjected to a blast equivalent to approximately 2270kg of TNT on 26 June 1996 in the form of a vehicle bomb. The blast caused severe damage to the façade of the structure along with many of the horizontal structural elements.

2.4. Experimental Case Studies of Explosions on Reinforced Concrete Columns and Flexure Members

Due to the cost and difficulties associated with experimental testing of structures under explosive loading, there has not been a substantial amount of past research in this field. The research that has been undertaken has attempted to define responses and response limits for structural elements by testing under actual high explosives or by using shock tubes or high velocity impactors to simulate explosive loading. Along with research into the response of structural members, there has been work done investigating methods of retrofit to mitigate the damage due to blast loads. This work has primarily focused on the use of polymer retrofits such as CFRP or GFRP. Below is a summary of some of the work that has been done investigating the response and retrofit of reinforced concrete flexural members under blast loads.

2.4.1. S.C. Woodson and J.T. Baylot

In an effort to better understand the response of reinforced concrete columns to blast effects, the authors built two quarter scale reinforced concrete models and subjected them to explosive loading. The models were two storeys tall, two bays wide and one bay deep with reinforced concrete construction for the columns, beams, and slabs and various masonry infill wall designs for cladding. Additional mass was added to the top of the structure to represent a four storey structure. The columns were designed for a low seismic region. The scale models were subjected to multiple explosions leading to various failure mechanisms.

The four corner columns of the models were over designed to withstand the expected blast loads while the exterior columns in the centre of two bays were designed as test columns. The test columns were 910mm in length cast integrally with the beam and slab connections at the floor levels. Two of each square cross section measuring 89mm and 76mm were used with one of each cross section in each scale model. Columns were reinforced with 400MPa 6.45mm² and 450MPa 32.3mm² deformed wire, and 440MPa 32.3mm² smooth wire lateral ties were used. Concrete strength with an average strength of 44.8MPa at the time of testing and a maximum aggregate size of 10mm was used in the construction (Woodson & Baylot, 1999).

The authors carried out five tests that represented different placement of explosives and different cladding designs. The results of these tests are as follows:

1. Open Frame – Large Standoff

In this model there was no masonry cladding used so all bays were open. 7.102kg of Composition C-4 (TNT equivalent of 9.730kg for pressure and 8.452kg for impulse) were placed on a light plywood platform located 228mm from the ground surface. The explosives were located 1524mm away from the centre of the test column and were aligned to be perpendicular to the face of the column. The resulting explosion caused little damage to the column aside from hairline cracks. No residual displacement was evident.

2. Open Frame – Small Standoff

The first test was repeated with exactly the same explosive mass located perpendicular to the column in an open frame at 1067mm standoff distance. This explosion caused some damage to the column including approximately 6.4mm residual displacement and larger cracks. The column was still able to support the mass of the structure and applied mass at the top of the structure.

3. Fully Clad

In this test, the same explosive properties of mass and location as in test two were used. The bays of the frames were fully clad with quarter scale masonry blocks. The column suffered considerable damage in the lower storey with approximately 114mm residual displacement and some damage in the upper storey with approximately 25mm residual displacement. The authors judged that the column was structurally deficient and not able to withstand the mass of the structure and the applied mass and that the load transfer to a spandrel beam was all that prevented collapse.

4. Partially Clad

This test used the same mass and location of explosive as tests 2 and 3 however the masonry cladding differed from test three. In this test, openings were left in the masonry that were equivalent to 33% of the bay area and represented windows in the structure. The lower storey column had a residual displacement of 76mm and the upper storey column had a residual displacement of 14mm. A similar load transfer to the spandrel beam as in test 3 was observed and the structure remained standing.

5. Parking Garage

Once again, the same mass and location of explosive as in tests 2 through 4 was used for this test. There was no masonry cladding used on the lower storey and complete infill masonry on the upper storey. This test setup was intended to represent a building with a main storey parking facility with open bays. The lower storey column did not see significant damage due to the lack of any cladding. The upper storey column had approximately 76mm residual displacement and the upper storey edge beam was needed to sustain the load transferred from the damaged column.

The conclusions of the experimental tests were that reinforced concrete frames responded better to explosive loading if there was little area to reflect the shock wave pressure, such as infill walls. Aside from columns, the authors also were concerned with the slabs and beams in the structure and found that the structures with openings had significantly more damage to the slabs than the fully clad structures. It was also concluded that the spandrel beam needs to be sufficient to accommodate the force transfer due to the loss of one or more columns.

2.4.2. J. Magnusson

Magnusson tested a total of 89 reinforced concrete beams under both static and air blast loading. 49 of beams were reinforced with 500MPa steel and 40 were reinforced solely with steel fibres that comprised 1% of the volume of the beams. The blast tests were simulated with an explosive driven shock tube.

In all of the static tests, the mode of failure was pure flexure for the beams. However, Magnusson found that in some cases under the blast loading the mode of failure would be shear failure. For the beams reinforced with steel fibres rather than traditional longitudinal bars, the shear mode of failure was prevented under blast loading. By comparing the support reactions of beams under static loading to those under dynamic blast loading, it was found that the beams could withstand larger forces under dynamic loading with the same degree of damage as static loading. Based on the displacement time histories recorded during the blast load tests, Magnusson observed that less stiff elements will deflect readily and gain high kinetic energy while more stiff elements will deflect less and attain high internal strain energy. The stiffer elements were more likely to fail in shear rather than flexure under the dynamic loads. Less stiff elements eventually failed in flexure (Magnusson, 2007).

2.4.3. L.C Muszynski and M.R. Purcell

The authors performed tests on both normal construction reinforced concrete beam, wall and column structures as well as the same structures fitted with externally applied carbon fibre sheets and glass fibre sheets exposed to explosive loading. The columns were cast integrally with the walls. Two structures were tested, one with carbon and glass external reinforcement and one normal construction. A series of explosive tests were undertaken to evaluate the effectiveness of the externally applied reinforcement (Muszynski & Purcell, 2003).

1. Window Testing

The first two tests were designed to test windows, not structural components. Large standoff distances, ranging from 31.5m to 65.1m were used with 806kg of TNT. These tests caused minimal damage to no damage in both structures.

2. Structure Testing

Two further explosive detonations were triggered, both using 806kg of TNT. The standoff distance was 14m and 13.5meters for the structure with external carbon and glass reinforcement and 15.9m for both tests on the structure with normal construction. The maximum reflected pressures on the structures ranged from 1786kPa to 2930kPa and reflected impulses ranged from 4262kPa-ms to 5241kPa-ms. Under the first large explosive test, the normal construction structure saw significant damage including the failure of columns in flexure and stress fractures between the columns and the walls. The structure with externally applied reinforcement saw less damage with some spalling of concrete on the columns and some stress fractures at the joints between the columns and walls. The second large explosive test caused complete collapse of the already damaged normal construction structure. The structure with external reinforcement remained intact while some of the carbon and fibre laminate ruptured. There was no spalling in the walls that were reinforced with either external carbon fibre or glass fibre.

The authors concluded that the use of carbon and glass fibre laminates increased the capacity of the structures to withstand air blast loads.

2.4.4. A. Schenker et al.

The authors carried out two full scale high explosive tests on reinforced concrete slabs fitted with sacrificial aluminium foam protection layers. The one way slabs spanned 3 m and were exposed to air blasts resulting from 1000kg of TNT detonated as hemispherical explosions located at a standoff distance of 20m. A total of four slabs were tested, two of which were high strength concrete slabs with 100MPa compressive strength concrete and two were normal strength concrete slabs with 30MPa compressive strength concrete. One slab in each strength group had a protective layer of aluminium foam designed to dissipate blast energy by plastic deformation and one control slab that was normal construction.

The authors recorded a reduction in acceleration, velocity and displacement in the slabs protected with the sacrificial aluminium foam layer in comparison with the normal construction slabs. There was a corresponding reduction in the strain at the center of the slabs for the protected slabs compared with the normal construction slabs. The authors concluded that the aluminium foam protective layer clearly modified the response of the slabs, however they were unable to draw conclusions on the efficiency of the foam for practical use (Schenker, et al., 2008).

2.4.5. F. Toutlemonde et al.

The authors of this study set out to explore the change in behaviour of concrete under high strain rates, specifically the apparent increase in compressive and tensile strength. Parameters including free water content and pore size of concrete matrix were explored by testing non reinforced concrete slabs in a shock tube. Concrete strengths varied from 35MPa to 115MPa for the tests and free water content was held constant at saturated (100%) or dry (0%). It was concluded that the free water within the pores of the concrete has a significant influence on the apparent strength increase in concrete under high strain rates. This strength increase is not apparent in shear and may cause a shear failure mechanism in a concrete member that would otherwise be flexural dominate under static loading (Toutlemonde, Rossi, Boulay, Gourraud, & Guedon, 1995).

2.4.6. A.G. Razaqpur, A. Tolba and E. Contestabile

Field tests using high explosives on 8 reinforced concrete slabs retrofitted with glass fibre reinforced polymers (GFRP) were undertaken. The slabs measured 1000mm by 1000mm and 70mm thick and were made of 40MPa concrete with top and bottom steel mesh. Of the eight slabs, five were normal

construction and three were retrofitted with the externally applied glass fibre laminates. The slabs were exposed to ANFO high explosive detonations at 3.1m standoff distances. Charge weights were 22.4kg (18.37kg TNT equivalent for pressure and impulse) and 33.4kg (27.39kg TNT equivalent).

The authors observed a significant improvement in performance for the slabs retrofitted with GFRP laminate compared with the normal construction slabs in terms of deflection, damage level, and residual strength for the 22.4kg explosions. There was no apparent improvement in performance of the retrofitted slabs under the 33.4 kg explosion when compared with the normal construction slabs (Razaqpur, Tolba, & Contestabile, 2006).

2.4.7. E. Jacques

Jacques conducted experimental shock tube testing on thirteen large scale reinforced concrete slab specimens. The tests explored the use of externally bonded CFRP retrofit on the response of slabs under blast loading. The slabs tested had a range of boundary conditions ranging from fixed on all sides to pin ended one-way bending. The tested specimens were constructed either as full square panels measuring 2440mm in width and height with thicknesses of 75mm or as one-way panels with a width of 440mm, a height of 2440mm and thicknesses of 80mm and 120mm. All specimens were reinforced with non-deformed 6.3mm steel wire.

Several of the slabs were retrofitted with externally bonded CFRP. The direction and reinforcement ratio of the CFRP retrofit was a parameter that was explored as part of the research project. Anchoring of the FRP to the concrete substrate was done on two of the specimens using FRP fan anchors placed at the end of the FRP reinforcement (Jacques E. , 2011).

Some of the conclusions that the author made are presented below:

1. Without exception, the FRP increased strength and stiffness of the flexure members.
2. The FRP retrofitted specimens remained essentially elastic up to the point of failure of the retrofit.
3. The moment reversal during rebound can result in damage in the case of members only retrofitted for inbound displacement.
4. Debonding of the FRP from the concrete substrate was the prevailing failure mode of the retrofitted specimens.

5. The FRP anchors were successful in delaying FRP debonding for diagonal crack induced debonding but had no impact on delaying plate-end interfacial debonding.
6. FRP was able to achieve a maximum strain of between 0.6% to 0.9% prior to debonding.
7. Failure of the FRP fan anchors was caused by shearing at the anchor bend at the interface between the FRP sheet retrofit and the concrete substrate.

2.4.8. A. Lloyd

The author constructed and tested fourteen half scale reinforced concrete columns under shock tube induced shock wave loading. Twelve of the columns measured 100mm by 150mm in cross-section with two columns measuring 150mm by 150mm in cross section. The columns were reinforced with four 10M longitudinal reinforcing bars at the corners extending the entire length with hooked ends for development. Transverse reinforcement, in the form of 6.3mm diameter closed hoops, were spaced at $h/2$ or $h/4$ for the entire length of the column. The columns with the smaller transverse steel spacing represented columns in seismic regions while the larger spacing represented columns in non-seismic regions. The columns were subjected to axial loads between 28% and 42% of the concentric capacity. Concrete strength was between 46.5MPa and 58MPa at the time of testing. Each column was subjected to multiple shock wave loading tests designed to load the column within the elastic range, up to yield, and to a damaged state. Some of the conclusions of the research are as follows (Lloyd, Performance of reinforced concrete columns under shock tube induced shock wave loading, 2010):

- There was a considerable variation in the axial load during the dynamic response of the columns. This was due to the ends of the columns moving in the direction of the column length with the horizontal displacement of the columns.
- There was no significant difference in the response between the seismic and non-seismic columns for these tests. This was attributed to the column slenderness and small area of the core concrete making confinement effects negligible. The loss of axial load with horizontal displacement also likely contributed to the lack of difference between responses of the two column types.
- The close spacing of the transverse hoops prevented reinforcement buckling during the dynamic response for both the seismic and non-seismic columns.
- The response of the columns can be accurately modeled using single-degree-of-freedom models that account for the change in axial load with horizontal displacement.

2.4.9. R. Burrell

Burrell constructed and tested a total of thirteen half-scale steel fibre reinforced concrete (SFRC) and ultra-high performance fibre reinforced concrete (UHPFRC) columns under shock tube induced shock wave loading. The specimens were 152.4mm by 152.4mm in cross-section with a total length of 2468mm and an unsupported length of 1980mm between partially fixed supports. All columns were reinforced with a total of four 10M longitudinal reinforcing bars and had square hoops comprised of 6.3mm non-deformed steel wire for transverse reinforcement spaced at 75mm or 38mm. The concrete strength ranged from 40.5MPa to 60.6MPa for the SFRC columns and 144.7MPa to 165.4MPa for the UHPFRC columns. The volumetric fibre content ranged from 0% to 1.5% for the SFRC concrete and 2.0% to 6.0% for the UHPFRC columns (Burrell, 2012).

Conclusions from this research include the following:

1. Steel fibres reduce maximum and residual displacement of columns subjected to shock wave loading.
2. The columns with closer spacing of transverse reinforcement had lower displacements under similar shock wave loading properties than the columns with the larger spacing.
3. Closer spacing of transverse reinforcement was effective in preventing the buckling of compression steel reinforcement that was seen in the columns with the larger spacing.
4. The use of SFRC was not sufficient to prevent buckling of compression reinforcement in columns with the larger spacing.
5. The use of SFRC was effecting in reducing the amount of spalling and crushing of concrete when compared with normal concrete.
6. The mode of failure of all columns with the ultra-high strength concrete was rupture of the tension reinforcement.

2.4.10. P.F Silva and B. Lu

The authors present the experimental results of a series of tests on one-way simply supported slabs subjected to spherical high explosive detonations. The six slabs were 1200mm square with 1048mm between supports and a thickness of 90mm. The slabs were reinforced with three 9.5mm diameter steel reinforcing bars in each direction on the unloaded face. Two of the slabs were retrofitted with externally bonded CFRP laminates. One of these slabs had CFRP on both faces and one had it only on the tension face. Two slabs were retrofitted with externally bonded steel reinforced fibre sheets with

one slab retrofitted on both sides and one only on the tension face. The slabs were tested with RDX high explosives with masses ranging from 0.45kg to 1.35kg and standoff distances from 300mm to 910mm. Some of the conclusions and observations that were made in the test program are presented below (Silva & Lu, 2007).

1. Retrofitting the slabs on both sides was shown to be a significantly better option than retrofitting only the tension side. The moment reversals seen during dynamic response were significant enough to cause damage to the slabs retrofitted only on one side.
2. For the slabs with two sides retrofitted, the shear capacity of the concrete was the limiting factor in resistance.
3. The SRP retrofit performed nearly identically to the FRP retrofitted slabs.

2.4.11. C.A. Ross, M.R. Purcell, and E.L. Jerome,

The authors presented the experimental results of tests on simply supported reinforced concrete beams retrofitted with CFRP subjected to high explosive loading. The authors tested a total of six 200mm by 200mm beams that were 2740mm long and two reinforced concrete slabs that was 200mm thick by 3050mm by 3050mm. Three of the beams were retrofitted with three plies of longitudinal CFRP on the unloaded face and the sides. One of the slab was retrofitted with CFRP sheets on the tension (unloaded) face. The beams were loaded with the blast wave from 110.7kg of ANFO high explosive at standoff distances ranging from 4.38m to 5.34m. The beams and slabs were constructed of high-strength, light weight nylon fibre reinforced concrete with a compressive strength of 84.3MPa. Some of the conclusions and observations presented by the authors are as follows (Ross, Purcell, & Jerome, 1997):

1. The moment reversal due to rebound resulted in more damage than the inbound displacement and moment due to the blast load. This is due to the retrofit only being applied to one side.
2. The slabs survive the blast tests with little damage. Yield line cracking indicated that the slabs were damaged on two surfaces by both the rebound displacement and inbound displacement.
3. The beams and slab that were retrofitted with CFRP did experience some FRP debonding from the concrete substrate.

2.4.12. M. Carriere et al.

The authors presented the results of explosive testing on ten reinforced concrete beams and columns. All beams had a cross section of 150mm by 150mm with a total length of 2100mm and an unsupported length between fixed supports of 1500mm. The beams were reinforced with four 6mm diameter steel reinforcing bars with 6mm diameter ties spaced at 100mm along the length with 50mm tie spacing at the support regions. Of the ten specimens tested, five were strengthened with externally bonded steel reinforced polymer (SRP) wraps and five were as-built and served as control tests. Concrete strength ranged from 38.1MPa to 40.5MPa at the time of testing. Reinforcing steel yield strength was 500MPa and ultimate strength was between 596MPa and 604MPa. The SRP had a specified modulus of elasticity of 77.9GPa and an ultimate strength of 1170MPa. It was applied to the beams as a wrap with fibres transverse to the member length. The SRP was bonded to the concrete substrate with epoxy. The specimens were loaded with a spherical detonation at a standoff distance of 2m. Explosive mass ranged from 15kg to 50kg of C-4.

The authors concluded that the members with the SRP wraps had a reduction in concrete crushing at the plastic hinge locations that was attributed either to the confinement provided by the SRP or the SRP providing armouring against spalling. They found that the SRP wrapped columns had increased ductility capacity which they attributed to the reduced concrete crushing. The SRP was successful in withstanding their maximum charge mass of 50kg at 2m without debonding from the concrete substrate. For the beams tested, they found that the SRP did not increase resistance when tensile failure of the internal reinforcement was the governing failure mode. The SRP did increase the resistance of the axially loaded members where concrete crushing was the failure mode in the control specimens. This was attributed to the confinement effects allowing compression concrete to reach a higher strain (Carroere, Wight, Heffernan, & Braimah, 2007).

2.4.13. J.O. Berger, P.J. Heffernan, and R.G. Wight

The authors tested eighteen reinforced concrete columns under high explosive driven blast loading. The columns had a 150mm square cross-section with a total length of 2100mm and an unsupported length of 1500mm between fixed supports. Longitudinal reinforcement was comprised of four 10mm diameter reinforcing bars. Transverse reinforcement was comprised of 6mm diameter hoops spaced at 100mm over the length of the column with 50mm spacing at the supports. The cover to the hoops was 20mm. Concrete strength was 44MPa at the time of testing and the internal reinforcing steel

had a yield strength of 450MPa with an ultimate strength of 630MPa. There were several retrofit schemes tested including the use of externally bonded steel reinforced fibre polymer (SRP) sheets and the use of externally bonded carbon fibre reinforced polymers (CFRP). The polymer retrofits were applied in different configurations including transverse wraps and longitudinal strips. Charge mass of the hemispherical explosions ranged from 49kg to 100kg of C4 at a range ranging from 4m to 5.25m. Some of the conclusions and observations of the testing program are provided below (Berger, Heffernan, & Wight, 2013).

1. The use of both longitudinal and transverse reinforcement significantly reduced damage.
2. The columns reinforced with SRP appeared to be more ductile than the columns reinforced with CFRP.
3. The explosive thrown projectiles that impacted the columns did not damage the SRP columns as much as the CFRP columns.
4. Debonding only occurred in a column that did not have a transverse wrap applied over the longitudinal polymers.
5. The columns retrofitted with both longitudinal and transverse polymers did have longitudinal polymer rupture for both cases of CFRP and SRP.

2.4.14. C. Wu et al.

The authors tested a total of six slabs under high explosive loading. Three groups of slabs were tested including two control specimens consisting of conventional reinforced concrete, two slabs with externally bonded pultruded fibre reinforced polymers, and two slabs with ultra-high performance concrete with and without steel reinforcement. The slabs were subjected to spherical blasts with charge masses ranging from 1kg to 20kg and standoff distances between 0.92m to 3m. All of the slabs with the exception of one of the slabs in the ultra-high performance concrete groups with no steel were reinforced with 12mm diameter mesh spaced at 100mm in the major bending plane and 200mm in the minor bending plane. Concrete cover was held constant at 10mm. All slabs were 100mm thick, 1000mm wide, and 2000mm long with an 1800mm unsupported length between pin supports. Concrete strength ranged from 39.5MPa for the conventional concrete to 151.6MPa for the ultra-high performance concrete. Two sets of pultruded CFRP plates with widths of 240mm and were applied using two layers of 1.4mm thick material extending the entire length of the slab. Some of the results of the experimental testing program are described below (Wu, Oehlers, Rebentrost, Leach, & Whittaker, 2009).

1. The normal strength reinforced concrete panels performed worse than the unreinforced ultra-high performance concrete with steel fibres.
2. The slabs retrofitted with pultruded plates performed better than the slabs without the pultruded plates.
3. The reinforced ultra-high performance concrete slab performed better than all the other slabs tested.
4. The pultruded CFRP retrofit debonded from the concrete substrate during testing.
5. Direct shear cracking was observed at the locations of the concentrated loads for the case of low standoff tests.

2.4.15. T. Rodriguez-Nikl et al.

The authors present the results of an experimental investigation into the response of carbon fibre reinforced polymer (CFRP) jacketed reinforced concrete columns under simulated blast loads. Experimental research was conducted using a system of hydraulic rams to accelerate masses towards the column specimens with a load shape consistent with blast loading. The researchers tested ten columns under impulses ranging from 6800kPa-ms to 15700kPa-ms. The columns were designed to represent columns in a moment frame structure in a low seismic region. The thickness of the CFRP jacket was a primary variable. The researchers concluded that the CFRP jackets significantly improve the response of the columns. Columns that experienced a brittle failure mode gained significant strength and ductility capacities with the addition of CFRP jacketing (Rodriguez-Nikl, et al., 2009).

2.4.16. C.P. Pantelides et al.

This paper presented the results of an experimental and analytical research project investigating the response of reinforced concrete panels subjected to high explosive blast loading. The authors tested five different types of panels with variables including concrete type (normal weight concrete and fibre reinforced concrete), reinforcement configuration, reinforcement type (steel or glass fibre reinforced polymer bars), and the use of external glass fibre reinforced polymer (GFRP) laminates. Each of the five types of panels had three thicknesses: 152mm, 254mm and 356mm. All panels were 1.2m square and supported on two sides. The explosive standoff distance varied between 0.97m and 1.02m with equivalent TNT mass varying between 6.2kg and 13.2kg (spherical detonation). Some of the conclusions and observations of the experimental testing project are as follows (Pantelides, Garfield, Richins, Larson, & Blakeley, 2012):

1. The panels with internal steel reinforcement and steel fibre concrete experienced minimal damage under the majority of the tests.
2. The panels with the GFRP laminates on both sides did not show significant damage aside from the thinner panel where the GFRP laminate on the unloaded face debonded from the concrete substrate. The authors recommended the use of additional anchorage for the GFRP to supplement the chemical bond of the epoxy to the substrate.
3. The panels with the internal GFRP bar reinforcement had moderate damage after the tests including radial cracks in the concrete with maximum crack widths of approximately 2mm.
4. The panels with the most damage were from the group without internal reinforcement aside from the steel fibre reinforcement. Some of the thinner panels with steel or glass reinforcement also suffered high damage during the tests. Damage included concrete cracking, fragmentation, radial cracks, concrete crushing, flexural cracking and plastic hinges along the vertical centerline.
5. The spacing of the internal reinforcement was found to be very important in the response of the panels with smaller spacing performing better.
6. For panels with similar reinforcement spacing and reinforcing ratios, there was no significant difference between the GFRP and steel reinforcement.
7. Increasing panel thickness was an effective method of reducing damage.

2.4.17. C.E. Davis et al.

The authors present the results of experimental testing of circular reinforced concrete bridge columns under high explosive driven blast loads. A total of ten half-scale columns with cross-section diameters of 457mm and 914mm were tested under various combinations of explosive mass and standoff distance. The parameters that were investigated included cross-section diameter, splice length, transverse reinforcement type (hoops, spiral or ties), transverse reinforcement spacing and volumetric reinforcement ratio of transverse reinforcement. Some of the conclusions and observations of the research are presented below (Davis, et al., 2009).

1. Increasing standoff distance is one of the best ways to improve the performance of blast-loaded reinforced concrete highway bridge columns. This limits the possibility of the blast causing local damage to the column or breaching of the concrete core.
2. The use of circular column cross-section effectively decreases the blast pressure and impulse on the column relative to and equivalent sized square column.

3. Increasing the volumetric reinforcement ratio of transverse reinforcement increases the column ductility capacity and shear capacity. The tests results showed better response for seismically detailed columns versus gravity load detailed columns.
4. Columns with spiral reinforcement performed better than columns with hoops or ties with the same spacing and reinforcement ratio.
5. Discrete hoops can be designed to perform better under blast loads if the hooked end extends deeper into the core. This observation is particularly related to the high shear areas where core concrete can become damaged. The authors recommend extending the hooked ends 20 bar diameters into the core of the column to reduce the possibility of anchorage pullout.
6. Splices of longitudinal reinforcement near the blast location for near field explosions increase the possibility of breaching.

2.4.18. P.A. Buchan and J.F. Chen

The authors summarized experimental research programs from several researchers investigating the use of FRP composite retrofits for concrete and masonry structures under blast loads. From the previous research studies, the authors concluded that FRP can considerably increase blast resistance capacity of concrete and masonry structures. They reported that FRP can be useful in strengthening, improving ductility and reducing fragmentation. The authors noted that there are limited quantities of previous research on FRP retrofits for blast loading and that the response of structures is not fully understood. They also note that there are limited design guidelines available and therefore it is difficult to have confidence in the FRP retrofit for a wide range of structural element types under blast loads. The authors have also found that the sensitive nature of the research resulted in many research reports and papers lacking critical information on charge weight and standoff distances or pressure and impulse values making comparisons of results difficult. (Buchan & Chen, 2007)

2.4.19. J. E. Crawford et al.

The authors present design methodologies for retrofitting reinforced concrete columns with externally bonded FRP wraps to improve column resistance under blast loads. The design methodologies are based on the results of numerical modeling and experimental testing of columns under static loading with blast load shapes and under high explosive driven blast loads. The authors incorporated the design methodologies into a proprietary software package that accounts for flexural

resistance and direct and diagonal shear resistance. Flexural resistance is governed by moment capacity of the column incorporating moment redistribution due the formation of plastic hinges and includes second order effects. Diagonal shear capacity is calculated as a function of the capacities of the concrete and steel shear capacities as well as the shear capacity of the FRP composite wrap. The direct shear capacity is a function of the concrete strength and the column section size along with the axial load and the flexural steel area and strength (Crawford, Malvar, Morrill, & Ferritto, 2001).

The paper presents a minimal amount of experimental test results. Some of the conclusions and observations from the testing are as follows:

1. Columns lacking sufficient transverse reinforcement failed to meet the required shear demand and failed catastrophically in shear.
2. Transverse FRP wrap retrofits enhanced both diagonal shear capacity and moment capacity. Retrofitted columns, while still exhibiting some shear damage, were able to develop a flexural response.
3. Columns that failed in shear exhibited a rigid body displacement response with very little damage at the column mid-height.
4. Properly designed retrofit for columns including transverse and longitudinal FRP were able to remain elastic with no permanent deformation under blast loads that caused complete failure to as-built columns.
5. Static testing of columns with and without transverse FRP retrofit wraps show that the addition of transverse FRP can significantly increase the resistance of shear deficient columns.

2.5. Previous Research on FRP Strengthening Retrofits for Flexure

The use of FRP laminates applied along the length of a concrete flexure member is common for increasing flexural resistance. The surface bonded FRP provides an additional layer of tension reinforcement to the flexure members and therefore increases moment capacity. Typically FRP materials behave elastically and have high ultimate stresses, however they tend to debond from the concrete substrate prior to reaching full capacity. In Canada, the CSA standard Design and Construction of Building Structures with Fibre-Reinforced Polymers (CSA Standard S806-12, 2012) recognizes this debonding occurs and provides guidance on ultimate stress of the fibres. The standard provides an equation for calculating the maximum strain in FRP as a function of the concrete

strength, the number of plies, the elastic modulus of the FRP, and the thickness of the layup. This calculated maximum strain is limited to be less than 0.007. In general, the maximum allowable strain in flexural FRP only allows for the consideration of a fraction of the material strength prior to debonding. The standard also provides methods for calculating the development length of the FRP required based on the same properties.

Several researchers have conducted experiments on reinforced concrete members, either in simple supported bending or as cantilevers, with surface bonded FRP of various parameters. These tests may be used to confirm maximum FRP strain for flexure members in the case of extreme loading due to an explosion. These strain limits validate the conservative values provided in the CSA S806-12 standard. Some of this research is summarized below.

2.5.1. W.Seim et al.

W. Seim et al. conducted tests on a total of thirteen one way simply supported reinforced concrete slabs with a single point load applied at mid-span (Seim, Karbhari, & Seible, 2001). All slabs were identically dimensioned and detailed with total lengths of 2290mm, widths of 480mm and thicknesses of 102mm. The length between simple supports was 2030mm. Each slab was reinforced with 3 #3 deformed steel reinforcement in the longitudinal direction providing a reinforcement ratio of 0.54%. There were a total of 23 #2 deformed steel reinforcing bars spaced at 100mm placed in the transverse direction beneath the longitudinal reinforcement. The depth from the loaded surface to the centroid of the longitudinal reinforcement was 81mm. Of the thirteen slabs, two served as control specimens while the remainder were retrofitted with various configurations of surface bonded fibre reinforced polymers. The remaining slabs were retrofitted with one of three types of surface bonded fibre reinforcement. Three slabs had unidirectional carbon fibre polymer sheets applied using a wet layup technique, one slab had a sprayed glass/polyester composite retrofit, and seven slabs had strips of pultruded carbon fibre laminates.

The researchers found that the use of the externally bonded retrofit could increase load capacity by up to 370% over the capacity of the control specimens depending on the configuration and material of the retrofit. They noted that increasing strength of the members resulted in a brittle failure mode characterized by sudden failure of the composite system either by loss of bond or rupture of the fibres. In the control specimens, the slabs showed post peak load ductility capacity consistent with under reinforced specimens characterized by yielding of the reinforcement. The retrofits using

pultruded strips were found to be limited by the capacity of the adhesive where maximum FRP strain was limited to 0.65%, approximately half of the experimentally measured average ultimate composite strain of 1.22%. The researchers speculated that premature debonding of the pultruded strips was due to possible imperfections in the fibre-epoxy composite resulting in microscopic dry spots or voids between fibres and the brittle epoxy bonding the pultruded strips to the concrete fracturing and creating stress concentrations leading to debonding. It was noted that if transverse fibres were present, there may have been a mechanism for stress redistribution away from concentrations leading to better response. The wet layup unidirectional carbon sheets also increased the slab capacity over the control specimens while allowing for more deformability than the pultruded strip retrofitted specimens. The specimens reinforced with carbon sheets either failed in rupture of the fibres or by shear-flexure induced delamination after the shear capacity of the slab was exceeded. Maximum strains recorded in the carbon FRP during the tests was 0.93% at the point of rupture where the composite ultimate capacity was measured to be 0.95% strain in pure tension. The slab retrofitted with sprayed on glass fibres failed due to material rupture at an unreported strain. The failure load for the sprayed glass fibre slab was higher than the control specimens but lower than all other slabs.

2.5.2. M. Maalej and K.S. Leong

M. Maalej and K.S. Leong tested a series of seventeen simply supported reinforced concrete beams under four point bending with loads at the third points (Maalej & Leong, 2005). Three sizes of beams were cast including six that were 115mm wide by 146mm deep by 1650mm long with 1500mm between supports (series A), six that were 230mm wide by 292mm deep by 3300mm long with 3000mm between supports (series B), and five that were 368mm wide by 467mm deep by 5280mm long with 4800mm between supports (series C). All beams had the same longitudinal tension reinforcement ratio of 1.71%, longitudinal compression steel reinforcement ratio of 1.14% and transverse shear steel reinforcement ratio of 0.82%. Each beam had a single layer of tension reinforcement comprised of three bars and a single layer of compression reinforcement comprised of two bars. Of the seventeen beams cast, eleven were retrofitted with externally bonded CFRP sheets. In series A, two beams served as control specimens while two were retrofitted with one layer of CFRP sheets and two were retrofitted with two layers of CFRP sheets. In series B, two beams were control specimens, two beams had two layers of CFRP sheets and two beams had four CFRP sheets. In series C, two beams were control specimens, two had three layers of CFRP sheets, and one had six

layers of CFRP sheets. In all cases, the CFRP was applied only to the tension face of the beams and was uniform over the entire length terminating 25mm, 50mm and 80mm from the supports for series A, B, and C respectively. The same unidirectional CFRP with composite ultimate strain values of 1.5% was used for all beams.

The researchers found that the use of the CFRP retrofit increased the load capacity of the beams over the control specimens in all cases. For all control beams, the failure mode was crushing of concrete while all retrofitted beams saw intermediate flexural crack induced debonding of the FRP from the concrete substrate. The stiffness of the retrofitted beams was higher than the control specimens as expected. Failure of the retrofitted beams occurred at a lower displacement than the control specimens and, in general, the more FRP used, the lower the failure displacement was. The maximum FRP strain at debonding was highest for the smaller beams at 0.99% for one layer of FRP and 0.67% for two layers of FRP. The mid-sized beams in series B saw maximum FRP strains at debonding of 0.80% for two layers and 0.58% for four layer. The larger beams in series C saw maximum FRP strains of 0.67% for three layers and 0.37% for six layers of FRP. The authors concluded that the beams size does not significantly influence the strengthening ratio of the FRP when the FRP reinforcement ratio is held constant and that the size of the beam does not seem to affect the energy ductility of the CFRP strengthened beams.

2.5.3. R. Kotynia et al.

R. Kotynia et al. conducted tests on ten rectangular reinforced concrete beams spanning 4200mm between simple supports loaded with two equal point loads at the third point locations (Kotynia, Baky, Neale, & Ebead, 2008). All of the beams had a rectangular cross section of 150x300mm and were reinforced with 10mm and 12mm diameter steel ribbed bars with 6mm diameter steel stirrups spaced at 100mm in the shear span locations and 200mm over the mid-span. All of the beams were reinforced with some configuration of externally bonded CFRP. The different configurations of FRP reinforcement tested included pure flexure reinforcement with strips or sheets and the use of U-wraps and L-plates of FRP material to increase FRP bond.

The researchers found that all beams failed due to intermediate crack induced debonding of the FRP from the concrete substrate. The width of the FRP strip was a significant factor in the debonding mechanism with the wide laminates leading to a debonding plane that was at the location of the depth of the concrete cover and the thinner strips having a debonding plane closer to the surface.

The use of mechanical anchorage with U-wraps or L-plates increased the strain capacity of the FRP prior to debonding resulting in higher ultimate load capacity. The researchers found that the average FRP debonding strain was between 36% and 56% of the specified ultimate composite material strain. The maximum and minimum strain in the FRP material at debonding recorded during all tests were 0.62% and 0.86%.

2.5.4. H.B. Pham and R. Al-Mahaidi

H.B. Pham and R. Al-Mahaidi conducted tests on twenty six reinforced concrete beams with surface bonded CFRP (Pham & Al-Mahaidi, 2006). All of the beams had a cross section of 140x260mm reinforced with 3-12mm diameter longitudinal steel bars on the bottom and 2-12mm diameter bars on the top with 10mm diameter stirrups spaced at 100mm. The beams were either tested in four point or three point bending configurations. In the four point bending configuration, the total clear span was 2300mm between simple supports with a 900mm between symmetric load points. The three point bending test had a span of 1600mm with 700mm and 900mm spans between the load and the supports. Several configurations of FRP reinforcement were tested including variations on number of layers, variations on FRP development length, the use of mechanical anchorage with a U-straps and the spacing of the U-straps.

The authors found that the beams experienced one of two brittle failure modes: debonding at the FRP end or debonding at a flexure-shear crack. The use of the U-straps for mechanical anchorage improved the bond between the concrete and the FRP. The authors found that prestressed U-straps performed slightly better than non-prestressed straps.

2.5.5. H. Niu, A. Vasques, and V.M. Karbhari

H. Niu, A. Vasques, and V.M. Karbhari conducted tests on fourteen reinforced concrete slabs of various configurations retrofits with various configurations of surface bonded CFRP laminates (Niu, Vasquez, & Karbhari, 2006). Each of the slabs were 4500mm by 960mm by 203mm and reinforced with either 3#6 steel bars or 7#4 steel bars. Concrete strength ranged from 30.3MPa to 35.9MPa at the time of testing. The slabs were tested as one-way members under either 3-point or 4-point bending configurations. Two of the slabs served as control specimens and the others were reinforced with various configurations of externally bonded CFRP either in the form of wet layup sheets or pultruded plates. The CFRP varied in number of plies and the width of the plies from member.

The authors found that the use of the FRP resulted in a significant increase in load capacity over the control specimens and that the increase in load capacity for the slabs was directly proportional to the amount of FRP. The failure of the FRP was in the form of debonding from either the concrete surface or interlayer debonding of the FRP laminates.

2.5.6. S. Lee

S. Lee conducted tests on nine simply supported large scale reinforced concrete beams retrofitted with various configurations of CFRP (Lee & Moy, 2007). Three different geometries of beams were tested: 75mm x 200mm x 2000mm, 150mm x 400mm x 4000mm and 300mm x 800mm x 7500mm. The reinforcement ratio for all beams was between 1.09% and 1.13% and the cover depth was constant at 0.75d. Three of each beam size was tested under four point bending with load points at the third span locations. In each group one of the specimens served as a control specimen and the other two were fitted with externally bonded CFRP applied using a wet layup technique.

The researchers found that the use of the FRP increased load capacity by between 25% and 29% in all specimens. The maximum recorded FRP strains ranged from 0.64% to 0.77%. Failure of all beams with FRP was due to debonding for the FRP from the concrete substrate.

2.5.7. S.P. Chiew, Q. Sun, and Y. Yu

S.P. Chiew, Q. Sun, and Y. Yu conducted tests on twelve simply supported reinforced concrete beams retrofitted with surface bonded GFRP sheets (Chiew, Sun, & Yu, 2007). All of the beams were 2800mm long with a cross-section measuring 200mm by 350mm and reinforced with two 16mm diameter steel bars on the tension side and two 10mm diameter bars on compression side. 10mm diameter steel bars were spaced at 150mm as shear reinforcement. All beams were tested under four point bending with loads applied either at 500mm from mid-span or at 200mm. Each load configuration had six beams with one serving as a control specimen and the remainder reinforced with externally bonded GFRP laminates applied using a wet-layup technique. Various laminate thickness and bond length were investigated for this project.

The maximum recorded GFRP strain at failure ranged from 0.57% to .90%. In all cases the use of GFRP resulted in a higher moment capacity and stiffness over the control specimens. The strength increase was linearly proportional to the amount of GFRP applied to the beams. The researchers found that

the length of the bond over the shear span had little effect on the bond characteristics of the material as long as the bond length was greater than 56% of the constant moment region length.

2.5.8. E. Ahmed, H.R. Sobuz, and N.M. Sutan

E. Ahmed, H.R. Sobuz, and N.M. Sutan conducted tests on six reinforced concrete beams with CFRP laminates surface bonded to the tension face over a 1900mm simply supported span (Ahmed, Soubuz, & Sutan, 2011). The beams were 150mm wide, 200mm deep, and reinforced with 2-10mm diameter reinforcing steel bars on the tension face and 206mm diameter bars on the compression face. The CFRP laminate was 100mm wide applied to the tension face of the beam in lengths varying between 1100mm and 1500mm. The beams were loaded until failure with two equal point loads centered on the beam and spaced 500mm apart.

The researchers found that the strength of the beams was significantly increased proportionally to the number of laminate layers. The failure mode of the tests was debonding of the laminates from the concrete surface and crushing of the compression concrete. The use of transverse U-shaped CFRP edge strips bonded over the longitudinal strips increased the flexural capacity of the beams by up to 33% by delaying debonding of the FRP from the concrete substrates.

2.5.9. M. Arduini and A. Nanni

M. Arduini and A. Nanni conducted tests on eighteen simply supported reinforced concrete beams retrofitted with CFRP sheets (Arduini & Nanni, 1997). The beams had spans of either 1100mm or 2100mm with cross sections of 320mm wide by 160mm deep for the short beams and 160mm wide by 320mm deep for the long beams. The FRP retrofit layup varied in number of sheets, length of development, width of sheets, and angle of fibres. The beams were tested under four point bending up to failure.

The researchers found that the FRP was effective in increasing the ultimate load capacity of the beams. Accurate predictions of the displacement-load relationship was possible using simple analysis methods incorporating the effects of the FRP. The maximum strains that were found in the FRP at failure was up to 1.2% for one test and as low as 0.3% for others. Failure was due to the debonding of the FRP from the concrete substrate.

2.5.10. G. Spadea, F. Bencardino, and R.N. Swamy

G. Spadea, F. Bencardino, and R.N. Swamy tested four simply supported reinforced concrete beams with surface bonded CFRP sheets (Spadea, Bencardino, & Swamy, 1998). The beams were all 140mm wide by 300mm deep with 4800mm span between simple supports. All beams were tested under four point bending up to failure. The CFRP varied by the use of U-shaped steel anchor plates bonded to the beams over top of the FRP and to the sides of the concrete at load points and at the ends of the FRP material.

The researchers found that the use of the externally bonded steel sheets for mechanical anchorage was effective in increasing FRP strain from 50% of ultimate (0.7% strain) at failure for the beam with no mechanical anchors up to 86% of ultimate (1.2% strain) for the beam with the most anchors.

2.5.11. J.F. Bonacci and M. Maalej

J.F. Bonacci and M. Maalej tested seven simply supported reinforced concrete beams with surface bonded CFRP under monotonic and cyclic loads (Bonacci & Maalej, 2000). Furthermore, the authors subjected four of the 7 beams to accelerated corrosion for up to 140 days. The beams were 3650mm long between simple supports with 270mm width and 400mm depth and tested under four point bending up to failure. The FRP varied in number of plies and the use of U-straps to enhance bonding.

The researchers found that the maximum FRP strain measured was 0.98% at the point of CFRP rupture for one specimen while others failed in debonding at between 0.61% and 0.80%. In general, it was found that the moment resistance was increased by 10-35% and deflections were reduced by between 10 and 32%. Prior to failure, local debonding initiated at the midspan region and progressed towards the support. Corroded beams had lower resistance and higher deflections.

2.5.12. M.J. Chajes et al.

M.J. Chajes et al. tested fourteen simply supported reinforced concrete beams retrofitted with aramid, glass and graphite FRP (Chajes, Thomson Jr, Januszka, & Finch Jr, 1994). The beams were all 1118mm between simple supports with 127mm width and 76.2mm depth reinforced with either one or two #3 bars. The FRP varied in layup by material, number of plies and the use of U-shaped end anchors. The beams were tested under four point bending up to failure.

The researchers found that the use of FRP constantly increased the strength and stiffness of the beams by between 43% and 63% of the control beam. Failure was in the form of either debonding of the FRP from the concrete substrate or rupture of the FRP material.

2.5.13. J. Pan, T.C.F. Chung, and C.K.Y. Leung

J. Pan, T.C.F. Chung, and C.K.Y. Leung tested nine simply supported reinforced concrete beams retrofitted with CFRP laminate. The beams were tested under various load distributions ranging from two load points to eight load points over the 1800mm span (Pan, Chung, & Leung, 2009). The beams had cross sections measuring 150mm wide by 200mm depth and were reinforced with two 10mm diameter deformed steel bars. The number of plies of FRP along with the effective shear span and the number of loading points were parameters of the tests.

One beam failed in rupture, five in intermediate crack induced debonding and three in plate end debonding. It was found that the more uniform the load, the higher the debonding moment was. The beams with more plies tended to have plate end debonding rather than intermediate crack induced debonding. The authors concluded that typical debonding models derived from direct shear tests are not reliable for predicting debonding strains under uniformly loaded members.

2.5.14. J. Pan, C.K.Y. Leung, and M. Luo

J. Pan, C.K.Y. Leung, and M. Luo conducted tests on eight reinforced concrete beams with externally bonded CFRP (Pan, Leung, & Luo, Effect of multiple secondary cracks on FRP debonding from the substrate of reinforced concrete beams, 2010). The beams were all 2000mm long with 150mm wide by 200mm deep cross sections with a clear span of 1800mm between simple supports. All beams were reinforced with two 10mm diameter deformed reinforcing steel bars on the tension side and two of the same bars on the compression side. The beams were reinforced with surfaced bonded CFRP applied as a two layers on the tension face. Three of the beams had pre-existing cracks cut into the tension face prior to the application of the FRP. These cracks were cut deep enough to sever the steel reinforcement.

The authors found that the maximum FRP strain was 1.05% at the point of debonding for the case of multiple flexural cracks. In the case of beams with a single pre-existing crack caused premature debonding, maximum strain was 0.73%.

2.6. Previous Research on Prestress Jacketing for Confinement

Confinement of columns with externally applied prestressing strands is a technique first developed at the University of Ottawa. This technique uses prestressing strands to apply lateral confinement pressure to the faces of columns. Using this technique, columns see the benefit of confinement without having to first be placed under high flexure and axial load combinations that would otherwise be needed to activate the confinement effects of internal hoop steel.

Several researchers have tested a variety of configurations of column properties and prestressing for columns under seismic loads (Saatcioglu and Yalcin, 2003; Mess, 1999; Yalcin, 1998; and Beausejour, 2000). These researchers either tested circular columns or used risers and spreader frames to apply confinement pressure to the face of square columns. They found that the use of prestressing strands for confinement has beneficial effects on columns including:

- Increasing ductility capacity
- preventing buckling of reinforcement
- Enhancing bond in splice length deficient columns
- Increasing shear capacity

2.7. Conclusions from Previous Research

The research on reinforced concrete columns under blast loads and retrofit of them is very limited. Retrofit technologies, such as the use of FRP have been studied for reinforced concrete flexure members under blast loads, but the results are not definitive and there are no specific guidelines for design of these retrofits. What guidelines there are have been adopted from seismic research. There is a need for new and innovative retrofit techniques beyond the use of FRP. This research addresses the limitations in knowledge for the use of FRP on reinforced concrete columns and provides several alternate and novel retrofit techniques. It also gives guidelines and design procedures that may be used in the design of the retrofits.

Chapter 3. Experimental Program

3.1. General

An experimental program was designed to test the response of reinforced concrete columns subjected to shock waves and investigate various retrofit techniques designed to enhance column strength or ductility. Sixteen columns were constructed and tested for this program. The results from two columns previously tested (Lloyd, 2010) were used as control specimens for part of the experimental program. Specimens were designed to represent seismic and non-seismic main storey construction with the primary difference being spacing of transverse reinforcement along the height of the columns. Axial load was applied to levels consistent with typical service loads in structures. Three primary retrofits were investigated:

1. Externally bonded FRP sheets for confinement and strengthening
2. External confinement prestressing
3. External bracing

The retrofits were selected to either enhance column strength or ductility or a combination of both. Within each retrofit group type, various retrofit techniques were explored. These techniques included: changing the fibre direction of the FRP to explore the benefits of confinement jacketing, longitudinal strengthening, or the combination of the two; applying prestressed steel jacketing independently and in combination with steel compression bracing; and varying the geometry of tension bracing systems and changing the initial axial load level on the column. The details of the test specimens, specimen construction, material properties, retrofit construction and retrofit procedure, instrumentation, test setup, and test procedure are described in this chapter.

3.2. Description of Test Specimens

Sixteen approximately half scale reinforced concrete columns were constructed for this research program and two columns from previous research were used as control specimens (Lloyd, 2010). The columns were designed to represent first storey exterior columns. Six columns were designed for seismic zones and twelve for non-seismic zones where the difference between the two types was the detailing of the transverse hoop spacing. Each column was rectangular in cross-section with a clear span between supports of 1980mm and a total height of 2438mm. Two different cross-sections were

used where four columns were 100x150mm in cross-section (weak axis bending) and fourteen columns were 150x150mm in cross-section.

3.3. Description of Column Groups

The columns were classified in three groups based on construction type and construction date. Group 1 columns had the smaller cross-section of 100x150mm. Group 2 columns had 150x150mm cross-sections with a uniform cross-section over the entire column length. Group 3 columns had a 150x150mm cross-section that was uniform over the span of the column between the supports with small concrete foundations monolithically cast with the column at the top and bottom of the column spans. Several different retrofits were used on the columns. These retrofits are briefly discussed in the following sections and described in more detail in section 3.7.

3.3.1. Group 1 Columns

Group 1 columns had a uniform 100x150mm cross-section over the entire length of the column and were tested under weak axis bending. Group 1 columns were reinforced with a total of four 483MPa 10M deformed longitudinal reinforcing steel bars located one in each corner of the column. The longitudinal steel bars were 2400mm in length with 90° hooks extending 75 mm into the core of the columns, providing a reinforcing ratio of 2.67%. The cover to the longitudinal bars was 20mm for all columns. Transverse reinforcement was provided in the form of rectangular closed hoops with 135° hooks extending approximately 40mm into the core of the column. In all cases, the transverse hoops were 580MPa non deformed steel wire with a diameter of 6.3mm and a cross sectional area of 31.2mm². The volumetric ratio for the transverse steel varied between 1.52% for the non-seismic columns 1-NS-C and 1-NS-J and 3.04% for the seismic columns 1-S-C and 1-S-J. Transverse hoops were spaced at $h/2$ (50mm) for the non-seismic columns and $h/4$ (25mm) for seismically detailed columns and was constant for the entire height of all columns. A 350kN concentric axial load was applied to all columns prior to testing. This axial load equated to 35.6% of concentric capacity for columns 1-NS-C and 1-S-C and 36.7% of concentric capacity for columns 1-NS-J and 1-S-J. A summary of the column properties and detailing can be found in Table 3-1.

Two of the columns within group 1 were as-built control columns (1-NS-C and 1-S-C) and two columns (1-NS-J and 1-S-J) were retrofitted with a single layer of externally bonded Fyfe Co. Tyfo SCH-41 CFRP and epoxy laminate wrapped around the column forming a jacket that extended the entire clear span

of the column. The profiles and cross-sections of the group 1 columns are shown in Figure 3-1 and Figure 3-2.

3.3.2. Group 2 Columns

Group 2 columns had a uniform 150x150mm cross-section over the entire length of the column. All group 2 columns were reinforced with a total of four 483MPa 10M deformed longitudinal reinforcing steel bars located one in each corner of the column. The longitudinal steel bars were 2400mm in length with 90° hooks extending 75mm into the core of the columns. The longitudinal reinforcing ratio was 1.78%. The cover to the longitudinal bars was 10mm for all columns. Transverse reinforcement was provided in the form of rectangular closed hoops with 135° hooks extending approximately 40mm into the core of the column. In all cases, the transverse hoops were 580MPa non deformed steel wire with a diameter of 6.3mm and a cross sectional area of 31.2mm². The volumetric ratio for the transverse steel varied between 1.01% for the non-seismic columns 2-NS-C, 2-NS-J, 2-NS-R and 2-NS-JR and 2.01% for the seismic columns 2-S-C, 2-S-J, 2-S-R and 2-S-JR. Transverse hoops were spaced at $h/2$ (75mm) for the non-seismic columns and $h/4$ (37.5mm) for seismically detailed columns and was constant for the entire height of all columns. A 422.8kN concentric axial load was applied to all columns prior to testing. This axial load equated to 25.1% of concentric capacity for the group 2 columns. A summary of the column properties and detailing can be found in Table 3-1.

Two of the columns within group 2 were as-built control columns (2-NS-C and 2-S-C). Two columns (2-NS-J and 2-S-J) were retrofitted with a single layer of externally bonded Fyfe Co. Tyfo SCH-41 CFRP and epoxy laminate wrapped around the column forming a jacket that extended the entire clear span of the column. Two columns (2-NS-R and 2-S-R) were retrofitted with a single layer of externally bonded Fyfe Co. Tyfo SCH-41 CFRP and epoxy laminate applied to both the loaded and unloaded face of the column as 120mm wide strips extending the entire clear span of the column. Two columns (2-NS-JR and 2-S-JR) were retrofitted with a single layer of externally bonded Fyfe Co. Tyfo SCH-41 CFRP and epoxy laminate applied to both the loaded and unloaded face of the column as 120mm wide strips extending the entire clear span of the column with a single layer of externally bonded Fyfe Co. Tyfo SCH-41 CFRP and epoxy laminate wrapped around the column and the longitudinal CFRP laminates forming a jacket that extended the entire clear span of the column. The profiles and cross-sections of the group 2 columns are shown in Figure 3-3 and Figure 3-4.

3.3.3. Group 3 Columns

Group 3 columns had a uniform 150x150mm cross-section over the entire span of the column between the supports with small foundation sections cast monolithically with the column at top and bottom of the column. All group 2 columns were reinforced with a total of four 497MPa 10M deformed longitudinal reinforcing steel bars located one in each corner of the column. The longitudinal steel bars were 2082mm in length with 90° hooks extending 200mm into the foundations at the top and bottom of the columns. The foundation sections were 584mm wide with a depth of 300mm and a height of 229mm. The foundations were reinforced with four rectangular 10M ties and eight 10M reinforcement bars. The hooks of the longitudinal reinforcement steel were placed inside the four rectangular foundation ties. The longitudinal reinforcing ratio for all columns was 1.78%. The cover to the longitudinal bars was 10mm for all columns. Transverse reinforcement was provided in the form of rectangular closed hoops with 135° hooks extending approximately 40mm into the core of the column. In all cases, the transverse hoops were 580MPa non deformed steel wire with a diameter of 6.3mm and a cross sectional area of 31.2mm². The 75mm spacing of transverse hoops resulted in a volumetric ratio for the transverse steel was 1.01% for columns 3-NS-C, 3-NS-CB, 3-NS-TB1, 3-NS-TB2 and 3-NS-TB3. Column 3-NS-W had transverse hoops spaced at 150mm resulting in a volumetric ratio for the transverse steel of 0.50%. A 381kN concentric axial load was applied to column 3-NS-C prior to testing corresponding to 31.3% of the column concentric capacity. Column 3-NS-W, 3-NS-CB, 3-NS-TB1 and 3-NS-TB2 had an initial axial load of 422.8kN corresponding to 34.7% of the concentric capacity. Column 3-NS-TB2 had an initial axial load of 141.0kN corresponding to 11.6% of the column concentric capacity. A summary of the column properties and detailing can be found in Table 3-1.

Column 3-NS-C was built as a non-seismic as-built control column for group 3. The profiles and cross-section for this column are shown in Figure 3-5.

Column 3-NS-W was retrofitted with a series of external fibre reinforced concrete wedges and prestressed high strength straps providing confinement. A total of 60 prestress straps with cross-section areas of 13.5mm² and average measured yield strengths of 1021MPa. The straps were initially prestressed to approximately 300MPa. The profiles and cross-section of column 3-NS-W is shown in Figure 3-6. Column 3-NS-CB was retrofitted with an external steel compression brace as well as similar fibre reinforced concrete wedges and external hoop prestressing as column 3-NS-W. The compression brace was made from cold rolled mild-steel with a specified yield strength of 410MPa

(ASTM, 2012). The brace was comprised of 25.4x50.8mm square steel compression members that braced the column at two locations along the length. The concrete wedges were cast with reliefs to accommodate the compression braces while still being in contact with the column sides. The profiles and cross-section of column 3-NS-CB is shown in

Figure 3-7. Columns 3-NS-TB1, 3-NS-TB2 and 3-NS-TB3 were retrofitted with external tension braces that braced the columns at two points along the span. The tension braces were comprised of high-strength steel threaded rods with a cross-section area of 621mm² and a specified yield strength of 1040MPa. High strength StressProof (Niagara Lasalle Corporation, 2012) shear pins with cross-sectional areas of 792mm² and specified yield strength of 689.5MPa were used at the supports and at the interior span brace points. Drawn over mandrel steel tubes with a specified yield strength of 414MPa (ASTM, 2012) were used to manufacture hinges to connect the tension brace members at the brace points and the supports. Columns 3-NS-TB1 and 3-NS-TB3 used steel bearing plates at the interior brace points that were 19mm in thickness. Column 3-NS-TB2 used steel bearing plates at the interior brace points that were 146mm in thickness. The profiles and cross-sections for columns 3-NS-TB1, 3-NS-TB2 and 3-NS-TB3 are shown in Figure 3-8 and Figure 3-9.

3.4. Description of Test Setup

Experimental testing was conducted using the University of Ottawa Shock Tube testing facility.

3.4.1. Shock Tube Operation

The shock tube used to apply shock wave loading can simulate a wide range of explosions with varying pressures and impulses. The shock tube is comprised of a variable length driver section, a double diaphragm firing control section with a spool section located between the diaphragms, an expansion section and a rigid end frame as shown in Figure 3-10 and Figure 3-11. The driver section is a series of 597mm inside diameter pipes with 19mm wall thickness. The length of the driver section is variable between 305mm and 5185mm in 305mm increments. Depending on the desired shock wave parameters, including peak pressure and total impulse, a driver length is selected. The spool section is a 90mm long FIKE Combination Disk Holder (Fyke Corporation, 2005). The flanges on the spool section are designed to mate with the flanges on the driver and the flange located between the spool and the beginning of the expansion section. The flanges have 30° inlets that mate with protrusions on the coupling flanges of the driver and expansion section (Figure 3-10). When held together with 20 Grade 8 high-strength threaded rods 31.75mm in diameter, the driver-spool-

expansion section flanges clamp the diaphragms in place. There are two diaphragms employed in the operation of the shock tube. Each diaphragm is comprised of up to three sheets of 813mm by 813mm grade 1100 aluminum foil of varying thickness. The foils have twenty 38mm holes in a circular pattern centered on a radius of 374.65mm with an angle of 18° between holes (Figure 3-12). These holes are provided to accommodate the twenty threaded rods that clamp the driver and spool to the expansion section. The expansion section connects to the driver-spool combination and to the rigid end frame. It begins with a 1920mm pipe splay as a transition from the circular driver cross section to the square expansion section cross section. Over a length of 6096mm the cross section of the expansion section increases from 597mm diameter circular to a 2032mm by 2032mm square opening at the rigid end frame. The entire length of the square expansion section is comprised of 12.7mm thick stiffened steel plates. The end frame is square with a 2032mm by 2032mm opening and a 203.2mm wide, 12.7mm thick stiffened steel plate surrounding the opening. There are a total of twenty 19 mm holes located in the end frame for connecting samples to the shock tube.

The operation of the shock tube involves charging the driver and spool simultaneously to desired pressures and then triggering firing by draining pressure from the spool section which causes rupture of the diaphragms. Diaphragm combinations are chosen based on rupture pressures of the diaphragms and desired pressures in the driver.

3.4.2. Shock Tube Capabilities

The shock tube has been shown to simulate a wide range of high explosive driven shock waves. The operating range, defined in terms of peak reflected pressure and reflected impulse over the positive phase, is shown in Figure 3-13 and in Table 3-2. In the tabulated values, equivalent TNT mass, standoff distances, and scaled distances for hemispherical explosions are provided for the maximum shock tube capacities.

For any combination of driver length and driver pressure, there is a unique pressure and impulse combination that will be generated. The relationships between driver pressure and reflected pressure for various driver lengths are shown in Figure 3-14. The relationship between driver pressure and reflected impulse for various driver lengths are shown in Figure 3-15. These relationships, which were defined in the initial shock tube calibration and updated as more test data became available (Lloyd, 2010 and Lloyd et al., 2010), were used to select the driver pressure and driver length to generate the desired shock wave properties.

3.4.3. Axial Load Mechanism

Axial load was applied to the columns using a hydraulic jack located between the base of the column and the slab on grade below the column. A steel block was placed at the top of the column between the column end and the 914mm strong floor above that formed the ceiling of the test area. Notched steel plates with round bars serving as rotational pins were placed between the column end and the hydraulic jack at the bottom and the column end and the steel bearing block at the top. The column axial load mechanism is shown in Figure 3-16. The axial load was applied prior to tightening the lateral supports of the column.

3.4.4. Lateral Load Transferring Devices

As the columns did not cover the entire opening of the shock tube at the test frame location, a load transferring device (LTD) was employed to collect the shock wave loading and apply it to the surface of the columns using a series of transverse steel beams and sheet metal. Two different load transfer devices were used in the testing program.

3.4.4.1. LTD A – Group 1 Columns

LTD A was used to test Group 1 columns. It consisted of fifteen 76.2mm by 76.2mm by 4.8mm thick hollow structural steel transfer beams which were attached to a 0.51mm thick steel sheet at 127mm on center. The beams were perpendicular to the column and were 2438mm long. The steel sheet covered the entire area of the end frame and the shock tube opening, 2438mm by 2438mm. Neither the steel beams nor the sheet metal were attached to the end frame during testing aside from being supported at the top to prevent falling during testing and therefore were free to move laterally and distribute the load entirely to the column being tested. The mass of the load transfer device was calculated to be 394kg. Figure 3-17 shows LTD A mounted on the shock tube.

3.4.4.2. LTD B – Group 2 and Group 3 Columns

LTD B was used on Group 2 and Group 3 columns. LTD B was made by modifying LTD A to reduce the mass of the system while still applying a relatively uniform load over the length of the column. It consisted of the same HSS transverse beams that were cut to a shorter length of 2083mm which was slightly wider than the 2032mm width of the shock tube opening. The top and bottom transverse bars had a 6.3mm thick, 177.8mm long, and 76.2mm wide steel plate welded at the centre of the beams where the column was loaded. The six interior transverse beams had 177.8mm long sections of the transverse beams welded to the top and bottom of the beams at the centre of the beams.

These beam sections were spaced 6.3mm proud of the transfer beams so that they would apply load to the column. The combination of the interior and top and bottom beams provided a total of fourteen load points that were relatively uniformly spaced over the length of the system. The beams were bolted to a 0.51mm thick steel sheet that served to collect the pressure from the shock wave loading and transfer it to the beams which, in turn, transferred the force to the column. The top beam was attached to the top lateral support of the column with a short length of chain to prevent the load transferring device from falling during testing. The total system mass of the load transfer device was calculated to be 209kg. Figure 3-18 shows LTD B mounted on the shock tube.

3.4.5. Support Conditions

For all tests the columns were restrained at both ends against lateral movement and to a degree against rotation. There were two types of lateral restraints employed for the tests.

3.4.5.1. Support A – Group 1 Columns

Group 1 columns were clamped between two steel plates at the top and bottom of the columns. The plates were welded to structural channels on the supports attached to the shock tube and to hollow structural steel sections on the outside of the columns. There were a total of eight 15.9mm bolts at each support location that clamped the column between the supports.

Figure 3-19 shows a column mounted to the shock tube with the Support A system and Figure 3-20 shows the entire mounting device for Group 1 columns including LTD A and Support A systems.

3.4.5.2. Support B – Group 2 and Group 3 Columns

The support system employed for Group 1 columns was considered deficient in rotational restraint and a new support system, Support B, was built for Group 2 and Group 3 columns. The support system consisted of hollow structural steel beams that extended the entire width of the shock tube test frame with perpendicular hollow structural steel beams welded to the ends of the beams. The beams were bolted to the shock tube at six locations along the width of the shock tube and the perpendicular beams were bolted to the sides of the shock tube at one location to provide a rotational resistance mechanism. At the location of the column support, a 12.7mm thick steel plate was welded to both sides of the beams and eight additional 15.9mm bolts were used to clamp the column between the supports. Figure 3-21 shows a column mounted to the shock tube with the Support B system and Figure 3-22 and Figure 3-23 shows the entire mounting device for Group 2 and Group 3 columns including LTD B and Support B systems.

3.5. Construction of Specimens

The construction of specimens was completed in multiple stages at the University of Ottawa Structures Laboratory. These stages included cutting and bending of steel ties, assembly of steel reinforcement cages, application of electrical resistance strain gauges, construction of formwork, and casting of concrete.

Bending of the steel ties was done using a hand bending jig designed to provide exact dimensions of the steel ties. The non-deformed 6.3mm steel wire was first cut to the length needed for each tie. Three 90° bends and two 135° bends completed a closed hoop tie. The steel ties were placed on the longitudinal reinforcement and tied in place with twisted steel wire. Strain gauges were applied to the longitudinal reinforcement by first smoothing the deformations on the bars followed by cleaning with a weak acid and weak base. The gauges were glued to the bars and wire leads were soldered and insulated with tape and silicone. Formwork was constructed from 19mm thick plywood. The columns were cast horizontally with a hand floated surface on the side of the column. After casting, the columns were covered and watered daily to limit heat during curing.

3.6. Material Properties

3.6.1. Concrete

Four concrete mixes were used for the columns. Mix one for columns 1-NS-C and 1-S-C had a compressive strength of 58MPa at the testing date. Mix two for columns 1-NS-J and 1-S-J had a compressive strength of 56.2MPa at the time of testing. Both of these concrete mixes used type 3 high early strength cement with a 10mm maximum size crushed siliceous aggregate.

Group 2 columns all were cast at the same date using the same ready mix concrete with 10mm maximum size crushed siliceous aggregate. Compressive strength of the concrete for group 2 columns was measured at 69.6MPa at the time of testing. Group 3 columns also were cast with ready mix concrete using 10mm maximum size siliceous aggregate. Compressive strength for group 3 columns was measured to be 49.5MPa at the time of testing.

Concrete compression strengths were measured using a Forney testing machine with a 2200kN maximum capacity. Sets of three cylinders were tested at different dates leading up to the testing program. Prior to testing, each cylinder was ground smooth on both ends to ensure a uniform

pressure distribution during testing. Compressive strengths for the concrete used in the columns are listed in Table 3-1.

3.6.2. Reinforcing Steel

Longitudinal steel reinforcement for all of the columns consisted of four 10M (10mm diameter) deformed reinforcing bars. Group 1 and group 2 columns had steel with a yield strength of 483MPa and group 3 columns had steel with a yield strength of 497MPa. Transverse steel ties were fabricated from 6.3mm diameter non deformed steel wire with a yield strength of 580MPa for all columns. Coupon tests were performed on sections of the steel bars to determine actual yield strength, ultimate strength, and strain at ultimate strength. The stress strain curves for all steel types are shown in Figure 3-24. Tabulated values for steel properties for each column are provided in Table 3-1.

3.6.3. CFRP

Two columns in Group 1 and six columns in Group 2 were retrofitted with externally bonded carbon fibre reinforced polymer (CFRP) sheets. The Tyfo SCH-41S-1 (Fyfe Co. LLC., 2009) system was used for all retrofits. This CFRP composite consists of a unidirectional carbon fibre fabric with a glass cross-fibre was saturated with a two part Tyfo S epoxy. The specified laminate material properties provided by the manufacture are 876MPa tensile strength, 72.4GPa tensile modulus, 1mm composite thickness, and 1.2% ultimate strain. The dry fibre properties are specified as 3790MPa ultimate tensile strength, 230GPa tensile modulus, and 1.7% ultimate strain. The tensile stress-strain relationship for the composite laminate is shown in Figure 3-25.

3.6.4. Confinement Prestress Material

Two columns in Group 3 were retrofitted with a combination of transverse hoop prestressing and fibre reinforced concrete wedges. The material properties for the concrete and steel used in the retrofit are provided below.

3.6.4.1. Concrete

The concrete wedges used in the retrofit were reinforced with 1% by volume Dramix ZP305 fibres to provide a minimal amount of crack control to the wedges to prevent damage during installation of the retrofit. The 30mm long, hooked-end fibres were made of steel wire with a tensile strength of 1100MPa. At the time of testing, the fibre-reinforced concrete had a compressive strength of 32MPa.

3.6.4.2. Steel

The steel prestressing material used in the retrofit was 0.71mm by 19mm in cross-section for a total area of 13.5mm². The strength of the material was measured to be 1021MPa at yield. When using the pinched coupler connection, the material was found to have an ultimate strength of 920MPa at rupture. The stress-strain relationship for the material with and without the coupler is shown in Figure 3-26.

3.6.5. Compression Brace Material

One column in Group 3 employed a compression brace retrofit. All of the material used in the compression brace was mild steel with an assumed yield strength of 350MPa and assumed elastic modulus of 200GPa.

3.6.6. Tension Brace Material

Three of the columns in Group 3 used a tension brace retrofit. This retrofit consisted of high strength 31.75mm steel threaded rods, custom built steel connectors, high strength steel shear pins, and mild steel bearing plates.

3.6.6.1. Threaded Brace Rods

The 31.75mm steel threaded rod used for the tension brace was tested under tension loading using a universal testing machine. The force-displacement relationship for the rods was recorded and used to find an effective cross-section area and equivalent stress-strain relationship. Using an assumed elastic modulus of 200GPa, an effective cross-section area of 621mm² was calculated. An assumed linear-elastic stress-strain relationship for the threaded rods up to an assumed ultimate stress of 720MPa is shown in Figure 3-27.

3.6.6.2. Connector Elements and Bearing Plates

Custom built connector elements and bearing plates were used to connect the threaded rods and to attach the retrofit to the foundations of the column. These elements were constructed using mild steel with an assumed yield stress of 350MPa and assumed elastic modulus of 200GPa.

3.6.6.3. Shear Pins

38.1mm diameter high strength shear pins were used at the locations of the interior bearing points of the brace retrofit and the column. These shear pins were constructed from StressProof steel

(Niagara Lasalle Corporation, 2012) with a specified yield strength of 689MPa, a specified ultimate tension strength of 793MPa, an elastic modulus of 200GPa, and an ultimate tensile strain of 8%.

3.7. Retrofit Procedures

Several different retrofit techniques were used in this research project. The following sections detail installation of the retrofits on the columns.

3.7.1. Externally Bonded CFRP

The application of the externally bonded CFRP was done using the manufacture's specified wet layup technique. A summary of the procedure is described below.

- Externally bonded CFRP was applied to the columns by first ensuring the column surface was smooth and clear of defects. The column surfaces were smoothed with a masonry grinding disk on an angle grinder. For the columns that had confinement retrofits, the corners were rounded to a 25mm radius with the use of an angle grinder. Any defects in the column surface were filled with an epoxy mixture thickened with silica fume.
- Once the surface treatment epoxy cured, the column surfaces were smoothed again with an angle grinder and a belt grinder.
- FRP was cut to the desired lengths and widths.
- With the dry FRP fabric on a plastic sheet, the material was saturated with epoxy using a foam roller.
- A layer of epoxy was applied to the surface of the column.
- The wet FRP fabric was placed on the wet surface of the column.
- A final layer of epoxy was applied to the FRP fabric that was placed on the column.
- Using a combination of a ribbed steel roller and plastic trowels, the surface of the FRP sheet was smoothed to the surface of the column and any air was forced from the system.
- The wet FRP layup was allowed to cure for several days prior to moving the columns and allowed to cure for at least 10 days prior to testing.

3.7.2. External Steel Prestressing

The first step in the external prestressing retrofit was casting the concrete wedges. Figure 3-28 shows the concrete wedge construction process. The following steps were done to construct the concrete wedges:

- 317.5mm diameter circular cardboard forms were cut to 305mm lengths.
- 25mm wide strips of the same forms were cut and glued to the interior surface of the circular forms at 30.5mm on centre spacing. These formed reliefs for the prestress steel straps.
- A 100mm wide, 305mm long strip of the cardboard form was cut and glued to the inside surface of the form to provide a relief in the location of the prestress anchors.
- 305mm long, 150mm by 150mm plywood forms were built to provide a relief for the concrete column when casting the concrete wedges.
- Four sections of 305mm long plywood were cut and installed at the corners of the plywood form that extended out at 45° to connect the plywood form to the cardboard form.
- Concrete was mixed in the University of Ottawa Structures lab and placed in the forms with the use of a small concrete vibrator to ensure even concrete distribution.
- The concrete was allowed to cure for seven days prior to removing of the formwork.
- The concrete wedges were labeled according to what form they came from.
- The concrete wedges were allowed to cure for 20 days prior to testing.

After the concrete wedges were cured, the application of the wedge-prestress retrofit was completed using the following steps:

- Steel straps were cut to the desired length and couplers were placed on the straps as shown in Figure 3-29.
- The bottom set of concrete wedges were placed on the column and spaced off the foundation by 20mm.
- A steel strap was loosely placed around the concrete wedges at one of the mid-height of the wedge reliefs.
- A pneumatic tool was used to prestress the strap and a separate pneumatic tool was used to pinch the coupler at three locations securing the strap to the column.
- The other nine straps on the set of wedges were installed in a staggered manor using the same procedure.
- The other wedges were installed using the same procedure.

The application of the retrofit to the column is shown in Figure 3-30.

3.7.3. Compression Brace

The compression brace retrofit employed the use of the concrete wedges and external prestress as well as an external steel brace. The same procedure for casting and installing the concrete wedges and prestress as described above was used with the exception of the additional reliefs left in the concrete wedges to accommodate the compression brace.

The compression brace was constructed from mild steel components. Figure 3-31, Figure 3-32, and Figure 3-33 show the individual components of the compression brace. These components were constructed by precisely measuring and cutting pieces to fit the column. The connection of the brace components was done with bolts and welds. The installation of the retrofit was done using the steps described below with pictures of the installation process provided in Figure 3-34.

- The column was mounted on the shock tube and axial load was applied.
- The diagonal compression brace components were installed by bolting the base plates to the column foundation using a minimum of two 12.7mm diameter concrete anchor bolts.
- The vertical brace members and the bearing plates were installed and bolted to the diagonal brace members. The vertical brace members were precisely ground to length to fit and clamped in place prior to bolting to the bearing plates and the diagonal members.
- The concrete wedges were installed using the procedure described in the previous section and the clamps holding the brace members to the column were removed.

3.7.4. Tension Brace

The same tension brace components were used to retrofit three columns. The first step in the retrofit was the construction of the brace components that are shown in Figure 3-35 through Figure 3-39. The construction was done using the following steps:

- The square steel stock forming parts of the hinge couplers and end bearing plates were cut to length.
- Holes were drilled in the material using a lathe.
- Holes were tapped to allow the connection of the threaded rods.
- Sections of the drawn over mandrel round steel that would form the hinges were cut to the desired width using a band saw.
- The existing holes in the hinge sections were trued using a lathe.

- The shear pins were cut to length.
- The bearing plate sections were cut to length.
- The sections of hexagonal steel bar that would form the turnbuckles were cut to length.
- Through holes were drilled in the hexagonal sections using a lathe
- One end of each of the hexagonal pieces was tapped with a right hand thread and one with a left hand thread creating turnbuckles.
- The sections of square, tapped steel that formed part of the hinges were ground down in the location of the welds to the round hinge components to allow for significant weld penetration.
- The round hinge pieces were ground at the weld location to allow for weld penetration.
- The hinge components were clamped together prior to welding.
- The hinge components were welded using a systematic procedure to reduce warping of the sections and limit weld induced stresses.
- The end bearing connection elements were welded together.
- The bearing plate elements were welded together.
- Threaded rod sections were cut to length with care to ensure a minimum of 50mm of rod extended into the tapped sections of the hinge components.

Installation of the retrofit was done in stages with the column off and on the shock tube. The installation of the retrofit is shown in Figure 3-40 and described below:

- With the column horizontal, the end bearing connectors were installed. At least four 12.7mm high strength threaded rods extending through the foundation were used to bolt the end elements to the foundation.
- The bottom and top hinge elements and shear pins were installed by bolting them to the end elements with a portion of the threaded rod tension brace.
- The column was mounted to the shock tube.
- The interior hinge elements were installed by bolting them to the bottom and top hinges using the threaded rod tension brace elements.
- The bearing plates were placed under the shear pins on the surface of the columns.
- The turnbuckle pieces were installed on the vertical components of the tension brace and lightly tightened.

- Axial load was applied to the column and the supports were secured.
- Any slack in the retrofit was removed by lightly tightening the turnbuckles taking care not to preload the system.

3.8. Instrumentation and Data Acquisition

Each column was instrumented with a series of electrical resistance strain gauges located at various locations on the internal longitudinal reinforcing steel. Additional strain gauges were also used on some of the external retrofit materials. The column displacement of the column at various points was measured using linear variable displacement transducers that were bolted to the column and mounted to a rigid frame outside of the test area. The pressure-time histories were measured using piezoelectric pressure sensors mounted on the shock tube flange close to the reflecting surface of the load transfer devices.

The instrumentation was monitored with a high speed digital oscilloscope that recorded time histories at 100,000 samples per second for a duration of approximately 3 seconds. The oscilloscope was set to record a loop of data for a 3 second duration at which point it would begin overwriting previously saved data. When the shock wave front reached a pressure sensor located near the reflecting surface, the data acquisition system would begin permanently saving the data including a baseline data history of 10% of the 3 second duration prior to the arrival of the shock wave front. The data acquisition system was also programed to send a trigger to a high speed camera set to record at 500 frames per second. This trigger would initiate recording of the video time history of the column response and synchronize it to the data time history recorded on the oscilloscope.

3.9. Test Procedure and Loading Program

Prior to testing, the load transfer mechanism was installed on the end of the shock tube. The bottom of each column was then placed on the hydraulic jack used to apply the axial load and the supports at the top and bottom of the column were installed. The axial load was applied prior to tightening the supports. The strain gauge leads were then connected to the data acquisition unit and displacement sensors were attached to the column and connected to the data acquisition system. The high speed camera and lights were set to record the column response. Photographs of the specimen before and after each test were taken to document damage.

Each column was exposed to multiple shots in order to explore the column response in both the elastic and inelastic regions. The driver pressure and driver length were selected to produce the desired reflected pressure and reflected impulse at the location of the test specimen.

Table 3-1: Column properties

Group	Name	Cross Section Width	Cross Section Height	Clear Cover	Longitudinal Reinforcement Ratio	Spacing of Transverse Reinforcement	Volumetric Ratio of Transverse Steel	f'_c	F_y	F_{yh}
		mm	mm	mm	%	mm	%	MPa	MPa	MPa
1	1-NS-C	150	100	20	2.67	50	1.52	58	483	580
	1-S-C	150	100	20	2.67	25	3.04	58	483	580
	1-NS-J	150	100	20	2.67	50	1.52	56.2	483	580
	1-S-J	150	100	20	2.67	25	3.04	56.2	483	580
2	2-NS-C	150	150	10	1.78	75	1.01	69.6	483	580
	2-S-C	150	150	10	1.78	37.5	2.01	69.6	483	580
	2-NS-J	150	150	10	1.78	75	1.01	69.6	483	580
	2-S-J	150	150	10	1.78	37.5	2.01	69.6	483	580
	2-NS-R	150	150	10	1.78	75	1.01	69.6	483	580
	2-S-R	150	150	10	1.78	37.5	2.01	69.6	483	580
	2-NS-JR	150	150	10	1.78	75	1.01	69.6	483	580
	2-S-JR	150	150	10	1.78	37.5	2.01	69.6	483	580
3	3-NS-C	150	150	10	1.78	75	1.01	49.5	497	580
	3-NS-W	150	150	10	1.78	150	0.50	49.5	497	580
	3-NS-CB	150	150	10	1.78	75	1.01	49.5	497	580
	3-NS-TB1	150	150	10	1.78	75	1.01	49.5	497	580
	3-NS-TB2	150	150	10	1.78	75	1.01	49.5	497	580
	3-NS-TB3	150	150	10	1.78	75	1.01	49.5	497	580

Table 3-1: Column properties - Continued

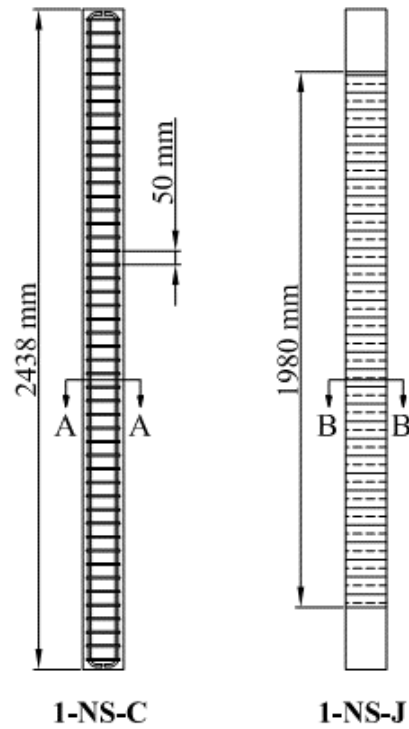
Group	Name	Initial Axial Load kN	P/P _o %	Initial Axial Strain %	Support Type	LTD Type	Retrofit Details
1	1-NS-C	350	35.6	0.0603	A	A	Non-seismic control (as-built)
	1-S-C	350	35.6	0.0603	A	A	Seismic control (as-built)
	1-NS-J	350	36.7	0.0611	A	A	One layer of SCH-41 CFRP wrap applied over the entire clear span of non-seismic column
	1-S-J	350	36.7	0.0611	A	A	One layer of SCH-41 CFRP wrap applied over the entire clear span of seismic column
2	2-NS-C	423	25.1	0.0465	B	B	Non-seismic control (as-built)
	2-S-C	423	25.1	0.0465	B	B	Seismic control (as-built)
	2-NS-J	423	25.1	0.0465	B	B	One layer of SCH-41 CFRP wrap applied over the entire clear span of non-seismic column
	2-S-J	423	25.1	0.0465	B	B	One layer of SCH-41 CFRP wrap applied over the entire clear span of seismic column
	2-NS-R	423	25.1	0.0465	B	B	One layer of SCH-41 CFRP longitudinal strengthening applied as 120mm wide strip on both loaded and unloaded faces terminating at supports of non-seismic column
	2-S-R	423	25.1	0.0465	B	B	One layer of SCH-41 CFRP longitudinal strengthening applied as 120mm wide strip on both loaded and unloaded faces terminating at supports of seismic column
	2-NS-JR	423	25.1	0.0465	B	B	One layer of SCH-41 CFRP longitudinal strengthening applied as 120mm wide strip on both loaded and unloaded faces terminating at supports with one layer of SCH-41 CFRP wrap applied over the entire clear span of non-seismic column
	2-S-JR	423	25.1	0.0465	B	B	One layer of SCH-41 CFRP longitudinal strengthening applied as 120mm wide strip on both loaded and unloaded faces terminating at supports with one layer of SCH-41 CFRP wrap applied over the entire clear span of seismic column

Table 3-1: Column properties - Continued

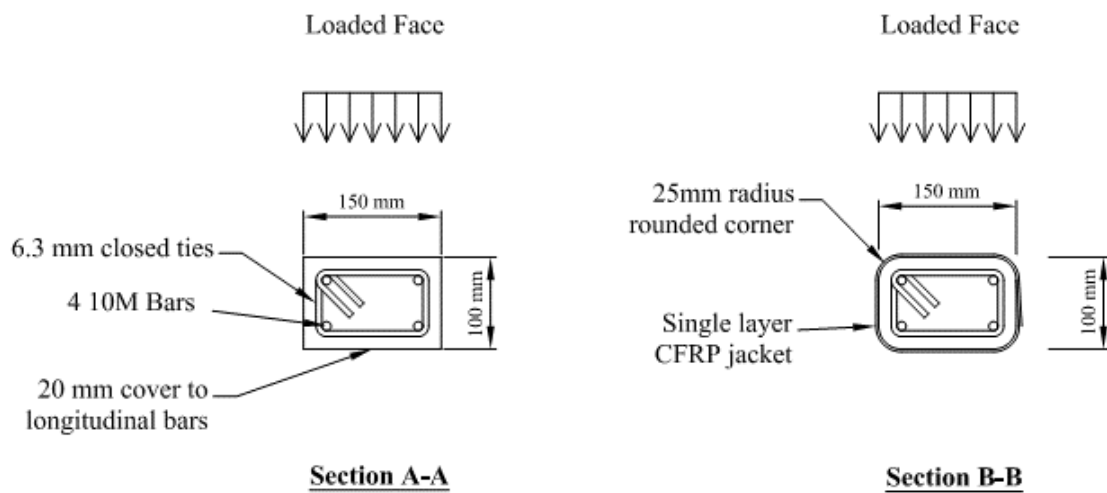
Group	Name	Initial Axial Load kN	P/P _o %	Initial Axial Strain %	Support Type	LTD Type	Retrofit Details
3	3-NS-C	381	31.3	0.0489	B	B	Non-seismic control (as-built)
	3-NS-W	423	34.7	0.0542	B	B	Six sets of concrete wedges with external prestress confinement strapping applied approximately every 32mm over the entire clear span of the non-seismic column
	3-NS-CB	423	34.7	0.0542	B	B	Six sets of concrete wedges with external prestress confinement strapping applied approximately every 32mm over the entire clear span of the non-seismic column with 1290mm ² compression brace steel on both sides of the column bracing the column at two points on the span
	3-NS-TB1	423	34.7	0.0542	B	B	621mm ² area tension braces on both sides of the non-seismic column bracing the column at two points on the span
	3-NS-TB2	423	34.7	0.0542	B	B	621mm ² area tension braces on both sides of the non-seismic column bracing the column at two points on the span with the brace points on the column span spaced away from the column face by 127mm further than columns 3-NS-TB1 and 3-NS-TB3
	3-NS-TB3	141	11.6	0.0181	B	B	621mm ² area tension braces on both sides of the non-seismic column bracing the column at two points on the span

Table 3-2: Maximum Shock Tube Capacities

Driver Length	Reflected Pressure	Reflected Impulse	Equivalent TNT Mass	Standoff	Scaled Distance
m	kPa	kPa-ms	Kg	m	m/kg ^{1/3}
305	78	217	8	12	6.0
915	92	410	42	18	5.2
1830	100	840	290	33	5.0
3355	103	1760	2500	67	4.9
4880	104	2690	10000	106	4.9

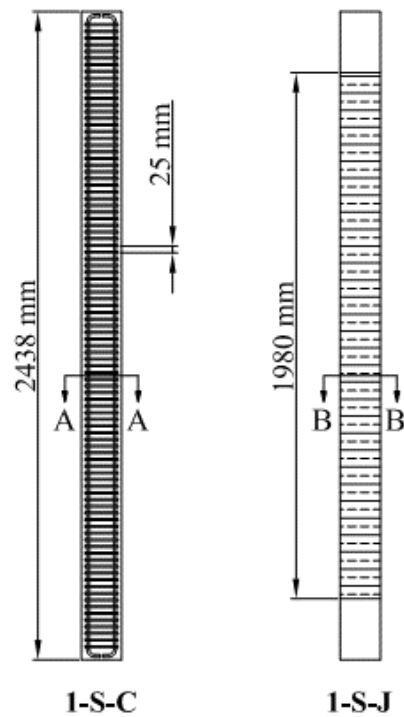


(a) Profiles showing Reinforcement and FRP Retrofit Details

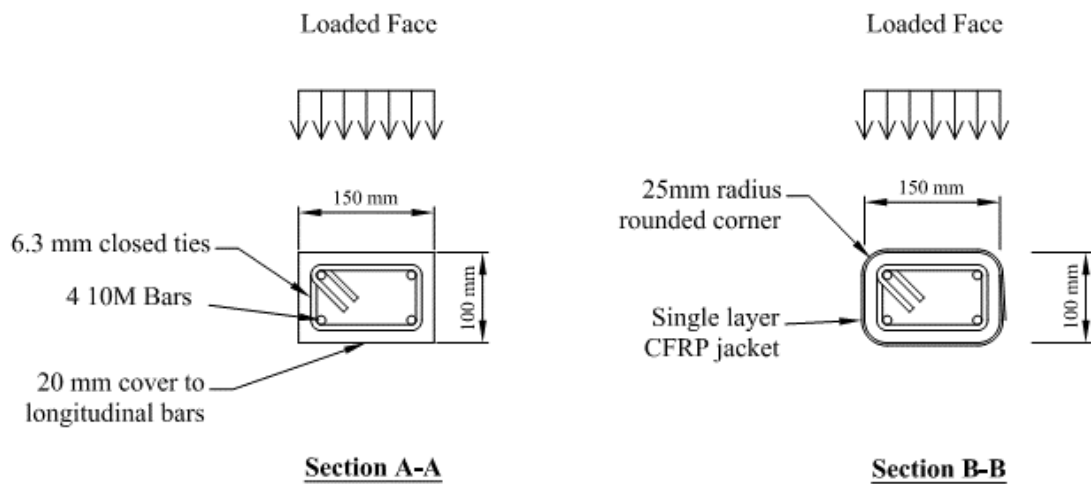


(b) Cross-Sections

Figure 3-1: Details of Group 1 Non-Seismic Columns

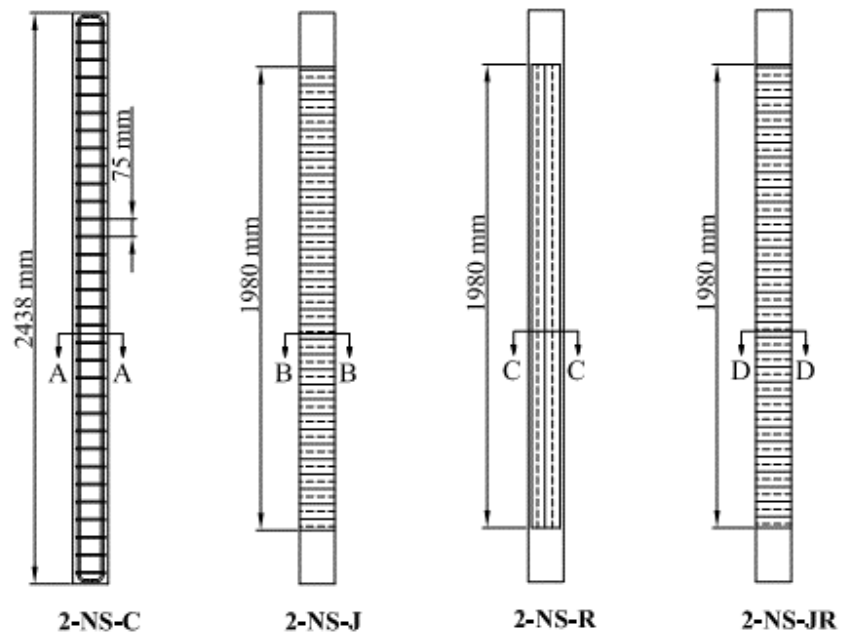


(a) Profiles showing Reinforcement and FRP Retrofit Details

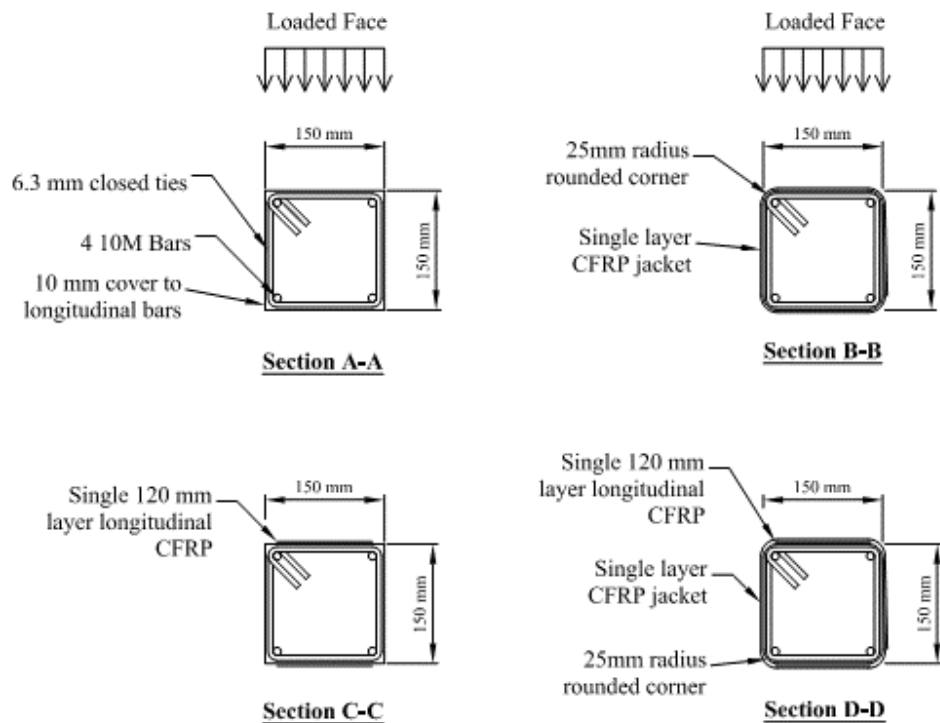


(b) Cross-Sections

Figure 3-2: Details of Group 1 Seismic Columns

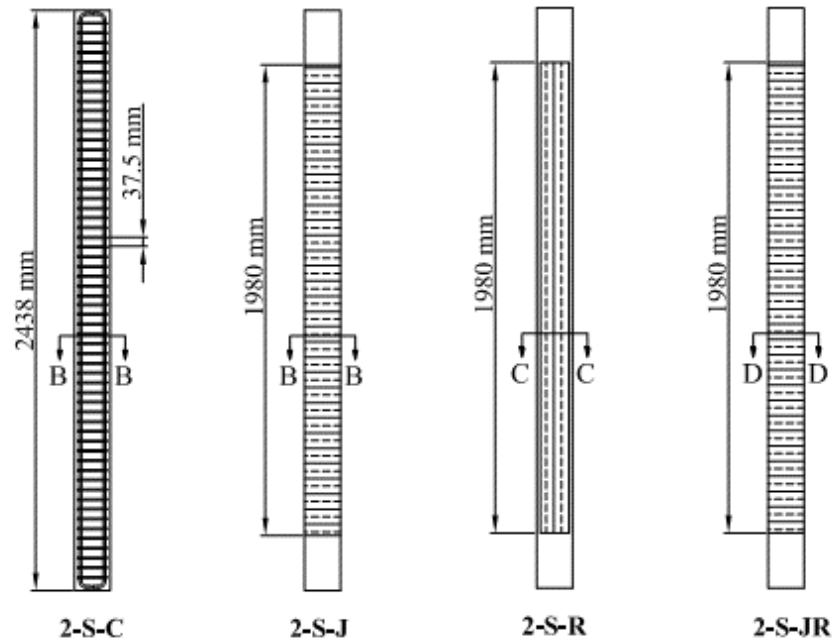


(a) Profiles showing Reinforcement and FRP Retrofit Details

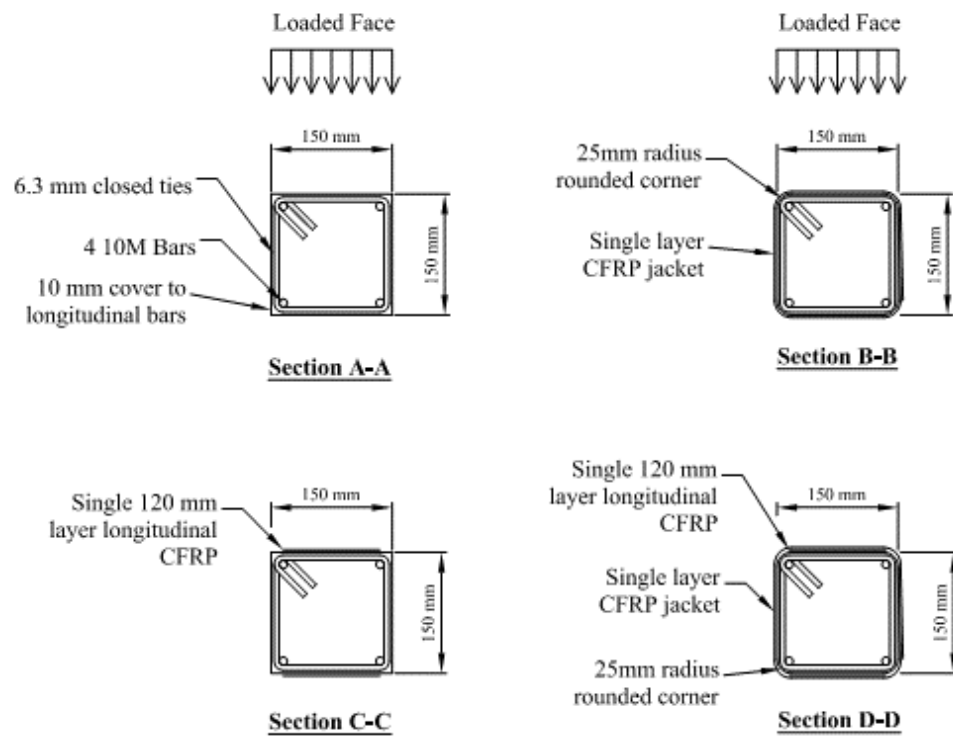


(b) Cross-Sections

Figure 3-3: Details of Group 2 Non-Seismic Columns

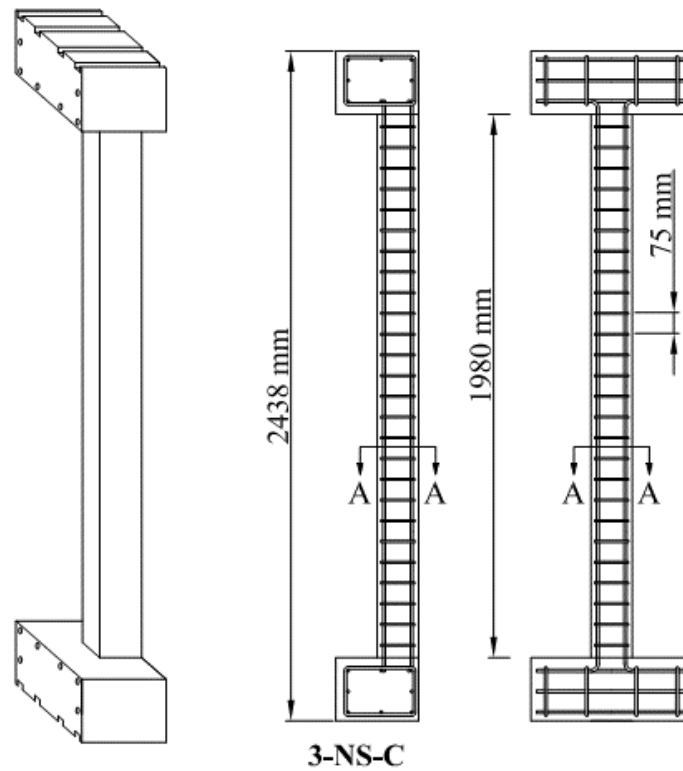


(a) Profiles showing Reinforcement and FRP Retrofit Details

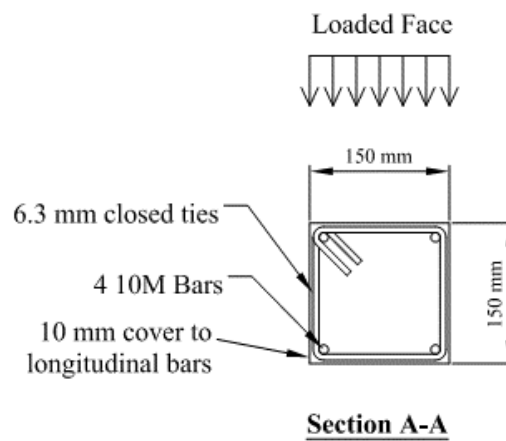


(b) Cross-Sections

Figure 3-4: Details of Group 2 Seismic Columns

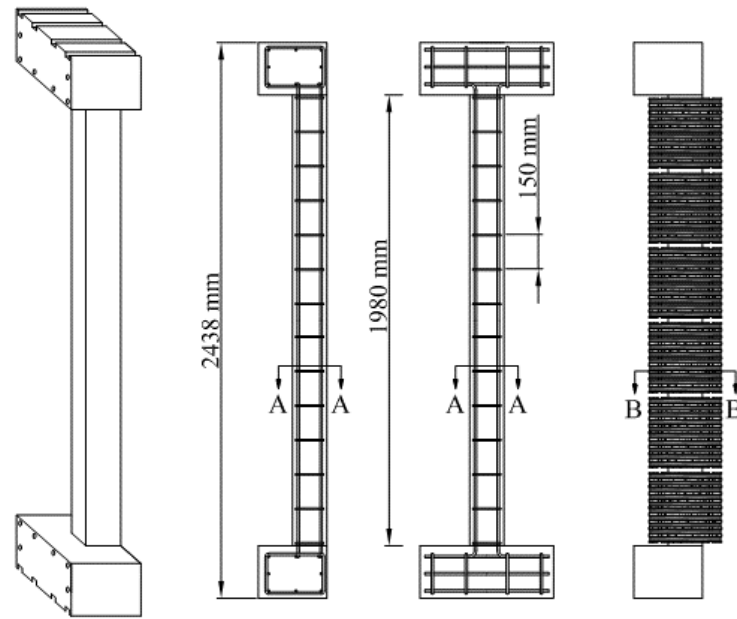


(a) Isometric View and Profiles showing Reinforcement Details



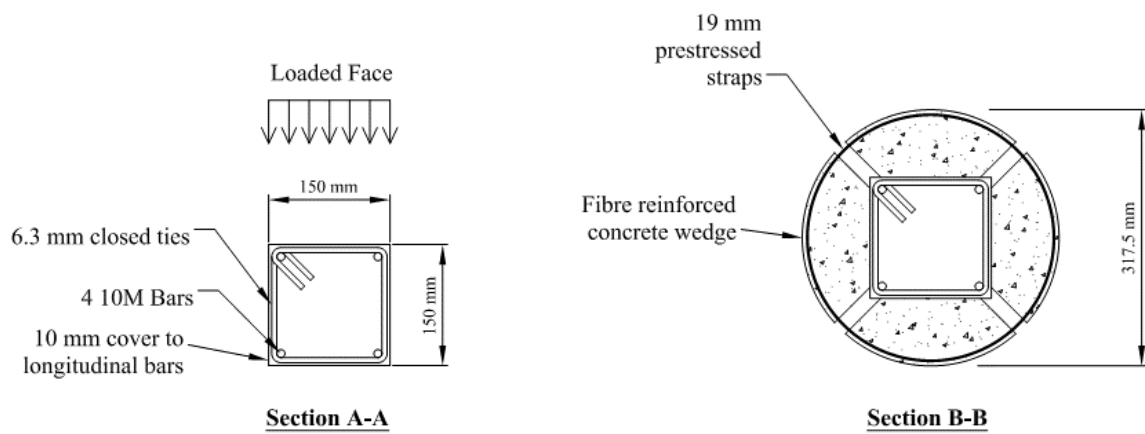
(b) Cross-Section

Figure 3-5: Details of Column 3-NS-C



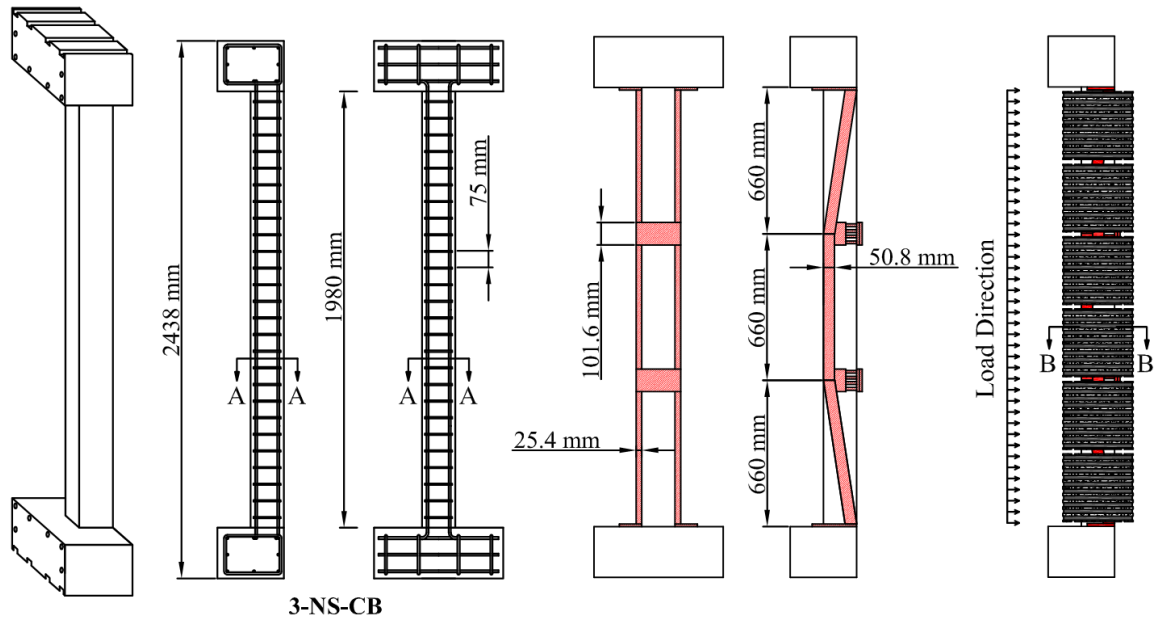
3-NS-W

(a) Isometric View and Profiles showing Reinforcement and Concrete Wedges Details



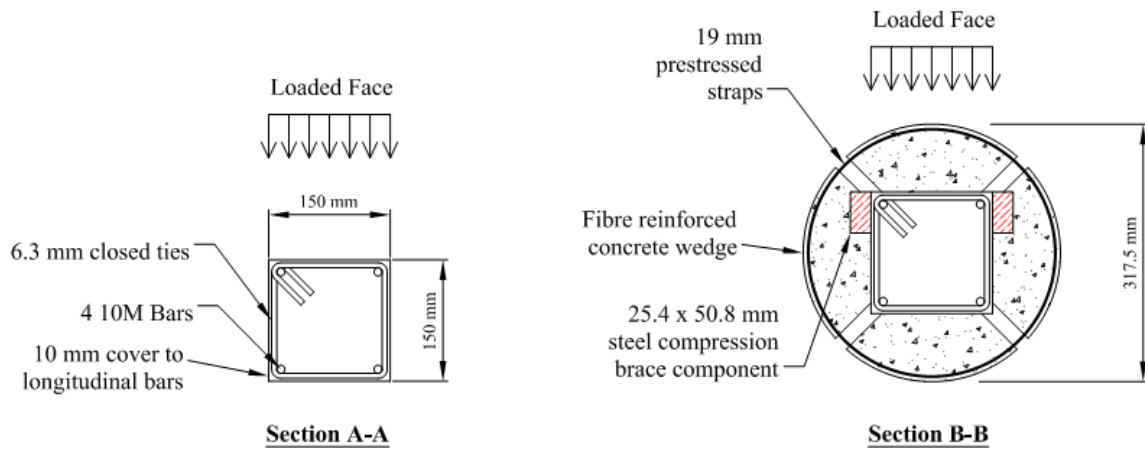
(b) Cross-Section

Figure 3-6: Details of Column 3-NS-W



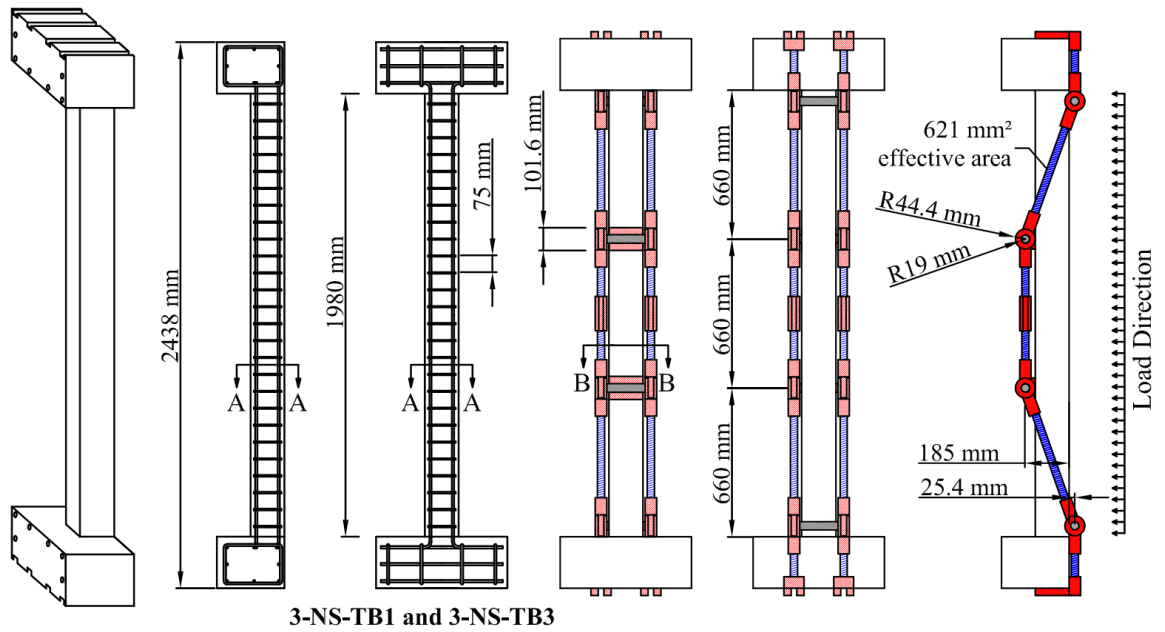
(a) Isometric View and Profiles showing Reinforcement, Compression Brace and Concrete

Wedges Details

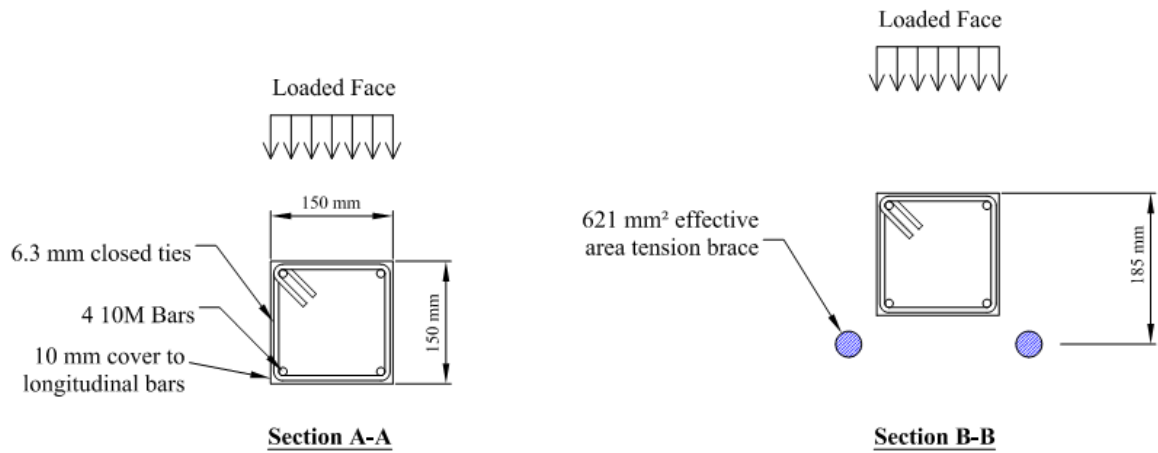


(b) Cross-Section

Figure 3-7: Details of Column 3-NS-CB

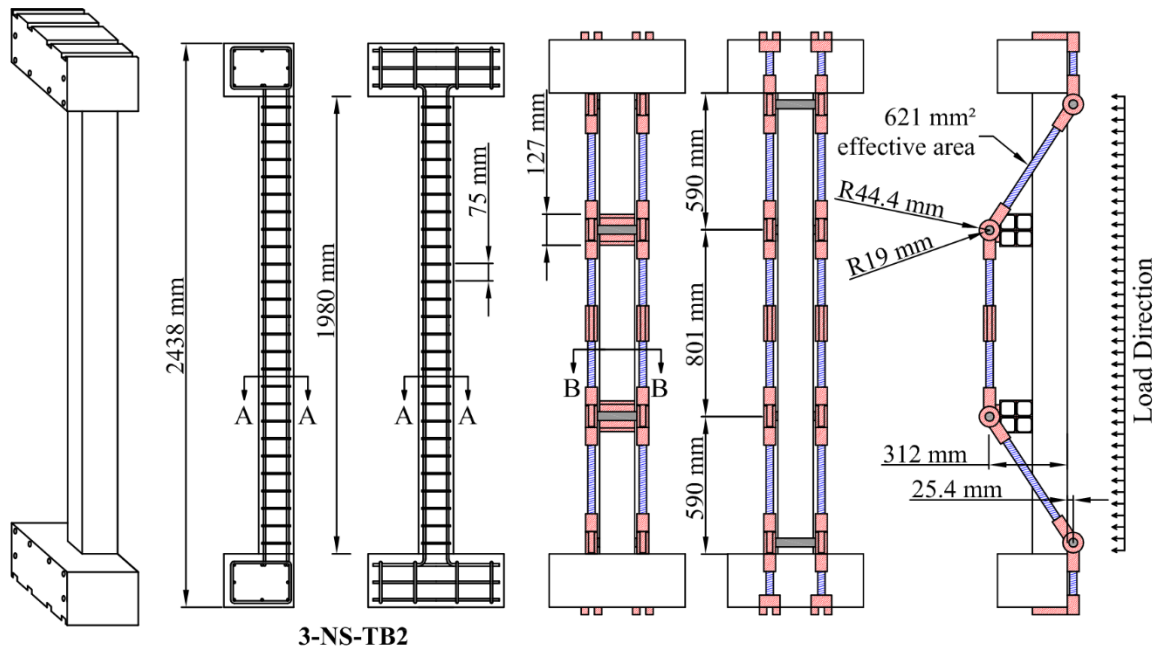


(a) Isometric View and Profiles showing Reinforcement and Tension Brace Details

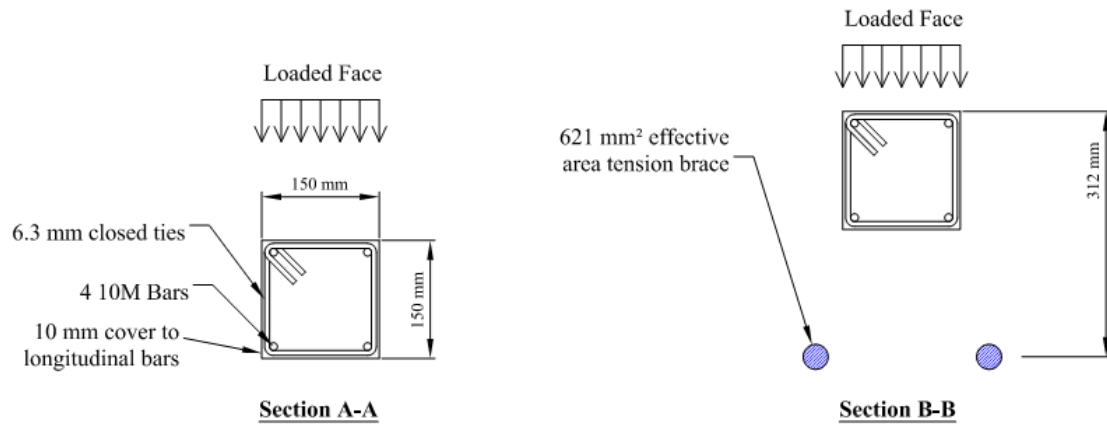


(b) Cross-Section

Figure 3-8: Details of Columns 3-NS-TB1 and 3NS-TB3



(a) Isometric View and Profiles showing Reinforcement and Tension Brace Details



(b) Cross-Section

Figure 3-9: Details of Column 3-NS-TB2



Figure 3-10: Driver Section, Diaphragms, and Expansion Section of Shock Tube



Figure 3-11: Shock Tube Opening, Test Frame and Expansion system

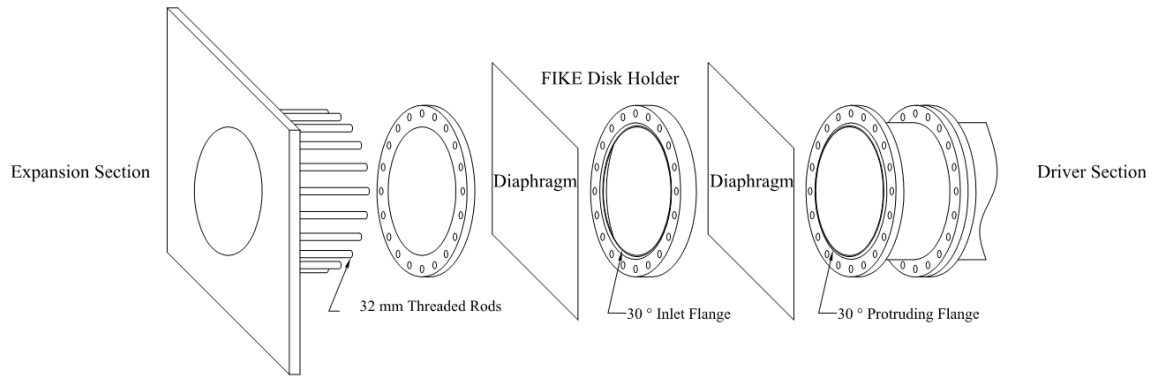


Figure 3-12: Diaphragm Section and Aluminum Diaphragms

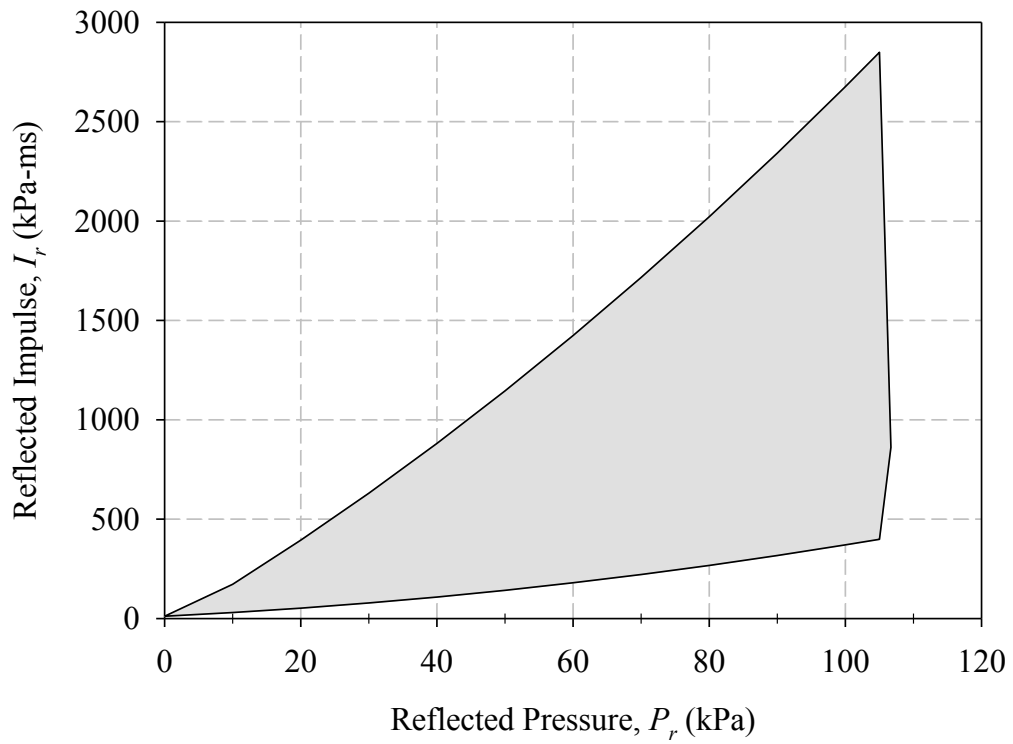


Figure 3-13: Range of Shock Tube Capacities

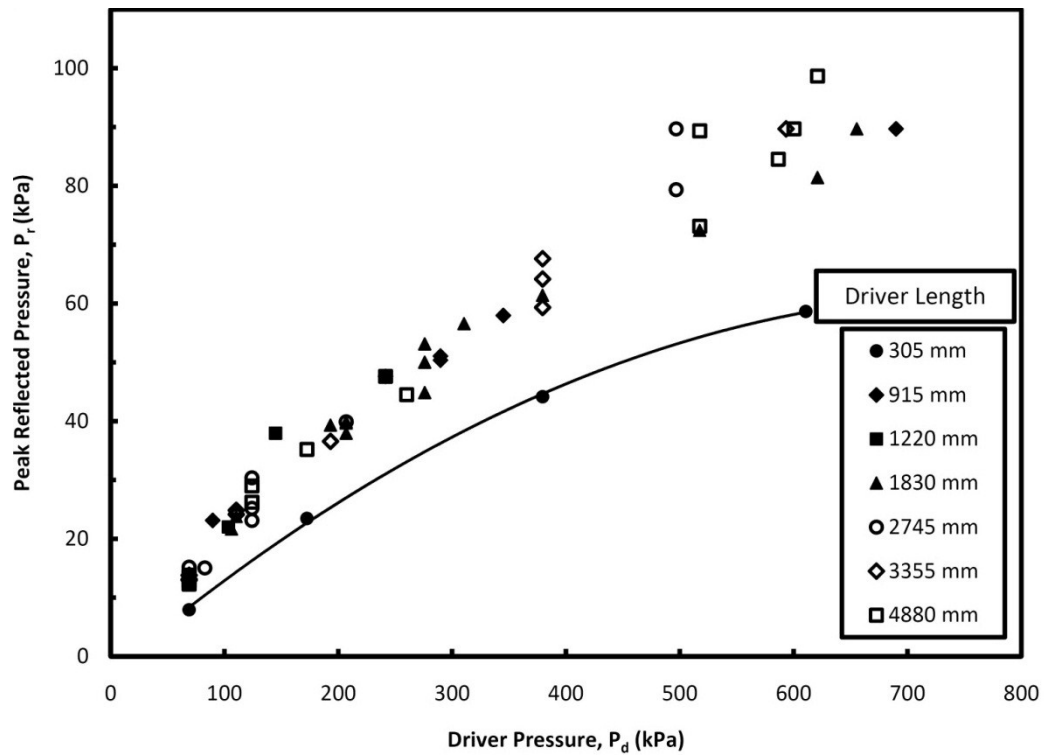


Figure 3-14: Relationship between Driver Pressure and Reflected Pressure

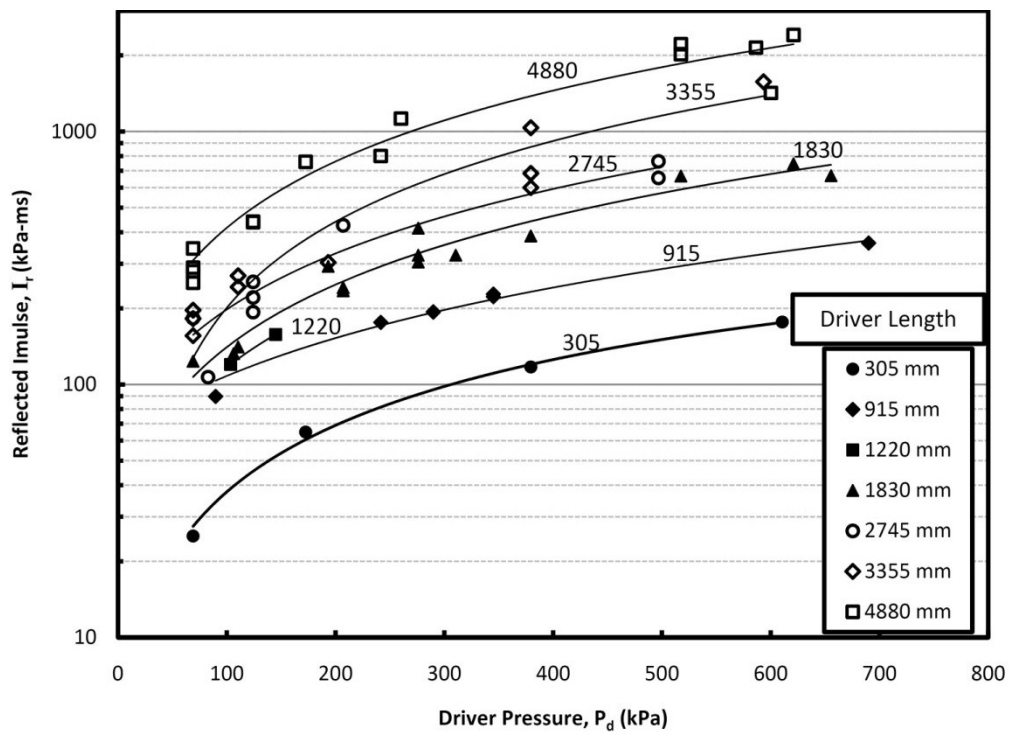


Figure 3-15: Relationship between Driver Pressure and Reflected Impulse

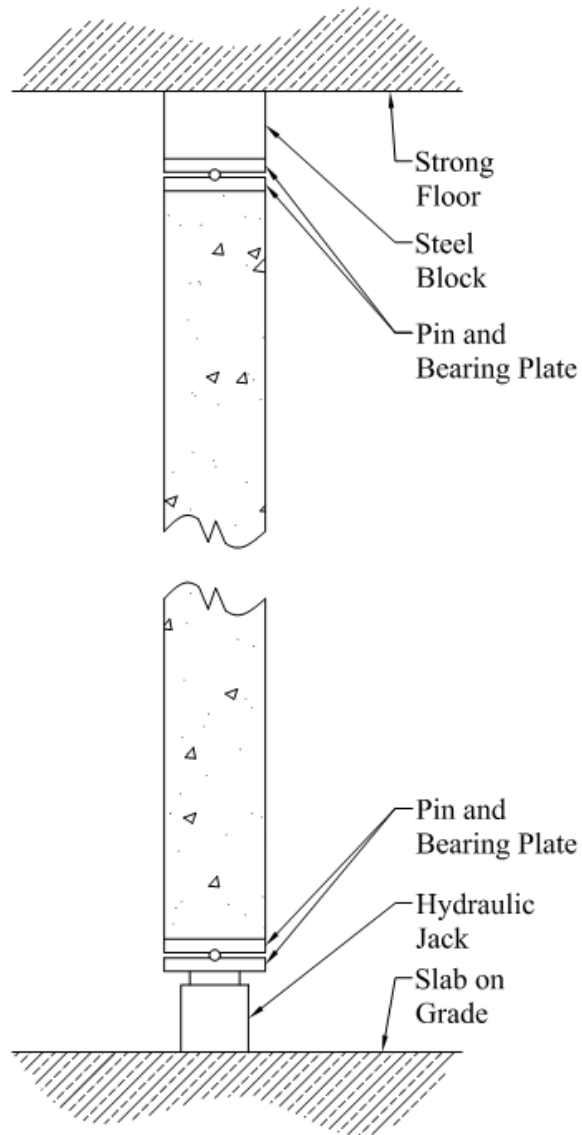
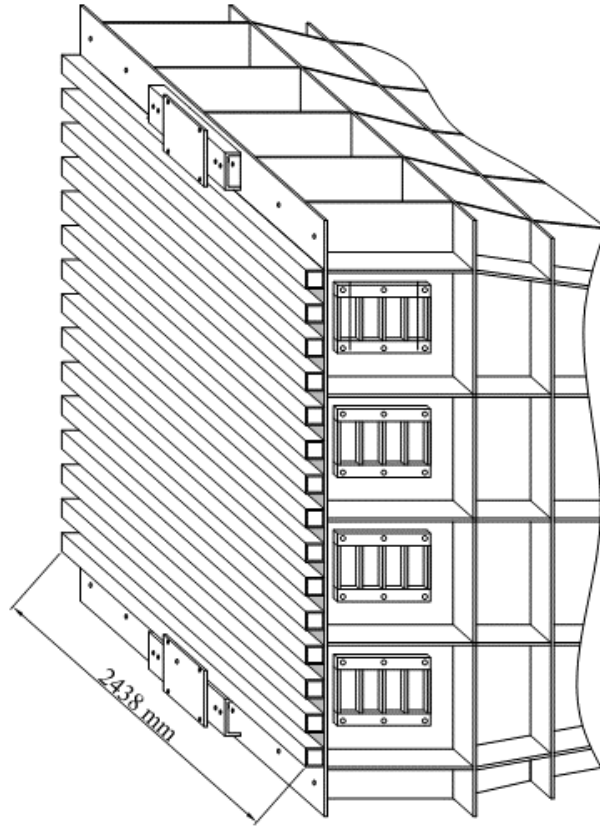
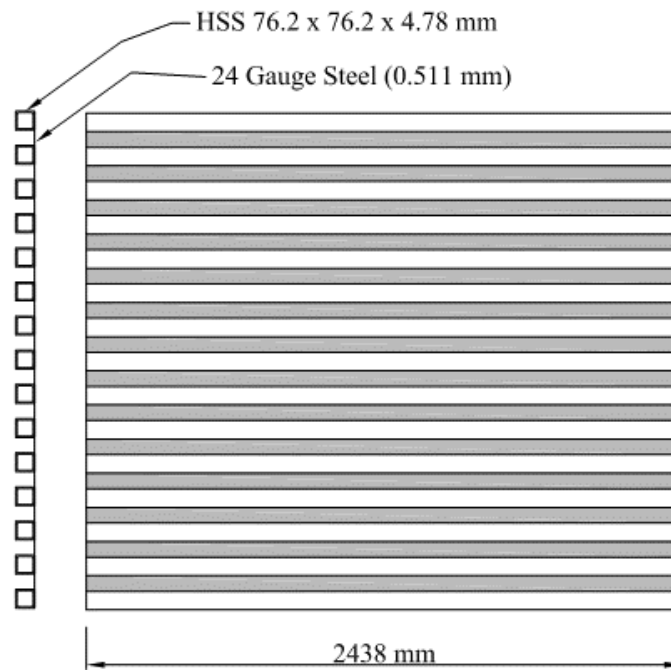


Figure 3-16: Axial Loading Mechanism

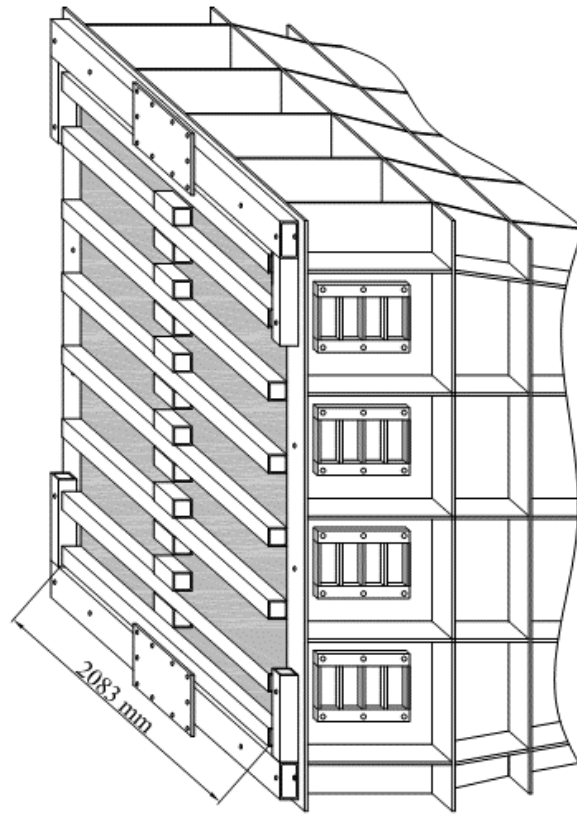


(a) Isometric View

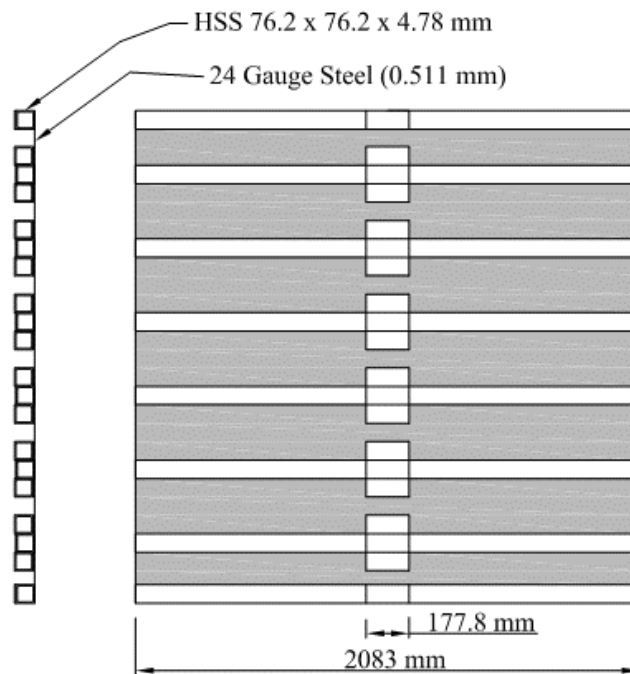


(b) Cross-Section and Elevation

Figure 3-17: Load Transfer Device for Group 1 Columns – LTD A



(a) Isometric View

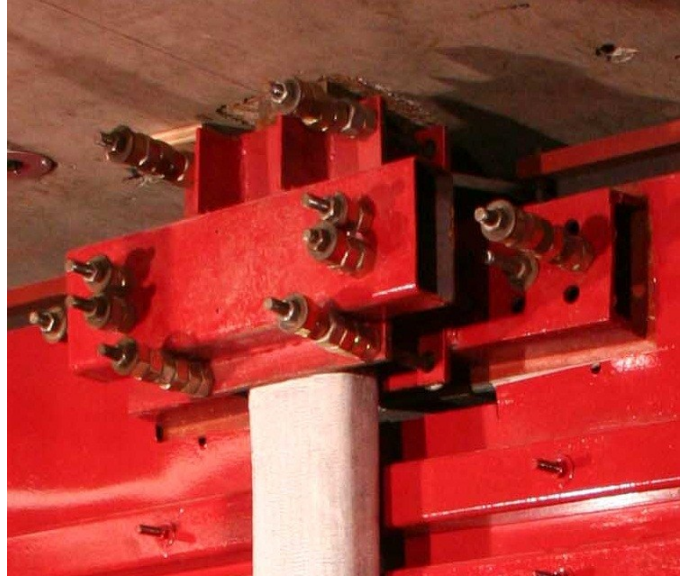


(b) Cross-Section and Elevation

Figure 3-18: Load Transfer Device for Group 2 and Group 3 Columns – LTD B



(a) Side View



(b) Front View

Figure 3-19: Column End Support for Group 1 Columns – Support A

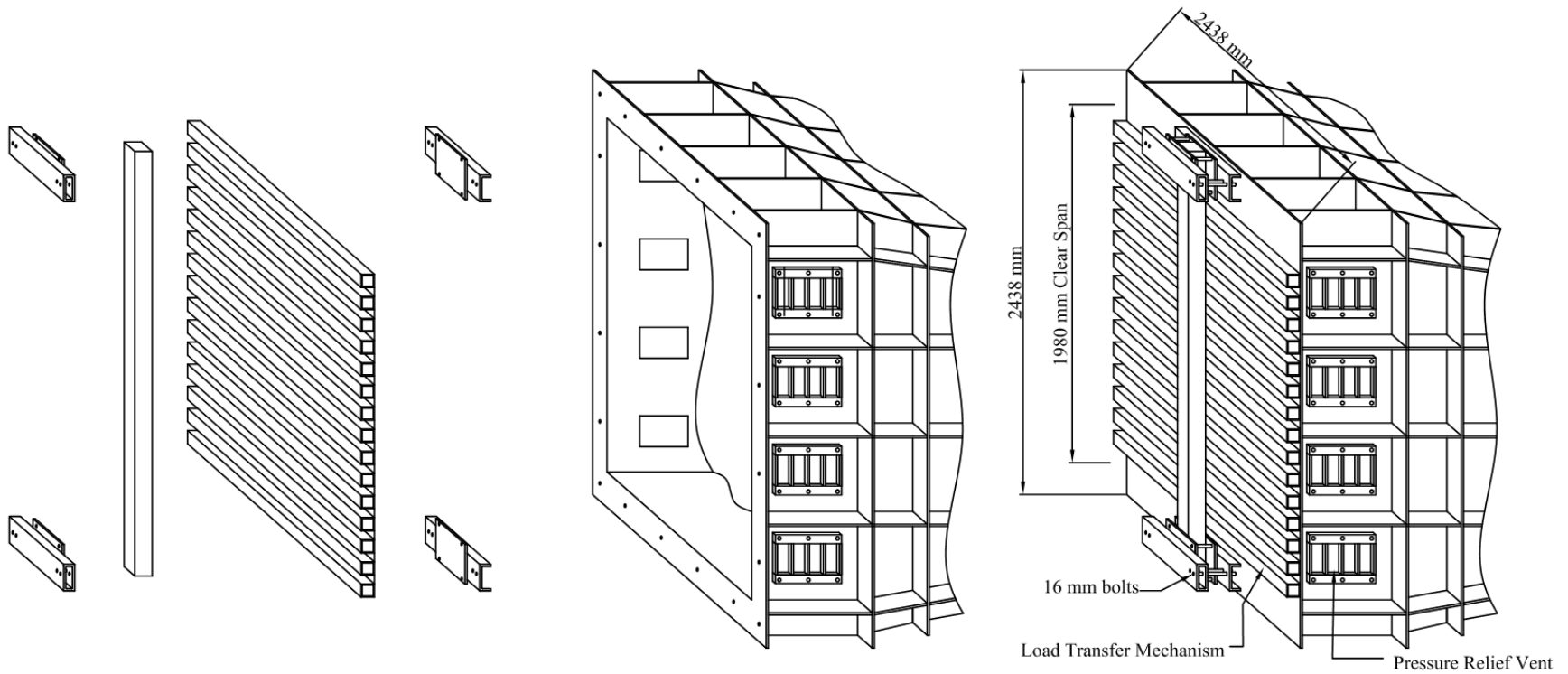


Figure 3-20: Group 1 Column Test Setup showing Supports A, Column, Load Transfer Device A, Shock Tube and Installed Column and Assembled Test Apparatus



(a) Side View



(b) Elevation

Figure 3-21: Column End Supports for Group 2 and Group 3 Columns – Support B

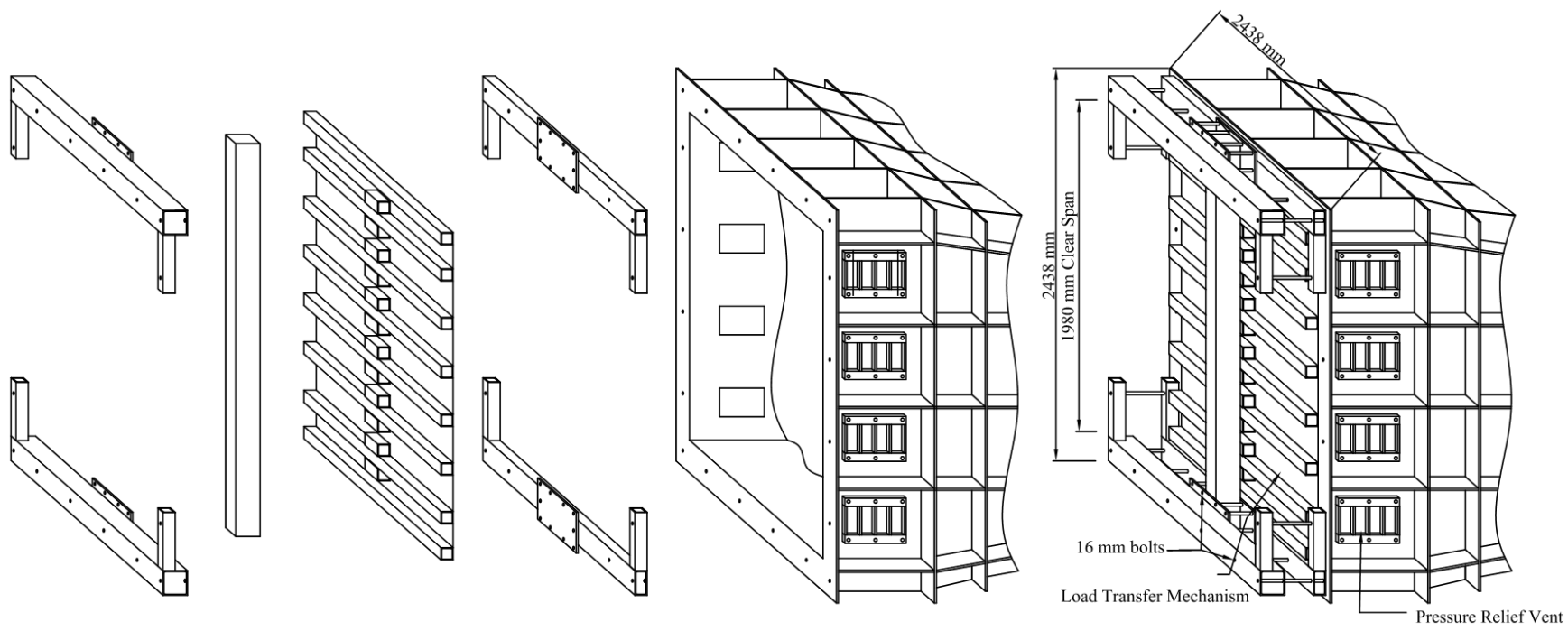


Figure 3-22: Group 2 Column Test Setup showing Supports B, Column, Load Transfer Device B, Shock Tube and Installed Column and Assembled Test Apparatus

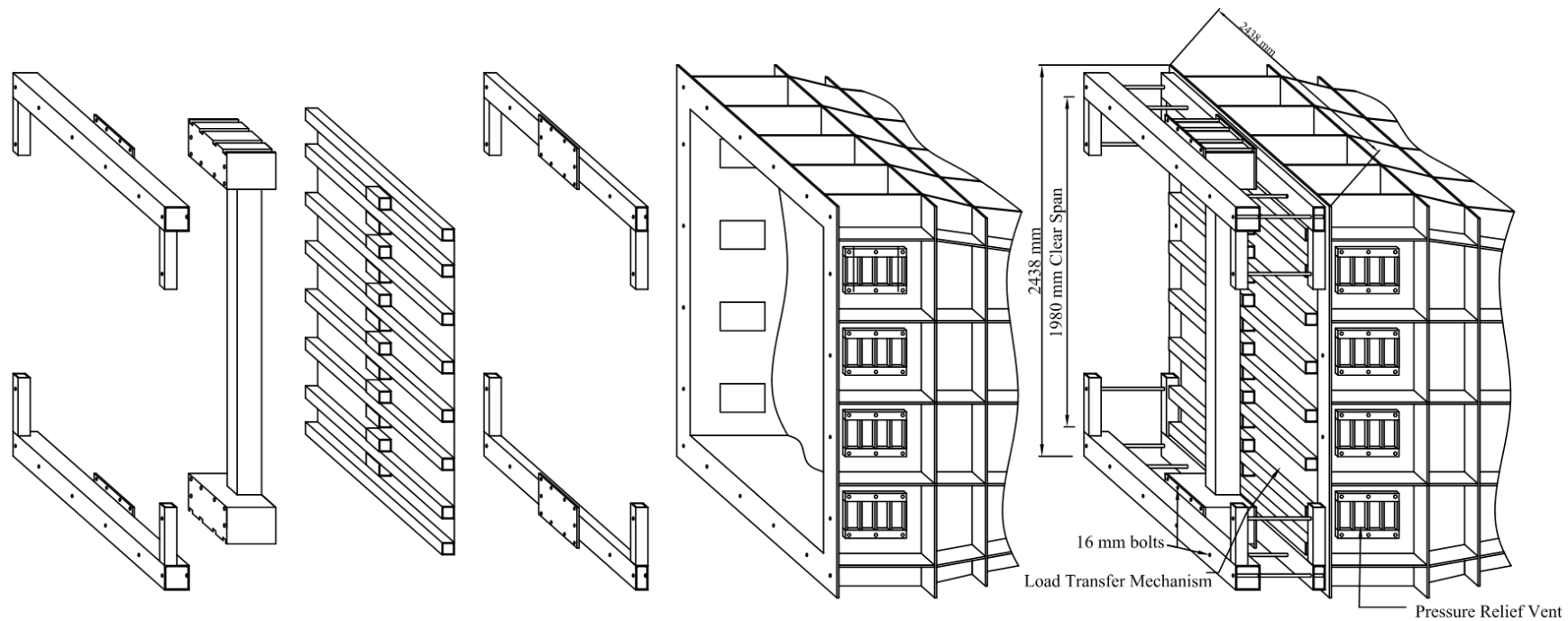


Figure 3-23: Group 3 Column Test Setup showing Supports B, Column, Load Transfer Device B, Shock Tube and Installed Column and Assembled Test Apparatus

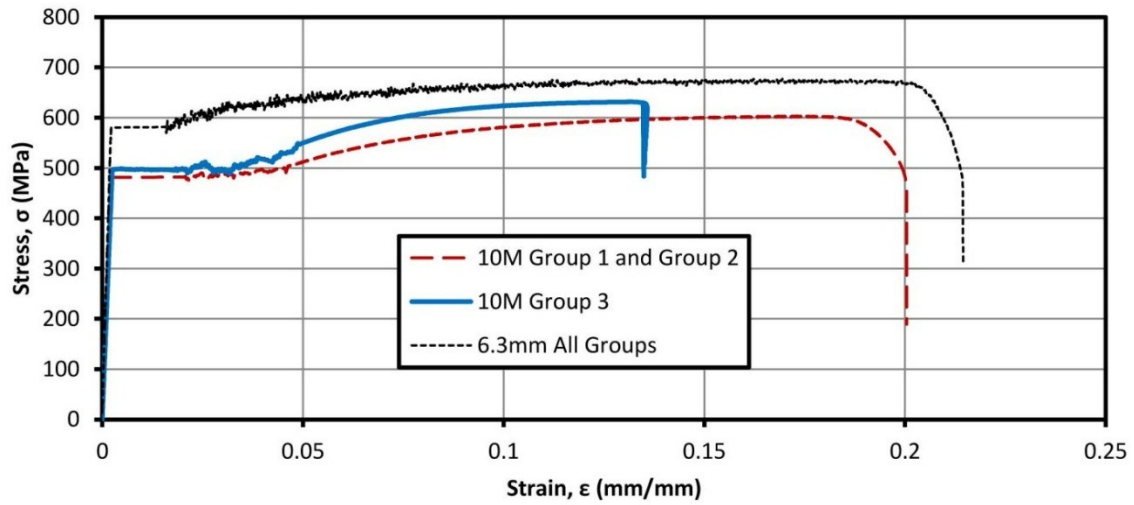


Figure 3-24: Reinforcing Steel Material Properties

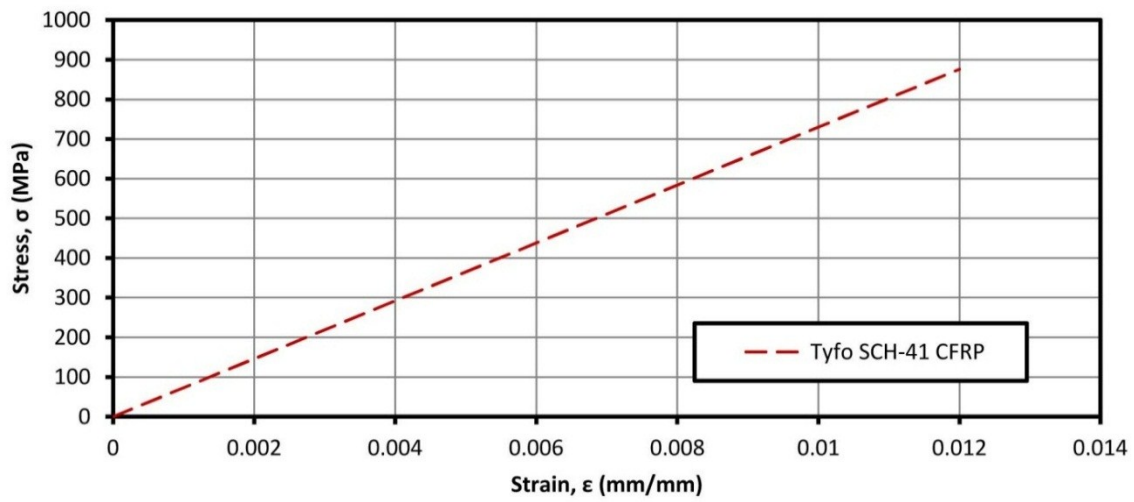


Figure 3-25: CFRP Material Properties

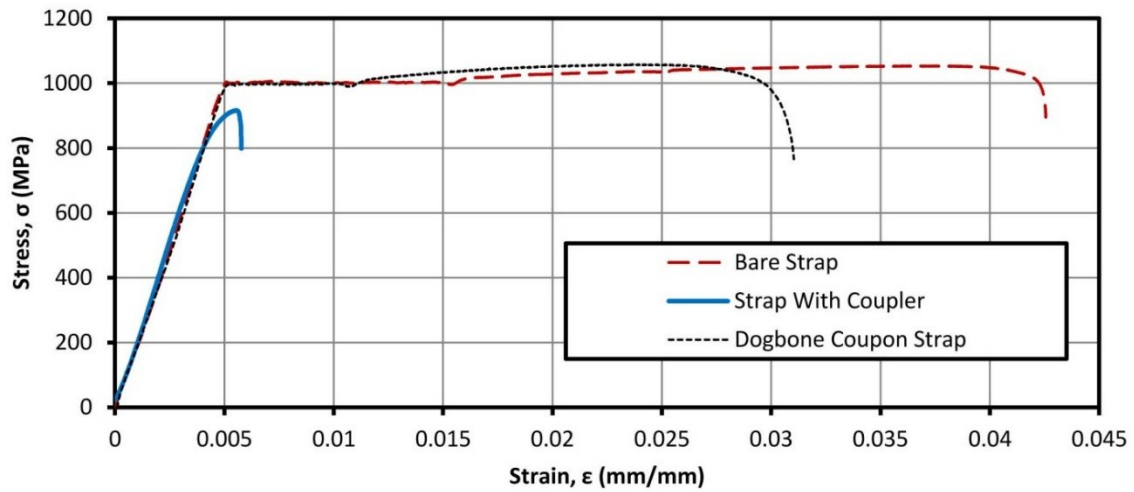


Figure 3-26: Prestress Strap Material Properties

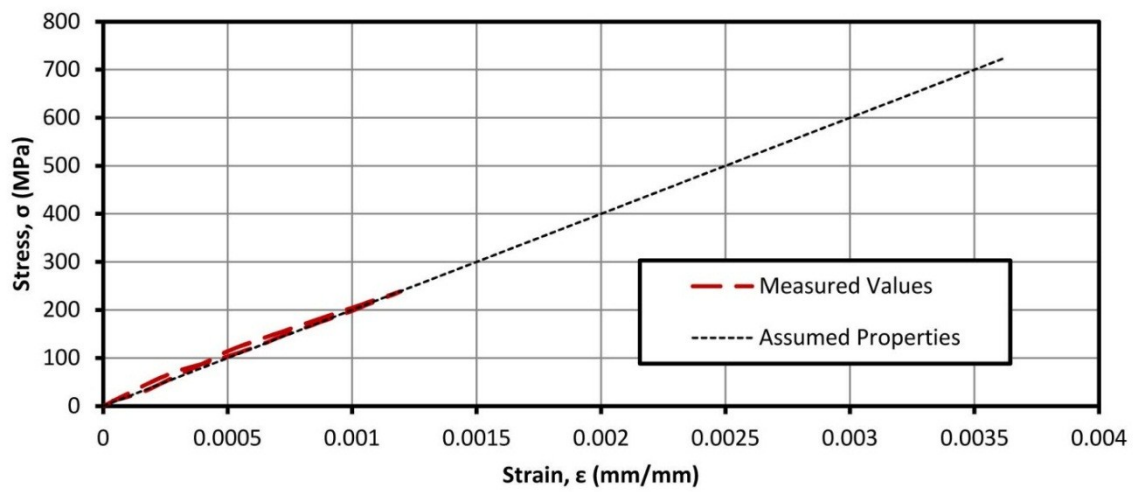


Figure 3-27: Tension Brace Threaded Rod Material Properties



a) Casted in Forms



b) Removed from Forms



c) Prior to Installation

Figure 3-28: Construction of Fibre Reinforced Concrete Wedges



a) Straps Cut to Length



b) Straps with Couplers

Figure 3-29: Prestress Straps Prior to Installation



a) Wedges Placed for Installation



b) Installing Prestressed Strap



c) Installing Prestressed Strap



d) Completed Column

Figure 3-30: Installation of Fibre Reinforced Concrete Wedges and Prestressed Straps

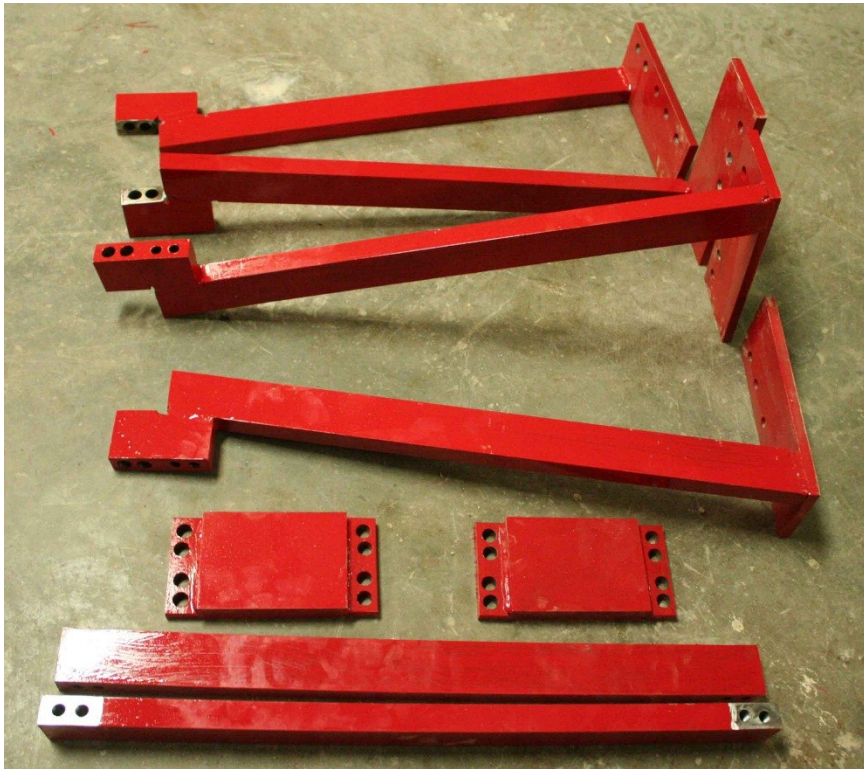
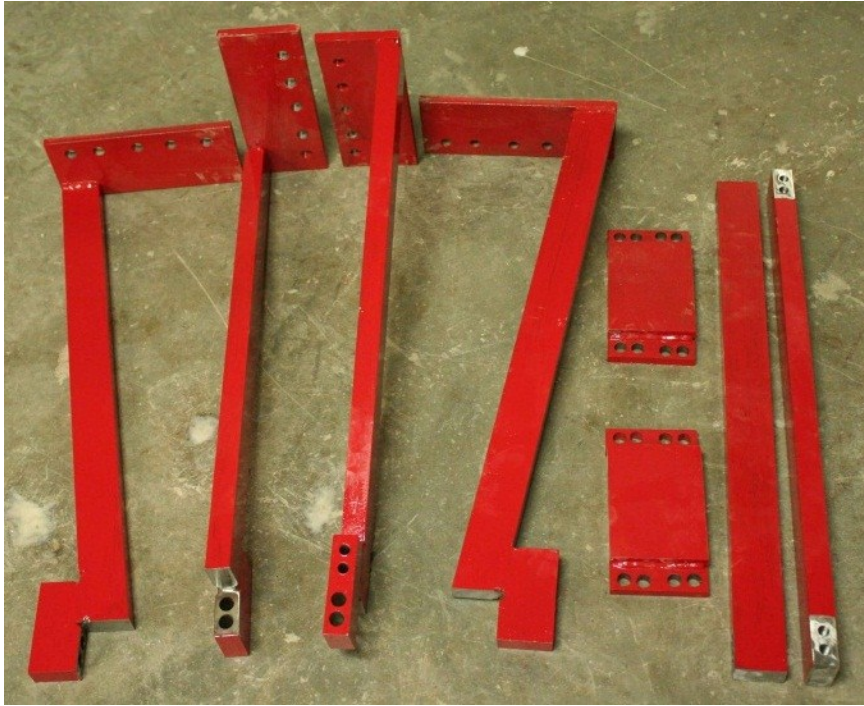
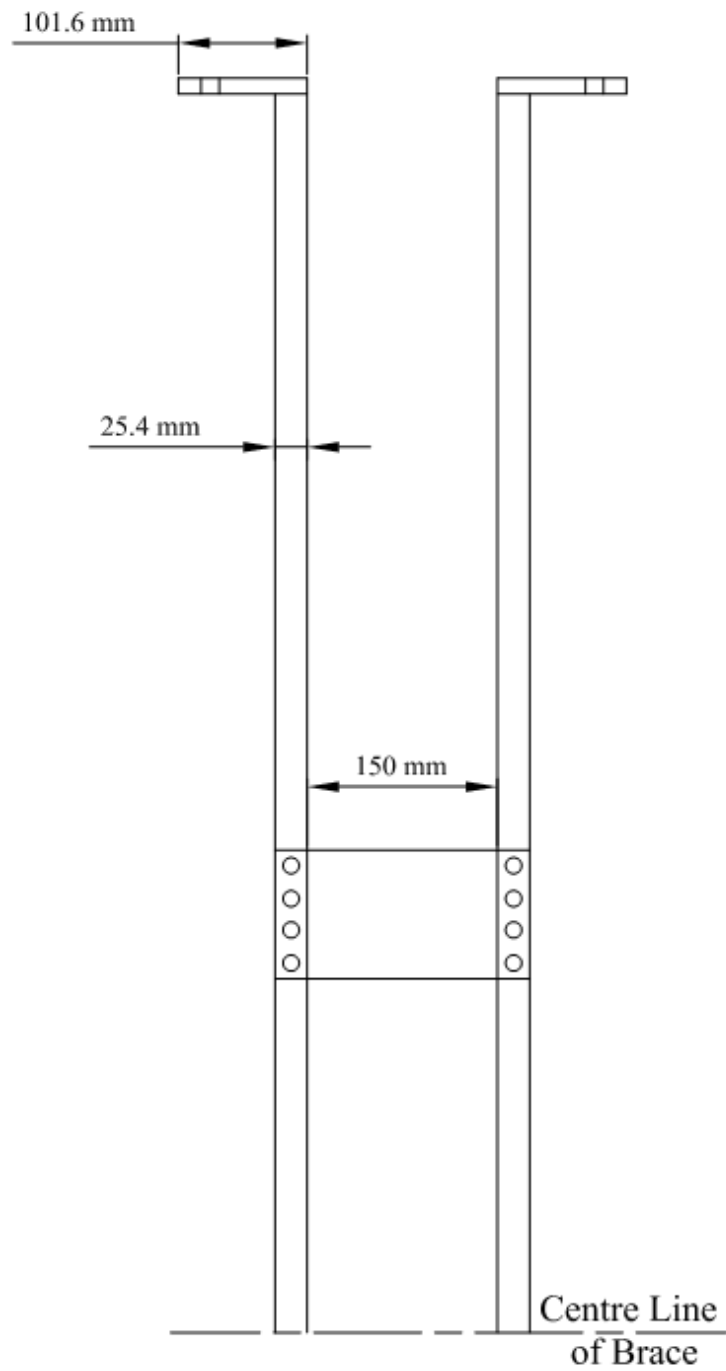
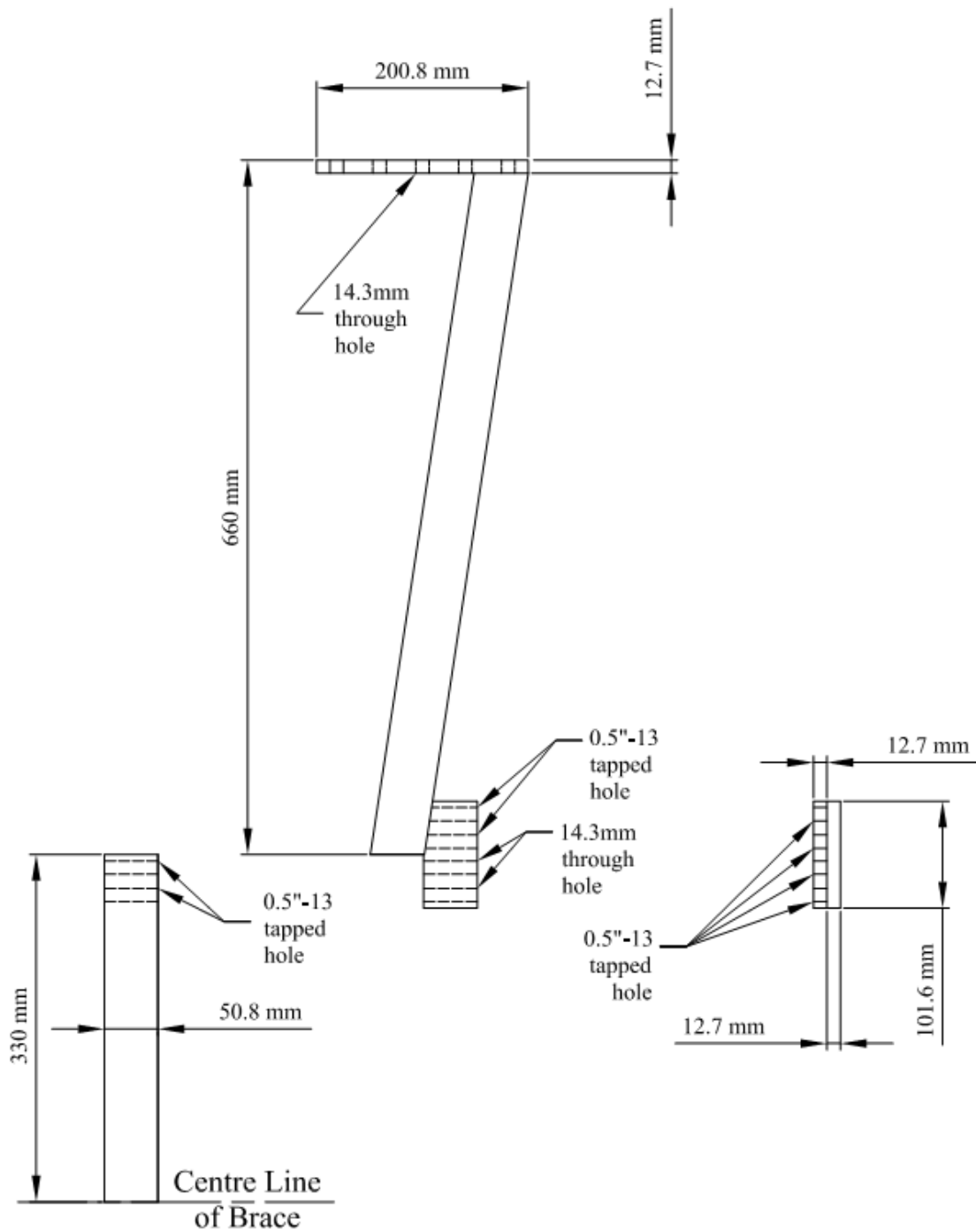


Figure 3-31: Two Views of Compression Brace Components showing Diagonal Components, Vertical Components and Bearing Plates



Front View

Figure 3-32: Compression Brace Components Front View



Side View

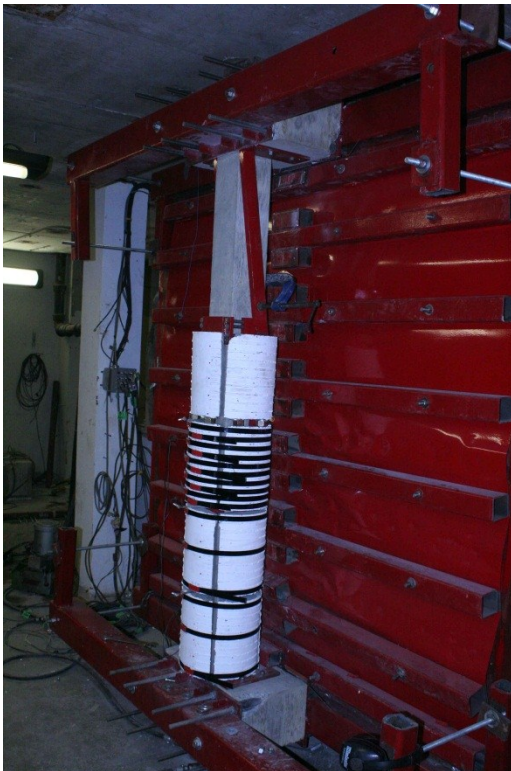
Figure 3-33: Compression Brace Components Side View



a) Diagonal Members



b) Vertical Members and Bearing Plates



c) Wedges and Prestressed Straps



d) Completed Column

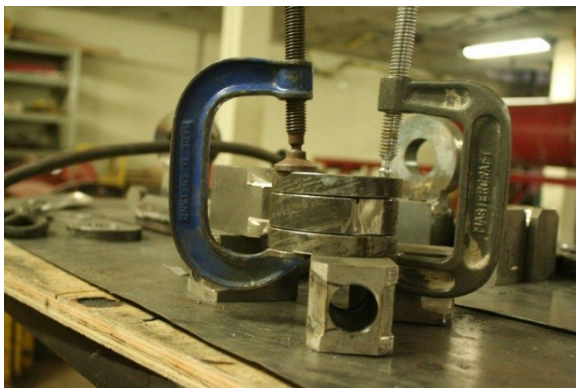
Figure 3-34: Installation of Compression Braces and Fibre Reinforced Concrete Wedges



a) Components Prior to Welding



b) Hinge Coupler Prior to Welding

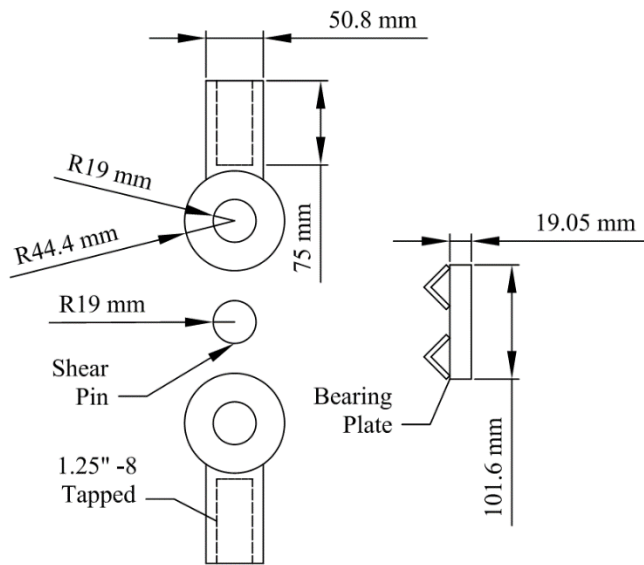


c) Hinge Coupler Prior to Welding

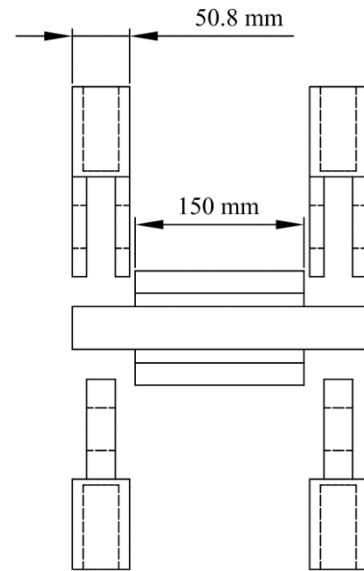


d) Completed Hinges and Shear Pin

Figure 3-35: Construction of Tension Brace Components

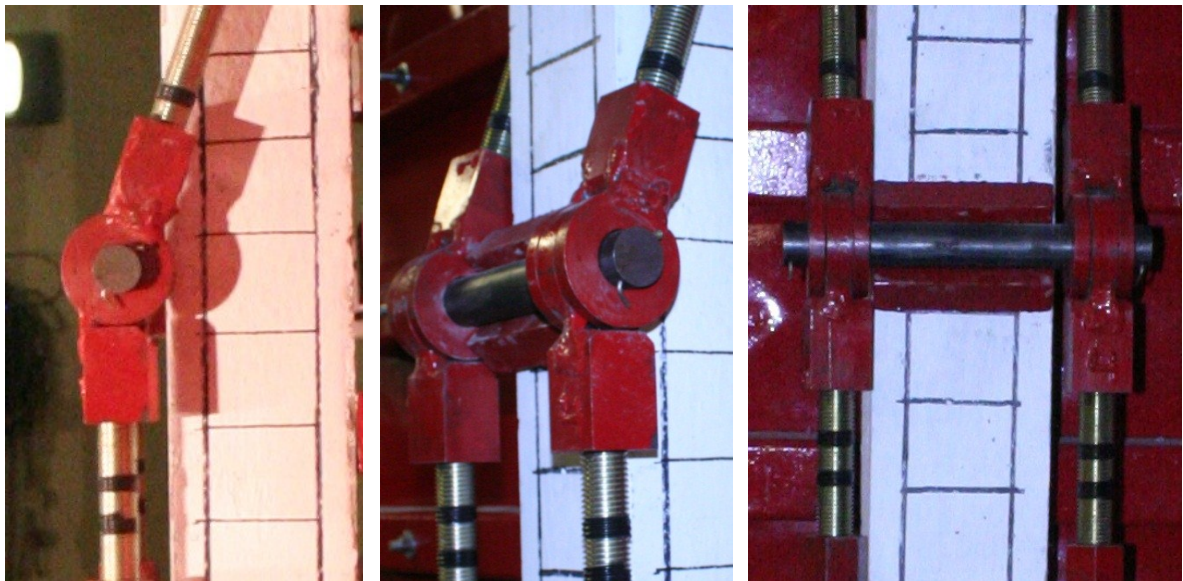


Side View



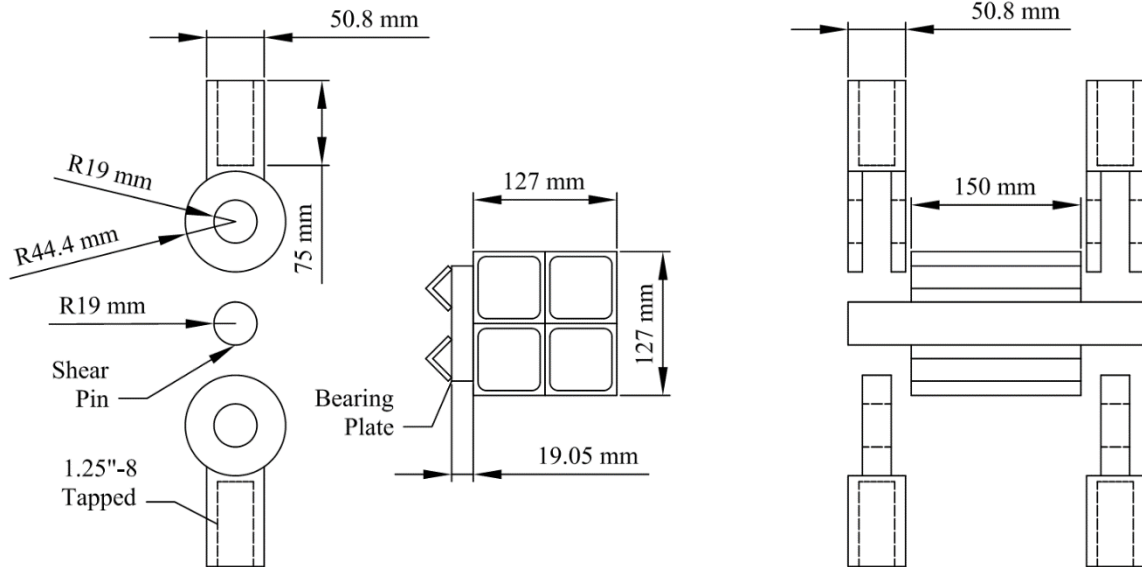
Front View

a) Side and Front of Tension Brace Interior Hinge Components



b) Interior Hinge Components and Bearing Plate on Column

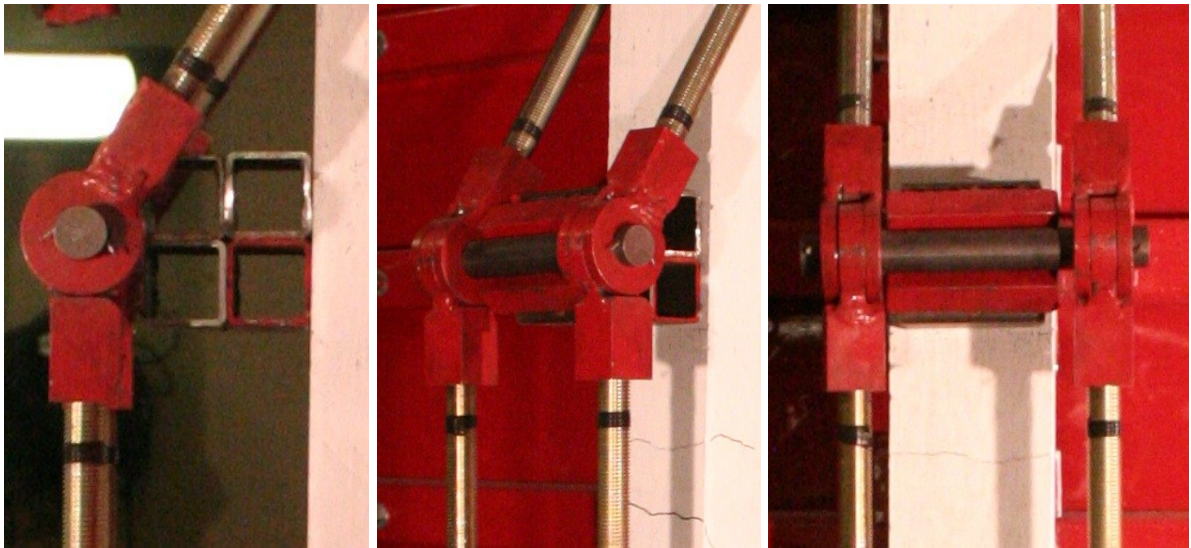
Figure 3-36: Interior Tension Brace Hinge Components and Bearing Plate for Columns 3-NS-TB1 and 3NSTB3



Side View

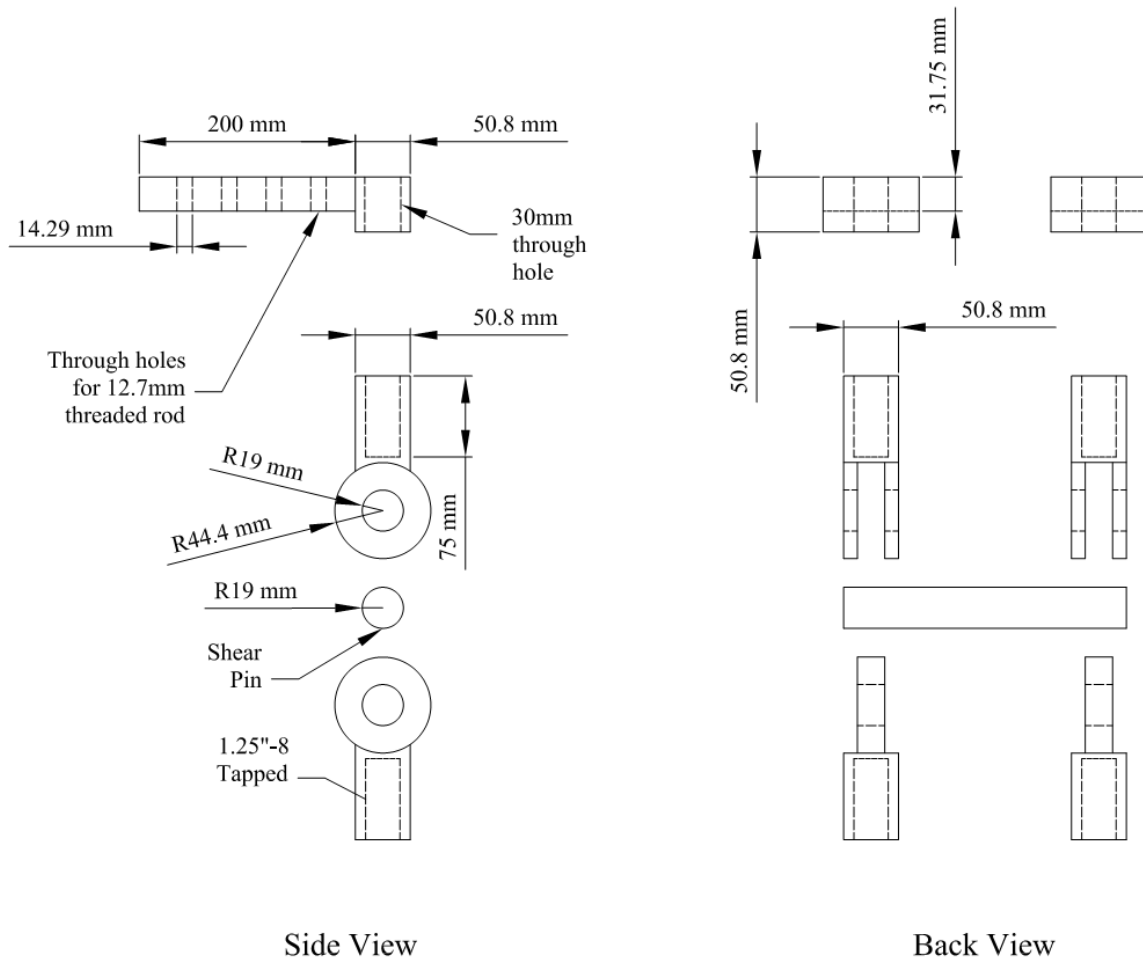
Front View

a) Side and Front of Tension Brace Interior Hinge Components



b) Interior Hinge Components and Bearing Plate on Column

Figure 3-37: Interior Tension Brace Hinge Components and Bearing Plate for Columns 3-NS-TB2

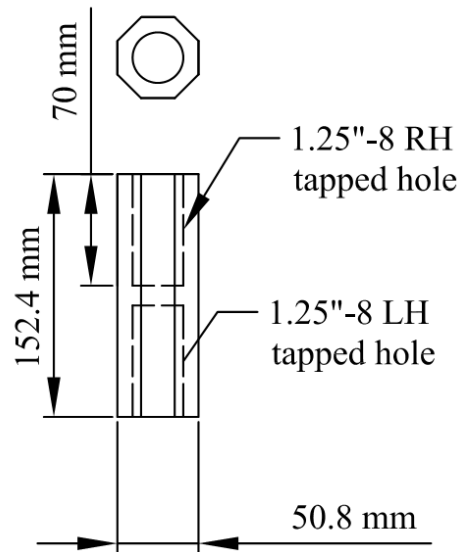


a) Side and Back views of Tension Brace End Hinge and Bearing Connector



b) End Hinge and Bearing Connector Installed on Column

Figure 3-38: End Tension Brace Hinge and Component and Bearing Connector

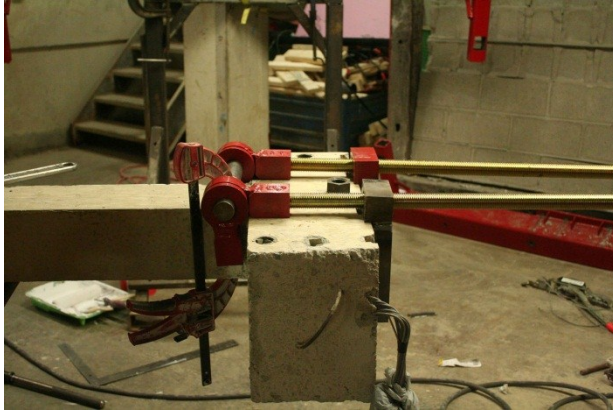


a) Tension Brace Turnbuckle



b) Tension Brace Turnbuckle Installed on Column

Figure 3-39: Tension Brace Turnbuckle



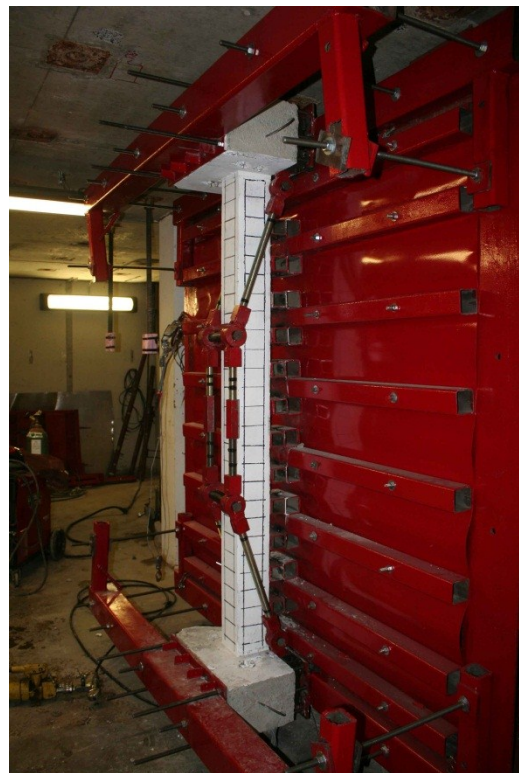
a) Installation of End Connections



b) Installation of Braces



c) Completed Brace Installation



d) Completed Column

Figure 3-40: Installation of Tension Braces

Chapter 4. Experimental Results and Discussion

4.1. General

The observed results of the experimental tests on 18 reinforced concrete columns are presented in this chapter. Each of the columns was tested with a series of shock waves increasing in magnitude until significant damage was observed in the column or the capacity of the shock tube was reached. The measured displacement at mid-height of the column as well as heights $1/3$ and $1/6$ of the height for selected columns is presented. Strain in longitudinal reinforcing steel at various locations as well as strain in FRP and braces where applicable is reported. Plots of the applied reflected pressure and impulse time histories are shown for each test along with displacement time histories are shown. In this section, each column is designated according to the naming defined in the previous chapter along with the test number augmenting the column designation. For example, 2NS-C-2 designates test 2 on column 2NSC.

Plots of the displacement, pressure and impulse time histories are shown in Appendix A. Plots of strain time histories for each test may be found in Appendix B.

4.1.1. Load Transferring Device

All of the given pressure and impulse values are those that were recorded acting on the load transferring device (LTD). The LTD was employed to transfer all of the pressure over the face of the shock tube opening onto the column as a uniformly distributed load over the length of the column. This has the effect of amplifying the applied load to the column. It is important to note that a column without a similar LTD to increase tributary area would not respond as significantly to the same applied pressure. The LTD was modified between some of the test specimens resulting in a change in mass. Columns within group 1 used LTD-A and columns in group 2 and group 3 used LTD-B. The two load transferring devices are described in detail in Section 3.4.4.

4.1.2. Supports

The lateral supports were intended to provide some level of fixed rotational restraints to the ends of the column. There were two different support systems employed in the test program. Group 1 columns used the Support-A system and group 2 and group 3 columns used the Support-B system. Details of the two support system configurations are discussed in the Section 3.4.5. Section 4.6 describes the measured rotational stiffness of the two support systems.

4.1.3. Axial Load

Some level of axial load was applied to all columns prior to testing. The axial application mechanism is discussed in detail in the Section 3.4.3. Section 4.4 discusses the variation in axial load during the dynamic response of the columns.

4.2. Experimental Results

A summary of the testing regimen for each column may be found in Table 4-1. This table includes the driver length (L_d) and driver pressure (P_d) used in each test along with the recorded maximum reflected pressure (P_r), total reflected impulse over the positive phase duration (I_r), positive phase duration (t_d), maximum displacements at gauge locations (d_{max}), the time to reach maximum displacements (t_{max}), the relative residual displacement (d_{res}) after each shot at gauge locations, and the maximum support rotation (θ_{max}). In Table 4-1 and in all figures showing displacement-time histories, the displacement values are presented as relative to the specific shot. This means that all previous residual displacements are not added to the presented results. Table 4-2 through Table 4-19 gives a summary of the strain measured at various locations on each of the columns for each shot. The initial strain in the column due to the applied axial load, ϵ_{axial} in Equation [4-1], is calculated using the suggested value for the elastic modulus of concrete (E_c) given Canadian Standard Association (CSA) Standard A23.3 for the “Design of Concrete Structures” (2004) shown in Equation [4-2]. In Equation 4-3, P is the axial load, and A_T is the transformed area of the column given in Equation [4-3]. In Equation [4-3], A_g is the gross column cross-section area, n is the ratio of the elastic modulus for steel, E_s , and the elastic modulus for concrete, E_c , given in Equation [4-4], and A_s is the area of the steel rebar in the column.

$$\epsilon_{axial} = \frac{P}{A_T E_c} \quad [4-1]$$

$$E_c = 4500\sqrt{f'_c} \text{ (MPa)} \quad [4-2]$$

$$A_T = A_g + (n - 1)A_s \quad [4-3]$$

$$n = \frac{E_s}{E_c} \quad [4-4]$$

The strain values presented in Table 4-2 through Table 4-19 are relative strains for each shot. The relative maximum strain, ϵ_{max} , and relative residual strain, ϵ_{res} , are found from the experimental time histories by selecting initial strain as zero. The true strain for any shot is taken as the sum of the

residual strains prior to the shot plus the initial axial strain (when applicable) and the relative strain value.

4.2.1. Column 1NS-C

Column 1NS-C was previously tested for a separate project (Lloyd, 2010) and is included here as it is a control column for the retrofitted columns in group 1. The column had a cross section measuring 100mm by 150mm and was bent about the weak axis. Concrete strength was measured as 58MPa at the time of testing. Closed steel hoops were spaced at 50mm along the length of the column and there were four 10M longitudinal reinforcing steel bars. A 350kN, or 36% of concentric capacity, axial load was applied to the column prior to each shock wave test. The axial load induced a compressive strain of -0.0603% as calculated using Equations [4-1] through [4-4] as reported in Table 4-2. Column 1NS-C is the control to Column 1NS-J and the non-seismic companion to Column 1S-C. The column was subjected to a total of 3 shock tube tests: 1NS-C-1, 1NS-C-2, and 1NS-C-3. The column mounted on the shock tube is shown in Figure 4-1a. Column 1NS-C used the end support type A with a clear span of 1980mm between supports and LTD type A as described in the previous chapter.

4.2.1.1. 1NS-C-1

Test 1NS-C-1 used a driver length of 3353mm and a driver pressure of 69kPa resulting in a reflected pressure of 13.7kPa and a reflected impulse of 179.8kPa-ms over a positive phase of 21.7ms as reported in Table 4-1 and shown in Figure A-1b. The maximum mid-height displacement was 14.6mm occurring 29.6ms after the start of the positive phase with a residual displacement of 5.5mm. The maximum support rotation based on the mid-height displacement was 0.84°. Displacement values and time histories are presented in Table 4-1 and Figure A-1a for this test.

Eight strain gauges placed at various locations on the internal longitudinal reinforcement were recorded during this test and are reported in Table 4-2 and Figure B-1a. The maximum tension strain occurred in gauge C3 and maximum compression strain occurred in gauge T3. Both gauges were located at mid-height of the column. Maximum relative tension strain was 0.1449% occurring 29.1ms after the start of loading and maximum compression strain was -0.0436% occurring 28.6ms after the start of loading. The residual tension and compression strain at these locations were 0.0231% and -0.0162% respectively.

There were some closed cracks on the column at mid-height on the tension face after the test. There was no other visible damage to the column following test 1. The post-test column is shown in Figure 4-1a.

4.2.1.2. 1NS-C-2

Test 1NS-C-2 used a driver length of 3353mm and a driver pressure of 110kPa resulting in a reflected pressure of 24.3kPa and a reflected impulse of 257.7kPa-ms over a positive phase of 26.4ms as reported in Table 4-1 and shown in Figure A-1d. The maximum mid-height displacement was 27.5mm occurring 35.1ms after the start of the positive phase with a residual displacement of 4.7mm. The maximum support rotation based on the mid-height displacement was 1.59°. Displacement values and time histories are presented in Table 4-1 and Figure A-1c for this test.

Eight strain gauges placed at various locations on the internal longitudinal reinforcement were recorded during this test and are reported in Table 4-2 and Figure B-1b. The maximum tension strain occurred in gauge C3 and maximum compression strain occurred in gauge T3. Both gauges were located at mid-height of the column. Maximum relative tension strain was 0.3507% occurring 29.9ms after the start of loading and maximum compression strain was -0.0665% occurring 35.1ms after the start of loading. The residual tension and compression strain at these locations were 0.0324% and -0.0410% respectively.

After the test there was no further apparent damage beyond that of test 1. The post-test column is shown in Figure 4-1b.

4.2.1.3. 1NS-C-3

Test 1NS-C-3 used a driver length of 3353mm and a driver pressure of 380kPa resulting in a reflected pressure of 65.1kPa and a reflected impulse of 765.3kPa-ms over a positive phase of 34.4ms as reported in Table 4-1 and shown in Figure A-2b. The maximum mid-height displacement was 171.1mm occurring 57.4ms after the start of the positive phase with a residual displacement of 126.0mm. The maximum support rotation based on the mid-height displacement was 9.81°. Displacement values and time histories are presented in Table 4-1 and Figure A-2a for this test.

Eight strain gauges placed at various locations on the internal longitudinal reinforcement were recorded during this test and are reported in Table 4-2 and Figure B-1c. Strain gauge C3 (unloaded face at mid-height) was damaged early in the test and did not provide reliable data for maximum strains. Strain gauge T3 (loaded face at mid-height) first showed compression behavior and then

tension as the column was significantly displaced. The maximum strain in gauge T3 was 0.2200% occurring 32.9ms after the start of loading with residual strain of 0.1642%.

After the test there was very obvious damage to the column. An interior plastic hinge formed approximately 150mm above the mid-height location where there were large and prevalent cracks in tension concrete visible and a significant amount of crushing of the cover concrete over a length of approximately 200mm. There was no reinforcement buckling at any location on the column. At the support location there was some crushing of the concrete cover up to a depth of approximately 10mm over a length of approximately 50mm. There was significant tension cracking on the loaded face of the column at the supports. The post-test column is shown in Figure 4-1c and Figure 4-1d.

4.2.2. Column 1S-C

Column 1S-C was previously tested for a separate project (Lloyd, 2010) and is included here as it is a control column for the retrofitted columns in group 1. The column had a cross section measuring 100mm by 150mm and was bent about the weak axis. Concrete strength was measured as 58MPa at the time of testing. Closed steel hoops were spaced at 25mm along the length of the column and there were four 10M longitudinal reinforcing steel bars. A 350kN, or 36% of concentric capacity, axial load was applied to the column prior to each shock wave test. The axial load induced a compressive strain of -0.0603% as calculated using Equations [4-1] through [4-4] as reported in Table 4-3. Column 1S-C is the control to Column 1S-J and the seismic companion to Column 1NS-C. The column was subjected to a total of 3 shock tube tests: 1S-C-1, 1S-C-2, and 1S-C-3. The column mounted on the shock tube prior to testing is shown in Figure 4-2a. Column 1NS-C used the end support type A with a clear span of 1980mm between supports and LTD type A as described in the previous chapter.

4.2.2.1. 1S-C-1

Test 1S-C-1 used a driver length of 3353mm and a driver pressure of 69kPa resulting in a reflected pressure of 13.0kPa and a reflected impulse of 155.4kPa-ms over a positive phase of 20.7ms as reported in Table 4-1 and shown in Figure A-3b. The maximum mid-height displacement was 10.9mm occurring 28.7ms after the start of the positive phase with a residual displacement of 2.7mm. The maximum support rotation based on the mid-height displacement was 0.63°. Displacement values and time histories are presented in Table 4-1 and Figure A-3a for this test.

Six strain gauges placed at various locations on the internal longitudinal reinforcement were recorded during this test and are reported in Table 4-3 and Figure B-2a. The maximum tension strain

occurred in gauge C3 which was located at the column mid-height. The corresponding compression steel gauge at the mid-height location was gauge T3. Gauge T3 did not record higher compression strains than the compression steel gauge at the support location (gauge T5). This indicates that gauge T3 was possibly damaged and recording inaccurate strain values. The maximum relative tension strain (gauge C3) was 0.1250% occurring 27.9ms after the start of loading with residual strain of 0.0064%. The maximum compression strain recorded at mid-height (T3) was -0.0207% occurring 34.7ms after the start of loading with -0.0102% residual.

There were some closed cracks on the column at mid-height on the tension face after the test. There was no other visible damage to the column following test 1. The post-test column is shown in Figure 4-2a.

4.2.2.2. 1S-C-2

Test 1S-C-2 used a driver length of 3353mm and a driver pressure of 110kPa resulting in a reflected pressure of 24.0kPa and a reflected impulse of 226.7kPa-ms over a positive phase of 23.3ms as reported in Table 4-1 and shown in Figure A-3d. The maximum mid-height displacement was 21.5mm occurring 29.6ms after the start of the positive phase with a residual displacement of 2.7mm. The maximum support rotation based on the mid-height displacement was 1.25°. Displacement values and time histories are presented in Table 4-1 and Figure A-3c for this test.

The maximum tension strain was recorded at the same locations as in test 1. Maximum tension strain in gauge C3 was 0.2882% occurring 28.3ms with a residual strain of 0.0237%. The maximum compression strain at the mid-height location was -0.0179% occurring 35.7ms after the start of loading with -0.0207% residual. Tabulated strain values for this test are found in Table 4-3. Strain-time histories for this test are shown in Figure B-2b.

After the test there was no further apparent damage beyond that of test 1. The post-test column is shown in Figure 4-2b.

4.2.2.3. 1S-C-3

Test 1S-C-2 used a driver length of 3353mm and a driver pressure of 380kPa resulting in a reflected pressure of 63.1kPa and a reflected impulse of 621.5kPa-ms over a positive phase of 26.3ms as reported in Table 4-1 and shown in Figure A-4b. The maximum mid-height displacement was 156.0mm occurring 53.7ms after the start of the positive phase with a residual displacement of

128.9mm. The maximum support rotation based on the mid-height displacement was 8.95°. Displacement values and time histories are presented in Table 4-1 and Figure A-4a for this test.

The tension steel strain gauge at mid-height of the column (gauge C3) was damaged early in the test and did not provide reliable data. The maximum compression strain at the mid-height location was -1.2490% occurring 26.0ms after the start of loading with -0.9900% residual. Tabulated strain values for this test are found in Table 4-3. Strain-time histories for this test are shown in Figure B-2c.

After the test there was very obvious damage to the column. An interior plastic hinge formed approximately 150mm above the mid-height location where there were large and prevalent cracks in tension concrete visible and a significant amount of crushing of the cover concrete over a length of approximately 250mm. There was no reinforcement buckling at any location on the column. At the support location there was some crushing of the concrete cover up to a depth of approximately 10mm over a length of approximately 50mm. The post-test column is shown in Figure 4-2c and Figure 4-2d.

4.2.3. Column 1NS-J

Column 1NS-J had a cross section measuring 100x150mm with a concrete strength of 56.2MPa at the time of testing. The column was reinforced with 4-10M longitudinal reinforcing steel bars with a yield strength of 482MPa. Closed rectangular 6.35mm diameter hoops with 582MPa yield strength were spaced at 50mm along the length of the column. The column was retrofitted with a single layer of CFRP jacketing over the clear span. An axial load of 350kN, or 37% of concentric capacity, was applied to the column prior to testing. The axial load induced a compressive strain of -0.0611% as calculated using Equations [4-1] through [4-4] as reported in Table 4-4. Column 1NS-J is the retrofitted companion to Column 1NS-C and the non-seismic companion to Column 1S-J. The column was subjected to a total of 3 shock tube tests: 1NS-J-1, 1NS-J-2, and 1NS-J-3. The column mounted on the shock tube prior to testing is shown in Figure 4-3a. Column 1NS-J used the support system Support-A and the load transferring system LTD-A.

4.2.3.1. 1NS-J-1

Test 1NS-J-1 used a driver length of 3353mm and a driver pressure of 70kPa resulting in a reflected pressure of 12.6kPa and a reflected impulse of 145.8kPa-ms over a positive phase of 20.9ms as reported in Table 4-1 and shown in Figure A-5b. The maximum mid-height displacement was 12.8mm occurring 28.0ms after the start of the positive phase. Displacement of the column at a location

279mm below mid-height (labeled as displacement at 1/6 height in Table 4-1 and Figure A-5a) was 7.7mm occurring 30.3ms after the start of loading. The relative residual displacements at the mid-height and 1/6 height were 0.0mm and -0.2mm respectively. The maximum support rotation based on the mid-height displacement was 0.74°. Displacement values and time histories are presented in Table 4-1 and Figure A-5a for this test.

Six strain gauges placed at various locations on the internal longitudinal reinforcement were recorded during this test and are reported in Table 4-4 and Figure B-3a. The maximum tension strain occurred in gauge T3 and maximum compression strain occurred in gauge C3. Both gauges were located at mid-height of the column. Maximum relative tension strain was 0.1218% occurring 27.0ms after the start of loading and maximum compression strain was -0.0797% occurring 33.9ms after the start of loading. The residual tension and compression strain at these locations were 0.0018% and -0.0133% respectively.

There was no visible damage to the column following test 1. The post-test column is shown in Figure 4-3b.

4.2.3.2. 1NS-J-2

Test 1NS-J-2 used a driver length of 3353mm and a driver pressure of 111kPa resulting in a reflected pressure of 19.4kPa, a reflected impulse over the positive phase of 240.8kPa-ms and a positive phase duration of 23.5ms. The maximum displacement at mid-height and 1/6 height was 25.6mm and 15.2mm occurring at 33.3ms and 32.4ms after the start of loading respectively. The maximum support rotation based on mid-height displacement was 1.48°. The relative residual displacements at mid-height and 1/6 height were 0.1mm and 0.0mm respectively. Tabulated results for pressure, impulse, positive phase duration, displacements, support rotation and times to maximum displacements are presented in Table 4-1. The displacement-time histories for mid-height and 1/6 height are shown in Figure A-5c. The reflected pressure and reflected impulse-time histories are shown in Figure A-5d.

The maximum tension and compression strain values were recorded at the same locations as in test 1. A maximum tension strain of 0.2705% occurred at mid-height (gauge T3) 27.5ms after the start of loading. A maximum compression strain of -0.0754% occurred at mid-height (gauge C3) 27.9ms after the start of loading. Relative residual strains of 0.0324% and -0.0248% were recorded at the mid-

height tension and compression gauges respectively after the test. Tabulated strain values for this test are found in Table 4-4. Strain-time histories for this test are shown in Figure B-3b.

After test 2 there were no visible signs of damage to the column. The post-test condition of the column is shown in Figure 4-3c and Figure 4-3d.

4.2.3.3. 1NS-J-3

Test 1NS-J-3 used a driver length of 3353mm and a driver pressure of 380kPa resulting in a reflected pressure of 58.8kPa, a reflected impulse over the positive phase of 685.8kPa-ms and a positive phase duration of 23.6ms. The maximum displacement at mid-height and 1/6 height was 160.9mm and 71.5mm occurring at 52.9ms and 53.5ms after the start of loading respectively. The maximum support rotation based on mid-height displacement was 9.23°. The relative residual displacements at mid-height and 1/6 height were 134.6mm and 60.1mm respectively. Tabulated results for pressure, impulse, positive phase duration, displacements, support rotation and times to maximum displacements are presented in Table 4-1. The displacement-time histories for mid-height and 1/6 height are shown in Figure A-6a. The reflected pressure and reflected impulse-time histories are shown in Figure A-6b.

The maximum tension was shown at the same location as in test 1 and test 2. The tension gauge (T3) at the mid-height location was damaged early in the time-history of this test so no maximum or residual values are known. The gauge that was previously the maximum compression gauge (C3) initially showed compression up to -0.0888% occurring 19.6ms after the start of the loading and then tension up to 0.1971% occurring 50.2ms after the start of the loading as shown in Figure B-3c and reported in Table 4-4. This shift from compression to tension behavior in the reinforcing steel was due to the significant deflection of the column ultimately resulting in considerable damage to the column. Gauge C3 had a residual strain of 0.1235% after the end of the test.

After test 3 there was considerable visible residual displacement as is shown in Figure 4-4a through d. Despite the existence of three plastic hinges, located at the column mid-height and at the supports, there was no sign of concrete crushing at mid-height where the column was wrapped with an FRP jacket. The longitudinal reinforcement did not rupture at mid-height. Along the entire height of the column the FRP jacket retained full bond to the concrete substrate. At mid-height and a surrounding region of approximately 100mm on either side, large cracks of up to 3mm width opened on the tension face following the direction of the fibres in the FRP jacket. The concrete within the support

region of the column was significantly cracked and showed crushing of the compression concrete at the hinge location up to approximately 10mm depth. A single large crack on the tension face of the column opened at the hinge region at the supports where there was no FRP jacketing.

4.2.4. Column 1S-J

Column 1S-J had a cross section measuring 100x150mm with a concrete strength of 56.2MPa at the time of testing. The column was reinforced with 4-10M longitudinal reinforcing steel bars with a yield strength of 482MPa. Closed rectangular 6.35mm diameter hoops with 582MPa yield strength were spaced at 25mm along the length of the column. The column was retrofitted with a single layer of CFRP jacketing over the clear span. An axial load of 350kN, or 37% of concentric capacity, was applied to the column prior to testing. The axial load induced a compressive strain of -0.0611% as calculated using Equations [4-1] through [4-4] as reported in Table 4-5. Column 1S-J is the retrofitted companion to Column 1S-C and the seismic companion to Column 1NS-J. The column was subjected to a total of 3 shock tube tests: 1S-J-1, 1S-J-2, and 1S-J-3. The column mounted on the shock tube prior to testing is shown in Figure 4-5a. Column 1S-J used the support system Support-A and the load transferring system LTD-A.

4.2.4.1. 1S-J-1

Test 1S-J-1 used a driver length of 3353mm and a driver pressure of 69kPa resulting in a reflected pressure of 14.3kPa and a reflected impulse of 165.0kPa-ms over a positive phase of 24.1ms as reported in Table 4-1 and shown in Figure A-7b. The maximum mid-height displacement was 13.4mm occurring 32.5ms after the start of the positive phase. Displacement of the column at a location 279mm below mid-height (labeled as displacement at 1/6 height in Table 4-1 and Figure A-7a) was 8.0mm occurring 32.5ms after the start of loading. The relative residual displacements at the mid-height and 1/6 height were 6.6mm and 4.4mm respectively. Given the relatively high level of residual displacement for both displacement gauges when compared with those of the similar column 1NS-J after shot 1 and the high ratio of displacement at the 1/6 and mid-height locations indicating rigid body motion, it is likely that the residual is largely due to support settling during the test or movement of the displacement gauge mounting stand. The maximum support rotation based on the mid-height displacement was 0.78°. Displacement values and time histories are presented in Table 4-1 and Figure A-7a for this test.

Six strain gauges placed at various locations on the internal longitudinal reinforcement were recorded during this test and are reported in Table 4-5 and Figure B-4a. The maximum tension strain

occurred in gauge T3 and maximum compression strain occurred in gauge C3. Both gauges were located at mid-height of the column. Maximum relative tension strain was 0.0937% occurring 30.2ms after the start of loading and maximum compression strain was -0.0833% occurring 29.8ms after the start of loading. The residual tension and compression strain at these locations were 0.0179% and -0.0454% respectively.

There was no visible damage to the column following test 1. The post-test column is shown in Figure 4-5b.

4.2.4.2. 1S-J-2

Test 1S-J-2 used a driver length of 3353mm and a driver pressure of 110kPa resulting in a reflected pressure of 20.2kPa, a reflected impulse over the positive phase of 227.6kPa-ms and a positive phase duration of 23.5ms. The maximum displacement at mid-height and 1/6 height was 27.1mm and 15.9mm occurring at 40.0ms and 39.0ms after the start of loading respectively. The maximum support rotation based on mid-height displacement was 1.57°. The relative residual displacements at mid-height and 1/6 height were 5.9mm and 3.8mm respectively. Tabulated results for pressure, impulse, positive phase duration, displacements, support rotation and times to maximum displacements are presented in Table 4-1. The displacement-time histories for mid-height and 1/6 height are shown in Figure A-7c. The reflected pressure and reflected impulse-time histories are shown in Figure A-7d.

The maximum tension and compression strain values were recorded at the same locations as in test 1. A maximum tension strain of 0.2706% occurred at mid-height (gauge T3) 34.6ms after the start of loading. A maximum compression strain of -0.1704% occurred at mid-height (gauge C3) 49.9ms after the start of loading. Relative residual strains of 0.0245% and -0.1267% were recorded at the mid-height tension and compression gauges respectively after the test. Tabulated strain values for this test are found in Table 4-5. Strain-time histories for this test are shown in Figure B-4b.

After test 2 there were no visible signs of damage to the column. The post-test condition of the column is shown in Figure 4-5c and Figure 4-5d.

4.2.4.3. 1S-J-3

Test 1S-J-3 used a driver length of 3353mm and a driver pressure of 380kPa resulting in a reflected pressure of 52.5kPa, a reflected impulse over the positive phase of 555.1kPa-ms and a positive phase duration of 22.6ms. The maximum displacement at mid-height and 1/6 height was 135.3mm and

59.7mm occurring at 51.7ms and 52.7ms after the start of loading respectively. The maximum support rotation based on mid-height displacement was 7.78° . The relative residual displacements at mid-height and 1/6 height were 110.3mm and 49.3mm respectively. Tabulated results for pressure, impulse, positive phase duration, displacements, support rotation and times to maximum displacements are presented in Table 4-1. The displacement-time histories for mid-height and 1/6 height are shown in Figure A-8a. The reflected pressure and reflected impulse-time histories are shown in Figure A-8b.

The maximum tension was shown at the same location as in test 1 and test 2. The tension gauge (T3) at the mid-height location was damaged early in the time-history of this test so no maximum or residual values are known. The gauge that was previously the maximum compression gauge (C3) initially showed compression up to -0.1665% occurring 21.0ms after the start of the loading and then tension up to 0.1385% occurring 50.8ms after the start of the loading as shown in Figure B-4c and reported in Table 4-5. This shift from compression to tension behavior in the reinforcing steel was due to the significant deflection of the column ultimately resulting in considerable damage to the column. Gauge C3 had a residual strain of 0.0500% after the end of the test.

After test 3 there was considerable visible residual displacement as is shown in Figure 4-6a through d. Despite the existence of three plastic hinges, located at the column mid-height and at the supports, there was no sign of concrete crushing at mid-height where the column was wrapped with an FRP jacket. The longitudinal reinforcement did not rupture at mid-height. Along the entire height of the column the FRP jacket retained full bond to the concrete substrate. At mid-height and a surrounding region of approximately 100mm on either side, large cracks of up to 3mm width opened on the tension face following the direction of the fibres in the FRP jacket. The concrete within the support region of the column was significantly cracked and showed crushing of the compression concrete at the hinge location up to approximately 10mm depth.

4.2.5. Column 2NS-C

Column 2NS-C had a cross section measuring 150x150mm with a concrete strength of 69.6MPa at the time of testing. The column was reinforced with 4-10M longitudinal reinforcing steel bars with a yield strength of 482MPa. Closed rectangular 6.35mm diameter hoops with 582MPa yield strength were spaced at 75mm along the length of the column. An axial load of 422.8kN, or 25.1% of the column concentric capacity, was applied to the column prior to testing. The axial load induced a compressive strain of -0.0465% as calculated using Equations [4-1] through [4-4] as reported in Table

4-6. Column 2NS-C is the control column to columns 2NS-J (non-seismic FRP jacket), 2NS-R (non-seismic FRP reinforced), and 2NS-JR (non-seismic FRP reinforced with FRP jacket). It is also the non-seismic companion column to the seismic control column 2S-C. Column 2NS-C is also considered a group 3 control column for the destructive test (test 2NS-C-3) as the control column in group 3 (3NS-C) was not tested under similar pressure and impulse loading. The column was subjected to a total of 3 shock tube test: 2NS-C-1, 2NS-C-2, and 2NS-C-3. The column mounted on the shock tube prior to testing is shown in Figure 4-7a. The column was tested using support system Support-B and the load transfer system LTD-B.

4.2.5.1. 2NS-C-1

Test 2NS-C-1 used a driver length of 2743mm and a driver pressure of 70kPa resulting in a reflected pressure of 14.0kPa and a reflected impulse of 140.4kPa-ms over a positive phase of 18.0ms as reported in Table 4-1 and shown in Figure A-9b. The maximum mid-height displacement was 8.2mm occurring 16.6ms after the start of the positive phase. The relative residual displacement at the mid-height was 1.6mm. The maximum support rotation based on the mid-height displacement was 0.47°. Displacement values and time histories are presented in Table 4-1 and Figure A-9a for this test.

Seven strain gauges placed at various locations on the internal longitudinal reinforcement were recorded during this test and are reported in Table 4-6 and Figure B-5a. The maximum tension strain occurred in gauge T3 and maximum compression strain occurred in gauge C3. Both gauges were located at mid-height of the column. Maximum relative tension strain was 0.1127% occurring 16.9ms after the start of loading and maximum compression strain was -0.0914% occurring 16.0ms after the start of loading. The residual tension and compression strain at these locations were 0.0107% and -0.0108% respectively. Strain gauges near the support on the loaded (tension) and unloaded (compression) side of the column measured 0.0204% and -0.0134% maximum tension (gauge C1) and compression (gauge T1) strain both at 8.8ms after the start of loading. The residual strains in gauges C1 and T1 were 0.0082% and -0.0041% respectively.

Following the test there were no visible signs of damage to the column including no noticeable cracking. The post-test column is shown in Figure 4-7b.

4.2.5.2. 2NS-C-2

Test 2NS-C-2 used a driver length of 2743mm and a driver pressure of 243kPa resulting in a reflected pressure of 41.8kPa and a reflected impulse of 478.4kPa-ms over a positive phase of 28.6ms as

reported in Table 4-1 and shown in Figure A-9d. The maximum mid-height displacement was 36.5mm occurring 20.9ms after the start of the positive phase. The relative residual displacement at the mid-height was 11.5mm. The maximum support rotation based on the mid-height displacement was 2.11°. Displacement values and time histories are presented in Table 4-1 and Figure A-9c for this test.

Both the tension and compression strain gauges at mid-height were damaged during the test and did not give reliable measurements. The maximum tension and compression strains near the supports were 0.0871% (gauge C1) and -0.0476% (gauge T1) occurring 18.4ms and 29.6ms, respectively, after the start of loading. The residual strains at these locations were 0.0117% for gauge C1 and 0.0136 for gauge T1. Tabulated strain values for this test are found in Table 4-6. Strain-time histories for this test are shown in Figure B-5b.

Following the test there was evidence of closed tension cracks around the mid-height of the column over a length of approximately 600mm. These cracks were spaced approximately every 75mm which is consistent with the transverse reinforcement spacing. There was no concrete crushing at any location on the column. Some cracking was observed at the support locations, however, since the tension face was obscured by the load transfer device, the extent of cracking was difficult to determine. The post-test column is shown in Figure 4-7c and Figure 4-7d.

4.2.5.3. 2NS-C-3

Test 2NS-C-3 used a driver length of 2743mm and a driver pressure of 546kPa resulting in a reflected pressure of 81.0kPa and a reflected impulse of 1073.5kPa-ms over a positive phase of 40.5ms as reported in Table 4-1 and shown in Figure A-10b. The maximum mid-height displacement was 158.0mm occurring 35.6ms after the start of the positive phase. The relative residual displacement at the mid-height was 136.5mm. The maximum support rotation based on the mid-height displacement was 9.07°. Displacement values and time histories are presented in Table 4-1 and Figure A-10a for this test.

The maximum tension and compression strains near the supports were 0.3717% (gauge C1) and -0.639% (gauge T1) occurring 16.2ms and 14.4ms, respectively, after the start of loading. The residual strains at these locations were 0.1394% for gauge C1 and 0.0514 for gauge T1. Tabulated strain values for this test are found in Table 4-6. Strain-time histories for this test are shown in Figure B-5c.

Following the test significant damage to the column was observed. There were three distinct plastic hinges in the column with the mid-height hinge being more significantly damaged than the support

location hinges. The mid-height hinge was characterized by crushing of the concrete, buckling of the compression reinforcement, and distributed cracking on the tension face. Crushing of the concrete extended beyond cover and into the core between the transverse hoop locations on the column compression side. This crushing depth was largest at the locations between the hoops where it extended to approximately 20-25mm depth. The crushing was localized to approximately 225mm. Compression reinforcement was distinctly buckled between the hoops at mid-height. The distributed tension cracking extended over a length of approximately 800mm with the most significant cracking occurring within 150mm on either side of mid-height. The column experienced less damage in the plastic hinge region at the supports than at mid-height. Although there was large rotation of the column at the supports and significant distributed tension cracks, there was little crushing of compression concrete and no reinforcement buckling. The post-test column is shown in Figure 4-8a through Figure 4-8d.

4.2.6. Column 2S-C

Column 2S-C had a cross section measuring 150x150mm with a concrete strength of 69.6MPa at the time of testing. The column was reinforced with 4-10M longitudinal reinforcing steel bars with a yield strength of 482MPa. Closed rectangular 6.35mm diameter hoops with 582MPa yield strength were spaced at 37.5mm along the length of the column. An axial load of 422.8kN, or 25.1% of the column concentric capacity, was applied to the column prior to testing. The axial load induced a compressive strain of -0.0465% as calculated using Equations [4-1] through [4-4] as reported in Table 4-7. Column 2S-C is the control column to columns 2S-J (seismic FRP jacket), 2S-R (seismic FRP reinforced), and 2S-JR (seismic FRP reinforced with FRP jacket). It is also the seismic companion column to the non-seismic control column 2NS-C. The column was subjected to a total of 3 shock tube test: 2S-C-1, 2S-C-2, and 2S-C-3. The column mounted on the shock tube prior to testing is shown in Figure 4-9a. The column was tested using support system Support-B and the load transfer system LTD-B.

4.2.6.1. 2S-C-1

Test 2S-C-1 used a driver length of 2743mm and a driver pressure of 71kPa resulting in a reflected pressure of 14.4kPa and a reflected impulse of 131.1kPa-ms over a positive phase of 18.4ms as reported in Table 4-1 and shown in Figure A-11b. The maximum mid-height displacement was 4.7mm occurring 18.4ms after the start of the positive phase. The relative residual displacement at the mid-height was 1.4mm. The maximum support rotation based on the mid-height displacement was 0.27°. Displacement values and time histories are presented in Table 4-1 and Figure A-11a for this test.

Five strain gauges placed at various locations on the internal longitudinal reinforcement were recorded during this test and are reported in Table 4-7 and Figure B-6a. The maximum compression strain occurred in gauge C3 located at the mid-height of the column. The tension steel gauge at the mid-height of the column did not give reliable readings and is not reported. The maximum compression strain in gauge C3 was -0.0665% occurring 17.7ms after the start of loading with a residual of -0.0207%. The maximum tension and compression strains in gauges located near the supports, C4 and T4 respectively, were 0.0195% and -0.0148% occurring 17.2ms and 14.4ms after the start of loading. These gauges recorded residual strains of -0.0101% for C4 and 0.0069% for T4.

Following the test there were no visible signs of damage to the column including no noticeable cracking. The post-test column is shown in Figure 4-9b.

4.2.6.2. 2S-C-2

Test 2S-C-2 used a driver length of 2743mm and a driver pressure of 243Pa resulting in a reflected pressure of 47.7kPa and a reflected impulse of 687.5kPa-ms over a positive phase of 45.8ms as reported in Table 4-1 and shown in Figure A-11d. The maximum mid-height displacement was 37.3mm occurring 21.1ms after the start of the positive phase. The relative residual displacement at the mid-height was 9.7mm. The maximum support rotation based on the mid-height displacement was 2.16°. Displacement values and time histories are presented in Table 4-1 and Figure A-11c for this test.

The compression strain gauge at the mid-height was damaged during the test and did not give reliable measurements. The maximum tension and compression strains near the supports were 0.2043% (gauge C4) and -0.1023% (gauge T4) occurring at 19.3ms and 18.9ms, respectively, after the start of loading. Residual strains at these locations were 0.0218% for gauge C4 and -0.0243% for gauge T4. Tabulated strain values for this test are found in Table 4-7. Strain-time histories for this test are shown in Figure B-6b.

Following the test there was evidence of closed tension cracks around the mid-height of the column over a length of approximately 400mm. These cracks were spaced approximately every 37.5mm which is consistent with the transverse reinforcement spacing. There was no concrete crushing at any location on the column. Some cracking was observed at the support locations, however, since the tension face was obscured by the load transfer device, the extent of cracking was difficult to determine. The post-test column is shown in Figure 4-9c and Figure 4-9d.

4.2.6.3. 2S-C-3

Test 2S-C-3 used a driver length of 2743mm and a driver pressure of 542kPa resulting in a reflected pressure of 70.0kPa and a reflected impulse of 686.7kPa-ms over a positive phase of 23.0ms as reported in Table 4-1 and shown in Figure A-12b. The maximum mid-height displacement was 129.0mm occurring 28.0ms after the start of the positive phase. The relative residual displacement at the mid-height was 98.8mm. The displacement gauge disengaged from the column during the initial positive thrust so the reported displacement time-history that is shown was extrapolated from individual frames of the high speed video taken during the test. Due to the low camera resolution, this displacement-time history should only be considered accurate to within approximately 5mm of the true displacement. The maximum support rotation based on the mid-height displacement was 7.42°. Displacement values and time histories are presented in Table 4-1 and Figure A-12a for this test.

There was no reliable strain data recorded during this test as all existing gauges were damaged early during the column response.

Following the test significant damage to the column was observed. There were three distinct plastic hinges in the column with the mid-height hinge being more significantly damaged than the support location hinges. The mid-height hinge was characterized by crushing of the concrete and distributed cracking on the tension face. Crushing of the concrete extended beyond cover and into the core between the transverse hoop locations on the column compression side. This crushing depth was largest at the locations between the hoops where it extended to approximately 15mm depth. The crushing was localized to approximately 115mm. Compression reinforcement did not buckle. The distributed tension cracking extended over a length of approximately 550mm with the most significant cracking occurring within 150mm on either side of mid-height. There was approximately twice the number of tension cracks as in column 2NS-C-3. The column experienced less damage in the plastic hinge region at the supports than at mid-height. Although there was large rotation of the column at the supports and significant distributed tension cracks, there was little crushing of compression concrete and no reinforcement buckling. The post-test column is shown in Figure 4-8a through Figure 4-8d.

4.2.7. Column 2NS-J

Column 2NS-J had a cross section measuring 150x150mm with a concrete strength of 69.6MPa at the time of testing. The column was reinforced with 4-10M longitudinal reinforcing steel bars with a yield strength of 482MPa. Closed rectangular 6.35mm diameter hoops with 582MPa yield strength were spaced at 75mm along the length of the column. The column was retrofitted with a single layer of CFRP jacketing over the clear span. An axial load of 422.8kN, or 25.1% of the column concentric capacity, was applied to the column prior to testing. The axial load induced a compressive strain of -0.0465% as calculated using Equations [4-1] through [4-4] as reported in Table 4-8. Column 2NS-J is a companion column to columns 2NS-C (non-seismic control), 2NS-R (non-seismic FRP reinforced), and 2NS-JR (non-seismic FRP reinforced with FRP jacket). It is also the non-seismic companion column to the seismic FRP jacketed column 2S-J and to the non-seismic prestressed concrete wedge jacketed column 3NS-W. The column was subjected to a total of 4 shock tube test: 2NS-J-1, 2NS-J -2, 2NS-J-3 and 2NS-J-4. The column mounted on the shock tube prior to testing is shown in Figure 4-11a. The column was tested using support system Support-B and the load transfer system LTD-B.

4.2.7.1. 2NS-J-1

Test 2NS-J-1 used a driver length of 2743mm and a driver pressure of 70kPa resulting in a reflected pressure of 13.5kPa and a reflected impulse of 168.3kPa-ms over a positive phase of 21.7ms as reported in Table 4-1 and shown in Figure A-13b. The maximum mid-height displacement was 8.0mm occurring 18.5ms after the start of the positive phase. The relative residual displacement at the mid-height was 2.4mm. The maximum support rotation based on the mid-height displacement was 0.46°. Displacement values and time histories are presented in Table 4-1 and Figure A-13a for this test.

Six strain gauges placed at various locations on the internal longitudinal reinforcement were recorded during this test and are reported in Table 4-8 and Figure B-7a. The maximum compression strain occurred in gauge C3 located at the mid-height of the column. The tension steel gauge at the mid-height of the column did not give reliable readings and is not reported. The maximum compression strain in gauge C3 was -0.1273% occurring 18.3ms after the start of loading with a residual of -0.0605%. The maximum tension and compression strains in gauges located near the supports, C4 and T4 respectively, were 0.0269% and -0.0195% occurring 15.5ms and 15.9ms after the start of loading. These gauges recorded residual strains of -0.0129% for C4 and 0.0064% for T4.

Following the test there were no visible signs of damage to the column including no noticeable cracking. The post-test column is shown in Figure 4-11b.

4.2.7.2. 2NS-J-2

Test 2NS-J-2 used a driver length of 2743mm and a driver pressure of 244kPa resulting in a reflected pressure of 44.8kPa and a reflected impulse of 509.9kPa-ms over a positive phase of 28.9ms as reported in Table 4-1 and shown in Figure A-13d. The maximum mid-height displacement was 36.3mm occurring 21.4ms after the start of the positive phase. The relative residual displacement at the mid-height was 10.0mm. The maximum support rotation based on the mid-height displacement was 2.10°. Displacement values and time histories are presented in Table 4-1 and Figure A-13c for this test.

The compression strain gauge at the mid-height was damaged during the test and did not give reliable measurements. The maximum tension and compression strains near the supports were 0.1500% (gauge C4) and -0.0971% (gauge T4) occurring at 18.6ms and 19.8ms, respectively, after the start of loading. Residual strains at these locations were 0.0262% for gauge C4 and -0.0243% for gauge T4. Tabulated strain values for this test are found in Table 4-8. Strain-time histories for this test are shown in Figure B-7b.

After the test there was no visible damage to the column. Any cracks that may have existed in the concrete were covered by the FRP jacket. There was no loss of bond of the FRP jacket at any location on the column. The post-test column is shown in Figure 4-11c and Figure 4-11d.

4.2.7.3. 2NS-J-3

Test 2NS-J-3 used a driver length of 2743mm and a driver pressure of 548kPa resulting in a reflected pressure of 73.2kPa and a reflected impulse of 643.0kPa-ms over a positive phase of 21.2ms as reported in Table 4-1 and shown in Figure A-14b. The maximum mid-height displacement was 85.5mm occurring 24.6ms after the start of the positive phase. The relative residual displacement at the mid-height was 38.7mm. The maximum support rotation based on the mid-height displacement was 4.94°. Displacement values and time histories are presented in Table 4-1 and Figure A-14a for this test.

The maximum tension and compression strains near the supports were 1.1434 % (gauge C4) and -0.1561% (gauge T4) occurring at 23.6ms and 19.1ms, respectively, after the start of loading. Residual strains at these locations were 0.4291% for gauge C4 and -0.0499% for gauge T4. Tabulated strain values for this test are found in Table 4-8. Strain-time histories for this test are shown in Figure B-7c.

Following the test there was visible residual displacement and evidence of damage to the column at the mid-height and at the support region. At the mid-height there were distributed tension cracks that followed the fibre orientation of the FRP jacket. These cracks were closely spaced and occurred over a length of approximately 400mm. At the support location there was some damage to the concrete outside of the jacketed regions. This damage was in the form of concrete cracking and some crushing of the concrete where the column was clamped to the steel supports. There was no evidence of concrete crushing over the entire unsupported column length. There was no evidence of debonding of the FRP jacket from the concrete substrate at any location over the length of the column. The post-test column is shown in Figure 4-12a through Figure 4-12d.

4.2.7.4. 2NS-J-4

Test 2NS-J-4 used a driver length of 2743mm and a driver pressure of 686kPa. Due to a problem with triggering the data acquisition system, no data or video was recorded for this test. From other similar driver pressure and driver length combination tests, the reflected pressure was likely between 80kPa and 85kPa with a reflective impulse between 840kPa and 1070kPa. The residual displacement was measured after the test at approximately 390mm.

After the test it was observed that the tension reinforcing steel ruptured at three locations: the mid-height hinge and the two hinges at the supports. There was no debonding or rupture of any of the FRP jacket fibres. At the mid-height hinge location a single crack opened at the location of the reinforcement rupture that extended the entire depth of the column. There was no concrete crushing at this location. There was no reinforcement buckling of the compression reinforcement. There was distributed cracking extending approximately 200mm on either side of the reinforcement rupture location as evident by gaps between the fibres of the jacketing. These cracks were up to 3mm wide. At the support locations there was crushing and significant cracking of the concrete outside of the jacket region within the supports. There was evidence of distributed cracking extending approximately 200mm beyond the rebar rupture location that was similar to that seen at mid-height. The post-test column is shown in Figure 4-13a through Figure 4-13d.

4.2.8. Column 2S-J

Column 2S-J had a cross section measuring 150x150mm with a concrete strength of 69.6MPa at the time of testing. The column was reinforced with 4-10M longitudinal reinforcing steel bars with a yield strength of 482MPa. Closed rectangular 6.35mm diameter hoops with 582MPa yield strength were spaced at 37.5mm along the length of the column. The column was retrofitted with a single layer of

CFRP jacketing over the clear span. An axial load of 422.8kN, or 25.1% of the column concentric capacity, was applied to the column prior to testing. The axial load induced a compressive strain of -0.0465% as calculated using Equations [4-1] through [4-4] as reported in Table 4-9. Column 2S-J is a companion column to columns 2S-C (seismic control), 2S-R (seismic FRP reinforced), and 2S-JR (seismic FRP reinforced with FRP jacket). It is also the seismic companion column to the non-seismic FRP jacketed column 2NS-J and to the non-seismic prestressed concrete wedge jacketed column 3NS-W. The column was subjected to a total of 4 shock tube test: 2S-J-1, 2S-J-2, 2S-J-3 and 2S-J-4. The column mounted on the shock tube prior to testing is shown in Figure 4-14a. The column was tested using support system Support-B and the load transfer system LTD-B.

4.2.8.1. 2S-J-1

Test 2S-J-1 used a driver length of 2743mm and a driver pressure of 72kPa resulting in a reflected pressure of 14.6kPa and a reflected impulse of 188.2kPa-ms over a positive phase of 21.6ms as reported in Table 4-1 and shown in Figure A-15b. The maximum mid-height displacement was 7.7mm occurring 17.3ms after the start of the positive phase. The relative residual displacement at the mid-height was 2.8mm. The maximum support rotation based on the mid-height displacement was 0.45°. Displacement values and time histories are presented in Table 4-1 and Figure A-15a for this test.

Three strain gauges placed at various locations on the internal longitudinal reinforcement were recorded during this test and are reported in Table 4-9 and Figure B-8a. The maximum tension strain occurred in gauge T3 located at the mid-height of the column. The compression steel gauge at the mid-height of the column did not give reliable readings and is not reported. The maximum tension strain in gauge T3 was 0.1090% occurring 16.8ms after the start of loading with a residual of 0.0195%. The maximum tension and compression strains in gauges located near the supports, C1 and T2 respectively, were 0.0138% and -0.0171% occurring 16.8ms and 8.6ms after the start of loading. These gauges recorded residual strains of -0.0137% for C1 and 0.0164% for T1.

Following the test there were no visible signs of damage to the column including no noticeable cracking. The post-test column is shown in Figure 4-14b.

4.2.8.2. 2S-J-2

Test 2S-J-2 used a driver length of 2743mm and a driver pressure of 244kPa resulting in a reflected pressure of 43.7kPa and a reflected impulse of 420.3kPa-ms over a positive phase of 25.7ms as reported in Table 4-1 and shown in Figure A-15d. The maximum mid-height displacement was

25.1mm occurring 18.5ms after the start of the positive phase. The relative residual displacement at the mid-height was 5.5mm. The maximum support rotation based on the mid-height displacement was 1.45°. Displacement values and time histories are presented in Table 4-1 and Figure A-15c for this test.

The maximum tension strain at mid-height (gauge T3) was 0.7073% occurring 18.4ms after the start of loading with 0.1071% relative residual displacement. The maximum tension and compression strains near the supports were 0.0639% (gauge C1) and -0.0584% (gauge T1) occurring at 18.0ms and 18.7ms, respectively, after the start of loading. Residual strains at these locations were 0.0217% for gauge C1 and -0.0221% for gauge T1. Tabulated strain values for this test are found in Table 4-9. Strain-time histories for this test are shown in Figure B-8b.

After the test there was no visible damage to the column. Any cracks that may have existed in the concrete were covered by the FRP jacket. There was no loss of bond of the FRP jacket at any location on the column. The post-test column is shown in Figure 4-14c and Figure 4-14d.

4.2.8.3. 2S-J-3

Test 2S-J-3 used a driver length of 2743mm and a driver pressure of 548kPa resulting in a reflected pressure of 75.3kPa and a reflected impulse of 787.2kPa-ms over a positive phase of 22.2ms as reported in Table 4-1 and shown in Figure A-16b. The maximum mid-height displacement was 99.5mm occurring 24.8ms after the start of the positive phase. The relative residual displacement at the mid-height was 46.0mm. The maximum support rotation based on the mid-height displacement was 5.74°. Displacement values and time histories are presented in Table 4-1 and Figure A-16a for this test.

The tension steel strain gauge (T3) at mid-height was damaged early during the response and did not produce reliable results. The maximum tension and compression strains near the supports were 0.9389 % (gauge C1) and -0.3645% (gauge T1) occurring at 26.1ms and 32.5ms, respectively, after the start of loading. Residual strains at these locations were 0.2425% for gauge C1 and -0.2032% for gauge T1. Tabulated strain values for this test are found in Table 4-9. Strain-time histories for this test are shown in Figure B-8c.

Following the test there was visible residual displacement and evidence of damage to the column at the mid-height and at the support region. At the mid-height there were distributed tension cracks that followed the fibre orientation of the FRP jacket. These cracks were closely spaced and occurred

over a length of approximately 300mm. At the support location there was some damage to the concrete outside of the jacketed regions. This damage was in the form of concrete cracking and some crushing of the concrete where the column was clamped to the steel supports. There was no evidence of concrete crushing over the entire unsupported column length. There was no evidence of debonding of the FRP jacket from the concrete substrate at any location over the length of the column. The post-test column is shown in Figure 4-15a through Figure 4-15d.

4.2.8.4. 2S-J-4

Test 2S-J-4 used a driver length of 2743mm and a driver pressure of 683kPa resulting in a reflected pressure of 85.0kPa and a reflected impulse of 839.2kPa-ms over a positive phase of 19.9ms as reported in Table 4-1 and shown in Figure A-16d. During the test the displacement gauge disengaged from the column resulting in displacement readings only valid in the initial positive displacement thrust. Displacements were interpolated from still images taken from the high speed video camera recording the response of the column. Due to limitations in the resolution of the camera, this displacement should only be considered accurate to within 5mm over the entire range of the time history. The maximum mid-height displacement was 256.5mm occurring 64.0ms after the start of the positive phase. The relative residual displacement at the mid-height was 241.5mm. The maximum support rotation based on the mid-height displacement was 14.52°. Displacement values and time histories are presented in Table 4-1 and Figure A-16c for this test.

There was no reliable strain data recorded during this test as all existing gauges were damaged early during the column response.

After the test it was observed that the tension reinforcing steel ruptured at three locations: the mid-height hinge and the two hinges at the supports. There was no debonding or rupture of any of the FRP jacket fibres. At the mid-height hinge location a single crack opened at the location of the reinforcement rupture that extended the entire depth of the column. There was no concrete crushing at this location. There was no reinforcement buckling of the compression reinforcement. There was distributed cracking extending approximately 200mm on either side of the reinforcement rupture location as evident by gaps between the fibres of the jacketing. These cracks were up to 3mm wide. At the support locations there was crushing and significant cracking of the concrete outside of the jacket region within the supports. There was evidence of distributed cracking extending approximately 200mm beyond the rebar rupture location that was similar to that seen at mid-height. The post-test column is shown in Figure 4-16a through Figure 4-16d.

4.2.9. Column 2NS-R

Column 2NS-R had a cross section measuring 150x150mm with a concrete strength of 69.6MPa at the time of testing. The column was reinforced with 4-10M longitudinal reinforcing steel bars with a yield strength of 482MPa. Closed rectangular 6.35mm diameter hoops with 582MPa yield strength were spaced at 75mm along the length of the column. The column was retrofitted with a single layer of CFRP reinforcement bonded to both the loaded and unloaded face extending over the clear span. An axial load of 422.8kN, or 25.1% of the column concentric capacity, was applied to the column prior to testing. The axial load induced a compressive strain of -0.0465% as calculated using Equations [4-1] through [4-4] as reported in Table 4-10. Column 2NS-R is a companion column to columns 2NS-C (non-seismic control), 2NS-J (non-seismic FRP jacketed), and 2NS-JR (non-seismic FRP reinforced with FRP jacket). It is also the non-seismic companion column to the seismic FRP reinforced column 2S-R. The column was subjected to a total of 3 shock tube test: 2NS-R-1, 2NS-R -2, and 2NS-R-3. The column mounted on the shock tube prior to testing is shown in Figure 4-17a. The column was tested using support system Support-B and the load transfer system LTD-B.

4.2.9.1. 2NS-R-1

Test 2NS-R-1 used a driver length of 2743mm and a driver pressure of 70kPa resulting in a reflected pressure of 13.7kPa and a reflected impulse of 189.3kPa-ms over a positive phase of 22.1ms as reported in Table 4-1 and shown in Figure A-17b. The maximum mid-height displacement was 8.6mm occurring 18.4ms after the start of the positive phase. The relative residual displacement at the mid-height was 2.5mm. The maximum support rotation based on the mid-height displacement was 0.50°. Displacement values and time histories are presented in Table 4-1 and Figure A-17a for this test.

Seven strain gauges placed at various locations on the internal longitudinal reinforcement and on the tension FRP surface at mid-height were recorded during this test and are reported in Table 4-10 and Figure B-9a. The maximum tension and compression strain in steel occurred in gauges T3 and C3, respectively, located at the mid-height of the column. The maximum tension strain in gauge T3 was 0.1378% occurring 16.4ms after the start of loading with a residual of 0.0220%. The maximum compression strain in gauge C3 was -0.0997% occurring 16.5ms after the start of loading with a residual of -0.0247%. The maximum strain in the FRP (gauge FRP-T) at mid-height was 0.1135% occurring 16.4ms after the start of loading with a residual of 0.0197%. As the surface bonded FRP is at a larger distance from the neutral axis of the section than the steel gauge, if plane section response and complete bond of the FRP to the substrate is assumed, the strain in the FRP should be larger than

the strain in the tension steel. In this case the strain in the FRP is the lower value. This is likely due to the fidelity of using single strain gauges to determine section strain. As no loss of bond between the FRP and the concrete substrate was observed, it is assumed that at least one of the tension strain gauges reported incorrect values.

Following the test there were no visible signs of damage to the column including no noticeable cracking and no loss of bond between the FRP and the concrete substrate. The post-test column is shown in Figure 4-17b.

4.2.9.2. 2NS-R-2

Test 2NS-R-2 used a driver length of 2743mm and a driver pressure of 242kPa resulting in a reflected pressure of 47.2kPa and a reflected impulse of 371.7kPa-ms over a positive phase of 20.2ms as reported in Table 4-1 and shown in Figure A-17d. The maximum mid-height displacement was 24.3mm occurring 17.5ms after the start of the positive phase. The relative residual displacement at the mid-height was 4.6mm. The maximum support rotation based on the mid-height displacement was 1.41°. Displacement values and time histories are presented in Table 4-1 and Figure A-17c for this test.

The maximum tension and compression strains at mid-height were 0.7991% (gauge T3) and -0.4148% (gauge C3) occurring at 16.7ms and 17.9ms, respectively, after the start of loading. Residual strains at these locations were 0.1219% for gauge T3 and -0.1654% for gauge C3. The maximum strain in the FRP at mid-height (gauge FRP-T) was 0.5027% occurring 16.7ms after the start of loading with 0.0400% residual strain. Similarly to test 1, the FRP strain gauge recorded lower strain than the tension steel strain gauge. This discrepancy likely reflects an error in either one or both of the strain gauges rather than loss of material bond or some other complex section behavior. Tabulated strain values for this test are found in Table 4-9. Strain-time histories for this test are shown in Figure B-9b.

Following the test there were some hairline cracks visible at the mid-height tension face of the concrete. The extent of cracking was obscured by the surface bonded FRP. There were no signs of further damage including concrete crushing and FRP debonding from the concrete substrate. The post-test column is shown in Figure 4-17c and Figure 4-17d.

4.2.9.3. 2NS-R-3

Test 2NS-R-3 used a driver length of 2743mm and a driver pressure of 547kPa resulting in a reflected pressure of 99.4kPa and a reflected impulse of 716.0kPa-ms over a positive phase of 23.0ms as

reported in Table 4-1 and shown in Figure A-18b. The maximum mid-height displacement was 127.6mm occurring 29.9ms after the start of the positive phase. The relative residual displacement at the mid-height was 93.4mm. The maximum support rotation based on the mid-height displacement was 7.35°. Displacement values and time histories are presented in Table 4-1 and Figure A-18a for this test.

Both the tension and compression steel gauges at mid-height of the column were damaged early during the dynamic response of the test. Neither of these gauges produced reliable information. The maximum strain in the FRP occurred at the point of FRP debonding from the concrete substrate. Maximum strain in the FRP (gauge FRP-T) was 1.1163% occurring 21.1ms after the start of loading. As the FRP debonded from the concrete substrate, there was no residual strain that could be measured in the FRP. Tabulated strain values for this test are found in Table 4-10. Strain-time histories for this test are shown in Figure B-9c.

Following the test it was observed that the FRP on the unloaded face debonded from the concrete substrate over nearly the entire column length. The FRP on the loaded face buckled at the location of the mid-height plastic hinge. There was significant cracking of the concrete at mid-height with some cracks opening up to 5mm. The concrete in compression at mid-height crushed into the core region to a maximum depth of approximately 20mm. The crushing of concrete extended over a region of approximately 300mm. The most significant crushing occurred in the region between the transverse reinforcement. The compression reinforcement steel buckled between hoops at the mid-height of the column. At the support locations the FRP on the loaded side (tension side at supports) did not debond from the substrate. There was some distributed tension cracks that extended approximately 300mm along the height of the column from the supports. There was only minimal crushing of the cover compression concrete at the support regions. The post-test column is shown in Figure 4-18a through Figure 4-18d.

4.2.10. Column 2S-R

Column 2S-R had a cross section measuring 150x150mm with a concrete strength of 69.6MPa at the time of testing. The column was reinforced with 4-10M longitudinal reinforcing steel bars with a yield strength of 482MPa. Closed rectangular 6.35mm diameter hoops with 582MPa yield strength were spaced at 37.5mm along the length of the column. The column was retrofitted with a single layer of CFRP reinforcement bonded to both the loaded and unloaded face extending over the clear span. An axial load of 422.8kN, or 25.1% of the column concentric capacity, was applied to the column prior

to testing. The axial load induced a compressive strain of -0.0465% as calculated using Equations [4-1] through [4-4] as reported in Table 4-11. Column 2NS-R is a companion column to columns 2S-C (seismic control), 2S-J (seismic FRP jacketed), and 2S-JR (seismic FRP reinforced with FRP jacket). It is also the non-seismic companion column to the non-seismic FRP reinforced column 2NS-R. The column was subjected to a total of 3 shock tube test: 2S-R-1, 2S-R -2, and 2S-R-3. The column mounted on the shock tube prior to testing is shown in Figure 4-19a. The column was tested using support system Support-B and the load transfer system LTD-B.

4.2.10.1. 2S-R-1

Test 2S-R-1 used a driver length of 2743mm and a driver pressure of 70kPa resulting in a reflected pressure of 14.5kPa and a reflected impulse of 157.7kPa-ms over a positive phase of 18.3ms as reported in Table 4-1 and shown in Figure A-19b. The maximum mid-height displacement was 7.3mm occurring 16.8ms after the start of the positive phase. The relative residual displacement at the mid-height was 2.1mm. The maximum support rotation based on the mid-height displacement was 0.42°. Displacement values and time histories are presented in Table 4-1 and Figure A-19a for this test.

Six strain gauges placed at various locations on the internal longitudinal reinforcement and on the tension FRP surface at mid-height were recorded during this test and are reported in Table 4-11 and Figure B-10a. The maximum tension and compression strain in steel occurred in gauges T3 and C3, respectively, located at the mid-height of the column. The maximum tension strain in gauge T3 was 0.1137% occurring 17.1ms after the start of loading with a residual of 0.0146%. The maximum compression strain in gauge C3 was -0.1051% occurring 17.0ms after the start of loading with a residual of -0.0287%. The maximum strain in the FRP (gauge FRP-T) at mid-height was 0.1392% occurring 17.1ms after the start of loading with a residual of 0.0174%.

Following the test there were no visible signs of damage to the column including no noticeable cracking and no loss of bond between the FRP and the concrete substrate. The post-test column is shown in Figure 4-19b.

4.2.10.2. 2S-R-2

Test 2S-R-2 used a driver length of 2743mm and a driver pressure of 239kPa resulting in a reflected pressure of 46.5kPa and a reflected impulse of 474.2kPa-ms over a positive phase of 24.7ms as reported in Table 4-1 and shown in Figure A-19d. The maximum mid-height displacement was 22.7mm occurring 16.4ms after the start of the positive phase. The relative residual displacement at

the mid-height was 5.4mm. The maximum support rotation based on the mid-height displacement was 1.31°. Displacement values and time histories are presented in Table 4-1 and Figure A-19c for this test.

The maximum tension and compression strains at mid-height were 0.6225% (gauge T3) and -0.5636% (gauge C3) occurring at 17.4ms and 17.0ms, respectively, after the start of loading. Residual strains at these locations were 0.0981% for gauge T3 and -0.3326% for gauge C3. The maximum strain in the FRP at mid-height (gauge FRP-T) was 0.4943% occurring 16.3ms after the start of loading with 0.0248% residual strain. Similarly to the strain gauge readings for column 2NS-R, the FRP strain gauge recorded lower strain than the tension steel strain gauge. This discrepancy likely reflects an error in either one or both of the strain gauges rather than loss of material bond or some other complex section behavior. Tabulated strain values for this test are found in Table 4-11. Strain-time histories for this test are shown in Figure B-10b.

Following the test there were some hairline cracks visible at the mid-height tension face of the concrete. The extent of cracking was obscured by the surface bonded FRP. There were no signs of further damage including concrete crushing and FRP debonding from the concrete substrate. The post-test column is shown in Figure 4-19c and Figure 4-19d.

4.2.10.3. 2S-R-3

Test 2S-R-3 used a driver length of 2743mm and a driver pressure of 551kPa resulting in a reflected pressure of 85.3kPa and a reflected impulse of 776.0kPa-ms over a positive phase of 22.8ms as reported in Table 4-1 and shown in Figure A-20b. The maximum mid-height displacement was 97.3mm occurring 26.8ms after the start of the positive phase. The relative residual displacement at the mid-height was 51.4mm. The maximum support rotation based on the mid-height displacement was 5.61°. Displacement values and time histories are presented in Table 4-1 and Figure A-20a for this test.

The compression gauge at mid-height was damaged early during test 3 and produced no reliable results. The maximum tension strain at mid-height was 1.1340% (gauge T3) occurring at 11.4ms after the start of loading with a residual strain of 0.2260%. The maximum strain in the FRP occurred at the point of FRP debonding from the concrete substrate. Maximum strain in the FRP (gauge FRP-T) was 1.1202% occurring 13.8ms after the start of loading. As the FRP debonded from the concrete

substrate, there was no residual strain that could be measured in the FRP. Tabulated strain values for this test are found in Table 4-11. Strain-time histories for this test are shown in Figure B-10c.

Following the test it was observed that the FRP on the unloaded face debonded from the concrete substrate over nearly the entire column length. The FRP on the loaded face buckled at the location of the mid-height plastic hinge. There was significant cracking of the concrete at mid-height with some cracks opening up to 2mm. The concrete in compression at mid-height crushed into the core region to a maximum depth of approximately 15mm. The crushing of concrete extended over a region of approximately 200mm. The most significant crushing occurred in the region between the transverse reinforcement. The compression reinforcement steel buckled between hoops at the mid-height of the column. At the support locations the FRP on the loaded side (tension side at supports) did not debond from the substrate. There was some distributed tension cracks that extended approximately 300mm along the height of the column from the supports. There was only minimal crushing of the cover compression concrete at the support regions. The post-test column is shown in Figure 4-20a through Figure 4-20d.

4.2.11. Column 2NS-JR

Column 2NS-JR had a cross section measuring 150x150mm with a concrete strength of 69.6MPa at the time of testing. The column was reinforced with 4-10M longitudinal reinforcing steel bars with a yield strength of 482MPa. Closed rectangular 6.35mm diameter hoops with 582MPa yield strength were spaced at 75mm along the length of the column. The column was retrofitted with a single layer of CFRP reinforcement bonded to both the loaded and unloaded face extending over the clear span and a single layer of CFRP jacketing wrapped on top of the FRP reinforcement extending over the entire clear span. An axial load of 422.8kN, or 25.1% of the column concentric capacity, was applied to the column prior to testing. The axial load induced a compressive strain of -0.0465% as calculated using Equations [4-1] through [4-4] as reported in Table 4-12. Column 2NS-JR is a companion column to columns 2NS-C (non-seismic control), 2NS-J (non-seismic FRP jacketed), and 2NS-R (non-seismic FRP reinforced). It is also the non-seismic companion column to the seismic FRP reinforced and FRP jacketed column 2S-JR. The column was subjected to a total of 3 shock tube test: 2NS-JR-1, 2NS-JR - 2, and 2NS-JR-3. The column mounted on the shock tube prior to testing is shown in Figure 4-21a. The column was tested using support system Support-B and the load transfer system LTD-B.

4.2.11.1. 2NS-JR-1

Test 2NS-JR-1 used a driver length of 2743mm and a driver pressure of 72kPa resulting in a reflected pressure of 13.4kPa and a reflected impulse of 130.1kPa-ms over a positive phase of 17.9ms as reported in Table 4-1 and shown in Figure A-23b. The maximum mid-height displacement was 7.3mm occurring 16.8ms after the start of the positive phase. The relative residual displacement at the mid-height was -2.7mm. For this test the mount of the displacement gauge to the column was loose allowing for some slip. This slip accounts for the negative residual displacement and could have introduced error in the maximum displacement of up to 5mm. The maximum support rotation based on the mid-height displacement was 0.42°. Displacement values and time histories are presented in Table 4-1 and Figure A-23a for this test.

Seven strain gauges placed at various locations on the internal longitudinal reinforcement and on the tension FRP surface at mid-height were recorded during this test and are reported in Table 4-12 and Figure B-11a. The maximum tension and compression strain in steel occurred in gauges T3 and C3, respectively, located at the mid-height of the column. The maximum tension strain in gauge T3 was 0.1218% occurring 17.1ms after the start of loading with a residual of 0.0022%. The maximum compression strain in gauge C3 was -0.0825% occurring 17.1ms after the start of loading with a residual of -0.0072%. The maximum strain in the longitudinal FRP (gauge FRP-T) at mid-height was 0.1346% occurring 17.1ms after the start of loading with a residual of -0.0002%.

Following the test there were no visible signs of damage to the column including no noticeable cracking and no loss of bond between the FRP and the concrete substrate. The post-test column is shown in Figure 4-21b.

4.2.11.2. 2NS-JR-2

Test 2NS-JR-2 used a driver length of 2743mm and a driver pressure of 239kPa resulting in a reflected pressure of 40.6kPa and a reflected impulse of 355.6kPa-ms over a positive phase of 20.0ms as reported in Table 4-1 and shown in Figure A-21d. The maximum mid-height displacement was 32.6mm occurring 17.8ms after the start of the positive phase. The relative residual displacement at the mid-height was 0.3mm. The maximum support rotation based on the mid-height displacement was 1.89°. Displacement values and time histories are presented in Table 4-1 and Figure A-21c for this test. For this test the mount of the displacement gauge to the column was loose allowing for some slip. This slip accounts for the sharp peaks in the displacement-time history shown in Figure A-21c.

The maximum tension and compression strains at mid-height were 0.6392% (gauge T3) and -0.4478% (gauge C3) occurring at 17.3ms and 17.7ms, respectively, after the start of loading. Residual strains at these locations were 0.1277% for gauge T3 and -0.4478% for gauge C3. The maximum strain in the FRP at mid-height (gauge FRP-T) was 0.6065% occurring 17.0ms after the start of loading with 0.0645% residual strain. Similarly to the some of the strain gauge readings for column 2NS-R and 2S-R, the FRP strain gauge recorded lower strain than the tension steel strain gauge. This discrepancy likely reflects an error in either one or both of the strain gauges rather than loss of material bond or some other complex section behavior. Tabulated strain values for this test are found in Table 4-12. Strain-time histories for this test are shown in Figure B-11b.

Following the test there were no signs of damage including concrete crushing and FRP debonding from the concrete substrate. Any cracks in the concrete that may have existed were covered by the FRP and not visible. The post-test column is shown in Figure 4-21c and Figure 4-21d.

4.2.11.3. 2NS-JR-3

Test 2NS-JR-3 used a driver length of 2743mm and a driver pressure of 550kPa resulting in a reflected pressure of 79.8kPa and a reflected impulse of 678.5kPa-ms over a positive phase of 23.7ms as reported in Table 4-1 and shown in Figure A-22b. The maximum mid-height displacement was 70.1mm occurring 23.4ms after the start of the positive phase. The relative residual displacement at the mid-height was 14.7mm. The maximum support rotation based on the mid-height displacement was 4.05°. Displacement values and time histories are presented in Table 4-1 and Figure A-22a for this test.

The tension steel strain gauge (T3) at mid-height was damaged early in the test and did not provide reliable data. The maximum compression strain at mid-height was -1.3993 % (gauge C3) occurring 21.8ms after the start of loading with a residual strain of -0.8575%. The maximum strain in the FRP at mid-height (gauge FRP-T) was 1.4519% occurring 15.6ms after the start of loading with. This maximum strain occurred at the point of FRP rupture so no residual strain could be reported. Tabulated strain values for this test are found in Table 4-12. Strain-time histories for this test are shown in Figure B-11c.

Following the test the column appeared largely intact with only a slightly visible residual displacement. There were no signs of concrete crushing or damage to the FRP jacket fibres or the bond. Based on the high strain recorded in gauge FRP-T as well as the sudden failure of the gauge

after 15.6ms and the existence of moderate residual displacement, it was thought that the longitudinal FRP may have ruptured beneath the FRP jacket. A small layer of the FRP jacket on the tension face of the column at mid-height was removed and FRP rupture was confirmed. All of the longitudinal fibres ruptured at approximately mid-height of the column. There was no sign of loss of bond between the longitudinal fibres and the concrete substrate outside of the rupture area. The post-test column is shown in Figure 4-22a through Figure 4-22d.

4.2.12. Column 2S-JR

Column 2S-JR had a cross section measuring 150x150mm with a concrete strength of 69.6MPa at the time of testing. The column was reinforced with 4-10M longitudinal reinforcing steel bars with a yield strength of 482MPa. Closed rectangular 6.35mm diameter hoops with 582MPa yield strength were spaced at 75mm along the length of the column. The column was retrofitted with a single layer of CFRP reinforcement bonded to both the loaded and unloaded face extending over the clear span and a single layer of CFRP jacketing wrapped on top of the FRP reinforcement extending over the entire clear span. An axial load of 422.8kN, or 25.1% of the column concentric capacity, was applied to the column prior to testing. The axial load induced a compressive strain of -0.0465% as calculated using Equations [4-1] through [4-4] as reported in Table 4-13. Column 2S-JR is a companion column to columns 2S-C (seismic control), 2S-J (seismic FRP jacketed), and 2S-R (seismic FRP reinforced). It is also the seismic companion column to the non-seismic FRP reinforced and FRP jacketed column 2NS-JR. The column was subjected to a total of 3 shock tube test: 2S-JR-1, 2S-JR -2, and 2S-JR-3. The column mounted on the shock tube prior to testing is shown in Figure 4-23a. The column was tested using support system Support-B and the load transfer system LTD-B.

4.2.12.1. 2S-JR-1

Test 2S-JR-1 used a driver length of 2743mm and a driver pressure of 70kPa resulting in a reflected pressure of 13.8kPa and a reflected impulse of 168.0kPa-ms over a positive phase of 20.9ms as reported in Table 4-1 and shown in Figure A-23b. The maximum mid-height displacement was 8.1mm occurring 17.1ms after the start of the positive phase. The relative residual displacement at the mid-height was -2.2mm. The maximum support rotation based on the mid-height displacement was 0.47°. Displacement values and time histories are presented in Table 4-1 and Figure A-23a for this test.

Seven strain gauges placed at various locations on the internal longitudinal reinforcing steel and external longitudinal FRP were recorded during this test and are reported in Table 4-13 and Figure B-12a. The strain gauges placed on the longitudinal fibres of the FRP strengthening retrofit were

damaged during the placement of the FRP jacket. As the jacket covered the longitudinal fibres, there was no way to replace the strain gauges prior to testing. The maximum tension and compression strain in steel occurred in gauges T3 and C3, respectively, located at the mid-height of the column. The maximum tension strain in gauge T3 was 0.1468% occurring 16.3ms after the start of loading with a residual of 0.0212%. The maximum compression strain in gauge C3 was -0.0847% occurring 16.7ms after the start of loading with a residual of -0.0255%.

Following the test there were no visible signs of damage to the column including no noticeable cracking and no loss of bond between the FRP and the concrete substrate. The post-test column is shown in Figure 4-23b.

4.2.12.2. 2S-JR-2

Test 2S-JR-2 used a driver length of 2743mm and a driver pressure of 239kPa resulting in a reflected pressure of 47.0kPa and a reflected impulse of 443.4kPa-ms over a positive phase of 26.1ms as reported in Table 4-1 and shown in Figure A-23d. The maximum mid-height displacement was 31.4mm occurring 17.2ms after the start of the positive phase. The relative residual displacement at the mid-height was 4.9mm. The maximum support rotation based on the mid-height displacement was 1.82°. Displacement values and time histories are presented in Table 4-1 and Figure A-23c for this test.

The maximum tension and compression strains at mid-height were 1.5674% (gauge T3) and -0.7355% (gauge C3) occurring at 18.7ms and 19.6ms, respectively, after the start of loading. Residual strains at these locations were 0.7390% for gauge T3 and -0.5202% for gauge C3. Although there was no strain gauge recording strain on the longitudinal FRP for this test, strain compatibility between the compression and the tension strain gauges can be used to approximate maximum strain in the FRP. Using the assumptions of full composite behaviour between the FRP and the concrete substrate and constant curvature across the section, the maximum relative FRP strain at mid-height is calculated as 2.5907%. By subtracting the initial axial strain magnitude from this value, the approximate maximum FRP strain of 2.5443% is determined. This value is significantly larger than the specified FRP ultimate strain of 1.2% (Fyfe Co., 2009) indicating that the FRP has ruptured at mid-height of the column. There were similar maximum displacements for this test (31.4mm) and column test 2NS-JR-2 (32.6mm) while the maximum FRP strain for 2NS-JR-2 was only 0.6065%, or approximately half the specified rupture strain. These similarities indicate that rather than FRP rupture in this test, it is possible that there were incorrect readings on the steel strain gauges and the FRP remained intact.

If the FRP had ruptured in this test, the column would behave similarly to columns 2NS-J-3 and 2S-J-3 for test 2S-JR-3 as the reinforcing FRP would not significantly contribute to the resistance. The maximum displacement, time to maximum displacement and residual displacement values for test 2S-JR-3 was similar to 2NS-J-3 and 2S-J-3 indicating that rupture of the FRP may have occurred in test 2S-JR-2. Tabulated strain values for this test are found in Table 4-13. Strain-time histories for this test are shown in Figure B-12b.

Following the test there were no signs of damage including concrete crushing and FRP debonding from the concrete substrate. It was not possible to determine if the longitudinal FRP had ruptured. Any cracks in the concrete that may have existed were covered by the FRP and not visible. The post-test column is shown in Figure 4-23c and Figure 4-23d.

4.2.12.3. 2S-JR-3

Test 2NS-JR-3 used a driver length of 2743mm and a driver pressure of 563kPa resulting in a reflected pressure of 88.2kPa and a reflected impulse of 793.2kPa-ms over a positive phase of 22.1ms as reported in Table 4-1 and shown in Figure A-22b. The maximum mid-height displacement was 102.7mm occurring 25.5ms after the start of the positive phase. The relative residual displacement at the mid-height was 51.8mm. The maximum support rotation based on the mid-height displacement was 5.92°. Displacement values and time histories are presented in Table 4-1 and Figure A-24a for this test.

Both the tension steel strain gauge (T3) and compression steel strain gauge (C3) at mid-height were damaged early in the test and did not provide reliable data. The tension and compression strain on the reinforcing steel near the supports (gauges C1 and T1 respectively) were 0.3285% and -0.0899% occurring 23.8ms and 31.5ms after the start of loading with 0.0575% and -0.0365% residual strain. Tabulated strain values for this test are found in Table 4-13. Strain-time histories for this test are shown in Figure B-12c.

Following the test the column appeared largely intact with visible residual displacement. There were no signs of concrete crushing or damage to the FRP jacket fibres or the bond. There was visible cracking at mid-height characterized by gaps between the fibres of the FRP jacket. These cracks were spaced at approximately 40mm over approximately 200mm centered at mid-height of the column. Based on the maximum displacement, residual displacement and time to maximum displacement values for this test compared with tests with only FRP jackets (2NS-J-3 and 2S-J-3) and the companion

non-seismic FRP reinforced and jacketed column (2NS-JR-3) it is likely that the FRP had ruptured at mid-height in the previous test (2S-JR-2). A discussion of this conclusion is presented in section 4.2.12.2. Based on the high residual displacement there was no doubt that the FRP had ruptured sometime before the end of test 2S-JR-3. Unlike for column 2NS-JR, the FRP jacketing was not removed to confirm this conclusion. The post-test column is shown in Figure 4-24a through Figure 4-24d.

4.2.13. Column 3NS-C

Column 3NS-C had a cross section measuring 150x150mm with a concrete strength of 49.5MPa at the time of testing. The column was reinforced with 4-10M longitudinal reinforcing steel bars with a yield strength of 497MPa. Closed rectangular 6.35mm diameter hoops with 582MPa yield strength were spaced at 75mm along the length of the column. An axial load of 381kN, or 31.3% of the column concentric capacity, was applied to the column prior to testing. The axial load induced a compressive strain of -0.0489% as calculated using Equations [4-1] through [4-4] as reported in Table 4-14. Column 3NS-C is the control column for all group 3 columns. For tests 1 and 2 on the column there were 3 displacement gauges used. These gauges were located at mid-height, 1/3 height and 1/6 height of the span of the column. The column was subjected to a total of 3 shock tube test: 3NS-C-1, 3NS-C-2, and 3NS-C-3. The column mounted on the shock tube prior to testing is shown in Figure 4-25a. The column was tested using support system Support-B and the load transfer system LTD-B.

4.2.13.1. 3NS-C-1

Test 3NS-C-1 used a driver length of 2743mm and a driver pressure of 72kPa resulting in a reflected pressure of 13.1kPa and a reflected impulse of 135.7kPa-ms over a positive phase of 18.5ms as reported in Table 4-1 and shown in Figure A-25b. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 6.4mm, 5.8mm, and 3.9mm respectively. These displacements occurred 17.4ms, 17.4ms, and 16.5ms after the start of the positive phase and resulted in 2.4mm, 2.4mm, and 2.0mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 0.37°. Displacement values and time histories are presented in Table 4-1 and Figure A-25a for this test.

Nine strain gauges placed at various locations on the internal longitudinal reinforcing steel were recorded during this test and are reported in Table 4-14 and Figure B-13a and Figure B-13b. The magnitude of the tension strain at mid-height recorded by gauge T3 was low for all tests on the

column and should not be considered as reliable. The maximum compression strain in steel occurred in gauge C3 located at the mid-height of the column. The maximum compression strain in gauge C3 was -0.0645% occurring 17.0ms after the start of loading with a residual of -0.0135%. The tension and compression strain on reinforcement at a location close to the supports was recorded in gauges C6 and T6 respectively. For these gauges the maximum strain was 0.0319% and -0.0350% occurring 11.6ms and 16.1ms after the start of loading. The residual strains at these locations were -0.0053% (C6) and 0.0072% (T6).

Following the test there were no signs of significant damage to the column including no crushing of concrete. There was minor distributed cracking at mid-height of the column. All cracks closed to hair line cracks after the test. The post-test column is shown in Figure 4-25b.

4.2.13.2. 3NS-C-2

Test 3NS-C-2 used a driver length of 2743mm and a driver pressure of 243kPa resulting in a reflected pressure of 39.4kPa and a reflected impulse of 378.1kPa-ms over a positive phase of 26.0ms as reported in Table 4-1 and shown in Figure A-25d. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 20.0mm, 18.8mm, and 12.6mm respectively. These displacements occurred 16.3ms, 16.0ms, and 16.4ms after the start of the positive phase and resulted in 6.2mm, 6.2mm, and 5.4mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 1.16°. Displacement values and time histories are presented in Table 4-1 and Figure A-25c for this test.

The maximum compression strain at mid-height was -0.1472% (gauge C3) occurring at 13.5ms after the start of loading with a residual strain of -0.0234%. The tension and compression strain on reinforcement at a location close to the supports was recorded in gauges C6 and T6 respectively. For these gauges the maximum strain was 0.0740% and -0.0774% occurring 8.9ms and 10.4ms after the start of loading. The residual strains at these locations were -0.0042% (C6) and 0.0121% (T6). Tabulated strain values for this test are found in Table 4-14. Strain-time histories for this test are shown in Figure B-13c and Figure B-13d.

Following the test there were no signs of significant damage to the column including no crushing of concrete. The distribute cracking extended to approximately half the column height centered around

the mid-height of the column. There was no sign of cracking at the support locations. All cracks were in the form of closed hair line cracks. The post-test column is shown in Figure 4-25c and Figure 4-25d.

4.2.13.3. 3NS-C-3

Test 3NS-C-3 used a driver length of 2743mm and a driver pressure of 690kPa resulting in a reflected pressure of 78.5kPa and a reflected impulse of 996.6kPa-ms over a positive phase of 26.7ms as reported in Table 4-1 and shown in Figure A-26b. For this test only the displacement at mid-height was measured. The maximum mid-height displacement was 152.0mm occurring 32.0ms after the start of the positive phase. The relative residual displacement at the mid-height was 138.1mm. The maximum support rotation based on the mid-height displacement was 8.73°. Displacement values and time histories are presented in Table 4-1 and Figure A-26a for this test.

The compression steel strain gauge at mid-height (C3) and the tension steel strain gauges near the support (C6) were damaged early in testing and produced no reliable results. The compression steel strain gauge near the support (T6) recorded -0.2067% maximum strain occurring 18.4ms after the start of loading with 0.0859% residual strain. Tabulated strain values for this test are found in Table 4-14. Strain-time histories for this test are shown in Figure B-14.

Following the test significant damage to the column was observed. There were three distinct plastic hinges in the column with the mid-height hinge being more significantly damaged than the support location hinges. The mid-height hinge was characterized by crushing of the concrete and distributed cracking on the tension face. Crushing of the concrete extended beyond cover and into the core between the transverse hoop locations on the column compression side. This crushing depth was largest at the locations between the hoops where it extended to approximately 25mm depth. The crushing was localized to approximately 150mm or two hoop spacings. Compression reinforcement buckled between the transverse reinforcement at mid-height. The distributed tension cracking extended over a length of approximately 800mm and was mostly closed cracking with the exception of the hinge region where concrete spalled from the tension face and damage over one tie spacing accounted for the majority of the column rotation. The column experienced less damage in the plastic hinge region at the supports than at mid-height. There was crushing of the concrete cover only at the interface of the column and the foundation ends and no sign of reinforcement buckling. A crack fan consisting of 3 or 4 significant cracks extended from the compression side of the interface of the column and the foundation and the tension side (loaded side) of the column. The maximum angle of

the tension cracks at the support location was approximately 45°. The post-test column is shown in Figure 4-26a through Figure 4-26d.

4.2.14. Column 3NS-W

Column 3NS-W had a cross section measuring 150x150mm with a concrete strength of 49.5MPa at the time of testing. The column was reinforced with 4-10M longitudinal reinforcing steel bars with a yield strength of 497MPa. Closed rectangular 6.35mm diameter hoops with 582MPa yield strength were spaced at 150mm along the length of the column. The column was retrofitted by confinement with concrete wedges and external hoop prestressing. Further details of the column and retrofit may be found in the previous chapter. An axial load of 422.8kN, or 34.7% of the column concentric capacity, was applied to the column prior to testing. The axial load induced a compressive strain of -0.0542% as calculated using Equations [4-1] through [4-4] as reported in Table 4-15. For tests 1 through 3 on the column there were 3 displacement gauges used and for test 4 only one displacement gauge at mid-height was used. These gauges were located at mid-height, 1/3 height and 1/6 height of the span of the column. Column 3NS-W is the companion column to Columns 3NS-C (group 3 control column), 2NS-J (group 2 non-seismic FRP jacketed) and 2S-J (group 2 seismic FRP jacketed). The column was subjected to a total of 4 shock tube test: 3NS-W-1, 3NS-W-2, 3NS-W-3, and 3NS-W-4. The column mounted on the shock tube prior to testing is shown in Figure 4-27a. The column was tested using support system Support-B and the load transfer system LTD-B.

4.2.14.1. 3NS-W-1

Test 3NS-W-1 used a driver length of 2743mm and a driver pressure of 48kPa resulting in a reflected pressure of 9.2kPa and a reflected impulse of 104.0kPa-ms over a positive phase of 18.4ms as reported in Table 4-1 and shown in Figure A-27b. Prior to the test it was observed that the displacement gauge mount for displacement at 1/3 height was loose and allowed free movement of up to 3mm of the displacement gauge. This could not be fixed as the concrete wedges and prestress system were in the way of access to the displacement gauge mount on the column. All displacement readings from this gauge for all tests on this column should be taken as accurate within 3mm. The loose connection is evident in the displacement-time plots for this test and the proceeding tests (Figure A-27a , Figure A-27c, and Figure A-28a) as the appearance of larger than mid-height displacements and of higher modes of response that are not realistic for the column. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 3.1mm, 4.2mm, and 2.1mm respectively. These displacements occurred 19.9ms, 20.2ms, and 22.3ms after

the start of the positive phase and resulted in 0.7mm, 0.8mm, and 0.5mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 0.18°. Displacement values and time histories are presented in Table 4-1 and Figure A-27a for this test.

Ten strain gauges placed at various locations on the internal longitudinal reinforcing steel and external prestress straps were recorded during this test and are reported in Table 4-15 and Figure B-15a and Figure B-15b. The maximum tension and compression strain in reinforcement steel occurred at mid-height in gauges C3 and T3 respectively. The maximum tension and compression strain was 0.0486% and -0.0369% occurring 17.0ms and 16.7ms after the start of loading with 0.0048% and -0.0053% residual strain for gauges C3 and T3 respectively. The maximum tension and compression strain on reinforcement near the supports was 0.0221% and -0.0104% occurring 17.2ms and 12.7ms after the start of loading with -0.0018% and 0.0038% residual strain for gauges T6 and C6 respectively. Two strain gauges were placed on the external confinement prestress steel straps at different locations on the column. The two gauges both recorded negative strains indicating that there was a loss of prestress during the test. The maximum relative strains for the two gauges were -0.0047% and -0.0058% occurring 6.5ms and 6.4ms after the start of loading with -0.0001% and -0.0002% residual for gauges S1 and S2 respectively. The loss of prestress during the response of the column likely can be correlated with settling of the interaction between the precast concrete wedges and the column surface.

Following the test there were no signs of damage to the column including no crushing of concrete, no visible cracks, no damage to the precast concrete wedges and no damage or obvious relaxing of the prestress straps. The post-test column is shown in Figure 4-27b.

4.2.14.2. 3NS-W-2

Test 3NS-W-2 used a driver length of 2743mm and a driver pressure of 242kPa resulting in a reflected pressure of 38.2kPa and a reflected impulse of 380.2kPa-ms over a positive phase of 20.6ms as reported in Table 4-1 and shown in Figure A-27d. Prior to the test it was observed that the displacement gauge mount for displacement at 1/3 height was loose and allowed free movement of up to 3mm of the displacement gauge. This could not be fixed as the concrete wedges and prestress system were in the way of access to the displacement gauge mount on the column. All displacement readings from this gauge for all tests on this column should be taken as accurate within 3mm. The loose connection is evident in the displacement-time plots for this test, the preceding test and the

proceeding tests (Figure A-27a, Figure A-27c, and Figure A-28a) as the appearance of larger than mid-height displacements and of higher modes of response that are not realistic for the column. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 20.5mm, 21.3mm, and 12.1mm respectively. These displacements occurred 22.2ms, 17.2ms, and 23.2ms after the start of the positive phase and resulted in 8.7mm, 8.3mm, and 7.3mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 1.19°. Displacement values and time histories are presented in Table 4-1 and Figure A-27c for this test.

Nine strain gauges placed at various locations on the internal longitudinal reinforcing steel and external prestress straps were recorded during this test and are reported in Table 4-15 and Figure B-15c and Figure B-15d. The maximum compression strain in reinforcement steel occurred at mid-height in gauges T3. The compression strain was -0.1359% occurring 16.0ms after the start of loading with -0.0198% residual strain for gauge T3. The tension strain gauge on reinforcing steel at mid-height (C3) was damaged early in the test and did not provide reliable data. The maximum tension and compression strain on reinforcement near the supports was 0.0538% and -0.0328% occurring 7.8ms and 7.3ms after the start of loading with 0.0063% and 0.0024% residual strain for gauges T6 and C6 respectively. Two strain gauges were placed on the external confinement prestress steel straps at different locations on the column. The two gauges both recorded negative strains indicating that there was a loss of prestress during the test. The maximum relative strains for the two gauges were -0.0125% and -0.0141% occurring 7.3ms and 7.4ms after the start of loading with -0.0036% and -0.0058% residual for gauges S1 and S2 respectively. The loss of prestress during the response of the column likely can be correlated with settling of the interaction between the precast concrete wedges and the column surface.

Following the test there were only minimal signs of damage to the column in the form of small, closed tension cracking at mid-height on the face of the column in the region visible between the concrete wedges. There were no signs of more significant damage including no crushing of concrete, no damage to the precast concrete wedges and no damage or obvious relaxing of the prestress straps. The post-test column is shown in Figure 4-27c and Figure 4-27d.

4.2.14.3. 3NS-W-3

Test 3NS-W-3 used a driver length of 2743mm and a driver pressure of 522kPa resulting in a reflected pressure of 75.7kPa and a reflected impulse of 739.3kPa-ms over a positive phase of 25.0ms as

reported in Table 4-1 and shown in Figure A-28b. Prior to the test it was observed that the displacement gauge mount for displacement at 1/3 height was loose and allowed free movement of up to 3mm of the displacement gauge. This could not be fixed as the concrete wedges and prestress system were in the way of access to the displacement gauge mount on the column. All displacement readings from this gauge for all tests on this column should be taken as accurate within 3mm. The loose connection is evident in the displacement-time plots for this test and the preceding tests (Figure A-27a, Figure A-27c, and Figure A-28a) as the appearance of larger than mid-height displacements and of higher modes of response that are not realistic for the column. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 63.0mm, 52.2mm, and 29.2mm respectively. These displacements occurred 26.9ms, 29.4ms, and 27.6ms after the start of the positive phase and resulted in 29.5mm, 22.9mm, and 15.5mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 3.64°. Displacement values and time histories are presented in Table 4-1 and Figure A-28a for this test.

Eight strain gauges placed at various locations on the internal longitudinal reinforcing steel and external prestress straps were recorded during this test and are reported in Table 4-15 and Figure B-16a and Figure B-16b. The tension strain gauge on reinforcing steel at mid-height (C3) was damaged early in test 2 and was not monitored. The compression strain gauge on reinforcing steel at mid-height (T3) was damaged early in the test and did not provide reliable data. The maximum tension and compression strain on reinforcement near the supports was 0.2881% and -0.1349% occurring 25.5ms and 29.0ms after the start of loading with 0.0374% and -0.0555% residual strain for gauges T6 and C6 respectively. Two strain gauges were placed on the external confinement prestress steel straps at different locations on the column. The two gauges both initially recorded negative strains followed by positive strains with maximum values occurring at approximately the same time as maximum displacements. This indicates that there was an initial loss of prestress during the test followed by a gain in prestress as the column displaced. The initial loss of prestress during the response of the column likely can be correlated with settling of the interaction between the precast concrete wedges and the column surface. The maximum relative strains for the two gauges were 0.0305% and 0.0675% occurring 24.4ms and 29.1ms after the start of loading with 0.0033% and 0.0446% residual for gauges S1 and S2 respectively.

After the test it was observed that there were several large cracks, up to 2mm wide, in the tension concrete at mid-height of the column. These cracks were visible in the space between the concrete wedges. There was some minor crushing of the compression concrete at mid-height with crushing depth not exceeding concrete cover. The residual displacement was visible. There was no damage to the concrete wedges or any of the prestress straps. The post-test column is shown in Figure 4-28a through Figure 4-28d.

4.2.14.4. 3NS-W-4

Test 3NS-W-4 used a driver length of 2743mm and a driver pressure of 690kPa resulting in a reflected pressure of 83.2kPa and a reflected impulse of 970.7kPa-ms over a positive phase of 26.7ms as reported in Table 4-1 and shown in Figure A-28d. Only the displacement gauge at the column mid-height was used for this test. The maximum mid-height displacement was 122.5mm occurring 39.1ms after the start of the positive phase resulting in 104.9mm residual displacement. The maximum support rotation based on the mid-height displacement was 7.06°. Displacement values and time histories are presented in Table 4-1 and Figure A-28c for this test.

Six strain gauges placed at various locations on the internal longitudinal reinforcing steel and external prestress straps were recorded during this test and are reported in Table 4-15 and Figure B-16c and Figure B-16d. Both of the mid-height reinforcing steel strain gauges were damaged in previous tests and not monitored in this test. Both the tension (T6) and compression (C6) strain gauges at the support location reinforcing steel were damaged early in the test and did not produce reliable data. The maximum recorded tension and compression strain in the reinforcing steel occurred in gauges C2 and T2, respectively, which were located 332.5mm away from the mid-height of the column. As this location is not a point of maximum stress along the length of the column, these gauges are not representative of the maximum steel strains in the column. The maximum recorded tension and compression strain was 0.2699% and -0.1144% occurring 16.8ms and 12.5ms after the start of loading with 0.0232% and -0.0115% residual strain for gauges C2 and T2 respectively. Two strain gauges were placed on the external confinement prestress steel straps at different locations on the column. The two gauges both initially recorded negative strains followed by positive strains with maximum values occurring at approximately the same time as maximum displacements. This indicates that there was an initial loss of prestress during the test followed by a gain in prestress as the column displaced. The initial loss of prestress during the response of the column likely can be correlated with settling of the interaction between the precast concrete wedges and the column

surface. The maximum relative strains for the two gauges were 0.0435% and 0.1917% occurring 39.8ms and 45.1ms after the start of loading with 0.0279% and 0.1625% residual for gauges S1 and S2 respectively.

After the test it was observed that the column was extremely damaged. At mid-height of the column the tension reinforcement ruptured. There was one tension crack that opened at the location of the rupture that extended to the compression reinforcement. This rupture and the tension crack occurred at the location of the gap between the rows of concrete wedges at mid-height. Once the concrete wedges were removed it was observed that there was no distributed tension cracking around the mid-height of the column, all damage was isolated to the location between the concrete wedges. There was concrete crushing on the compression side at mid-height that extended over approximately 100mm in length and did not exceed the concrete cover in depth. There was no reinforcement buckling. A crack fan consisting of 3 or 4 significant cracks extended from the compression side of the interface of the column and the foundation and the tension side (loaded side) of the column. The maximum angle of the tension cracks at the support location was approximately 45°. There was only minimal concrete crushing of the cover concrete at the support location that did not exceed the concrete cover depth.

The concrete wedges on either side of mid-height were bearing on each other on the compression side of the column after the test due to the large support rotation. This likely limited displacement of the column after the reinforcement ruptured. There was no damage to any of the concrete wedges after the test other than a small amount of concrete crushing at the location where the two mid-height wedges came in contact. There was no damage to any of the prestress straps at any location on the column. The post-test column is shown in Figure 4-29a through Figure 4-29d.

4.2.15. Column 3NS-CB

Column 3NS-CB had a cross section measuring 150x150mm with a concrete strength of 49.5MPa at the time of testing. The column was reinforced with 4-10M longitudinal reinforcing steel bars with a yield strength of 497MPa. Closed rectangular 6.35mm diameter hoops with 582MPa yield strength were spaced at 150mm along the length of the column. The column was retrofitted by confinement with concrete wedges and external hoop prestressing along with an external bracing system designed to work as a compression member. Further details of the column and retrofit may be found in the previous chapter. An axial load of 422.8kN, or 34.7% of the column concentric capacity, was applied to the column prior to testing. The axial load induced a compressive strain of -0.0542% as calculated

using Equations [4-1] through [4-4] as reported in Table 4-16. For all tests on the column there were 3 displacement gauges used. These gauges were located at mid-height, 1/3 height and 1/6 height of the span of the column. Column 3NS-CB is the companion column to Columns 3NS-C (group 3 control column), 3NS-TB1 (group 3 non-seismic tension brace column 1), 3NS-TB2 (group 3 non-seismic tension brace column 2), and 3NS-TB3 (group 3 non-seismic tension brace column 3). The column was subjected to a total of 5 shock tube test: 3NS-CB-1, 3NS-CB-2, 3NS-CB-3, 3NS-CB-4 and 3NS-CB-5. The column mounted on the shock tube prior to testing is shown in Figure 4-30a. The column was tested using support system Support-B and the load transfer system LTD-B.

4.2.15.1. 3NS-CB-1

Test 3NS-CB-1 used a driver length of 2743mm and a driver pressure of 69kPa resulting in a reflected pressure of 11.9kPa and a reflected impulse of 141.1kPa-ms over a positive phase of 19.5ms as reported in Table 4-1 and shown in Figure A-29b. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 4.1mm, 3.7mm, and 2.6mm respectively. These displacements occurred 19.6ms, 20.1ms, and 21.7ms after the start of the positive phase and resulted in 0.3mm, 0.2mm, and 0.1mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 0.24°. Displacement values and time histories are presented in Table 4-1 and Figure A-29a for this test.

Nine strain gauges placed at various locations on the internal longitudinal steel, external prestress straps and external vertical components of the compression brace were recorded during this test and are reported in Table 4-16 and Figure B-17a and Figure B-17b. The maximum tension and compression strain in the internal reinforcement occurred at mid-height in gauges C3 and T3 respectively. For these gauges the maximum strain was 0.0611% and -0.0364% occurring 18.9ms and 19.0ms after the start of loading. The residual strains at these locations were -0.0004% (C3) and -0.0068% (T3). At the location of the brace point the strains in the reinforcing steel were nearly as high as those at mid-height. The maximum tension and compression strains at the brace point (1/3 height) were 0.0412% and -0.0331% occurring 19.6ms and 20.0ms after the start of loading with residual strains of -0.0014% and -0.0065% for gauges C2 and T2 respectively. Gauges S1 and S2 measured strain in the external prestress straps at two locations on the column. The strains in these gauges were 0.0008% and -0.0018% occurring 26.2ms and 19.3ms after the start of loading with residual strains of -0.0004% and -0.0044% for gauges S1 and S2 respectively. These two gauges both

recorded negative strains indicating a loss of prestress in the straps. The loss of prestress during the response of the column likely can be correlated with settling of the interaction between the precast concrete wedges and the column surface. The strain in the vertical component of the compression brace was measured by gauge VB1 for this test. For this test and all proceeding tests VB1 recorded strains that indicated tension in the brace. This likely indicates that the gauge was not working properly and reported incorrect strains. In tests 3, 4 and 5 a second gauge (VB2) was installed on the vertical component of the compression brace and provided more reliable data.

Following the test there were no signs of damage to the column including no crushing of concrete, no visible cracking and no visible residual displacement. The post-test column is shown in Figure 4-30b.

4.2.15.2. 3NS-CB-2

Test 3NS-CB-2 used a driver length of 2743mm and a driver pressure of 242kPa resulting in a reflected pressure of 36.3kPa and a reflected impulse of 381.3kPa-ms over a positive phase of 23.3ms as reported in Table 4-1 and shown in Figure A-29d. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 17.1mm, 15.4mm, and 10.6mm respectively. These displacements occurred 19.8ms, 21.1ms, and 20.9ms after the start of the positive phase and resulted in 0.3mm, 0.2mm, and 0.1mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 0.99°. Displacement values and time histories are presented in Table 4-1 and Figure A-29c for this test.

The tension strain gauge on the internal reinforcement at mid-height (gauge C3) was damaged early in the test and did not produce reliable data. The maximum compression strain in the internal reinforcement at mid-height (T3) was -0.1087% occurring 21.5ms after the start of loading with residual strain of -0.0162. The maximum tension and compression strains at the brace point (1/3 height) were 0.2325% and -0.1038% occurring 21.0ms and 21.0ms after the start of loading with residual strains of 0.0123% and -0.0209% for gauges C2 and T2 respectively. Gauges S1 and S2 measured strain in the external prestress straps at two locations on the column. The strains in these gauges were 0.0141% and -0.0234% occurring 44.0ms and 64.8ms after the start of loading with residual strains of -0.0021% and -0.0146% for gauges S1 and S2 respectively. The strain in the vertical component of the compression brace was measured by gauge VB1 for this test. For this test, the preceding test and all proceeding tests VB1 recorded strains that indicated tension in the brace. This

likely indicates that the gauge was not working properly and reported incorrect strains. In tests 3, 4 and 5 a second gauge (VB2) was installed on the vertical component of the compression brace and provided more reliable data. Tabulated strain values are reported in Table 4-16 and strain-time histories are shown in Figure B-17c and Figure B-17d.

Following the test there were no signs of damage to the column including no crushing of concrete, no visible cracking and no visible residual displacement. The post-test column is shown in Figure 4-30c and Figure 4-30d.

4.2.15.3. 3NS-CB-3

Test 3NS-CB-3 used a driver length of 2743mm and a driver pressure of 552kPa resulting in a reflected pressure of 70.6kPa and a reflected impulse of 780.4kPa-ms over a positive phase of 27.0ms as reported in Table 4-1 and shown in Figure A-30b. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 50.0mm, 40.8mm, and 25.1mm respectively. These displacements occurred 25.7ms, 25.0ms, and 25.8ms after the start of the positive phase and resulted in 13.2mm, 12.1mm, and 10.4mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 2.89°. Displacement values and time histories are presented in Table 4-1 and Figure A-30a for this test.

The maximum compression strain in the internal reinforcement at mid-height (T3) was -0.4111% occurring 33.6ms after the start of loading with residual strain of -0.2687. The maximum tension and compression strains at the brace point (1/3 height) were 0.3245% and -0.1239% occurring 15.0ms and 15.0ms after the start of loading with residual strains of 0.0009% and -0.0060% for gauges C2 and T2 respectively. Gauges S1 and S2 measured strain in the external prestress straps at two locations on the column. The strains in these gauges were -0.0020% and -0.0176% occurring 20.0ms and 26.1ms after the start of loading with residual strains of -0.0031% and -0.0113% for gauges S1 and S2 respectively. One of the strain gauges placed on the vertical component of the compression brace (VB1) for this test was likely damaged and likely did not record reliable data. For this test, all preceding tests and all proceeding tests VB1 recorded strains that indicated tension in the brace. This likely indicates that the gauge was not working properly and reported incorrect strains. Strain gauge VB2 was placed at the same location (mid-height on the vertical component of the compression brace) for this test. The maximum strain in gauge VB2 was -0.1116% occurring 25.5ms after the start of loading with a residual strain of -0.0556%. As this gauge was not monitored during the first two

tests, there are no cumulative residual strains reported in the tabulated values. Tabulated strain values are reported in Table 4-16 and strain-time histories are shown in Figure B-18a and Figure B-18b.

Following the test there were minimal signs of damage to the column in the form of closed tension cracks that were visible at the mid-height location of the column between the external concrete wedges. There was no crushing of concrete and no visible residual displacement. The post-test column is shown in Figure 4-31a and Figure 4-31b.

4.2.15.4. 3NS-CB-4

Test 3NS-CB-4 used a driver length of 2743mm and a driver pressure of 692kPa resulting in a reflected pressure of 84.0kPa and a reflected impulse of 983.3kPa-ms over a positive phase of 27.4ms as reported in Table 4-1 and shown in Figure A-30d. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 72.5mm, 56.6mm, and 33.1mm respectively. These displacements occurred 26.9ms, 26.0ms, and 26.8ms after the start of the positive phase and resulted in 20.1mm, 17.3mm, and 13.3mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 4.19°. Displacement values and time histories are presented in Table 4-1 and Figure A-30c for this test.

The maximum compression strain in the internal reinforcement at mid-height (T3) was -0.5398% occurring 38.7ms after the start of loading with residual strain of -0.3955. The maximum tension and compression strains at the brace point (1/3 height) were 0.3225% and -0.1115% occurring 23.4ms and 13.1ms after the start of loading with residual strains of 0.0048% and -0.0078% for gauges C2 and T2 respectively. Gauges S1 and S2 measured strain in the external prestress straps at two locations on the column. The strains in these gauges were 0.0182% and 0.0511% occurring 11.6ms and 28.3ms after the start of loading with residual strains of 0.0125% and -0.0015% for gauges S1 and S2 respectively. Both of the strain gauges on the prestress straps measured maximum strains in tension indicating that the prestress jacketing was activated and provided additional confinement pressure during response. One of the strain gauges placed on the vertical component of the compression brace (VB1) for this test was likely damaged and likely did not record reliable data. For this test and all preceding tests, VB1 recorded strains that indicated tension in the brace. This likely indicates that the gauge was not working properly and reported incorrect strains. Strain gauge VB2 was placed at the same location (mid-height on the vertical component of the compression brace)

for this test. The maximum strain in gauge VB2 was -0.1951% occurring 23.8ms after the start of loading with a residual strain of -0.0545%. As this gauge was not monitored during the first two tests, there are no cumulative residual strains reported in the tabulated values. Tabulated strain values are reported in Table 4-16 and strain-time histories are shown in Figure B-18c and Figure B-18d.

Following the test there were signs of damage to the column in the form of tension cracks up to 1mm wide that were visible at the mid-height location of the column between the external concrete wedges and visible residual displacement. There was no crushing of concrete visible at any of the exposed locations on the column. The post-test column is shown in Figure 4-31c and Figure 4-31d.

4.2.15.5. 3NS-CB-5

Test 3NS-CB-5 used a driver length of 4877mm and a driver pressure of 698kPa resulting in a reflected pressure of 84.4kPa and a reflected impulse of 1669.8kPa-ms over a positive phase of 59.1ms as reported in Table 4-1 and shown in Figure A-31b. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 103.3mm, 77.8mm, and 43.1mm respectively. These displacements occurred 36.7ms, 35.8ms, and 39.3ms after the start of the positive phase and resulted in 44.5mm, 34.0mm, and 21.2mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 5.96°. Displacement values and time histories are presented in Table 4-1 and Figure A-31a for this test.

The maximum compression strain in the internal reinforcement at mid-height (T3) was -0.5773% occurring 58.0ms after the start of loading with residual strain of -0.2170. The maximum tension and compression strains at the brace point (1/3 height) were 0.4041% and -0.0901% occurring 33.7ms and 11.1ms after the start of loading with residual strains of 0.0146% and 0.1028% for gauges C2 and T2 respectively. Gauges S1 and S2 measured strain in the external prestress straps at two locations on the column. Strain gauge S1 was damaged early in the test and did not produce reliable data. The strain in gauge S2 was 0.1125% occurring 11.9ms after the start of loading with residual strain of 0.0011%. Gauge S2 on the prestress strap measured maximum strain in tension indicating that the prestress jacketing was activated and provided additional confinement pressure during response. One of the strain gauges placed on the vertical component of the compression brace (VB1) for this test was likely damaged and likely did not record reliable data. For this test and all preceding tests, VB1 recorded strains that indicated tension in the brace. This likely indicates that the gauge was not working properly and reported incorrect strains. Strain gauge VB2 was placed at the same location

(mid-height on the vertical component of the compression brace) for this test. The maximum strain in gauge VB2 was -0.5096% occurring 36.1ms after the start of loading with a residual strain of -0.3212%. As this gauge was not monitored during the first two tests, there are no cumulative residual strains reported in the tabulated values. Tabulated strain values are reported in Table 4-16 and strain-time histories are shown in Figure B-19a and Figure B-19b.

Following the test there were significant signs of damage to the column in the form of a single tension crack approximately 5mm wide that were visible at the mid-height location of the column between the external concrete wedges and visible residual displacement. There was some minor crushing of the concrete cover visible on the compression face of the column at mid-height. There was no sign visible damage at the support regions. None of the prestress straps were damaged and none of the concrete wedges were damaged. There was no rupture of the longitudinal reinforcement or buckling of the compression reinforcement. The post-test column is shown in Figure 4-32a through Figure 4-32d.

4.2.16. Column 3NS-TB1

Column 3NS-TB1 had a cross section measuring 150x150mm with a concrete strength of 49.5MPa at the time of testing. The column was reinforced with 4-10M longitudinal reinforcing steel bars with a yield strength of 497MPa. Closed rectangular 6.35mm diameter hoops with 582MPa yield strength were spaced at 150mm along the length of the column. The column was retrofitted with an external bracing system designed to work as a tension member. This column differed from column 3NS-TB2 by the geometry of the tension brace as column 3NS-TB2 had the interior bearing points spaced an additional 127mm from the unloaded column face. 3NS-TB1 differed from column 3NS-TB3 by the level of the initial axial load. Further details of the column and retrofit may be found in the previous chapter. An axial load of 422.8kN, or 34.7% of the column concentric capacity, was applied to the column prior to testing. The axial load induced a compressive strain of -0.0542% as calculated using Equations [4-1] through [4-4] as reported in Table 4-17. For all tests on the column there were 3 displacement gauges used. These gauges were located at mid-height, 1/3 height and 1/6 height of the span of the column. Column 3NS-TB1 is the companion column to Columns 3NS-C (group 3 control column), 3NS-CB (group 3 non-seismic compression brace), 3NS-TB2 (group 3 non-seismic tension brace column 2), and 3NS-TB3 (group 3 non-seismic tension brace column 3). The column was subjected to a total of 5 shock tube test: 3NS-TB1-1, 3NS-TB1-2, 3NS-TB1-3, 3NS-TB1-4 and 3NS-

TB1-5. The column mounted on the shock tube prior to testing is shown in Figure 4-33a. The column was tested using support system Support-B and the load transfer system LTD-B.

4.2.16.1. 3NS-TB1-1

Test 3NS-TB1-1 used a driver length of 2743mm and a driver pressure of 72kPa resulting in a reflected pressure of 11.2kPa and a reflected impulse of 155.0kPa-ms over a positive phase of 20.5ms as reported in Table 4-1 and shown in Figure A-32b. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 6.3mm, 5.0mm, and 2.8mm respectively. These displacements occurred 21.2ms, 21.5ms, and 21.9ms after the start of the positive phase and resulted in 0.3mm, 0.4mm, and 0.3mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 0.37°. Displacement values and time histories are presented in Table 4-1 and Figure A-32a for this test.

Nine strain gauges placed at various locations on the internal longitudinal steel were recorded during this test and are reported in Table 4-17 and Figure B-20a and Figure B-20b. The maximum tension and compression strain in the internal reinforcement occurred at mid-height in gauges T3A and C3 respectively. For these gauges the maximum strain was 0.0954% and -0.0616% occurring 18.5ms and 18.6ms after the start of loading. The residual strains at these locations were 0.0111% (T3A) and -0.0094% (C3). A second tension strain gauge (T3B) at mid-height was monitored for this test. For T3B the maximum strain was 0.0914% occurring 18.6ms after the start of loading with 0.0049% residual strain. At the location of the brace point the strains in the reinforcing steel were nearly as high as those at mid-height. The maximum tension and compression strains at the brace point (1/3 height) were 0.0504% and -0.0546% occurring 19.1ms and 19.8ms after the start of loading with residual strains of 0.0106% and -0.0083% for gauges T2 and C2 respectively.

Following the test there were no signs of damage to the column including no crushing of concrete, no visible cracking and no visible residual displacement and the tension brace rods were still tight. The post-test column is shown in Figure 4-33b.

4.2.16.2. 3NS-TB1-2

Test 3NS-TB1-2 used a driver length of 2743mm and a driver pressure of 212kPa resulting in a reflected pressure of 34.8kPa and a reflected impulse of 375.9kPa-ms over a positive phase of 22.9ms as reported in Table 4-1 and shown in Figure A-32d. The maximum mid-height displacement, 1/3

height displacement, and 1/6 height displacement were 24.4mm, 21.3mm, and 12.9mm respectively. These displacements occurred 19.2ms, 18.7ms, and 17.5ms after the start of the positive phase and resulted in 2.6mm, 2.4mm, and 1.8mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 1.41°. Displacement values and time histories are presented in Table 4-1 and Figure A-32c for this test.

Nine strain gauges placed at various locations on the internal longitudinal steel were recorded during this test and are reported in Table 4-17 and Figure B-20c and Figure B-20d. The maximum tension and compression strain in the internal reinforcement occurred at mid-height in gauges T3B and C3 respectively. For these gauges the maximum strain was 0.3001% and -0.1058% occurring 15.8ms and 14.8ms after the start of loading. The residual strains at these locations were 0.0164% (T3B) and -0.0082% (C3). A second tension strain gauge (T3A) at mid-height was monitored for this test. For T3A the maximum strain was 0.2647% occurring 15.9ms after the start of loading with 0.0148% residual strain. At the location of the brace point the strains in the reinforcing steel were nearly as high as those at mid-height. The maximum tension and compression strains at the brace point (1/3 height) were 0.2543% and -0.1176% occurring 17.8ms and 16.7ms after the start of loading with residual strains of 0.0216% and 0.0014% for gauges T2 and C2 respectively.

Following the test there were minimal signs of damage to the column in the form of closed cracking at the mid-height and the supports. There were two small tension cracks at the mid-height location and two small tension cracks at each of the column-foundation locations. There was no other damage to the column including no crushing of concrete and no visible residual displacement and the tension brace rods were still tight. The post-test column is shown in Figure 4-33c and Figure 4-33d.

4.2.16.3. 3NS-TB1-3

Test 3NS-TB1-3 used a driver length of 2743mm and a driver pressure of 556kPa resulting in a reflected pressure of 73.8kPa and a reflected impulse of 888.1kPa-ms over a positive phase of 28.4ms as reported in Table 4-1 and shown in Figure A-33b. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 55.1mm, 42.3mm, and 24.7mm respectively. These displacements occurred 21.5ms, 19.5ms, and 17.7ms after the start of the positive phase and resulted in 5.7mm, 4.8mm, and 3.5mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height

displacement was 3.19°. Displacement values and time histories are presented in Table 4-1 and Figure A-33a for this test.

Seven strain gauges placed at various locations on the internal longitudinal steel were recorded during this test and are reported in Table 4-17 and Figure B-21a and Figure B-21b. The compression (C3) gauge at mid-height and one of the two tension gauges (T3A) were damaged early during the response and did not provide reliable data. The maximum tension strain in the internal reinforcement occurred at mid-height in gauge T3B. For gauge T3B, the maximum strain was 0.6361% occurring 10.2ms after the start of loading with 0.0812% residual strain. At the location of the brace point the strains in the reinforcing steel were nearly as high as those at mid-height. The maximum tension and compression strains at the brace point (1/3 height) were 0.2712% and -0.1460% occurring 10.2ms and 18.3ms after the start of loading with residual strains of 0.0099% and -0.0153% for gauges T2 and C2 respectively.

Following the test there was signs of damage to the column in the form of closed cracking at the mid-height and the supports and crushing of the cover concrete at mid-height over a length of approximately 150mm. The crushing depth was approximately 10mm. There was no damage to the column at the support locations including no significant tension cracks and no concrete crushing. The tension brace system was undamaged and the tension rods were still tight. The post-test column is shown in Figure 4-34a and Figure 4-34b.

4.2.16.4. 3NS-TB1-4

Test 3NS-TB1-4 used a driver length of 2743mm and a driver pressure of 691Pa resulting in a reflected pressure of 82.6kPa and a reflected impulse of 982.6kPa-ms over a positive phase of 27.8ms as reported in Table 4-1 and shown in Figure A-33d. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 72.6mm, 55.0mm, and 34.5mm respectively. These displacements occurred 23.9ms, 20.4ms, and 19.8ms after the start of the positive phase and resulted in 14.4mm, 4.4mm, and 6.8mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 4.19°. Displacement values and time histories are presented in Table 4-1 and Figure A-33c for this test.

Six strain gauges placed at various locations on the internal longitudinal steel were recorded during this test and are reported in Table 4-17 and Figure B-21c and Figure B-21d. The remaining tension

strain gauge in the longitudinal reinforcement at mid-height (T3B) was damaged early in the test and did not provide reliable data. The maximum tension and compression strains at the brace point (1/3 height) were 0.2330% and -0.1378% occurring 10.9ms and 10.5ms after the start of loading with residual strains of -0.0025% and 0.0129% for gauges T2 and C2 respectively.

Following the test there was little change in the state of the column from test 3. The crushing of concrete cover over approximately 150mm was not more significant. There was more distributed cracking around the mid-height of the column with closed cracks spaced at approximately 75mm extending over a length of approximately 500mm centred around mid-height. There was no damage to the column at the support locations including no significant tension cracks and no concrete crushing. The tension brace system was undamaged and the tension rods were still tight. The post-test column is shown in Figure 4-34c and Figure 4-34d.

4.2.16.5. 3NS-TB1-5

Test 3NS-TB1-5 used a driver length of 4877mm and a driver pressure of 693Pa resulting in a reflected pressure of 79.0kPa and a reflected impulse of 1610.2kPa-ms over a positive phase of 60.4ms as reported in Table 4-1 and shown in Figure A-34b. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 75.8mm, 57.0mm, and 34.7mm respectively. These displacements occurred 24.7ms, 21.3ms, and 19.9ms after the start of the positive phase and resulted in 12.2mm, 8.8mm, and 4.7mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 4.38°. Displacement values and time histories are presented in Table 4-1 and Figure A-34a for this test.

Six strain gauges placed at various locations on the internal longitudinal steel were recorded during this test and are reported in Table 4-17 and Figure B-22. The maximum tension and compression strains at the brace point (1/3 height) were 0.1925% and -0.1214% occurring 16.4ms and 13.4ms after the start of loading with residual strains of -0.0406% and 0.0869% for gauges T2 and C2 respectively.

Following the test there was an increase in the damage observed in test 4. The concrete crushing at mid-height extended into the core to an approximate depth of 20mm with loss of the concrete cover on the sides of the column up to the section mid-depth over approximately 100mm in length centred around mid-height of the column. The longitudinal reinforcement did not buckle or rupture at any

location in the column. There was no crushing of concrete at the support location and only minor closed tension cracks. There was no sign of damage to the tension brace system and all tension rods were tight after the test. It was observed that the mounting bolts for the displacement gauges were bent by 5mm to 10mm after the test. The 8mm diameter anchor bolts for mounting the gauges were installed such that approximately 75mm of unsupported length existed between the displacement gauge connection and the column surface. The bending of these bolts was likely due to the large inertia forces associated with the acceleration of the column at the gauge connection points and the mass of the displacement gauges. The mounting bolts bending could have resulted in a 5mm to 10mm overestimation of maximum and residual displacement. This error was likely not isolated to test 5, rather it should be averaged over tests 3, 4, and 5 where the most significant column displacements were seen. These large column displacements would have resulted in large accelerations at the gauge points and therefore large inertia forces that could have caused permanent damage to the mounting bolts. Therefore, it would be appropriate to assume that the displacement records for tests 3, 4, and 5 are accurate to within 3mm each with the experimental time-histories erring on the side of larger displacements. The post-test column is shown in Figure 4-35a through Figure 4-35d.

4.2.17. Column 3NS-TB2

Column 3NS-TB2 had a cross section measuring 150x150mm with a concrete strength of 49.5MPa at the time of testing. The column was reinforced with 4-10M longitudinal reinforcing steel bars with a yield strength of 497MPa. Closed rectangular 6.35mm diameter hoops with 582MPa yield strength were spaced at 150mm along the length of the column. The column was retrofitted with an external bracing system designed to work as a tension member. This column differed from column 3NS-TB1 by the geometry of the tension brace as column 3NS-TB2 had the interior bearing points spaced an additional 127mm from the unloaded column face. 3NS-TB2 differed from column 3NS-TB3 by the level of the initial axial load and the spacing of the brace points further from the surface of the column. Further details of the column and retrofit may be found in the previous chapter. An axial load of 422.8kN, or 34.7% of the column concentric capacity, was applied to the column prior to testing. The axial load induced a compressive strain of -0.0542% as calculated using Equations [4-1] through [4-4] as reported in Table 4-18. For all tests on the column there were 3 displacement gauges used. These gauges were located at mid-height, approximately 1/3 height and 1/6 height of the span of the column. Due to the change in geometry of the tension brace, the bearing points in the column were not at 1/3 of the column height locations. These bearing points were centered 397mm above

and below the mid-height of the column, which is 62mm closer to the supports than in columns 3NS-TB1 and 3NS-TB3. The displacement gauge at the brace point ($d_{1/3}$) was also located 397mm below the mid-height of the column. For simplicity, this displacement gauge is referred to using the same name ($d_{1/3}$) as the other columns. Column 3NS-TB2 is the companion column to Columns 3NS-C (group 3 control column), 3NS-CB (group 3 non-seismic compression brace), 3NS-TB1 (group 3 non-seismic tension brace column 1), and 3NS-TB3 (group 3 non-seismic tension brace column 3). The column was subjected to a total of 5 shock tube test: 3NS-TB2-1, 3NS-TB2-2, 3NS-TB2-3, 3NS-TB2-4 and 3NS-TB2-5. The column mounted on the shock tube prior to testing is shown in Figure 4-36a. The column was tested using support system Support-B and the load transfer system LTD-B.

4.2.17.1. 3NS-TB2-1

Test 3NS-TB2-1 used a driver length of 2743mm and a driver pressure of 71kPa resulting in a reflected pressure of 13.5kPa and a reflected impulse of 151.4kPa-ms over a positive phase of 20.2ms as reported in Table 4-1 and shown in Figure A-35b. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 1.5mm, 1.8mm, and 1.2mm respectively. These displacements occurred 18.3ms, 16.0ms, and 16.0ms after the start of the positive phase and resulted in -0.2mm, 0.1mm, and 0.1mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 0.09°. Displacement values and time histories are presented in Table 4-1 and Figure A-35a for this test.

Only two of the strain gauges on the internal steel reinforcement were intact after installation of the column. Strain gauge C5 was located 665mm above the mid-height of the column on the loaded face. Maximum strain in C5 was -0.0128% occurring 23.7ms after the start of loading with -0.0013% residual strain. Strain gauge T3 was located at mid-height on the unloaded face of the column. Maximum strain in gauge T3 was 0.0137% occurring 11.3ms after the start of loading with -0.0074% residual strain. Tabulated values and strain-time histories are reported in Table 4-18 and Figure B-23a.

Following the test there were no signs of damage to the column including no crushing of concrete, no visible cracking and no visible residual displacement and the tension brace rods were still tight. The post-test column is shown in Figure 4-36b.

4.2.17.2. 3NS-TB2-2

Test 3NS-TB2-2 used a driver length of 2743mm and a driver pressure of 243kPa resulting in a reflected pressure of 34.6kPa and a reflected impulse of 447.0kPa-ms over a positive phase of 23.5ms as reported in Table 4-1 and shown in Figure A-35d. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 21.1mm, 18.2mm, and 12.1mm respectively. These displacements occurred 18.5ms, 19.1ms, and 18.9ms after the start of the positive phase and resulted in 6.2mm, 5.0mm, and 4.8mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 1.22°. Displacement values and time histories are presented in Table 4-1 and Figure A-35c for this test.

Only two of the strain gauges on the internal steel reinforcement were intact after installation of the column. Strain gauge C5 was located 665mm above the mid-height of the column on the loaded face. Maximum strain in C5 was -0.0404% occurring 16.5ms after the start of loading with 0.0083% residual strain. Strain gauge T3 was located at mid-height on the unloaded face of the column. Maximum strain in gauge T3 was 0.2343% occurring 17.0ms after the start of loading with 0.0097% residual strain. Tabulated values and strain-time histories are reported in Table 4-18 and Figure B-23b.

Following the test there was only minimal signs of damage in the form of two closed cracks at mid-height. There was no further damage including no crushing of concrete, no visible residual displacement and the tension brace rods were still tight. The post-test column is shown in Figure 4-36c and Figure 4-36d.

4.2.17.3. 3NS-TB2-3

Test 3NS-TB2-3 used a driver length of 2743mm and a driver pressure of 551kPa resulting in a reflected pressure of 68.4kPa and a reflected impulse of 812.8kPa-ms over a positive phase of 25.7ms as reported in Table 4-1 and shown in Figure A-36b. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 44.0mm, 34.0mm, and 23.4mm respectively. These displacements occurred 19.5ms, 19.0ms, and 17.5ms after the start of the positive phase and resulted in 16.8mm, 15.7mm, and 9.6mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 2.55°. Displacement values and time histories are presented in Table 4-1 and Figure A-36a for this test.

Strain gauge C5 was located 665mm above the mid-height of the column on the loaded face. Maximum strain in C5 was -0.1302% occurring 17.9ms after the start of loading with -0.0007% residual strain. Strain gauge T3 was damaged early in the response and did not provide reliable data. Tabulated values and strain-time histories are reported in Table 4-18 and Figure B-23c.

After the test it was observed that there was minor crushing of the concrete cover on the compression side of the column at mid-height. The crushing extended over approximately 75mm length centred around mid-height. There was new closed tension cracking at mid-height with distributed cracking extending over approximately 400mm spaced at approximately 75mm. There was no crushing or cracking at the support locations of the column. There was no damage to the tension brace system and the rods were still tight. The post-test column is shown in Figure 4-37a and Figure 4-37b.

4.2.17.4. 3NS-TB2-4

Test 3NS-TB2-4 used a driver length of 2743mm and a driver pressure of 686kPa resulting in a reflected pressure of 80.3kPa and a reflected impulse of 956.3kPa-ms over a positive phase of 27.7ms as reported in Table 4-1 and shown in Figure A-36d. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 51.6mm, 38.0mm, and 29.0mm respectively. These displacements occurred 20.3ms, 19.1ms, and 18.3ms after the start of the positive phase and resulted in 13.6mm, 10.6mm, and 5.0mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 2.99°. Displacement values and time histories are presented in Table 4-1 and Figure A-36c for this test.

Strain gauge C5 was located 665mm above the mid-height of the column on the loaded face. Maximum strain in C5 was -0.1523% occurring 16.3ms after the start of loading with -0.0001% residual strain. Tabulated values and strain-time histories are reported in Table 4-18 and Figure B-23d.

After the test it was observed that the crushing of the concrete cover on the compression side of the column at mid-height extended over a length of approximately 150mm to 200mm and extended to the sides of the column up to half the section depth. There was no new cracking of the column however, the mid-height cracks were no longer completely closed with up to 1mm crack width. There was no crushing or cracking at the support locations of the column. There was no damage to the

tension brace system and the rods were still tight. Some of the concrete on the foundations was cracked and slightly damaged around the location of the mounting holes for attaching the column to the shock tube and supports. This likely did not have an effect on the test. The post-test column is shown in Figure 4-37c and Figure 4-37d.

4.2.17.5. 3NS-TB2-5

Test 3NS-TB2-5 used a driver length of 4877mm and a driver pressure of 690kPa resulting in a reflected pressure of 83.9kPa and a reflected impulse of 1894.0kPa-ms over a positive phase of 60.4ms as reported in Table 4-1 and shown in Figure A-37b. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 47.8mm, 35.7.0mm, and 29.1mm respectively. These displacements occurred 21.2ms, 19.1ms, and 18.7ms after the start of the positive phase and resulted in 11.2mm, 10.8mm, and 7.6mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 2.76°. Displacement values and time histories are presented in Table 4-1 and Figure A-37a for this test.

Strain gauge C5 was located 665mm above the mid-height of the column on the loaded face. Maximum strain in C5 was -0.1959% occurring 22.1ms after the start of loading with -0.0004% residual strain. Tabulated values and strain-time histories are reported in Table 4-18 and Figure B-24.

After the test it was observed that the crushing of the concrete cover on the compression side of the column at mid-height extended over a length of approximately 250mm and extended to the sides of the column up to half the section depth. On the compression face of the column the crushing depth extended into the core of the column with a maximum depth of approximately 20mm. There were new cracks on the tension face of the column in the mid-height region. Cracking was spaced at approximately the internal transverse reinforcement spacing (75mm) extending over approximately 600mm in length. There was no crushing or cracking at the support locations of the column. There was no damage to the tension brace system and the rods were still tight. There was more damage to the concrete foundations including some more cracks and spalling of concrete at one corner. This likely did not have an effect on the test. It was observed that the mounting bolts for the displacement gauges were bent by 5mm to 10mm after the test. The 8mm diameter anchor bolts for mounting the gauges were installed such that approximately 75mm of unsupported length existed between the displacement gauge connection and the column surface. The bending of these bolts was likely due to the large inertia forces associated with the acceleration of the column at the gauge connection points

and the mass of the displacement gauges. The mounting bolts bending could have resulted in a 5mm to 10mm overestimation of maximum and residual displacement. This error was likely not isolated to test 5, rather it should be averaged over tests 3, 4, and 5 where the most significant column displacements were seen. These large column displacements would have resulted in large accelerations at the gauge points and therefore large inertia forces that could have caused permanent damage to the mounting bolts. Therefore, it would be appropriate to assume that the displacement records for tests 3, 4, and 5 are accurate to within 3mm each with the experimental time-histories erring on the side of larger displacements. The post-test column is shown in Figure 4-38a though Figure 4-38d.

4.2.18. Column 3NS-TB3

Column 3NS-TB3 had a cross section measuring 150x150mm with a concrete strength of 49.5MPa at the time of testing. The column was reinforced with 4-10M longitudinal reinforcing steel bars with a yield strength of 497MPa. Closed rectangular 6.35mm diameter hoops with 582MPa yield strength were spaced at 150mm along the length of the column. The column was retrofitted with an external bracing system designed to work as a tension member. This column differed from column 3NS-TB2 by the geometry of the tension brace as column 3NS-TB2 had the interior bearing points spaced an additional 127mm from the unloaded column face and by the level of the initial axial load. 3NS-TB31 differed from column 3NS-TB1 by the level of the initial axial load. Further details of the column and retrofit may be found in the previous chapter. An axial load of 141.0kN, or 11.6% of the column concentric capacity, was applied to the column prior to testing. The axial load induced a compressive strain of -0.0181% as calculated using Equations [4-1] through [4-4] as reported in Table 4-19. For all tests on the column there were 3 displacement gauges used. These gauges were located at mid-height, 1/3 height and 1/6 height of the span of the column. Column 3NS-TB3 is the companion column to Columns 3NS-C (group 3 control column), 3NS-CB (group 3 non-seismic compression brace), 3NS-TB1 (group 3 non-seismic tension brace column 1), and 3NS-TB2 (group 3 non-seismic tension brace column 2). The column was subjected to a total of 5 shock tube test: 3NS-TB3-1, 3NS-TB3-2, 3NS-TB3-3, 3NS-TB3-4 and 3NS-TB3-5. The column mounted on the shock tube prior to testing is shown in Figure 4-39a. The column was tested using support system Support-B and the load transfer system LTD-B.

4.2.18.1. 3NS-TB3-1

Test 3NS-TB3-1 used a driver length of 2743mm and a driver pressure of 72kPa resulting in a reflected pressure of 11.2kPa and a reflected impulse of 146.0kPa-ms over a positive phase of 21.0ms as reported in Table 4-1 and shown in Figure A-38b. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 4.5mm, 4.0mm, and 2.5mm respectively. These displacements occurred 19.3ms, 21.4ms, and 22.9ms after the start of the positive phase and resulted in 0.9mm, 0.5mm, and 0.7mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 0.26°. Displacement values and time histories are presented in Table 4-1 and Figure A-38a for this test.

Eight strain gauges at various locations on the internal reinforcement were monitored during this test. The maximum tension and compression strain occurred at mid-height in gauges T3A and C3, respectively. Maximum strains of 0.0326% and -0.0381% occurring 17.7ms and 15.5ms after the start of loading with 0.0026% and -0.0044% residual strains were recorded for gauges T3A and C3 respectively. Tabulated values and strain-time histories are reported in Table 4-19 and Figure B-25a and Figure B-25b.

Following the test there were no signs of damage to the column including no crushing of concrete, no visible cracking and no visible residual displacement and the tension brace rods were still tight. The post-test column is shown in Figure 4-39b.

4.2.18.2. 3NS-TB3-2

Test 3NS-TB3-2 used a driver length of 2743mm and a driver pressure of 241kPa resulting in a reflected pressure of 36.9kPa and a reflected impulse of 436.6kPa-ms over a positive phase of 23.7ms as reported in Table 4-1 and shown in Figure A-38d. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 23.9mm, 22.8mm, and 14.7mm respectively. These displacements occurred 17.9ms, 19.2ms, and 18.2ms after the start of the positive phase and resulted in 3.5mm, 3.3mm, and 2.9mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 1.38°. Displacement values and time histories are presented in Table 4-1 and Figure A-38c for this test.

Eight strain gauges at various locations on the internal reinforcement were monitored during this test. The maximum tension and compression strain occurred at mid-height in gauges T3A and C3, respectively. Maximum strains of 0.2104% and -0.0854% occurring 17.7ms and 8.6ms after the start of loading with 0.0171% and -0.0197% residual strains were recorded for gauges T3A and C3 respectively. Tabulated values and strain-time histories are reported in Table 4-19 and Figure B-25c and Figure B-25d.

After the test it was observed that there were three closed cracks at the column mid-height spaced approximately 150mm apart. There was no cracking at the support locations. There were no further signs of damage including no crushing of concrete, no visible residual displacement and the tension brace rods were still tight. The post-test column is shown in Figure 4-39c and Figure 4-39d.

4.2.18.3. 3NS-TB3-3

Test 3NS-TB3-3 used a driver length of 2743mm and a driver pressure of 549kPa resulting in a reflected pressure of 67.9kPa and a reflected impulse of 879.5kPa-ms over a positive phase of 28.3ms as reported in Table 4-1 and shown in Figure A-39b. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 53.7mm, 47.5mm, and 29.2mm respectively. These displacements occurred 20.4ms, 20.4ms, and 18.3ms after the start of the positive phase and resulted in 7.6mm, 11.4mm, and 7.6mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 3.11°. Displacement values and time histories are presented in Table 4-1 and Figure A-39a for this test.

Eight strain gauges at various locations on the internal reinforcement were monitored during this test. The maximum tension and compression strain occurred at mid-height in gauges T3A and C3, respectively. Maximum strains of 0.3143% and -0.0873% occurring 11.8ms and 7.1ms after the start of loading with -0.0095 % and -0.0310% residual strains were recorded for gauges T3A and C3 respectively. Tabulated values and strain-time histories are reported in Table 4-19 and Figure B-26a and Figure B-26b.

After the test it was observed that more closed cracks on the unloaded face formed. These cracks were spaced at approximately 75mm over the majority of the column, stopping approximately 250mm from each support. There was crushing of the concrete cover at mid-height on the loaded face over a length of approximately 50mm with the crushing depth between 5mm and 10mm. There

was no crushing of concrete at either of the supports. There was no sign of cracking of the concrete at any location on the loaded side. The tension brace system was undamaged and the tension rods were still tight. The post-test column is shown in Figure 4-40a and Figure 4-40b.

4.2.18.4. 3NS-TB3-4

Test 3NS-TB3-4 used a driver length of 2743mm and a driver pressure of 686kPa resulting in a reflected pressure of 83.1kPa and a reflected impulse of 1037.2kPa-ms over a positive phase of 29.9ms as reported in Table 4-1 and shown in Figure A-39d. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 75.8mm, 56.9mm, and 34.7mm respectively. These displacements occurred 23.1ms, 21.4ms, and 19.1ms after the start of the positive phase and resulted in 16.6mm, 10.0mm, and 8.5mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 4.38°. Displacement values and time histories are presented in Table 4-1 and Figure A-39c for this test.

Seven strain gauges at various locations on the internal reinforcement were monitored during this test. The maximum tension and compression strain occurred at mid-height in gauges T3A and C3, respectively. Maximum strains of 0.2610% and -0.0680% occurring 10.8ms and 21.1ms after the start of loading with 0.0087% and -0.0277% residual strains were recorded for gauges T3A and C3 respectively. Tabulated values and strain-time histories are reported in Table 4-19 and Figure B-26c and Figure B-26d.

After the test it was observed that more closed cracks on the unloaded face formed in the mid-height region. These new cracks were spaced at approximately 37mm over a length of approximately 400mm. There were cracks on the sides of the column at the support region that formed a fan extending from the interface of the column and the foundation on the tension side (loaded side) to the approximate location of the compression reinforcement. The maximum angle of the cracks within the fan was approximately 45°. The load transfer device prevented inspection of the loaded face of the column at the supports to determine the extent of cracking. There was no crushing of concrete at the support locations. The compression concrete at mid-height was crushed to a greater extent than after test 3. The crushing depth remained approximately 10mm but the length of the crushing zone extended to approximately 150mm. There was no buckling of the compression reinforcement. There was no damage to the tension brace system, however the tension rods were somewhat loose and were re-tightened prior to test 5. Similarly to what was observed in columns 3NS-TB1 and 3NS-

TB2, the displacement gauge mounting bolts showed permanent deformation. This bending of the bolts, due to the inertia forces in the displacement gauges, would have resulted in an overestimation of the maximum and residual displacements by up to 3mm-5mm. The post-test column is shown in Figure 4-40c and Figure 4-40d.

4.2.18.5. 3NS-TB3-5

Test 3NS-TB3-4 used a driver length of 4877mm and a driver pressure of 688kPa resulting in a reflected pressure of 83.1kPa and a reflected impulse of 1849.6kPa-ms over a positive phase of 63.3ms as reported in Table 4-1 and shown in Figure A-40b. The maximum mid-height displacement, 1/3 height displacement, and 1/6 height displacement were 76.0mm, 58.8mm, and 35.1mm respectively. These displacements occurred 24.9ms, 22.2ms, and 19.7ms after the start of the positive phase and resulted in 31.5mm, 19.4mm, and 12.1mm residual displacement for the gauges at mid-height, 1/3 height and 1/6 height respectively. The maximum support rotation based on the mid-height displacement was 4.39°. Displacement values and time histories are presented in Table 4-1 and Figure A-40a for this test.

Three strain gauges at various locations on the internal reinforcement were monitored during this test. Gauge C3 was damaged early in the test and did not provide reliable data. For gauge T3A, the maximum strain was 0.2759% occurring 11.0ms after the start of loading with -0.0348% residual strain. Tabulated values and strain-time histories are reported in Table 4-19 and Figure B-27.

After the test it was observed that the cracking of concrete on the unloaded face at mid-height became more significant. There was some spalling of the concrete cover on the tension face corners at mid-height. There was more crushing of the concrete at the mid-height with the crushing extending into the core to a depth of approximately 30mm. The compression reinforcement at mid-height buckled in one of the zones between transverse reinforcement. There was no crushing of concrete at the support regions and the tension cracks remained closed. The tension brace system was not damaged however there was damage to the connection of one of the tension braces and the foundation. The anchor bolts connecting the brace at this location began to pull out of the foundation. This allowed for the rotation of the support mechanism of the tension brace system and would have allowed for some minimal movement of the tension rods. It was observed that the mounting bolts for the displacement gauges were bent by 5mm to 10mm after the test. The 8mm diameter anchor bolts for mounting the gauges were installed such that approximately 75mm of unsupported length existed between the displacement gauge connection and the column surface.

The bending of these bolts was likely due to the large inertia forces associated with the acceleration of the column at the gauge connection points and the mass of the displacement gauges. The mounting bolts bending could have resulted in a 5mm to 10mm overestimation of maximum and residual displacement. This error was likely not isolated to test 5, rather it should be averaged over tests 3, 4, and 5 where the most significant column displacements were seen. These large column displacements would have resulted in large accelerations at the gauge points and therefore large inertia forces that could have caused permanent damage to the mounting bolts. Therefore, it would be appropriate to assume that the displacement records for tests 3, 4, and 5 are accurate to within 3mm each with the experimental time-histories erring on the side of larger displacements. The post-test column is shown in Figure 4-41a through Figure 4-41d.

4.3. Repeatability of Pressure and Impulse

Similar reflected pressure and impulse combinations were used for columns tested within the same companion group to allow for direct comparison of results. To assess the repeatability of the pressure and impulse loading on the columns, normalized values for each group were determined. Normalized pressure and impulse values were calculated by averaging the pressure and impulse values for each similar test and dividing individual values by the mean. The normalized values are plotted in Figure 4-42 and Figure 4-43 for the reflected pressure and reflected impulse respectively. The normalized values were sorted from lowest to highest and plotted on an equal interval percentile scale. The linear distribution of the normalized values along with the low standard deviation and high R^2 values indicate a normal distribution of pressure and impulse with low deviation from the mean. There are some outliers for both pressure and impulse indicating that some of the tests significantly varied from the mean. For pressure, the maximum variation from the mean was approximately 28% for two tests with normal variations within roughly 10% of the mean. For impulse the maximum variation on the low end was approximately 31% and the maximum variation on the high end was approximately 58% with normal variations within 15%. The high variation for the high end of the impulse can be attributed to the unreliable negative phase of the shock waves generated with the shock tube. In some cases the positive phase is extended over a long duration with very low pressures before the negative phase is reached. In extreme cases, the negative phase does not occur until after secondary or tertiary shock fronts arrive. Both of these cases have the effect of increasing the impulse over the positive phase and result in the large variation at the high end of the normalized impulse plot. The effect of this increase in impulse on the column is likely small because of the long positive phases generated with the driver lengths used in this testing program. In general, most columns reached

maximum displacement at approximately the same time as the end of the positive phase. After the initial thrust of the pressure loading, the primary force in the column is the inertia force so even a longer positive phase, and corresponding higher impulse, would have minimal effect on the column response.

4.4. Axial Load Variation

Axial load was applied using a hydraulic jack that applied compression stress at the ends of the columns. As the columns deflected horizontally, the projected length of the column shortened. This shortening resulted in a change in axial load during response as the columns moved away from the hydraulic jack. An attempt at measuring the axial displacement, and the corresponding change in axial load, was made for the group 2 columns. This was done by installing spring loaded wire displacement transducers at the top and bottom of the columns and monitoring the initial axial displacement due to the application of load as well as the change in axial displacement during response. It was found after the tests that the wire transducers had a significant lag in response time and did not produce results that could be considered accurate enough to present in detail. None the less, the variation in axial load for one of the column tests is presented in Figure 4-44 and Figure 4-45.

Figure 4-44 shows the mid-height displacement and the axial displacement of column 2NS-C-2 plotted against time. There is a direct correlation between the magnitude of the horizontal displacement and the axial displacement for this test. Based on the initial axial deformation due to the application of axial load, this column lost all axial load once the axial displacement was 1.41mm. In Figure 4-44 it appears that this corresponds to approximately 28mm of mid-height displacement, however, due to the lag in response time for the axial displacement gauges, the time to loss of axial load and corresponding horizontal displacement is not accurate.

In Figure 4-45 the axial displacement is converted to axial load by assuming a linear relationship between the change in load and the change in axial displacement. This assumption is likely valid as the column was loaded axially well within the elastic capacity and, therefore, the material should be relatively linear. The axial load variation is plotted against the mid-height displacement. This figure shows that there is a fairly linear relationship between the change in axial load and the mid-height displacement. It also shows the lag in the response time for the wire transducers used to measure axial deformation as a region of constant axial load up to approximately 5mm horizontal displacement at mid-height. For this figure, any axial displacements that were higher than the initial axial deformation due to the application of load were assumed to correspond to zero axial load.

The variation of axial load with horizontal displacement is a significant factor in the response of the column. As the axial load drops, the column behaves more like a beam than a beam-column. This means that the neutral axis shifts towards the cover compression concrete resulting in a member that is less stiff with lower moment capacity (for the columns axially loaded below balanced conditions). The loss of axial load also reduces the second order moments on the column and, as these moments become most significant with large displacements, considerably changes the column response. Some of the parameters explored in this testing program are significantly affected by axial load. Specifically, the effects of confinement of the core concrete with variable spacing of transverse reinforcement and confinement using external retrofits such as FRP jacketing and prestress confinement are limited by the loss of axial load. The effect of variable axial load is explored in more detail in the following chapter.

4.5. Comparison of Column Response

There were several different parameters explored for this research. These include internal detailing of reinforcement and the use of various retrofit techniques. This section discusses the test parameters for all three groups.

4.5.1. Group 1 Columns

Group 1 columns were tested three times. The average pressures and impulses for the first, second and third test were 13.4kPa, 161.5kPa-ms; 22.0kPa, 238.2kPa-ms and 59.9kPa, 656.9kPa-ms, respectively. The group consisted of non-seismic and seismic columns with and without a single layer of FRP jacketing over the entire unsupported length. The displacement-time histories for shots 1, 2, and 3 for all four columns in the group are shown in Figure 4-46, Figure 4-47 and Figure 4-48. From these figures it is evident that there is no significant difference in the maximum or residual displacements across the group for any test and there is no obvious difference in the period of response. The maximum and residual displacement values for each test are shown in bar charts in Figure 4-49 and Figure 4-50 to provide a simple direct comparison.

The only difference in the two control columns in this group is the spacing of the transverse reinforcement. For the non-seismic column, the reinforcement was spaced at 50mm and for the seismic column the reinforcement was spaced at 25mm. The transverse reinforcement serves the purpose of confining the concrete core, delaying reinforcement buckling and increasing shear capacity. Since the columns were so slender, with a weak axis section depth of 100mm, they could displace a significant amount before critical points such as yield moment or ultimate support

rotation. This large displacement capacity is thought to have resulted in significant loss of axial load prior to the columns reaching a damage state. The axial load variation was discussed in section 4.4. Once the axial load was reduced, the member displaced in pure flexure and the neutral axis shifted to the edge of the core of the column. This put much of the core concrete in tension and negated any effects of increased confinement due to the variable spacing of the transverse reinforcement. This assumption is verified by looking at the strain values recorded during the test at mid-height on the compression and tension reinforcement. For example, in test 1NS-C-2 the maximum real tension strain was 0.2904% while the maximum compression strain was -0.1268%. This value was calculated by adding the initial axial strain to the maximum values presented in Table 4-2. Using these strain values along with the assumption that the core depth extended to the middle of the longitudinal reinforcement, only 15.2mm of the 50mm core was in compression. Since the cover to the middle of the longitudinal reinforcement was 25mm, the total compression zone depth is approximately 40mm with the majority of that depth, and the high strain region, comprised of cover concrete.

The columns that were retrofitted with FRP jackets had seismic and non-seismic detailing of the transverse reinforcement. The core confinement along with the FRP confinement effectively created two confined regions within the column. If the axial load remained high during response, it is likely that the FRP would have significantly contributed to both the strength and the ductility capacity of the column. However, since the axial load dropped with displacement, the same argument for the as-built columns is made for the retrofitted columns: the neutral axis shifted and beam bending behavior was the response. For the FRP to be effective in confining the column, it needs to be activated by the lateral pressure of the concrete due to a triaxial stress state. Since there was very little axial load, there was very little lateral pressure activating the FRP and, in turn, creating a confined concrete state in the column. This argument explains the lack of strengthening and stiffening of the column with the addition of FRP jacketing.

Although it does not appear that there were any significant strength benefits due to the FRP jacketing or the seismic detailing of the transverse reinforcement, there were noticeable benefits to the column ductility capacity. The seismic and non-seismic detailed as-built columns did not have considerable differences in the damaged state after test 3. This is likely due to the small core to gross section area ratio of the column. There was still significant loss of the concrete cover in both columns with no crushing of the core concrete. Neither column had reinforcement buckling due to the close spacing of the transverse reinforcement and the small crushing depth. The main difference in the

column was the distributed tension cracking. The cracks were more frequent in the seismic column indicating that there was potentially more energy dissipation capacity as the damage was spread over a larger area. The FRP jackets displayed the most benefit to ductility capacity for this group. For both types of columns, these jackets prevented concrete crushing and distributed the tension cracking over a large region. The lack of concrete crushing indicates that there is a significant increase in ductility capacity in the columns with FRP jacketing.

4.5.2. Group 2 Columns

Several different parameters were tested with group 2 columns including the effects of seismic detailing, the effects of FRP jacketing, the effects of longitudinal surface bonded FRP for strengthening, and the combination of longitudinal FRP and FRP jacketing. This section compares the results of the column tests for each parameter.

4.5.2.1. Non-Seismic Columns

The mid-height displacement-time history plots for the non-seismic group 2 columns shots 1 through 3 are shown in Figure 4-51, Figure 4-52 and Figure 4-53. For the first test there was little difference in response of the columns. During the test all the columns remained well within their elastic limits. Therefore, the additional resistance of from the external longitudinal FRP and the benefits of the additional confinement from FRP jacketing were minimal and not readily observed.

The second test shows that the columns with the FRP strengthening retrofit (2NS-R and 2NS-JR) had lower maximum displacements and lower residual displacements than the control column and the jacketed column. From the displacement-time plots of this test, it is evident that the longitudinal strengthening was effective in increasing the strength of the columns and increasing the stiffness of the columns. The stiffness increase is evident from the lower maximum displacement under the similar loading. There was no significant difference between the control column and the jacketed column. This test pushed the control column and the jacketed column to the elastic limit based on the existence of residual displacement and the state of the columns after the test. Since the axial load is thought to have dropped significantly during the column response, the lack of effectiveness of the FRP jacketing for this test is realistic. The benefits of confinement from the FRP only become significant on the concrete strength and stiffness when there is the presence of axial load.

The third test pushed all columns beyond their elastic limits. The largest maximum and residual displacements were seen in the control column followed consecutively by columns 2NS-R, 2NS-J and

2NS-JR. The control column having the largest displacements is expected due to the lack of strengthening or ductility enhancing retrofits. This column saw significant concrete crushing and the formation of distinct plastic hinges. The column with the FRP strengthening alone, 2NS-R, had high displacements because of the debonding of the FRP from the concrete substrate. The FRP debonded at 14.2ms after the start of the loading according to the strain-time history. This is approximately half the time to the maximum displacement. Up to the point of the FRP debonding, the column stored considerable strain energy in the external FRP retrofit. Once the FRP debonded, the strain energy was released and the column became significantly weaker and less stiff. The inertia force and external loading in the column was significant enough after the debonding to propel the column to a considerably damaged state. This column also had significant crushing of concrete and distinct plastic hinges formed after the test. The FRP jacketed column did not see damage to the retrofit during this test. Although the confinement effects on concrete strength and stiffness may have been minimal once the axial load was diminished, the jacket served to prevent crushing of the cover concrete. This had the effect of increasing the resistance of the column above that of the control column and also providing an increase in the ductility capacity of the column. Column 2NS-JR had an external longitudinal FRP retrofit on both the loaded and unloaded faces that was wrapped with an FRP jacket. The FRP jacket prevented premature debonding of the FRP and allowed full activation of the strengthening retrofit. This had the effect of reducing the total displacement due to an increase in strength and stiffness in the column. Once the longitudinal FRP ruptured, the FRP jacket served to prevent crushing of the concrete which both increased strength and provided ductility capacity to the column.

4.5.2.2. Seismic Columns

The mid-height displacement-time history plots for the seismic group 2 columns shots 1 through 3 are shown in Figure 4-54, Figure 4-55 and Figure 4-56. For the first test there was little difference in response of the retrofitted columns, however, the control column had lower maximum displacement. The low response for the control column can likely be explained by it being tested under a lower impulse than the other tests, by between 17% and 30%. During the test all the columns remained well within their elastic limits. Therefore, the additional resistance of from the external longitudinal FRP and the benefits of the additional confinement from FRP jacketing were minimal and not readily observed.

The second test shows that the column with the FRP strengthening (2S-R) and the column with the FRP jacket (2S-J) had similar displacements that were lower than the column with the FRP strengthening and jacket (2S-JR) and the control column (2S-C). In section 4.2.12.2 the likelihood that the FRP strengthening in column 2S-JR ruptured during this test is discussed. If, as it is thought, the FRP did rupture, the high displacement for the column is explained by the loss of the stiffness and strength during response. The displacement levels, both maximum and residual, for column 2S-JR are similar to that of column 2S-J indicating that the loss of FRP in 2S-JR was likely. In the discussion of results for the non-seismic group an argument for why the control column had similar displacement values as the jacketed column was made. In the seismic group the control column had significantly higher displacements than the jacketed column. This is likely due to the higher impulse loading on the column which was between 31% and 40% higher than the other columns in the group. The column with the external FRP strengthening retrofit had the lowest displacement in this test. This is because the FRP did not debond from the concrete substrate and the column retained high stiffness and resistance throughout the test.

The third test pushed all columns beyond their elastic limits. The largest maximum and residual displacements were seen in the control column followed all other columns that had similar maximum and residual displacements. The control column having the largest displacements is expected due to the lack of strengthening or ductility enhancing retrofits. This column saw significant concrete crushing and the formation of distinct plastic hinges but no reinforcement buckling was observed. The column with the FRP strengthening alone, 2NS-R, had high displacements because of the debonding of the FRP from the concrete substrate. The FRP debonded at 15.2ms after the start of the loading according to the strain-time history. This is approximately half the time to the maximum displacement. Similarly to the response of the non-seismic column, the debonding of the FRP significantly weakened the column and the external loading and inertia force was high enough to cause considerable damage to the column. The column with the external FRP jacket (2S-J) had approximately the same response as its non-seismic companion column (2NS-J). Using the same argument as for the non-seismic column, the loss of axial load resulted in little increase in stiffness and strength of the concrete material properties due to confinement, however, the prevention of concrete crushing and buckling increased resistance and ductility capacity of the column over the control column. The column with the FRP strengthening and jacketing (2S-JR) had approximately the same response as the column with only the FRP jacket. This strengthens the argument that the FRP

ruptured in test 2. If the FRP ruptured at mid-height, this column would perform similarly to the column with only FRP jacketing.

4.5.2.3. Control Columns

Columns 2NS-C and 2S-C were the non-seismic and seismic control columns for group 2. Each column was subjected to three shock tube tests. The displacement-time histories for each test on the two control columns along with the typical pressure-time histories for each shot is shown in Figure 4-57. The first test on the columns caused the columns to respond essentially within the elastic range. The second test resulted in small residual displacements for both columns indicating that the elastic limit had been exceeded, but not significantly. There was no significant difference in the response of the two columns in terms of maximum displacements, residual displacements, or qualitative damage assessment for the first two tests.

The third test resulted in significant damage to both columns. The maximum displacement for the seismic column was 129.0mm which was 29mm less than the non-seismic column maximum displacement of 158mm. Similarly, the residual displacement for the seismic column (98.8mm) was 37.7mm less than the residual displacement of the non-seismic column (134.6mm). Qualitatively, the damage to the non-seismic column was more significant than the seismic column. There was concrete crushing over a longer length and to a greater depth for column 2NS-C when compared with column 2S-C. The non-seismic column also had longitudinal reinforcement buckling at mid-height on the compression side whereas the seismic column did not. The tension cracks occurred more frequently and were distributed over a longer length for the seismic column indicating the distribution of damage and energy absorption was distributed over a larger area for the seismic column.

4.5.2.4. Jacketed Columns

Columns 2NS-J and 2S-J were the non-seismic and seismic columns with FRP jacketing for group 2. Each column was subjected to four shock tube tests of which there is data for the first three. The displacement-time histories for each of the first three tests on the two jacketed columns along with the typical pressure-time histories for each shot are shown in Figure 4-58. The first test on the columns caused the columns to respond essentially within the elastic range. There was no significant difference in the response of the two columns in test 1. The second test resulted in small residual displacements for both columns indicating that the elastic limit had been exceeded, but not significantly. In the second test, the maximum and residual displacements for the non-seismic column

were higher than the seismic column. For this test there was some discrepancy in the reflected impulse that was used to load both columns. The non-seismic column was loaded with 509.9kPa-ms reflected impulse which was 89.6kPa-ms higher than the 420.3kPa-ms impulse on the seismic column. This discrepancy in impulse likely was the cause of the higher displacement values for the non-seismic column in test 2. Qualitatively, there was no visible difference in the state of the columns after test 1 and test 2.

Test 3 resulted in significant residual displacement for both columns. For this test, the seismic column had higher displacements (both residual and maximum) than the non-seismic column. This was likely due to a discrepancy in matching the impulse loading on both columns. For this test, the seismic column was loaded with 787.2kPa-ms reflected impulse which was 144.2kPa-ms higher than the 643.0kPa-ms impulse on the non-seismic column. After the test there was no noticeable difference in the damage to the columns. Since both columns did not have crushing of concrete or reinforcement buckling and the tension cracking was distributed similarly, the FRP jacketing appeared to have effectively confined the columns to approximately the same degree.

The fourth test resulted in very similar damage states to both columns. Both columns had reinforcement rupture at the three hinge locations with no signs of damage to the FRP jacket. The FRP jacket effectively prevented crushing of the compression concrete and allowed for distribution of tension cracks over a long length. There was no reinforcement buckling for either column. As the displacement-time history and pressure-time history were not recorded for column 2NS-J-4, no direct quantitative comparison of the response can be made.

4.5.2.5. FRP Reinforced Columns

Columns 2NS-R and 2S-R were the non-seismic and seismic columns reinforced with external longitudinal FRP in group 2. The columns were subjected to three shock tube tests. The mid-height displacement-time histories and typical pressure-time histories for the columns are shown in Figure 4-59 for all three tests. Tests 1 and 2 both resulted in little damage to the columns with near zero residual displacements indicating that the columns remained in the elastic range. There was no significant difference in the displacement-time histories for the columns for the first two tests.

Test 3 resulted in significant damage to both columns. The external FRP on the unloaded face of both columns debonded from the concrete substrate. Both columns had buckling of compression reinforcement and crushing of concrete at mid-height. The non-seismic column had higher maximum

and residual displacements than the seismic column indicating that once the FRP debonded from the concrete, the seismic column was better able to withstand the remaining inertia forces and external pressure loading. This is reflected in the smaller concrete crushing depth for the seismic column that occurred over a smaller length. Both columns were damaged well beyond a serviceable limit, as is indicated by the visible damage and the large residual displacement. The post-test condition of the columns indicates that for both seismic and non-seismic columns, the post FRP debonding response of the columns is highly inelastic. In test 3, both columns had debonding of the FRP at approximately the same relative maximum strain of 1.1163% and 1.1202% for the non-seismic and seismic columns respectively.

The displacement-time histories show that the FRP strengthening retrofit results in similar behavior for both column types until debonding occurs. The response up to the point of debonding is largely elastic. Following debonding, similar behavior occurs for both column types resulting in significant damage. This behavior indicates that the FRP strengthening retrofit should only be considered as valid for columns that are not pushed to the point of debonding. The fact that both columns had approximately the same debonding strain indicates that there is no special FRP retrofit design procedure needed to account for the internal reinforcement detailing differences.

4.5.2.6. FRP Reinforced and Jacketed Columns

Columns 2NS-JR and 2S-JR were the non-seismic and seismic columns reinforced with external longitudinal FRP and with an external FRP jacket in group 2. The columns were subjected to three shock tube tests. The mid-height displacement-time histories and typical pressure-time histories for the columns are shown in Figure 4-60 for all three tests. Tests 1 resulted in little damage to the columns with near zero residual displacements indicating that the columns remained in the elastic range. There was no significant difference in the displacement-time histories for the columns after the first test. It should be noted that the displacement gauge mount at mid-height for the non-seismic column was slightly loose during all three tests. The result of this is sharper peaks and more constant slopes in the displacement-time histories visible in Figure 4-60.

Test two resulted in similar displacement-time histories for both columns; however, there was a non-zero residual displacement for the seismic column. As was discussed in sections 4.2.12.2 and 4.2.12.3, it is likely that the FRP ruptured in column 2S-JR during test 2. This was not the case for the non-seismic column in the same test. If the FRP did rupture during this test, it would have ruptured at a lower strain value than what was found as the debonding strain from columns 2NS-R and 2S-R. Based

on similar maximum displacements for columns 2NS-JR and 2S-JR for test 2, the maximum FRP strain in column 2S-JR at the point of rupture was likely in the range of 0.6, which is approximately the maximum FRP strain from column 2NS-JR in the test. If the FRP did rupture during test 2 at a relatively low strain level it was likely due to an unnoticed defect in the material, poor application of the FRP, or a defect in the column surface resulting in a stress concentration.

Test 3 resulted in significantly different responses of the two columns. The non-seismic column had lower maximum displacement and significantly lower residual displacement than the seismic column. If the FRP did rupture in test two for the seismic column, this is the expected response as the non-seismic column would be significantly stronger and stiffer. The fact that both columns saw rupture of FRP indicates that the FRP jacket was successful in preventing early debonding of the longitudinal FRP from the concrete substrate. This allowed the FRP strengthening retrofit to be fully activated. Further comparisons of responses between the two columns are not valid for this test considering the likelihood of premature FRP rupture in the seismic column.

4.5.3. Jacketed and Hoop Prestressed Columns

Column 3NS-W is a companion column to columns 2NS-J and 2S-J as all three are retrofitted with a confinement system. Columns 3NS-C and 2NS-C serve as control columns for column 3NS-W. Column 3NS-W was retrofitted with external prestressed confinement with concrete wedges used to convert the cross section from square to circular in an attempt to provide uniform confinement pressure on all surfaces of the column. The displacement-time histories for the mid-height displacement gauge and the typical pressure-time histories for these columns are shown in Figure 4-61 through Figure 4-64. Bar charts showing comparisons of maximum and residual displacements for the jacketed columns and their control columns are shown in Figure 4-65 and Figure 4-66.

The jacketed columns were subjected to four tests each. The first test, shown in Figure 4-61, shows that the three columns from group 2 (2NS-C, 2NS-J, and 2S-J) all had similar maximum and residual displacements after shot 1 and were tested well within the elastic range. Column 3NS-W had less than half the maximum displacement of the group 2 columns under the same loading. This likely is primarily the result of the significant additional mass that came from the additional concrete wedges used for 3NS-W. This mass increased the inertial resistance of the column which resulted in a requirement for higher external loading to cause the same displacement response as the columns in group 2. Column 3NS-W was also tested under slightly lower maximum pressure and impulse loads

than the columns from group 2 which also contributed to the lower displacement response. After the test, none of the columns showed signs of damage.

Test 2, shown in Figure 4-62, brought the columns within group 2 slightly above the elastic limit resulting in moderate residual displacements. Columns 2NS-C and 2NS-J had very similar maximum and residual displacements. Column 2S-J had lower maximum and residual displacements than the other columns in group 2 due to a lower impulse loading as was discussed in section 4.5.2.4. Column 3NS-W had the lowest maximum displacement of all the columns; however, it was not significantly lower than column 2S-J. . Column 3NS-W was also tested under slightly lower maximum pressure and impulse loads than the columns from group 2 which also contributed to the lower displacement response. After the test, none of the columns showed signs of damage beyond the existence of moderate residual displacement indicating that the elastic limit in all columns had been exceeded.

Test 3, shown in Figure 4-63, resulted in the complete destruction of the control column with significant concrete crushing, reinforcement buckling and significant residual displacement. The jacketed columns from group 2 both had similar maximum displacements and residual displacements that were higher than the values from column 3NS-W. The jacketing retrofit successfully prevented any crushing of the concrete in the group 2 jacketed columns. There was small crushing of concrete that did not exceed the cover depth for column 3NS-W. This crushing was at mid-height in the region between the concrete wedges. Columns 2NS-J and 2S-J both had distributed tension cracks over the mid-height unloaded side of the column indicating that the damage was spread over a large area allowing for significant energy dissipation. Column 3NS-W showed only tension cracks forming in the small region at mid-height in between the concrete wedges. The length of this region, approximately 20mm, was significantly smaller than the length of the distributed cracking in the group 2 jacketed columns. This indicates that the damage was isolated to a small area at the maximum stress location in the column and all energy dissipation due to damage of the column had to be contained to this region.

Test 4, shown in Figure 4-64, used column 3NS-C as the control column as column 2NS-C was not tested under the corresponding pressure and impulse combination. There was no data for column 2NS-J recorded for this test and the displacement-time history for column 3NS-C was interpolated from the high speed video recording. Test 4 loaded all columns well past ultimate column capacity and direct comparisons of the magnitude of the maximum and residual displacement are not a valid indicator of the level of performance of the columns. This test resulted in the complete destruction

of all three columns. The control column had significant crushing of the compression concrete and reinforcement steel buckling. The post-elastic response of this column was crushing and buckling controlled. Once the concrete crushed and reinforcement buckled, a weak hinge was formed allowing for significant rotation of the column with little resistance. As the column rotated at the hinge location, the neutral axis moved deeper into the core due to the lack of compression reinforcement and compression concrete on the loaded face. This shift in the neutral axis lowered the moment resistance and lowered the strain demand on the tension reinforcement. Due to the compression failure of this column, it was not possible to rupture the tension reinforcement. The tension reinforcement did rupture in both the jacketed columns (2S-J and 3NS-W). The confinement retrofits had the effect of largely eliminating compression concrete loss and preventing the compression reinforcement from buckling. As the column formed hinges in the high stress regions, all rotation was due to the inelastic deformation of the tension reinforcement. This would have resulted in a rotational hinge that likely had a near constant moment-curvature relationship up to the point of reinforcement rupture. Once the reinforcement did rupture, there was no resistance in the columns and large displacements were achieved. Because the reinforcement did rupture, column 2S-J had higher maximum and residual displacements than column 3NS-C. This does not indicate that the control column was stronger or stiffer or had higher ductility capacity than the FRP jacketed column. The prestressed jacketed column had lower maximum and residual displacements than the other columns. This is likely due to the fact that once the reinforcement ruptured at mid-height, the column rotated to such an extent that the concrete wedges began to bear against each other at the mid-height location. This provided an additional post-ultimate displacement resistance that prevented the column displacements from exceeding that of the control column.

4.5.4. Braced Columns

Four columns were retrofitted with external bracing in different configurations. Of these columns 3NS-TB1, 3NS-TB2 and 3NS-TB3 had tension braces and column 3NS-CB had a compression brace system with concrete wedges and prestressed steel confinement. All of the columns had non-seismic internal steel detailing. The displacement-time histories for the five tests on these columns are shown in Figure 4-67 through Figure 4-71 and bar charts comparing maximum and residual displacements are shown in Figure 4-72 and Figure 4-73. Columns 3NS-C and 2NS-C served as control columns for this set.

The first test on these columns, shown in Figure 4-67, resulted in elastic deformations for all columns. Column 3NS-TB2 had the lowest maximum displacement in this test while all other columns had similar maximum displacements. The more favorable geometry of the tension brace used in column 3NS-TB2 likely caused the reduction in maximum displacement. All displacements were less than 7mm maximum and direct comparisons are difficult due to slight variations in the magnitudes of pressure and impulse values resulting in significant variations in percentage differences in the loading quantities due to low pressure loading.

The second test, Figure 4-68, once again showed little variation in the maximum displacements in all columns. There is a noticeable increase in the period of response for column 3NS-CB. This is likely due to the increased mass of the retrofit system causing an increase in inertial resistance and decrease in the natural frequency of the column. Unlike the tension brace columns, the compression brace has the effect of reducing the applied axial load to the column during response. This also would have had the effect of increasing the natural period of the compression brace column. This test was an elastic test for all columns resulting in little damage to the columns with only closed flexural cracks at the mid-height location evident in the tension brace columns and the control column.

The third test, Figure 4-69, resulted in significant damage to the control column (2NS-C-3) with crushing of the concrete, buckling of the compression reinforcement and significant residual displacement. All of the braced columns had similar maximum displacements with 3NS-TB2 having the lowest. Similarly to what was observed in test 2, the compression brace column had a longer period of response than the other braced column. All of the braced columns had similar residual displacement values that were less than 20mm. The damage to the compression brace column was only in the form of closed cracks at the mid-height location between the concrete wedges. The tension braced columns all had some crushing of the compression concrete at mid-height that was observed after the test. This crushing is likely the result of the increased axial load in the column due to the vertical component of the reaction forces in the tension braces. The crushing was most significant in 3NS-TB2 followed by 3NS-TB1 and 3NS-TB3. Column 3NS-TB3 likely had the least amount of compression concrete damage due to the low initial axial load on the column. All of the concrete crushing was minimal only effecting concrete cover over a small length at mid-height.

The fourth test on the columns, Figure 4-70, resulted in the complete destruction of the control column (3NS-C). This column had significant crushing of compression concrete into the core, buckling of longitudinal reinforcement, distributed cracking at the mid-height and support locations, and high

residual displacements. In Figure 4-70 the displacement-time history of the control column is slightly offset from the other columns. This is likely due to the inaccuracies involved in interpolating the displacement-time history from the high speed video record of the test. Columns 3NS-TB1 and 3NS-TB3 had nearly identical displacement-time histories. These two columns shared the same tension brace geometry with the only difference being the initial axial load level. As the tension braces induce a high axial load on the column during response, the initial axial load difference did not seem to have a significant effect to the response of these columns. The maximum displacements for columns 3NS-TB1, 3NS-TB3 and 3NS-CB were all similar. Once again, it is evident that the compression column had a longer period of response than the tension braced columns. The lowest maximum displacement was seen in column 3NS-TB2. This column had the tension brace bearing points spaced an additional 127mm out from the unloaded face of the column as the other tension braced columns. This caused larger horizontal components of the forces in the brace (restoring forces) at the same displacement level as the other tension braces resulting in lower maximum displacements of the column. The residual displacements of all the braced columns were in the range of 20mm. After the test it was observed that a single crack in the tension face of the compression column had opened to approximately 1mm wide at the mid-height location between the concrete wedges. The tension braced columns had more tension cracks form during the test that were closed after the test was over. Concrete crushing extended over a longer length for columns 3NS-TB2 and 3NS-TB3 but did not exceed the cover depth.

Test 5, shown in Figure 4-71, did not have a control column as a companion to the braced columns. The maximum and residual displacements of the compression braced column were higher than the other columns during this test. The period of the response of the compression column was also longer as was the case in the previous tests. Columns 3NS-TB1 and 3NS-TB3 which shared the same tension brace geometry had nearly identical responses up to similar maximum displacements. Column 3NS-TB1 rebounded more than 3NS-TB3 and had lower residual displacement. This may be due to the higher initial axial load on the column creating a stiffer member and providing a mechanism to restore the residual displacement. Column 3NS-TB2 had the lowest maximum displacement and residual displacement of the group. This is likely due to the geometry of the tension brace producing higher horizontal restoring forces. The maximum and residual displacements of column 3NS-TB2 were less than half of those of column 3NS-CB. After the test column 3NS-CB had one large tension crack at mid-height that was open to approximately 5mm. There was also very minor crushing of the concrete cover for this column at mid-height. The tension braced columns all had more significant

concrete crushing than was observed after test 4. This crushing extended up to 30mm in depth and was distributed over lengths up to 250mm.

In general, the tension brace and compression brace retrofit systems were successful in increasing the capacity of the columns and reducing maximum and residual displacements. The tension braces appeared to perform better than the compression brace. It appears that the initial axial load level had little impact on the response of the columns that had the same tension brace geometry. Spacing the bearing points further from the column surface and, therefore, increasing the horizontal restoring force of the tension braces had a significant effect on reducing maximum and residual displacements in the column over the other tension braced columns. The tension brace system did induce large axial loads on the column that resulted in some concrete crushing at mid-height during the tests. The compression brace system seemed to largely eliminate concrete crushing in the column.

4.6. Support Rigidity

Two types of supports were used in this testing program. Group 1 columns were tested using support type A while group 2 and group 3 columns were tested using support type B. The details of the support types are provided in the previous chapter. An attempt at end rotational fixidity was made for both types of supports; however, in both cases there was rotation of the ends of the columns. This was evident by the damage states in the columns after testing as well as by the strain gauge readings. For both types of supports the maximum stress location occurred at mid-height of the column rather than at the ends of the column as would be expected for perfect fixed end constraints.

The rotational rigidity of the supports used in group 1 was not directly measured. By using the strain data recorded at mid-height and at the support location on the compression and tension steel for an elastic test, the level of support was approximated. Figure 4-74 shows the curvature-time histories for column 1NS-C-1 at the support and mid-height locations. The curvature was calculated by dividing the difference in strain in the tension and compression steel by the distance between them. Since the column remained elastic during this test, the curvature values were assumed to be proportional during the entire response. Therefore, the ratio of the internal moment at the ends of the column and the mid-height of the column should be the same as the curvature ratios at the same location. The ratios of the curvatures are shown in Figure 4-74 along with the average ratio of 0.199 over the duration of the test. This indicates that approximately five times the internal moment was seen at mid-height of the column as was seen at the support locations. This relationship is used in determining the column resistance in the following chapter.

The rotational resistance of the supports used in the group 2 and group 3 columns was directly measured. This was done by clamping a steel beam of known sectional properties in one of the supports and pulling on the end of the beam at a known location. The force applied to this cantilever test and the displacement at the location of the force was directly measured. The flexural displacement of the beam was calculated and subtracted from the total tip displacement with the difference being the displacement due to rigid body rotation at the support. The moment-rotation relationship was calculated at a number of displacement points and the corresponding rotational stiffness was found. The rotational stiffness, K_θ , was calculated as the applied moment divided by the end rotation. The moment-rotation and rotational stiffness-rotation relationships for the group 2 and group 3 supports are shown in Figure 4-75.

Table 4-1: Pressure, Impulse, Maximum Displacement, and Time to Maximum Displacement Values

Specimen		Driver Length	Driver Pressure	Reflected Pressure	Reflected Impulse	Positive Phase Duration	Maximum Displacement			Time to Maximum			Residual Displacement			Max Support Rotation
		L_d	P_d	P_r	I_r	t_d	d_{max}			t_{max}			d_{res}			θ_{max}
		mm	kPa	kPa	kPa-ms	ms	1/2	1/3	1/6	1/2	1/3	1/6	1/2	1/3	1/6	degrees
1NS-C-	1	3353	69	13.7	179.8	21.7	14.6			29.6			5.5			0.84
	2	3353	110	24.3	257.7	26.4	27.5			35.1			4.7			1.59
	3	3353	380	65.1	765.3	34.4	171.1			57.4			126.0			9.81
1S-C-	1	3353	69	13.0	155.4	20.7	10.9			28.7			2.7			0.63
	2	3353	110	24.0	226.7	23.3	21.5			29.6			2.7			1.25
	3	3353	380	63.1	621.5	26.3	156.0			53.7			128.9			8.95
1NS-J-	1	3353	70	12.6	145.8	20.9	12.8	7.7		28.0	30.3		0.0	-0.2		0.74
	2	3353	111	19.4	240.8	23.7	25.6	15.2		33.3	32.4		0.1	0.0		1.48
	3	3353	380	58.8	685.8	23.6	160.9	71.5		52.9	53.5		134.6	60.1		9.23
1S-J-	1	3353	69	14.3	165.0	24.1	13.4	8.0		32.5	33.6		6.6	4.4		0.78
	2	3353	110	20.2	227.6	23.5	27.1	15.9		40.0	39.0		5.9	3.8		1.57
	3	3353	380	52.5	555.1	22.6	135.3	59.7		51.7	52.7		110.3	49.3		7.78
2NS-C-	1	2743	70	14.0	140.4	18.0	8.2			16.6			1.6			0.47
	2	2743	243	41.8	478.4	28.6	36.5			20.9			11.5			2.11
	3	2743	546	81.0	1073.5	40.5	158.0			35.6			136.5			9.07
2S-C-	1	2743	71	14.4	131.1	18.4	4.7			18.4			1.4			0.27
	2	2743	242	47.7	687.5	45.8	37.3			21.1			9.7			2.16
	3	2743	542	70.0	686.7	23.0	129.0			28.0			98.8			7.42
2NS-J-	1	2743	70	13.5	168.3	21.7	8.0			18.5			2.4			0.46
	2	2743	244	44.8	509.9	28.9	36.3			21.4			10.0			2.10
	3	2743	548	73.2	643.0	21.2	85.5			24.6			38.7			4.94
2S-J-	1	2743	72	14.6	188.2	21.6	7.7			17.3			2.8			0.45
	2	2743	244	43.7	420.3	25.7	25.1			18.5			5.5			1.45
	3	2743	548	75.3	787.2	22.2	99.5			24.8			46.0			5.74
	4	2743	683	85.0	839.2	19.9	256.5			64.0			241.5			14.52

Table 4-1: Pressure, Impulse, Maximum Displacement, and Time to Maximum Displacement Values - Continued

Specimen		Driver Length	Driver Pressure	Reflected Pressure	Reflected Impulse	Positive Phase Duration	Maximum Displacement			Time to Maximum			Residual Displacement			Max Support Rotation
		L_d	P_d	P_r	I_r	t_d	d_{max}			t_{max}			d_{res}			θ
		mm	kPa	kPa	kPa-ms	ms	1/2	1/3	1/6	1/2	1/3	1/6	1/2	1/3	1/6	degrees
2NS-R-	1	2743	70	13.7	189.3	22.1	8.6			18.4			2.5			0.50
	2	2743	242	47.2	371.7	20.2	24.3			17.5			4.6			1.41
	3	2743	547	99.4	716.0	23.0	127.6			29.9			93.4			7.35
2S-R-	1	2743	70	14.5	157.7	18.3	7.3			16.8			2.1			0.42
	2	2743	239	46.5	474.2	24.7	22.7			16.4			5.4			1.31
	3	2743	551	85.3	776.0	22.8	97.3			26.8			51.4			5.61
2NS-JR-	1	2743	72	13.4	130.1	17.9	7.3			20.6			-2.7			0.42
	2	2743	239	40.6	355.6	20.0	32.6			17.8			0.3			1.89
	3	2743	550	79.8	678.5	23.7	70.1			23.4			14.7			4.05
2S-JR-	1	2743	70	13.8	168.0	20.9	8.1			17.1			2.2			0.47
	2	2743	239	47.0	443.4	26.1	31.4			17.2			4.9			1.82
	3	2743	563	88.2	793.2	22.1	102.7			25.5			51.8			5.92
3NS-C-	1	2743	72	13.1	135.7	18.5	6.4	5.8	3.9	17.4	17.4	16.5	2.4	2.4	2.0	0.37
	2	2743	243	39.4	378.1	26.0	20.0	18.8	12.6	16.3	16.0	16.4	6.2	6.2	5.4	1.16
	3	2743	690	78.5	996.6	26.7	152.0			32.0			138.1			8.73
3NS-W-	1	2743	48	9.2	104.0	18.4	3.1	4.2	2.1	19.9	20.2	22.3	0.7	0.8	0.5	0.18
	2	2743	242	38.2	380.2	20.6	20.5	21.3	12.1	22.2	17.2	23.2	8.7	8.3	7.3	1.19
	3	2743	522	75.7	739.3	25.0	63.0	52.2	29.2	26.9	29.4	27.6	29.5	22.9	15.5	3.64
	4	2743	690	83.2	970.7	26.7	122.5			39.1			104.9			7.06
3NS-CB-	1	2743	69	11.9	141.1	19.5	4.1	3.7	2.6	19.6	20.1	21.7	0.3	0.2	0.1	0.24
	2	2743	242	36.3	381.3	23.3	17.1	15.4	10.6	19.8	21.1	20.9	4.8	4.7	4.3	0.99
	3	2743	552	70.6	780.4	27.0	50.0	40.8	25.1	25.7	25.0	25.8	13.2	12.1	10.4	2.89
	4	2743	692	84.0	983.3	27.4	72.5	56.6	33.1	26.9	26.0	26.8	20.1	17.3	13.3	4.19
	5	4877	698	84.4	1669.8	59.1	103.3	77.8	43.1	36.7	35.8	39.3	44.5	34.0	21.2	5.96

Table 4-1: Pressure, Impulse, Maximum Displacement, and Time to Maximum Displacement Values - Continued

Specimen		Driver Length	Driver Pressure	Reflected Pressure	Reflected Impulse	Positive Phase Duration	Maximum Displacement			Time to Maximum			Residual Displacement			Max Support Rotation
		L_d	P_d	P_r	I_r	t_d	d_{max}			t_{max}			d_{res}			θ
		mm	kPa	kPa	kPa-ms	ms	1/2	1/3	1/6	1/2	1/3	1/6	1/2	1/3	1/6	degrees
3NS-TB1-	1	2743	72	11.2	155.0	20.5	6.3	5.0	2.8	21.2	21.5	21.9	0.3	0.4	0.3	0.37
	2	2743	212	34.8	375.9	22.9	24.4	21.3	12.9	19.2	18.7	17.5	2.6	2.4	1.8	1.41
	3	2743	556	73.8	888.1	28.4	55.1	42.3	24.7	21.5	19.5	17.7	5.7	4.8	3.5	3.19
	4	2743	691	82.6	982.6	27.8	72.6	55.0	34.5	23.9	20.4	19.8	14.4	4.4	6.8	4.19
	5	4877	693	79.0	1610.2	60.4	75.8	57.0	34.7	24.7	21.3	19.9	12.2	8.8	4.7	4.38
3NS-TB2-	1	2743	71	13.5	151.4	20.2	1.5	1.8	1.2	18.3	16.0	16.0	-0.2	0.1	0.1	0.09
	2	2743	243	34.6	447.0	23.5	21.1	18.2	12.1	18.5	19.1	18.9	6.2	5.0	4.8	1.22
	3	2743	551	68.4	812.8	25.7	44.0	34.0	23.4	19.5	19.0	17.5	16.8	15.7	9.6	2.55
	4	2743	686	80.3	956.3	27.7	51.6	38.0	29.0	20.3	19.1	18.3	13.6	10.6	5.0	2.99
	5	4877	690	83.9	1894.0	60.4	47.8	35.7	29.1	21.2	19.1	18.7	11.2	10.8	7.6	2.76
3NS-TB3-	1	2743	72	11.2	146.0	21.0	4.5	4.0	2.5	19.3	21.4	22.9	0.9	0.5	0.7	0.26
	2	2743	241	36.9	436.6	23.7	23.9	22.8	14.7	17.9	19.2	18.2	3.5	3.3	2.9	1.38
	3	2743	549	67.9	879.5	28.3	53.7	47.5	29.2	20.4	20.4	18.3	7.6	11.4	7.6	3.11
	4	2743	686	83.1	1037.2	29.9	75.8	56.9	34.7	23.1	21.4	19.1	16.6	10.0	8.5	4.38
	5	4877	688	83.1	1849.6	63.3	76.0	58.8	35.1	24.9	22.2	19.7	31.5	19.4	12.1	4.39

Table 4-2: Strain Values for 1NS-C

Initial Strain: -0.0603%			Relative Strain Values										
Gauge			Shot 1			Shot 2				Shot 3			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	ϵ_{max}	t_{max}	ϵ_{res}	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$
	mm		%	ms	%	%	ms	%	%	%	ms	%	%
C3	0	U	0.1449	29.1	0.0231	0.3507	29.9	0.0324	0.0555	-	-	-	-
T3	0	L	-0.0436	28.6	-0.0162	-0.0665	35.1	-0.0248	-0.0410	0.2200	32.9	0.1642	0.1231
T1	-965	L	0.0499	26.7	-0.0071	0.1266	28.1	0.0051	-0.0020	1.3911	25.8	0.3320	0.3300
T2	-740	L	-0.0067	35.9	-0.0099	0.0175	27.7	0.0047	-0.0052	0.3995	38.2	0.2396	0.2343
C2	-740	U	-0.0061	28.1	0.0078	-0.0300	30.4	-0.0085	-0.0007	0.1079	49.3	0.0761	0.0754
T5	965	L	-0.0108	21.1	0.0072	-0.0164	21.1	-0.0009	0.0063	0.6533	48.4	0.5633	0.5696
C1	-965	U	0.0612	26.8	0.0010	0.1613	28.0	0.0099	0.0109	-	-	-	-
C5	965	U	-0.0487	23.6	0.0019	-0.0727	31.3	-0.0133	-0.0114	0.3714	33.1	-0.0442	-0.0556

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel.

Table 4-3: Strain Values for 1S-C

Initial Strain: -0.0603%			Relative Strain Values										
Gauge			Shot 1			Shot 2				Shot 3			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	ϵ_{max}	t_{max}	ϵ_{res}	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$
	mm		%	ms	%	%	ms	%	%	%	ms	%	%
C3	0	U	0.1250	27.9	0.0064	0.2882	28.3	0.0237	0.0301	-	-	-	-
T3	0	L	-0.0207	34.7	-0.0102	-0.0179	35.7	-0.0207	-0.0309	-1.2490	26.0	-0.9900	-1.0209
T2	-740	L	-0.0030	37.0	-0.0030	-0.0253	35.8	0.0015	-0.0015	0.1813	36.1	0.1623	0.1608
C4	740	U	0.0101	20.9	0.0084	-0.0167	35.8	0.0088	0.0172	0.0832	13.6	0.0861	0.1033
T5	965	L	-0.0462	22.0	-0.0062	-0.0723	35.3	-0.0106	-0.0167	-0.1010	12.6	-0.2198	-0.2366
C1	-964	U	0.0301	24.2	-0.0061	0.1344	26.6	-0.0021	-0.0082	-	-	-	-

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel.

Table 4-4: Strain Values for 1NS-J

Initial Strain: - 0.0611%			Relative Strain Values										
Gauge			Shot 1			Shot 2				Shot 3			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	ϵ_{max}	t_{max}	ϵ_{res}	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma \epsilon_{res}$	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma \epsilon_{res}$
	mm		%	ms	%	%	ms	%	%	%	ms	%	%
C3	0	L	-0.0797	33.9	-0.0133	-0.0754	27.9	-0.0044	-0.0177	0.1971	50.2	0.1411	0.1235
C1	-964	L	0.0106	29.7	0.0002	0.0238	27.9	0.0012	0.0015	0.9932	41.4	0.7261	0.7276
T2	-740	U	0.0290	25.1	0.0018	0.0505	31.4	0.0001	0.0019	0.2096	48.0	0.0686	0.0705
T3	0	U	0.1218	27.0	0.0018	0.2705	27.5	0.0047	0.0065	-	-	-	-
C2	-740	L	-0.0238	20.8	0.0008	-0.0267	21.9	0.0023	0.0031	0.0624	40.1	0.0433	0.0655
T1	-964	U	0.0041	24.9	0.0011	0.0068	49.1	0.0000	0.0010	0.2010	52.1	0.1379	0.1389

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel.

Table 4-5: Strain Values for 1S-J

Initial Strain: -0.0611%			Relative Strain Values										
Gauge			Shot 1			Shot 2				Shot 3			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	ϵ_{max}	t_{max}	ϵ_{res}	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma \epsilon_{res}$	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma \epsilon_{res}$
	mm		%	ms	%	%	ms	%	%	%	ms	%	%
C3	0	L	-0.0833	29.8	-0.0454	-0.1704	49.9	-0.1267	-0.1721	0.1385	50.8	0.0500	-0.1222
C1	-964	L	0.0111	21.5	-0.0136	0.0268	29.6	-0.0039	-0.0175	1.5872	51.2	-	-
T2	-740	U	0.0287	36.4	0.0110	0.0365	39.2	0.0090	0.0200	0.1434	50.6	0.0484	0.0684
T3	0	L	0.0937	30.2	0.0179	0.2706	34.6	0.0245	0.0425	-	-	-	-
C2	-740	L	-0.0327	38.9	-0.0258	-0.0087	30.3	-0.0115	-0.0373	0.2639	47.8	0.1614	0.1241
T1	-964	U	0.0097	39.4	0.0063	0.0062	55.7	0.0043	0.0106	0.3285	43.2	0.1496	0.1601

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel.

Table 4-6: Strain Values for 2NS-C

Initial Strain: -0.0465%			Relative Strain Values										
Gauge			Shot 1			Shot 2				Shot 3			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	ϵ_{max}	t_{max}	ϵ_{res}	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$
	mm		%	ms	%	%	ms	%	%	%	ms	%	%
T1	-940	U	-0.0134	8.8	-0.0041	-0.0476	29.6	0.0136	0.0095	-0.0639	14.4	0.0514	0.0610
C1	-940	L	0.0204	8.8	0.0082	0.0871	18.4	0.0117	0.0198	0.3717	16.2	0.1394	0.1592
C2	-660	L	-0.0454	18.9	-0.0247	-0.0780	11.4	-0.0121	-0.0368	-0.0684	9.0	0.1805	0.1437
T3	0	U	0.1127	16.9	0.0107	-	-	-	-	-	-	-	-
C3	0	L	-0.0914	16.0	-0.0108	-	-	-	-	-	-	-	-
T4	-940	U	-0.0142	17.4	-0.0026	-0.0674	18.9	-0.0058	-0.0084	-	-	-	-
C4	940	L	0.0173	17.2	0.0113	0.1045	17.4	0.0153	0.0266	-0.1353	11.7	0.1185	0.1451

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel.

Table 4-7: Strain Values for 2S-C

Initial Strain: -0.0465%			Relative Strain Values						
Gauge			Shot 1			Shot 2			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	ϵ_{max}	t_{max}	ϵ_{res}	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$
	mm		%	ms	%	%	ms	%	%
T1	-940	U	-0.0143	14.2	0.0113	-0.0874	19.5	-0.0208	-0.0096
C1	-940	L	0.0196	13.3	-0.0064	0.1126	19.2	0.0206	0.0143
C3	0	L	-0.0665	17.7	-0.0207	-	-	-	-
T4	-940	U	-0.0148	14.4	0.0069	-0.1023	18.9	-0.0243	-0.0174
C4	940	L	0.0195	17.2	-0.0101	0.2043	19.3	0.0218	0.0116

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel.

Table 4-8: Strain Values for 2NS-J

Initial Strain: -0.0465%			Relative Strain Values										
Gauge			Shot 1			Shot 2				Shot 3			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	ϵ_{max}	t_{max}	ϵ_{res}	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$
	mm		%	ms	%	%	ms	%	%	%	ms	%	%
T1	-940	U	-0.0060	11.1	0.0141	-0.0607	19.9	-0.0073	0.0068	-0.0943	23.0	-0.0232	-0.0164
C1	-940	L	0.0139	11.0	-0.0127	0.1249	19.1	0.0244	0.0118	0.5839	23.1	0.1079	0.1197
C2	-660	L	-0.0366	18.3	-0.0162	-0.0504	10.8	0.0058	-0.0104	0.1112	23.5	0.0488	0.0384
C3	0	L	-0.1273	18.2	-0.0605	-	-	-	-	-	-	-	-
T4	940	U	-0.0195	15.9	0.0064	-0.0971	19.8	-0.0243	-0.0178	-0.1561	19.1	-0.0499	-0.0678
C4	-940	L	0.0269	15.5	-0.0129	0.1500	18.6	0.0262	0.0133	1.1434	23.6	0.4291	0.4424

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel.

Table 4-9: Strain Values for 2S-J

Initial Strain: -0.0465%			Relative Strain Values										
Gauge			Shot 1			Shot 2				Shot 3			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	ϵ_{max}	t_{max}	ϵ_{res}	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$
	mm		%	ms	%	%	ms	%	%	%	ms	%	%
T1	-940	U	-0.0171	8.6	0.0164	-0.0584	18.7	-0.0221	-0.0056	0.9389	26.1	-0.2032	-0.2088
C1	-940	L	0.0138	16.8	-0.0137	0.0639	18.0	0.0217	0.0080	-0.3645	32.5	0.2425	0.2505
T3	0	U	0.1090	16.8	0.0195	0.7073	18.4	0.1071	-	-	-	-	-

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel.

Table 4-10: Strain Values for 2NS-R

Initial Strain: -0.0465%			Relative Strain Values										
Gauge			Shot 1			Shot 2				Shot 3			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	<i>ε_{max}</i>	<i>t_{max}</i>	<i>ε_{res}</i>	<i>ε_{max}</i>	<i>t_{max}</i>	<i>ε_{res}</i>	<i>Σε_{res}</i>	<i>ε_{max}</i>	<i>t_{max}</i>	<i>ε_{res}</i>	<i>Σε_{res}</i>
	mm		%	ms	%	%	ms	%	%	%	ms	%	%
T1	-940	U	-0.0118	10.4	0.0145	-0.0361	15.4	-0.0122	0.0023	-0.3014	22.7	-0.2801	-0.2778
C1	-940	L	0.0073	15.0	-0.0112	0.0281	13.8	0.0224	0.0111	0.9222	21.1	0.2540	0.2651
FRP-T	0	UF	0.1135	16.4	0.0197	0.5027	16.7	0.0400	0.0597	1.1163	12.2	-	-
T3	0	U	0.1378	16.4	0.0220	0.7991	16.7	0.1219	0.1439	-	-	-	-
C3	0	L	-0.0997	16.5	-0.0247	-0.4148	17.9	-0.1654	-0.1901	-	-	-	-
T4	940	U	-0.0193	16.7	0.0120	-0.0728	16.2	0.0116	0.0237	-0.2523	22.9	-0.2060	-0.1824
C4	940	L	0.0200	17.1	-0.0169	0.1056	16.5	-0.0077	-0.0246	1.5486	14.9	0.3945	0.3698

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel; UF-Unloaded Face on longitudinal FRP.

Table 4-11: Strain Values for 2S-R

Initial Strain: -0.0465%			Relative Strain Values										
Gauge			Shot 1			Shot 2				Shot 3			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	<i>ε_{max}</i>	<i>t_{max}</i>	<i>ε_{res}</i>	<i>ε_{max}</i>	<i>t_{max}</i>	<i>ε_{res}</i>	<i>Σε_{res}</i>	<i>ε_{max}</i>	<i>t_{max}</i>	<i>ε_{res}</i>	<i>Σε_{res}</i>
	mm		%	ms	%	%	ms	%	%	%	ms	%	%
C1	-940	L	0.0389	15.3	0.0165	0.0681	16.5	0.0371	0.0536	0.8205	24.7	0.2272	0.2809
T1	-940	U	-0.0316	14.5	-0.0143	-0.0456	16.9	-0.0272	-0.0416	-0.1590	23.5	-0.0591	-0.1007
C3	0	L	-0.1051	17.0	-0.0287	-0.5636	17.0	-0.3326	-0.3613	-2.4030	17.1	-0.3168	-0.6781
T3	0	U	0.1137	17.1	0.0146	0.6225	17.4	0.0981	0.1127	-	-	-	-
C4	940	L	-0.0127	9.3	0.0075	-	-	-	-	-	-	-	-
FRP-T	0	UF	0.1392	17.1	0.0174	0.4943	16.3	0.0248	0.0421	1.1202	13.8	-	-

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel; UF-Unloaded Face on longitudinal FRP.

Table 4-12: Strain Values for 2NS-JR

Initial Strain: -0.0465%			Relative Strain Values										
Gauge			Shot 1			Shot 2				Shot 3			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	ϵ_{max}	t_{max}	ϵ_{res}	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$
	mm		%	ms	%	%	ms	%	%	%	ms	%	%
T1	-940	U	0.0054	18.7	0.0000	-0.0223	6.6	0.0069	0.0069	-0.0837	19.7	-0.0043	0.0026
C1	-940	L	-0.0098	18.8	0.0037	-0.0114	9.5	0.0069	0.0106	0.1697	20.1	0.0182	0.0288
FRP-T	0	UF	0.1346	17.1	-0.0002	0.6065	17.0	0.0645	0.0643	1.4519	15.6	-	-
T3	0	U	0.1218	17.1	0.0022	0.6392	17.3	0.1277	0.1299	-	-	-	-
C3	0	L	-0.0825	17.2	-0.0072	-0.4478	17.7	-0.2382	-0.2454	-1.3993	21.8	-0.8575	-1.1029
T4	940	U	-0.0707	15.6	-0.0051	-0.1401	15.4	-0.0142	-0.0193	-0.1890	14.2	-0.1597	-0.1790
C4	940	L	0.0607	15.6	-0.0005	0.1492	16.8	-0.0086	-0.0091	1.6572	19.5	0.3746	0.3655

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel.

Table 4-13: Strain Values for 2S-JR

Initial Strain: -0.0465%			Relative Strain Values										
Gauge			Shot 1			Shot 2				Shot 3			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	ϵ_{max}	t_{max}	ϵ_{res}	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$
	Mm		%	ms	%	%	ms	%	%	%	ms	%	%
T1	-940	U	0.0106	18.9	0.0078	-0.0056	19.6	-0.0091	-0.0013	-0.0899	31.5	-0.0365	-0.0378
C1	-940	L	-0.0074	19.2	-0.0103	0.0184	17.2	0.0242	0.0139	0.3285	23.8	0.0575	0.0714
C2	-660	U	-0.0459	16.8	-0.0164	-0.1012	17.5	0.0079	-0.0084	-0.1632	12.7	0.0167	0.0082
T3	0	L	0.1468	16.3	0.0212	1.5674	18.7	0.7178	0.7390	-	-	-	-
C3	0	U	-0.0847	16.7	-0.0255	-0.7355	19.6	-0.4947	-0.5202	-	-	-	-
T4	940	L	-0.0390	15.1	0.0146	-0.1317	16.9	-0.0002	0.0144	-0.2582	24.0	-0.1355	-0.1211
C4	940	U	0.0292	14.4	-0.0105	0.1063	16.4	-0.0068	-0.0173	-	-	-	-

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel.

Table 4-14: Strain Values for 3NS-C

Initial Strain: -0.0489%			Relative Strain Values										
Gauge			Shot 1			Shot 2				Shot 3			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	ϵ_{max}	t_{max}	ϵ_{res}	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$
	Mm		%	ms	%	%	ms	%	%	%	ms	%	%
T1	-665	U	0.0277	19.7	0.0087	0.0566	13.5	0.0183	0.0270	0.0313	38.1	0.0080	0.0350
C1	-665	L	-0.0231	19.4	-0.0081	-0.0673	13.5	-0.0130	-0.0210	0.1801	32.9	0.1080	0.0870
T2	-332.5	U	0.0763	16.5	0.0124	0.3182	14.9	0.0254	0.0378	-	-	-	-
C2	-332.5	L	-0.0539	16.5	-0.0108	-0.1308	15.5	-0.0209	-0.0317	-	-	-	-
T3 [†]	0	U	0.0072	16.1	0.0146	0.0173	19.1	0.0143	0.0290	-	-	-	-
C3	0	L	-0.0645	17.0	-0.0135	-0.1472	13.5	-0.0234	-0.0369	-	-	-	-
T5	665	U	0.0546	17.1	0.0136	0.1828	19.6	0.0041	0.0177	0.3180	32.0	0.0560	0.0737
C5	665	L	-0.0421	17.2	-0.0109	-0.0954	13.1	-0.0137	-0.0246	-0.1362	8.4	0.0632	0.0385
T6	-970	U	-0.0350	16.1	0.0072	-0.0774	10.4	0.0121	0.0194	-0.2067	18.4	0.0859	0.1053
C6	-970	L	0.0319	11.6	-0.0053	0.0740	8.9	-0.0042	-0.0095	-	-	-	-

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel.

† Strain gauge T3 likely recorded not reliable strain readings for all tests.

Table 4-15: Strain Values for 3NS-W

Initial Strain: -0.0542%			Relative Strain Values										
Gauge			Shot 1			Shot 2				Shot 3			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	ϵ_{max}	t_{max}	ϵ_{res}	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$
	mm		%	ms	%	%	ms	%	%	%	ms	%	%
T1	-665	L	-0.0143	21.8	-0.0027	-0.0512	20.2	-0.0056	-0.0083	-0.0477	13.4	0.0061	-0.0021
C1	-665	U	0.0243	21.2	0.0044	0.0626	20.4	0.0101	0.0145	0.0477	13.0	-0.0327	-0.0181
T2	-332.5	L	-0.0272	20.1	-0.0039	-0.1309	18.6	-0.0215	-0.0254	-0.1336	18.4	-0.0080	-0.0334
C2	-332.5	U	0.0339	20.4	0.0041	0.2719	20.9	0.0292	0.0332	0.2511	13.8	-0.0187	0.0145
T3	0	L	-0.0369	16.7	-0.0053	-0.1359	16.0	-0.0198	-0.0250	-	-	-	-
C3	0	U	0.0486	17.0	0.0048	-	-	-	-	-	-	-	-
S1†	-920	S	-0.0047	6.5	-0.0001	-0.0125	7.3	-0.0036	-0.0038	0.0305	24.4	0.0033	-0.0005
S2†	-64	S	-0.0058	6.4	-0.0002	-0.0141	7.4	-0.0058	-0.0060	0.0675	29.1	0.0446	0.0386
T6	-970	L	0.0221	17.2	-0.0018	0.0538	7.8	0.0063	0.0045	0.2881	25.5	0.0374	0.0419
C6	-970	U	-0.0104	12.7	0.0038	-0.0328	7.3	0.0024	0.0062	-0.1349	29.0	-0.0555	-0.0494

Gauge			Shot 4			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$
	mm		%	ms	%	%
T1	-665	L	0.0801	35.6	0.0474	0.0453
C1	-665	U	0.0767	10.9	-0.0164	-0.0346
T2	-332.5	L	-0.1144	12.5	-0.0115	-0.0449
C2	-332.5	U	0.2699	16.8	0.0232	0.0377
S1†	-920	S	0.0435	39.8	0.0279	0.0274
S2†	-64	S	0.1917	45.1	0.1625	0.2010

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel; S-Side of column on prestress steel.

† Initial strain not applicable for this gauge.

Table 4-16: Strain Values for 3NS-CB

Initial Strain: -0.0542%			Relative Strain Values										
Name	Gauge	Location **	Shot 1			Shot 2				Shot 3			
	X*		ϵ_{max}	t_{max}	ϵ_{res}	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma \epsilon_{res}$	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma \epsilon_{res}$
	mm		%	ms	%	%	ms	%	%	%	ms	%	%
T2	-332.5	L	-0.0331	20.0	-0.0065	-0.1038	21.0	-0.0209	-0.0274	-0.1239	15.0	-0.0060	-0.0334
C2	-332.5	U	0.0412	19.6	-0.0014	0.2325	21.0	0.0123	0.0109	0.3245	15.0	0.0009	0.0118
T3	0	L	-0.0364	19.0	-0.0068	-0.1087	21.5	-0.0162	-0.0229	-0.4111	33.6	-0.2687	-0.2916
C3	0	U	0.0611	18.9	-0.0004	-	-	-	-	-	-	-	-
VB1†,††	0	VB	-0.0038	17.6	0.0003	0.0143	21.9	0.0002	0.0006	0.0421	25.7	-0.0033	-0.0028
VB2†	0	VB	-	-	-	-	-	-	-	-0.1116	25.5	-0.0556	-
S1†	-920	S	0.0008	26.2	-0.0004	0.0141	44.0	-0.0021	-0.0025	-0.0020	20.0	-0.0031	-0.0056
S2†	-64	S	-0.0018	19.3	-0.0044	-0.0234	64.8	-0.0146	-0.0190	0.0176	26.1	-0.0113	-0.0303
T6	-970	L	-0.0181	28.1	-0.0044	-0.0606	29.6	-0.0196	-0.0240	0.1609	22.0	0.0344	0.0104
C6	-970	U	0.0158	28.6	0.0010	0.0587	30.1	0.0129	0.0139	-0.0554	19.4	-0.0138	0.0000

Name	Gauge	Location **	Shot 4				Shot 5			
	X*		ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma \epsilon_{res}$	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma \epsilon_{res}$
	mm		%	ms	%	%	%	ms	%	%
T2	-332.5	L	-0.1115	13.1	-0.0078	-0.0412	-0.0901	11.5	0.1028	0.0616
C2	-332.5	U	0.3225	23.4	0.0048	0.0166	0.4041	33.7	0.0146	0.0312
T3	0	L	-0.5398	38.7	-0.3955	-0.6871	-0.5773	58.0	-0.2170	-0.9040
VB1†,††	0	VB	0.0610	25.9	-0.0048	-0.0075	0.2048	33.1	0.0831	0.0755
VB2†	0	VB	-0.1951	23.8	-0.0545	-0.0545	-0.5096	36.1	-0.3212	-
S1†	-920	S	0.0182	11.6	0.0125	0.0069	-	-	-	-
S2†	-64	S	0.0511	28.3	-0.0015	-0.0318	0.1125	11.9	0.0011	-0.0306
T6	-970	L	0.2268	22.0	-0.0176	-0.0072	0.1919	16.0	0.0543	0.0471
C6	-970	U	-0.0473	19.3	0.0142	0.0142	-0.0368	6.9	0.0687	0.0830

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel; VB-Side of column on vertical external compression brace; S-Side of column on prestress steel.

† Initial strain not applicable for this gauge.

†† Gauge was likely damaged and not producing reliable data.

Table 4-17: Strain Values for 3NS-TB1

Initial Strain: -0.0542%			Relative Strain Values										
Gauge			Shot 1			Shot 2				Shot 3			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	<i>ε_{max}</i>	<i>t_{max}</i>	<i>ε_{res}</i>	<i>ε_{max}</i>	<i>t_{max}</i>	<i>ε_{res}</i>	<i>Σε_{res}</i>	<i>ε_{max}</i>	<i>t_{max}</i>	<i>ε_{res}</i>	<i>Σε_{res}</i>
	mm		%	ms	%	%	ms	%	%	%	ms	%	%
C1	-665	L	-0.0121	24.8	-0.0058	-0.0574	16.4	0.0010	-0.0048	-0.0717	18.2	-0.0019	-0.0067
C2	-332.5	L	-0.0546	19.8	-0.0083	-0.1176	16.7	0.0014	-0.0069	-0.1460	18.3	-0.0153	-0.0223
T2	-332.5	U	0.0504	19.1	0.0106	0.2543	17.8	0.0216	0.0322	0.2712	10.2	0.0099	0.0421
C3	0	L	-0.0616	18.6	-0.0094	-0.1058	14.8	-0.0082	-0.0176	-	-	-	-
T3A	0	U	0.0954	18.5	0.0111	0.2647	15.9	0.0148	0.0259	-	-	-	-
T3B	0	U	0.0914	18.6	0.0049	0.3001	15.8	0.0164	0.0213	0.6361	10.2	0.0812	0.0976
C4	332.5	L	-0.0517	17.6	0.0057	-0.1065	15.3	0.0024	0.0081	-0.1684	21.4	-0.0004	0.0077
C5	665	L	-0.0243	14.9	0.0054	-0.0646	21.1	-0.0111	-0.0057	-0.1283	20.7	0.0083	0.0026
C6	-970	L	0.0268	13.7	-0.0064	-0.0204	20.4	0.0075	0.0011	-0.0259	13.5	0.0022	0.0033

Gauge			Shot 4				Shot 5			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	<i>ε_{max}</i>	<i>t_{max}</i>	<i>ε_{res}</i>	<i>Σε_{res}</i>	<i>ε_{max}</i>	<i>t_{max}</i>	<i>ε_{res}</i>	<i>Σε_{res}</i>
	mm		%	ms	%	%	%	ms	%	%
C1	-665	L	-0.0788	12.2	0.0115	0.0048	0.0134	22.3	0.0567	0.0615
C2	-332.5	L	-0.1378	10.5	0.0129	-0.0094	-0.1214	13.4	0.0869	0.0774
T2	-332.5	U	0.2330	10.9	-0.0025	0.0396	0.1925	16.4	-0.0406	-0.0010
C4	332.5	L	-0.1583	21.0	0.0131	0.0208	-0.1564	21.7	0.0775	0.0983
C5	665	L	-0.1528	20.3	0.0164	0.0190	-0.1719	21.7	0.0453	0.0643
C6	-970	L	0.1164	5.0	0.0092	0.0125	0.1554	7.6	0.0202	0.0327

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel.

Table 4-18: Strain Values for 3NS-TB2

Initial Strain: -0.0542%			Relative Strain Values										
Gauge			Shot 1			Shot 2				Shot 3			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	ϵ_{max}	t_{max}	ϵ_{res}	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$
	mm		%	ms	%	%	ms	%	%	%	ms	%	%
C5	665	L	-0.0128	23.7	-0.0013	-0.0404	16.5	0.0083	0.0070	-0.1302	17.9	-0.0007	0.0063
T3	0	U	0.0137	11.3	-0.0074	0.2343	17.0	0.0097	0.0023	-	-	-	-

Gauge			Shot 4				Shot 5			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$	ϵ_{max}	t_{max}	ϵ_{res}	$\Sigma\epsilon_{res}$
	mm		%	ms	%	%	%	ms	%	%
C5	665	L	-0.1523	16.3	-0.0001	0.0061	-	22.1	-0.0004	0.0057

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel.

Table 4-19: Strain Values for 3NS-TB3

Initial Strain: -0.0181%			Relative Strain Values										
Gauge			Shot 1			Shot 2				Shot 3			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	<i>ε_{max}</i>	<i>t_{max}</i>	<i>ε_{res}</i>	<i>ε_{max}</i>	<i>t_{max}</i>	<i>ε_{res}</i>	<i>Σε_{res}</i>	<i>ε_{max}</i>	<i>t_{max}</i>	<i>ε_{res}</i>	<i>Σε_{res}</i>
	mm		%	ms	%	%	ms	%	%	%	ms	%	%
C1	-665	L	-0.0152	25.4	-0.0012	-0.0348	17.1	0.0061	0.0049	-0.0533	16.8	0.0043	0.0093
C2	-332.5	L	0.0339	15.0	0.0040	0.1583	16.7	0.0195	0.0236	0.2241	12.5	0.0053	0.0288
C3	0	L	-0.0381	15.5	-0.0044	-0.0854	8.6	-0.0063	-0.0107	-0.0873	7.1	-0.0310	-0.0417
T3A	0	U	0.0326	17.7	0.0026	0.2104	17.7	0.0171	0.0197	0.3143	11.8	-0.0095	0.0101
C4	332.5	L	0.0268	20.4	0.0012	0.1732	17.1	0.0157	0.0169	0.2287	10.2	0.0009	0.0178
C5	665	L	-0.0380	18.0	-0.0040	-0.0990	16.4	-0.0022	-0.0061	-0.1431	12.0	0.0027	-0.0035
T5	665	U	-0.0220	20.3	-0.0027	-0.0878	16.9	-0.0029	-0.0056	-0.1404	14.8	-0.0054	-0.0110
C6	-970	L	0.0347	18.9	0.0046	0.2351	17.1	0.0222	0.0268	-	-	-	-

Gauge			Shot 4				Shot 5			
<i>Name</i>	<i>X*</i>	<i>Location**</i>	<i>ε_{max}</i>	<i>t_{max}</i>	<i>ε_{res}</i>	<i>Σε_{res}</i>	<i>ε_{max}</i>	<i>t_{max}</i>	<i>ε_{res}</i>	<i>Σε_{res}</i>
	mm		%	ms	%	%	%	ms	%	%
C1	-665	L	-0.0509	26.4	0.0106	0.0199	0.0391	21.2	0.0174	0.0373
C2	-332.5	L	0.1667	15.0	0.0030	0.0318	-	-	-	-
C3	0	L	-0.0680	21.1	-0.0277	-0.0694	-0.1389	21.0	-0.0337	-0.1031
T3A	0	U	0.2610	10.8	0.0087	0.0189	0.2759	11.0	-0.0348	-0.0160
C4	332.5	L	0.2150	9.5	-0.0080	0.0098	0.2135	9.5	-0.0288	-0.0190
C5	665	L	-0.1373	14.4	-0.0008	-0.0043	-0.1541	11.4	0.1169	0.1126
T5	665	U	-0.1358	12.0	0.0058	-0.0052	-0.1189	11.8	0.0424	0.0372

* Distance along the length from the mid-height of the column to the placement of the strain gauge where positive is up.

** Placement of the strain gauge where: U-unloaded side longitudinal steel; L-loaded side longitudinal steel.



a) After Shot 1



b) After Shot 2



c) After Shot 3 – Side

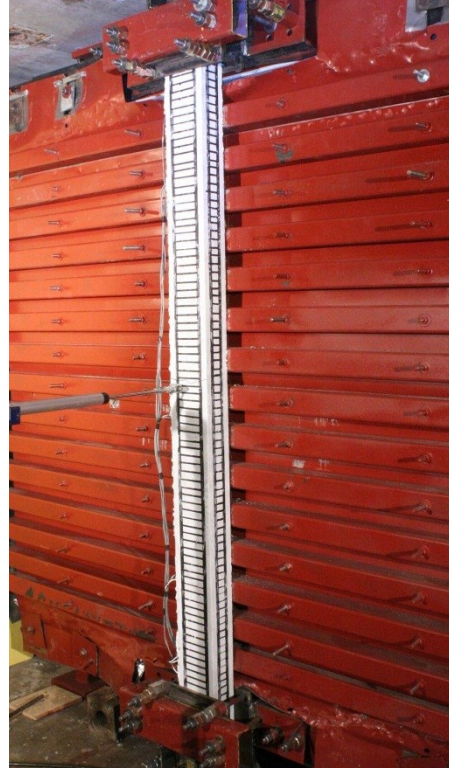


d) After Shot 3 – Front

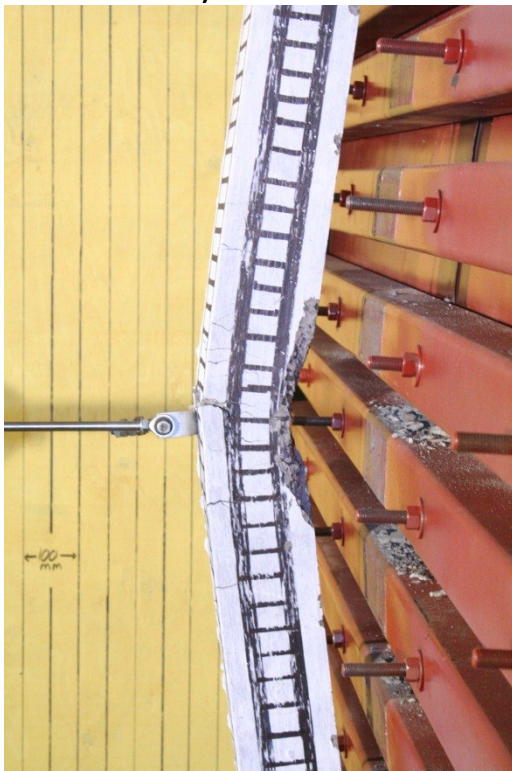
Figure 4-1: Column 1NS-C After Shot 1, Shot 2, and Shot 3



a) After Shot 1



b) After Shot 2



c) After Shot 3 – Side

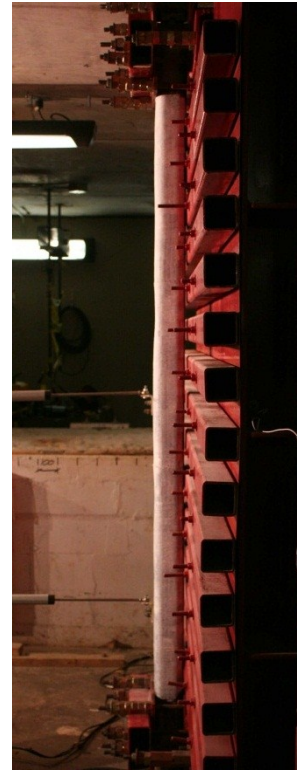


d) After Shot 3 – Front

Figure 4-2: Column 1S-C After Shot 1, Shot 2, and Shot 3



a) Before Shot 1



b) After Shot 1

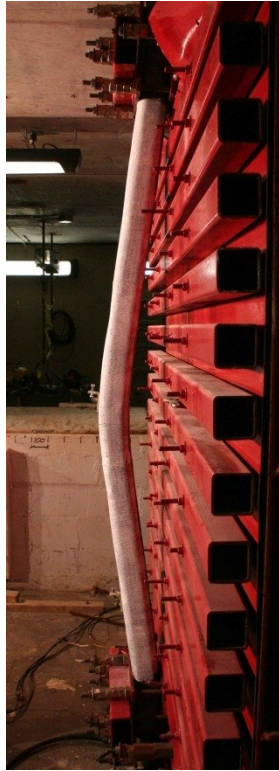


c) After Shot 2 – Side



d) After Shot 2 – Front

Figure 4-3: Column 1NS-J Before Testing and After Shot 1 and Shot 2



a) After Shot 3 – Side



b) After Shot 3 – Front

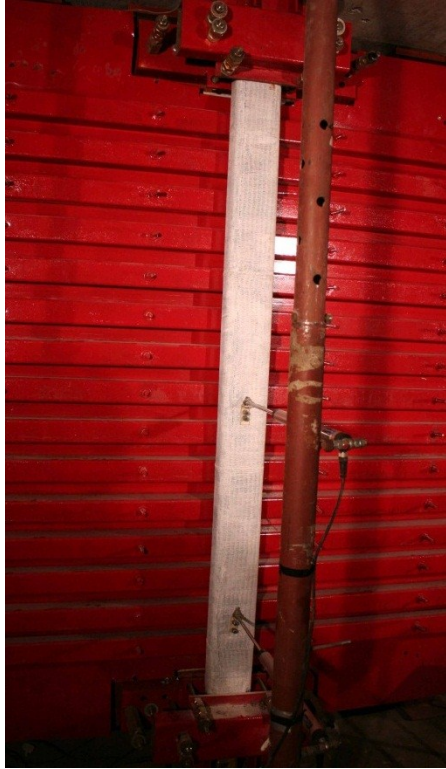


c) After Shot 3 – Mid-Height



d) After Shot 3 – Support

Figure 4-4: Column 1NS-J After Shot 3



a) Before Shot 1



b) After Shot 1



c) After Shot 2 – Side



d) After Shot 2 – Front

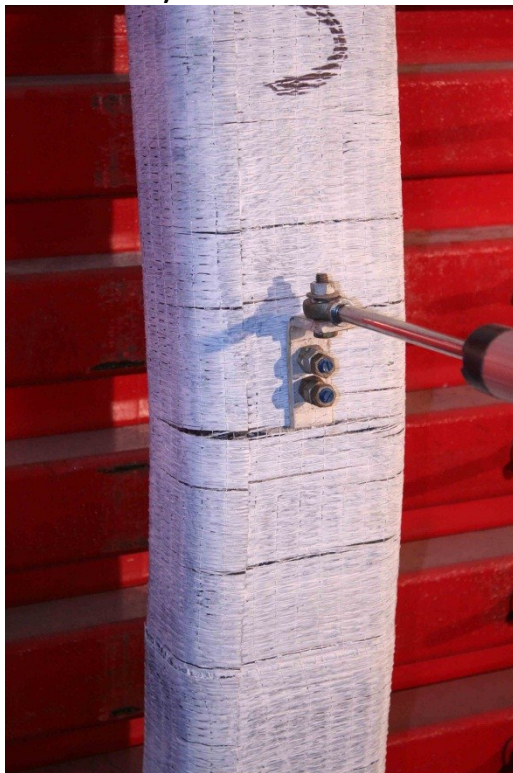
Figure 4-5: Column 1S-J Before Testing and After Shot 1 and Shot 2



a) After Shot 3 – Side



b) After Shot 3 – Front



c) After Shot 3 – Mid-Height

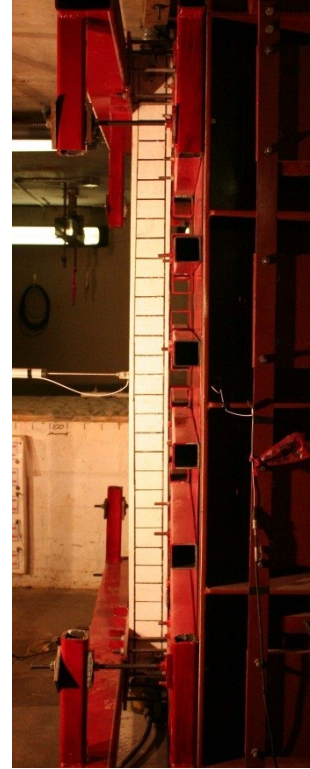


d) After Shot 3 – Support

Figure 4-6: Column 1S-J After Shot 3



a) Before Shot 1



b) After Shot 1



c) After Shot 2 – Side

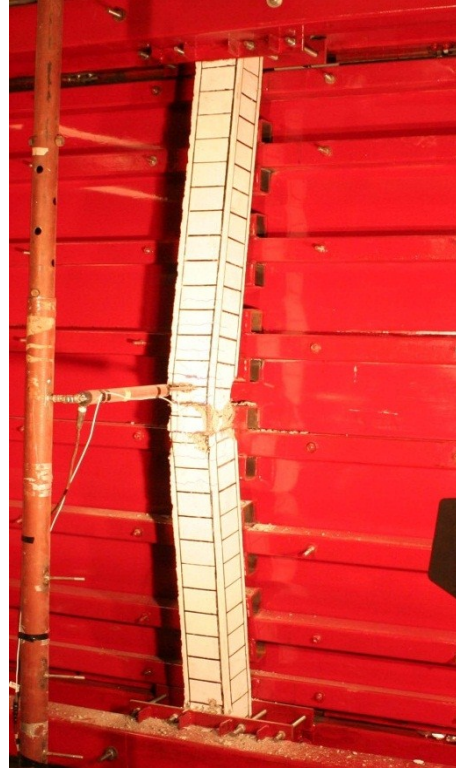


d) After Shot 2 – Front

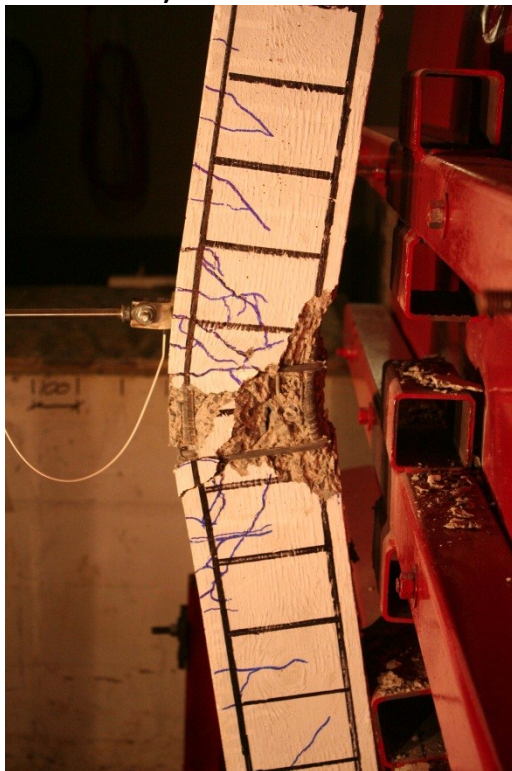
Figure 4-7: Column 2NS-C Before Testing and After Shot 1 and Shot 2



a) After Shot 3 – Side



b) After Shot 3 – Front

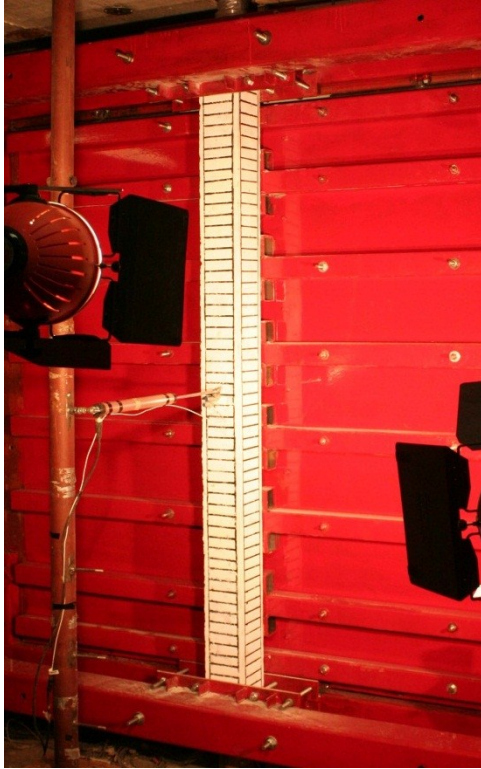


c) After Shot 3 – Mid-Height



d) After Shot 3 – Support

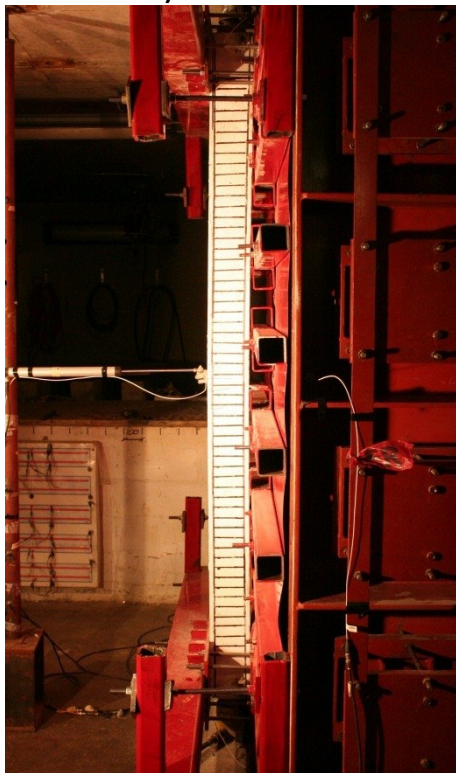
Figure 4-8: Column 2NS-C After Shot 3



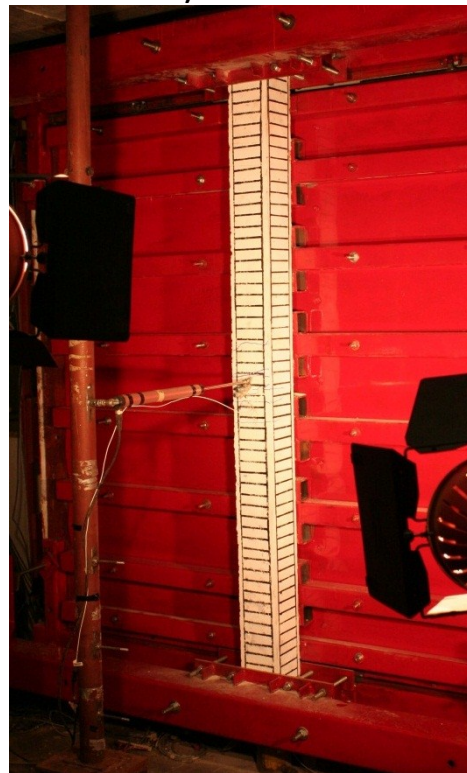
a) Before Shot 1



b) After Shot 1

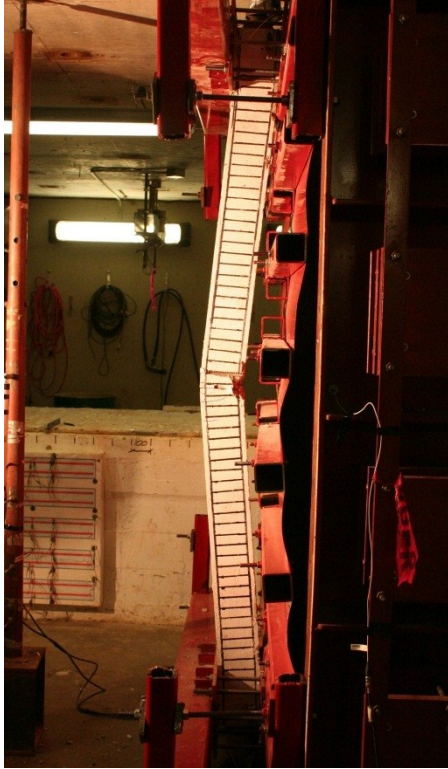


c) After Shot 2 – Side



d) After Shot 2 – Front

Figure 4-9: Column 2S-C Before Testing and After Shot 1 and Shot 2



a) After Shot 3 – Side



b) After Shot 3 – Front

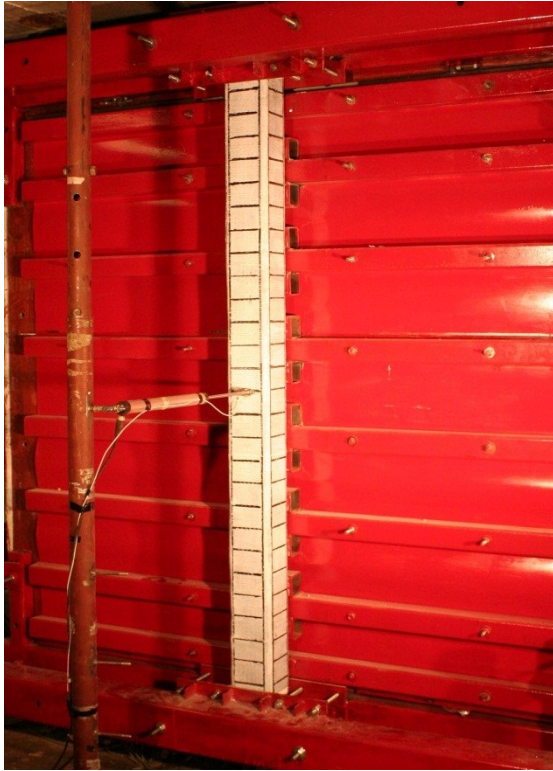


c) After Shot 3 – Mid-Height

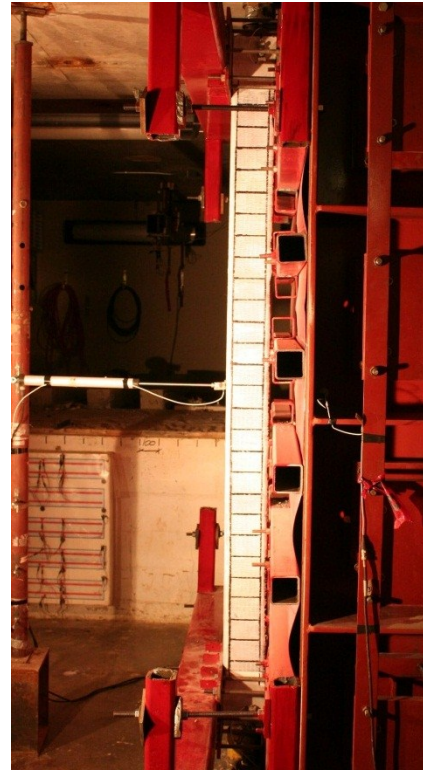


d) After Shot 3 – Support

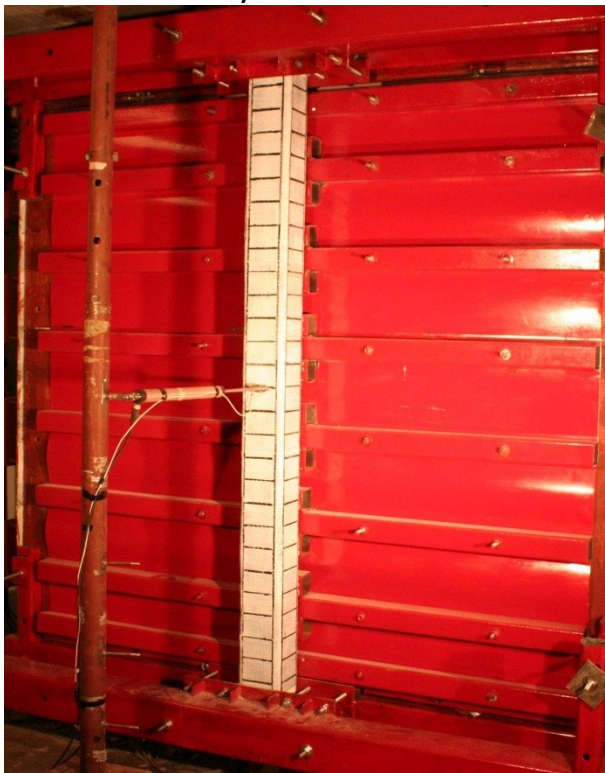
Figure 4-10: Column 2S-C After Shot 3



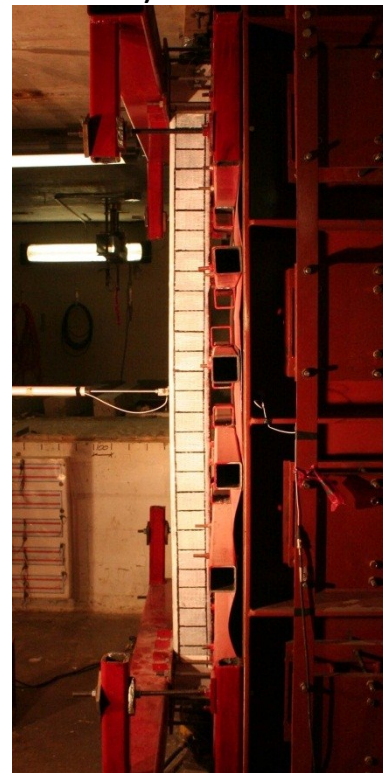
a) Before Shot 1



b) After Shot 1



c) After Shot 2 – Side



d) After Shot 2 – Front

Figure 4-11: Column 2NS-J Before Testing and After Shot 1 and Shot 2



a) After Shot 3 – Side



b) After Shot 3 – Front

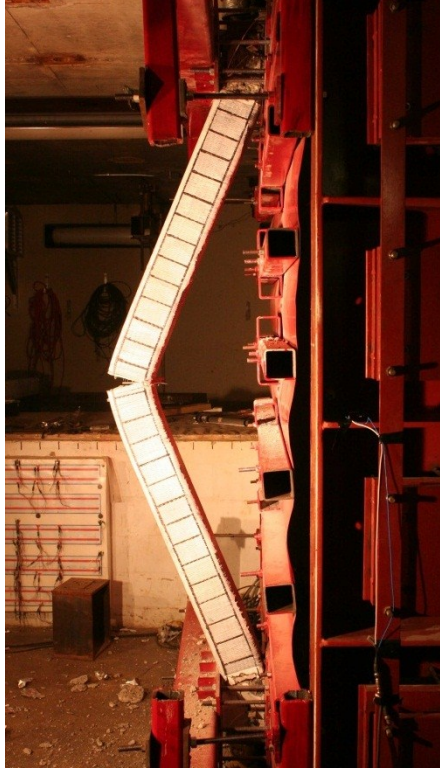


c) After Shot 3 – Mid-Height



d) After Shot 3 – Support

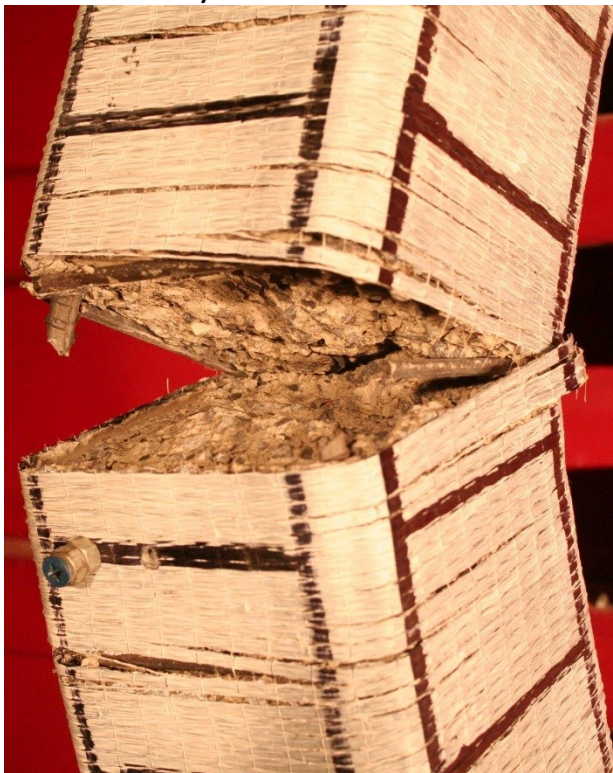
Figure 4-12: Column 2NS-J After Shot 3



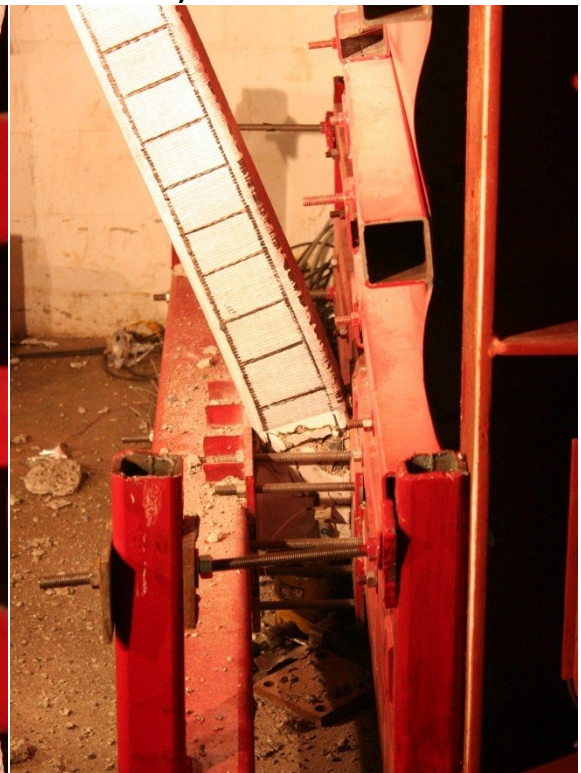
a) After Shot 4 – Side



b) After Shot 4 – Front

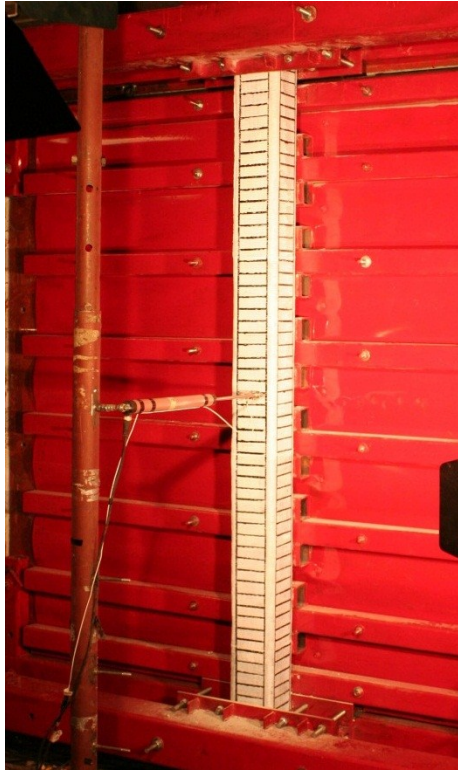


c) After Shot 4 – Mid-Height

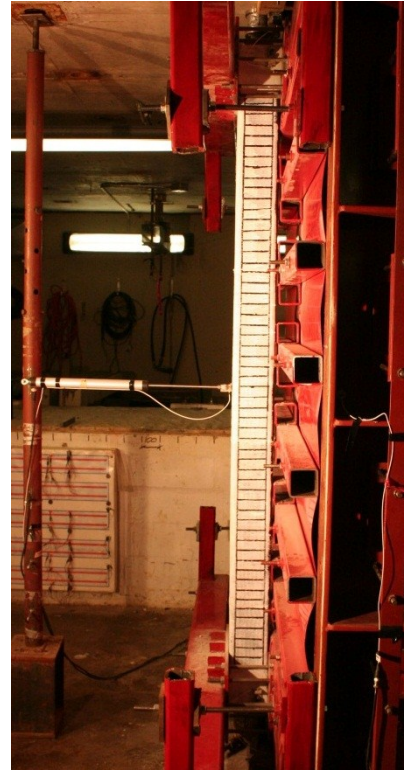


d) After Shot 4 – Support

Figure 4-13: Column 2NS-J After Shot 4



a) Before Shot 1



b) After Shot 1

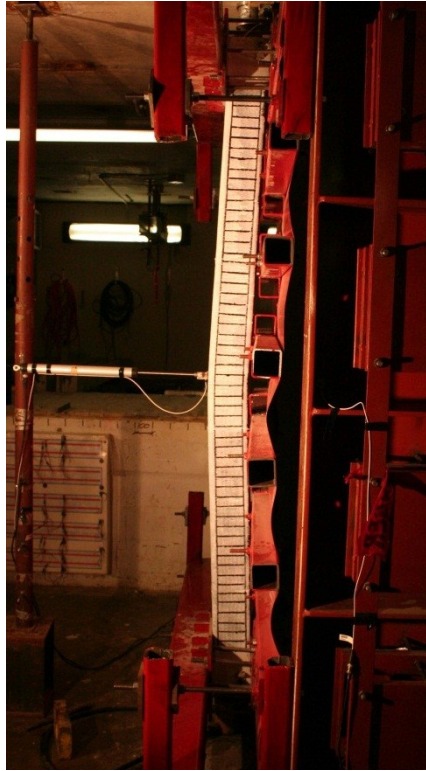


c) After Shot 2 – Side



d) After Shot 2 – Front

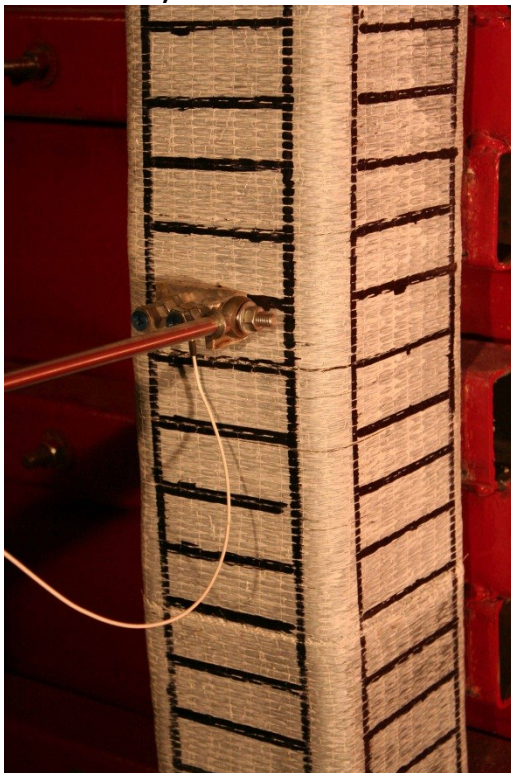
Figure 4-14: Column 2S-J Before Testing and After Shot 1 and Shot 2



a) After Shot 3 – Side



b) After Shot 3 – Front

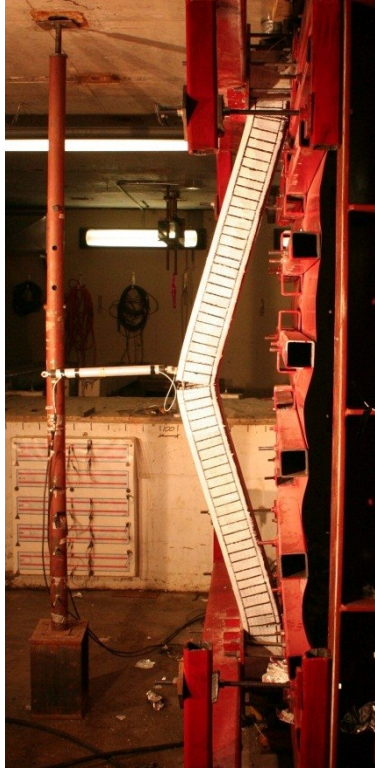


c) After Shot 3 – Mid-Height



d) After Shot 3 – Support

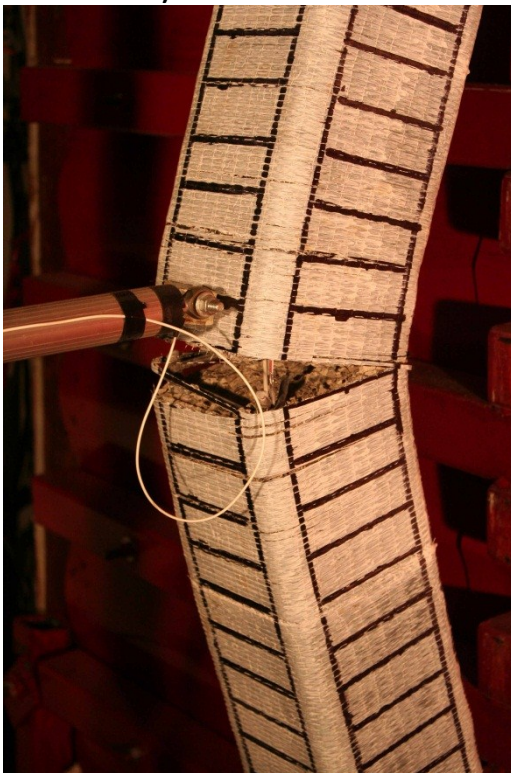
Figure 4-15: Column 2S-J After Shot 3



a) After Shot 4 – Side



b) After Shot 4 – Front

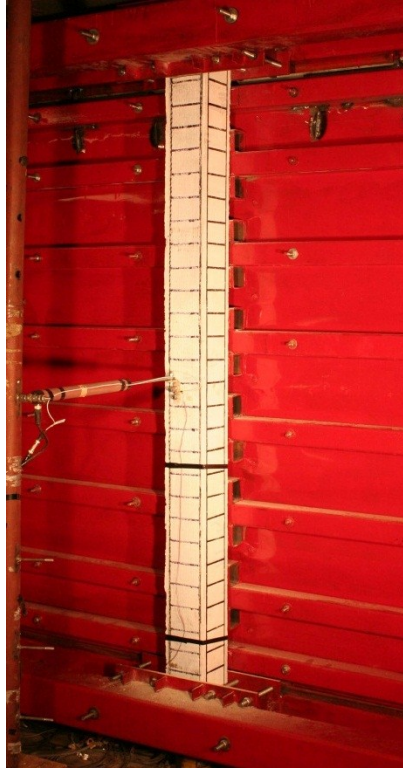


c) After Shot 4 – Mid-Height

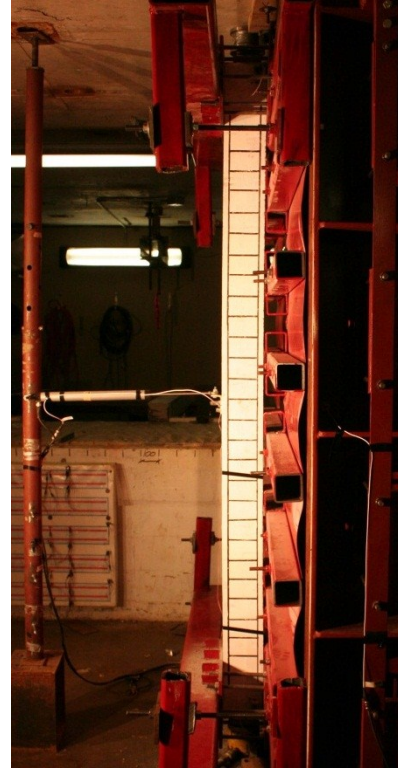


d) After Shot 4 – Support

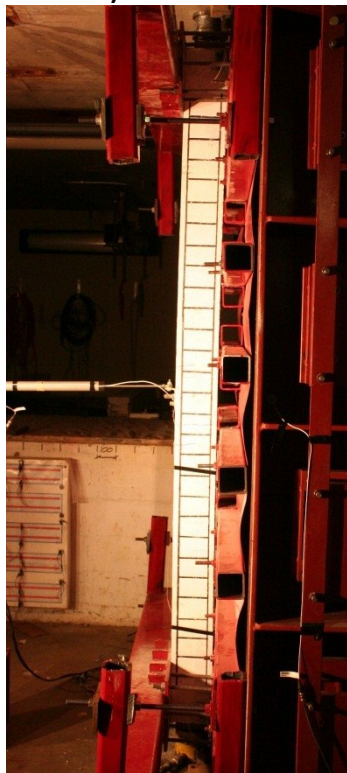
Figure 4-16: Column 2S-J After Shot 4



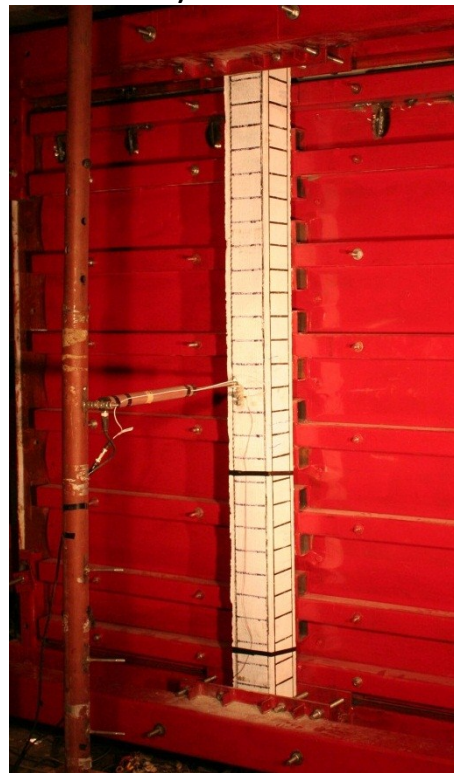
a) Before Shot 1



b) After Shot 1



c) After Shot 2 - Side

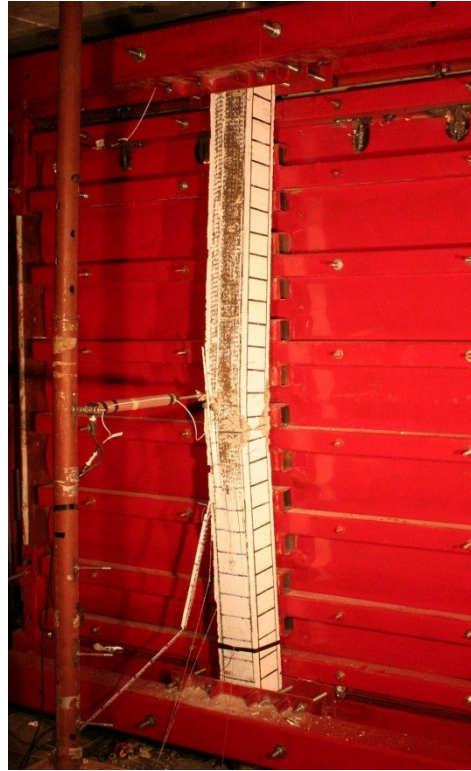


d) After Shot 2 - Front

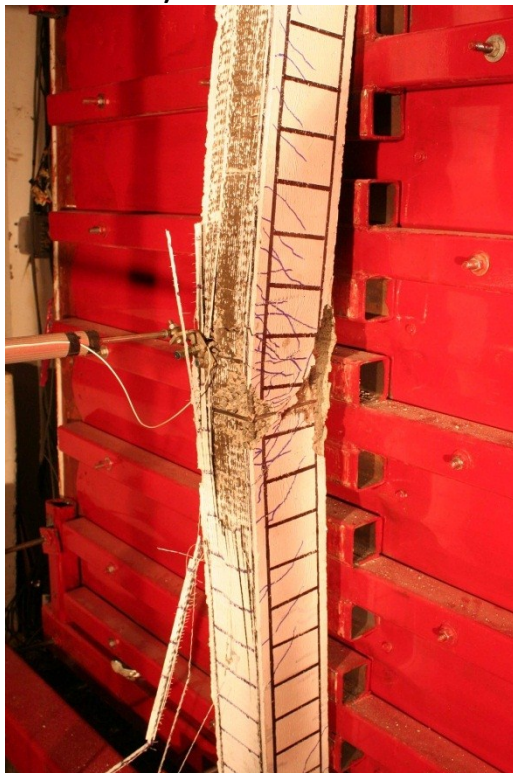
Figure 4-17: Column 2NS-R Before Testing and After Shot 1 and Shot 2



a) After Shot 3 – Side



b) After Shot 3 – Front

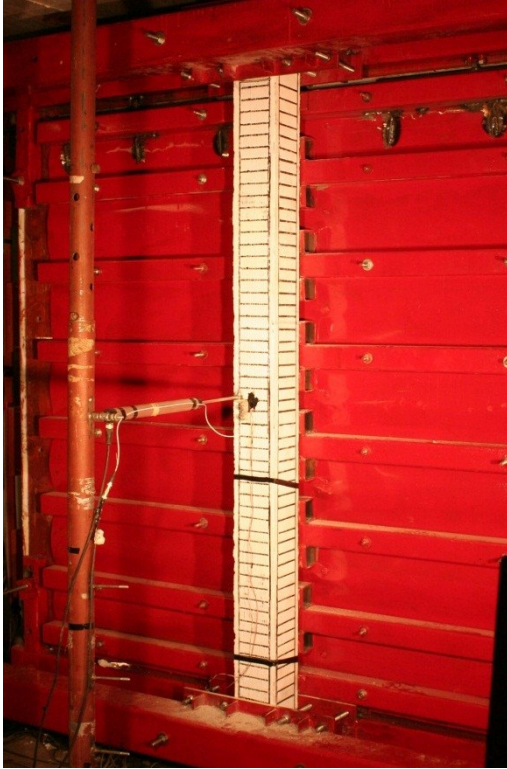


c) After Shot 3 – Mid-Height

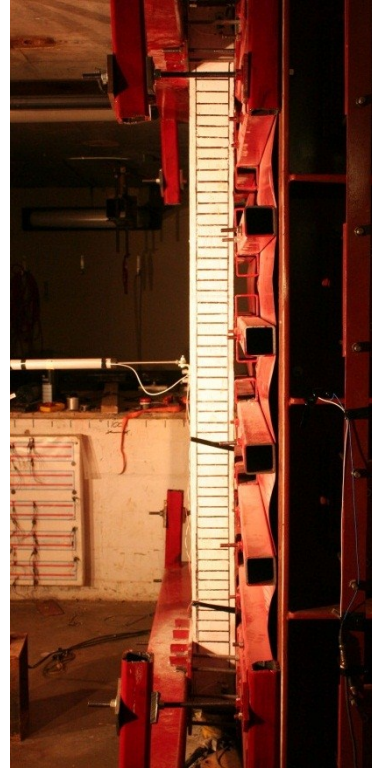


d) After Shot 3 – Support

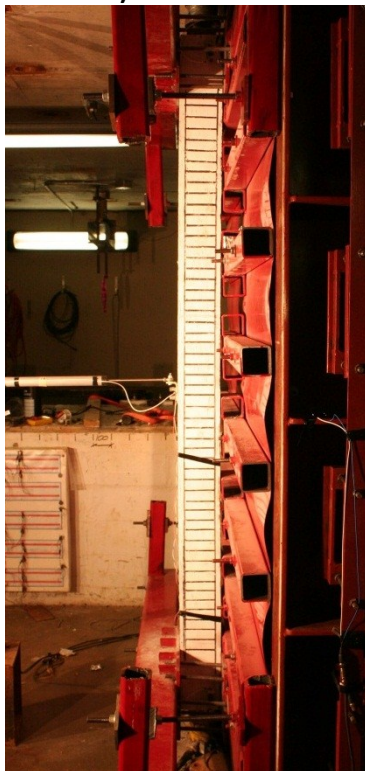
Figure 4-18: Column 2NS-R After Shot 3



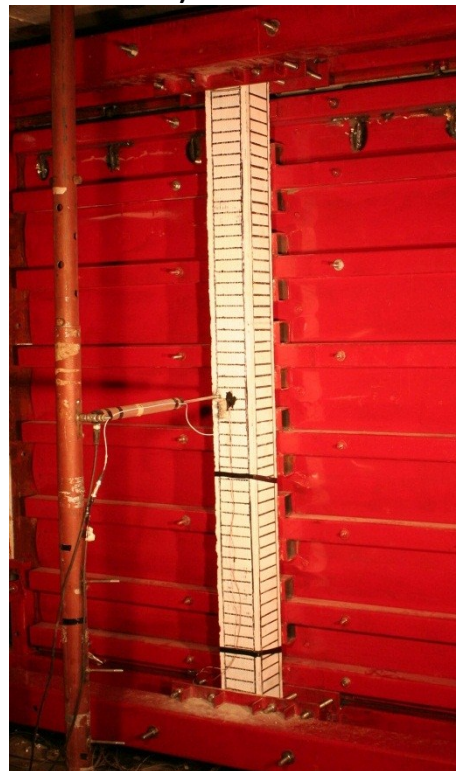
a) Before Shot 1



b) After Shot 1



c) After Shot 2 – Side

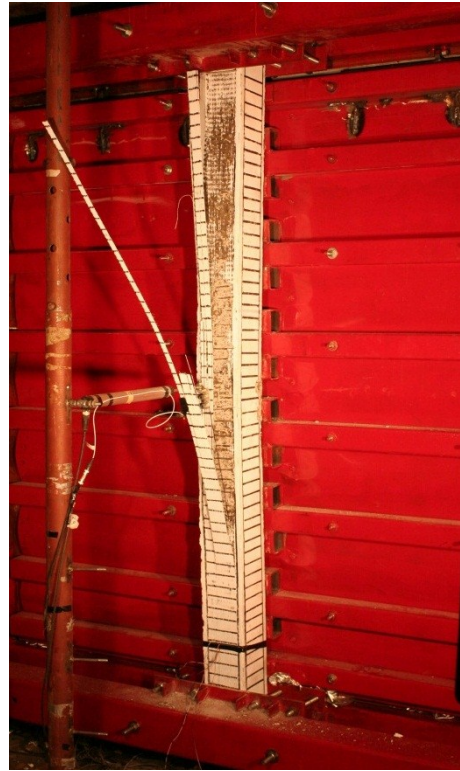


d) After Shot 2 – Front

Figure 4-19: Column 2S-R Before Testing and After Shot 1 and Shot 2



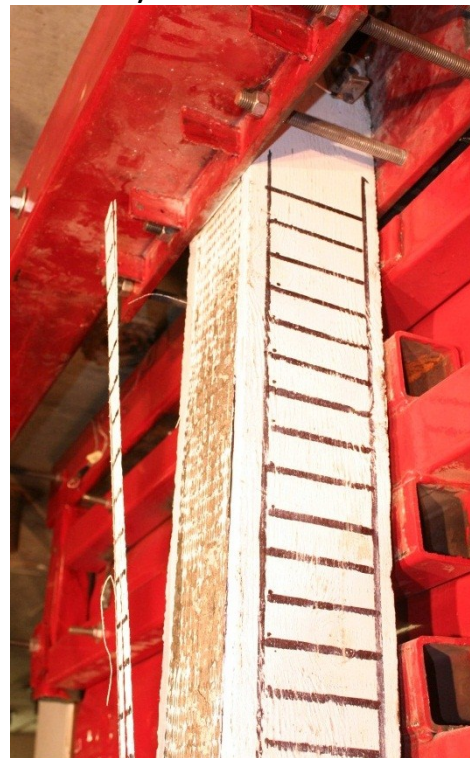
a) After Shot 3 – Side



b) After Shot 3 – Front



c) After Shot 3 – Mid-Height

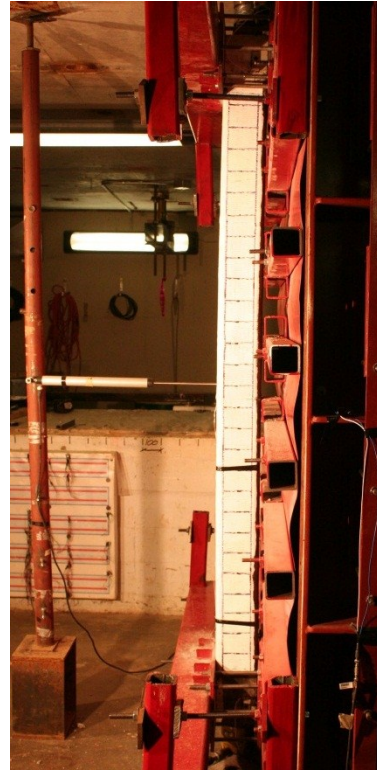


d) After Shot 3 – Support

Figure 4-20: Column 2S-R After Shot 3



a) Before Shot 1



b) After Shot 1



c) After Shot 2 – Side

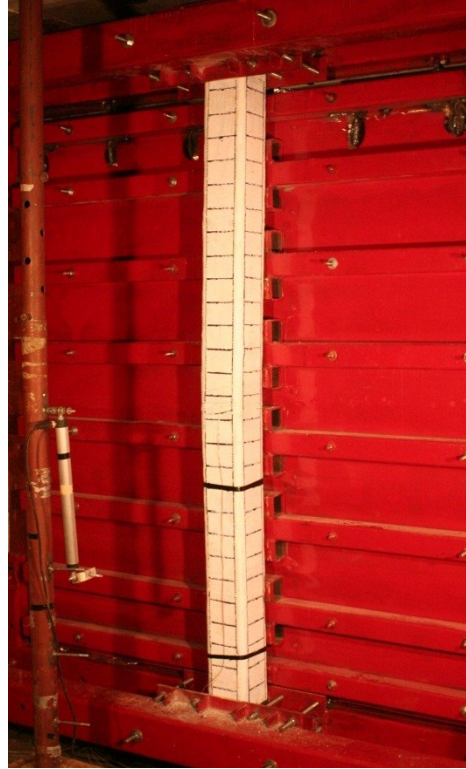


d) After Shot 2 – Front

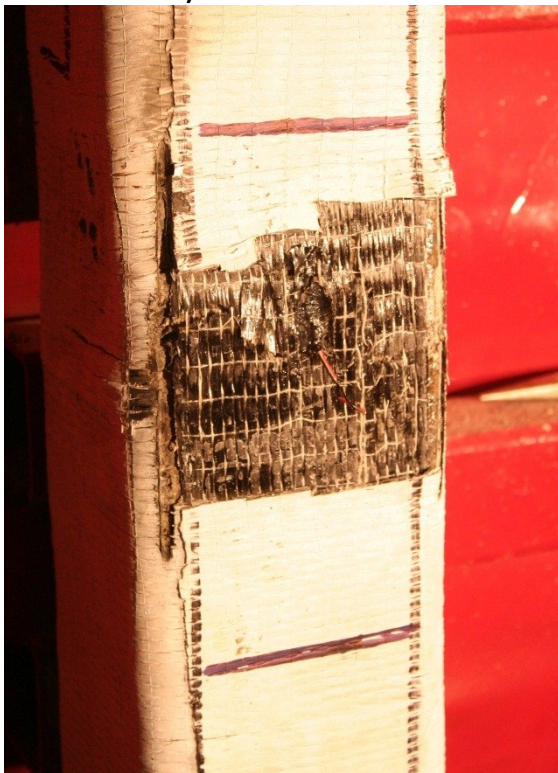
Figure 4-21: Column 2NS-JR Before Testing and After Shot 1 and Shot 2



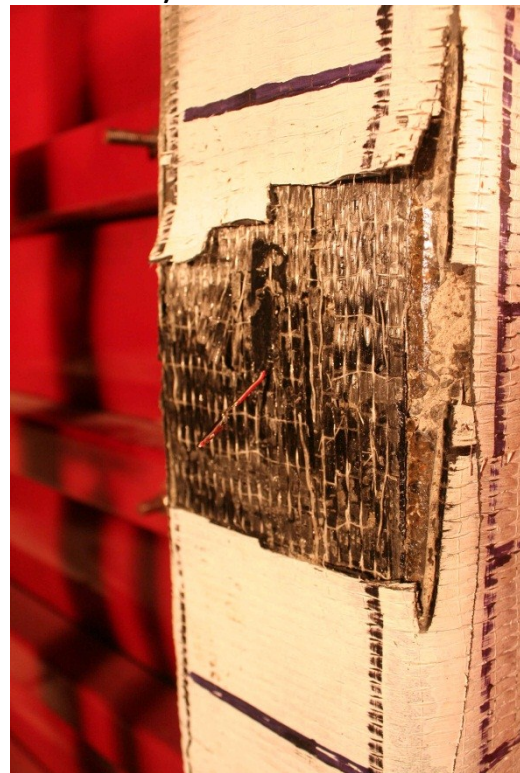
a) After Shot 3 – Side



b) After Shot 3 – Front



c) After Shot 3 – Mid-Height

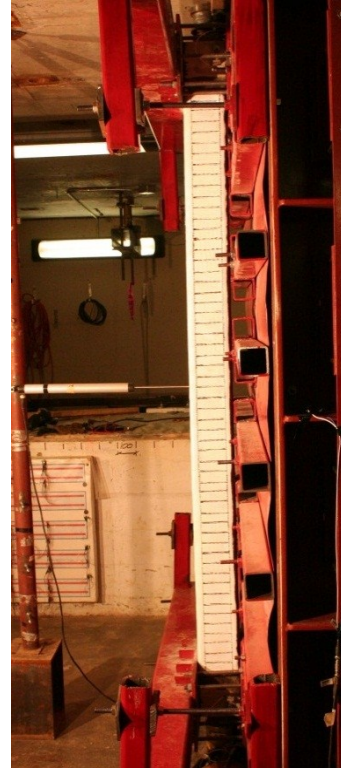


d) After Shot 3 – Mid-Height

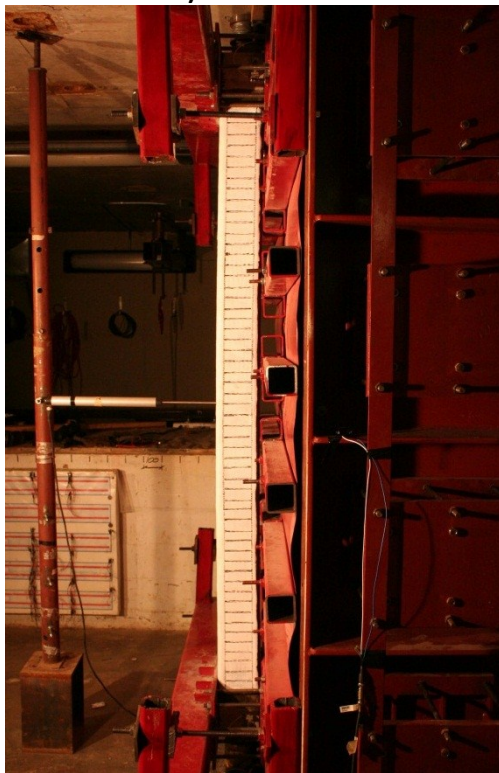
Figure 4-22: Column 2NS-JR After Shot 3



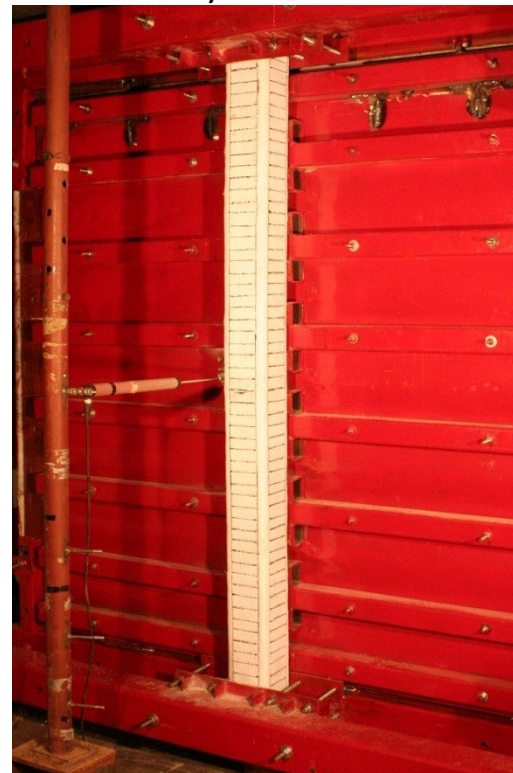
a) Before Shot 1



b) After Shot 1

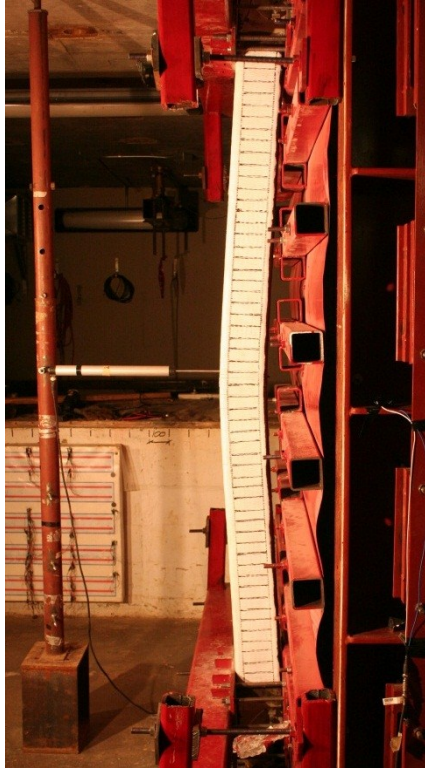


c) After Shot 2 – Side

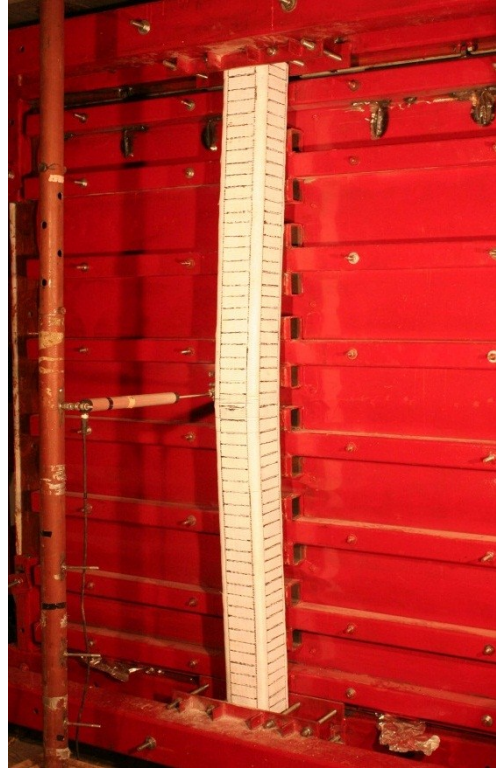


d) After Shot 2 – Front

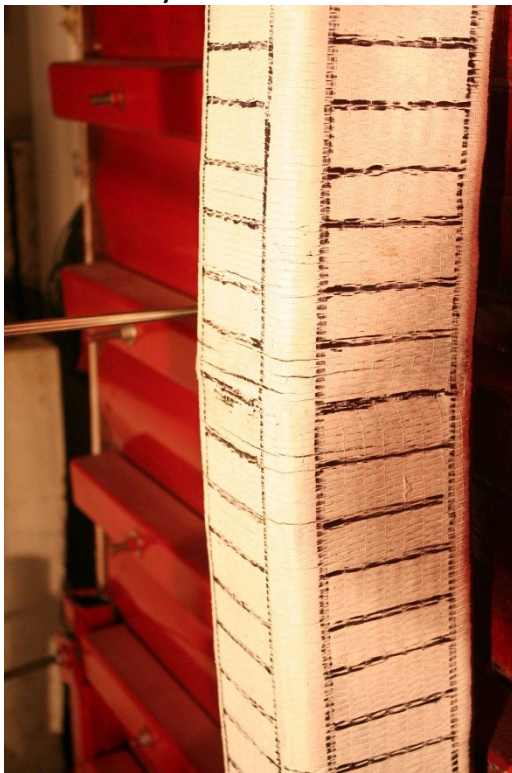
Figure 4-23: Column 2S-JR Before Testing and After Shot 1 and Shot 2



a) After Shot 3 – Side



b) After Shot 3 – Front



c) After Shot 3 – Mid-Height



d) After Shot 3 – Mid-Height

Figure 4-24: Column 2S-JR After Shot 3



a) Before Shot 1



b) After Shot 1



c) After Shot 2 – Side



d) After Shot 2 – Front

Figure 4-25: Column 3NS-C Before Testing and After Shot 1 and Shot 2



a) After Shot 3 – Side



b) After Shot 3 – Front



c) After Shot 3 – Mid-Height

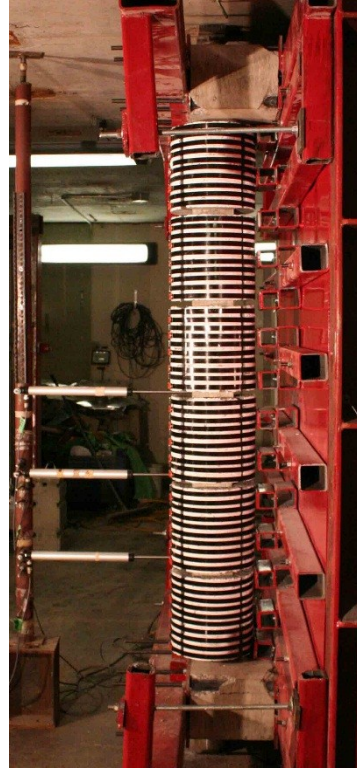


d) After Shot 3 – Support

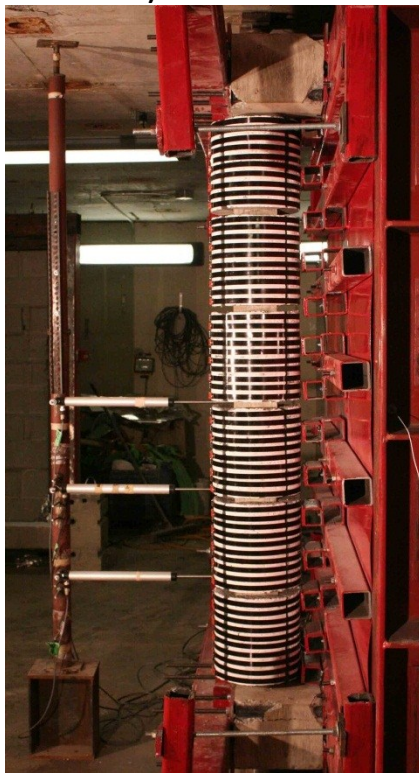
Figure 4-26: Column 3NS-C After Shot 3



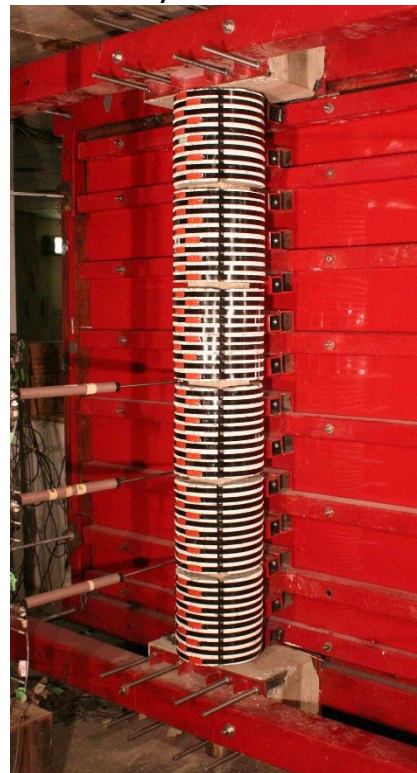
a) Before Shot 1



b) After Shot 1

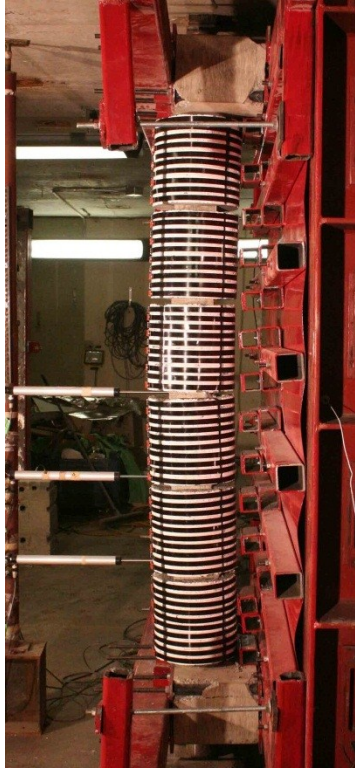


c) After Shot 2 – Side

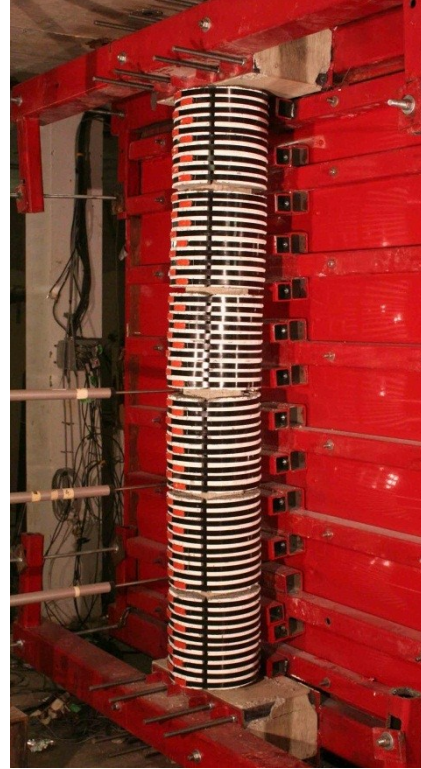


d) After Shot 2 – Front

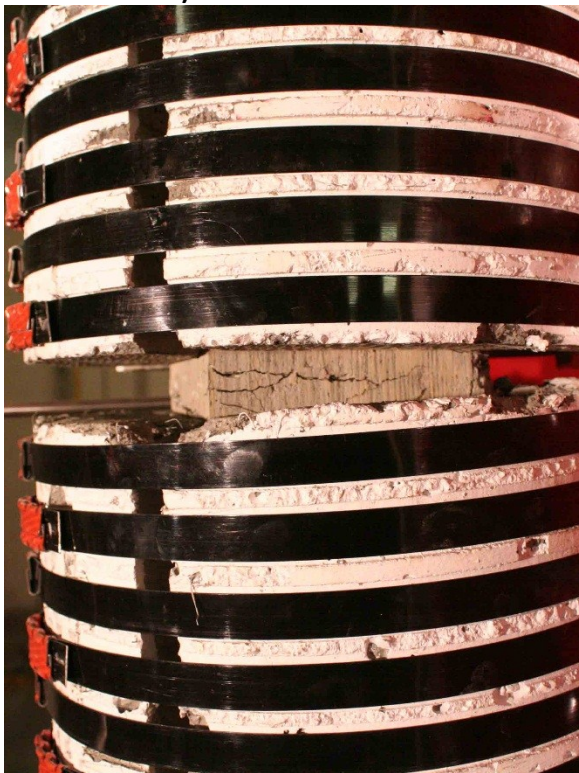
Figure 4-27: Column 3NS-W Before Testing and After Shot 1 and Shot 2



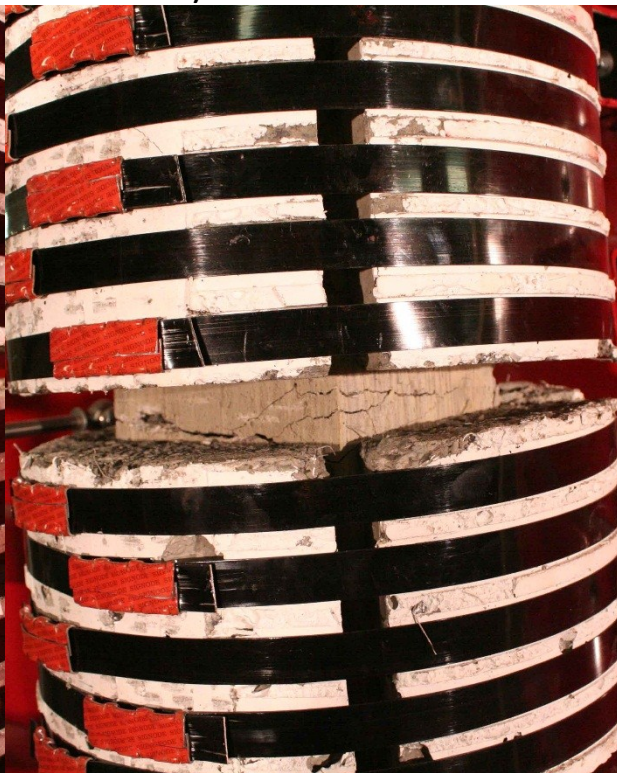
a) After Shot 3 – Side



b) After Shot 3 – Front



c) After Shot 3 – Mid-Height



d) After Shot 3 – Mid-Height

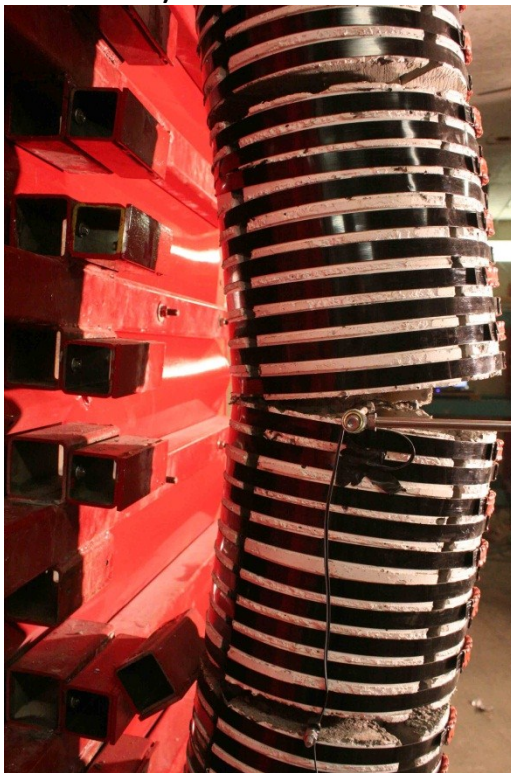
Figure 4-28: Column 3NS-W After Shot 3



a) After Shot 4 – Side



b) After Shot 4 – Front



c) After Shot 4 – Mid-Height

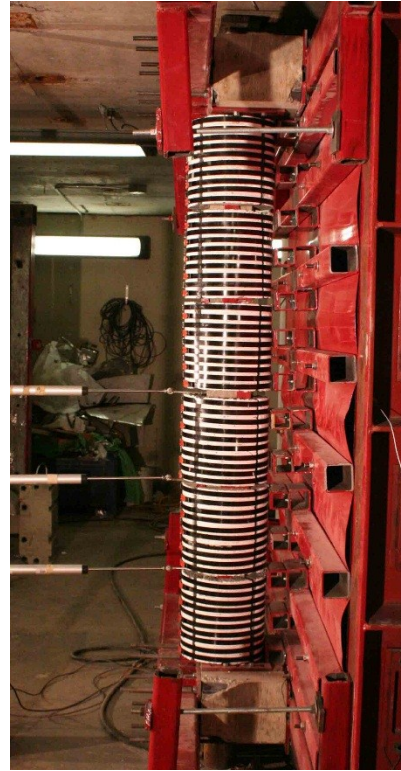


d) After Shot 4 – Side

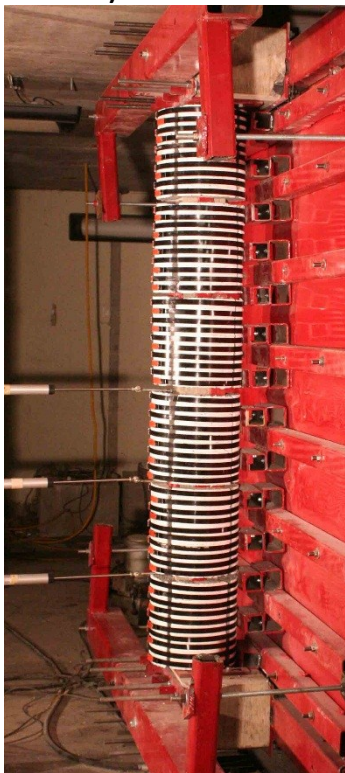
Figure 4-29: Column 3NS-W After Shot 4



a) Before Shot 1



b) After Shot 1



c) After Shot 2 – Side



d) After Shot 2 – Front

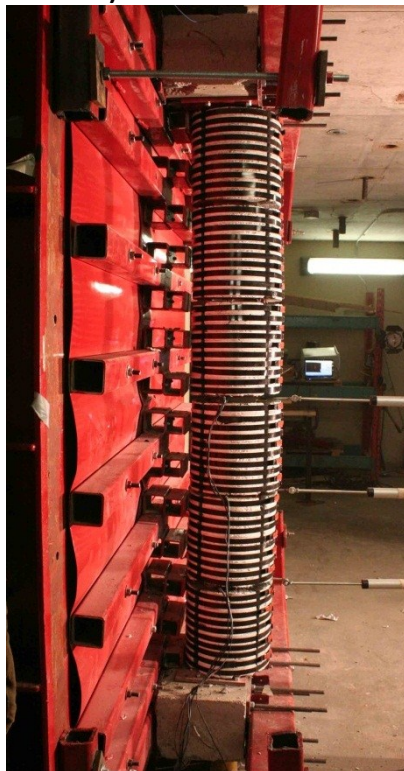
Figure 4-30: Column 3NS-CB Before Testing and After Shot 1 and Shot 2



a) After Shot 3 – Side



b) After Shot 3 – Front

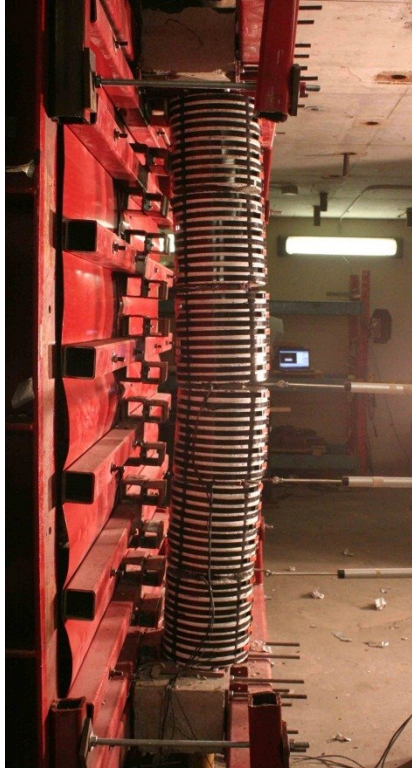


c) After Shot 4 – Side

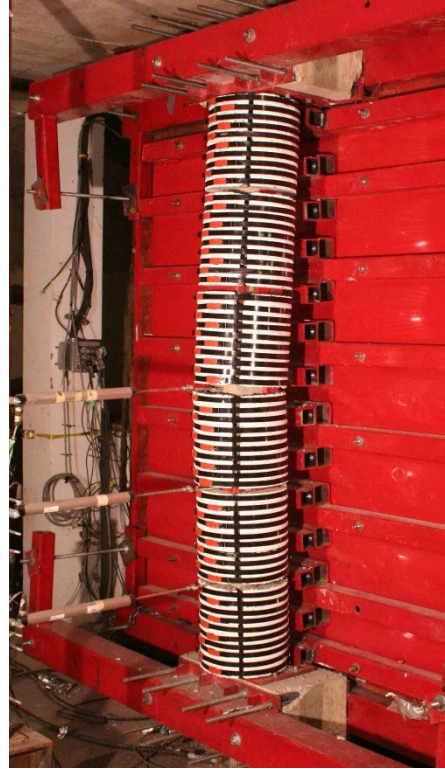


d) After Shot 4 – Front

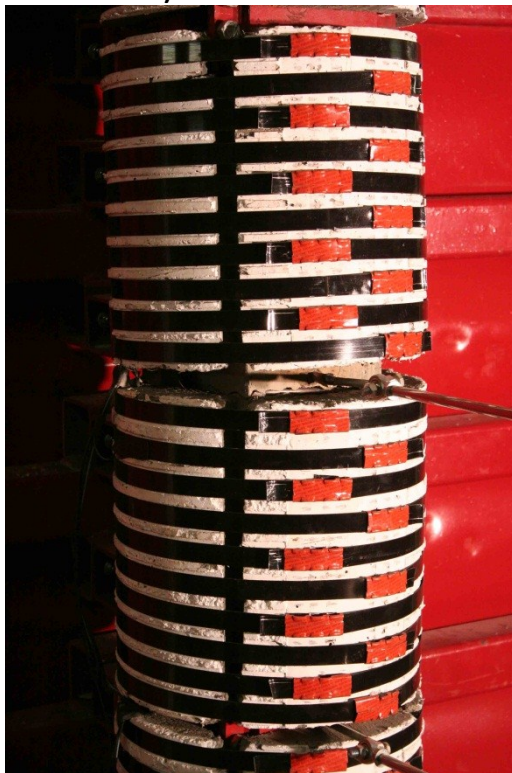
Figure 4-31: Column 3NS-CB After Shot 3 and Shot 4



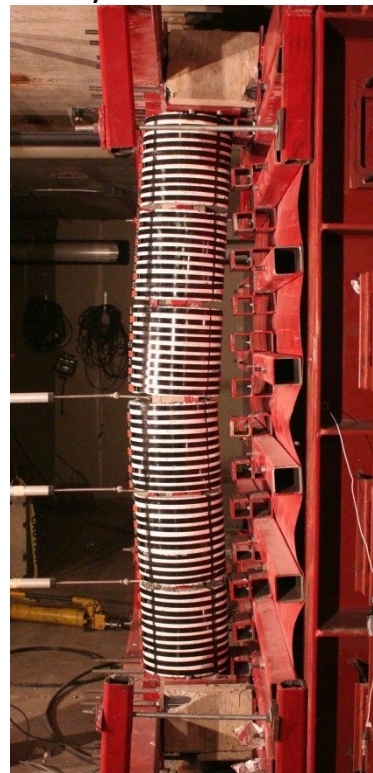
a) After Shot 5 – Side



b) After Shot 5 – Front



c) After Shot 5 – Mid-Height



d) After Shot 5 – Side

Figure 4-32: Column 3NS-CB After Shot 5



a) Before Shot 1



b) After Shot 1



c) After Shot 2 – Side



d) After Shot 2 – Front

Figure 4-33: Column 3NS-TB1 Before Testing and After Shot 1 and Shot 2



a) After Shot 3 – Side



b) After Shot 3 – Front



c) After Shot 4 – Side

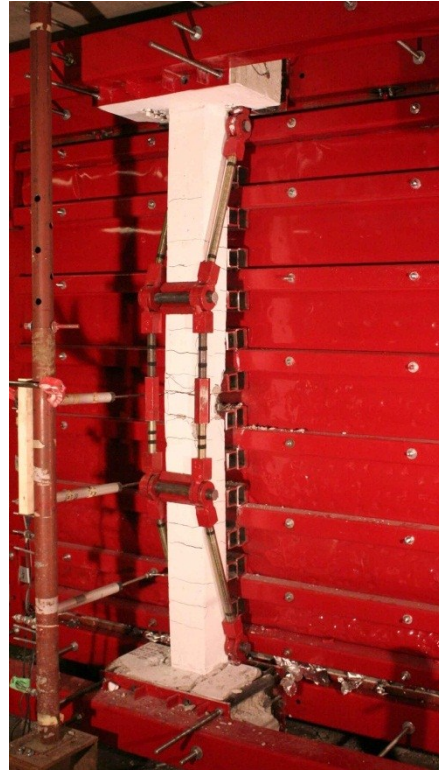


d) After Shot 4 – Front

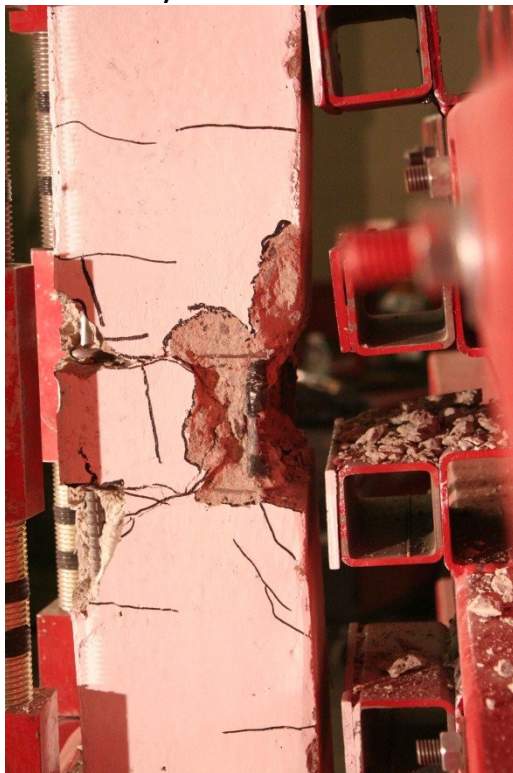
Figure 4-34: Column 3NS-TB1 After Shot 3 and Shot 4



a) After Shot 5 – Side



b) After Shot 5 – Front

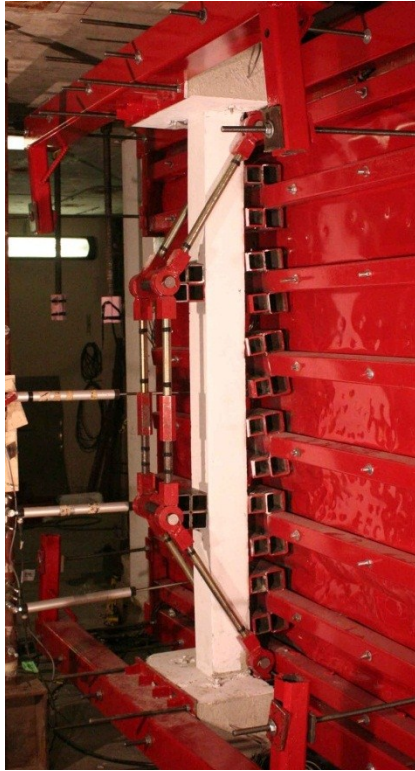


c) After Shot 5 – Mid-Height

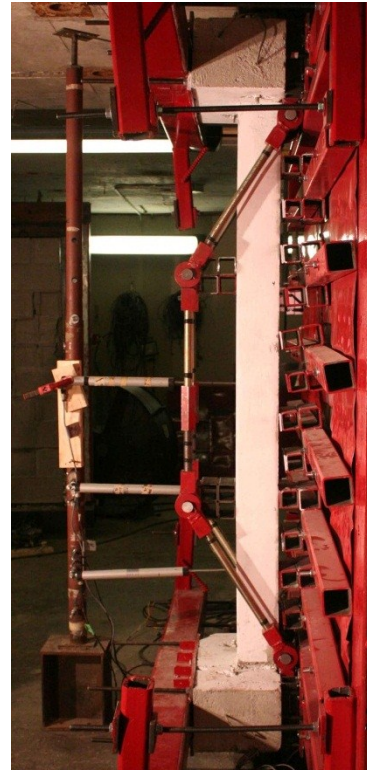


d) After Shot 5 – Support

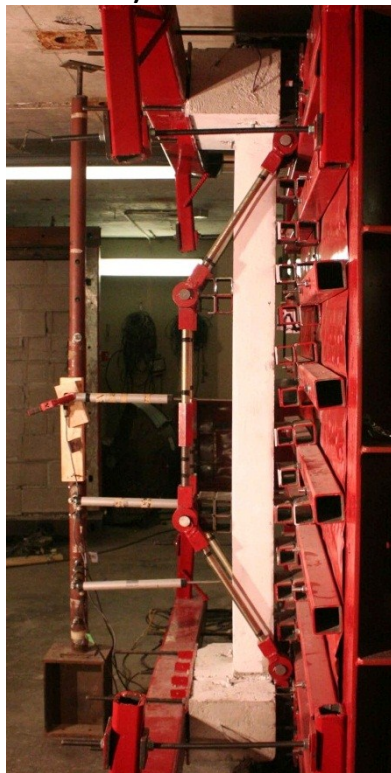
Figure 4-35: Column 3NS-TB1 After Shot 5



a) Before Shot 1



b) After Shot 1

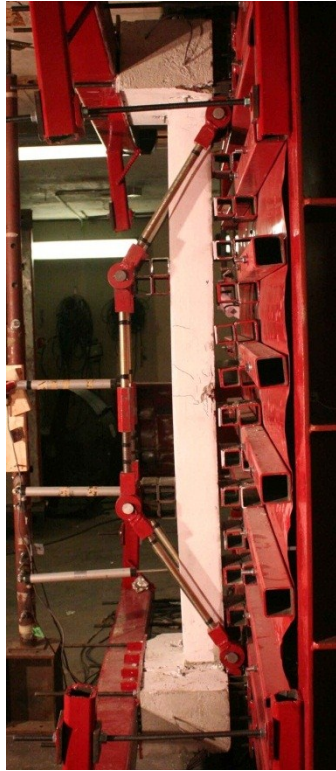


c) After Shot 2 – Side



d) After Shot 2 – Front

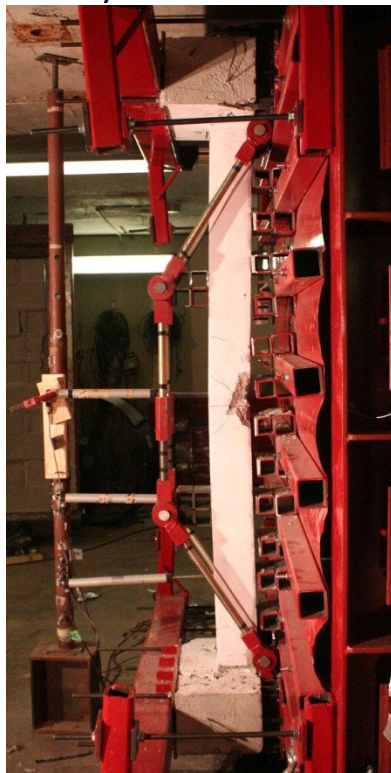
Figure 4-36: Column 3NS-TB2 Before Testing and After Shot 1 and Shot 2



a) After Shot 3 – Side



b) After Shot 3 – Front



c) After Shot 4 – Side

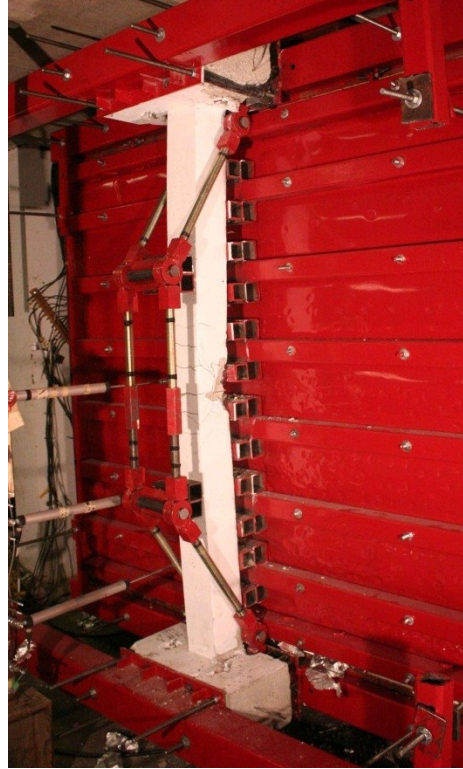


d) After Shot 4 – Front

Figure 4-37: Column 3NS-TB2 After Shot 3 and Shot 4



a) After Shot 5 – Side



b) After Shot 5 – Front



c) After Shot 5 – Front

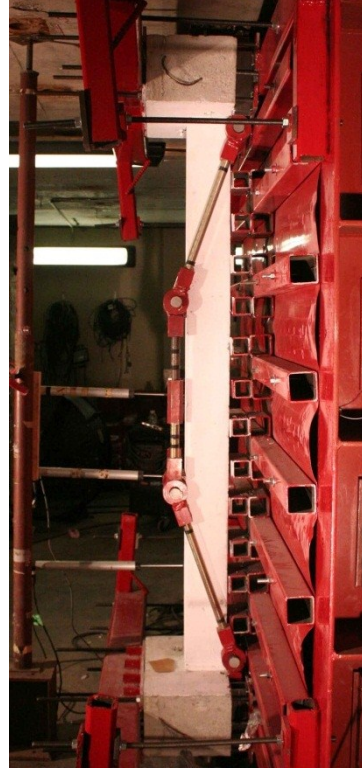


d) After Shot 5 – Support

Figure 4-38: Column 3NS-TB2 After Shot 5



a) Before Shot 1



b) After Shot 1



c) After Shot 2 – Side



d) After Shot 2 – Front

Figure 4-39: Column 3NS-TB3 Before Testing and After Shot 1 and Shot 2



a) After Shot 3 – Side



b) After Shot 3 – Front



c) After Shot 4 – Side

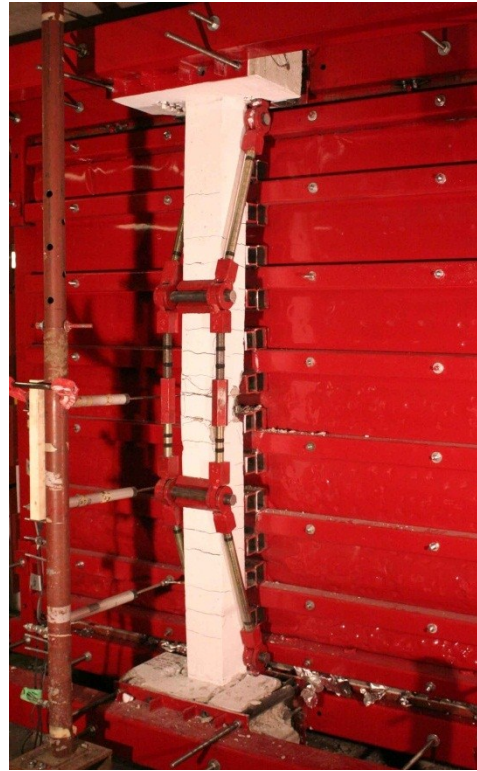


d) After Shot 4 – Front

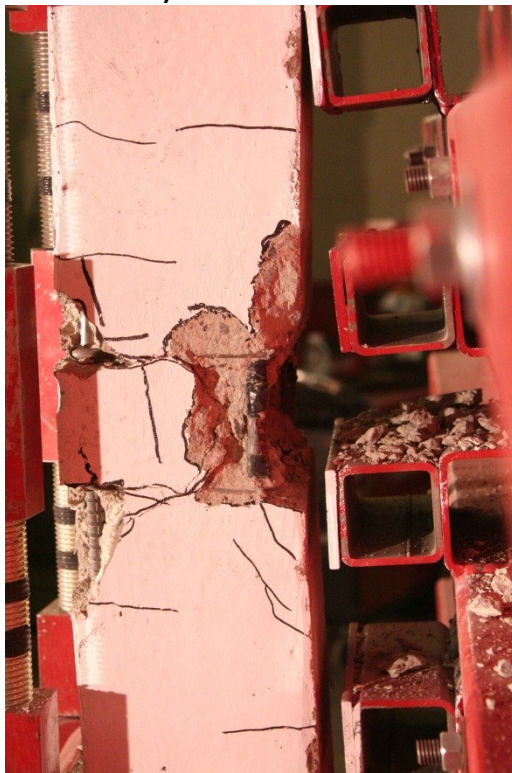
Figure 4-40: Column 3NS-TB3 After Shot 3 and Shot 4



a) After Shot 5 – Side



b) After Shot 5 – Front



c) After Shot 5 – Mid-Height



d) After Shot 5 – Support

Figure 4-41: Column 3NS-TB3 After Shot 5

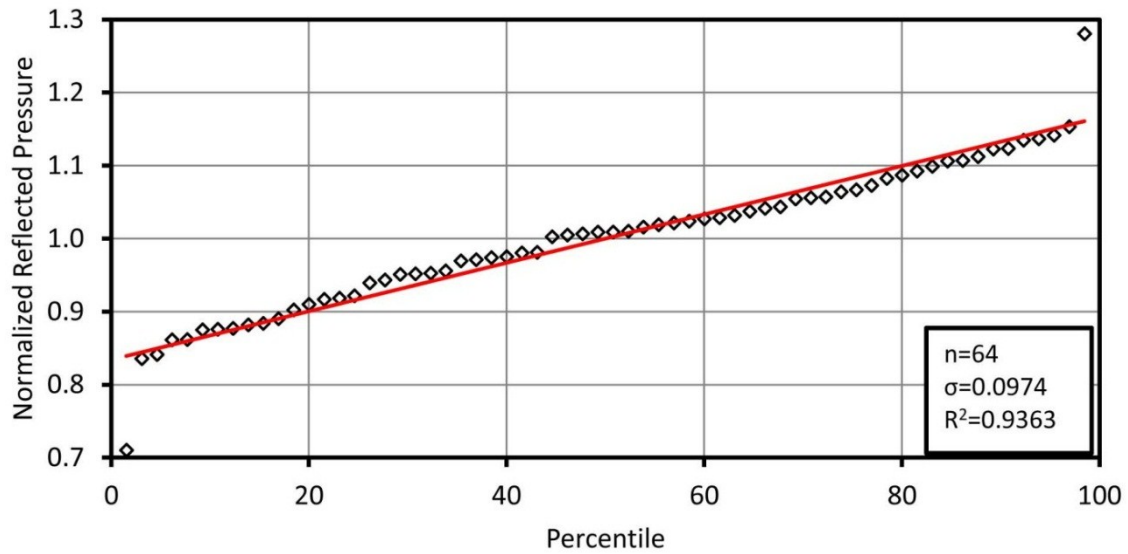


Figure 4-42: Repeatability of Reflected Pressure

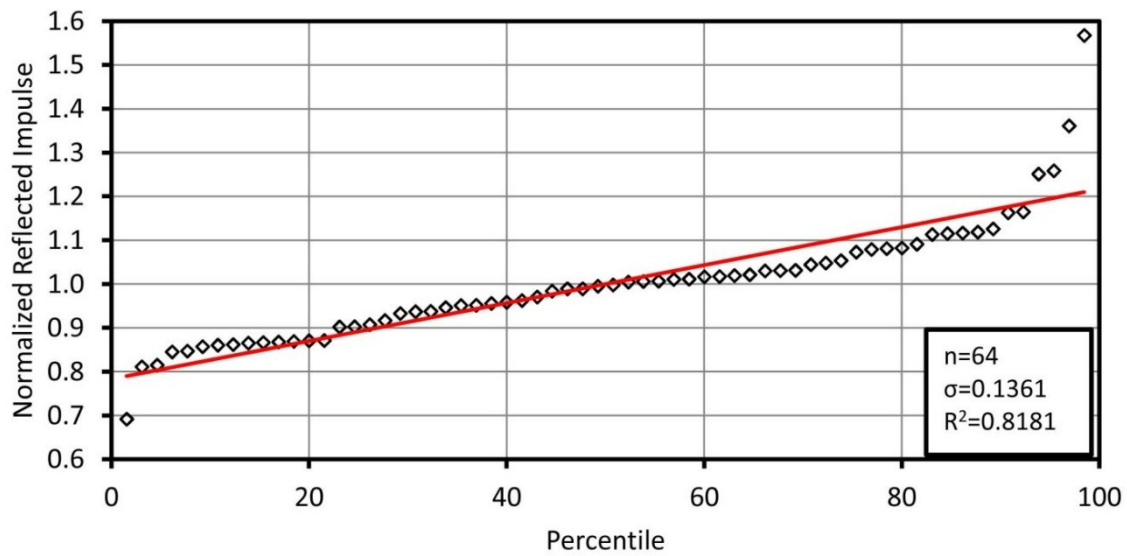


Figure 4-43: Repeatability of Reflected Impulse

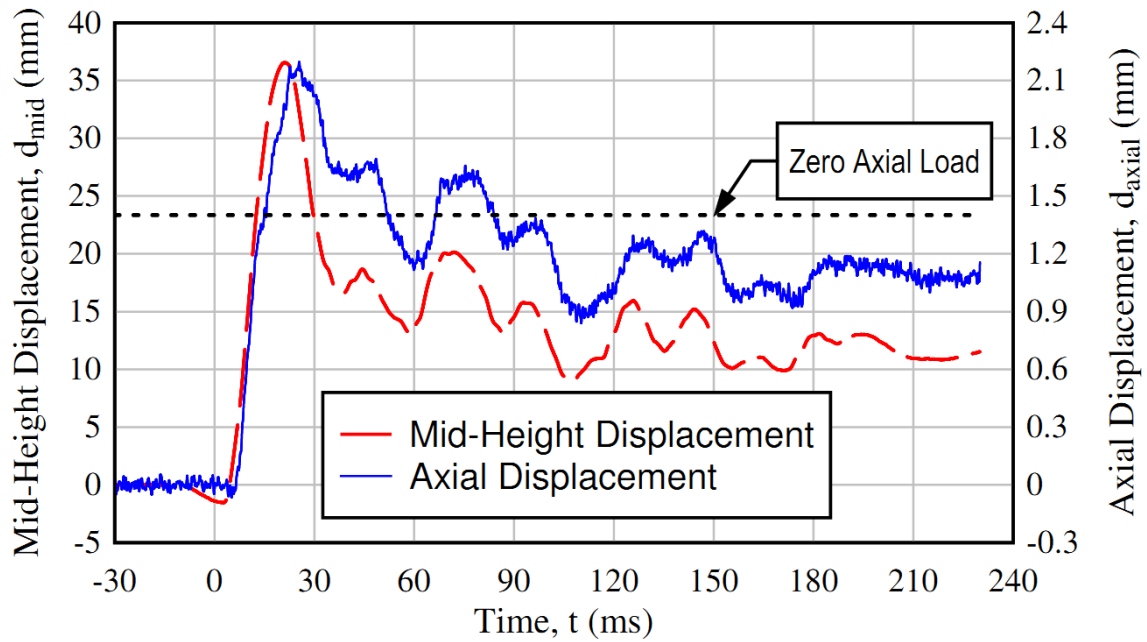


Figure 4-44: Axial Displacements and Mid-Height Displacements for Column 2NS-C-2

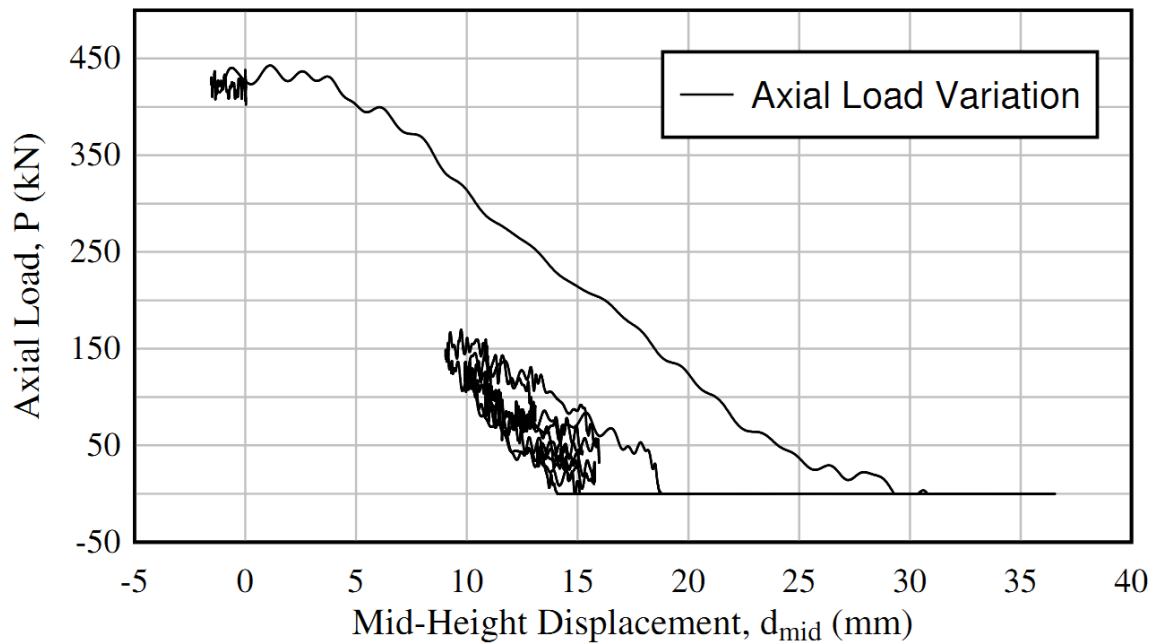


Figure 4-45: Axial Load Variation versus Mid-Height Displacement for Column 2NS-C-2

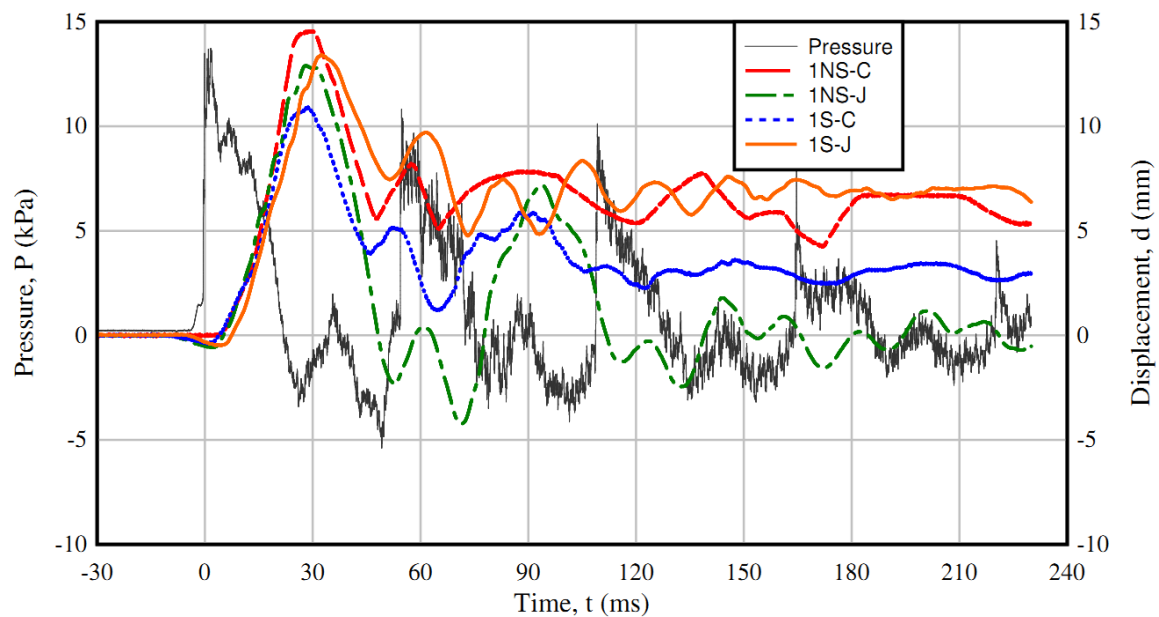


Figure 4-46: Comparison of Displacements - Group 1 Shot 1

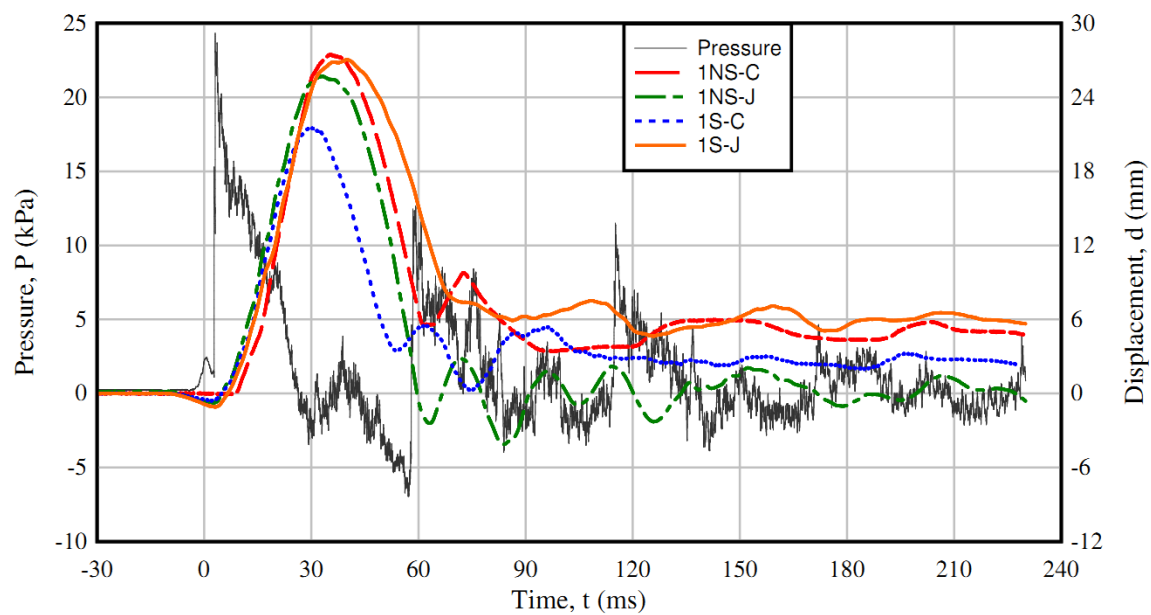


Figure 4-47: Comparison of Displacements - Group 1 Shot 2

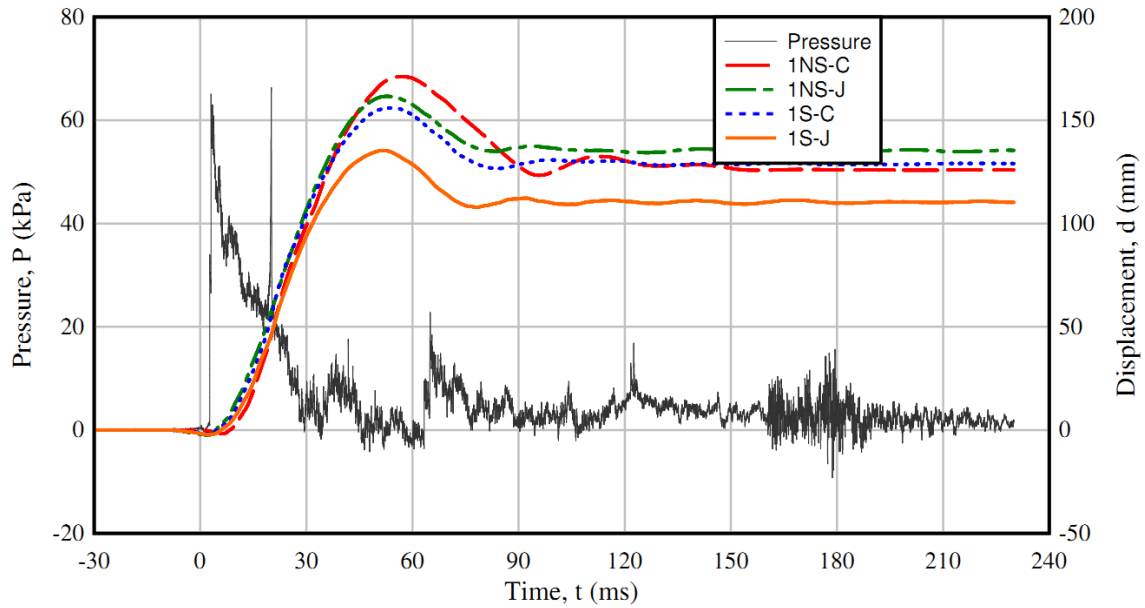


Figure 4-48: Comparison of Displacements - Group 1 Shot 3

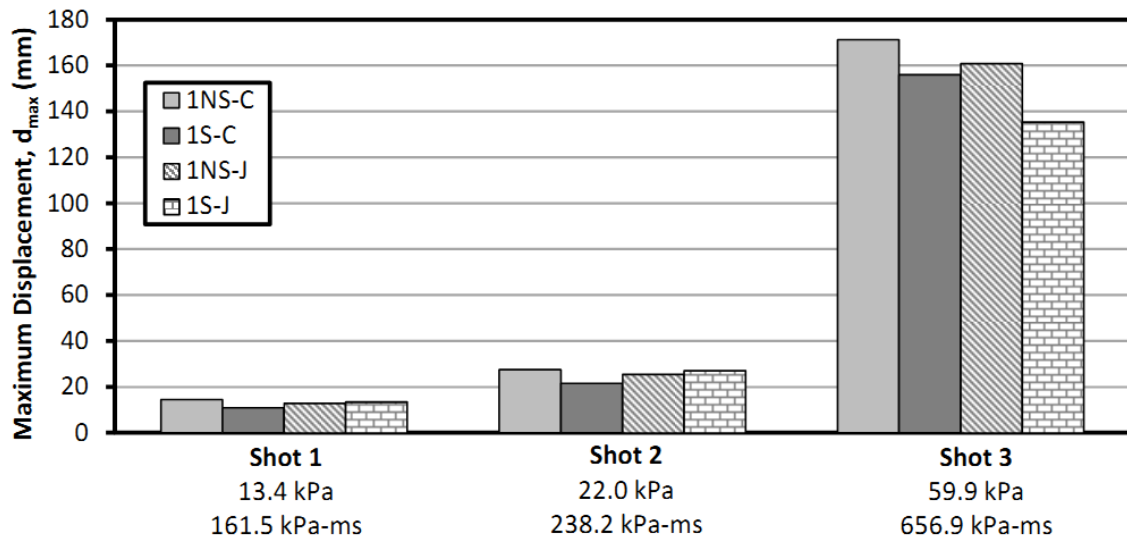


Figure 4-49: Comparison of Maximum Displacements - Group 1

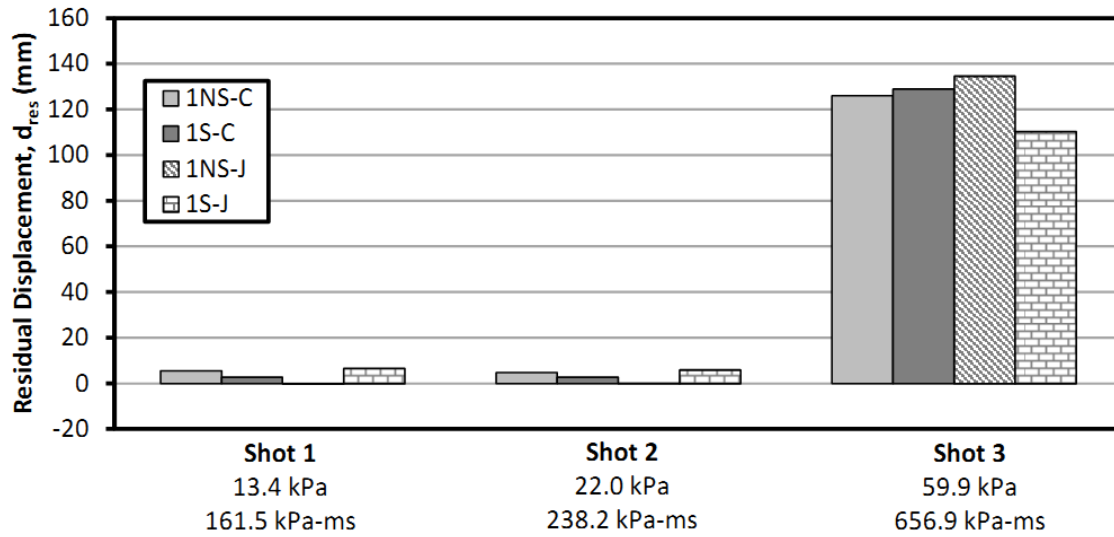


Figure 4-50: Comparison of Residual Displacements - Group 1

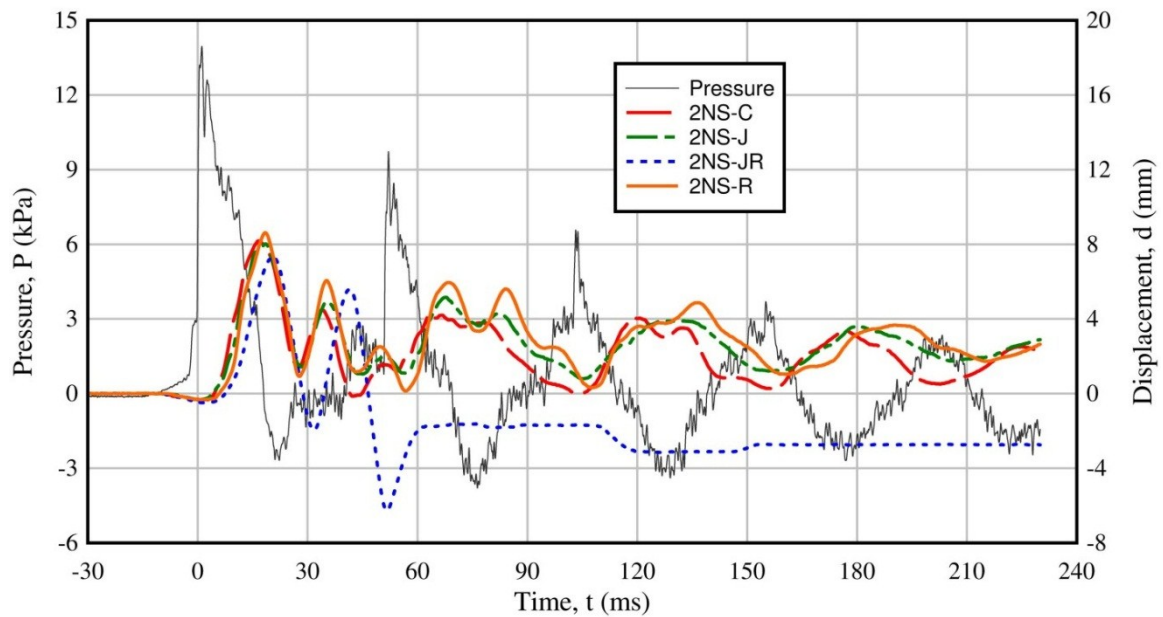


Figure 4-51: Comparison of Displacements - Group 2 Non-Seismic Shot 1

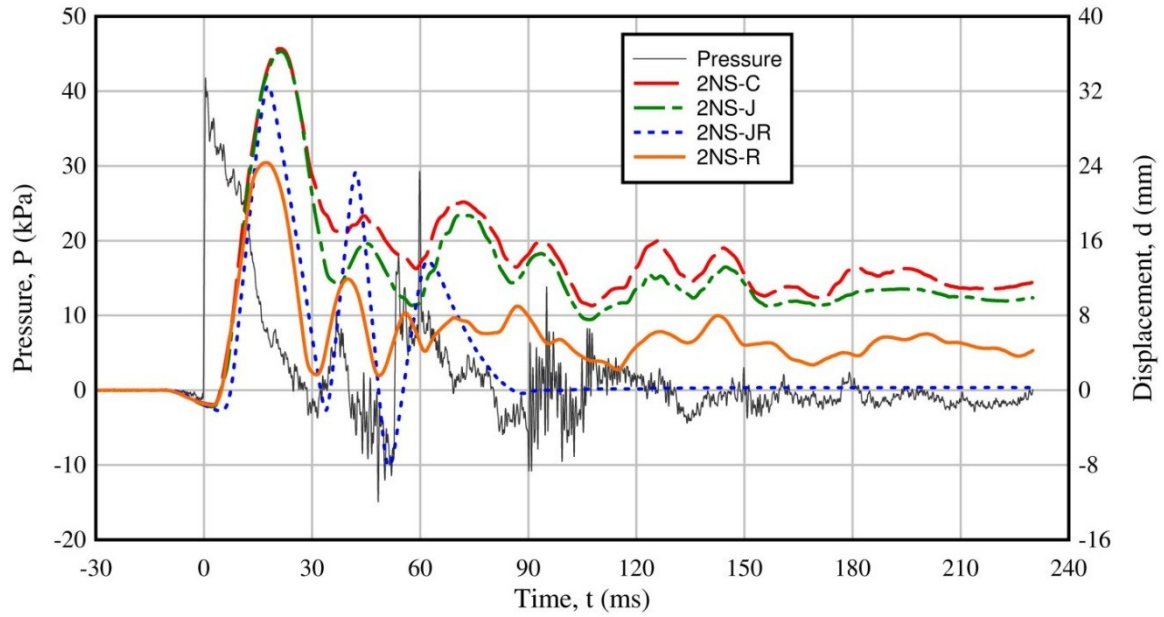


Figure 4-52: Comparison of Displacements - Group 2 Non-Seismic Shot 2

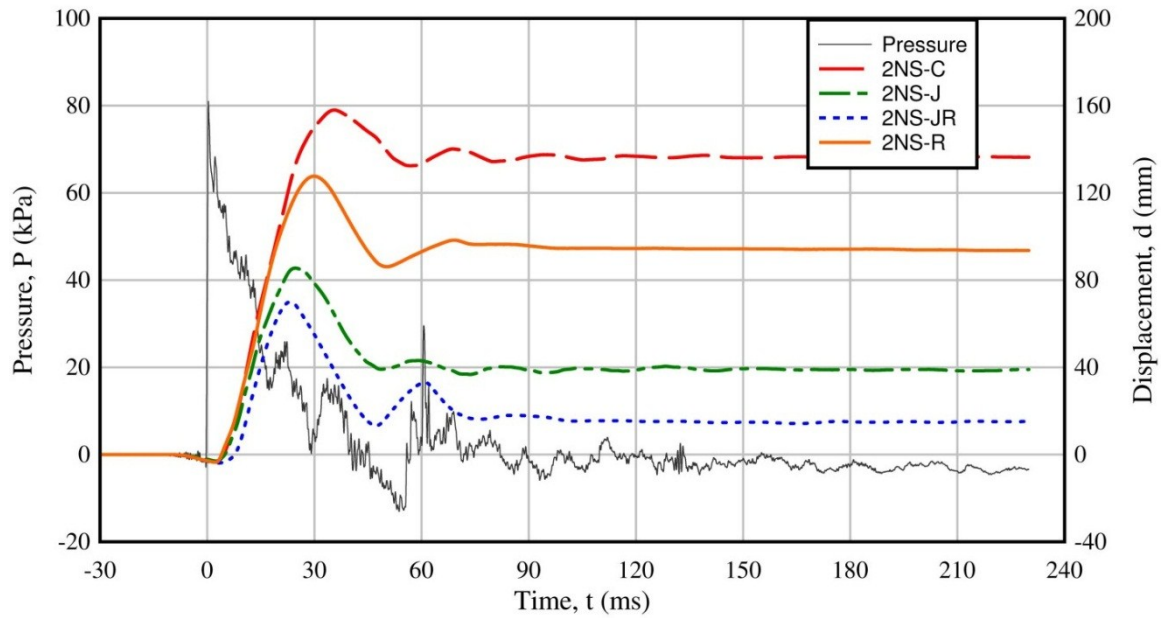


Figure 4-53: Comparison of Displacements - Group 2 Non-Seismic Shot 3

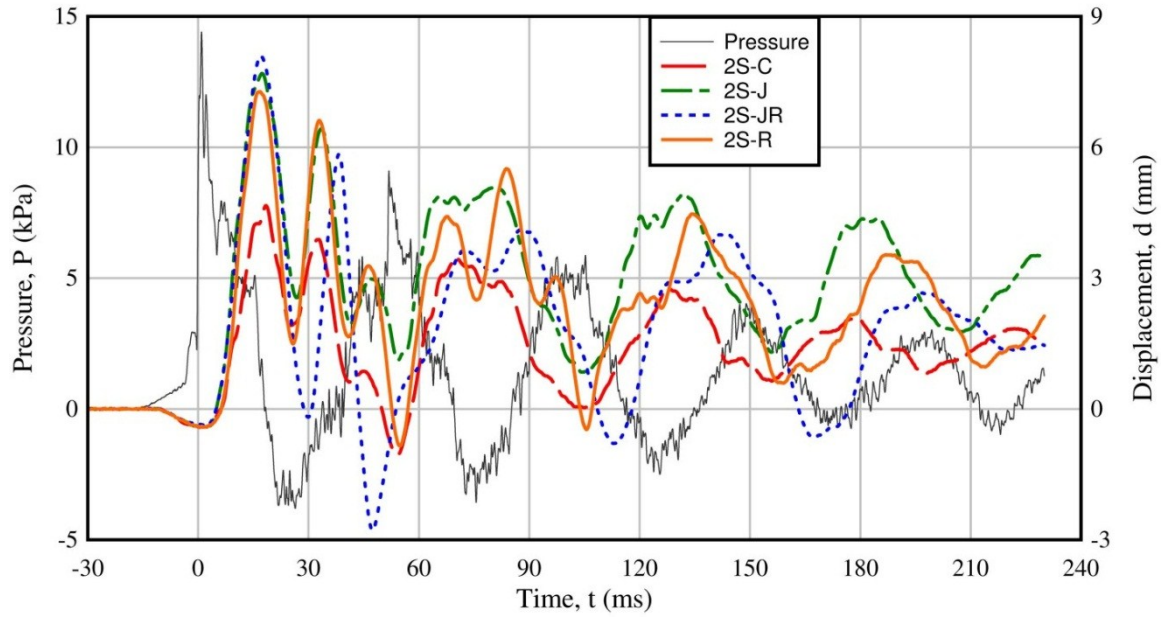


Figure 4-54: Comparison of Displacements - Group 2 Seismic Shot 1

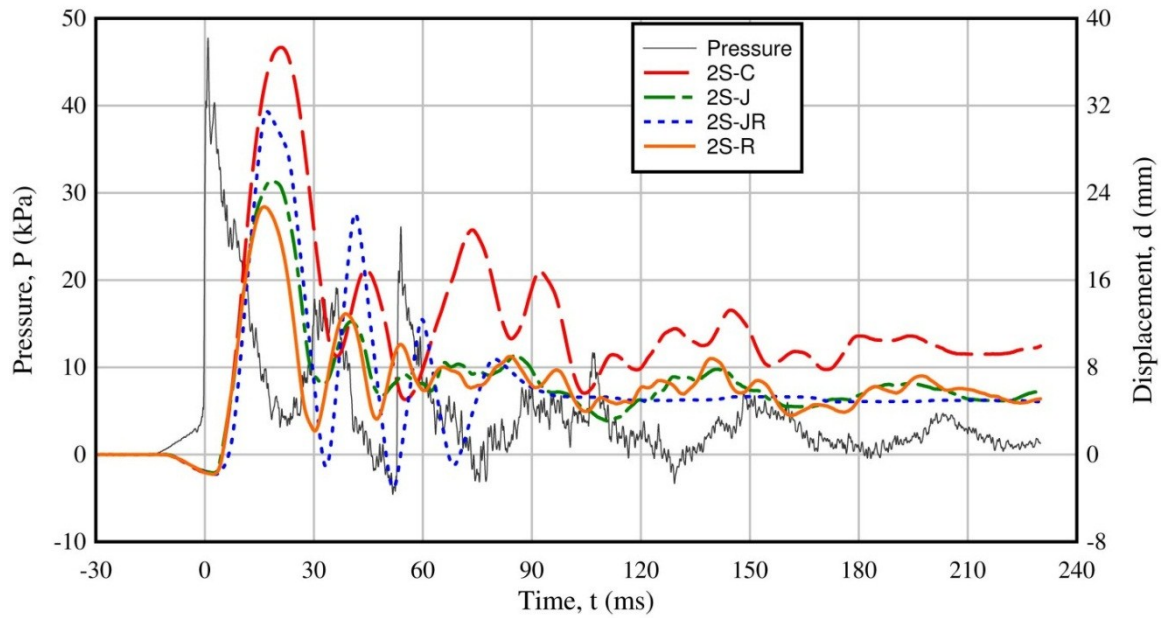


Figure 4-55: Comparison of Displacements - Group 2 Seismic Shot 2

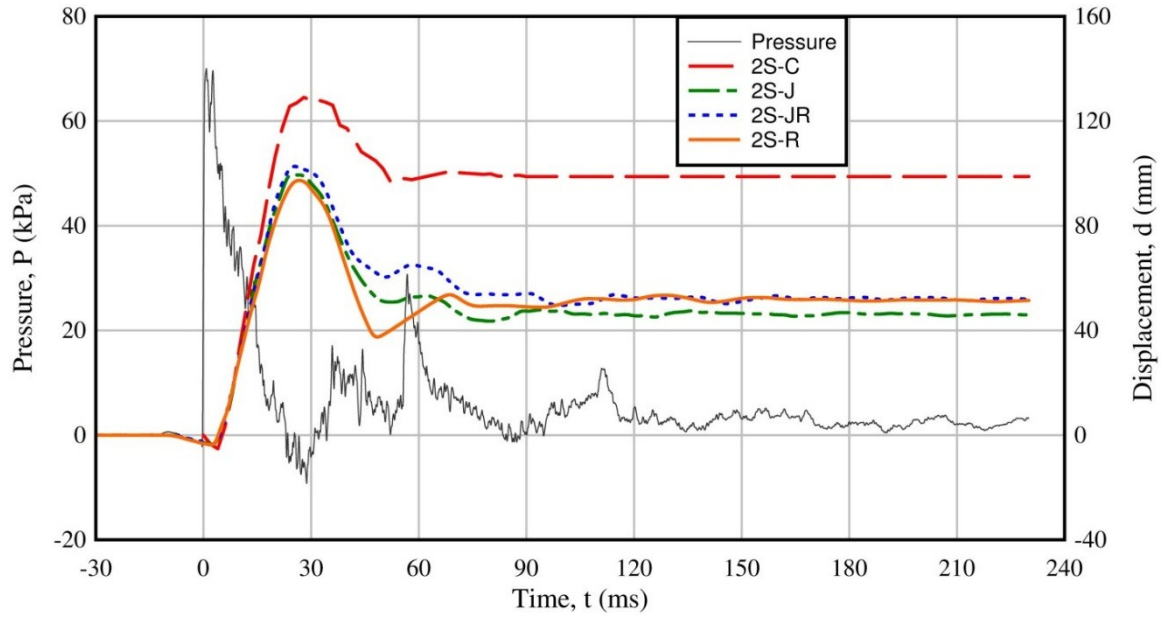


Figure 4-56: Comparison of Displacements - Group 2 Seismic Shot 3

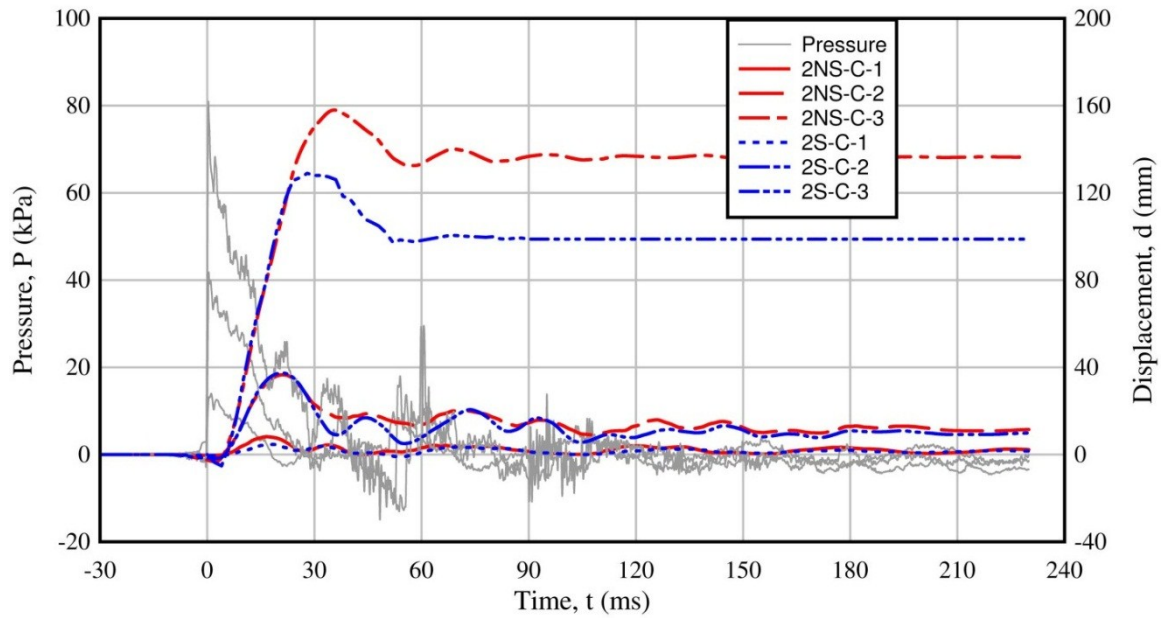


Figure 4-57: Comparison of Displacements - Group 2 Control Columns Seismic vs Non-Seismic

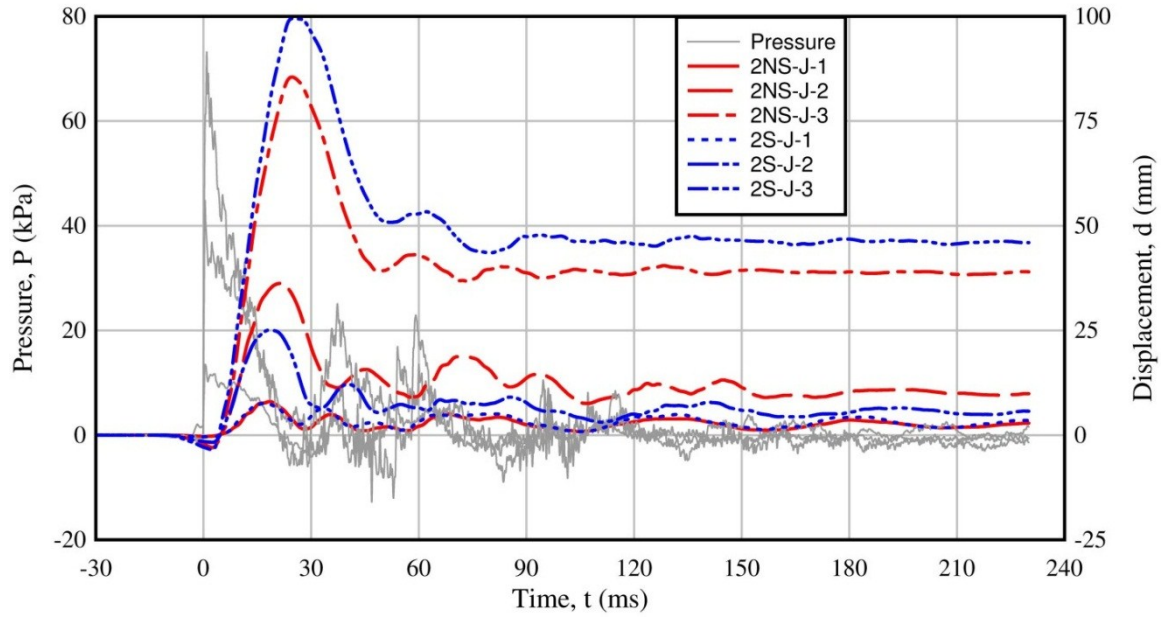


Figure 4-58: Comparison of Displacements - Group 2 Jacketed Columns Seismic vs Non-Seismic

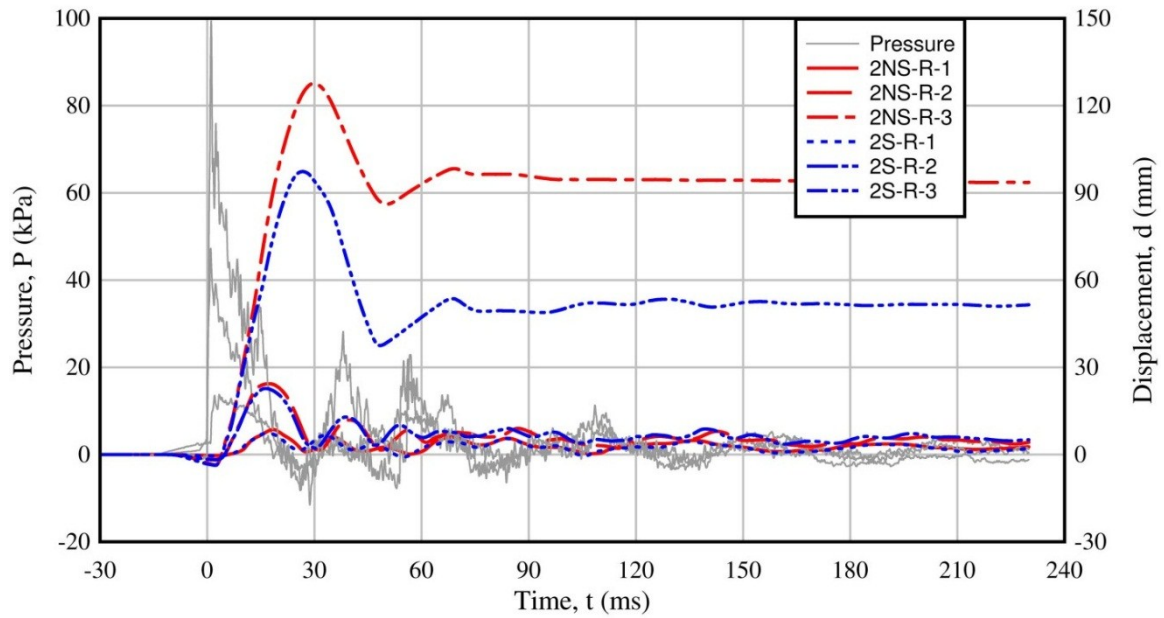
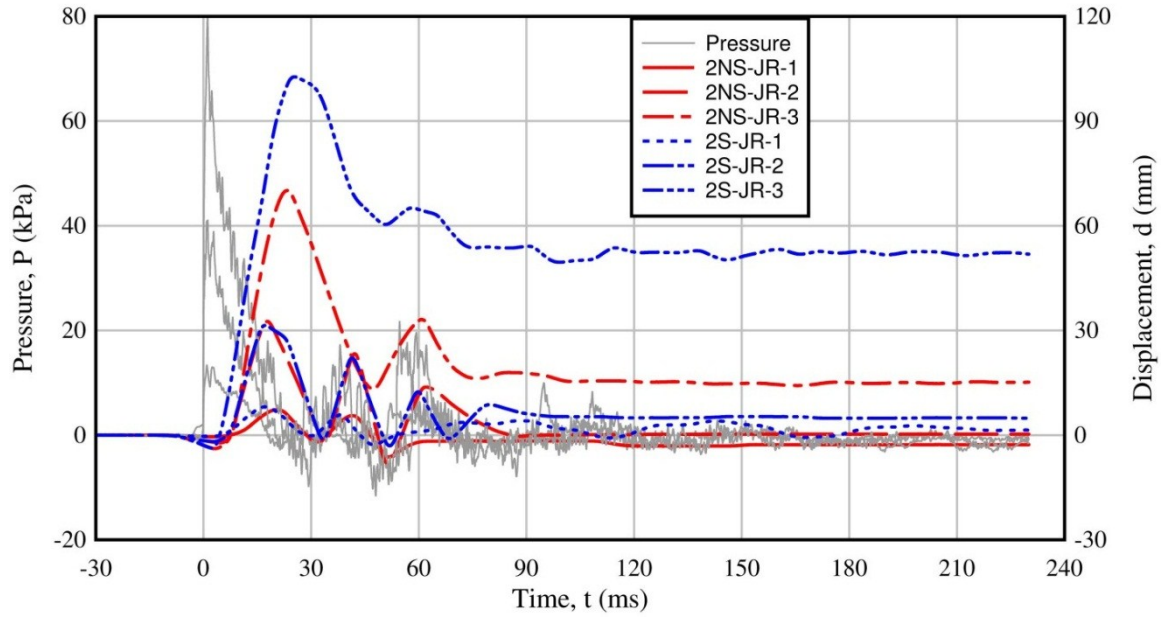
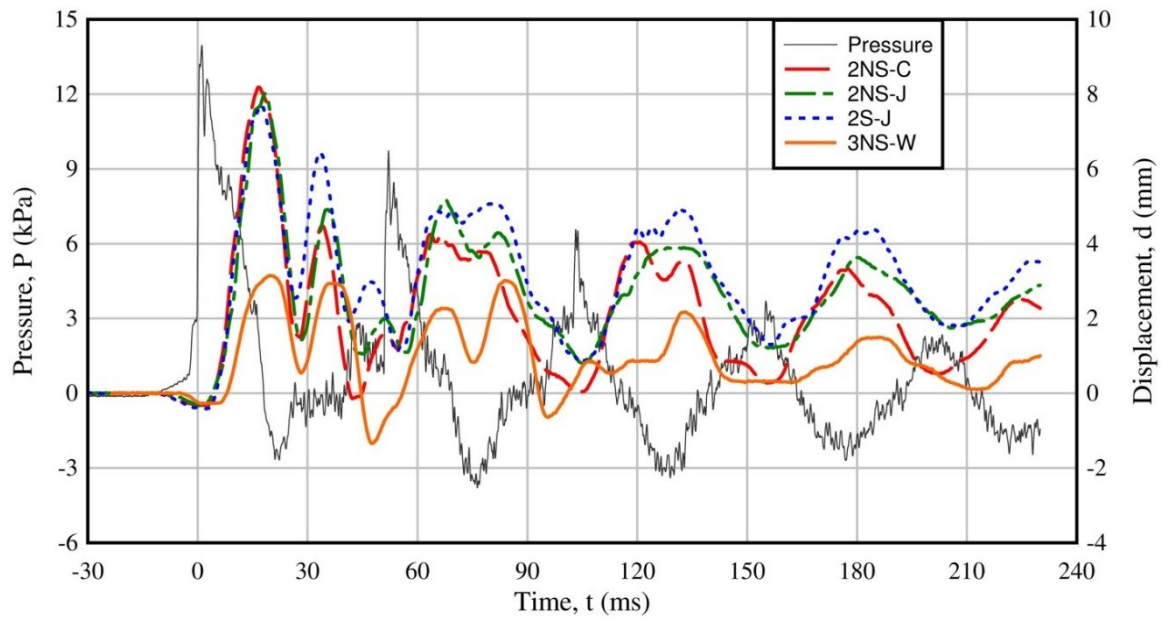


Figure 4-59: Comparison of Displacements - Group 2 FRP Reinforced Columns Seismic vs Non-Seismic



**Figure 4-60: Comparison of Displacements - Group 2 FRP Reinforced and Jacketed Columns
Seismic vs Non-Seismic**



**Figure 4-61: Comparison of Displacements – FRP Jacketed Columns and Wedge Prestressed
Column Shot 1**

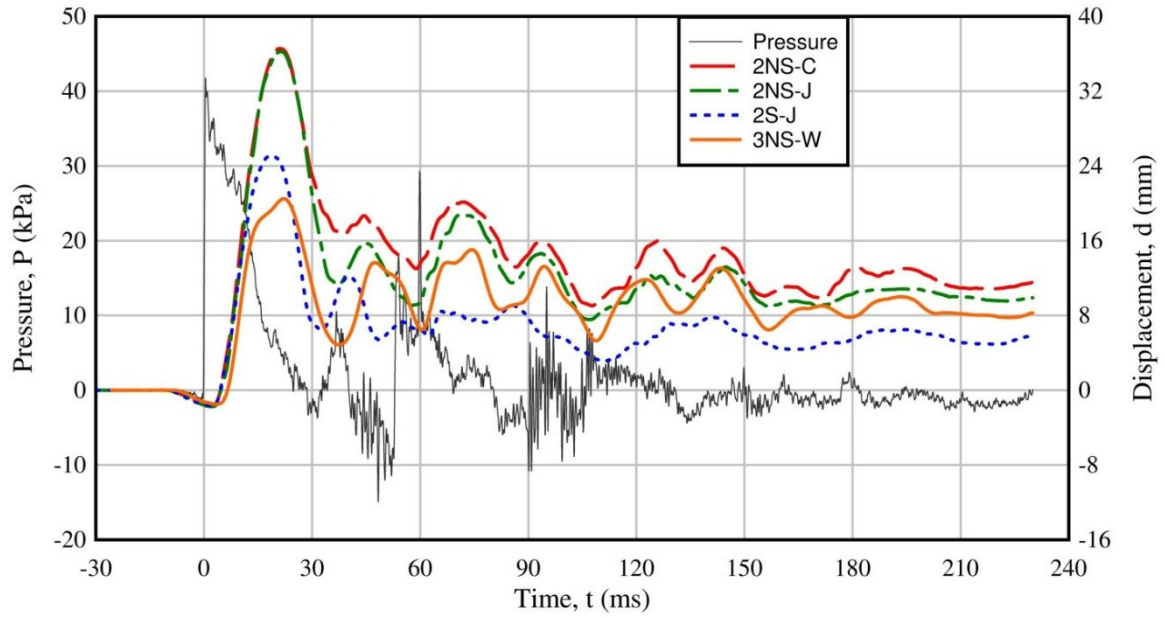


Figure 4-62: Comparison of Displacements – FRP Jacketed Columns and Wedge Prestressed Column Shot 2

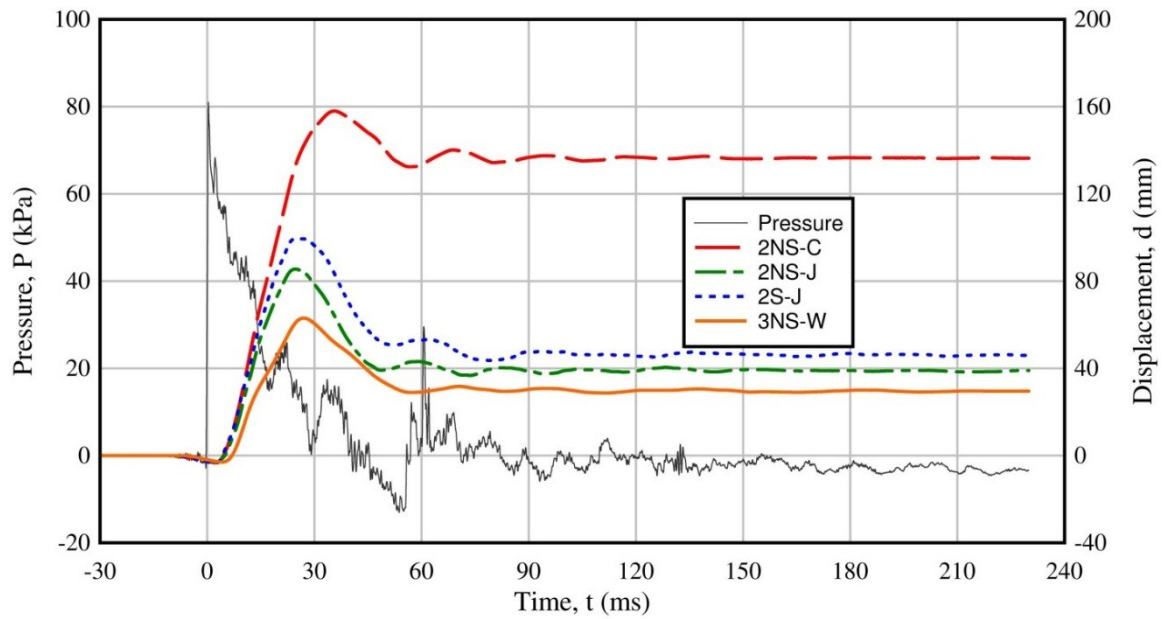


Figure 4-63: Comparison of Displacements – FRP Jacketed Columns and Wedge Prestressed Column Shot 3

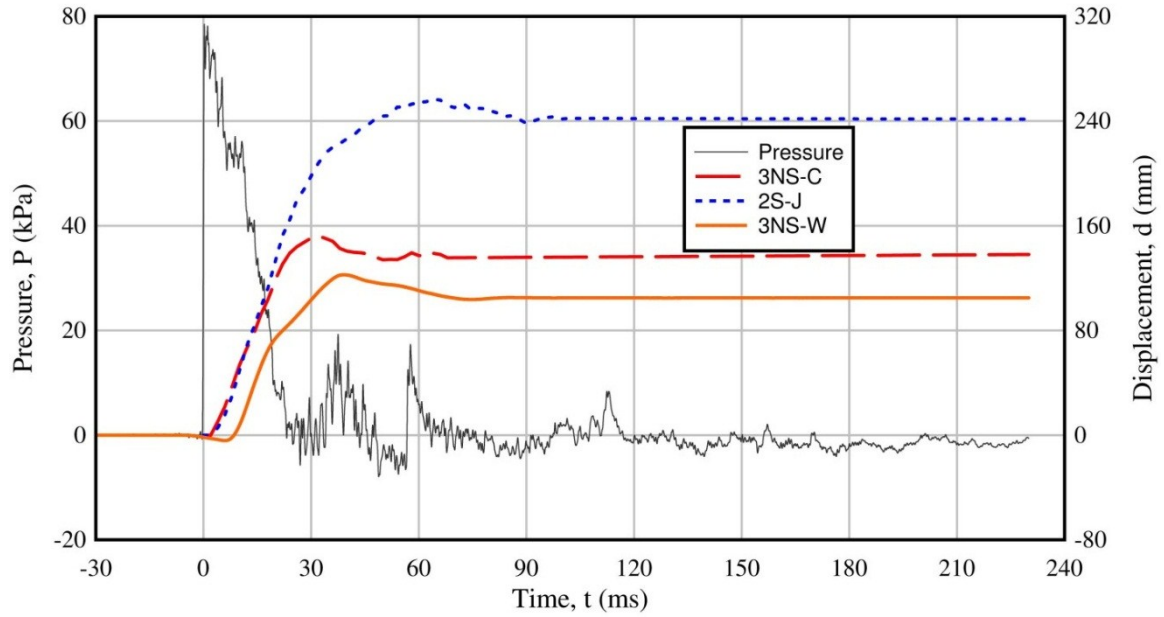


Figure 4-64: Comparison of Displacements – FRP Jacketed Columns and Wedge Prestressed Column Shot 4

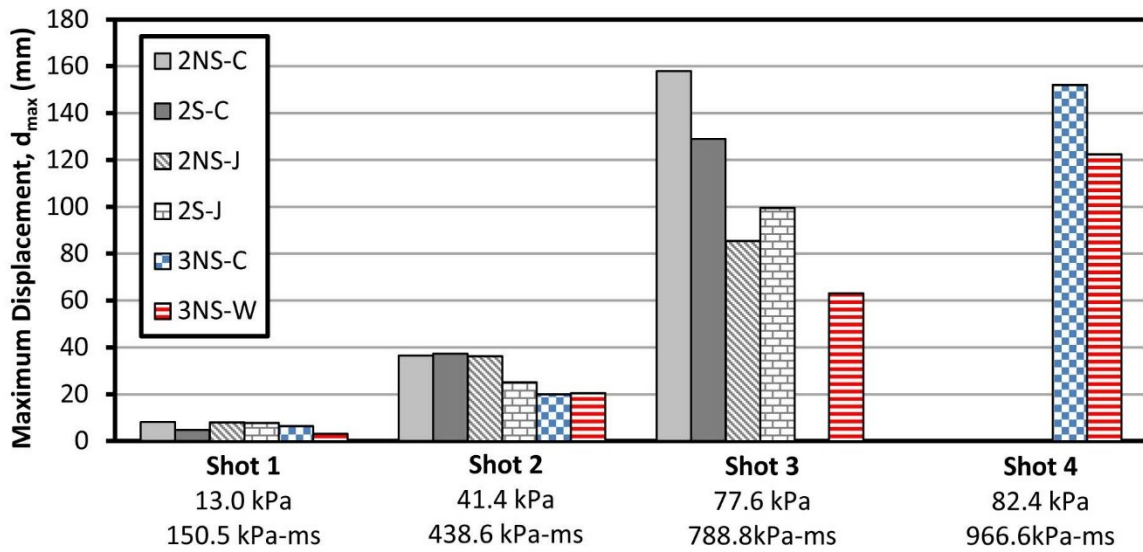


Figure 4-65: Comparison of Maximum Displacements – Jacketed Columns

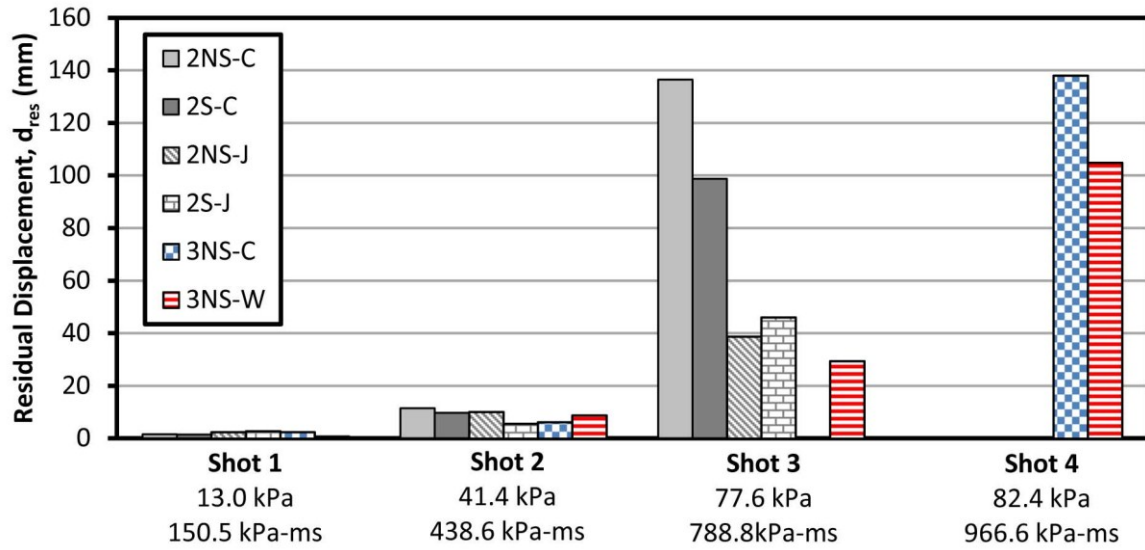


Figure 4-66: Comparison of Residual Displacements – Jacketed Columns

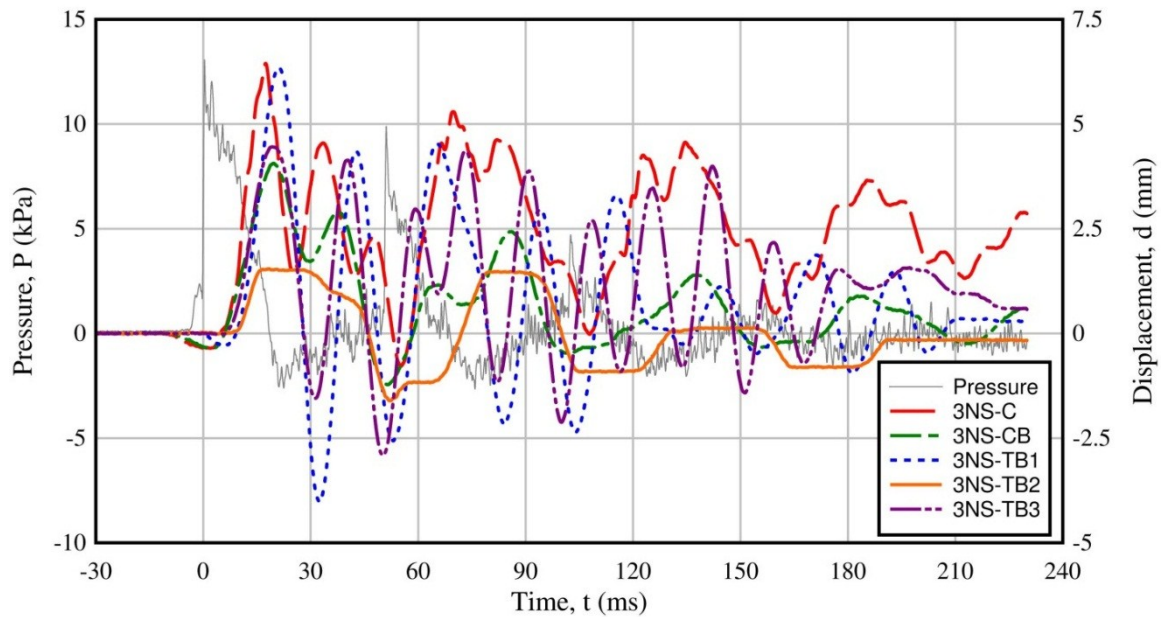


Figure 4-67: Comparison of Displacements – Braced Columns Shot 1

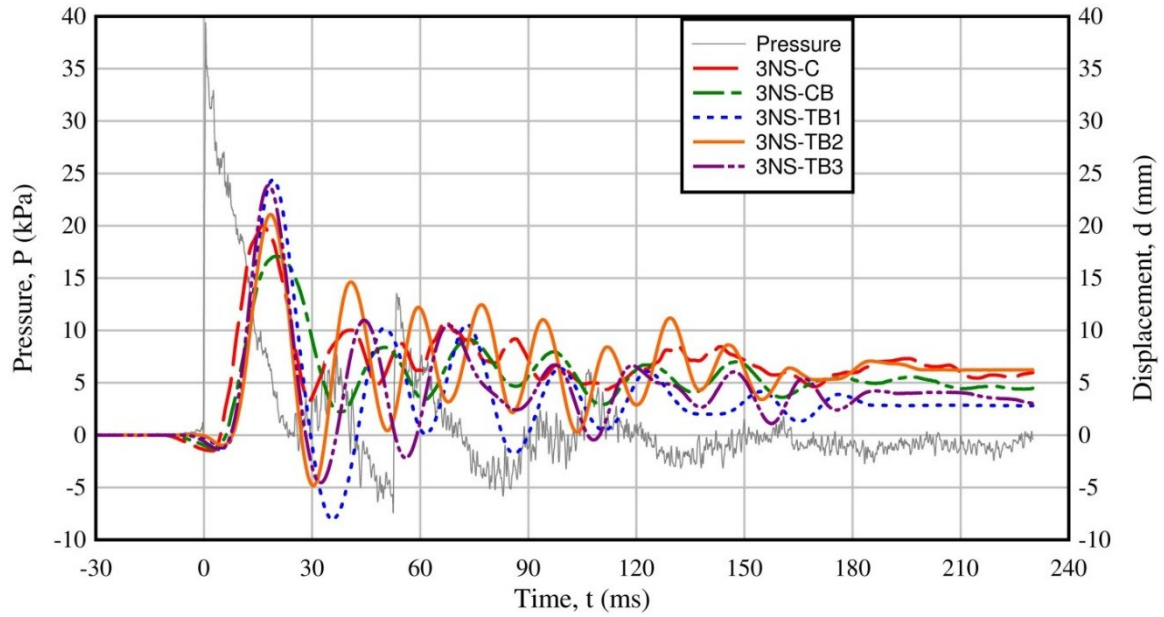


Figure 4-68: Comparison of Displacements – Braced Columns Shot 2

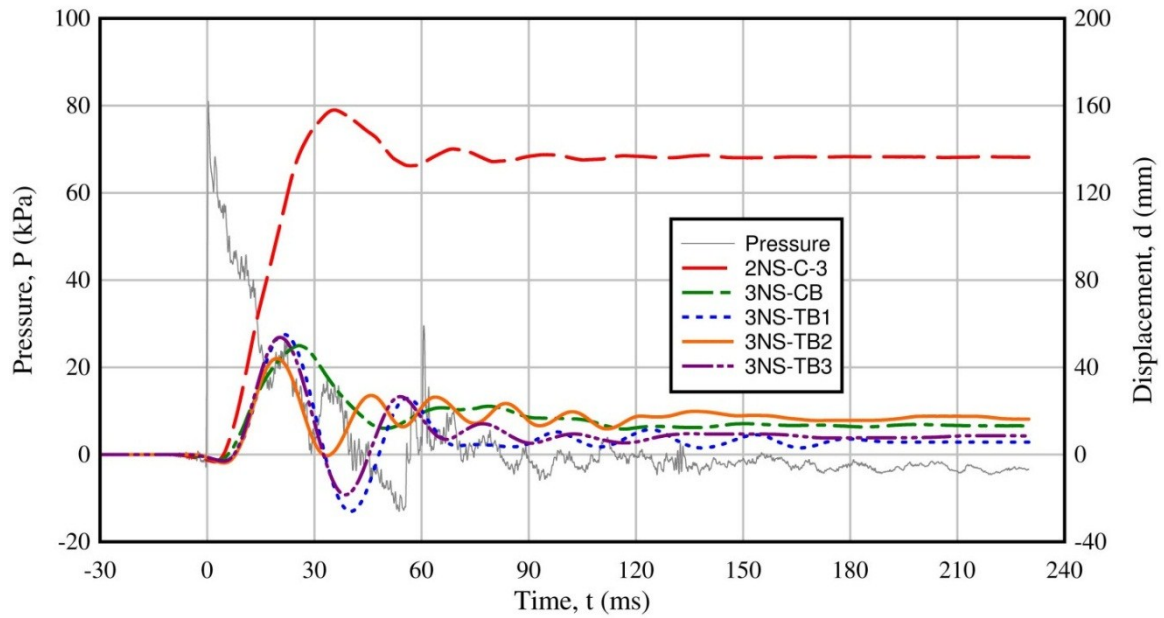


Figure 4-69: Comparison of Displacements – Braced Columns Shot 3

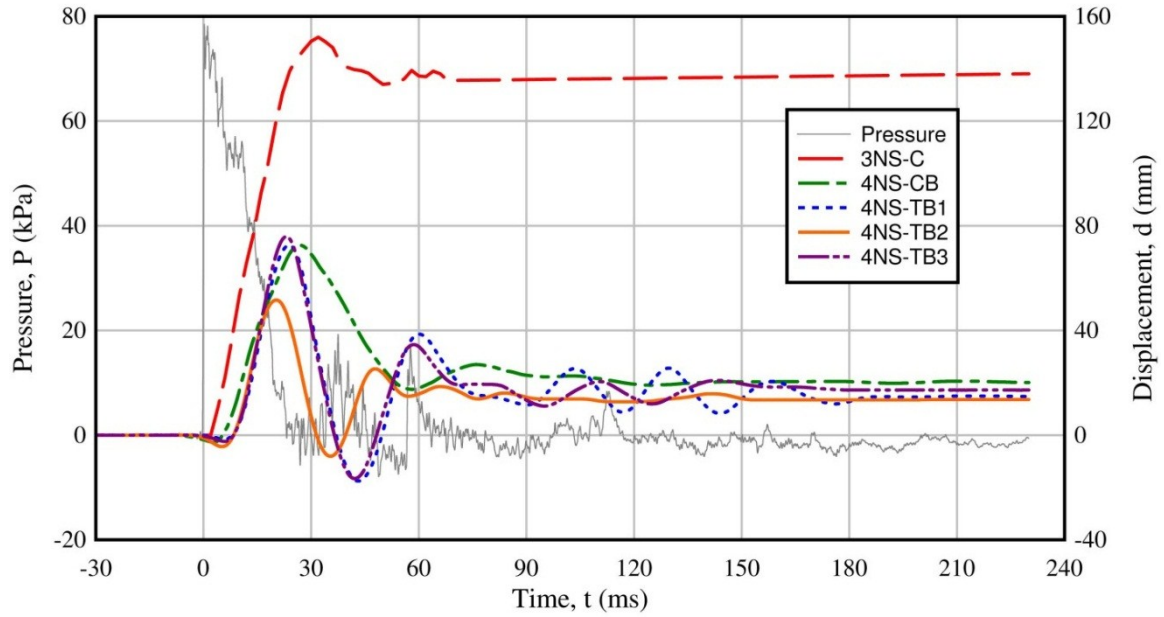


Figure 4-70: Comparison of Displacements – Braced Columns Shot 4

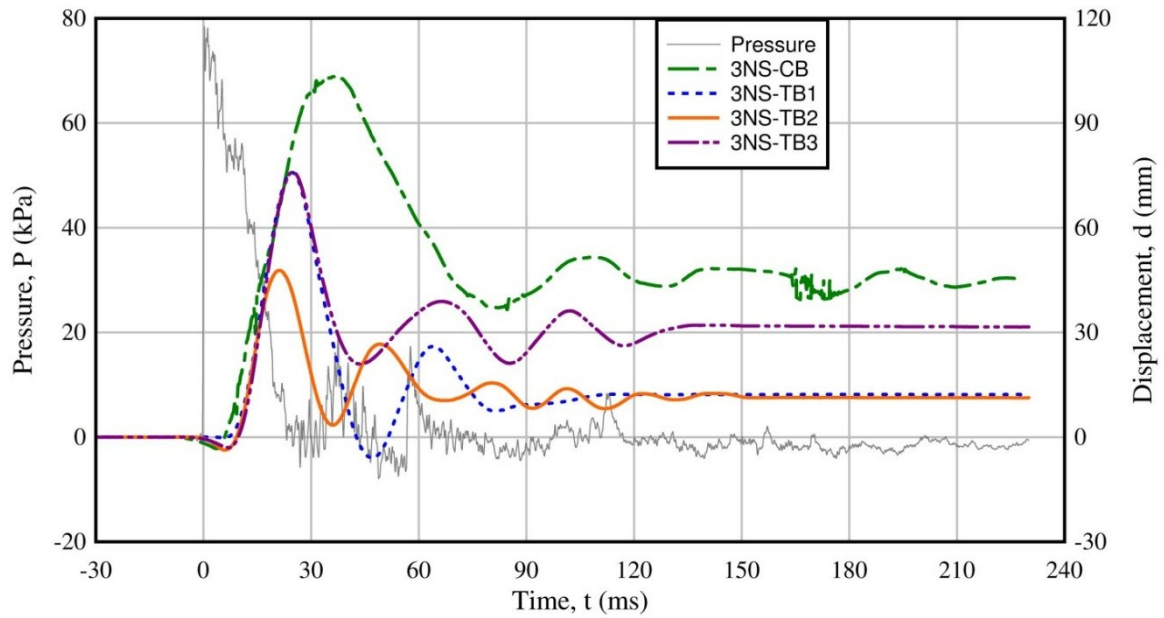


Figure 4-71: Comparison of Displacements – Braced Columns Shot 5

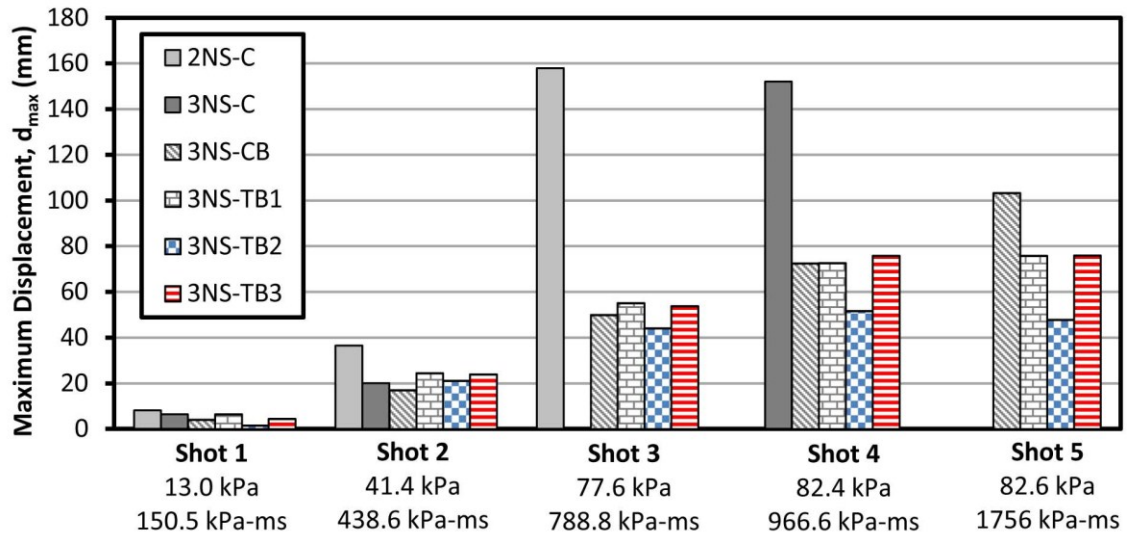


Figure 4-72: Comparison of Maximum Displacements - Braced Columns

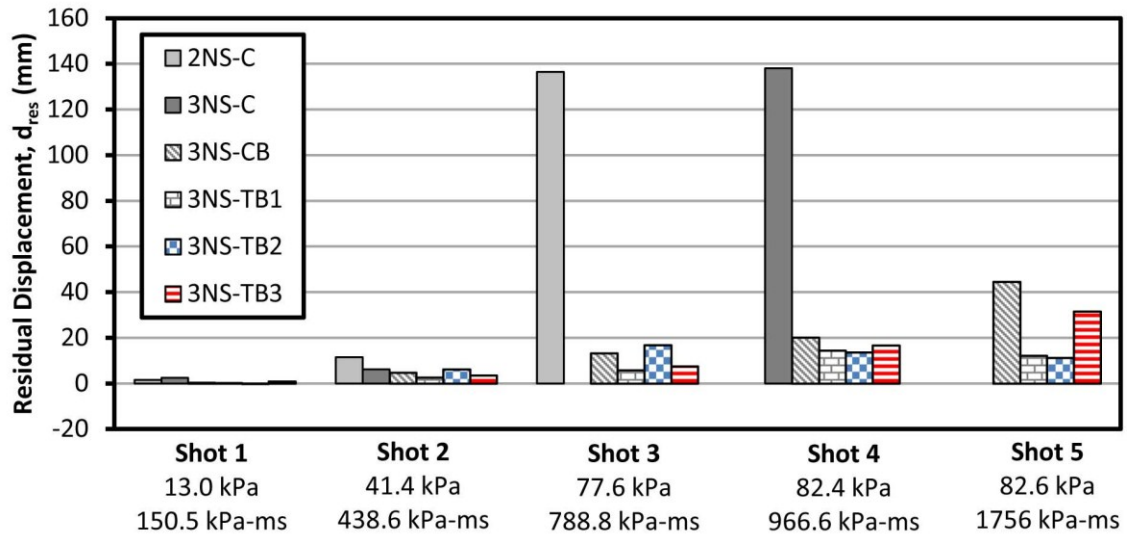


Figure 4-73: Comparison of Residual Displacements - Braced Columns

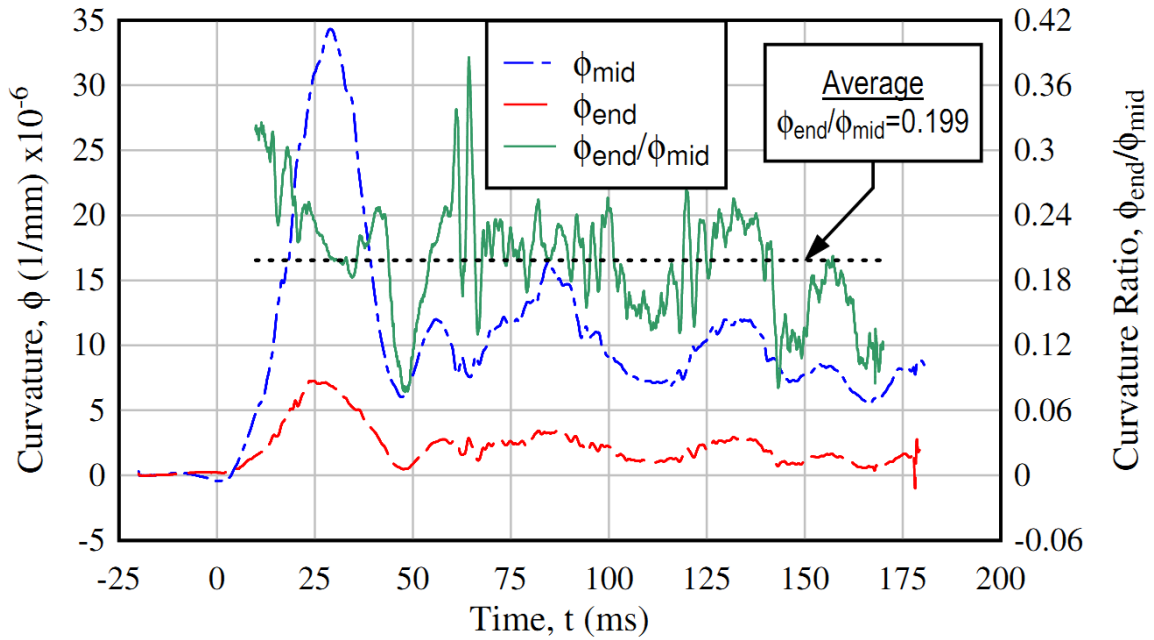


Figure 4-74: Curvatures and Curvature Ratios for Group 1 Support Stiffness

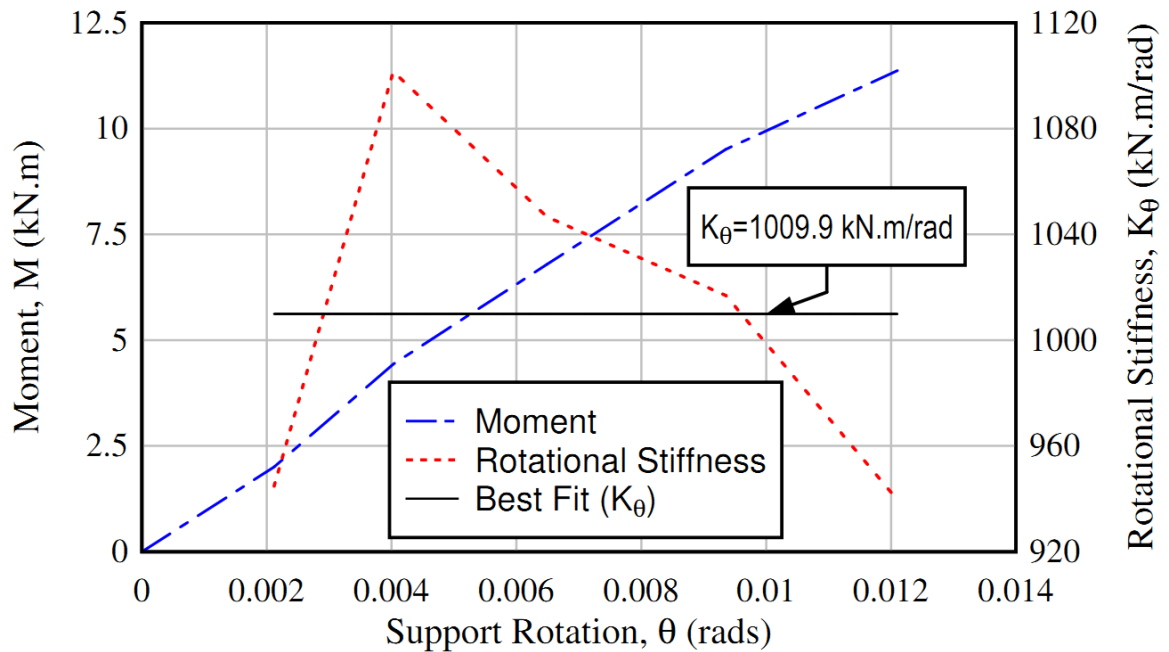


Figure 4-75: Rotational Support Stiffness for Groups 2 and 3

Chapter 5. Single Degree of Freedom Modeling

5.1. General

Analysis of the columns in this study was conducted using an equivalent single-degree-of-freedom dynamic system with non-linear material properties and non-linear geometry. This analysis was conducted in several steps:

1. Material properties were defined and appropriate material models were selected incorporating effects of concrete confinement, inelasticity, and dynamic increase factors on material strength and stiffness.
2. Sectional analysis was performed for a wide range of axial load levels to determine the moment-curvature-axial load relationship along with the axial load-axial deformation relationship.
3. Resistance-displacement curves (or just resistance curves) were determined using incrementally increasing curvature analysis with non-linear second order effects and variation in axial load due to projected length shorting and/or axial force from bracing elements.
4. SDOF equivalency conversion factors as a function of displacement were calculated.
5. Dynamic system properties were determined including system mass, system loaded area, applied pressure from the experimentally recorded pressure-time histories, and the previously calculated resistance-displacement curves.
6. SDOF dynamic analysis was conducted by integration of the equation of motion using the dynamic system properties as the input.
7. SDOF predicted displacement results were compared with the experimental displacement results.

The results of the analysis indicate that the modeling procedures were able to accurately predict the response of all types of columns in this study.

5.2. Single-Degree-of-Freedom Analysis

Structural response to shock wave loading is a transient dynamic problem. The combination of external loading and inertia forces are resisted by internal elastic and damping forces within the

structure and structural members. As the primary interest is typically the maximum displacement which occurs on the first vibration period of response, damping is generally ignored as it has negligible effects on early response for under damped systems (Department of Defense, 2008). The relationship between natural period of the global structure, natural period of structural members, duration of the shock wave loading, geometry of the structure, and location of the loading are significant in determining how the structure will respond. Due to the relatively small mass and large stiffness of individual structural members, natural periods of the members tend to be significantly smaller than those of the entire structure. Since shock wave durations are short, in the order of milliseconds for conventional high explosives, the ratio of shock wave duration to natural period of structural member is far closer to unity than that of the duration to the global natural period. As a result, the total forces on a member can be amplified significantly due to dynamic forces, specifically inertia forces and the response of local members to shock wave loading.

The SDOF equation of motion is given in Equation [5-1] below (Biggs, 1964).

$$K_{LM}(u)m\ddot{u}(t) + Ru(t) = AP_r(t) \quad [5-1]$$

where $K_{LM}(u)$ is the load-mass factor as a function of displacement which converts the continuous system to an equivalent SDOF system, m is the total system mass, R is the displacement dependent resistance of the system, $AP_r(t)$ is the external forcing function of pressure multiplied by area, and $u(t)$ is the field variable of displacement at the SDOF ordinate with $\ddot{u}(t)$ as its second derivative, acceleration.

The load-mass factor for conversion to SDOF is calculated as a function of the mode shape (ϕ) and the distribution of mass and applied load (Biggs, 1964). Equations [5-2] through [5-4] show the calculation of the load-mass factor.

$$K_M = \frac{\int_0^L m\phi^2(x)dx}{mL} \quad [5-2]$$

$$K_L = \frac{\int_0^L p\phi(x)dx}{pL} \quad [5-3]$$

$$K_{LM} = \frac{K_M}{K_L} \quad [5-4]$$

5.2.1. Numerical Integration of the Equation of Motion

The software RCblast (Jacques, 2014 and Jacques et al., 2013) was used for the dynamic analysis of the equation of motion. This software uses the average acceleration method to integrate the equation of motion. The dynamic system properties including mass, area, piecewise resistance-displacement curves, piecewise load-mass transformation factors, and applied pressure time-histories were calculated separately and inputted into the software. The calculation of these input factors is described in the following sections.

5.2.2. Loaded Area and System Mass

The areas used as loaded are for the dynamic analysis were 4.54m^2 for group 1 columns using LTD-A and 4.23m^2 for group 2 and group 3 columns using LTD-B. Group 1 columns had a total system mass of 465kg which included 394kg for LTD-A and 71kg for the column specimen determined using an assumed concrete density of 2400kg/m^3 . Group 2 columns had a total system mass of 316kg which included 209kg for LTD-B and 107kg for the column mass. Column 3NS-C had the same mass as that for group 2. Column 3NS-W had a total system mass of 560kg which included 209kg for LTD-B, plus 27kg for the spacer, in addition to 107kg for the column plus 24-9kg for the wedges. Column 3NS-CB had a total mass of 610kg which included 209kg for LTD-B plus 27kg for the spacer, in addition to 107kg for the column plus 24-9kg for the wedges plus the 50kg brace elements. Columns 3NS-TB1 to 3NS-TB3 had a total system mass of 393kg which included 209kg for LTD-B plus 27kg for the spacer in addition to 107kg for the column plus an assumed mass of 50kg for the brace elements. The masses and loaded areas used for analysis are reported in Table 5-1.

5.2.3. Hysteretic Model

The default hysteretic model built into RCblast was that proposed by Clough (1966). As the columns did not have a clear linear elastic behaviour up to yield region due to the variation in axial load (as described in detail later in this chapter), this model only approximates the hysteretic behaviour. The primary interest in the dynamic analysis was the maximum displacement. This value occurs on the peak of the first thrust and does not require the definition of an unloading (or hysteresis) model to determine. The majority of error in post peak response predictions that arise using the Clough model will occur with post-peak predictions of small displacement values that do not yield but are acting as damaged in the Clough model. The large displacement (and high damage) post-peak predictions are not significantly affected by the use of this simplified hysteretic model. As the free-vibration response

was not critical and not of primary concern in this analysis, the Clough model was considered sufficient for the hysteretic analysis.

5.3. Resistance-Displacement Relationships

Resistance-displacement relationships, or resistance curves, were developed using an incremental second order analysis with non-linear material properties and variable axial loads. The process for generation of resistance curves was as follows:

Step 1: Run a series of sectional analyses under various axial load cases to determine moment-curvature and axial load-vertical deformation relationships for the columns.

Step 2: Run second order incremental lateral load analysis for columns under various axial load states to determine the total lateral load-lateral displacement relationships for the columns.

Step 3: Using the results from Step 2 and the axial load-vertical deformation relationships for the sections, piece together composite resistance curves that incorporate the variation of axial load with lateral deformations observed in the experimental program.

5.3.1. Material Models

Several different material models were incorporated into the analysis to address all the different material responses present in the test program. These material models address the effects of confinement on concrete and steel properties, inelasticity of steel, and the degree of confinement provided by external and internal transverse steel or FRP. The unconfined and confined concrete models are shown in Figure 5-1. The reinforcing steel models for compression and tension steel are shown in Figure 5-2.

5.3.1.1. Unconfined Concrete

Unconfined concrete is the cover concrete for the columns. For this analysis, the depth of cover was equal to the clear cover plus half the tie diameter. Figure 5-3a shows a typical column cross-section with the unconfined region referenced. The material model for both confined and unconfined concrete was developed by Saatcioglu and Razvi (1992) with the unconfined concrete model based on Hognestad (1951). The unconfined model uses a parabolic stress-strain relationship for the ascending curve up to the maximum concrete stress, f'_c , at the prescribed strain at maximum, ϵ_{01} , which was taken as 0.002 for this analysis. Following the maximum stress, the curve decreases

linearly to $0.85f'_c$ at the prescribed ultimate strain, ε_{cu} , value which was taken as 0.0035 for this analysis. The ascending and descending stress strain curves for unconfined concrete are shown in Equations [5-5] and [5-6].

$$f_c = f'_{co} \left[2 \frac{\varepsilon_c}{\varepsilon_{01}} - \left(\frac{\varepsilon_c}{\varepsilon_{01}} \right)^2 \right] \quad \text{for } \varepsilon_c \leq \varepsilon_{01} \quad [5-5]$$

$$f_c = f'_{co} - (\varepsilon_c - \varepsilon_{01}) \left(\frac{0.15f'_{co}}{\varepsilon_{085} - \varepsilon_{01}} \right) \quad \text{for } \varepsilon_{01} \leq \varepsilon_c \leq \varepsilon_{cu} \quad [5-6]$$

5.3.1.2. Steel Hoop Confined Concrete

Confinement of concrete has the effect of increasing strength, strain at maximum strength, and post peak material ductility capacity over that of unconfined concrete as shown for the confined curve in Figure 5-1. The confined region of a steel hoop reinforced column is shown as the interior of the column in Figure 5-3a. The confined concrete model for concrete confined with steel hoops is given in the equations below (Saatcioglu and Razvi, 1992). In this model, the confinement pressure, f_l , is a function of the hoop dimensions, spacing, and yield strength. The effective confinement pressure is calculated by multiplying f_l by the factor k_2 which is a function of the core geometry and hoop spacing. The maximum confined concrete stress, f'_{cc} , is calculated using the specified strength and confinement pressure. The strain at maximum stress, ε_1 , is increased beyond the specified strain at specified concrete strength (ε_{01}) based on a factor of the effective confinement pressure divided by the specified concrete strength (K). The strain at 85% of concrete strength, ε_{85} , is increased beyond the specified ε_{085} value by a function of the transverse reinforcement ratio (ρ) and the strain at maximum confined concrete strength (ε_1). The stress strain curve for the descending branch is linear beginning at ε_1 and passing through ε_{85} at 85% of f'_{cc} . This descending branch continues until stress is 20% of f'_{cc} where it remains constant for all further increasing strains.

$$f_c = f'_{cc} \left[2 \frac{\varepsilon_c}{\varepsilon_1} - \left(\frac{\varepsilon_c}{\varepsilon_1} \right)^2 \right]^{1/(1+2k)} \quad \text{for } \varepsilon_c \leq \varepsilon_1 \quad [5-7]$$

$$k = \frac{k_1 f_{le}}{f'_{co}} \quad [5-8]$$

$$f'_{cc} = f'_{co} + k_1 k_2 f_l \quad [5-9]$$

$$f_l = \frac{\sum A_s f_{yh}}{s b_c} \quad [5-10]$$

$$k_1 = 6.7 (k_2 f_l)^{-0.17} \quad [5-11]$$

$$k_2 = 0.26 \sqrt{\frac{b_c}{s} \frac{b_c}{s_l} \frac{1}{f_l}} \quad [5-12]$$

$$\varepsilon_1 = \varepsilon_{01}(1 + 5k) \quad [5-13]$$

$$\varepsilon_{85} = 260\rho\varepsilon_1 + \varepsilon_{085} \quad [5-14]$$

$$\rho = \frac{\sum A_{sh}}{s(b_{cx} + b_{cy})} \quad [5-15]$$

5.3.1.3. FRP Confinement Pressure Model Modification

In the case of FRP jacketed columns, the jacket provides additional confinement pressures above those of the steel hoops. Figure 5-3b shows a typical cross section of a jacketed column with the different confined regions identified. For columns with external FRP confinement, the cover concrete is modeled using the confined concrete model (described above) with FRP confinement pressures as described in Equation [5-16] where t_j is the FRP jacket thickness, f_{Fj} is the stress in the confinement FRP, and D is the cross-section depth of the column. The coefficient k_1 is calculated using Equation [5-17] with $k_c=0.4$ for square columns. The stress in the FRP, f_{Fj} is calculated as a function of the axial load to concentric load capacity of the column as shown in Equations [5-18] to [5-20] (Saatcioglu, 2012). For the core concrete, the lateral pressure from the FRP is added to the lateral pressure from the hoop steel.

$$f_l = \frac{2t_j f_{Fj}}{D} \quad [5-16]$$

$$k_1 = 6.7(k_c f_l)^{-0.17} \quad [5-17]$$

$$\frac{P}{P_o} < 0.15, f_{Fj} = 0.003E_f \quad [5-18]$$

$$0.15 < \frac{P}{P_o} < 0.3, f_{Fj} = 0.002E_f \frac{P}{P_o}. \quad [5-19]$$

$$\frac{P}{P_o} > 0.3, f_{Fj} = 0.006E_f. \quad [5-20]$$

5.3.1.4. Confinement Pressure Model Modification for Prestress Steel

The use of prestressed steel on concrete wedges for confinement of a square section, as shown in Figure 5-3c, required the development of new confinement pressure and confinement efficiency equations. The confinement pressure from the prestressed steel on the concrete wedges is given in Equation [5-21] for circular sections. The same rational used for confinement of circular sections with circular hoops in the original confinement model (Saatcioglu and Razvi, 1992) was used for this,

however, the total force from the confinement steel was modified to account for the steel exiting the wedge area (for a single side of the cross section) at a 45° angle as shown in Figure 5-4. The confinement efficiency factor, k_2 , was assumed to be 1.0. Equations [5-22] to [5-24] were developed from the axial load dependent FRP stress equations presented above. These equations provide the stress in the prestressed steel as a function of the axial load and the initial prestressing. The transverse reinforcing ratio, ρ , is given in Equation [5-25]. The confinement stress from prestressed steel can be added to that of the internal hoops to get a total confinement stress for the core concrete.

$$f_l = \frac{\sqrt{2}A_s f_{ps}}{bs_{ps}} \quad [5-21]$$

$$\frac{P}{P_o} < 0.15, f_{ps} = f_{pso} + 0.3f_{pu} \leq f_{pu} \quad [5-22]$$

$$0.15 < \frac{P}{P_o} < 0.3, f_{ps} = 2f_{pu} \frac{P}{P_o} + f_{pso} \leq f_{pu}. \quad [5-23]$$

$$\frac{P}{P_o} > 0.3, f_{ps} = f_{pso} + 0.6f_{pu} \leq f_{pu} \quad [5-24]$$

$$\rho = \frac{\sqrt{2}A_{ps}\pi}{s_{ps}b} \quad [5-25]$$

5.3.1.5. Reinforcing Steel

Reinforcing steel was modeled using either a tension strain hardening model or a compression buckling model described below.

5.3.1.5.1. Steel in Tension

For steel in tension, the stress-strain curve is described by three regions: elastic, plastic, and strain hardening. For the elastic region, the stress-strain relationship for steel is linear up to yield strain as shown in Equation [5-26]. The plastic plateau region is linear between yield strain and the onset of strain hardening as shown in Equation [5-27]. The strain hardening region described by Equation [5-28] is assumed to be parabolic up to the ultimate strain at ultimate stress (Yalcin and Saatcioglu, 2000).

$$f_s = E\varepsilon_s \quad \text{for } \varepsilon_s \leq \varepsilon_y \quad [5-26]$$

$$f_s = f_y + (\varepsilon_s - \varepsilon_y) \left(\frac{f_{sh} - f_y}{\varepsilon_{sh} - \varepsilon_y} \right) \quad \text{for } \varepsilon_y \leq \varepsilon_s \leq \varepsilon_{sh} \quad [5-27]$$

$$f_s = f_y + (f_u - f_y) \left[2 \frac{\varepsilon_s - \varepsilon_{sh}}{\varepsilon_u - \varepsilon_{sh}} - \left(\frac{\varepsilon_s - \varepsilon_{sh}}{\varepsilon_u - \varepsilon_{sh}} \right)^2 \right] \quad \text{for } \varepsilon_{sh} \leq \varepsilon_s \leq \varepsilon_u \quad [5-28]$$

5.3.1.5.2. Steel in Compression

Yalcin and Saatcioglu (2000) provide a compression steel buckling model that is a function of spacing the aspect ratio of the spacing of lateral ties to the compression bar diameter. When the aspect ratio is large (greater than 8.0), the compression bar is assumed to buckle at the yield point. This is described in Equations [5-29] to [5-31].

$$f_s = f_y - (\varepsilon_s - \varepsilon_y) \left[-23000 + 11000 \ln \left(\frac{s}{d_b} \right) \right] \quad \text{for } \varepsilon_y \leq \varepsilon_s \leq \varepsilon_{S/Du} \quad [5-29]$$

$$f_{S/Du} = 28f_y \left(\frac{s}{d_b} \right)^{-1.7} \quad [5-30]$$

$$\varepsilon_{S/Du} = \left[40 - 6 \ln \left(\frac{s}{d_b} \right) \right] \varepsilon_y \quad [5-31]$$

When the aspect ratio is between 4.5 and 8.0, the compression bar is assumed to be able to enter the strain hardening region but will not achieve full ultimate strain or ultimate strength prior to buckling. This region is described by Equations [5-32] to [5-34] given below.

$$f_s = f_y + (f_{S/Du} - f_{sh}) \left[2 \frac{\varepsilon_s - \varepsilon_{sh}}{\varepsilon_{S/Du} - \varepsilon_{sh}} - \left(\frac{\varepsilon_s - \varepsilon_{sh}}{\varepsilon_{S/Du} - \varepsilon_{sh}} \right)^2 \right] \quad \text{for } \varepsilon_s > \varepsilon_{sh} \quad [5-32]$$

$$f_{S/Du} = f_{sh} + (f_u - f_{sh}) [48e^{-0.9(s/d_b)}] \quad [5-33]$$

$$\varepsilon_{S/Du} = \varepsilon_{sh} + (\varepsilon_u - \varepsilon_{sh}) [6e^{-0.4(s/d_b)}] \quad [5-34]$$

When the aspect ratio is less than 4.5, the unsupported length of the bar is considered sufficient to prevent buckling and allow full stress-strain response up to ultimate stress and ultimate strain. Therefore, when the aspect ratio is less than 4.5, the tension steel model is used to describe the stress-strain behavior of the compression steel. This is given in Equations [5-26] to [5-28]. For cases with external confinement either FRP or prestress steel, the compression steel was considered sufficiently supported against buckling to allow the tension steel model to be used for compression bars.

5.3.1.6. Bracing Steel

There were two types of bracing steel used in this study: compression brace steel and tension brace steel.

5.3.1.6.1. Threaded Rod for Tension Brace

The threaded rod for the tension brace was Grade 5 high strength threaded rod (approximate equivalent to ISO class 9.8) with a specified yield stress of 710MPa. This material has a specified ultimate strain of between 0.05 and 0.10. An assumed elastic-perfect plastic behaviour was used for the tension braces as described in Equations [5-35] and [5-36].

$$f_{sb} = E_b \varepsilon_{sb} \quad \text{for } \varepsilon_{sb} \leq \varepsilon_{yb} \quad [5-35]$$

$$f_{sb} = f_{yb} \quad \text{for } \varepsilon_{sb} > \varepsilon_{yb} \quad [5-36]$$

5.3.1.6.2. Mild Steel for Compression Brace

The compression braces were comprised of mild steel with an assumed elastic-perfect plastic behavior as described in Equations [5-37] and [5-38].

$$f_{scb} = E_{cb} \varepsilon_{scb} \quad \text{for } \varepsilon_{scb} \leq \varepsilon_{ycb} \quad [5-37]$$

$$f_{scb} = f_{ycb} \quad \text{for } \varepsilon_{scb} > \varepsilon_{ycb} \quad [5-38]$$

5.3.1.7. FRP

The FRP was assumed to follow the manufacturer's specified elastic modulus up to the point of rupture or debonding, whichever occurs first. Rupture strain was assumed to be equal to the manufacturer's specified rupture strain and debonding strain was set equal to the debonding strain observed in the experiments.

5.3.2. Dynamic Increase Factors

Dynamic increase factors (DIF) accounting for the effects high strain rates have on materials were applied to concrete, reinforcing steel and bracing steel in this analysis. The DIFs used were based on the approximate strain rates for far range design (Department of Defense, 2008). Lloyd et al. (2010) determined that the shock tube simulates far field explosions associate with flexural strain rates on the order of $0.1s^{-1}$. Jacques (2011) concluded that the use of approximate strain rates and corresponding DIFs for analysis was sufficient in analyzing flexure-dominant members in shock tube testing without the need for time consuming and labour intensive iterations of the resistance curve with updated DIFs.

The dynamic increase factors used in analysis were taken from the United Facilities Criteria (UFC 3-340-02) Structure to Resist the Effects of Accidental Explosions (Department of Defense, 2008). These values are largely the same as those in the CSA S850-12 Design and Assessment of Buildings Subjected

to Blast Loads (CSA, 2012). The DIF used for concrete was 1.19 applied to f'_c . The DIF for reinforcing steel was 1.17 to yield strength and 1.05 to ultimate strength. The DIF for the compression brace material was 1.19 applied to yield strength which was taken from the A36 structural steel material in tension category. The DIF for the tension brace was 1.05 applied to yield strength which was taken from the A514 structural steel in tension category.

No dynamic increase factors were used for FRP or for material applied transverse to the section as jacketing. The composite FRP material properties are considered independent of strain rate (Harding and Welsh, 1983). This was confirmed experimentally under shock tube testing of reinforced concrete slabs by Jacques (2011). The jacketing material sees a loss of strain with displacement due to the variable axial load during dynamic response. This is discussed in detail later in this chapter. The effect of this loss of axial load was the reduction in lateral pressure resulting from Poisson's effect and, therefore, the strain rate becomes negligible.

5.3.3. Sectional Analysis for Moment-Curvature

A FORTRAN 95 program was written to perform sectional analysis on the columns to generate moment-curvature relationships. Sectional analysis was performed using the following procedure:

1. Sectional geometry was defined. This included section size, reinforcement size and location, strengthening FRP geometry if present, confinement FRP geometry if present, and Prestress steel geometry if present.
2. Material properties and axial load were inputted and material models were used to compute full stress-strain relationships.
3. The cross-section was divided into a number of slices perpendicular to the axis of bending (by default 200 slices were used). At each slice centroid, the area of each material in the slice was calculated. Figure 5-3 shows the material locations for the different sections used in this study.
4. The reinforcing steel and the longitudinal FRP locations and centroids were computed.
5. The axial load was applied concentrically to the column cross-section and the strain in the column was determined such that the total force from all material contributions was equal to the axial load while ensuring strain compatibility across the section.
6. While holding axial load constant, the strain on the extreme fibre that could become in tension was incremented by a default value of $\varepsilon_{01}/20$. While holding the new extreme fibre

strain constant, the strain on the compression fibre was changed and strains at the centroid of each slice and reinforcement element were calculated. Using the centroid strains, the stress in each material was determined. Using the stress and the area of each material at each centroid location, the force was calculated. The sum of all concrete slice and reinforcement element forces was found and compared with the axial load. If it was not equal, the compression strain was modified using the bisection method and the step was repeated. If the internal and external forces had converged the step was complete.

7. The moment was calculated as the sum of moments of each slice or reinforcement element force and the moment from the external axial load about the extreme compression fibre. The curvature was calculated as the difference between extreme fibre strains divided by the section height.

This process was repeated with incrementally higher strains up to the point of sectional failure. Failure of the section was defined as any of:

- Rupture of reinforcement elements.
- Rupture (or debonding) of FRP.
- Inability to converge internal and external forces.
- Post-peak moment capacity less than 5% of the maximum sectional moment capacity.

While computing the sectional response of the columns, key moment-curvature points including FRP rupture (or debonding), reinforcement buckling, and sectional yield were recorded. Sectional yield was defined as the first case of:

- Reinforcement yielding in tension
- FRP rupturing (debonding)
- Cover concrete crushing reaching ϵ_{01}
- Cover concrete reaching ϵ_1 for jacketed columns

As all of the columns exhibited variations in axial load during their dynamic response, the analysis was designed to incorporate multiple load levels. For columns that were not braced with tension braces, the axial load dropped during response from initial load to zero in the extreme cases. For the tension braced columns (3NS-TB1 through 3NS-TB3), the axial load increased during response. Therefore, the moment-curvature relationships for a wide range of axial loads were determined to

address this change in load during response. Figure 5-5 a) and b) show the variation in moment-curvature for column 2NSC. This column had no jacketing with confinement reinforcement spaced at 75mm ($h/2$) along the height of the section. As the axial load dropped from the initial load of 423kN to zero, both the stiffness over the initial region up to yield and the maximum moment decreased. Shortly after yield, the concrete cover crushed resulting in a drop in moment capacity. The effect of this cover crushing was less prominent and more delayed for lower axial loads than for higher axial loads. The post yield curvature ductility, defined as the ratio of curvature to yield curvature, increased as axial load decreased. The post yield moment capacity dropped at higher rates with higher axial load values. These properties are consistent with columns below balanced conditions and were consistent for all the non-jacketed columns in this study. Figure 5-6 a) and b) show the moment-curvature relationship for the FRP jacketed column 2S-J under axial loads ranging from initial 423kN to zero. In this case, similar pre-yield behaviour with variation of axial load as the non-jacketed column was observed: an increase in stiffness and increase in yield moment with increase in axial load. The jacket effect on the post-yield behaviour was apparent with the lack of cover crushing, and the lack of difference in ductility capacity for different axial loads. This behaviour was consistent for all jacketed columns (including columns with FRP and prestressed jackets) for this study.

5.3.4. Resistance-Displacement Curve Development

Three separate analysis procedures were developed in VBA (Visual Basic for Applications) to determine resistance-displacement relationships for the columns in this study: one for non-braced columns, one for compression braced columns, and one for tension braced columns. All of the methods relied on the elastic and plastic flexural analyses of the column due to lateral loads. Therefore, plasticity models needed to be incorporated in the analysis. In all the cases, the mode shape of the response was used to convert the continuous system into an equivalent SDOF system.

5.3.4.1. Plasticity

If the curvature in either of the two possible plastic hinge locations, mid-span or end, exceeded the yield curvature, a plastic hinge of length L_{PL} was assigned to the region. Over this plastic hinge length it was assumed that all curvatures remained constant and greater than yield curvature. The plastic hinge length was defined by an equation given by Sawyer (1964) as shown in Equation [5-39] for all columns other than the prestress confined columns. In Equation [5-39] d is the effective depth of the cross section and z is the shear span.

$$L_{PL} = 0.25d + 0.075z \quad [5-39]$$

For columns 3NS-W and 3NS-CB the hinge length was taken as twice the gap between the wedges as approximated from the experimental results. Based on observations of the test results, all of the damage was concentrated on the gap region. This damage was in the form of concrete crushing on the compression face and cracking on the tension face along. Unlike other columns, these columns did not have cracks forming a fan pattern over a discernable length, rather, they had a single main crack or a pair of cracks that formed inside the gap region. This isolated the damage to a small region which needed to be accounted for in the plastic hinge length used in analysis. This value, 50mm, is less than the hinge length calculated in the above expression for the other columns and is conservative in terms of assessing displacement ductility capacity as the shorter the hinge, the less plastic rotation it will have.

5.3.4.2. Development of Resistance Curves for Non-Braced Columns

For all of the columns without braces including retrofitted columns, those with FRP retrofits and the column with prestressed confinement retrofit, the flexural resistance to lateral loads and axial load induced second order moments was determined using the following steps.

1. Determine sectional response for various axial loads.
2. Define column geometry.
3. Discretize the column along the height into a number of discrete elements with length d_y .
4. Apply an end curvature to the column and calculate the corresponding end moment at the supports. Using the support rotational stiffness, k_θ , found in the experimental study, calculate the specified end rotation.
5. Assume a mid-height curvature and determine the corresponding moment from the moment-curvature relationship.
6. Determine the total lateral load that would result in the mid-height moment using the moment distribution of $M_w(y) = wy(L - y)/2$ and the support moment as the boundary condition, where y is the length along the column, w is the magnitude of the uniformly distributed lateral load, and L is the length of the column.
7. Distribute curvatures along the length of the column according to the corresponding moment values.

8. Integrate curvatures to calculate rotations along the length of the column while applying the mid-height rotation equal to zero boundary condition.
9. Integrate rotations to calculate displacements along the length of the column.
10. Compute the second order moments due to axial load and displacements as $M(y) = Px(y)$
11. Return to Step 6 and add second order moments to moment distribution and continue until convergence of displacement.
12. Compare calculated end rotation from Step 8 with the specified end rotation found in Step 4. If not converged, use the bisection method to determine new trial mid-height curvature and return to Step 5. If converged, continue.
13. Determine mode shape and equivalent SDOF conversion factors (load-mass factors).
14. Calculate the axial displacement of the column due to lateral flexural displacement where
 - the incremental displacement along the length of the column in the vertical direction is $\Delta y = d_y - \tan\theta\Delta x$,
 - Δx is the relative lateral displacement over the element length, d_y , and
 - the total vertical displacement due to flexure (y_{fp}) is the sum of Δy over the column length.
15. Increase end curvature and return to step 4.

5.3.4.2.1. Variable Axial Load Resistance Curves for Non-Braced Columns

The procedure above was repeated 20 times with different axial loads ranging from initial load to zero. Each time, the initial axial deformation was computed for each initial load (y_{ap}). It was assumed that, for a given axial load, when the sum of the axial flexural deformation and the initial axial deformation equaled the initial axial deformation under the initial load (y_{apo}), the load resistance-displacement value would be valid. Therefore, each of the 20 curves used by default contributed one point to the actual resistance-displacement curve. The exception to this being the zero axial load case which was considered to be the actual resistance curve for regions where, $y_{ap} + y_{af} \geq y_{apo}$. The variable axial load resistance curve is referred to as the composite curve for brevity at times in this document.

The composite curve along with the resistance curves for various axial load levels for column 2S-J is shown in Figure 5-7. In Figure 5-7 a) it is apparent that this transition from high initial axial load to

zero axial load occurs within the early response region where the column is elastic or only slightly plastic. The effect of this transition in axial load is to decrease the strength and stiffness of the column (for the case of columns below balanced point). The relationship between the displacement ductility of the column and the axial load transition is a function of the column geometry, the support conditions, the initial axial load level, and the strength. For the case of the columns in this study, the transition between high axial loads to zero axial load occurred prior to yielding. For columns with lower length to depth aspect ratios or fully fixed supports, this load transition would happen at higher displacement ductilities. In Figure 5-7 b) it is apparent that the loss of axial load provides for higher ultimate displacement ductility. This is due to the negation of second order effects.

Figure 5-8 shows the curvature ductilities for the mid-span and end regions of the column overlaid upon the composite resistance curve for column 2S-J. In Figure 5-8 a) the point of first yield is shown to be in the mid-span of the column at approximately 16mm deflection. At this point, the axial load has dropped to less than 25% of the initial axial load on the column. At approximately 18mm displacement, the end of the column begins to hinge as well. At this point the entire axial load is lost and the column is bending in pure flexure. Because of the beam bending response, the column is able to achieve very high ductilities with curvature ductility ratios of up to 30 and displacement ductility ratios of up to approximately 14 as shown in Figure 5-8 b). Figure 5-8 a) and b) also have four points of interest identified as “Mode Shape Points”. These points represent locations of elasticity with high axial load (Point 1), elasticity with maximum resistance (Point 2), elastic-plastic transition region with no axial load (Point 3) and extreme inelasticity with no axial load (Point 4). The mode shapes for these regions are shown in Figure 5-9. In Figure 5-9 the low rotational restraint of the support systems is shown as the column mode shape appears closer to a pin-ended member than a fully fixed member for the elastic mode shapes.

The two elastic mode shapes (points 1 and 2) are similar. As the end supports allowed rotation, the end mode shape shows rotation that appears closer to simple supported than fixed. This behavior is consistent with the curvature ductilities at the two locations where mid-span ductility was consistently higher than support ductility. Once plasticity occurs, it becomes more apparent that the majority of displacement is due to rotation over short lengths at the mid-span and end of the column. The mode-shape for point 4, where the column is highly plastic, shows what may be mistaken for fully fixed support conditions, in fact, the supports have rotated approximately the same amount as

in the curves for point 3 and there is significant additional rotation over the hinges that dwarfs the support rotation.

Figure 5-9 also reports the load-mass factor (K_{LM}) which can be roughly used in assessing the elasticity of response as it relates the integral of the square of mode shape to the integral of the mode shape. In general, once plasticity occurs, the load-mass factor will begin to drop. Idealized systems with fully fixed supports, constant rigidity (EI), no axial load, and no plastic hinge lengths will have load-mass factors of 0.77 for elastic, 0.78 for elastic-plastic and 0.66 for plastic response (Biggs, 1964). In the case of this column, the mode shapes range from 0.777 to 0.761 for elastic, 0.741 for elastic-plastic, and 0.728 for fully plastic. The consistently higher mode shapes in this analysis are largely due to the presence of a plastic hinge length.

Although Figure 5-7 through Figure 5-9 only shown the behaviour of column 2SJ, the response can be generalized to the other non-braced columns in this study. Columns with no jacketing may have somewhat lower post yield resistance and ductility capacity, but as they exhibit no axial load they will behave like under-reinforced beams and have tension steel controlled failure that is similar to that shown for column 2S-J. Columns with jacketing including columns from Group 1 and column 3NS-W will have similar responses in general, however differences in geometry, support conditions, axial load, plastic hinge length, confinement efficiency, and material properties will provide different magnitudes of response. Columns with strengthening FRP will exhibit similar response up to the point of FRP rupture or debonding where there will be a significant drop in resistance as the longitudinal FRP is removed from the system and the columns follow the response of their non-strengthened companions.

Figure 5-10 through Figure 5-23 show the resistance-displacement and load-mass factor-displacement curves for all the non-braced columns including all four Group 1 columns, all eight Group 2 columns, and columns 3NS-C and 3NS-W from Group 3. Figure 5-18 and Figure 5-19 show the resistance curves for the FRP strengthened columns 2NS-R and 2S-R. In these curves, once FRP is considered debonded from the column, the response follows that of columns 2NS-C and 2S-C, respectively. In both of these cases debonding occurred well after the transition to zero axial load was complete and the distinct bump in the resistance curve over the transition range is apparent despite the strengthening FRP's presence. As the FRP does move the column from under-reinforced towards balanced conditions for the zero axial load region, there is still an increase in resistance post

load transitioning up to the point of FRP failure. Figure 5-20 and Figure 5-21 show similar responses for columns 2NS-JR and 2S-JR, however, in these cases the post-FRP failure region did not follow that of their companion non-strengthened columns 2NS-J and 2S-J exactly. As the ruptured FRP was observed to have not debonded from the column surface beyond the rupture location, the post rupture response had hinge regions with no strengthening FRP and non-hinge regions with strengthening FRP. This has only a slight effect on the response as it stiffens the elastic regions of the column but provides no increased strength capacity. The vast majority of the post-rupture displacement can be attributed to hinge rotation, the effect of FRP on the elastic regions is negligible.

5.3.5. Development of Resistance Curve for Column 3NS-CB (Compression Brace)

For the compression braced column 3NS-CB, the resistance-displacement relationship was determined using an approach that incremented the column displacement and solved for an equilibrium condition at each incrementation. The incremental displacement ordinate used was the rotation of the supports. This ordinate, rather than the mid-height displacement, was selected to facilitate the calculation of support moment and simplify the analysis procedure. The following steps were used.

1. Determine sectional response for various axial loads ranging from initial axial load to zero.
2. Define column geometry.
3. Define brace properties including brace geometry using the brace center line, brace area, any initial gaps in brace or between brace and column supports, and yield stress and strain.
4. Discretize the column along the height into a number of discrete elements with length d_y .
5. Set starting axial load equal to initial axial load.
6. Apply an end curvature to the column and calculate the corresponding end moment at the supports. Using the support rotational stiffness, k_θ , found in the experimental study, calculate the specified end rotation.
7. Assume a mid-height curvature and determine the corresponding moment from the moment-curvature relationship.
8. Calculate the total lateral load that would result in the mid-height moment using the moment distribution of $M_w(y) = wy(L - y)/2$ and the support moment as the boundary condition where y is the length along the column, w is the magnitude of the uniformly distributed lateral load, and L is the length of the column.

9. Distribute curvatures along the length of the column according to the corresponding moment values.
10. Integrate curvatures to calculate rotations along the length of the column while applying the mid-height rotation equal to zero boundary condition.
11. Integrate rotations to calculate displacements along the length of the column.
12. Compute brace forces by change in brace geometry based on interior brace point deformation.
13. Compute axial load at that deformation as the difference between initial axial load, vertical brace force, and load lost due to vertical flexural deformation.
14. Compute the second order moments due to axial load and displacements as $M(y) = Px(y)$
15. Compute the restoring moment due to vertical brace force as the total vertical brace force multiplied by the distance from the column center to the center line of the brace at the support location, $M_{RVB} = F_{BV}x_b$.
16. Compute the restoring moment due to the horizontal brace forces as the horizontal brace force at the brace point multiplied by distance along the column, $M_{RVH}(y) = F_{BH}y$, for points between the support and the interior brace and as the horizontal force multiplied by the distance between the brace point and the support, $M_{RVH}(y) = F_{BH}y_b$, for points between two interior braces.
17. Return to Step 8 and add second order moments to moment distribution and continue until convergence of displacement.
18. Compare calculated end rotation from Step 11 with the specified end rotation found in Step 6. If not converged, use the bisection method to determine new trial mid-height curvature and return to Step 7. If converged, continue.
19. Determine mode shape and equivalent SDOF conversion factors (load-mass factors).
20. Increase end curvature and return to step 6.

The procedure above was repeated 20 times with different axial loads ranging from initial load to zero. Following similar logic as for the non-braced columns, a composite resistance curve was constructed based on the load present at each deformation level. Once the vertical load dropped to zero, the zero axial load moment-curvature relationship was used for the remaining analysis.

Figure 5-24 a) and b) show the resistance curve and the load-mass factor for column 3NS-CB. In developing this curve, an initial total gap between all brace components and between the brace components and the supports was taken as 2mm. This value was approximated based on experimental observations. The 2mm looseness in the brace resulted in a slightly reduced stiffness up to approximately 10mm lateral displacement when the brace was activated. Without this looseness, this drop in stiffness would not be apparent. The brace looseness did not have a significant effect on maximum resistance which coincides with yielding of the brace. Following the brace yield, there is a reducing resistance-displacement relationship up to the point when the brace geometry is overwhelmed and there is no longer any horizontal restoring force present. At this point the column is assumed to follow the response curve of column 3NS-W which had the same amount of external prestress condiment but no bracing elements. The use of the compression brace provided a maximum resistance of 229.5kN. This was an increase of 100.3kN over the 129.2kN resistance of the companion prestress confined column 3NS-W and an increase of 101.4kN over the 128.1kN resistance of the companion control column 3NS-C.

5.3.6. Development of Resistance Curves for Columns 3NS-TB1 through 3NS-TB3 (Tension Brace)

For columns with tension braces, the resistance-displacement relationships were determined using the following steps.

1. Determine sectional response for various axial loads ranging from initial axial load to maximum concentric capacity. Also determine axial load-axial strain relationship for all load levels.
2. Define column geometry.
3. Define brace properties including brace geometry using the brace center line, brace area, and yield stress and strain.
4. Discretize the column along the height into a number of discrete elements with length d_y .
5. Set starting axial load equal to initial axial load.
6. Apply an end curvature to the column and calculate the corresponding end moment at the supports. Using the support rotational stiffness, k_θ , found in the experimental study, find the specified end rotation.
7. Assume a mid-height curvature and determine the corresponding moment from the moment-curvature relationship.

8. Find the total lateral load that would result in the mid-height moment using the moment distribution of $M_w(y) = wy(L - y)/2$ and the support moment as the boundary condition where y is the length along the column, w is the magnitude of the uniformly distributed lateral load, and L is the length of the column.
9. Distribute curvatures along the length of the column according to the corresponding moment values.
10. Integrate curvatures to find rotations along the length of the column while applying the mid-height rotation equal to zero boundary condition.
11. Integrate rotations to find displacements along the length of the column.
12. Calculate the axial displacement of the column due to lateral flexural displacement where the incremental displacement along the length of the column in the vertical displacement is $\Delta y = d_y - \tan\theta\Delta x$ and Δx is the relative lateral displacement over then element length, d_y . The total vertical displacement due to flexure (y_{fp}) is the sum of Δy over the column length.
13. Compute brace axial deformation ($y_{b\theta}$) due to end rotation as equal to $y_{b\theta} = 2\left(\frac{b_y}{2} + x_{BEnd}\right)\tan\theta_{End}$ where b_y is the sectional height of the column and $\frac{b_y}{2}$ the assumed center of rotation at the end point, x_{BEnd} is the offset of the brace at the end of the column from the column loaded face to the brace centroid, and θ_{End} is the end rotation of the column at the support.
14. Compute the brace forces due to new brace geometry using total axial deformation of braces and lateral deformation at brace points found in Step 11. The vertical component of the force of the diagonal brace is set equal to the vertical force in the vertical brace as the compatibility equation.
15. Update total axial deformation of the column using axial deformation due to vertical brace force and axial deformation due to lateral flexure and find the corresponding axial load at that deformation using axial load-axial strain relationships.
16. Compute the second order moments due to axial load and displacements as $M_{aP}(y) = Px(y)$
17. Compute the second order moments due to vertical brace force and displacements as

$$M_{aB}(y) = P\left(x(y) + \frac{b_y}{2} + x_{BEnd}\right)$$
18. Compute the restoring moment due to the horizontal brace forces as the horizontal brace force at the brace point multiplied by distance along the column , $M_{RVH}(y) = F_{BH}y$, for

points between the support and the interior brace and as the horizontal force multiplied by the distance between the brace point and the support, $M_{RVH}(y) = F_{BH}y_b$, for points between two interior braces.

19. Compute the moment applied to the supports (outside of the column) due to the horizontal brace force as $M_{BSup} = F_{BH}L_{Sup}$ where F_{BH} is the horizontal brace force component found in Step 14, and L_{Sup} is the length of the support acting as the moment arm. Figure 5-25 shows the forces and dimensions that contribute to M_{BSUP} .
20. Compute the rotation at the support due to M_{BSup} as $\theta_{BSup} = \frac{M_{BSup}}{k_\theta}$
21. Update the end rotation in the column in Step 6 by adding θ_{BSup} found in Step 20.
22. Update the mid-height moment demand on M_w as the difference between the specified mid-height moment (moment at the specified curvature from Step 7) and the destabilizing and restoring moments found in Steps 16, 17 and 18.
23. Return to Step 8 and repeat until convergence of lateral load (w) is achieved.
24. Check convergence of specified end rotation (Step 6) and calculated end rotation (Step 10). If not converged, use bisection method to assume new mid-height curvature and return to Step 7. If converged, continue.
25. Determine mode shape and equivalent SDOF conversion factors.
26. Increase end curvature and return to step 6.

5.3.7. Composite Resistance Curves for Braced Columns

The procedure above was repeated 20 times with different axial loads ranging from initial load to maximum concentric axial load. Although the axial load due to the jack force decreased with increasing flexural displacement, the vertical brace force increased considerably and the vertical reaction force was a compression force in the column. At each axial load increment, the initial axial deformation was computed for each initial load (y_{ap}) and the difference between the initial axial deformation (y_{ap0}) and the axial deformation for the load increment was taken as looseness in the brace for the resistance calculation. Each of the 20 curves used contributed one point to the axial resistance-displacement response for the column when the total axial load on the column (brace force plus jack force) was equal to the axial load used to determine the moment-curvature relationship.

Figure 5-26 shows moment-curvature relationships for columns 3NS-TB1 through 3NS-TB3 up to the concentric axial load capacity, and Figure 5-27 shows the moment-axial load interaction diagrams for the same columns with the 423kN initial axial load for columns 3NS-TB1 and 3NS-TB2 overlaid. Using column 3NS-TB1 as an example, the column is only slightly below the balanced condition with balanced load being approximately 580kN. As the axial load increases beyond the initial load, there is an increase in stiffness and yield moment up to the balanced axial load. After this point the moment capacity decreases with increasing axial load. The curvature ductility capacity decreases with an increase in axial load.

The axial load and axial strain vs. concentric axial load plot for Column 3NS-TB1 is shown in Figure 5-28. The relationship between concentric load and axial strain is approximately linear and could be expressed as a function of the modulus of elasticity for concrete and the transformed sectional area using constant values for elastic modulus. Since this information was known in the development of moment-curvature relationships for various axial loads, it was used in the analysis to compute axial deformation due to load.

Figure 5-29 a) shows the construction of the composite resistance curve for column 3NS-TB1 with the resistance curves for various axial loads plotted alongside the composite curve. Figure 5-29 b) shows the variation in axial load (jack force plus vertical brace force) with horizontal deformation under various initial axial loads. In Figure 5-29 a), the composite resistance curve was built following the steps listed previously up to the point of the crushing of concrete or yield of the tension brace. After this point, a simplified method was employed to account for crushing of the concrete cross-section over the mid-height hinge region. The total axial deformation due to load was computed using a smaller cross-section area. This reduction in area, equal to the area of the section in compression minus the area lost due to concrete crushing, resulted in larger axial deformations which reduced resistance and increased displacement. In Figure 5-29 b) it is apparent that the total brace force converges as the diagonal brace yields. This brace force convergence occurs at approximately 50mm for column 3NS-TB1. At this point, the brace force remains constant and the axial load due to the jack force has been reduced to zero.

Figure 5-30 shows the distributions of curvature ductility ratios over the height of column 3NS-TB1 for three different regions of response: Elastic, slightly plastic, highly plastic. As the axial load on the column (due to the applied load and the brace force) is different for each of these response levels,

the yield ductilities for the column changes as reported in Figure 5-30. For each of these curvature distributions, the moment distributions for each destabilizing or restoring component are shown in Figure 5-31 through Figure 5-33. In Figure 5-30 it is apparent that the end curvature of the column remains elastic throughout the entire response, with curvature ductility values less than 0.1. This elastic response is due to the large moment induced in the support (outside of the column) due to the lateral brace force and support length. As the ductility increases, the curvatures in the span between the support and mid-height decrease up to the point of negative curvatures at the brace point for very high damage. These negative curvatures remain below yield and do not induce damage at the brace points. In Figure 5-31 through Figure 5-33 the moment distributions due to axial load, vertical brace force, horizontal brace restoring force, applied uniformly distributed load, and the total moment distribution are shown. As the column displaces, the magnitude of moment caused by the axial load changes, due to a decrease in jack force and an increase in eccentricity. At large displacements, this moment reduces to zero as the jack force is removed. The moment caused by the vertical component of brace force increases with larger displacements as the moment is the product of an increasing vertical force and increasing eccentricity. The restoring moment due to the horizontal brace increases with deformation up to the point of brace yielding. The destabilizing moment from the uniformly distributed load associated with blast wave increases to balance the restoring force of the brace. The total moment along the length of the column is the sum of all destabilizing and restoring moments.

The rotation distribution and displacement distribution of column 3NS-TB1 for the same three points used in Figure 5-30 through Figure 5-33 are shown in Figure 5-34 and Figure 5-35. It is evident that the majority of the rotation of the column in the inelastic region comes from the rotation over the plastic hinge length. For the highly plastic case, there is double curvature of rotation as the brace induces negative curvatures at the brace point. This effect is relatively negligible on total displacement as the non-hinge displaced shape is essentially linear up to the hinge region where high curvatures result in high change in rotation and displacement. The elastic displaced shape is very similar to that of the unbraced displaced shape for the companion column 3NS-C in the elastic region. This is shown in Figure 5-36 where the mode shapes for the elastic response of the two columns are superimposed. The similarity in elastic mode shapes is an important distinction that will be used to develop simplified design methodologies in the next chapter.

Figure 5-37, Figure 5-38, and Figure 5-39 show the resistance-displacement and the load-mass factor-displacement curves for columns 3NS-TB1, 3NS-TB2, and 3NS-TB3, respectively. The highest resistance is from column 3NS-TB2 which had the most favorable geometry for horizontal restoring forces. Column 3NS-TB3 had slightly less resistance as column 3NS-TB1 due to a lower initial axial load. The magnitude of resistance for these columns is between 3.8 and 4.6 times higher than the control column resistance (3NS-C shown in Figure 5-22).

5.4. Displacement Predictions for Columns

Table 5-2 summarizes the results of the SDOF predictions along with the experimental results. Maximum displacement at mid-height and other gauge locations (if available) are presented along with the time to maximum displacement values. The ratios of maximum SDOF predicted displacement to maximum recorded displacement for all tests are presented in Table 5-3. The experimental and SDOF displacement-time histories for all tests on all columns are presented in Figure C-1 through Figure C-18. Figure 5-40 through Figure 5-43 show scatter plots of the experimental to SDOF predicted displacements for different ranges of response.

5.4.1. Displacement Predictions based on Magnitude of Displacement

The total average ratio of predicted to experimental mid-span displacements of the 62 size population was 0.93 with a standard deviation of 0.42, a coefficient of variation of 44.6% and minimum and maximum values of 0.47 and 3.16, respectively. Comparing the displacement ratio over the entire range does disservice to the representation of accuracy of predictions for moderate sized displacements. Very small displacements, for example those under 10mm as shown in Figure 5-41 and represented in Table 5-3, show higher standard deviation (0.76), higher coefficient of variation (65.6%), and more extreme minimum (0.52) and maximum (3.16) values than other ranges. This is because the displacement ratio is very sensitive to relatively minor differences between experimental and SDOF displacements due to experimental and analytical sources of error discussed further in Section 5.5. By comparison, the standard deviation for the different ranges above 10mm (10 to 40mm, 40 to 80mm and above 80mm) ranged from 0.16 to 0.22 and the coefficient of variation for these ranges was 24.2% and less.

5.4.2. Group 1 Column Displacement Predictions

For the 12 tests conducted on group 1 columns, the average displacement ratio was 1.05 with 0.19 standard deviation, 17.7% coefficient of variation and minimum and maximum values of 0.72 and

1.35 respectively. All of the columns had similar levels of accuracy in the predication of results over all displacement ranges.

5.4.3. Group 2 Column Displacement Predictions

Group 2 column displacements were under-predicted with an average over the 25 tests of 0.78. The standard deviation was 0.21 with a coefficient of variation of 27.4% and minimum and maximum values of 0.47 and 1.22 respectively. In general, the displacement predictions for the columns in group 2 were low for the small displacement shots and fairly accurate for the higher displacement shots. This can be seen in the displacement-time histories for group 2 columns in Figure C-5 through Figure C-12. Examination of Table 5-2 shows that for these columns, the low displacement under-predictions (Shot 1) were off by less than 4mm for all cases. The low experimental displacements for these tests (between 4.7 and 8.6mm) amplified this relatively small prediction error when represented as displacement ratios.

5.4.4. Control Column Predictions

The 14 control column tests across all groups provided SDOF predicated displacement to test comparisons with an average displacement ratio of 0.93, a standard deviation of 0.25, a coefficient of variation of 26.5% and minimum and maximum values of 0.55 to 1.35. The SDOF predictions were generally accurate for predicting the maximum displacement of control columns.

5.4.5. Group 2 Seismic and Non-Seismic Columns

There was no real difference in the accuracy of predictions between the seismic and non-seismic columns in group 2 and that of group 2 as a whole.

5.4.6. Tension Braced Columns

The average displacement ratio for the 15 tension braced column tests was 1.00 with a high standard deviation of 0.68, a high coefficient of variation of 68.1%, and a high range of minimum and maximum values of 0.56 and 3.16 respectively. Generally, the displacement predictions were more accurate for the lower displacement levels for the columns and under predicted for higher displacement levels. Section 5.5 discusses several experimental and analytical sources of error that pertain to the accurate prediction of displacements for columns in this group. In general, the resistance curves were extremely sensitive to axial deformation and support rotation. These quantities are difficult to calculate with the same confidence as flexural deformation; so inaccuracies in predicted displacements can be expected. Furthermore, things like accumulation of damage, doweling of bolts

into supports preventing support vertical translation, and inaccuracies in assumptions of the center of rotation of supports all lead to non-conservative displacement predictions.

Some of the inaccuracies in prediction can be attributed to very low experimental displacements. For example, 3NS-TB2 shot 1 had an experimentally recorded maximum displacement of 1.5mm and a SDOF prediction of 4.7mm resulting in the maximum displacement ratio for the set of 3.16 despite the small relative displacement difference of 3.2mm. The predictions for displacements are generally more accurate for the shots 1 and 2 for the three columns. This is an important distinction as these tests represent response prior to yielding of the column which is the critical design region for columns.

5.4.7. Compression Braced Columns

There was one compression braced column that had 5 shots. The average displacement ratio for these shots was 1.15 with a standard deviation of 0.52, a coefficient of variation of 45.5%, and a minimum and maximum value of 0.82 and 2.06 respectively. The first test had an experimental displacement of 4.1mm and a SDOF predicted displacement of 8.4mm resulting in the maximum displacement ratio for the set of 2.06 despite the small relative difference of 4.5mm. If this point was removed from the data set, the displacement ratios would have an average of 0.92 with a standard deviation of 0.15, a coefficient of variation of 16.6% and minimum and maximum values of 0.82 and 1.15. Considering the first point as an outlier shows that the predictions for the compression braced column are accurate over all ranges.

5.4.8. Columns Retrofitted for Confinement

There were five columns in total that were retrofitted with a confinement retrofit alone. Two columns in group 1, 1NS-J and 1S-J, used FRP confinement. Two columns in group 2, 2NS-J and 2S-J, used FRP confinement. One column in group 3, 3NS-W, used prestress steel confinement. There were a total of 16 shots on these columns with an average displacement ratio of 0.94, a standard deviation of 0.25, a coefficient of variation of 26.3%, and minimum and maximum values of 0.57 and 1.47 respectively. All of the predictions for group 1 columns were between 0.72 and 1.22 representing accurate predictions for all displacement ranges. In group 2, column 2NS-J used accurate predictions for all tests. Column 2S-J had a high displacement ratio for the first shot where experimental displacement was 7.7mm and SDOF displacement was 4.6mm resulting in a displacement ratio of 0.60 showing the sensitivity of this measurement at low displacement levels. Column 2S-J was

subjected to 4 shots. For shot 4 the column had a maximum displacement of 256.5mm recorded experimentally and a maximum predicted displacement of 146.7mm resulting in a displacement ratio of 0.57. Despite the low displacement ratio, the SDOF method accurately predicted blowout of the column. Column 3NS-W was subjected to 4 shots. Of those shots, 3 were used in the statistical analysis as shot 4 resulted in blowout of the experimental test and blowout from the SDOF analysis such that no result could be predicted. The first shot for the column resulted in a high displacement ratio of 1.47 despite the relative difference between SDOF and experimental displacements of 1.4mm due to the very low experimental displacement of 3.5mm. The other two displacements were accurately predicted.

In general, the prediction of confinement retrofit response was accurate for all columns over all ranges of response. The standard deviation and coefficient of variation for the displacement ratio values for this set are skewed by the overly sensitive small displacement values.

5.4.9. FRP Strengthened Columns

Four columns in group 2 were strengthened with FRP with a total of 12 shots. Two of these columns, 2NS-R and 2S-R, were retrofitted only with longitudinal FRP while two, 2NS-JR and 2S-JR, were retrofitted with the combination of longitudinal and confinement FRP. The average displacement ratio for these columns was 0.74 with a standard deviation of 0.20, a coefficient of variation of 27.1% and minimum and maximum values of 0.47 and 1.15 respectively. These results show consistent minor under-prediction of displacement for these columns across all displacement ranges.

5.4.10. Validation of Composite Curves

As a final validation of the use of composite resistance curves which account for the variable axial load during the response of the column, the dynamic analysis of column 2S-J was conducted for both the composite resistance curve and the resistance curve under full axial load. These resistance curves are shown in Figure 5-44. The results of the dynamic analysis are shown in Figure 5-45a through Figure 5-45d for shots 1 through 4 respectively. The predicted displacements for both resistance curves is nearly identical for the first shot shown in Figure 5-45a due to the low displacement level causing little variation between resistance curves. As the pressures increase, the displacement response increases and the column displacement predictions diverge considerably. Assessing the larger displacements in Figure 5-45b through Figure 5-45d shows that the larger displacements are

poorly predicted by the full axial load resistance curve while better predictions are made with the composite curve.

5.5. Sources of Error

Two different kinds of error influenced the accuracy of the results. The first is experimental error and the second is inaccurate analytical assumptions. The following section describes these sources and their impact on the accuracy of the results.

5.5.1. Experimental Error

There were three main sources of experimental error that affect the accuracy of the predicted results.

5.5.1.1. Movement of the Shock Tube

When the diaphragms holding back the pressurized driver air ruptures, there is recoil which moves the entire shock tube and column assembly away from the fixed-in place displacement gauge mount. This has the effect of recording a positive displacement that begins prior to the arrival of the shock front at the specimen. Figure 5-46 shows the movement of the shock tube under three different magnitudes of shock waves taken from a different set of data outside the scope of this study. The movement of the shock tube begins approximately 10ms prior to the shock front arrival at the specimen and continues until approximately 15ms after the shock front arrival at which point the movement reverses. This reversal is due to the impact of the shock front reflecting on at the specimen location and pushing the shock tube forwards. As series of internal reflections within the shock tube continue to cause vibrations of the shock tube for approximately the next 100ms. The magnitude of the displacements of the shock tube are up to approximately 3mm for the pressures given in Figure 5-46. The shock tube displacements are typically much less than the maximum displacements in this study, but still enough to have an impact on the results. The primary impact that this shock tube movement has on the results is an apparent delay in the displacement-time history response of the columns being tested. This delay is really the difference between the positive displacement of the column (moving towards the fixed in place displacement gauge mounts) and the initial negative displacement of the shock tube (moving away from the fixed in place gauge mounts). This delay is apparent on all experimental displacement-time histories for all columns (see Figure 5-47 between -10ms and +15ms as an example). The second effect the shock tube movement has on the recorded response is an increase in maximum displacement due to the typical coincidence of maximum

specimen displacement with maximum shock tube displacement. Finally, the velocity (and acceleration) of the recorded response from zero displacement to maximum displacement will be greater than the actual values due to the apparent delay in response and the increase in maximum. It is estimate that these effects could result in errors between 1 and 10% of the recorded displacement values depending on the specimen and the applied pressure.

5.5.1.2. Loose or Flexible Displacement Gauge Mounts

The connections of displacement gauges to the column were occasional loose or too flexible. Figure 5-47 shows how two displacement gauges (at 1/3 height and 1/6 height) respond when the gauge mount is poorly installed. In this cause the vibration of the gauge mount itself is visible on the displacement-time history as what appears to be higher frequency response of a different mode. In reality, this high frequency vibration is an error in measurement. The amplitude of the gauge vibration also introduces an error in the maximum displacement. In several of the columns in group 3, due to the size of the retrofits, the gauge mounts were long bolts that were observed to bend during testing resulting in up to 10mm of movement at the location of the LVDT. This has the effect of the recorded displacements being higher than the actual displacements. Depending on the gauge and the test specimen and pressure, the error in displacement readings from this source is estimated at up to 20%. This high error primarily effects the results of columns 3NS-W, 3NS-CD, 3NS-TB1, 3NS-TB2, and 3NS-TB3.

5.5.1.3. Gap between the Load Transfer Device (LTD) and Column Surface

The final source of experimental error affects columns in group 3, primarily columns 3NS-TB1 through 3NS-TB3. For these columns there was a gap between the LTD and the column surface of approximately 10mm prior to testing. This gap was due to the LTD being bent in a prior round of testing and the spacers used to fill the gap between the LTD beams not being sized correctly to reduce this gap. The effect of this gap is an actual delay between the shock front arrival time and the time at which the column is loaded. Figure 5-48 shows this delay at approximately 10ms for column 3NS-TB1 Shot 1. This delay also has the apparent delay from the shock tube movement, therefore the exact delay is not known. The primary effect of this delay would be to offset the displacement-time history by the magnitude of the delay making all predicted displacement-time histories appear to start significantly earlier than the actual experimental values. As the LTD has negligible resistance, due to the conservation of energy principles, this likely does not have a significant effect on the maximum displacement recorded.

5.5.2. Analytical Error

There were three main sources of error that contributed to inaccuracies in the analysis.

5.5.2.1. Rebound into Negative Displacements

The shock tube and the LTD width prevented rebound of the columns into negative displacements. This was not accounted for in the analysis and any rebound less than zero would have been resisted by the LTD in flexure as it was supported by the sides of the shock tube. This has no impact on maximum displacement predictions and only minimal impact on the predictions of response for small displacements after the maximum.

5.5.2.2. Load transfer device separation on rebound

The LTD was not physically connected to the specimen. This means that on rebound, it would separate from the surface of the column and not act as a supported mass in some cases. Furthermore, the negative pressures from shock wave would only act on the LTD mass, not the column mass and resistance. The initial displacement up to maximum displacement is not effected by the LTD separation as maximum displacement was found to occur within the positive phase of the shock wave. Post peak response is effected by the separation of the LTD, however, the free vibration response of the column is suspect but was not of primary concern for this analysis.

5.5.2.3. Hysteresis and Yield Points

As was mentioned in Section 5.2.3, the hysteretic model does not address the yield of members occurring well after non-linearity of the resistance-displacement curve due to variable axial load (see Figure 5-8 for an example of the yield point on a resistance curve). Therefore, the post-peak response (unloading and free vibration) is only an approximation of the actual member response.

5.6. Summary

A detailed description of the analysis performed in this study was provided in this chapter. SDOF dynamic modeling was used with robustly developed resistance-displacement and SDOF conversion factor (load-mass factor) curves. The results of the analysis show that the SDOF method is able to reliably predict displacements of all the columns in this study.

Table 5-1: Mass and Loaded Area of Columns for SDOF Analysis

Column	Loaded Area (m ²)	Column Mass (kg)	LTD Mass (kg)	Retrofit Mass (kg)	Total Mass (kg)
Group 1	4.54	71	394	-	465
Group 2	4.23	107	209	-	316
3NS-C	4.23	107	209	-	316
3NS-W	4.23	107	236	217	560
3NS-CB	4.23	107	236	267	610
3NS-TB1	4.23	107	236	50	393
3NS-TB2	4.23	107	236	50	393
3NS-TB3	4.23	107	236	50	393

Table 5-2: Shock Wave Properties, Experimental Displacements and Time to Maximum Displacements, and SDOF Predicted Maximum Displacements and Time to Maximum Displacements

Specimen		Shock Wave Properties			Experimental						Single Degree of Freedom					
					Maximum Displacement (at location relative to height of column)			Time to Maximum (at location relative to height of column)			Maximum Displacement (at location relative to height of column)			Time to Maximum (at location relative to height of column)		
		P_r	I_r	t_d	$d_{max EXP}$			$t_{max EXP}$			$d_{max SDOF}$			$t_{max SDOF}$		
					1/2	1/3	1/6	1/2	1/3	1/6	1/2	1/3	1/6	1/2	1/3	1/6
		kPa	kPa-ms	ms	mm	mm	mm	ms	ms	ms	mm	mm	mm	ms	ms	ms
1NS-C-	1	13.7	179.8	21.7	14.6			29.6			15.7			23.8		
	2	24.3	257.7	26.4	27.5			35.1			24.9			29.1		
	3	65.1	765.3	34.4	171.1			57.4			213.4			72.6		
1S-C-	1	13	155.4	20.7	10.9			28.7			14.8			22.4		
	2	24	226.7	23.3	21.5			29.6			22.9			24.6		
	3	63.1	621.5	26.3	156			53.7			141.3			49.4		
1NS-J-	1	12.6	145.8	20.9	12.8		7.7	28		30.3	15.6		6.9	23.0		23.0
	2	19.4	240.8	23.7	25.6		15.2	33.3		32.4	26.9		10.9	26.0		26.0
	3	58.8	685.8	23.6	160.9		71.5	52.9		53.5	150.9		48.9	47.2		47.2
1S-J-	1	14.3	165	24.1	13.4		8	32.5		33.6	16.3		6.4	23.4		23.4
	2	20.2	227.6	23.5	27.1		15.9	40		39	24.2		8.8	25.0		25.0
	3	52.5	555.1	22.6	135.3		59.7	51.7		52.7	97.2		30.7	36.8		36.8
2NS-C-	1	14	140.4	18	8.2			16.6			4.5			9.8		
	2	41.8	478.4	28.6	36.5			20.9			25.6			17.8		
	3	81	1073.5	40.5	158			35.6			171.5			40.4		
2S-C-	1	14.4	131.1	18.4	4.7			18.4			5.7			12.0		
	2	47.7	687.5	45.8	37.3			21.1			28.9			17.4		
	3	70	686.7	23	129			28			93.9			26.2		
2NS-J-	1	13.5	168.3	21.7	8			18.5			6.3			11.6		
	2	44.8	509.9	28.9	36.3			21.4			30.5			19.6		
	3	73.2	643	21.2	85.5			24.6			81.3			24.9		
2S-J-	1	14.6	188.2	21.6	7.7			17.3			4.6			10.2		
	2	43.7	420.3	25.7	25.1			18.5			17.8			14.8		
	3	75.3	787.2	22.2	99.5			24.8			119.7			28.8		
	4	85	839.2	19.9	256.5			64			146.7			29.2		

Table 5-2: Shock Wave Properties, Experimental Displacements and Time to Maximum Displacements, and SDOF Predicted Maximum Displacements and Time to Maximum Displacements – Continued

Specimen		Shock Wave Properties			Experimental						Single Degree of Freedom					
					Maximum Displacement (at location relative to height of column)			Time to Maximum (at location relative to height of column)			Maximum Displacement (at location relative to height of column)			Time to Maximum (at location relative to height of column)		
		P_r	I_r	t_d	$d_{max EXP}$			$t_{max EXP}$			$d_{max SDOF}$			$t_{max SDOF}$		
					1/2	1/3	1/6	1/2	1/3	1/6	1/2	1/3	1/6	1/2	1/3	1/6
		kPa	kPa-ms	ms	mm	mm	mm	ms	ms	ms	mm	mm	mm	ms	ms	ms
2NS-R-	1	13.7	189.3	22.1	8.6			18.4			5.1			11.4		
	2	47.2	371.7	20.2	24.3			17.5			16.3			12.4		
	3	99.4	716	23	127.6			29.9			80.9			25.6		
2S-R-	1	14.5	157.7	18.3	7.3			16.8			6.5			12.0		
	2	46.5	474.2	24.7	22.7			16.4			19.5			13.0		
	3	85.3	776	22.8	97.3			26.8			96.2			27.2		
2NS-JR-	1	13.4	130.1	17.9	7.3			20.6			5.7			11.0		
	2	40.6	355.6	20	32.6			17.8			15.3			11.6		
	3	79.8	678.5	23.7	70.1			23.4			80.9			25.6		
2S-JR-	1	13.8	168	20.9	8.1			17.1			4.2			9.6		
	2	47	443.4	26.1	31.4			17.2			19.5			13.0		
	3	88.2	793.2	22.1	102.7			25.5			74.2			24.4		
3NS-C-	1	13.1	135.7	18.5	6.4	5.8	3.9	17.4	17.4	16.5	3.9	3.4	1.9	10.4	10.4	10.4
	2	39.4	378.1	26	20	18.8	12.6	16.3	16	16.4	17.3	13.4	6.5	15.6	15.6	15.6
	3	78.5	996.6	26.7	152			32			blowout			blowout		
3NS-W-	1	9.2	104	18.4	3.1	4.2	2.1	19.9	20.2	22.3	4.5	3.8	2.1	16.6	16.6	16.6
	2	38.2	380.2	20.6	20.5	21.3	12.1	22.2	17.2	23.2	17.7	13.7	6.9	18.0	18.0	18.0
	3	75.7	739.3	25	63	52.2	29.2	26.9	29.4	27.6	61.0	45.0	22.0	28.0	28.0	28.0
	4	83.2	970.7	26.7	122.5			39.1			blowout			blowout		
3NS-CB-	1	11.9	141.1	19.5	4.1	3.7	2.6	19.6	20.1	21.7	8.4	7.1	3.8	19.6	19.6	19.6
	2	36.3	381.3	23.3	17.1	15.4	10.6	19.8	21.1	20.9	19.7	15.2	7.4	16.8	16.8	16.8
	3	70.6	780.4	27	50	40.8	25.1	25.7	25	25.8	43.4	31.6	13.9	20.8	20.8	20.8
	4	84	983.3	27.4	72.5	56.6	33.1	26.9	26	26.8	59.2	42.7	18.6	24.2	24.2	24.2
	5	84.4	1669.8	59.1	103.3	77.8	43.1	36.7	35.8	39.3	89.0	63.8	27.3	34.2	34.2	34.2

Table 5-2(Cont'd): Shock Wave Properties, Experimental Displacements and Time to Maximum Displacements, and SDOF Predicted Maximum Displacements and Time to Maximum Displacements – Continued

Specimen		Shock Wave Properties			Experimental						Single Degree of Freedom					
					Maximum Displacement (at location relative to height of column)			Time to Maximum (at location relative to height of column)			Maximum Displacement (at location relative to height of column)			Time to Maximum (at location relative to height of column)		
		P_r	I_r	t_d	$d_{max EXP}$			$t_{max EXP}$			$d_{max SDOF}$			$t_{max SDOF}$		
					1/2	1/3	1/6	1/2	1/3	1/6	1/2	1/3	1/6	1/2	1/3	1/6
		kPa	kPa-ms	ms	mm	mm	mm	ms	ms	ms	mm	mm	mm	ms	ms	ms
3NS-TB1-	1	11.2	155	20.5	6.3	5	2.8	21.2*	21.5*	21.9*	7.0	5.9	5.5	15.2	15.2	15.2
	2	34.8	375.9	22.9	24.4	21.3	12.9	19.2*	18.7*	17.5*	20.7	15.9	14.3	15.0	15.0	15.0
	3	73.8	888.1	28.4	55.1	42.3	24.7	21.5*	19.5*	17.7*	40.6	29.5	26.2	13.2	13.2	13.2
	4	82.6	982.6	27.8	72.6	55	34.5	23.9*	20.4*	19.8*	44.1	31.9	28.2	13.2	13.2	13.2
	5	79	1610.2	60.4	75.8	57	34.7	24.7*	21.3*	19.9*	42.6	30.9	27.4	13.2	13.2	13.2
3NS-TB2-	1	13.5	151.4	20.2	1.5	1.8	1.2	18.3*	16*	16*	4.7	3.7	2.2	12.4	12.4	12.4
	2	34.6	447	23.5	21.1	18.2	12.1	18.5*	19.1*	18.9*	17.0	11.9	6.8	13.2	13.2	13.2
	3	68.4	812.8	25.7	44	34	23.4	19.5*	19*	17.5*	29.1	19.1	10.6	12.4	12.4	12.4
	4	80.3	956.3	27.7	51.6	38	29	20.3*	19.1*	18.3*	33.3	21.8	12.0	11.6	11.6	11.6
	5	83.9	1894	60.4	47.8	35.7	29.1	21.2*	19.1*	18.7*	34.9	22.8	12.6	11.6	11.6	11.6
3NS-TB3-	1	11.2	146	21	4.5	4	2.5	19.3*	21.4*	22.9*	8.5	7.1	3.9	16.2	16.2	16.2
	2	36.9	436.6	23.7	23.9	22.8	14.7	17.9*	19.2*	18.2*	26.8	20.5	10.4	16.4	16.4	16.4
	3	67.9	879.5	28.3	53.7	47.5	29.2	20.4*	20.4*	18.3*	44.7	32.4	15.0	13.8	13.8	13.8
	4	83.1	1037.2	29.9	75.8	56.9	34.7	23.1*	21.4*	19.1*	50.8	36.7	16.8	14.2	14.2	14.2
	5	83.1	1849.6	63.3	76	58.8	35.1	24.9*	22.2*	19.7*	49.9	36.0	16.6	14.6	14.6	14.6

* See section 5.5.1.3 for a discussion on high time to maximum and delay in start of response for tension braced columns.

Table 5-3: Summary of Experimental and SDOF Predicted Responses for Various Column Groups

Data Set	SDOF Mid-Height Displacement Ratio, $\frac{d_{maxSDOF}}{d_{maxEXP}}$					n
	Mean	Std Dev	COV	Min	Max	
All Specimens	0.93	0.42	44.6%	0.47	3.16	62
Group 1 Columns	1.05	0.19	17.7%	0.72	1.35	12
Group 2 Columns	0.78	0.21	27.4%	0.47	1.22	25
Control Columns	0.93	0.25	26.5%	0.55	1.35	14
Group 2 Non-Seismic Columns	0.77	0.21	27.5%	0.47	1.15	12
Group 2 Seismic Columns	0.80	0.23	28.2%	0.52	1.22	13
Tension Braced Columns	1.00	0.68	68.1%	0.56	3.16	15
Compression Braced Column	1.15	0.52	45.5%	0.82	2.06	5
Confinement Retrofit Columns	0.94	0.25	26.3%	0.57	1.47	16
FRP Strengthened Columns	0.74	0.20	27.1%	0.47	1.15	12
All Retrofitted Columns	0.93	0.45	48.9%	0.47	3.16	48
$d_{max} < 10\text{mm}$	1.16	0.76	65.6%	0.52	3.16	14
$10\text{mm} < d_{max} < 40\text{mm}$	0.91	0.22	24.2%	0.47	1.35	22
$40\text{mm} < d_{max} < 80\text{mm}$	0.76	0.16	21.5%	0.56	1.15	13
$d_{max} > 80\text{ mm}$	0.89	0.21	23.7%	0.57	1.25	13

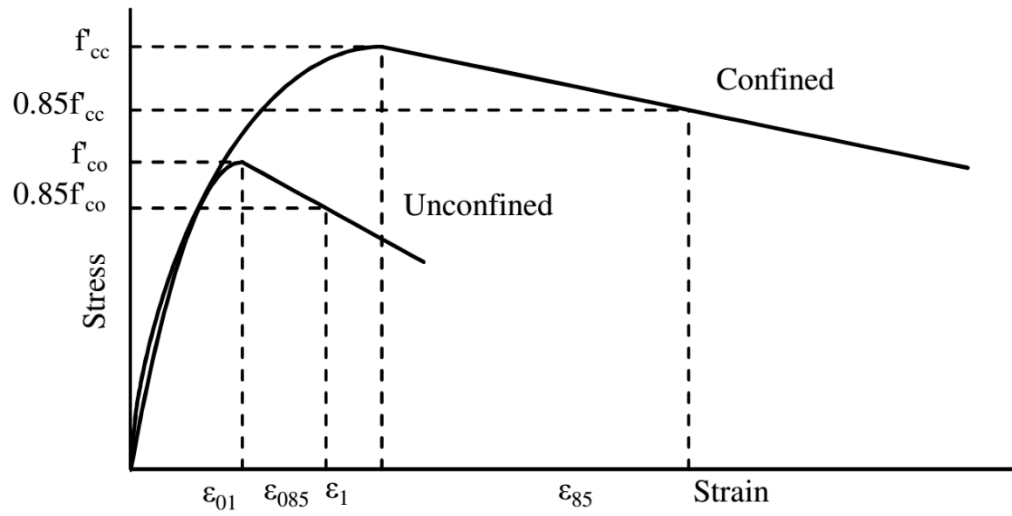


Figure 5-1: Confined and Unconfined Concrete Material Model

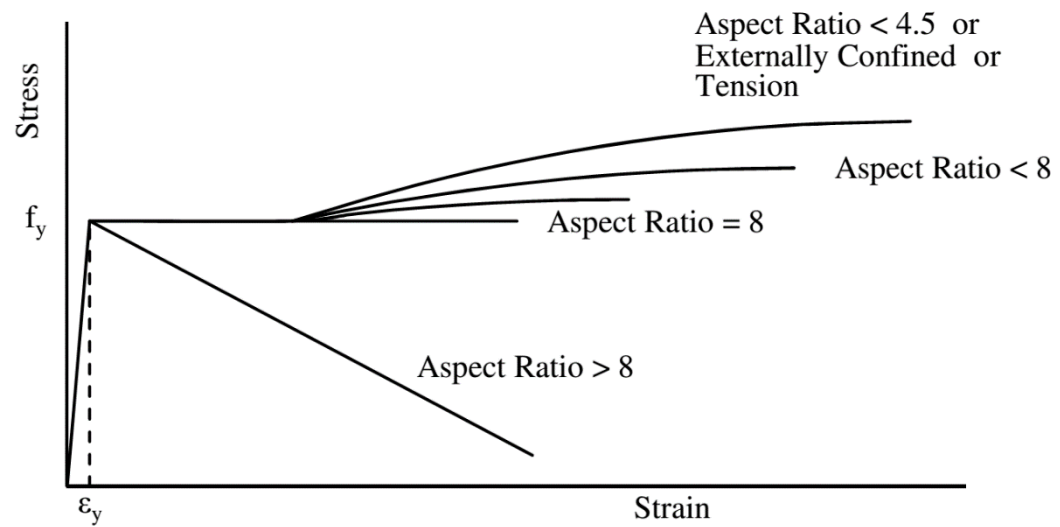
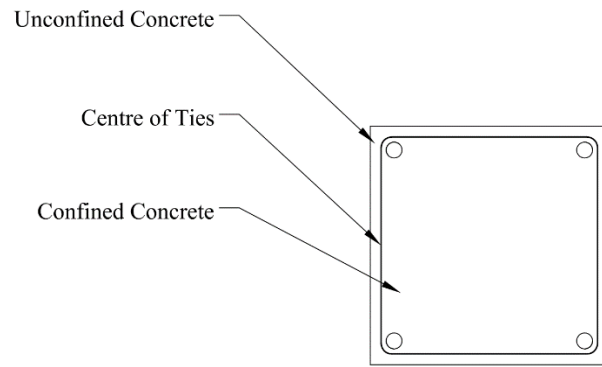
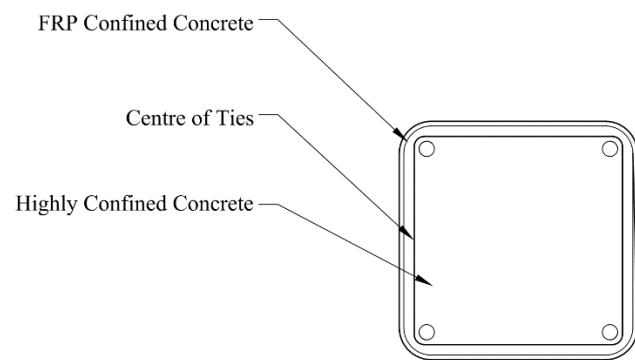


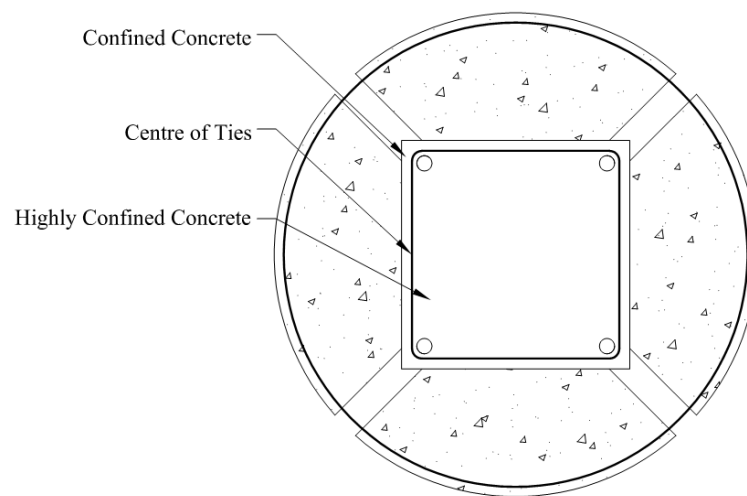
Figure 5-2: Compression and Tension Reinforcing Steel Material Model



a) Columns Not Confined with FRP or Prestressed Steel Jackets



b) Columns Confined with FRP Jackets



c) Columns Confined with Prestressed Steel Jackets

Figure 5-3: Locations of Confined and Unconfined Concrete in Different Sections

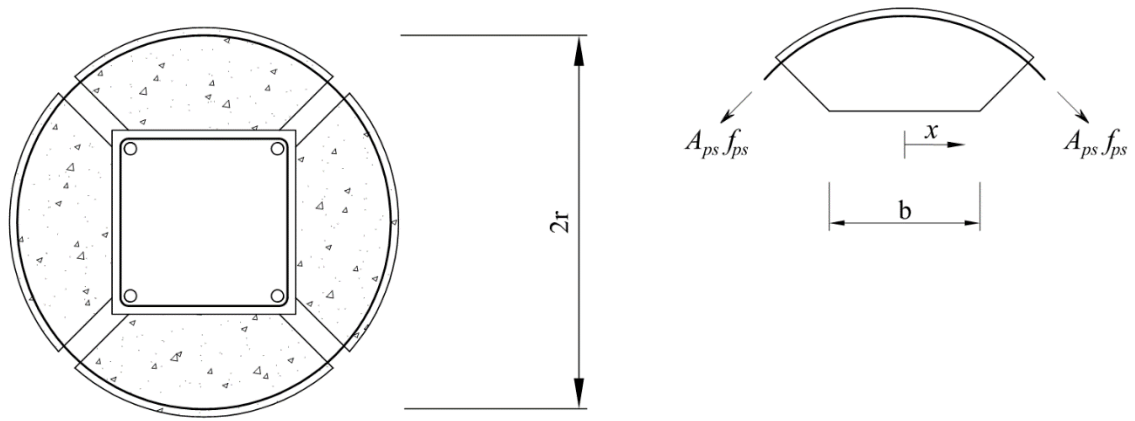


Figure 5-4: Geometry of Confinement of Column with Prestressed Steel and Wedges for Development of Equivalent Confinement Pressure

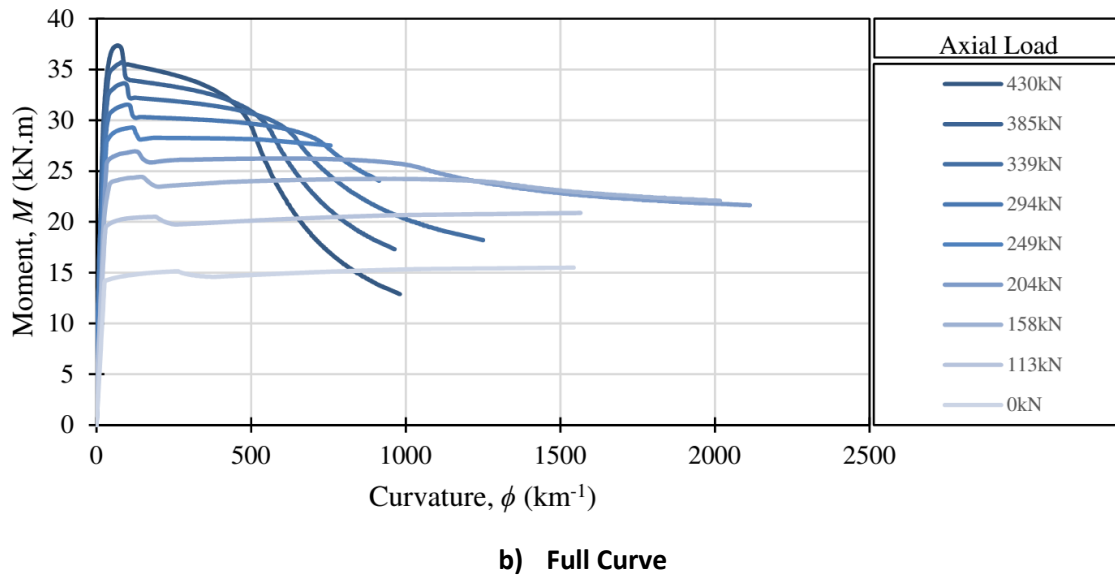
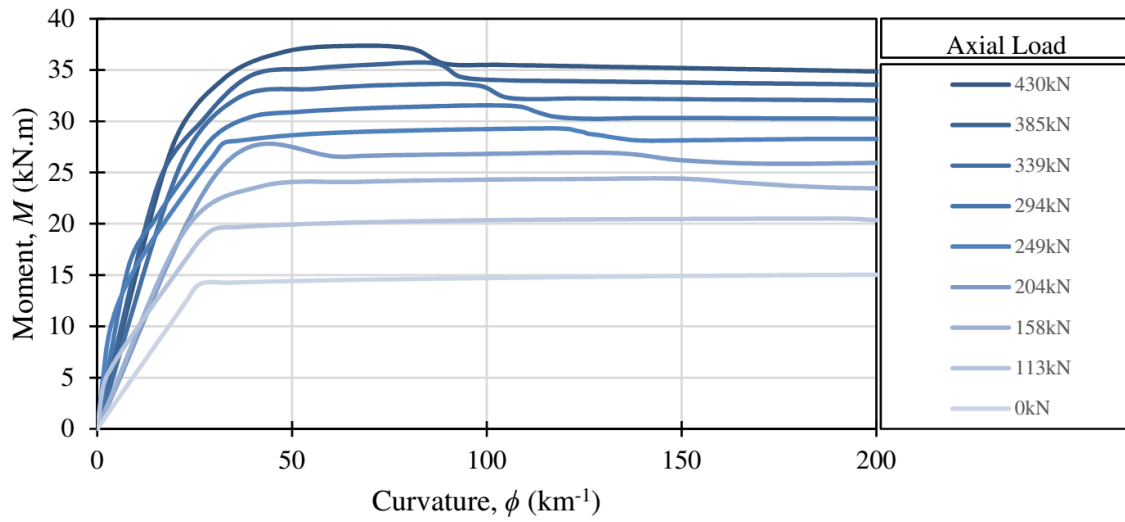
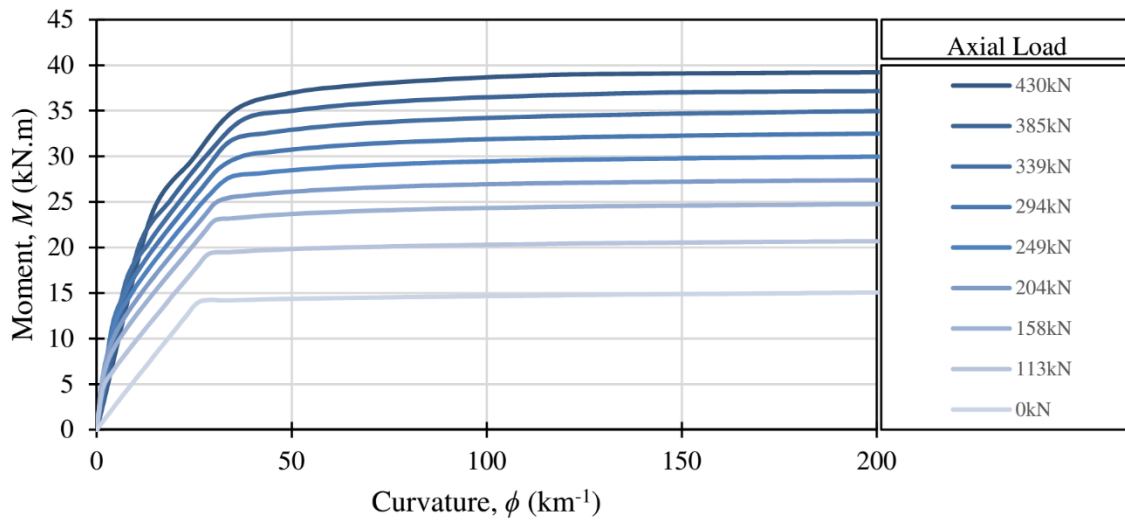
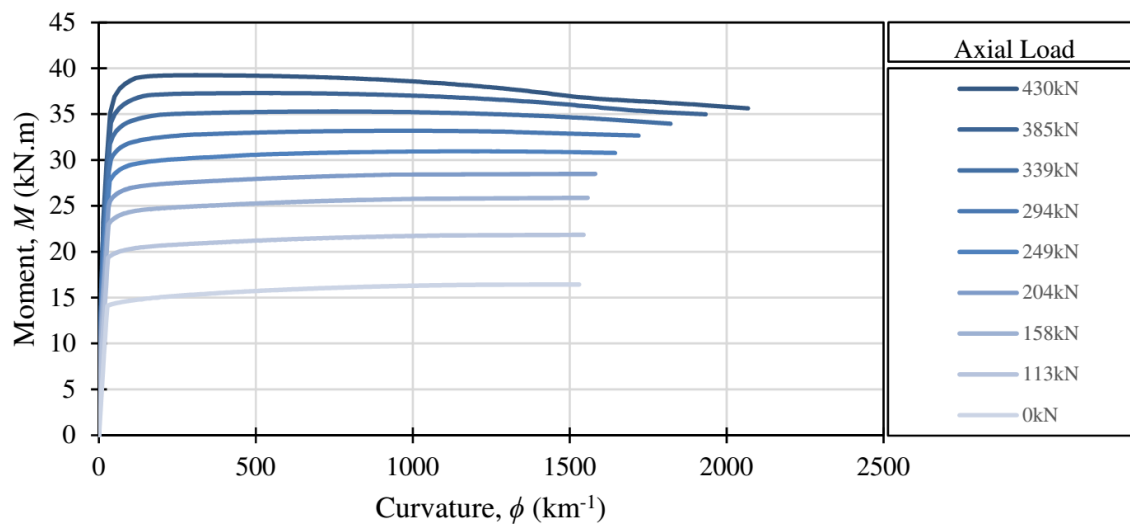


Figure 5-5: Moment-Curvature Relationship for Column 2NSC under Various Axial Loads



a) Initial (Elastic) Region



b) Full Curve

Figure 5-6: Moment-Curvature Relationship for Column 2SJ under Various Axial Loads

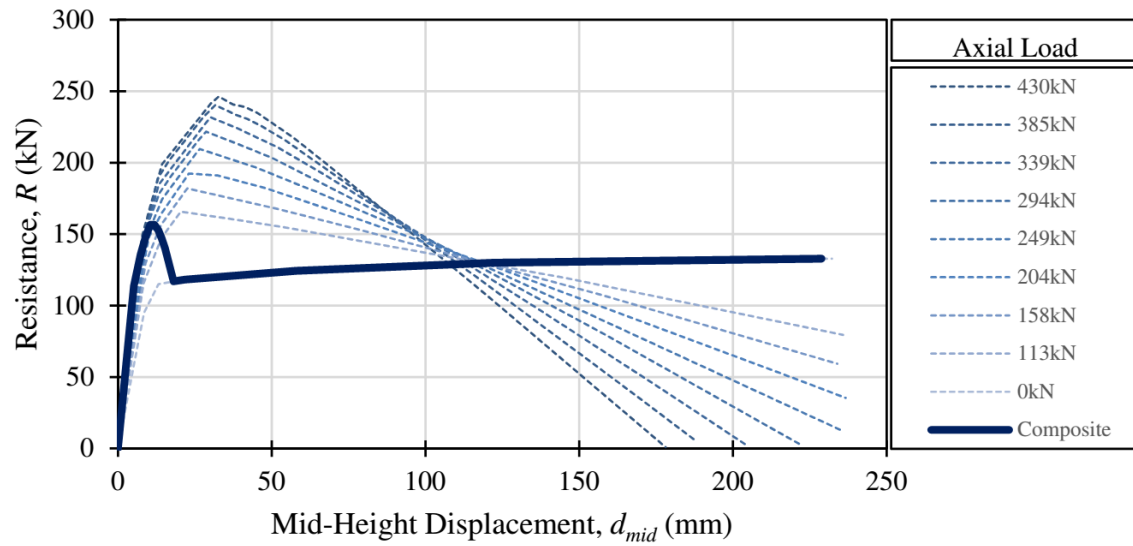
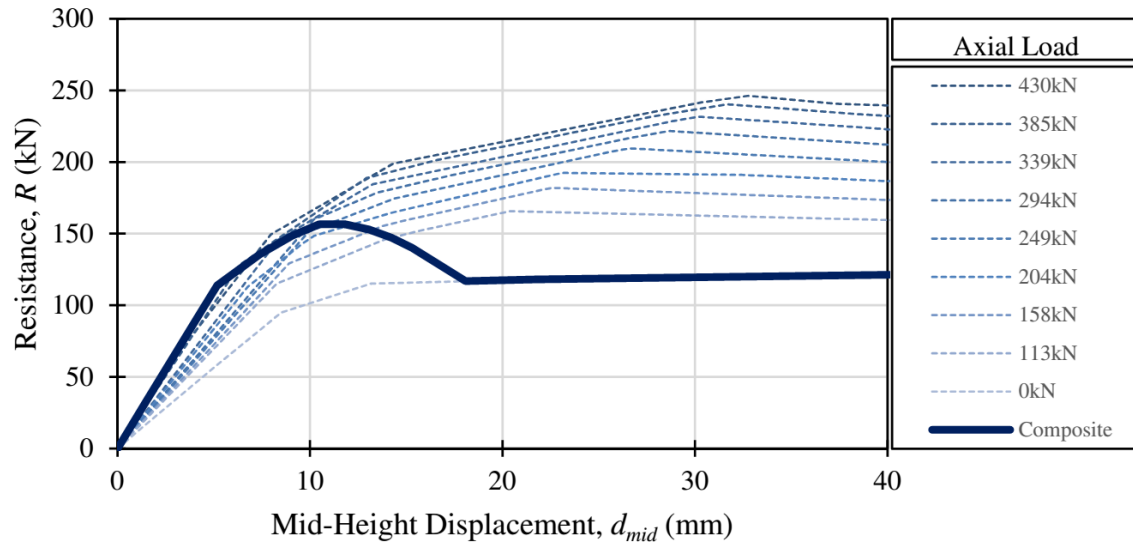
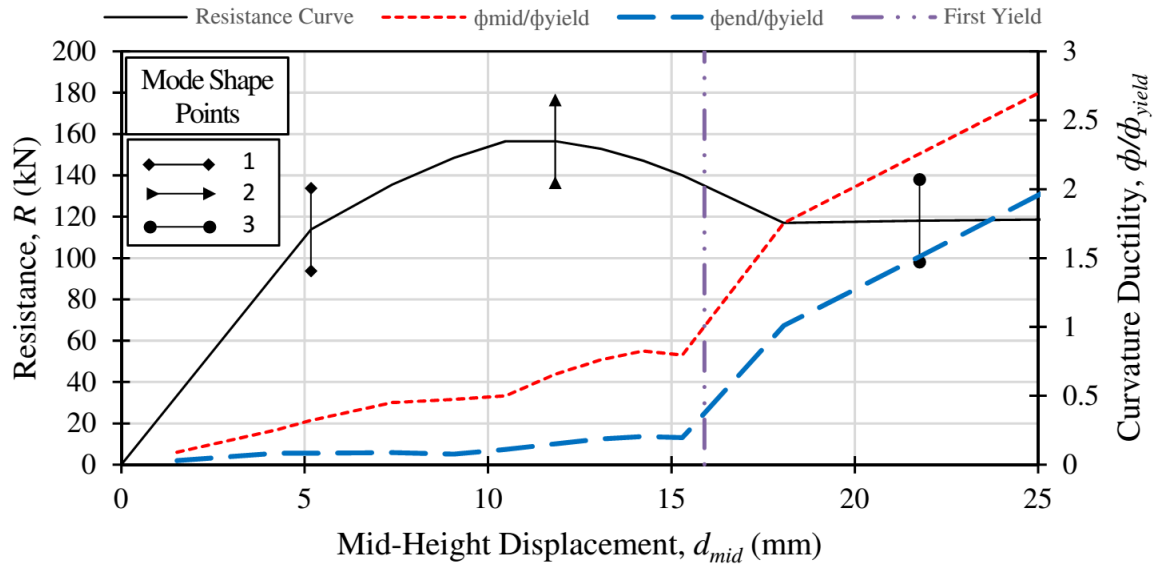
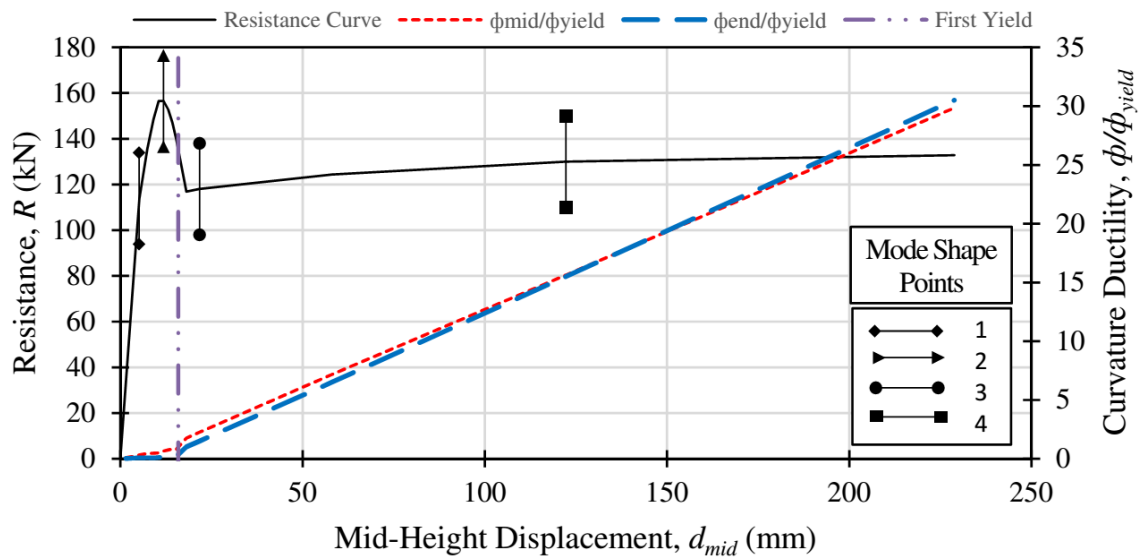


Figure 5-7: Construction of Composite Resistance Curve for Column 2S-J using Variation in Axial Loads



a) Early Response Region



b) Full Curve

Figure 5-8: Curvature Ductilities for End and Mid-Span of Column 2S-J Plotted with Composite Resistance Curve and First Yield Point Identified along with Identification Several Points of Interest for Mode Shape Plots. (Mode Shapes Shown in Figure 5-9)

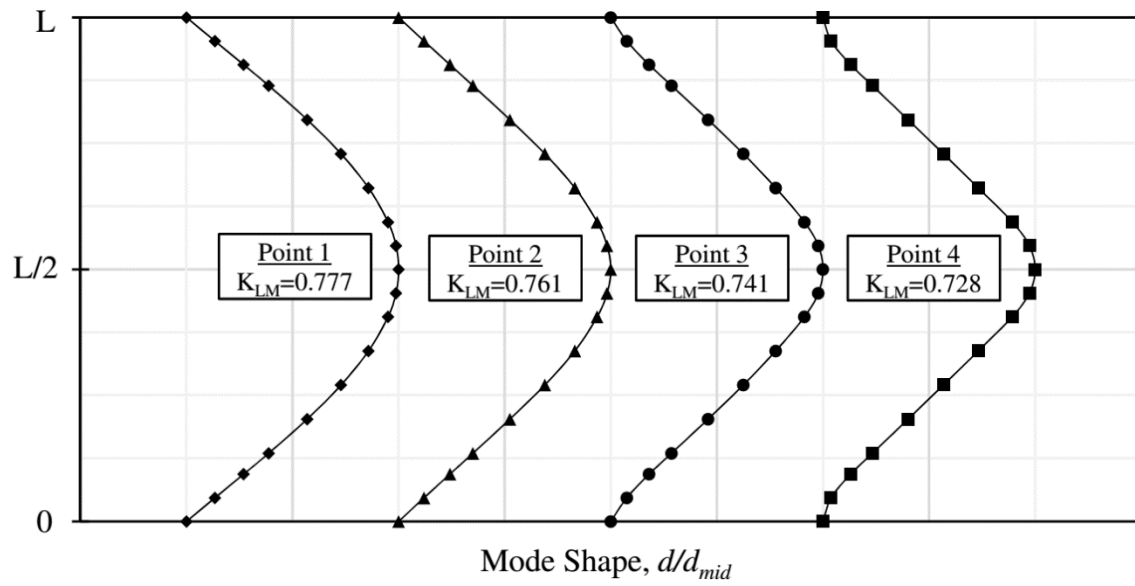
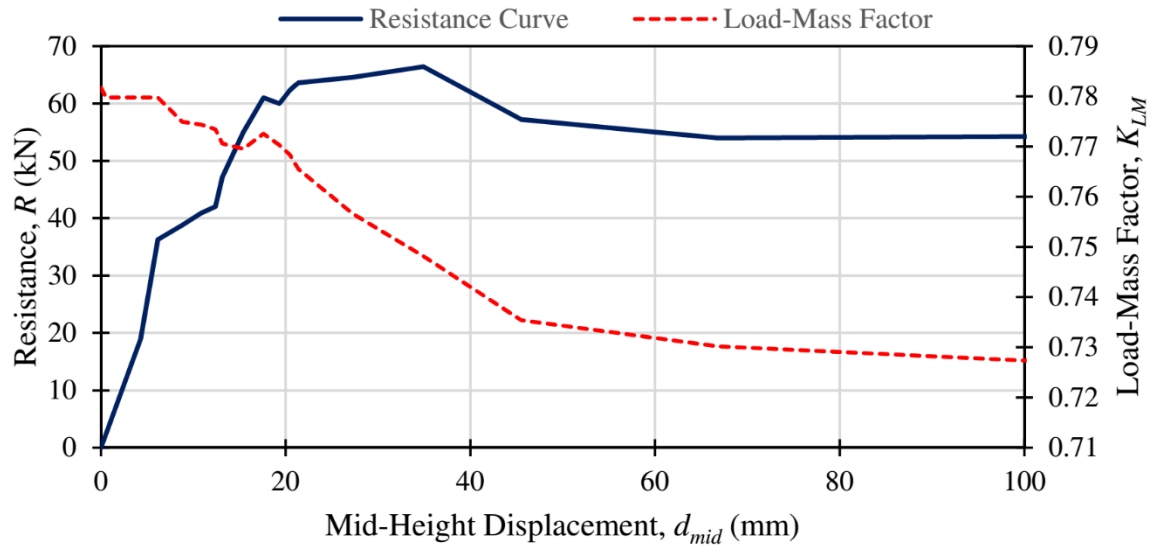
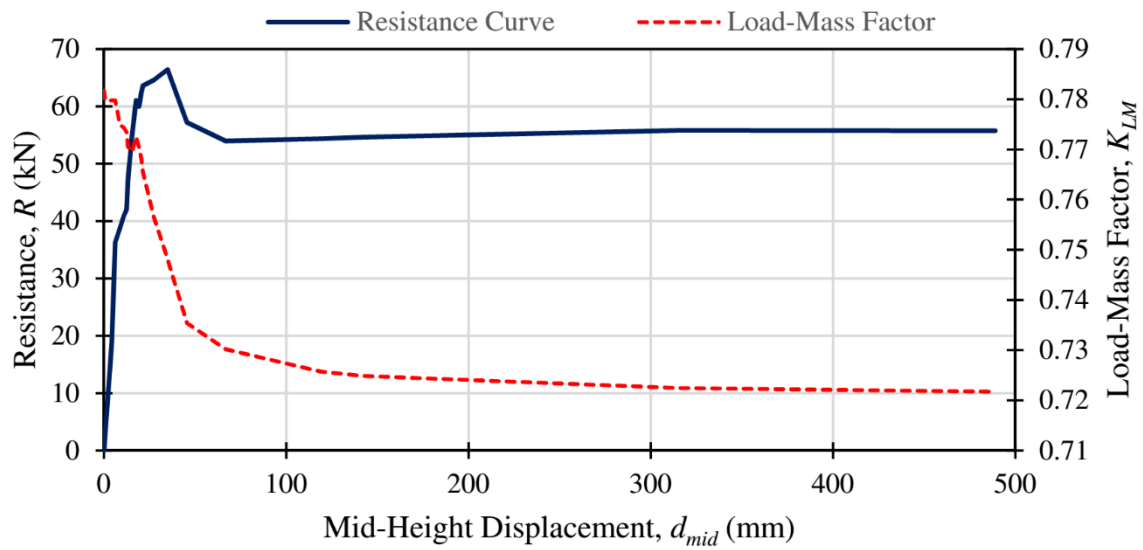


Figure 5-9: Mode Shapes for Four Points of Interest along Composite Resistance Curve for Column 2S-J. (Points of Interest Shown in Figure 5-8)

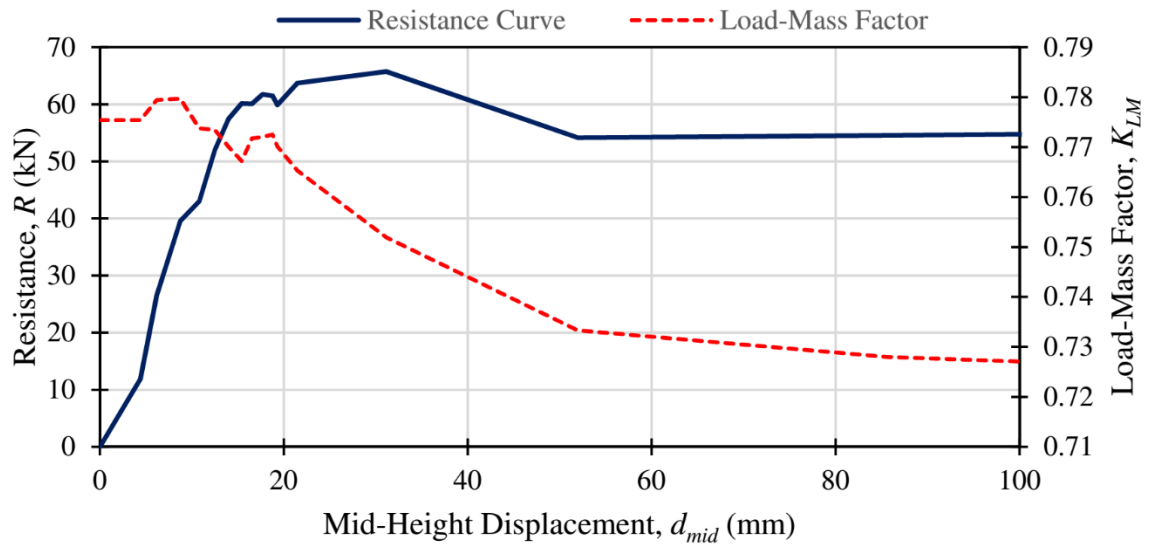


a) Initial Response Region

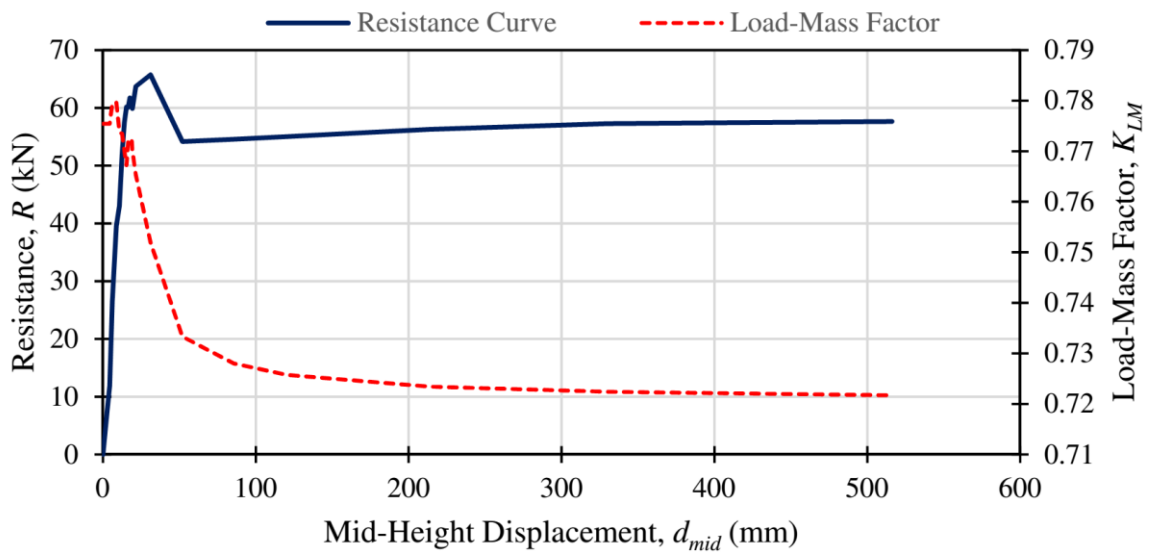


b) Full Curves

Figure 5-10: Column 1NS-C Resistance-Displacement and Load-Mass Factor Curves

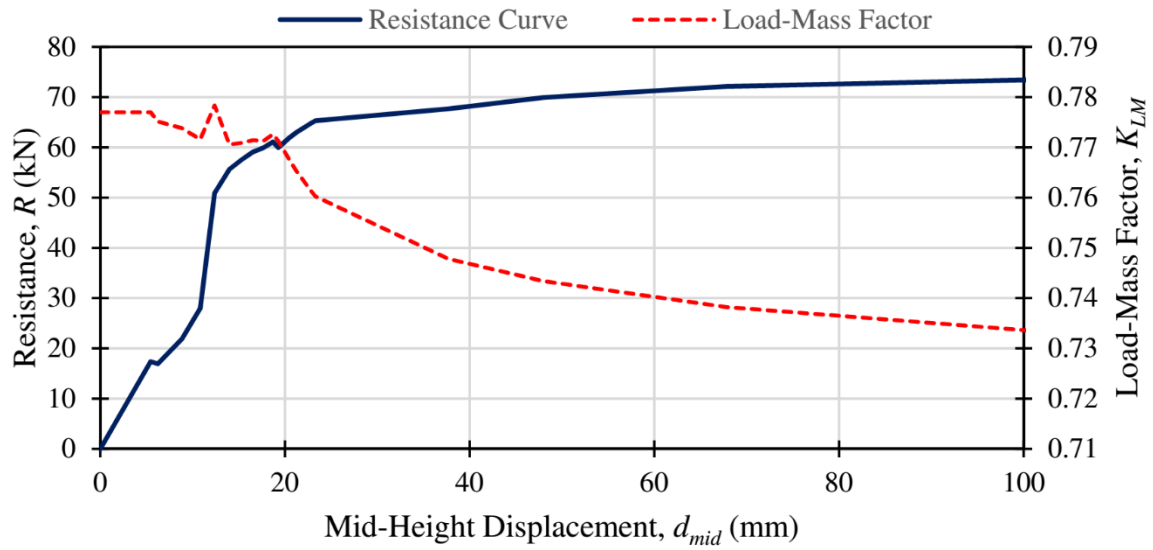


a) Initial Response Region

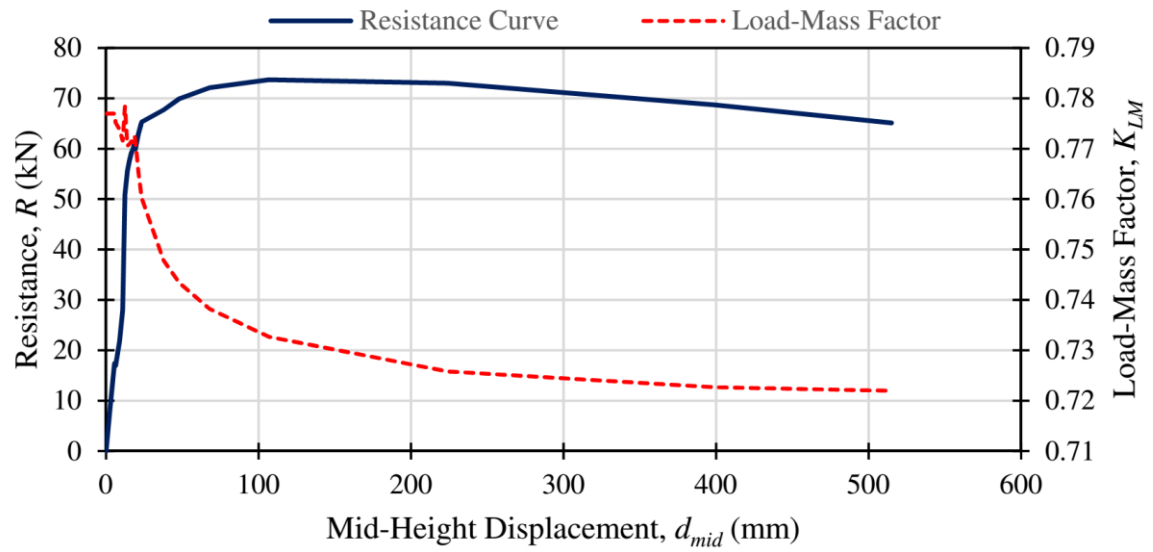


b) Full Curves

Figure 5-11: Column 1S-C Resistance-Displacement and Load-Mass Factor Curves

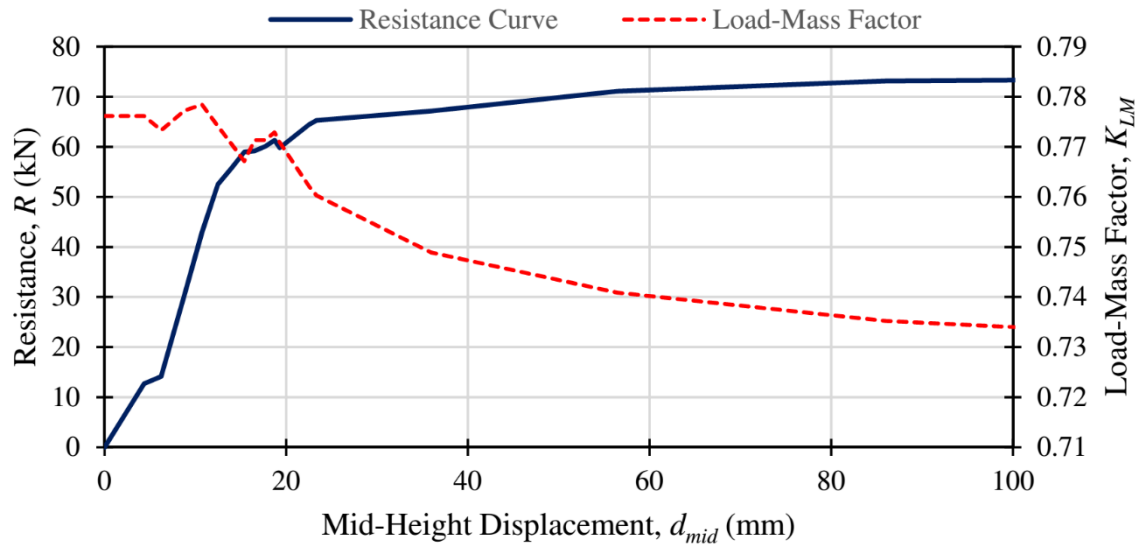


a) Initial Response Region

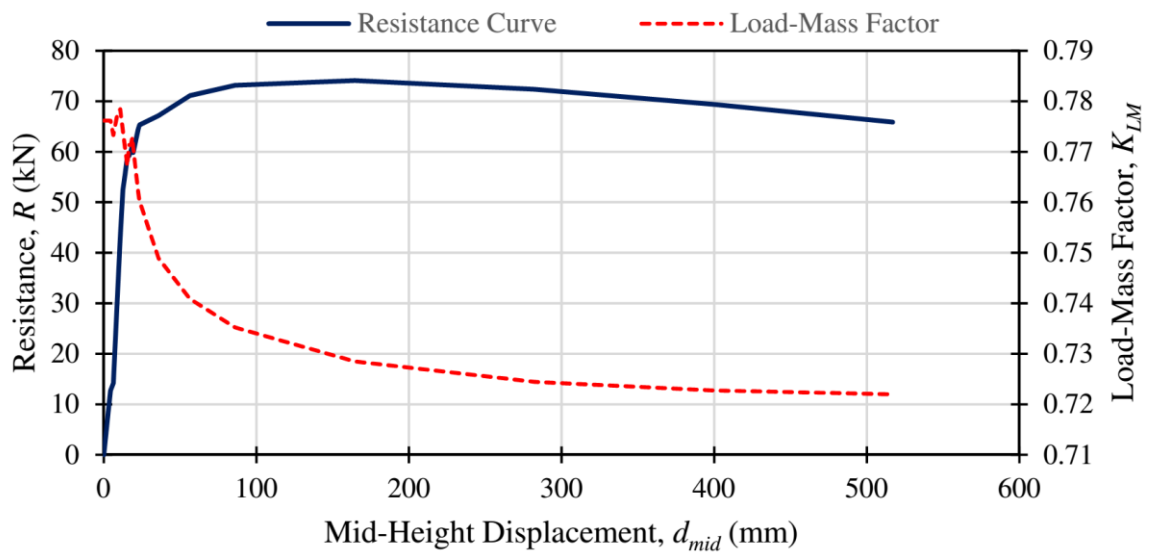


b) Full Curves

Figure 5-12: Column 1NS-J Resistance-Displacement and Load-Mass Factor Curves

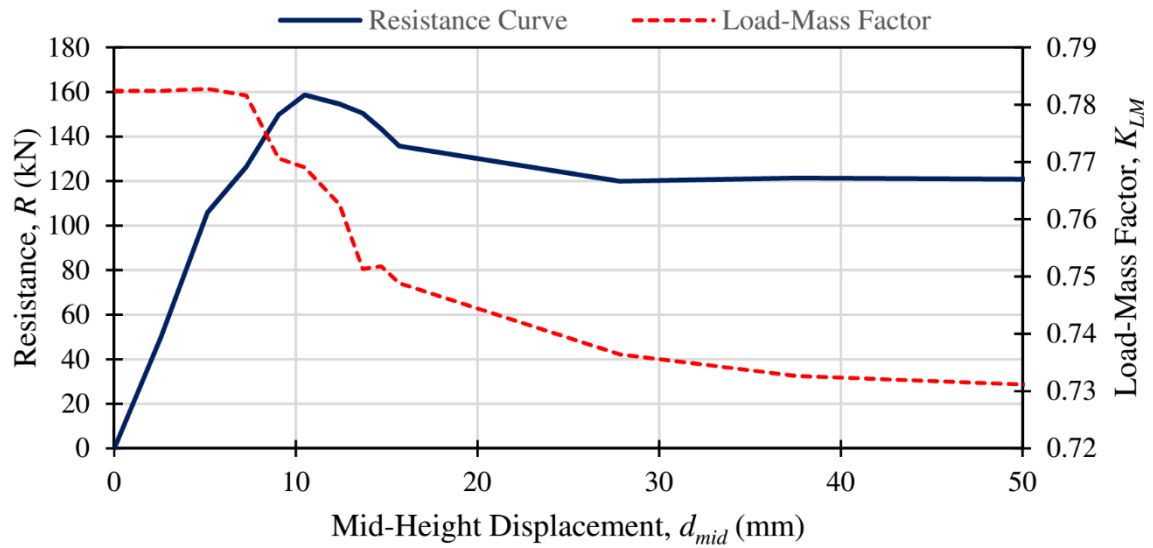


a) Initial Response Region

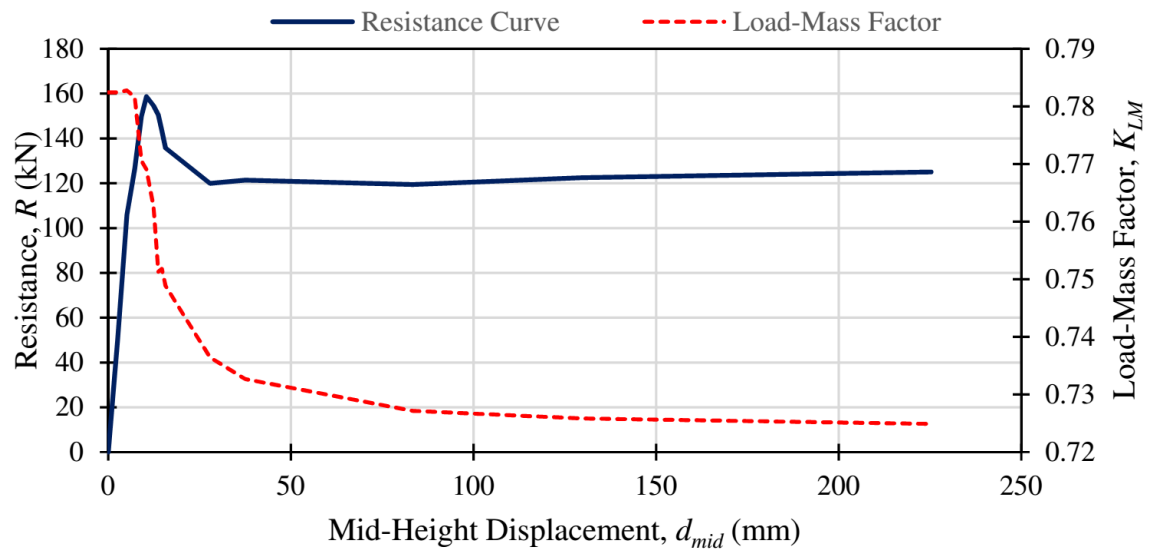


b) Full Curves

Figure 5-13: Column 1S-J Resistance-Displacement and Load-Mass Factor Curves

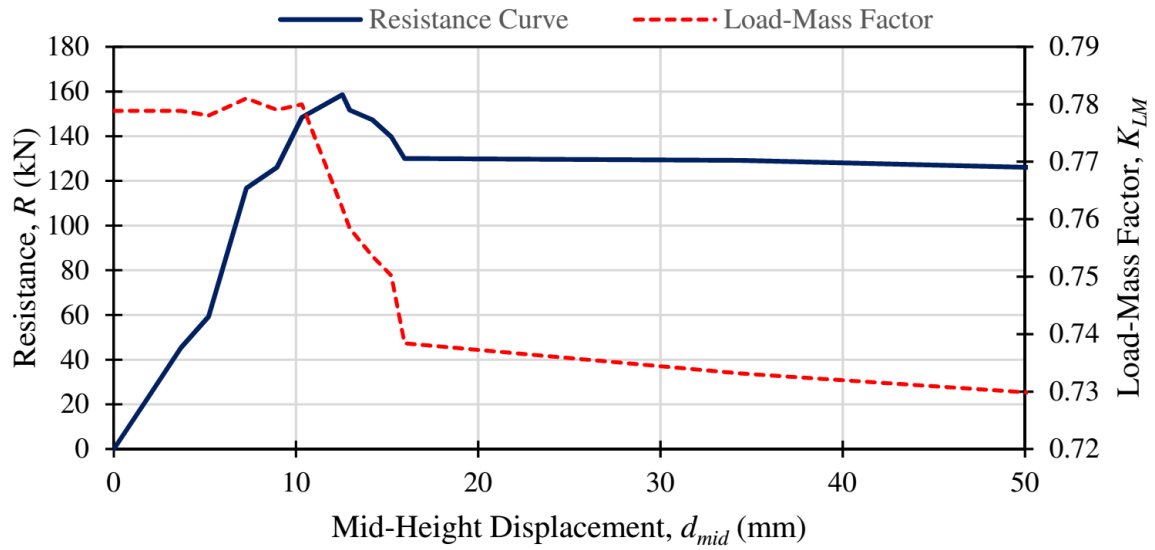


a) Initial Response Region

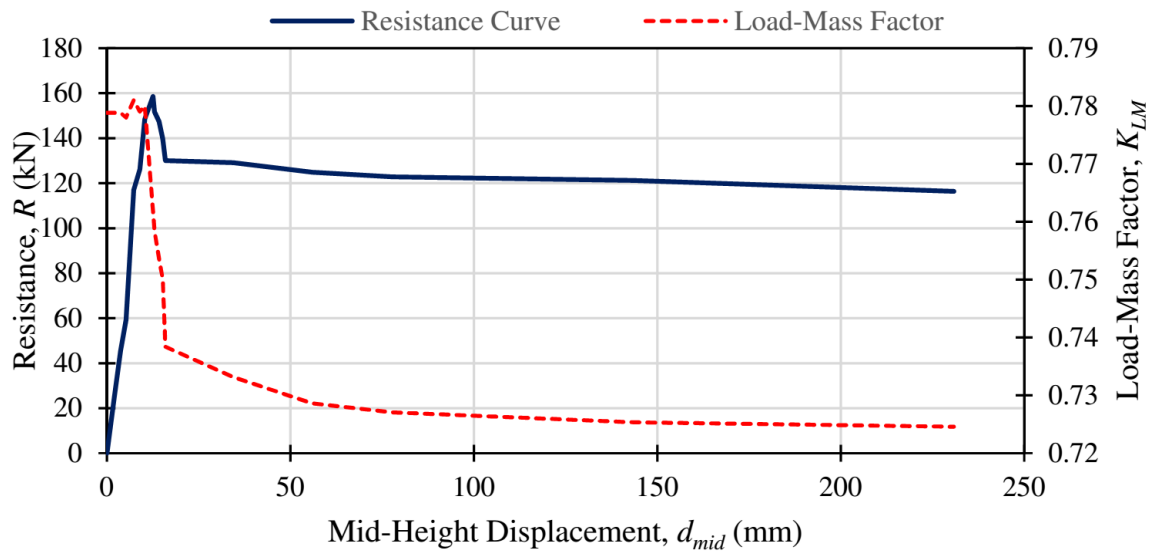


b) Full Curves

Figure 5-14: Column 2NS-C Resistance-Displacement and Load-Mass Factor Curves

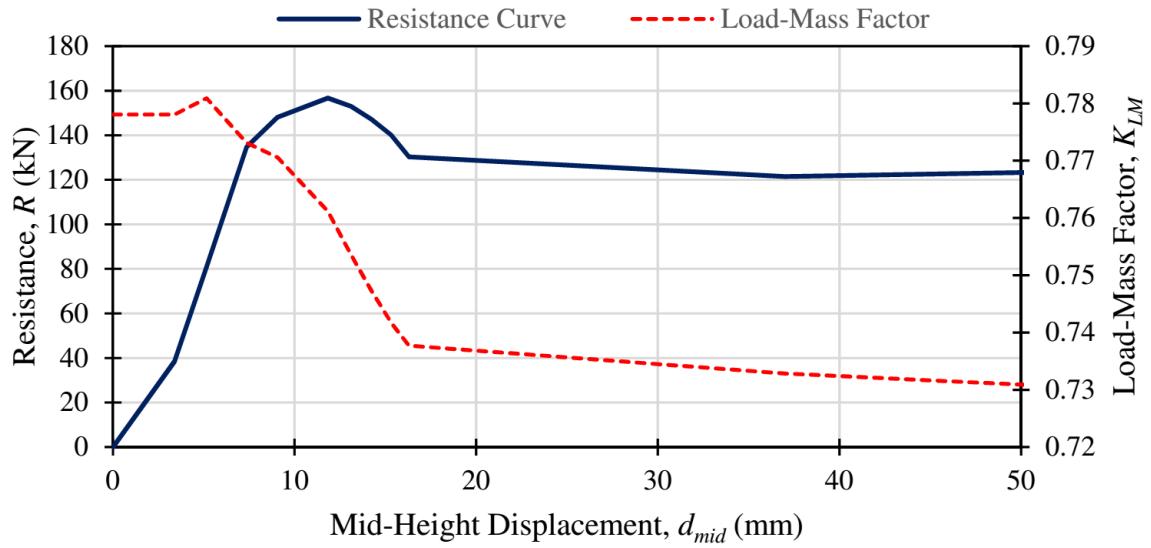


a) Initial Response Region

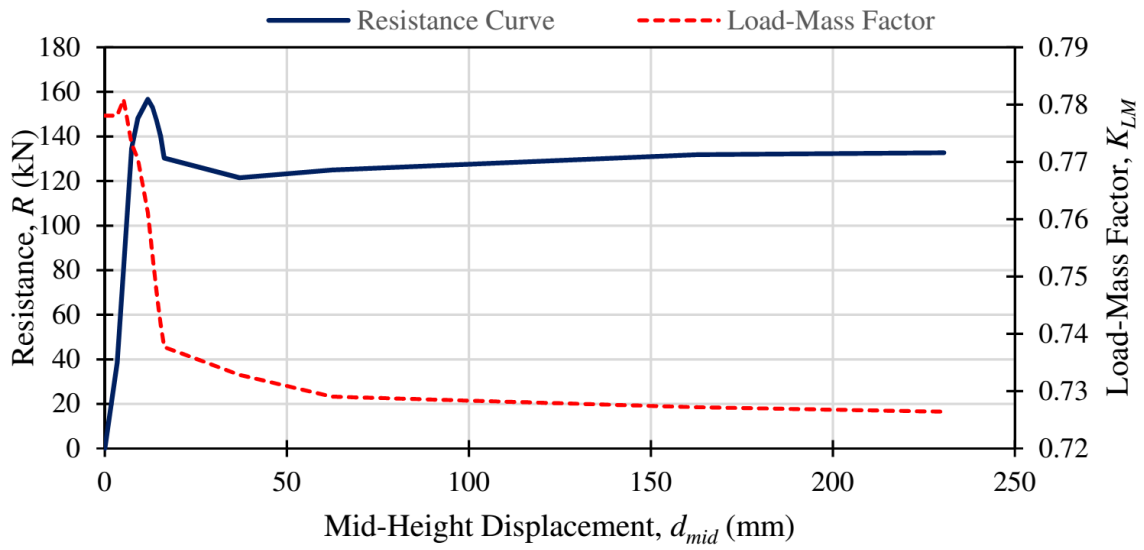


b) Full Curves

Figure 5-15: Column 2S-C Resistance-Displacement and Load-Mass Factor Curves

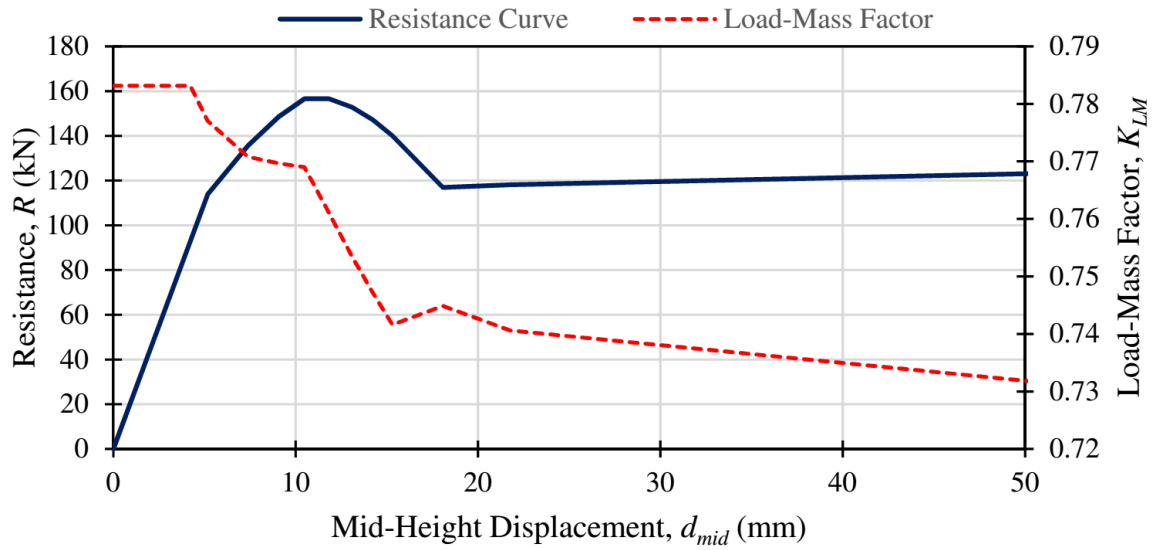


a) Initial Response Region

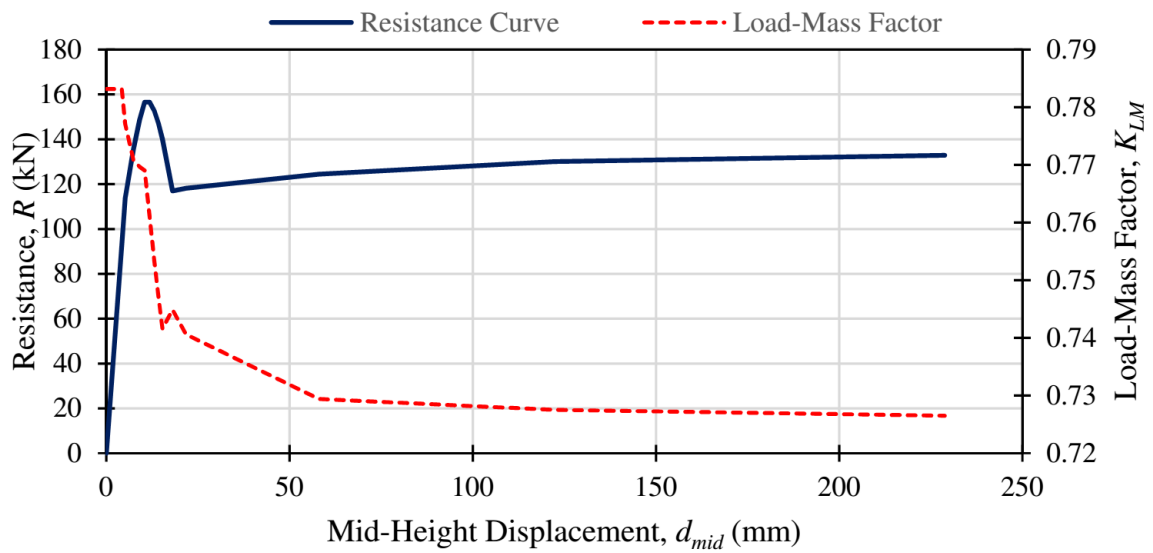


b) Full Curves

Figure 5-16: Column 2NS-J Resistance-Displacement and Load-Mass Factor Curves

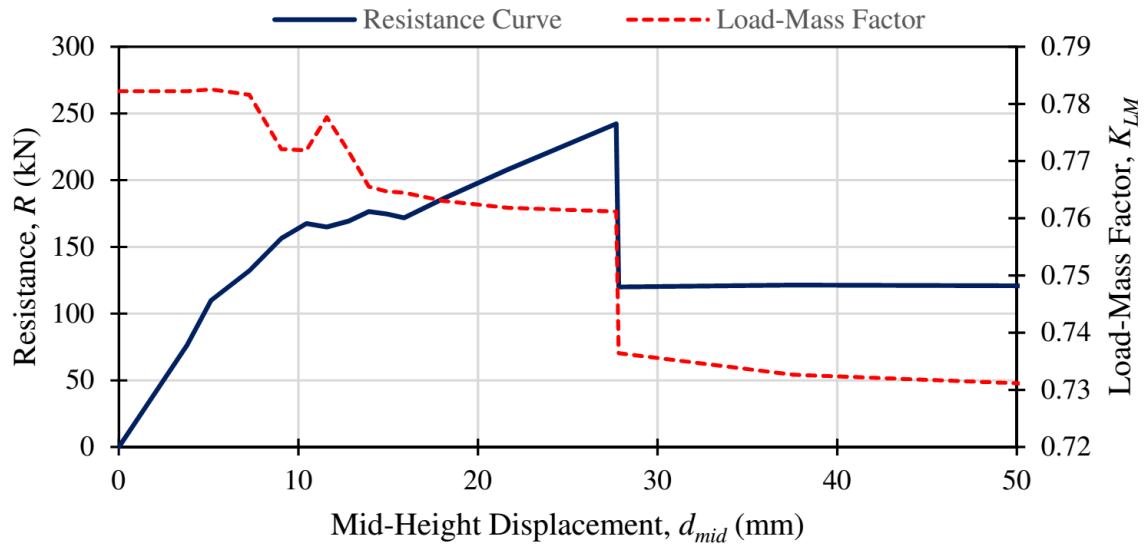


a) Initial Response Region

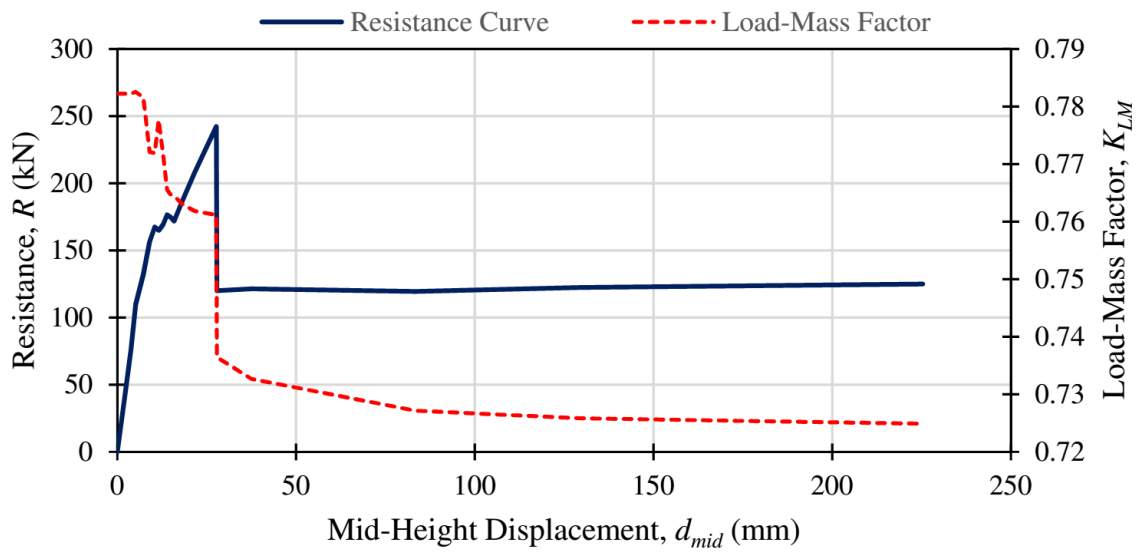


b) Full Curves

Figure 5-17: Column 2S-J Resistance-Displacement and Load-Mass Factor Curves

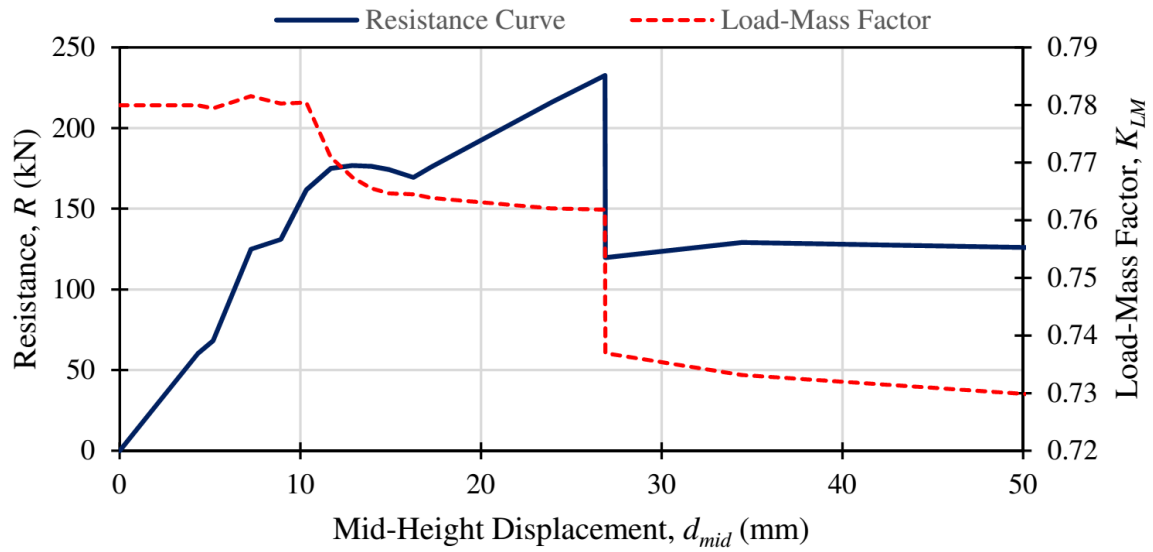


a) Initial Response Region

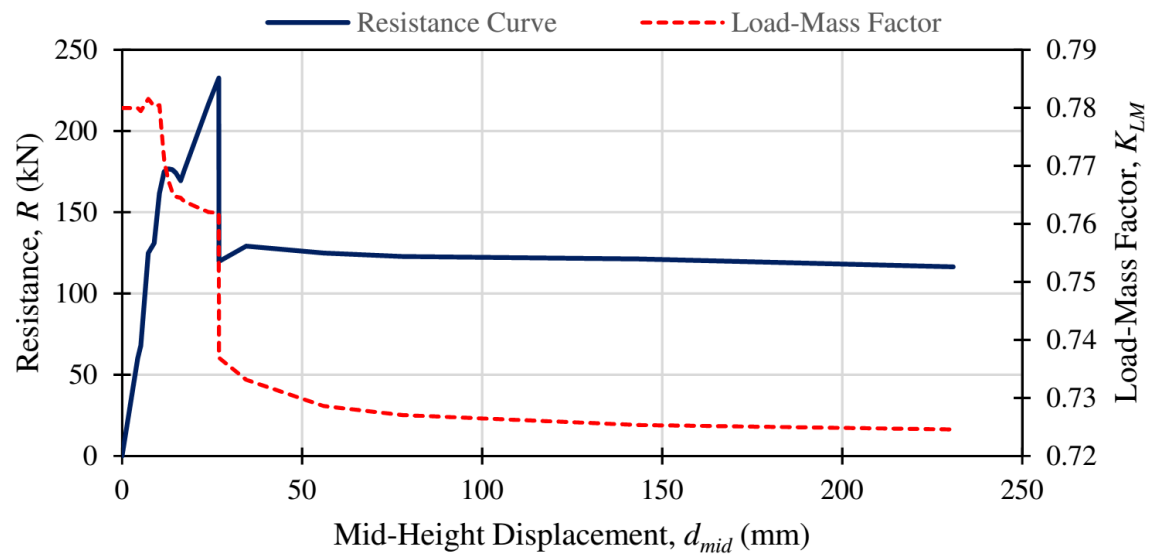


b) Full Curves

Figure 5-18: Column 2NS-R Resistance-Displacement and Load-Mass Factor Curves

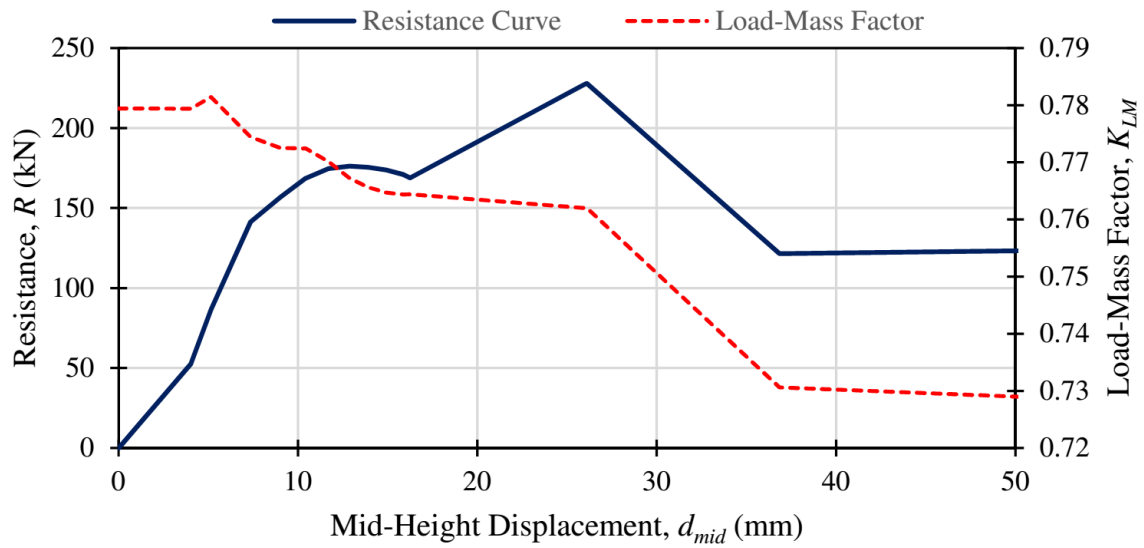


a) Initial Response Region

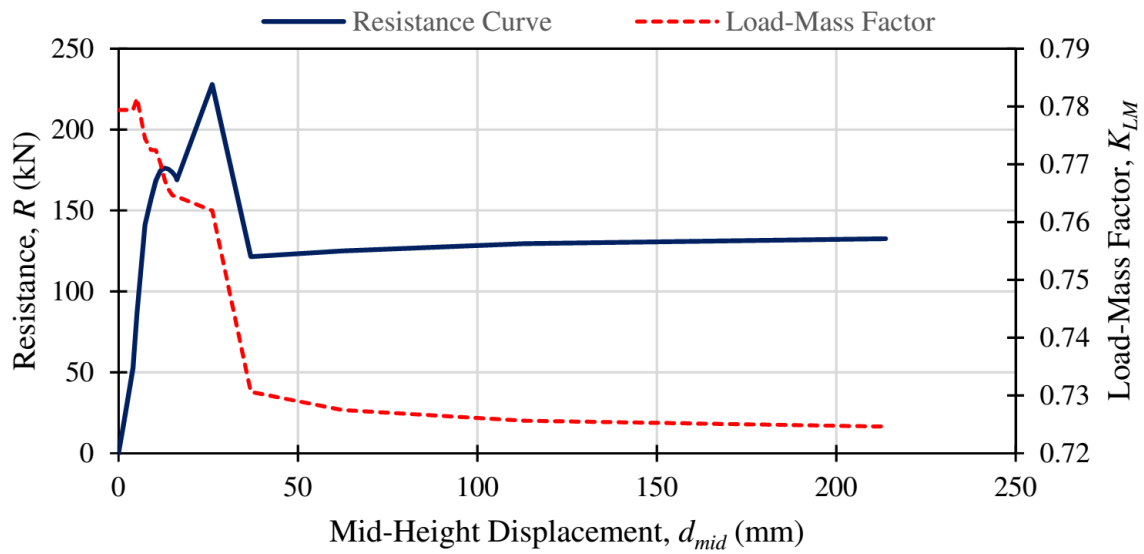


b) Full Curves

Figure 5-19: Column 2S-R Resistance-Displacement and Load-Mass Factor Curves

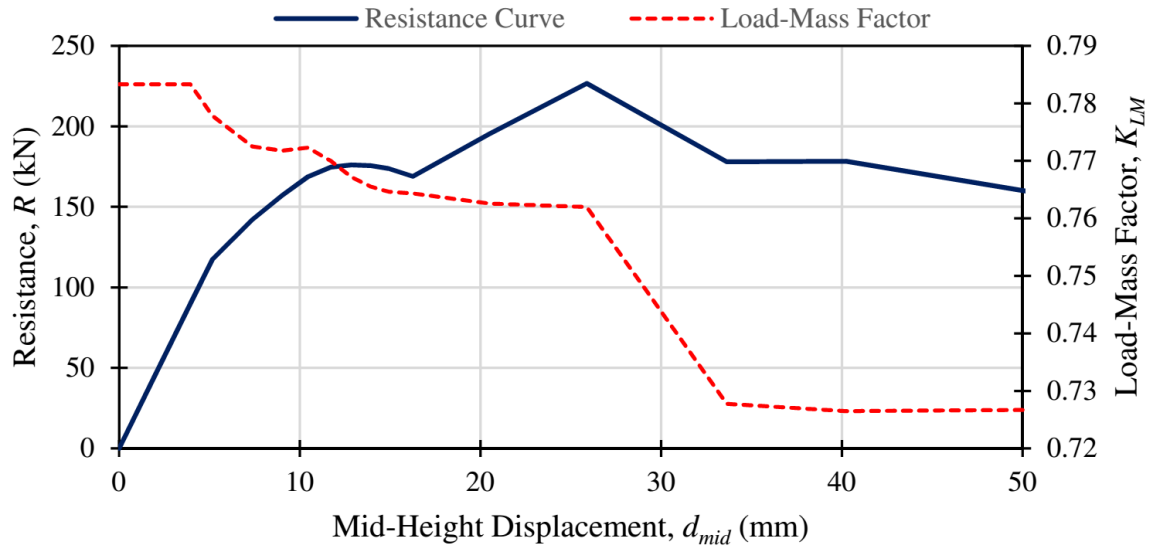


a) Initial Response Region

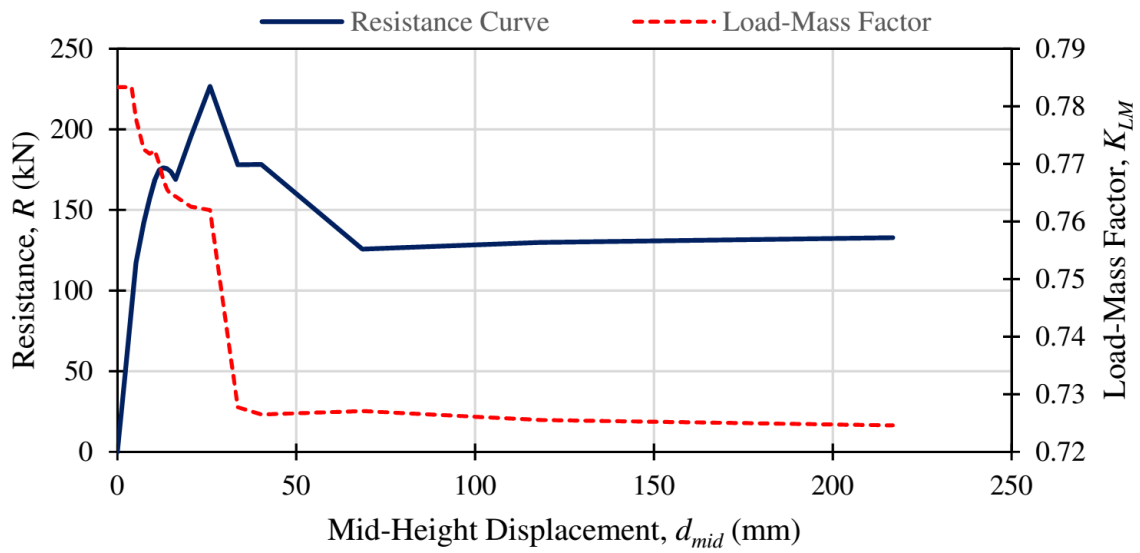


b) Full Curves

Figure 5-20: Column 2NS-JR Resistance-Displacement and Load-Mass Factor Curves

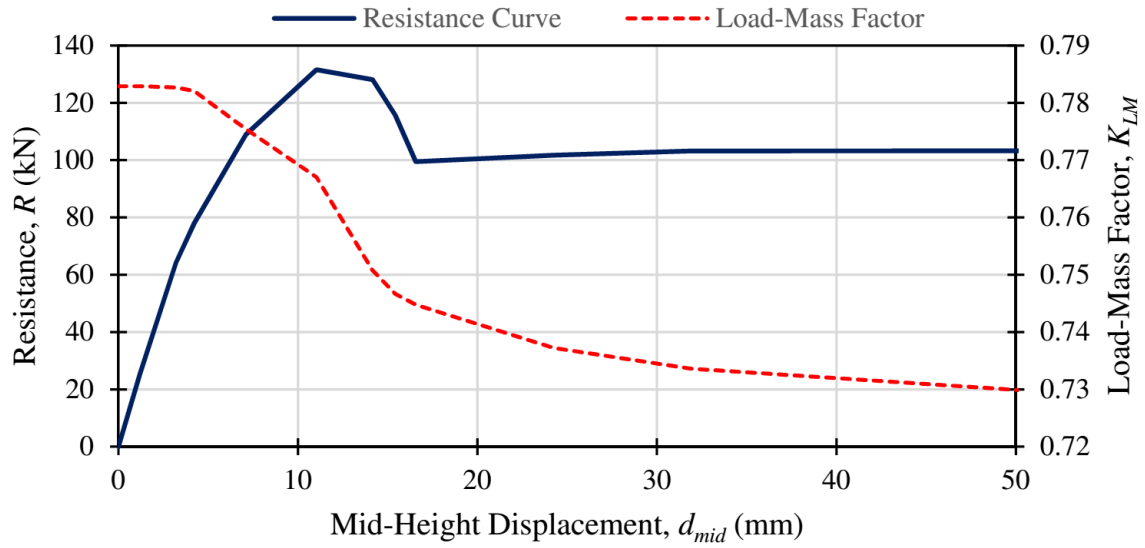


a) Initial Response Region

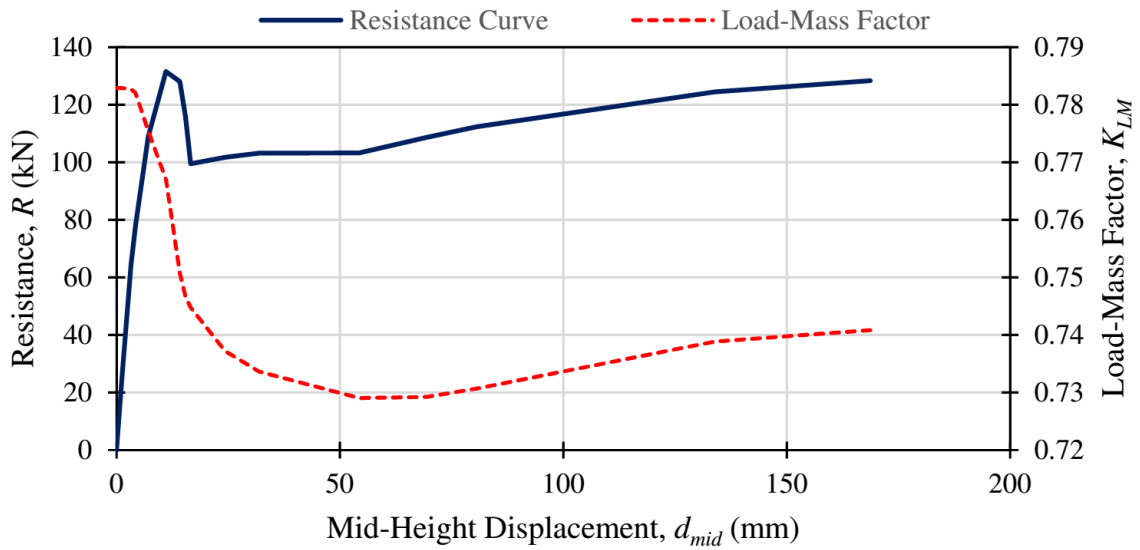


b) Full Curves

Figure 5-21: Column 2S-JR Resistance-Displacement and Load-Mass Factor Curves

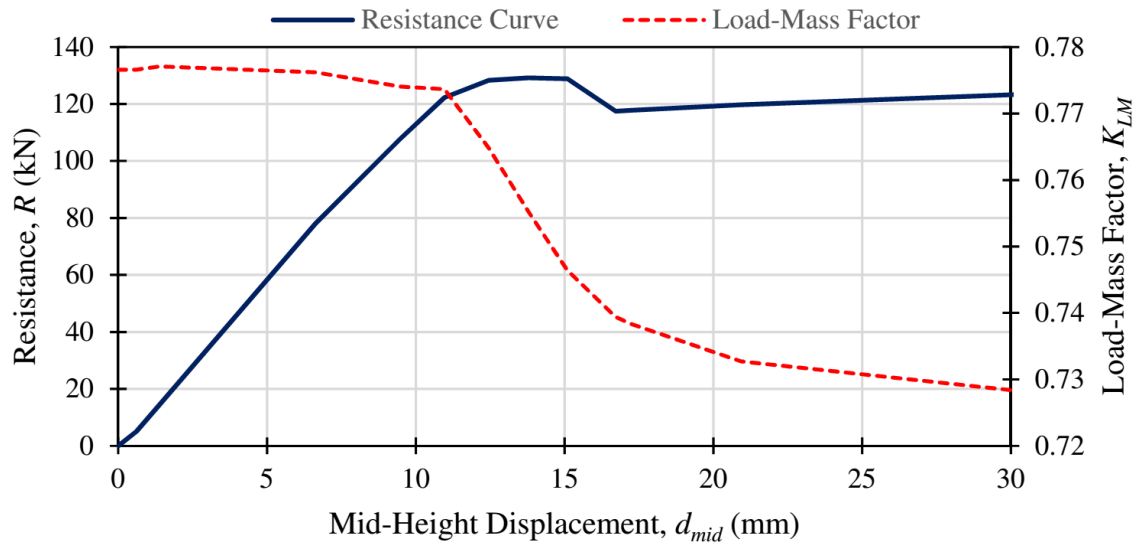


a) Initial Response Region

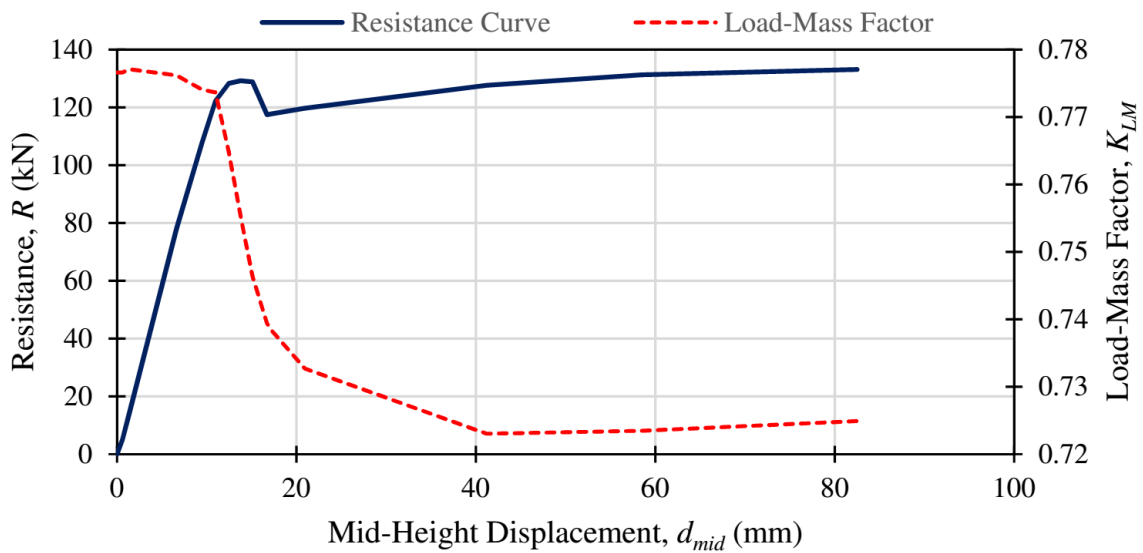


b) Full Curves

Figure 5-22: Column 3NS-C Resistance-Displacement and Load-Mass Factor Curves

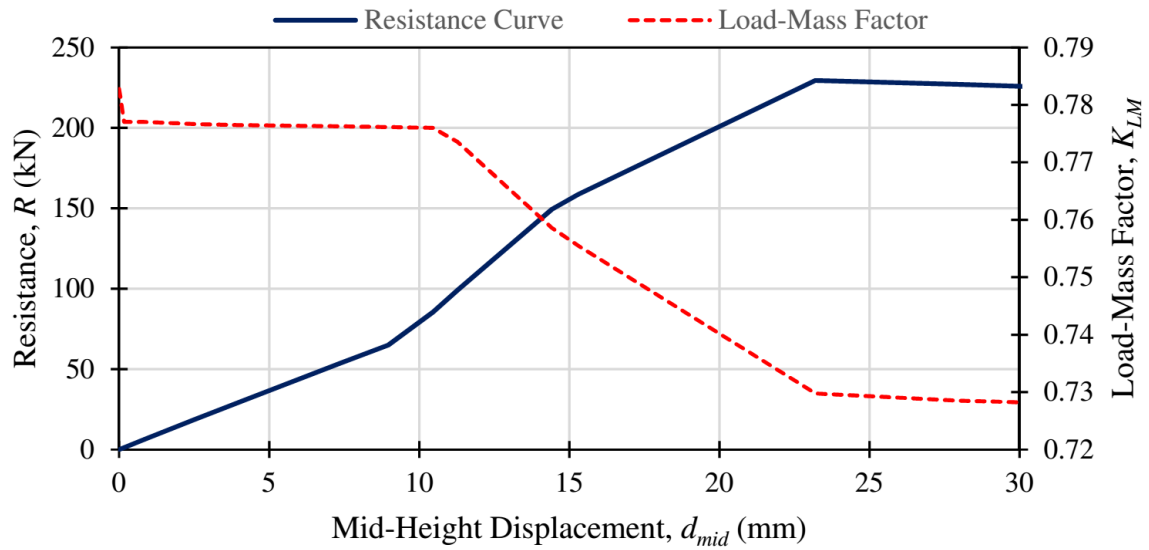


a) Initial Response Region

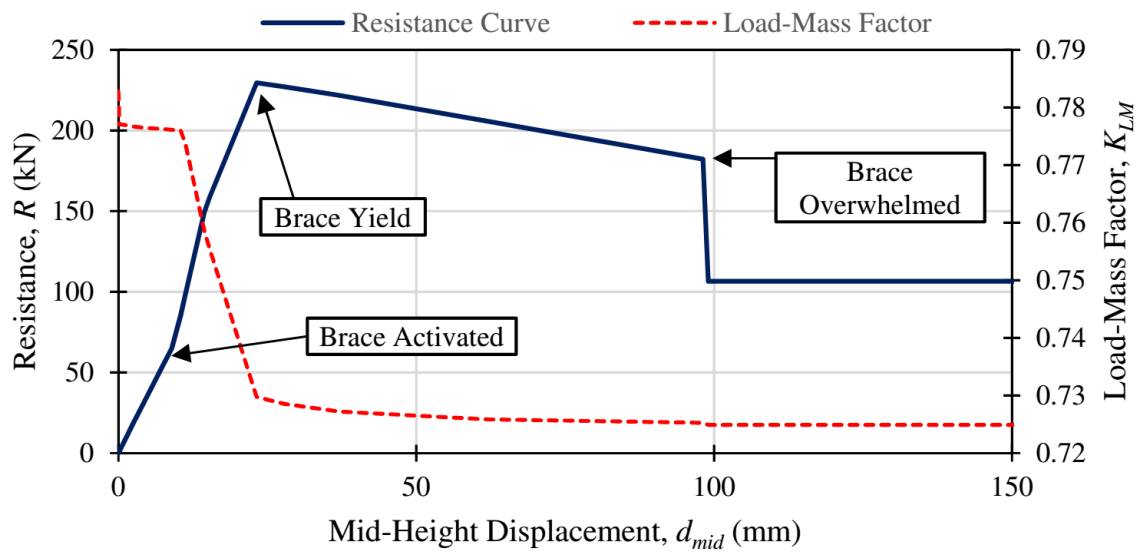


b) Full Curves

Figure 5-23: Column 3NS-W Resistance-Displacement and Load-Mass Factor Curves



a) Initial Response Region



b) Full Curves

Figure 5-24: Column 3NS-CB Resistance-Displacement and Load-Mass Factor Curves

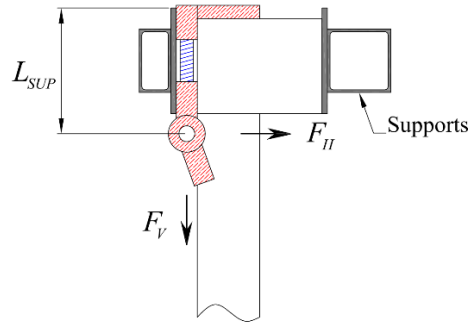
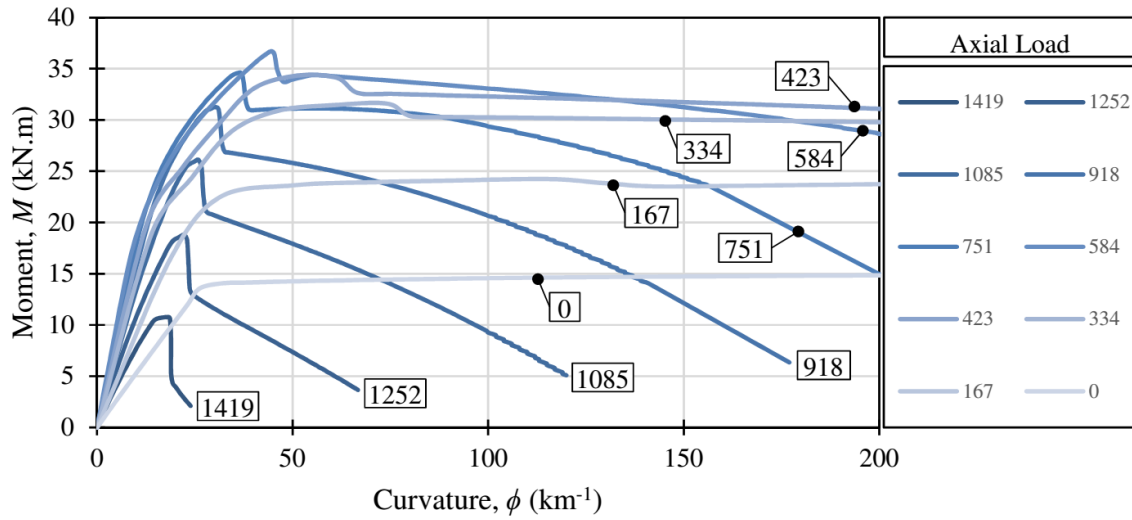
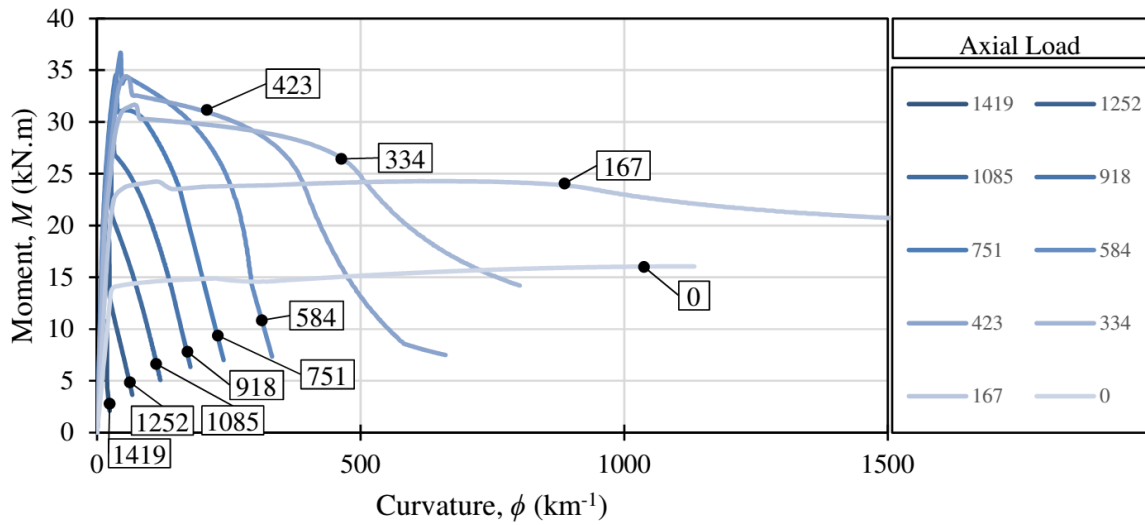


Figure 5-25: Forces and Dimensions Causing M_{BSUP} in Tension Braced Columns



a) Initial (Elastic) Region



b) Full Curve

Figure 5-26: Moment-Curvature Relationship for Columns 3NS-TB1 through 3NS-TB3 under Various Axial Loads

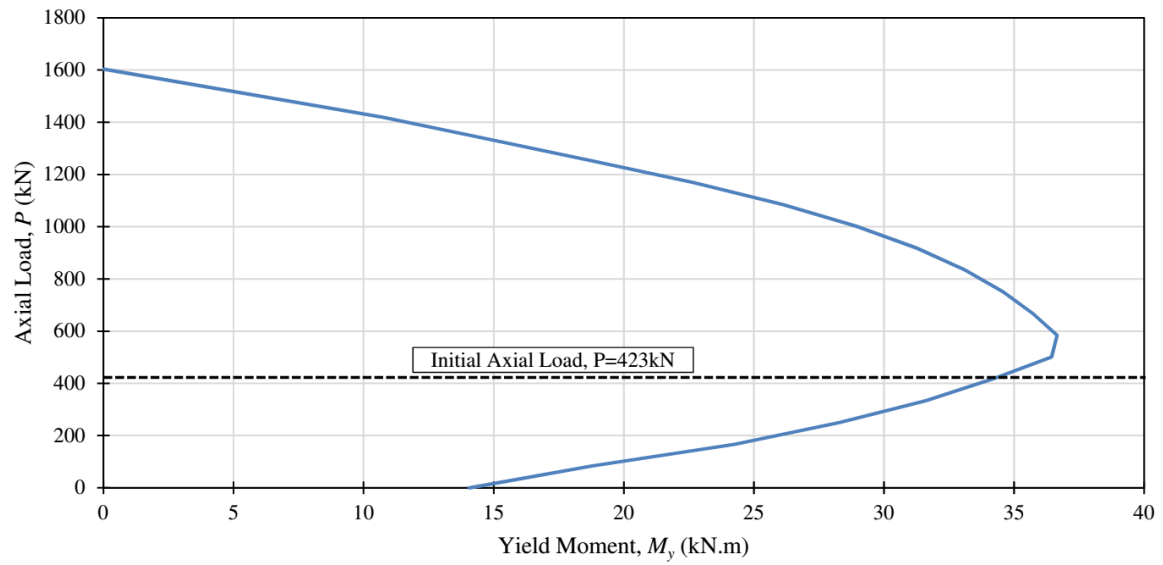


Figure 5-27: Axial Load-Bending Moment Interaction Diagram for Column 3NS-TB1

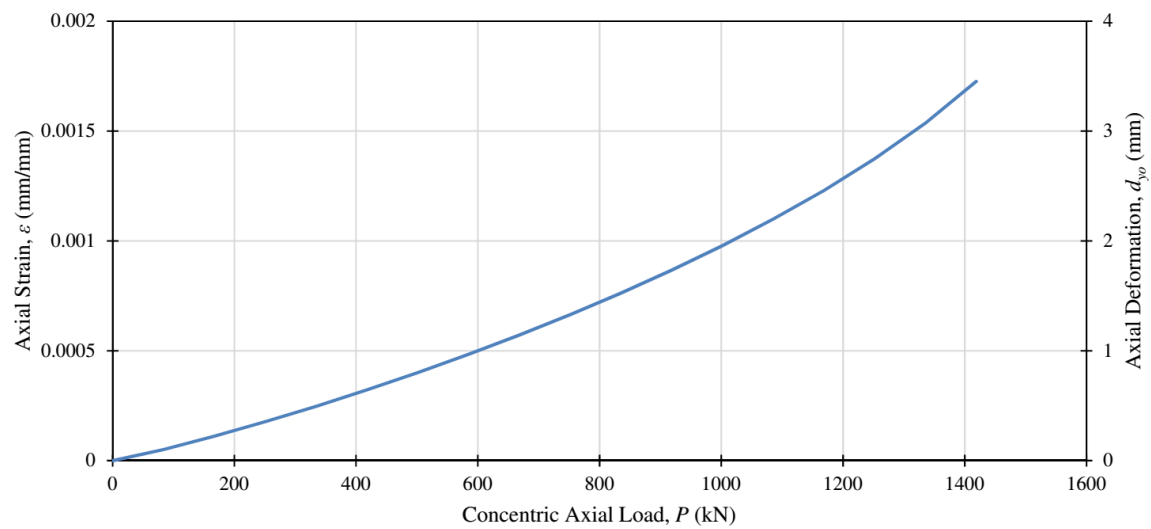
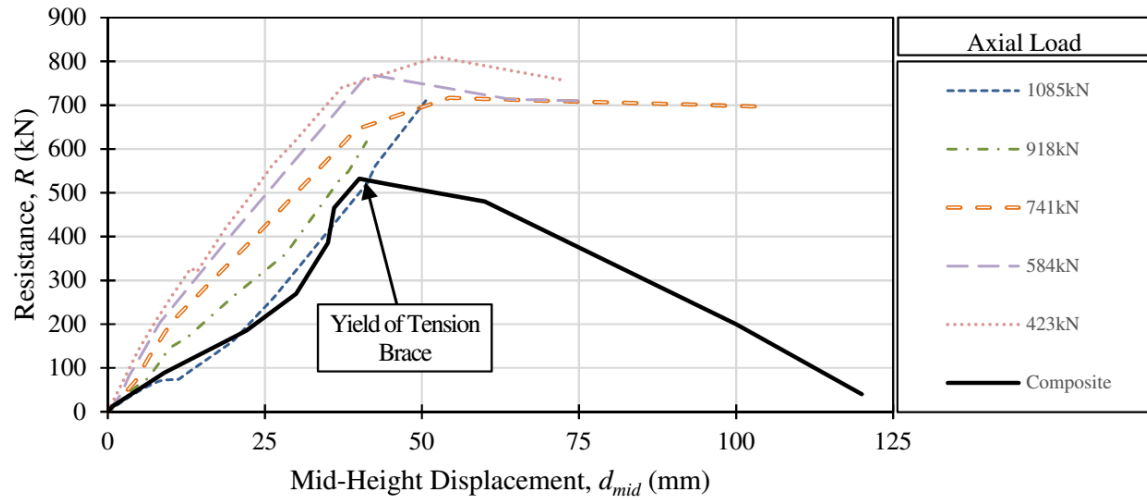
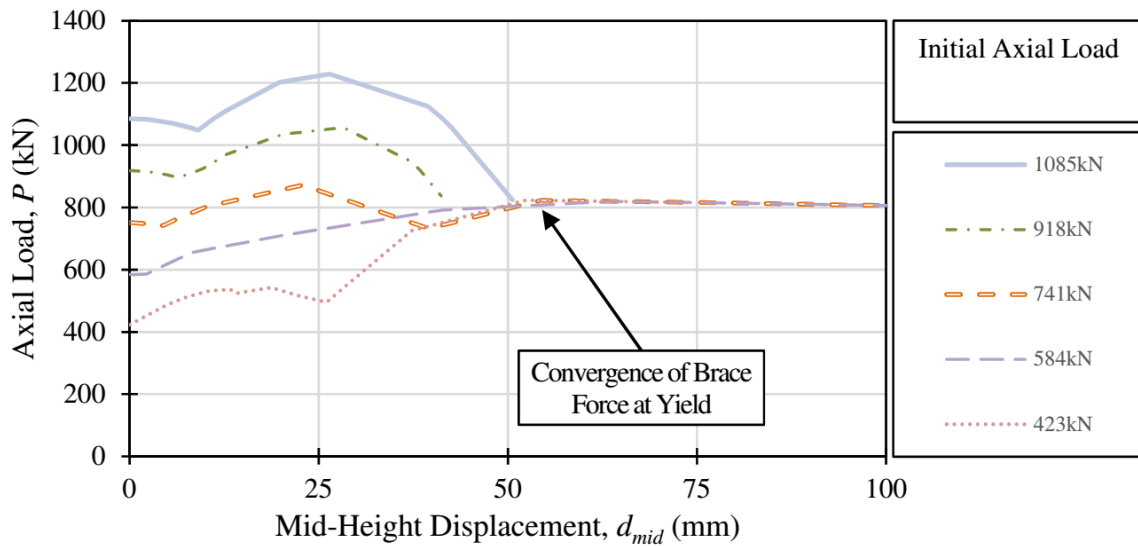


Figure 5-28: Axial Load-Axial Strain and Axial Deformation Relationship for Column 3NS-TB1



a) Resistance-Displacement Curves for Various Axial Loads



b) Axial Load and Mid-Height Displacement Relationships for Various Axial Loads

Figure 5-29: Construction of Composite Resistance Curve for Column 3NS-TB1 using Variation in Axial Loads

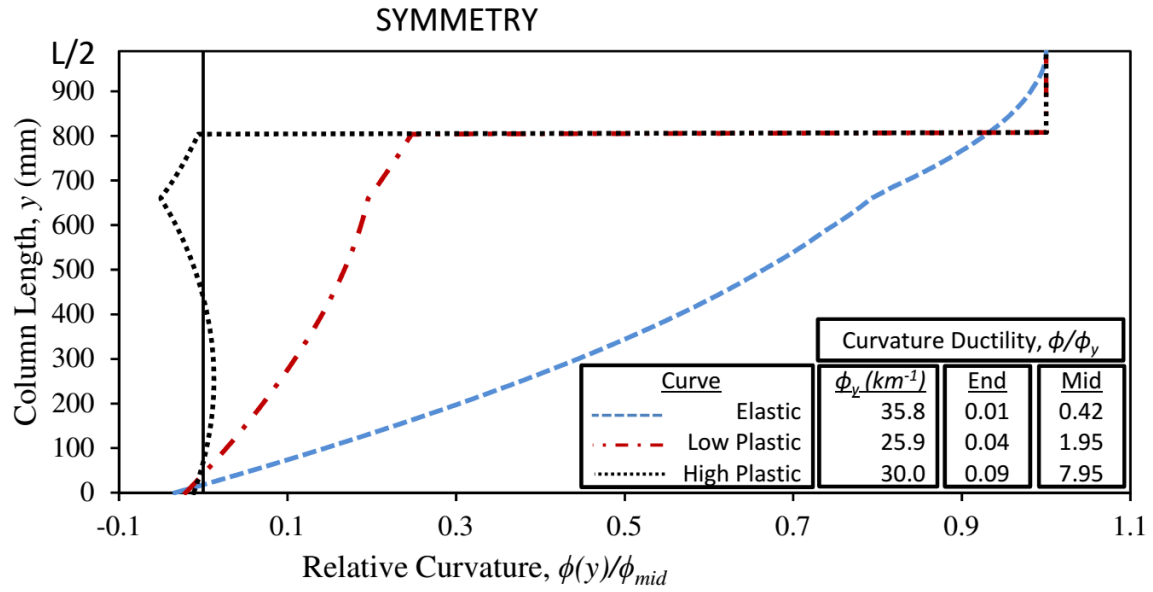


Figure 5-30: Curvature Distribution for Various Ductility Ratios Found During Development of Column 3NS-TB1 Resistance Curve

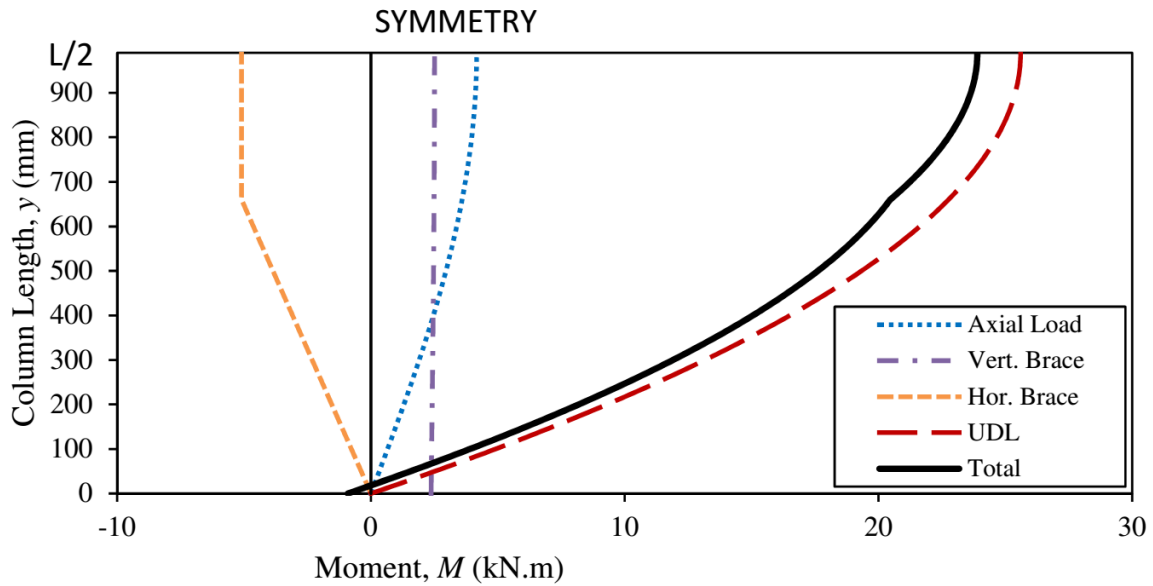


Figure 5-31: Distribution of Moment Components for Elastic Response Found During Development of Column 3NS-TB1 Resistance Curve

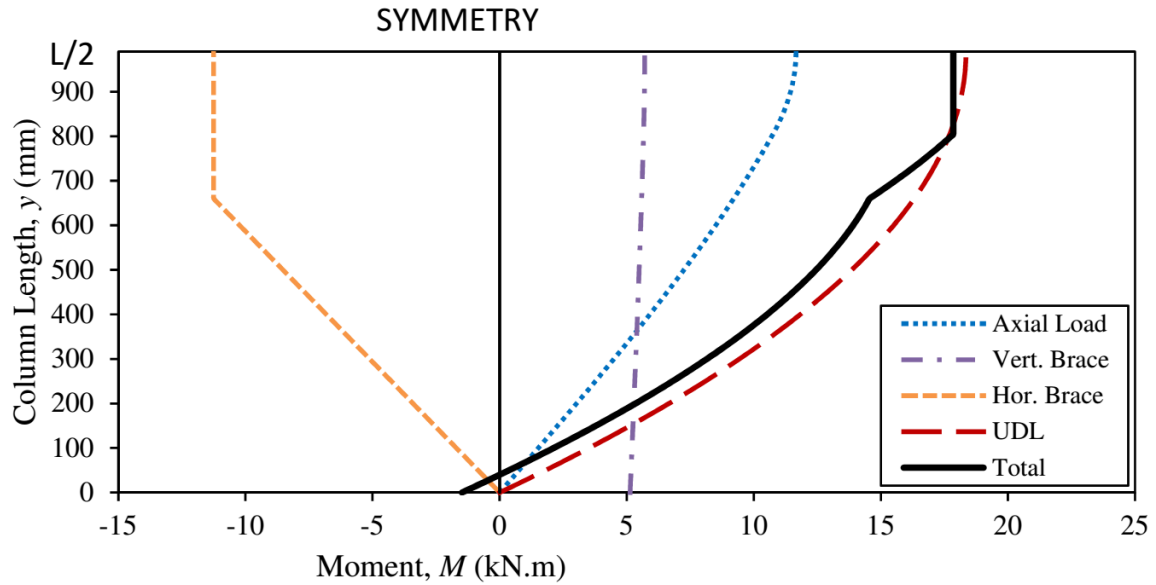


Figure 5-32: Distribution of Moment Components Found During Development of Column 3NS-TB1 Resistance Curve for Early Post Hinging of the Mid-Height

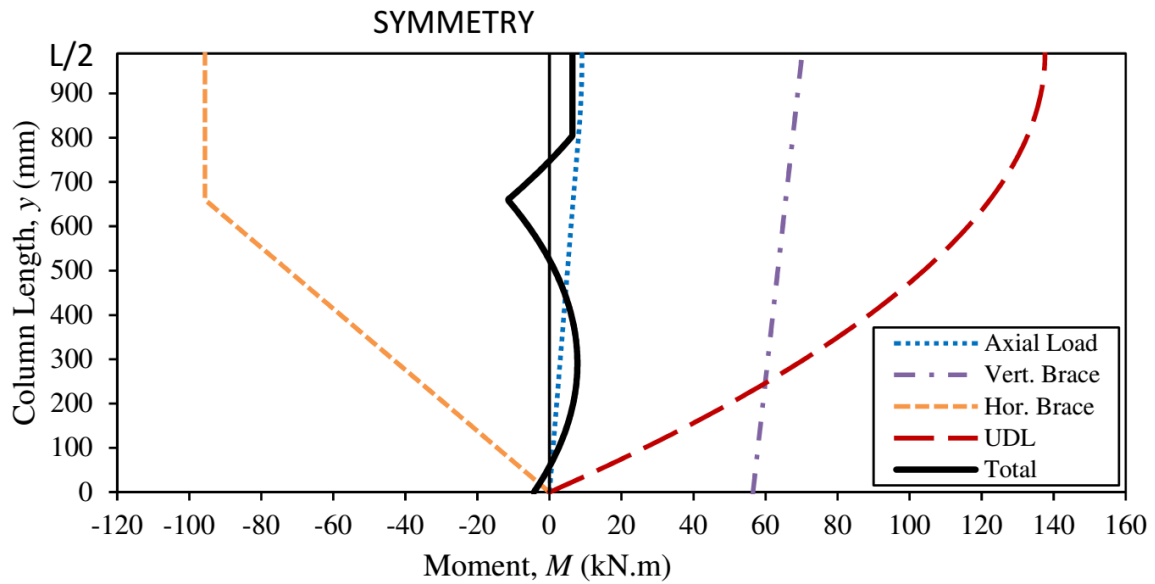


Figure 5-33: Distribution of Moment Components Found During Development of Column 3NS-TB1 Resistance Curve for Highly Plastic Response Region

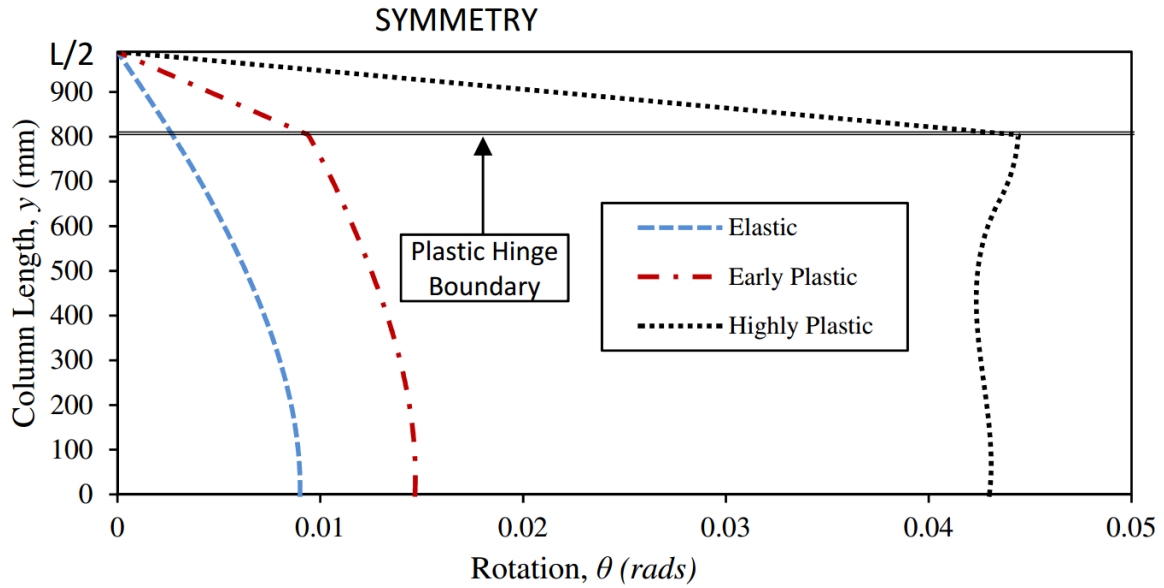


Figure 5-34: Distribution of Rotations Calculated During Development of Column 3NS-TB1 Resistance Curve for Various Response Regions

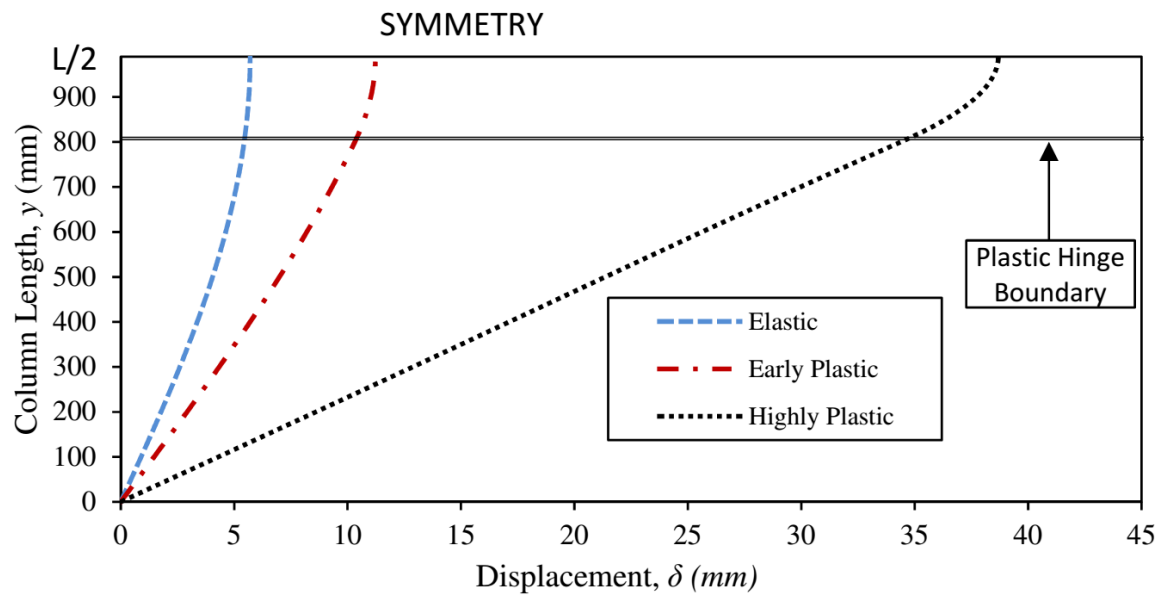


Figure 5-35: Displacements along the Length Observed During Development of Column 3NS-TB1 Resistance Curve for Various Response Regions

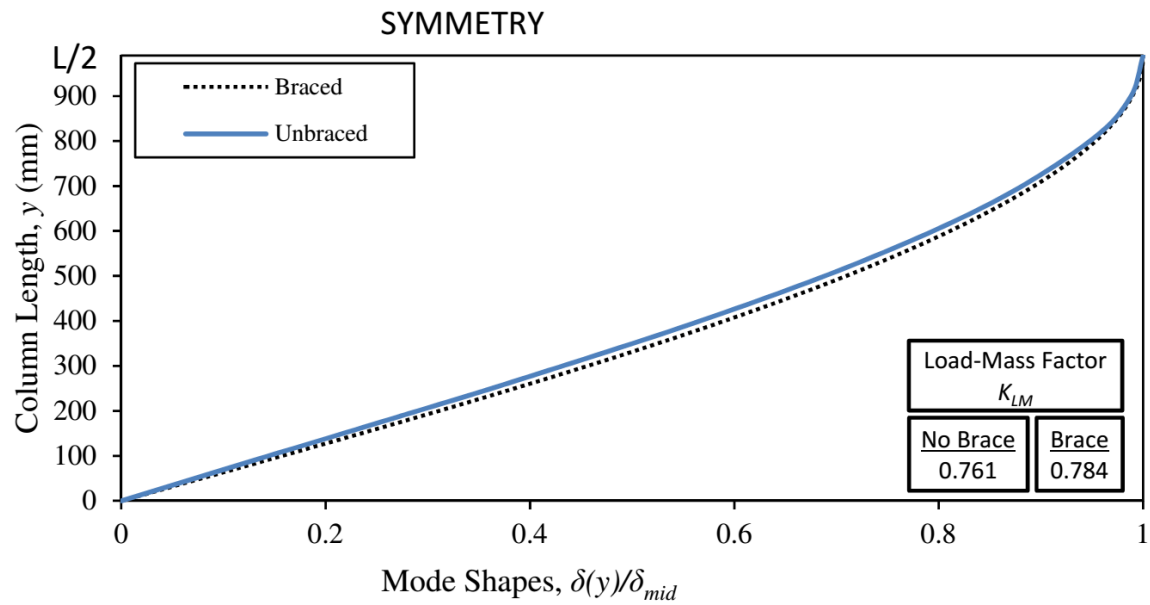
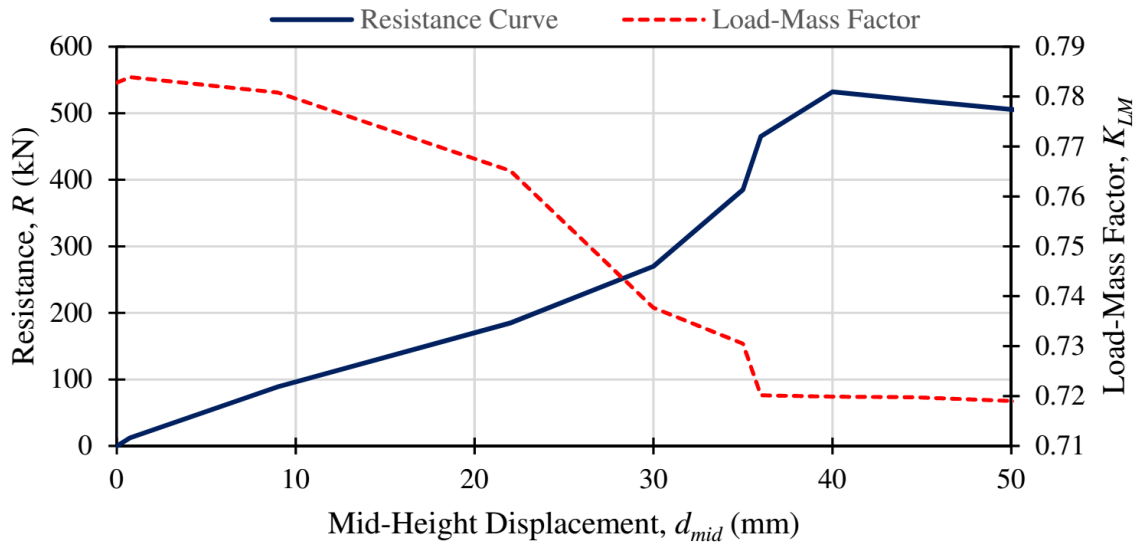
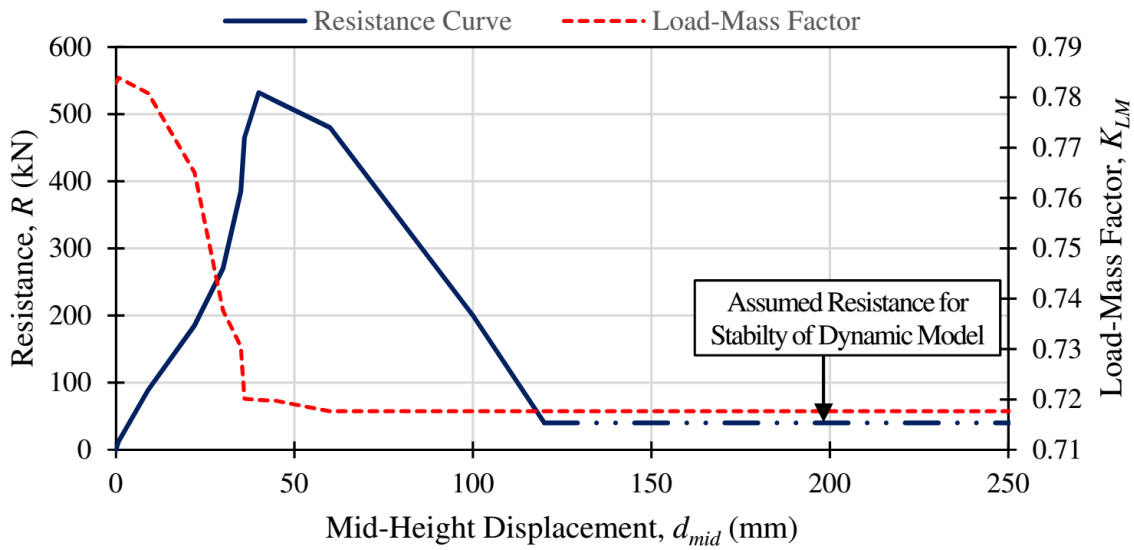


Figure 5-36: Elastic Mode Shape Calculated During Construction of Resistance Curve for Column 3NS-TB1 Compared with Elastic Mode Shape for Column 3NS-C

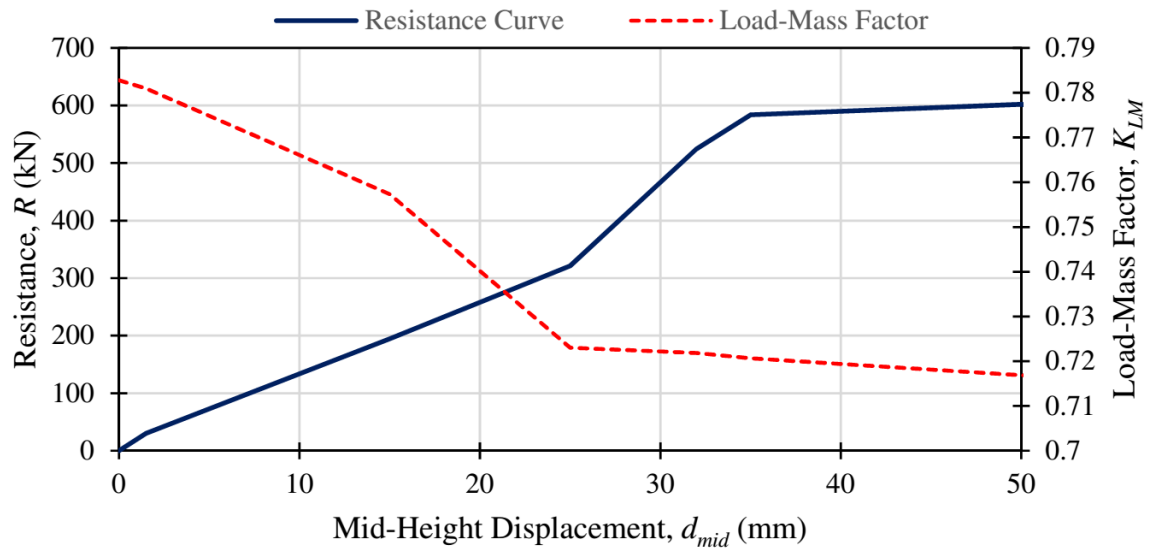


a) Initial Response Region

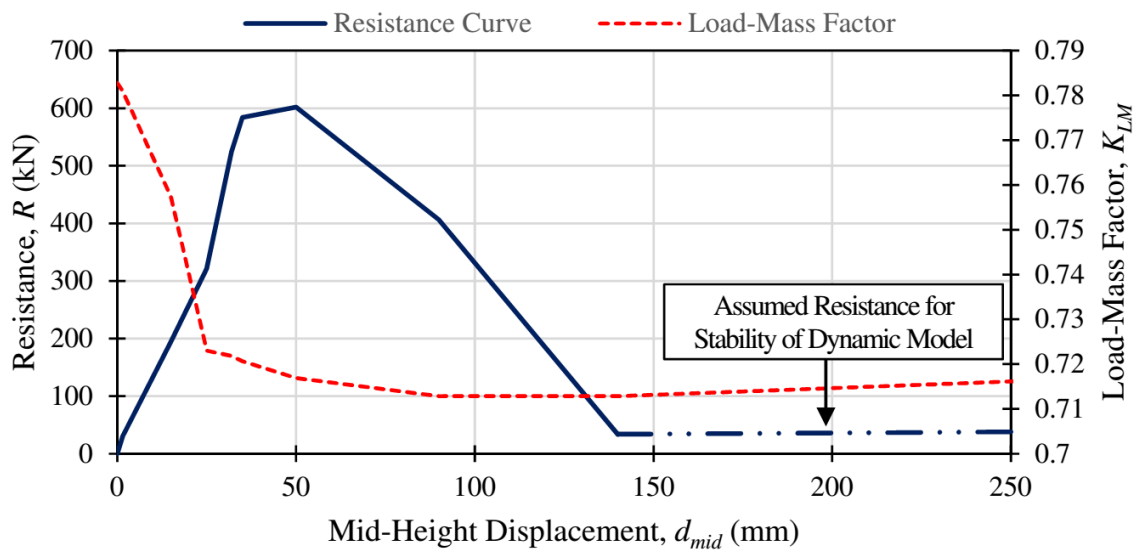


b) Full Curves

Figure 5-37: Column 3NS-TB1 Resistance-Displacement and Load-Mass Factor Curves

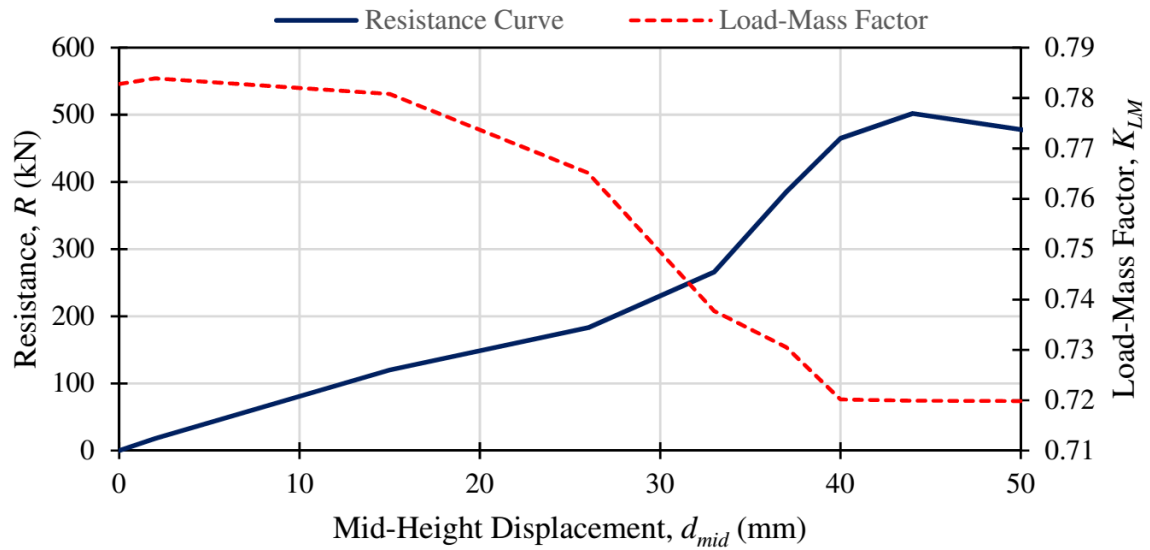


a) Initial Response Region

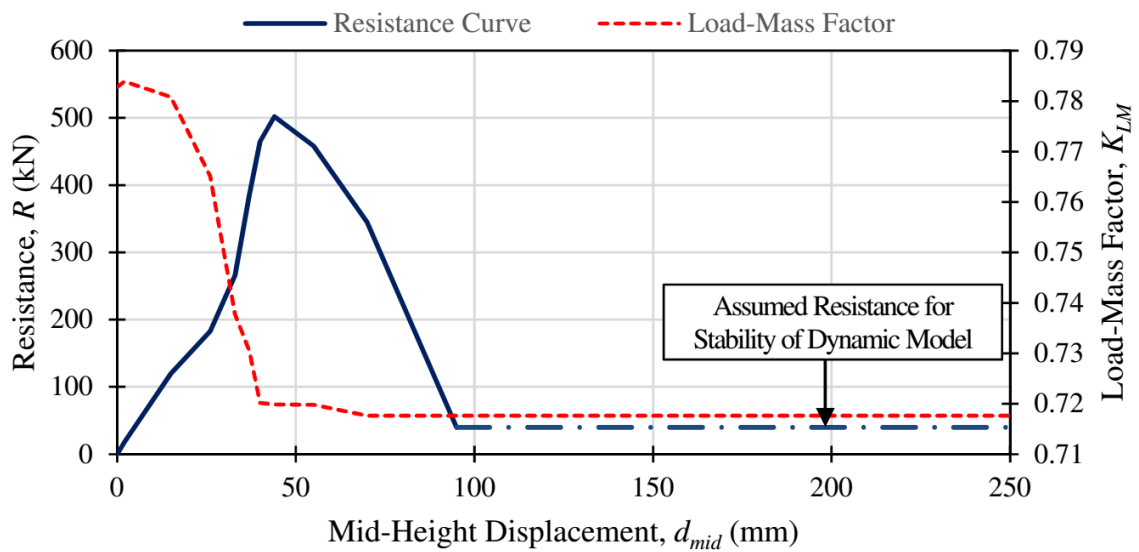


b) Full Curves

Figure 5-38: Column 3NS-TB2 Resistance-Displacement and Load-Mass Factor Curves



a) Initial Response Region



b) Full Curves

Figure 5-39: Column 3NS-TB3 Resistance-Displacement and Load-Mass Factor Curves

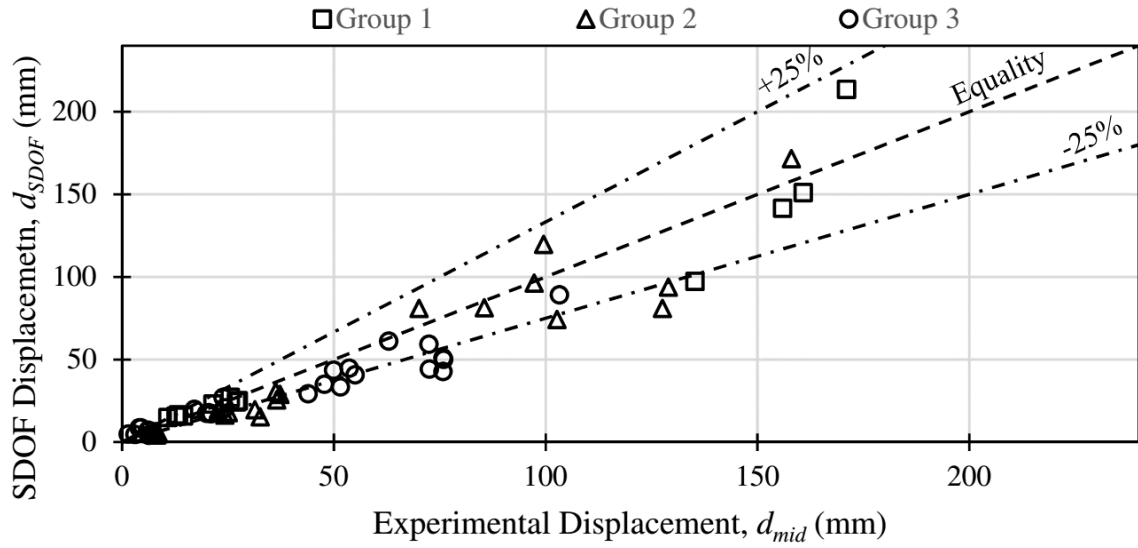


Figure 5-40: Maximum Mid-Height Displacements Compared with SDOF Predicted Maximum Displacements for All Tests

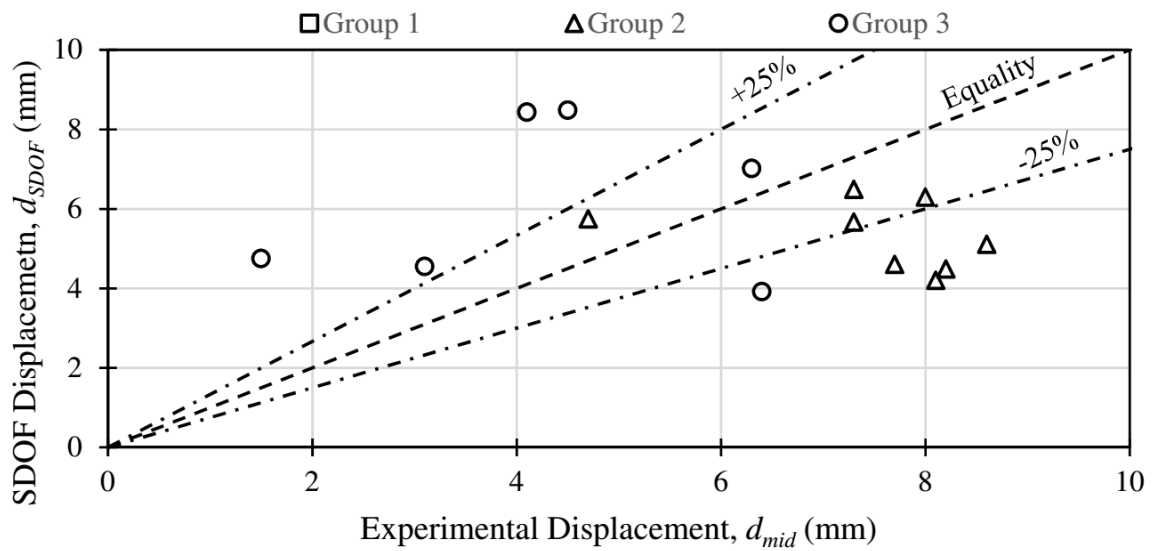


Figure 5-41: Maximum Mid-Height Displacements Compared with SDOF Predicted Maximum Displacements for All Tests – Values below 10mm

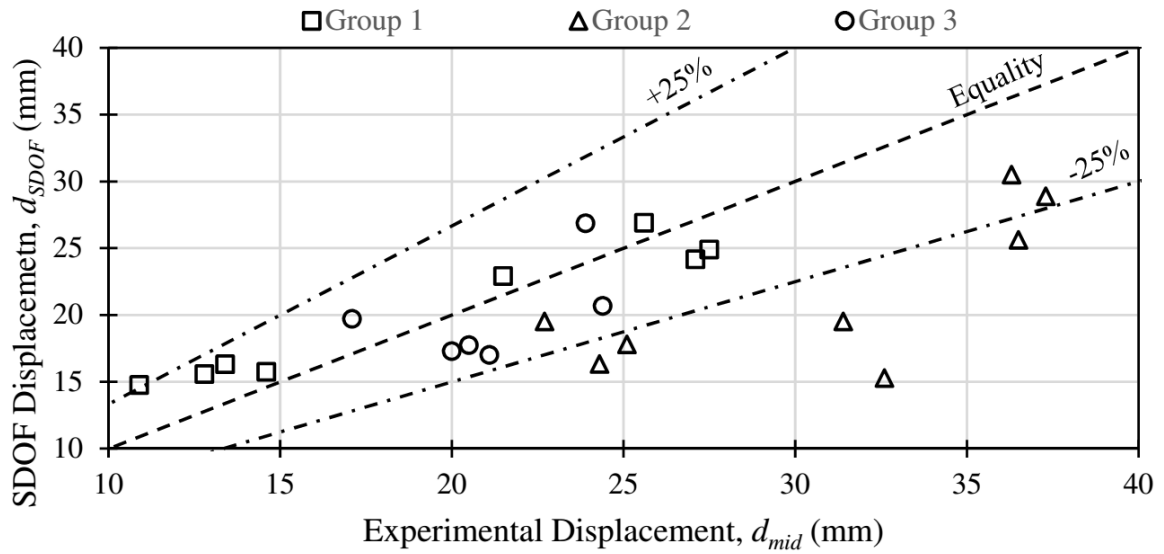


Figure 5-42: Maximum Mid-Height Displacements Compared with SDOF Predicted Maximum Displacements for All Tests – Values Between 10mm and 40mm

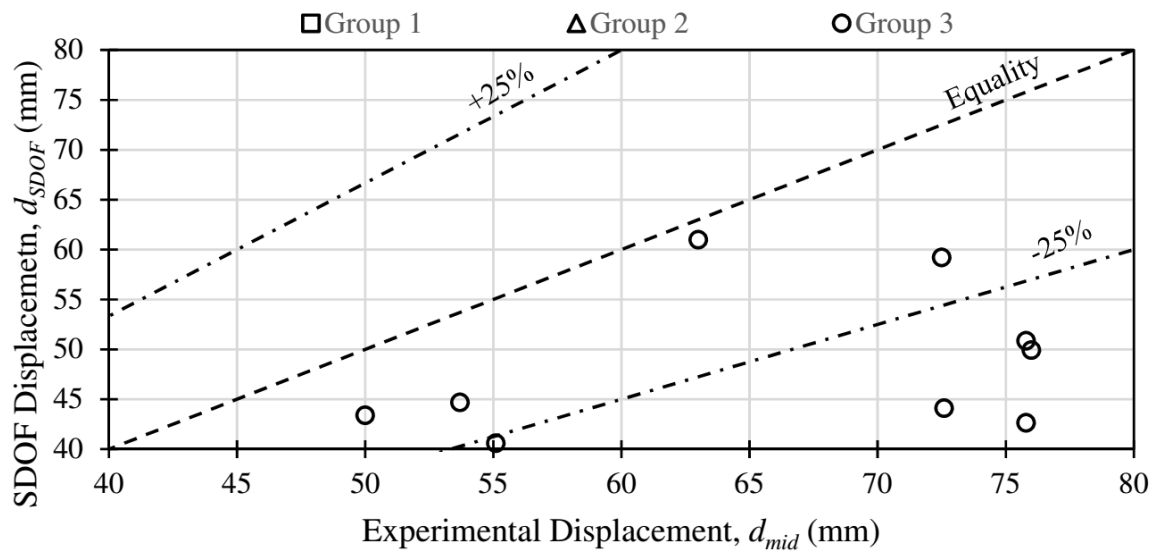


Figure 5-43: Maximum Mid-Height Displacements Compared with SDOF Predicted Maximum Displacements for All Tests – Values Between 40mm and 80mm

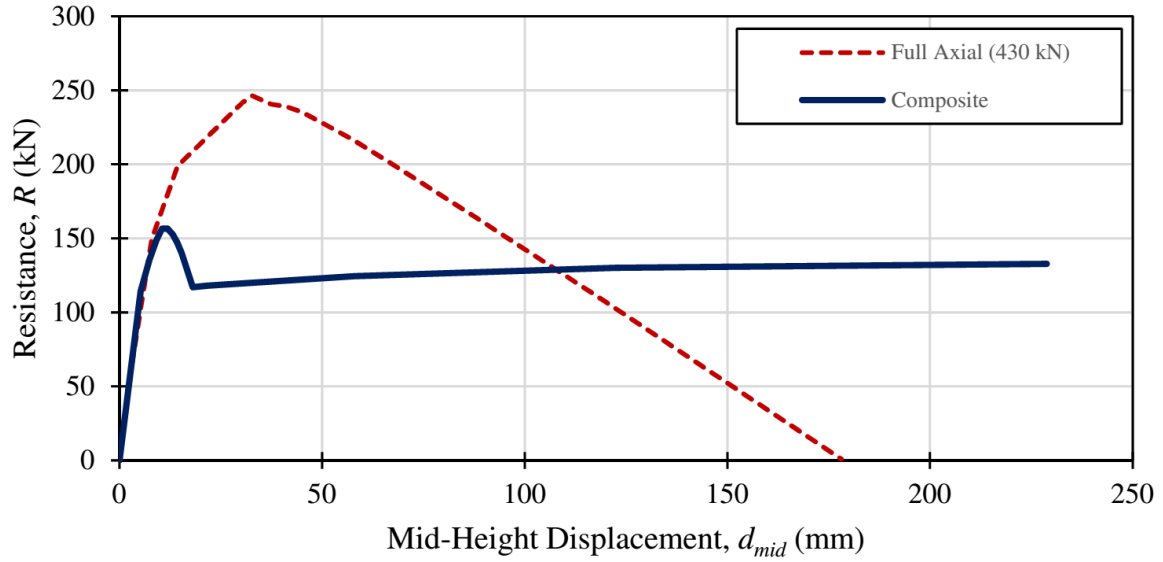


Figure 5-44: Composite Resistance Curve and Resistance Curve for Full Axial Load for Column 2S-J.

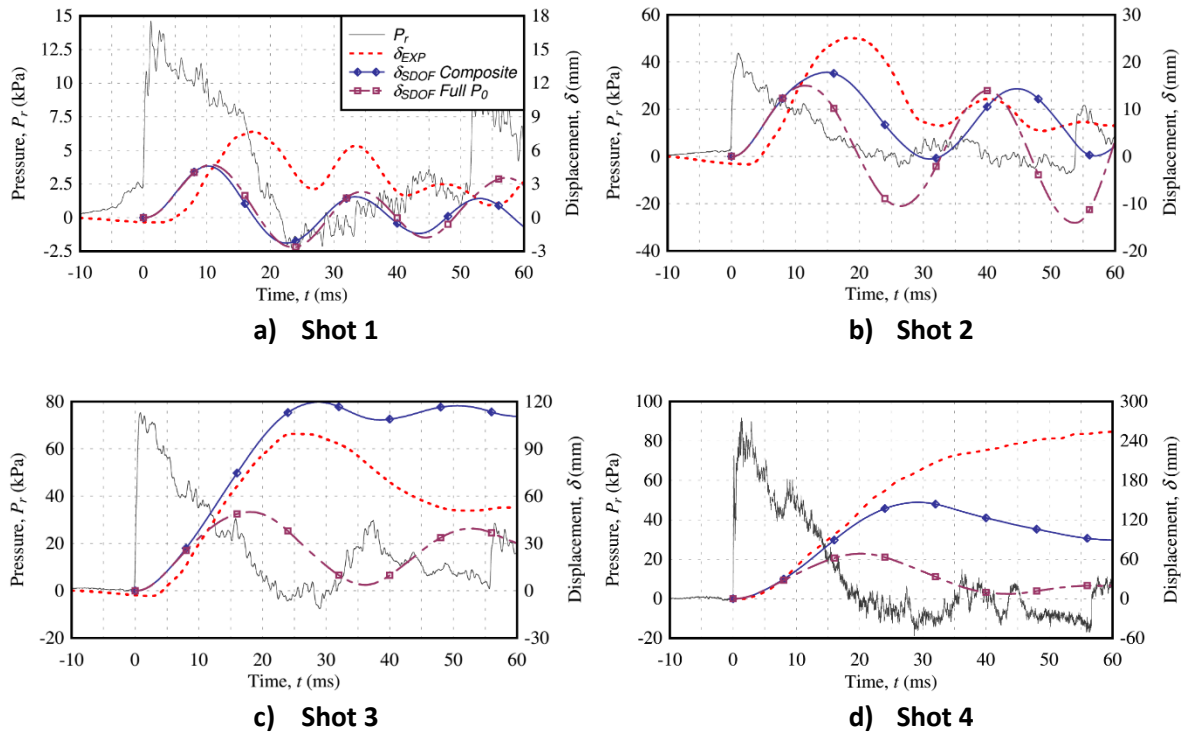


Figure 5-45: Experimental and SDOF Predicted Displacements for Shots 1 through 4 of Column 2S-J using Composite Resistance Curves and Resistance Curve for Full Axial Load.

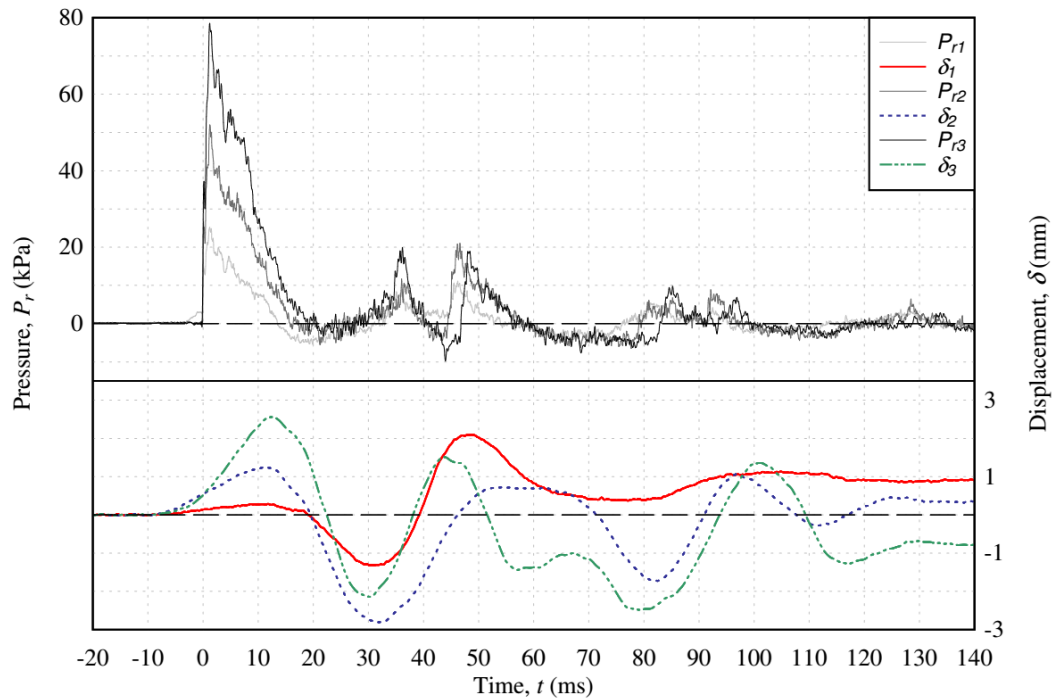


Figure 5-46: Displacement of Shock Tube Prior To and During Shock Wave Loading for Three Different Shock Tube Tests

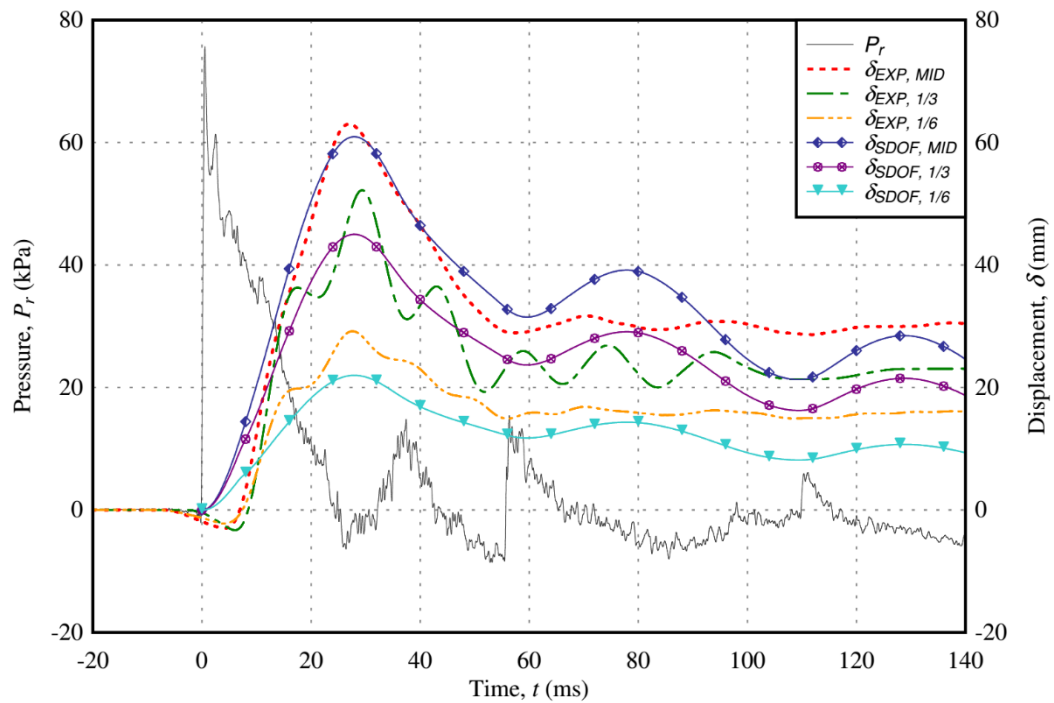


Figure 5-47: Example of Errors in Measurement of Displacement Response: Shock Tube Movement and Loose Displacement Gauge Mounts on Column 3NS-W Shot 3

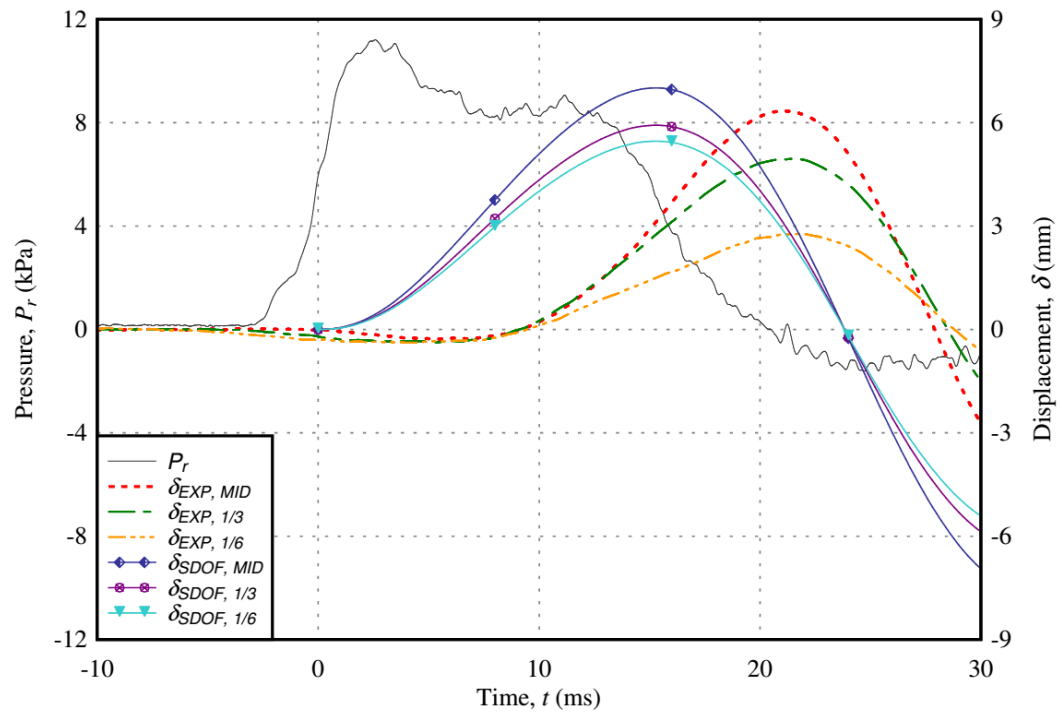


Figure 5-48: Example of Errors in Testing Effecting Displacement Response: Initial Gap between LTD and Column 3NS-TB1 Shot 1.

Chapter 6. Design Procedures and Recommendations

6.1. General

The experimental and analytical results of this study were used to develop guidelines and procedures for structural design. Each of the different retrofits strategies studied are addressed in the following sections.

6.2. Consideration of Axial Load Variation

The possibility of variation of axial load should be considered in a real structure. The observed axial load variation in the laboratory setting may be transferable to real world scenarios, however, special consideration of the specific structure is needed.

The observed vertical movement of the columns, which caused the axial load drop in the laboratory, occurred at a rate comparable to that at which gravity could propel a mass in free-fall. Therefore, for columns in simple structures without a significant amount of continuity, the mass of the structure above the column may not load the column uniformly during the entire dynamic response. The assessment of these columns should consider the possibility of complete or partial loss of axial load. This will have an effect on the stiffness, yield strength, yield deformation, and failure mode of the column. Detailed analysis may involve mult-degree-of-freedom, coupled-single-degree-of-freedom, or other techniques. A variable axial load SDOF model may be appropriate if care is taken to describe the axial load as a function of lateral displacement. In lieu of more complex modeling, it may be possible to assess the column under high and low constant axial load states and ensure that there is significant resistance for both conditions.

Columns that are part of a frame with more continuity may see axial load transferred through more complex mechanisms than simple supported masses. The axial load on the column could be transferred through continuous floor systems from the interior of the structure to the exterior columns. Therefore, for complex structures, the axial load could be applied more as a spring than as a supported mass. In this case, the variability of the axial load is more difficult to generalize. A designer should consider the response of the structural system as a whole, rather than individual columns, to determine the axial load variation during dynamic response.

In any case, a column in a real structure would likely have the combination of a larger cross-section depth and a smaller aspect ratio (column height divided by section depth). Therefore, these columns may have a smaller yield displacement than the columns tested in the laboratory for this project. If yield displacement is the design limit, a smaller maximum value would result in less vertical displacement and, therefore, less possibility of variation in axial load. For stout columns, a designer may find that it is not necessary to design with axial load variation as a consideration.

6.3. Design Procedures for FRP Retrofits

The three different types of FRP retrofits studied in this project are addressed below. These retrofits include:

- Longitudinal FRP for strengthening of columns,
- Transverse FRP for enhanced confinement of columns, and
- Longitudinal FRP with transverse FRP jacketing on top to enhance both strength and ductility of columns.

6.3.1. General FRP Retrofit Guidelines

Regardless of the type of FRP retrofit applied to a column, the following general guidelines should be considered. These guidelines are based on the observations from the experimental testing program as well as current design standards and practices.

- All columns should be designed to have a displacement ductility capacity of a minimum of 4 to attain a performance level similar to those sought for earthquake resistant columns unless the level of strength enhancement justifies a lower ductility.
- Columns with only longitudinal FRP should be avoided unless well anchored to concrete to avoid premature bond failure.
- Longitudinal FRP should be mechanically anchored to the column at terminal end locations or held in place with the use of jacketing FRP unless the strain limit used for flexural strengthening is well below the level at which debonding occurs.
- Design for the use of transverse FRP jacketing as mechanical anchorage of longitudinal FRP should consider the non-uniform distribution of transverse pressure from FRP jackets over the width of square and rectangular column sections.

- Conservative assumptions of low axial load should be used when determining confinement effectiveness of FRP jackets to account for the possible reduction of axial load during blast response associated with lateral displacement of the column.
- If possible, longitudinal FRP should continue beyond the column through the support region with a sufficient development length beyond locations of high stress.
- Columns strengthened with longitudinal FRP should have compression controlled failure rather than tension controlled failure. The stress in the tension FRP should be limited as much as possible and debonding or rupture of the material should be avoided.
- General design procedures for FRP should be followed according to the current Canadian Standard (CSA S806, 2012).

6.3.2. Strengthening with Longitudinal FRP

The use of only longitudinal FRP is not recommended without the use of mechanical anchorage. As columns typically have some degree of rotational fixity at the supports, care should be used to ensure continuity of FRP into the support region to allow for proper development of bond. If appropriate mechanical anchorage is provided, the column should be designed according to the Design and Construction of Building and Structures with Fibre-Reinforced Polymers (CSA S806, 2012) which sets strain limits for FRP at a maximum of 0.7%. The columns in this study achieved higher than the maximum allowable design strain prior to failure of the FRP retrofit. This supports using the established CSA S806 strain limit as a conservative approach.

6.3.3. Confinement with Transverse FRP

Saatcioglu and Razvi (2002) suggested a displacement based design procedure that provides the area ratio of transverse reinforcement in each cross-sectional direction (ρ_c) of a reinforced concrete column as a function of drift ratio (δ), the axial load to concentric axial capacity ratio (P/P_o), gross to core area ratio (A_g/A_c), confinement efficiency of the selected reinforcement arrangement (k_c), and the concrete strength to transverse steel strength ratio (f'_c/f_{yh}). This design equation (Equation [6-1]) was found to accurately determine the amount of confinement steel required to achieve drift ratios corresponding to a 20 % strength decay in the moment resistance while accounting for second order moments for column bending under double curvature due to seismic forces.

$$\rho_c = 14 \frac{f'_c}{f_{yh}} \left[\frac{A_g}{A_c} - 1 \right] \frac{1}{\sqrt{k_c}} \frac{P}{P_o} \delta \quad [6-1]$$

This design procedure was adopted by the Canadian standard for the Design and Assessment of Buildings Subjected to Blast Loads (CSA S850, 2012) for internal confinement reinforcement in new columns and the Design and Construction of Building Structures with Fibre-Reinforced Polymers standard (CSA S806, 2012) for internal FRP reinforcement and external FRP jacketing. It is commonly accepted that fully confined columns based on current building codes can achieve 4% drift ratio prior to a significant loss in moment capacity (Saatcioglu, 2012). Also, the drift ratio for columns of typical height to section depth ratios at yield is approximately 1% (Saatcioglu and Grira, 1999 and Saatcioglu and Baingo, 1999). Therefore, a 4% drift ratio approximately corresponds to a displacement ductility of 4. This relates to the National Building Code of Canada (NRCC, 2010) ductility related force modification factor $R_d=4$ for fully ductile reinforced concrete frames. The CSA S850 standard design equation for ductile blast resistant columns was adopted after substituting 0.04 in Equation as design drift ratio, reflecting the drift level associated with a ductile column. In CSA S806 the equation was expressed in terms of the required FRP jacket thickness (t_j) that would provide the desired level of ductility.

Typical building columns have a high level of rotational fixity at the column ends such that they approach fully fixed support conditions. In some cases, pin connections or other variations of end supports are possible. Figure 6-1 shows simplified elastic moment and displaced mode shapes and curvature distributions for pin-ended columns (a) and fully fixed columns (b). In the case of a pin-ended column, the drift ratio for elastic response is calculated as the ratio d_1/L_1 where d_1 is the displacement at mid-height and L_1 is half the column height. For the case of the fully fixed column there are two drift ratios that can be considered: d_1/L_1 and d_2/L_2 . In these two ratios the displacement term is displacement between the points of inflection and the length term is the length between inflection points. For an elastic member, these drift ratios are approximately equal. As the column approaches its fully ductile displacement capacity, the most significant component of displacement is a result of inelastic rotation within the plastic hinge regions. For fully ductile columns experiencing significant inelasticity, it is common practice to use two rigid chords with three hinge points as the plastic mode shape (Biggs, 1964 and Department of Defense, 2008). This is shown as the plastic displaced shapes in Figure 6-1 (a) and (b). For this inelastic displacement, the drift level is calculated as the mid-height displacement divided by half the column height for all types of end conditions. Since the primary displacement is due to the rotation over the plastic hinge length, L_p , when the

column approaches its fully ductile response, the drift capacity becomes essentially independent of the boundary conditions assuming that the curvature ductility capacity is the same for all hinge locations. This assumption is likely to be valid as reinforcing steel is generally evenly distributed in the positive and negative moment regions and confinement reinforcement extends evenly spaced over the entire length for properly detailed columns designed for blast.

Since it is commonly accepted that the ability to deform to 4% drift ratio is associated with fully ductile behaviour and that inelastic drift levels are not a function of boundary conditions for columns loaded under blast, the use of Equation [6-1] is valid for blast design of transverse reinforcement. Therefore, Equation [6-2] is obtained as the displacement based design equation in CSA S850 with 0.04 as the drift ratio. In this equation, rather than solving for the area ratio of transverse reinforcement in each cross-sectional direction (ρ_c), the area of transverse reinforcement is solved directly (A_{st}) and the spacing of transverse reinforcement (s) and the core dimension perpendicular to lateral force (h_c) are introduced as parameters to be consistent with CSA Standard A23.3 “Design of Concrete Structures” (CSA A23.3-2004).

$$A_{st} = 0.6sh_c \frac{f'_c}{f_{yh}} \left[\frac{A_g}{A_c} - 1 \right] \frac{1}{\sqrt{k_c}} \frac{P}{P_o} \quad [6-2]$$

6.3.3.1. FRP Jacket Design

For columns that do not meet the confinement reinforcement requirement of CSA S850, a viable approach is to use FRP jackets to improve inelastic deformability. CSA S806 has incorporated Equation [6-1] as a displacement based design tool to determine the required thickness of an FRP jacket to achieve a desired level of lateral drift ratio. In Equation [6-3], the area or area ratio of confinement steel is replaced with the FRP jacket thickness, t_j , and the gross to core area ratio is replaced by its lower limit as the entire section is now considered core concrete measured by width D . The confinement efficiency of the FRP jacket depends on the shape of the cross section. For circular sections $k_c=1.0$ and for square sections k_c is calculated using Equation [6-4] (Saatcioglu et al., 2008). In Equation [6-4] the confinement efficiency base value of 0.4 is multiplied by a factor relating the width of the column to the distance between peak lateral pressure points generated by transverse FRP reinforcement. This factor accounts empirical observations made by Ozbakkaloglu and Saatcioglu (2008) relating the effectiveness of continuous reinforcement to discrete reinforcement for confinement efficiency. The effective FRP stress in the transverse direction, f_{Fj} , is determined

based on the ratio of axial load to concentric capacity (P/P_o) as shown in Equations [6-5] through [6-7] (Saatcioglu et al., 2008). There is a possibility of decrease in axial load during the dynamic response of the columns, especially for columns displacing to high drift values associated with the ductility design approach. It is recommended that the lower limit for FRP confinement stress be employed (Equation [6-5]) for blast resistant design.

$$t_j = 2D \frac{f'_c}{f_{Fj}} \frac{P}{P_o} \frac{\delta}{\sqrt{k_c}} \quad [6-3]$$

$$k_c = 0.4 \sqrt{\frac{b_c}{s_l}} \leq 1.0 \quad [6-4]$$

$$\frac{P}{P_o} < 0.15, f_{Fj} = 0.003E_f \quad [6-5]$$

$$0.15 < \frac{P}{P_o} < 0.3, f_{Fj} = 0.002E_f \frac{P}{P_o} \quad [6-6]$$

$$\frac{P}{P_o} > 0.3, f_{Fj} = 0.006E_f \quad [6-7]$$

In calculating the drift capacity of the member, superposition of Equations [6-1] and [6-3] may be used to account for the effects of steel and FRP confinement. However, it is conservative to consider only the confinement effects of the FRP when determining the minimum thickness of FRP jacketing required with Equation [6-3]. For a fully ductile column, the design drift ratio (δ) in Equation [6-3] should be taken as 0.04 (4.0%) to represent a displacement ductility of 4.0.

Columns 2NS-C, 2S-C, 2NS-J and 2S-J were assessed using Equation [6-3]. These columns represented control columns with low confinement (2NS-C with transverse reinforcement spacing at $h/2$) and high confinement (2S-C with transverse reinforcement spacing at $h/4$) along with FRP jacketed companion columns with 1.0mm thick CFRP jacket uniformly applied over the length of the columns. Using the column geometry, reinforcement detailing and material properties, the displacement capacities for the as-built columns were computed using Equation [6-1] as 33mm and 78mm for the non-seismic and seismic columns respectively. The application of a single layer of CFRP increased the displacement capacities to 75mm and 120mm as computed from the superposition of drift values from Equations [6-1] and [6-3], reflecting the contributions of internal steel confining reinforcement and external CFRP jacket, respectively. As the experimental testing program was force controlled, experimental values of resistance force at computed displacement levels are not available. The

columns were initially loaded to approximately 30% of their concentric axial load capacity. This load was applied with a hydraulic jack that compressed the column between a two rigid supports. When the columns displaced laterally, the projected length of the columns decreased causing a significant drop in axial load with horizontal displacement. This change in axial load resulted in beam bending behaviour where the neutral axis shifted into the core of the column resulting in less confinement effects and a reduction in confinement efficiency. Due to this drop in axial load during the column response, the lower limit of 15% of the concentric axial capacity was taken as the axial load levels in Equation [6-3].

Column 2NS-J withstood pressures that resulted in approximately the computed inelastic displacement capacity with little damage and residual displacement, whereas the companion non-retrofitted column (2NS-C) was completely destroyed at the same level of pressure. Similarly, Column 2S-J sustained inelastic deformations approaching the computed value, developing approximately 100mm of total displacement and only a one third of the computed displacement capacity as residual displacement (approximately 40mm), whereas the companion non-retrofitted column (2S-C) suffered significant damage under the same pressure, sustaining in excess of 100mm of residual displacement. The use of force controlled testing without direct measurement of the resistance-displacement response of the columns prevents more in-depth experimental observations on the accuracy of Equations [6-1] and [6-3].

Another mode of failure in reinforced concrete columns within the plastic range of deformations is the buckling of compression reinforcement. This failure mode is significantly delayed in seismic resistant columns by ensuring sufficiently small spacing of column ties. Columns with wide spacing of internal transverse reinforcement, retrofitted with FRP jackets may be susceptible to the same failure mode if the longitudinal reinforcement is of a very large size relative to the tie spacing and the concrete cover and FRP jacket thicknesses are inadequate. However, the columns tested in the current experimental program did not show buckling of compression bars prior to achieving the target ductility level. Therefore, until further research results become available on the relationship between the bar size, tie spacing, cover thickness and FRP jacket thickness for FRP retrofitted columns, it may be assumed that this failure mode may not be critical for columns having similar A_g/A_c and longitudinal reinforcement ratios as those tested in the experimental program.

6.3.4. Strengthening and Confinement with Longitudinal and Transverse FRP

The experimental research showed that the use of transverse FRP on top of the longitudinal FRP provided enough mechanical anchorage to allow for complete bond of the longitudinal FRP to the column up to the point of rupture of the material. This rupture occurred at approximately 1.3% strain in the longitudinal FRP. In the case of columns with larger cross-section dimensions, the mechanical clamping effect that the transverse FRP places on the longitudinal FRP may be reduced due to the unfavorable geometry of the cross section. If the transverse FRP is considered sufficient to provide mechanical anchorage to the longitudinal FRP, it may be possible to consider an increased ultimate tension strain in the longitudinal FRP fibers. However, best practices would likely have designers apply the more conservative ultimate limits for FRP strain of 0.7% (or a smaller calculated value) as prescribed by the Canadian standards for FRP design (CSA S806, 2012). In this approach, instead of considering an increased FRP design strain, the designer could consider the combined effects of the transverse and longitudinal FRP on the column resistance while retaining an additional level of resiliency in the structure due to the enhanced bond. Designers should consider the following:

- The transverse FRP should be designed based on a displacement ductility approach as outlined in the previous section.
- The effect of the confinement FRP should be considered on the concrete material properties both in the cover concrete and in the core of the column. The concrete should have increased strength and material ductility capacity.
- The ability of the confinement FRP to resist buckling of longitudinal steel should be considered.
- The longitudinal FRP should add an additional tension force component to the cross section of the column and contribute to the moment resistance in sectional analysis.
- The column should be designed to achieve a displacement ductility ratio of 4 prior to the longitudinal FRP reaching its strain limit.
- To avoid brittle column failure due to longitudinal FRP failure at ultimate strain, the column should be designed to be compression dominant (over reinforced after the addition of FRP reinforcement). Care should be taken to ensure that the following is considered:
 - The column may lose significant axial load during dynamic response. The column should be checked at zero axial load to ensure that the flexural capacity is still governed by compression failure.

- The curvature ductility of the column may be considerably enhanced due to the confinement FRP increasing compression concrete performance by limiting spalling of cover, increasing compression strength, and preventing reinforcement buckling. This should be considered in the design when ensuring the column has sufficient displacement ductility prior to tension failure of the longitudinal FRP.

6.4. Design Guidelines for Prestress Confinement

The column retrofitted with external transverse prestressing for improved confinement (3NS-W) performed well under the experimental testing program in that:

- Buckling of reinforcement was prevented.
- Crushing of compression concrete and spalling of cover concrete was limited.
- Relative damage between the retrofitted column and the control column (3NS-C) under similar pressure and impulse combinations was lower.

This behaviour shows that the prestressed jacketed column is a successful retrofit technique for which some design guidelines should be established. The improvements in column flexural behaviour in this application results from the active confinement of compression concrete introduced through transverse prestressing, and the increase in column mass due to the use of precast concrete raisers. In the experimental phase of the current research project only one column was tested utilizing this technique for verification of the concept (though it was also used in a column with compression brace for buckling control). Therefore, while it is not possible to develop a general design procedure based on a single test for application to blast-resistant columns, the earlier procedure developed by Saatcioglu et al (2002) and Saatcioglu and Yalcin (2003) for the application of the same technique to seismic retrofitting can be employed to establish the required area of prestressing steel and the level of transverse prestressing for the required ductility demand. The deformation demands and the confinement requirements should be computed through dynamic analysis of the column with due considerations given to the added mass.

The following are suggested as part of the overall design considerations:

- Uniform spacing of the transverse reinforcement over the length of the column is encouraged.

- Conversion of rectangular cross sections to circular through the use of precast concrete risers or other mechanical means is necessary to ensure lateral pressure is uniformly applied over the column face.
- Gaps in the spacing of the risers and the prestress material should not occur in regions likely to form plastic hinges. If gaps do occur in these locations, the hinge length should be considered to be very small. By observation, the hinge length in the experimental testing was approximately two times the gap size between the concrete wedges at mid-height. This shorter hinge length reduces displacement ductility capacity.
- Transverse material should be prestressed to a low enough level to allow for lateral pressures during dynamic response.

6.5. Design Guidelines for Bracing

Either tension or compression braces may be used to retrofit columns. The choice of brace type depends primarily on the following factors:

1. The level of axial load on the column

Tension braced columns act as independent systems where the axial load and the vertical load induced by the brace elements are placed on the column itself. For columns that begin with a relatively low initial axial load, there is remaining axial capacity that is needed to balance the load from a tension brace. Columns with a high initial axial load will not be able to withstand significant increases in axial load due to tension brace forces and should be retrofitted with compression brace elements that do not induce axial load in the column.

2. The degree of redundancies in the structure and resistance to uplift

Columns in simple structures with few alternate load paths, limited continuity of floor systems between supports, and limited continuity between columns of adjoining levels should be retrofitted with tension braces. Compression braces rely on the ability to transfer vertical loads from the braced element into the support system where they will be resisted by gravity loads and the structures ability to transfer loads to other vertical load carrying members. Larger structures that have multiple levels, continuous floor systems, and interior columns not subjected to blast loads are candidates for the compression brace retrofit. If

vertical movement of the support region are possible under uplift loads associated with the compression braces, compression braces should not be used.

3. The amount of effort and cost required for each brace type.

Tension brace retrofits are likely significantly less expensive and require less labour than compression brace retrofits. Tension brace elements can fit any column geometry with only the length of the threaded rod bracing elements, width of the bearing plates, and width of the shear pins being custom to the column. These elements are simple to modify with little effort and cost and have high tolerances as any looseness can be overcome by tightening the turnbuckles. Compression brace components are custom fit to each column and require very low tolerances to be effective. Furthermore, buckling prevention of compression brace elements must be considered either through use of prestressed confinement and wedge elements or other mechanical anchorage of the elements to the column.

6.5.1. Tension Brace Retrofit Guidelines

General guidelines for tension brace retrofits are provided below.

- The strength of the brace material should be balanced with brace material ductility. Very high strength and low ductility material should be avoided in place of lower strength, higher ductility material with larger area.
- All couplers and shear pins should be designed to remain elastic up to the point of ultimate stress in the threaded rod brace material. This includes thread stripping, weld failure, block shear failure, tension yielding and other appropriate checks.
- The threaded rod should not be tightened to the point of significant preload on the column without further study. Preloading the column could promote rebound failure of the column.
- Bearing plates should be used to distribute forces to the column at the interior brace locations.
- Double shear planes (at a minimum) should be used at the hinge connection location to reduce eccentricity on the shear pin and reduce shear demand in the pin. Hinges with more than two shear planes per connection may be designed to further reduce shear demand on the shear pin.

- Hinges should be placed tight to the surface of the column to minimize the length of the shear pin. No cantilever of the shear pin between the bearing plate and the brace hinges should be present.
- Turnbuckles should be placed on the vertical bracing elements rather than the diagonal bracing elements. This reduces the chance of failure at the turnbuckle location as the vertical members have lower force demands than the diagonal members.
- Columns should be sufficiently ductile under high axial loads to withstand the combined axial and flexural demands if they are designed to behave in the inelastic range of deformations. The use of FRP jacketing (or other jacketing for confinement) is recommended for such columns to prevent concrete crushing failure.
- Brace support connections should be designed to fully transfer lateral loads into the floor systems, not as shear forces on the columns. Vertical loads should be distributed to the columns symmetrically to minimize out-of-plane eccentricity.
- In lieu of physical testing of threaded rod mechanical properties, conservative estimations on effective brace area, yield stress, and ultimate stress based on thread pitch, bolt diameter, and thread cut depth should be used.

Brace points should be placed at approximately 1/3 span locations.

6.5.1.1. Tension Brace Retrofit Design Assumptions

Due to the complicated interaction between the tension brace and the column discussed in the previous chapter, a simplified approach was developed to design tension braced columns to resist a given lateral load in lieu of more robust analysis. The design procedure uses the following assumptions:

1. The column responds with a mode shape approximately the same as that of a pin-ended member.

This assumption was validated in the analysis from the previous chapter. The eccentric axial load from the tension brace along with destabilizing moments on the supports due to eccentrically applied horizontal brace reaction forces induced enough end rotation to approximate simple supports.

2. All axial forces on the column has the same eccentricity as the vertical brace eccentricity at the column end.

This assumption allows for a simplification of the axial load induced end moments in design of the retrofit. Any of the concentric axial load on the column prior is conservatively moved to the location of brace eccentricity.

3. The column responds with a linear resistance-displacement response up to yield.

The resistance-displacement response of the column is only determined at the point of column yield in this design approach. The response up to yield is considered linear.

4. The maximum capacity of the column is achieved when yielding occurs at mid-height.

For columns with any degree of rotational restraint at the supports, yielding at mid-height is considered as the column yield point. The eccentric end moments due to the brace elements reduce the end moment demand on the column in proportion to the mid-height moment. Columns that do yield at the ends prior to mid-height are assumed to have sufficient ductility capacity to allow yielding at mid-height. Yielding at mid-height is the maximum allowable column response in this design procedure.

6.5.1.2. Simplified Design Procedure

The simplified design procedure for tension brace retrofitted columns is as follows.

1. Find the moment-axial load interaction diagram for the column.
 - a. As this is a simplified design procedure, the designer may opt to not include dynamic increase factors for strain rate effects in the determination of the interaction diagram. In general, this is a conservative assumption.
2. Determine an estimate of yield moment and rigidity of the column under high axial load associated with the braced system. The design axial load of the column including brace load will be greater than the service axial load on the column therefore:
 - a. An estimate of axial load should be found as a value between initial axial load and maximum axial load capacity.
 - b. The corresponding yield moment to the initial axial load assumption may be found from the moment-axial load diagram.
 - c. The rigidity of the column can be determined using the axial load dependent flexural rigidly equation below (Tikka and Mirza, 2008) or an appropriate alternative.

$$EI = \alpha_c E_c I_g + 0.8 E_s I_s \quad [6-8]$$

$$\alpha_c = 0.47 - 3.5 \frac{e}{h} \left[\frac{1}{1 + \beta \left(\frac{e}{h} \right)} \right] + 0.003 \frac{L}{h} \geq 0 \quad [6-9]$$

$$\beta = 7 \text{ for } \rho < 2 \quad \text{and} \quad \beta = 8 \text{ for } \rho > 2 \quad [6-10]$$

$$E_c = 4500 \sqrt{f'_c} \quad [6-11]$$

$$e = \frac{M_y}{P} \quad [6-12]$$

where E_c is the modulus of elasticity of concrete, E_s is the modulus of elasticity of steel, I_g is the gross moment of inertia of the cross-section, I_s is the moment of inertia of the reinforcing steel, α_c is a rigidly softening coefficient reflecting end eccentricity and slenderness, β is a coefficient reflecting the reinforcement ratio, $\frac{e}{h}$ is the eccentricity ratio for the column and is calculated based on a conservative assumption of yield end moment and single curvature bending, and $\frac{L}{h}$ is the aspect ratio (length to height of the section) of the column.

3. Determine the yield displacement at mid-height ($d_{y,mid}$) and the brace point ($d_{y,brace}$) of the column using an assumed simple supported uniformly distributed lateral load mode shape. The mid-span yield displacement is given by Equation [6-13] and, if the brace point is placed at the 1/3 span, the brace point displacement at yield is given by Equation [6-14].
 - a. Note that this is not an approximation of the response of the as-built column, rather a Step in the simplified design process for the braced system. This Step is based on the assumption that the sum of all distributed moments in the final braced system can be approximated by the moment distribution on a simply supported member with a uniformly distributed lateral load as was concluded in the analytical section of this project. The response of the as-built column should account for the axial load induced secondary moments and the realistic boundary conditions of the column.

$$d_{y,mid} = \frac{5M_y L^2}{48EI} \quad [6-13]$$

$$d_{y,brace} = 0.87 d_{y,mid} \quad [6-14]$$

4. Determine the required increase in stiffness needed such that the yield displacement is not exceeded under the design loads.
 - a. The system without the brace has a yield displacement of $d_{y,mid}$ at a resistance of $R = \frac{8M_y}{L}$ with a load-mass factor of $k_{LM} = 0.77$.
 - b. The system with the brace has the same yield displacement, $d_{y,mid}$, and load mass factor, $k_{LM} = 0.77$. The increase in stiffness is found by use of either a SDOF analysis or static analysis with dynamic load factors. In either case, the resistance of the system is varied until the maximum displacement is equal to the yield displacement, $d_{max} = d_{y,mid}$. Figure 6-2 shows the elastic dynamic load factor distribution as a function of the ratio of natural period of the SDOF system and the duration of a triangular impulsive load function (Department of Defense, 2008). The natural period of the SDOF system, T_N , is given by Equation [6-15] which is calculated using the stiffness, K in Equation [6-16] and the SDOF mass. The duration of the triangular load, T , found using the design impulse and pressure is given by Equation [6-17]. The mid-span displacement of the system is found using Equation [6-18] with the dynamic load factor (DLF) from Figure 6-2.

$$T_N = \frac{2\pi}{\sqrt{\frac{K}{k_{LM}m}}} \quad [6-15]$$

$$K = \frac{R}{d_{y,mid}} \quad [6-16]$$

$$T = \frac{2I_r}{P_r} \quad [6-17]$$

$$d_{max} = DLF \frac{5P_r A L^3}{384EI} \quad [6-18]$$

where K is the stiffness of the SDOF system, $k_{LM}m$ is the system mass multiplied by the load-mass factor, R is the total resistance of the system to lateral load, I_r is the design reflected impulse, P_r is the design reflected pressure, A is the loaded area of the system, and DLF is the dynamic load factor.

5. Find the ratio of the horizontal brace force to the vertical brace force. For braces at 1/3 span, this is found using Equation [6-19].

$$\frac{F_V}{F_H} = \frac{y_b}{d_{y,brace} + x_b} \quad [6-19]$$

where F_V is the total vertical brace force, F_H is the horizontal force from the brace at the brace point, y_b is the distance along the column from the end of the column to the interior brace point, $d_{y,brace}$ is the yield displacement of the column at the brace point, and x_b is horizontal distance between the vertical brace at the end of the column and the vertical brace at the interior brace point of the column as shown in Figure 6-3.

6. Determine the required horizontal force in the brace and the required vertical force in the brace using Equation [6-20] and the force ratio from Equation [6-19] in Equation [6-21].

$$F_H = \frac{RL - 8M_y}{8y_b} \quad [6-20]$$

$$F_V = \left(\frac{F_V}{F_H} \right) F_H \quad [6-21]$$

where R is the total resistance of the braced system (found in the SDOF or equivalent static procedure above), L is the length of the column, M_y is the column yield moment, and y_b is the distance from the end of the column to the brace point.

7. Verify the validity of the assumed yield moment and yield displacement found in Step 2 using the interaction diagram for the column and the axial load equal to the total vertical force in the brace.
 - a. If there is considerable difference between the assumed values and the values found using F_V , the process should begin again at Step 2.
 - b. If the vertical load is approaching concentric capacity of the column, the column needs modification to increase compression capacity or an alternate retrofit technique should be used.
8. Calculate the vertical displacement in the brace due to column shortening from flexure (Δy_{flex}), column shorting from axial load change (Δy_P), and end rotation of the floor system if applicable (Δy_θ) using Equations [6-22] through [6-25].

- a. For columns with extremely rigid floor systems (column joint systems) or for columns with connecting columns above and below that are retrofitted with tension braces, the vertical brace displacement due to end rotation may be taken as zero as all end rotation will occur in the column and no brace shortening will occur due to rotation.

$$\Delta y_{flex} = 0.043d_{y,mid} \quad [6-22]$$

$$\Delta y_P = \frac{(F_V - P_i)L}{E_c A_{tr}} \quad [6-23]$$

$$A_{tr} = A_g + A_s \left(\frac{E_s}{E_c} - 1 \right) \quad [6-24]$$

$$\Delta y_\theta = \frac{2}{3} x_{b,end} M_y L \quad [6-25]$$

where A_g is the gross area of the column section, A_{tr} is the transformed area of the column section, P_i is the initial axial load on the column (column load under service conditions), and $x_{b,end}$ is the eccentricity of the brace at the end of the column relative to the centroid of the cross-section. In determining the brace shortening due to support rotation, the rotation of the end of the simply supported system was used as the rotation angle and small angle theory was used to determine the vertical displacement at the offset brace due to the rotation.

9. Calculate the total force in the diagonal brace, F_D , using the Equations [6-26] through [6-29] where the $L_{b,V}$, $L_{b,H}$, and $L_{b,D}$ terms represent deformed brace components lengths in the vertical and horizontal directions along with the total deformed diagonal brace length.

$$L_{b,V} = y_b \left(1 - \frac{\Delta y_{flex} + \Delta y_P + \Delta y_\theta}{L} \right) \quad [6-26]$$

$$L_{b,H} = d_{y,brace} + x_b \quad [6-27]$$

$$L_{b,D} = \sqrt{L_{b,H}^2 + L_{b,V}^2} \quad [6-28]$$

$$F_D = F_V \frac{L_{b,D}}{L_{b,V}} \quad [6-29]$$

10. Using the force in the brace, find the required brace area.

Note that this method may or may not have the brace yield at the column yield displacement. If the brace has yielded (Based on the strain found using the deformed diagonal brace length), the brace ductility capacity should be confirmed. If desired, the process can be repeated using smaller column displacements until a design with no brace yielding is achieved. In this case a moment less than the yield moment of the column is used in the design procedure

6.5.2. Compression brace guidelines

General guidelines for compression brace retrofits are provided below.

- All components of compression brace systems should fit the column precisely for improved effectiveness.
- The largest angle of the diagonal brace elements possible should be used.
- The strength of the brace material should be balanced with brace material ductility. Very high strength and low ductility material should be avoided in place of lower strength, higher ductility material with larger area.
- Bearing plates should be used to distribute forces to the column at the interior brace locations.
- Brace points should be placed at approximately 1/3 span locations.
- The brace should be sufficiently affixed to the column to prevent buckling failure. This may be in the form of the retrofit tested in this study (with prestressed condiment steel and concrete wedges restraining buckling) or by other mechanical means.

6.5.2.1. Simplified Design Procedure

In lieu of more detailed calculations, compression braces may be designed using the following procedure.

1. Determine the zero axial load resistance-displacement curve for the as-built column and identify the yield displacement.
2. Determine the initial axial compression displacement (Δy_p), of the column using the elastic modulus for concrete specified in Equation [6-11], the transformed section area in Equation [6-24], the initial axial load on the column (P_i), and Equation [6-23] with F_v set equal to zero.
3. Determine the displacement at which all axial load on the column is zero by setting the vertical displacement due to flexure (Δy_{flex}) from Equation [6-22] equal to the initial

axial compression displacement (Δy_p) and solve for the horizontal displacement at mid-span in the column (d_{mid}). If this displacement is less than the yield displacement, the column will yield after all axial load is transferred to the brace. If it is greater than the yield displacement, the column will retain a portion of the axial load at yield.

4. Determine the displacement of the brace point ($d_{brace,yb}$) and at the mid-height of the column ($d_{mid,yb}$) at yield of the brace using Equations [6-30] to [6-32]

$$L_d = (y_b^2 + x_b^2)^{1/2} \quad [6-30]$$

$$d_{brace,yb} = x_b - L_d(1 - \varepsilon_{y,b}) \quad [6-31]$$

$$d_{mid,yb} = 1.15d_{brace} \quad [6-32]$$

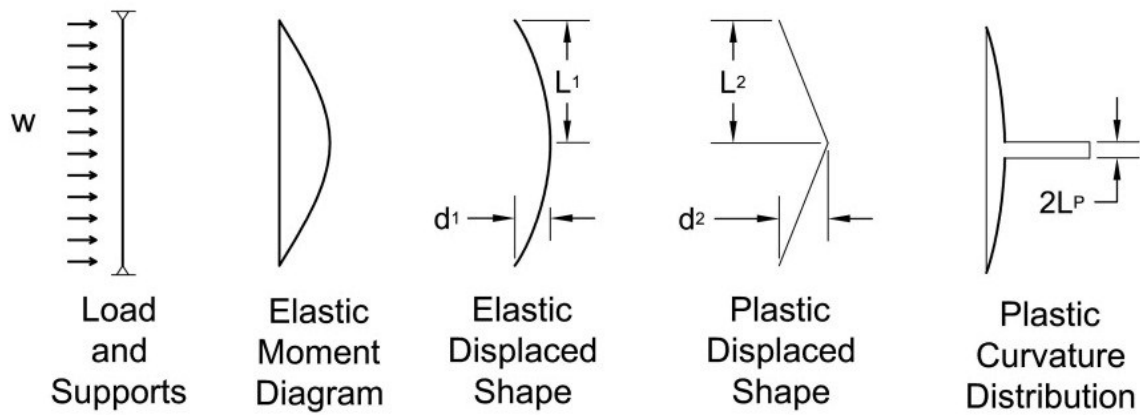
where L_d is the length of the diagonal brace component at the centroid, x_b is horizontal distance between the vertical brace at the end of the column and the vertical brace at the interior brace point of the column and y_b is the vertical distance as shown in Figure 6-4. In Equation [6-31], $\varepsilon_{y,b}$ is the yield strain of the compression brace. The above formulation assumes that the vertical movement of the column ends is zero, the column deforms with the non-braced mode shape, the brace resists force about its centroid, and there is no fabrication error in the brace length. These assumptions were based on the observations from the experimental and analytical program.

5. Select a brace area and compute the total horizontal brace force (sum of both brace points) at the yield of the brace using Equation [6-33]

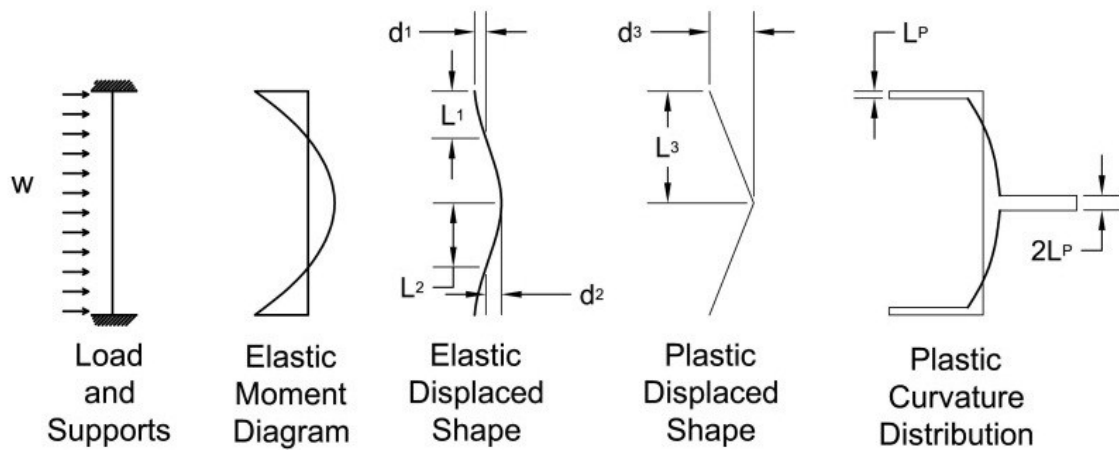
$$F_H = 2 \left[2A_b E_b \varepsilon_{y,b} \left(\frac{x_b - d_{brace,yb}}{L_d(1 - \varepsilon_{y,b})} \right) \right] \quad [6-33]$$

where A_b is the area of a single brace element and E_b is the brace modulus of elasticity. The two multiples of 2 account for braces on both sides of the column and two brace points on the column.

6. Add the brace force curve to the zero axial load column resistance-displacement curve to gain the total resistance curve for the retrofitted column. This is illustrated in Figure 6-5 and Figure 6-6 for columns with braces yielding before and after the yield of the unbraced column. The SDOF conversion factors (K_{LM}) may be assumed to be unchanged from the non-braced column.



(a) Pin-Ended Column



(b) Fully Fixed Column

Figure 6-1: Simplified Displaced Shapes and Curvature Distributions for Columns under Blast Loads.

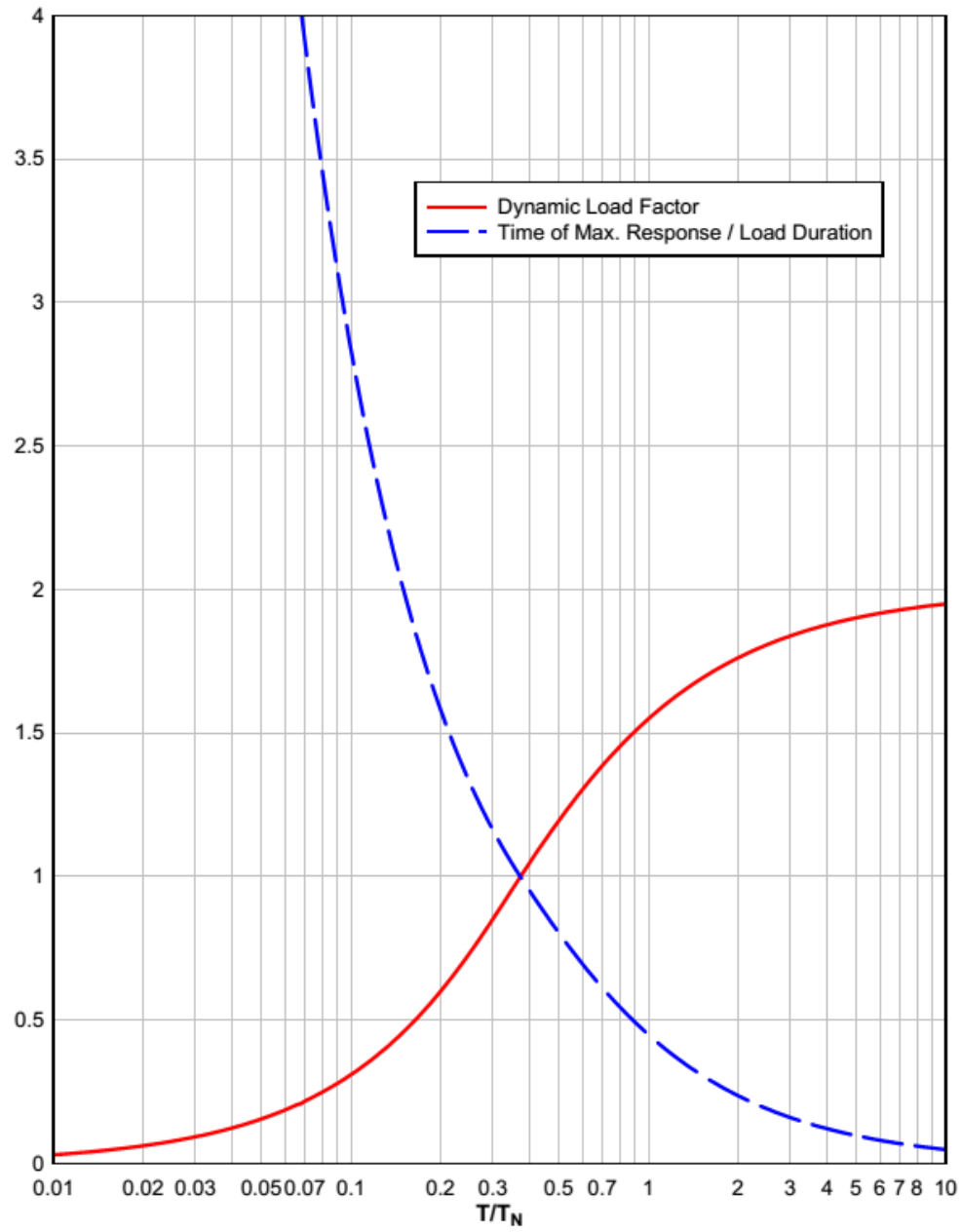


Figure 6-2: Dynamic Load Factor for Elastic Systems (Department of Defense, 2008).

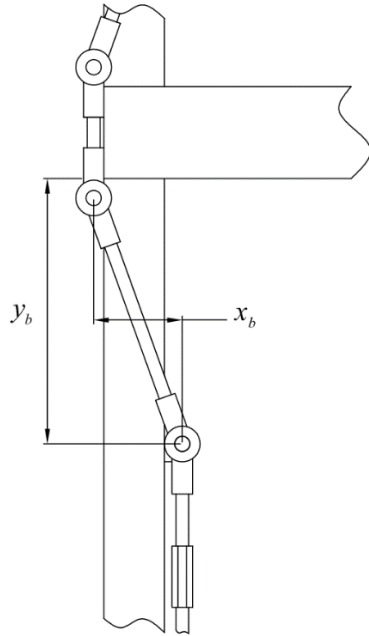


Figure 6-3: Typical Tension Brace Configuration on Column showing Distance to Brace Point and Brace Eccentricity.

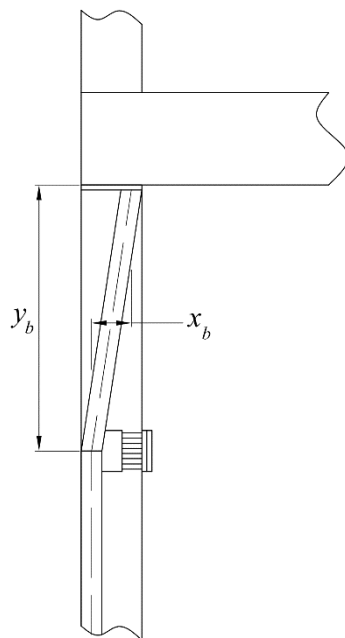


Figure 6-4: Typical Compression Brace Configuration on Column showing Distance to Brace Point and Brace Eccentricity.

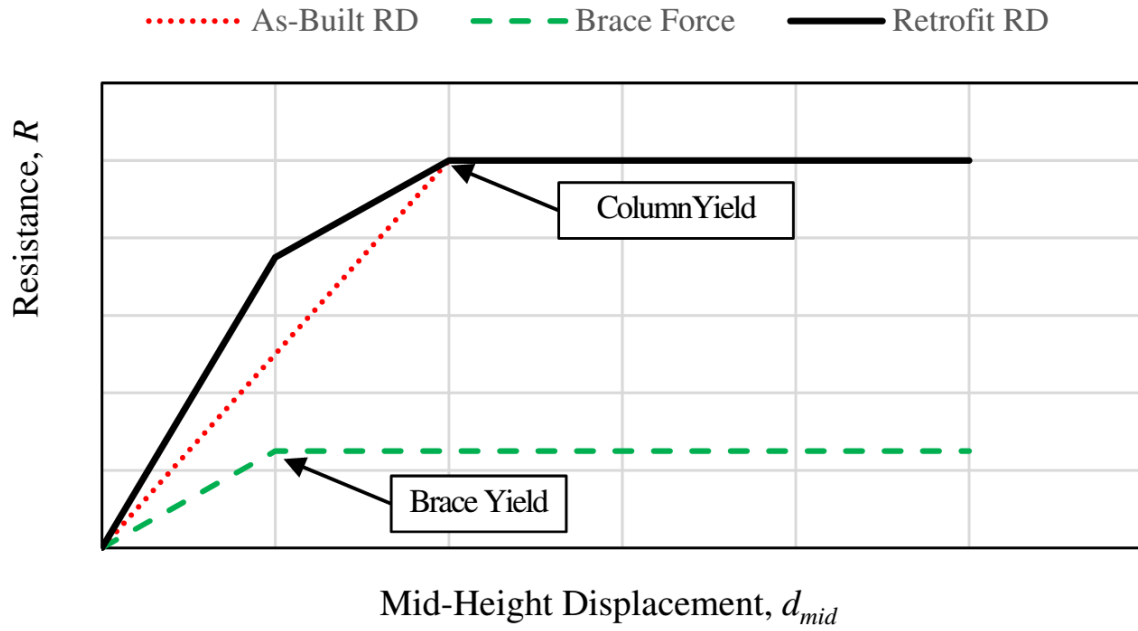


Figure 6-5: Resistance Curve for Compression Brace Retrofitted Column with Brace Yielding prior to Column Yielding

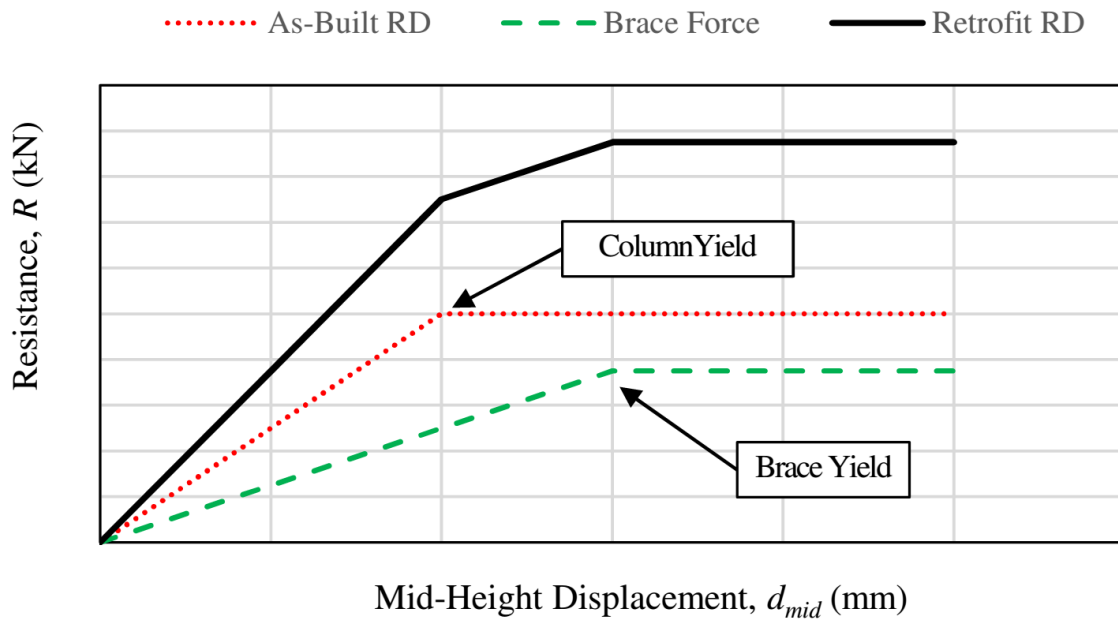


Figure 6-6: Resistance Curve for Compression Brace Retrofitted Column with Brace Yielding post Column Yielding

Chapter 7. Conclusions

7.1. General

Columns of multistory buildings are responsible from carrying gravity loads, as well as resisting lateral forces due to wind, earthquakes and possible bomb blasts. First storey columns play especially important roles as they are responsible from overall strength and stability of the column stacks above them. Bomb threats to critical infrastructure are often in the form of external surface explosions with increased effects on the closest columns to the ground. Therefore, the resistance of columns, especially at the first-storey level, is critical in reducing the likelihood of collapse of structures after a blast. Reinforced concrete columns may be deficient in lateral load resistance leading to the loss of part or all of the gravity load resistance of a structure after an explosion. These columns may be lacking in strength or in ductility capacity and, therefore, require retrofit to ensure they maintain capacity during and after the extreme event.

7.2. Summary

The research project, reported in this thesis, has the objective of developing new and innovative blast retrofit technologies, while also improving the existing methods. It consists of combined experimental and analytical research. The experimental phase involves extensive testing of large-scale reinforced concrete columns, retrofitted with different techniques. The techniques considered include; the use of FRP, transverse prestressing, and lateral bracing. The experimental research was conducted in the Blast Research Laboratory of the University of Ottawa with simulated blast loads generated by the University of Ottawa shock tube. This facility has been shown to accurately reproduce shock waves that are representative of those driven from high explosives. An experimental program was developed that used the shock tube along with a load transfer device and boundary conditions designed to represent realistic column connections to test a series of columns.

Sixteen reinforced concrete columns were constructed (along with the data from two further columns taken from a previous study conducted by the author). The columns were approximately half-scale in size and built to represent those designed for seismic and non-seismic regions in Canada. A series of columns was retrofitted with CFRP in various layups. Other columns were retrofitted with transverse prestressed confinement steel or with steel braces. Each column was tested under increasing magnitudes of blast pressure and impulse until failure was observed or the capacity of the

shock tube was exceeded. The columns were instrumented and monitored for displacements, strains, and pressures during testing. High speed videos of the tests were taken and were used to compliment the recorded data. The experimental results of the tests on the columns were presented in detail, and comparisons of responses across all column groups were provided.

Analysis of the results was conducted by modeling the dynamic systems as equivalent SDOF systems. This analysis procedure used a robust solution for the resistance-displacement response for all columns. These resistance curves were developed using direct integration of curvatures over the length of the column and accounted for the variation of axial load during response due to column flexure and the axial force component of any bracing element. CFRP retrofitted columns were analyzed by incorporating the effects of the retrofit material as tension force components in the sectional analysis or by transverse reinforcement for modification of the material properties of concrete due to their effects on confinement. The actual pressure-time histories from the experimental data were used to load the columns in the numerical analysis. The results of the numerical analysis were used in expanding on conclusions and observations made from the experimental program.

The analytical research also included the development and/or verification of design procedures based on the observations from this study and from current design standards. Design guidelines were devised for each retrofit technique developed/researched. These procedures and guidelines are intended to be used by researchers and practicing designers to better apply sound engineering principles in blast resistant column design.

7.3. Conclusions

The following conclusions may be drawn from the material presented in this study:

- Flexure-dominant reinforced concrete columns, with longitudinal reinforcement ratios typically used in practice, develop failures that initiate with compression crushing of concrete, followed by compression bar buckling unless closely spaced ties are used, when subjected to blast loads in excess of their flexural capacity.
 - The as-built (control) flexure dominant reinforced concrete columns in this study developed crushing of concrete on the compression face at the critical mid-height location when subject to blast pressures in excess of their flexural capacity. The columns that were detailed with internal hoops spaced at $h/4$ (1S-C and 2S-C) along

the height did not suffer from buckling of compression reinforcement unlike the columns with spacing at $h/2$ (1NS-C and 2NS-C). Prior to significant damage occurring, the columns in the seismic and non-seismic groups performed comparably. For the two columns with non-square cross-sections (1NS-C and 1S-C with dimensions 100mm x 150mm x 1980mm bent about the weak axis), significant damage occurred on the third test where maximum displacements were 171.1mm and 156.0mm with residual displacements of 126.0mm and 128.9mm for the non-seismic and seismic columns respectively. For the large cross-section columns (2NS-C and 2S-C with dimensions 150mm x 150mm x 1980mm) the damaging test resulted in maximum displacements of 158.0mm and 129.0mm with residuals of 136.5mm and 98.8mm for the non-seismic and seismic columns respectively. It was observed that the non-seismic columns had shorter plastic hinge lengths with less finely distributed fan cracking over these lengths than the seismic columns.

- Flexure dominant reinforced concrete columns with height to section depth ratios of about 13 develop support rotations in excess of 7% at or above the compression crushing of concrete, depending on the level of blast pressure.
 - For the control columns, the maximum support rotations varied from 7.4° to 9.8° on the failure test with compression controlled failure characterized by crushing of the concrete into the core of the column. Only the control non-seismic column in the third group (3NS-C with $s=h/2$ and 150mm x 150mm x 1980mm dimensions) had buckling of compression reinforcement. The failure tests on the columns had pressures ranging from 65kPa to 81kPa. The failure mode was observed to change to the rupturing of tension reinforcement when the applied pressure exceeded the flexural capacity significantly, resulting in blowout.
- Concrete confinement provided by seismic detailing may not result in significantly different response if the level of axial compression is reduced at peak lateral displacement. There was little difference observed between the response of seismic and non-seismic control specimens tested in the current investigation.
 - This was attributed to the axial load being lost during dynamic response resulting in beam bending behaviour in the inelastic response region and negating the majority of the effects of confinement on concrete material properties. The axial load was lost as the column ends pulled away from the supports. For the columns in this study,

this was found to occur at approximately 30 mm mid-height displacement. Up to this displacement, the axial load was reduced proportionally with lateral displacement.

- Significant reduction may occur in column axial load associated with gravity loading during blast response.
 - For the case of the columns in this study, the transition between high axial loads to zero axial load occurred prior to yielding. For columns with lower length to depth aspect ratios or fully fixed supports, this load transition would happen at higher displacement ductilities (within the inelastic range). This is due to the later having lower yield displacement. Yield displacement is an important design value for columns under blast loads where, in general, it would not be designed to be exceeded. The loss of axial load in real columns may not be a significant factor depending on the actual column geometry but this should be verified by the designer.
- The application of surface-bonded CFRP sheets as flexural reinforcement improves column behaviour.
 - The columns that were retrofitted with only longitudinal CFRP (2NS-R and 2S-R) for increasing strength performed better than their control companion columns (2NS-C and 2S-C) under similar loads for the tests when the FRP did not debond. For the penultimate tests prior to failure, these columns saw a reduction in support rotation between by between 33% and 39% under pressures of approximately 45kPa and impulses of 250kPa-ms. The FRP strain at the maximum location for the columns under this test was 0.5% for both columns. The third test, with pressures of 99kPa and 85kPa for 2NS-R and 2S-R, respectively, resulted in the FRP debonding from the concrete substrate on the unloaded side. The maximum strain in the FRP was found to be 1.1% for both columns at the point of debonding. This value is considerably higher than the maximum value specified by CSA S806-12 of 0.7%. After debonding, the columns both suffered considerable damage that was similar to the control column damage on the failure tests.
- CFRP jacketing of columns as a blast retrofit technology improves the behaviour of compression concrete at failure and may result in overall improvements in response.
 - There were four columns retrofitted with only transverse FRP jacketing. Two of the columns were from the first group (1NS-J and 1S-J) and two from the second group

(2NS-J and 2S-R). For these columns, the use of the FRP jacketing prevented all concrete crushing and removed all real variation between the seismic and non-seismic response. The columns in Group 1 did not see any reduction in displacement (either maximum or residual) over their control columns. This lack of difference was considered to be due to the slenderness of the column. The retrofitted columns in the second group had between 22% and 45% lower maximum displacements and between 53% and 72% lower residual displacements on the third test (the failure test for the control columns). This performance improvement was due to the complete prevention of concrete crushing in the retrofitted columns. The columns were tested under a higher pressure for a fourth test and the failure mode shifted from compression controlled to tension controlled with the rupture of longitudinal reinforcement. There was no damage to the transverse FRP at any stage in the response.

- Combined use of longitudinal CFRP sheets enclosed in CFRP jacketing is an effective blast retrofit strategy.
 - Columns 2NS-JR and 2S-JR were retrofitted with the combination of longitudinal and transverse FRP. These columns had the lowest displacements of all of the columns in Group 2 under similar tests. The transverse FRP in both columns was applied over top of the longitudinal FRP and enhanced the bond of the FRP to the concrete substrate. The FRP in both columns was able to strain to rupture at the critical mid-height location prior to bond failure occurring. The use of the FRP jacket served two functions in these columns: enhancing bond and preventing crushing of concrete. It was found that the combination of the two FRP layups was the most effective FRP retrofit.
- Lateral bracing of columns to develop restoring force response is a very effective technique for blast retrofit of concrete columns.
 - All of the columns retrofitted with bracing (either compression or tension) performed better than the columns retrofitted with FRP. In the case of the columns with bracing, the maximum shock tube capacity was reached prior to significant column damage occurring. Braced columns were able to withstand more than double the impulse on the non-braced columns (with and without FRP retrofit) along with pressures up to 85kPa over the area of the load transfer device. For these

columns, the maximum support rotation was limited to 6% for the compression brace column and 4.4% for the highest rotation on the tension braced columns. The columns retrofitted with bracing were able to survive with little damage blast pressures that resulted in complete blowout of control columns or FRP retrofitted columns.

- Tension bracing used as a blast retrofit technique should be implemented with as large of an eccentricity along the column height as practicable to increase the horizontal force component.
 - The column with spacers used to increase the horizontal component of the tension brace force (3NS-TB2) performed better than the other tension braced columns. This column had approximately 60% of the displacement of the other two tension braced columns under similar loads. This indicates that the larger aspect ratio simulated with the brace spacers leads to better performance.
- When bracing is used as a blast-retrofit method, the effects of the axial force components should be accounted for in design.
 - The compression braced column put considerable axial load demand onto the supports of the test setup. This axial load would be uplift in a real structure. Actual use of compression braced columns in real structures relies on confidence in resisting this uplift force due to high structure mass and high degree of structural indeterminacy. Tension braced columns should be designed to withstand the considerable temporary axial forces from the vertical brace component. This may necessitate an additional retrofit such as an FRP jacket, to improve column performance under high axial compression.
- Tension bracing is more effective in controlling lateral drift and strengthening the member, while also providing a less labour intensive solution than the compression bracing.
- The design of brace components should consider all brace elements and promote ductile failure of braces both in tension and compression.
- The tension brace system is an innovative and highly effective retrofit technique for reinforced concrete columns under blast loads. It is simple to construct, uses parts that can be universal to any column, is simple to install, and is non-intrusive in the structure.
- Movement of the shock tube during testing had a significant effect on the accuracy of the experimentally recorded displacement-time histories.

- The shock tube movement initially registers as a negative displacement on the displacement graphs followed by a positive displacement that tends to be approximately in phase with the actual column displacement. This results in an over prediction of displacement of column response. The higher the pressure and impulse in the shock tube, the more movement the shock tube had. For the high pressure, high impulse braced columns that had small maximum displacements, this had the most effect on the accuracy of the experimental displacement recordings. With this movement and the presence of overly flexible displacement gauge mounts for the braced columns, it was surmised that upwards of 30% recording error may have been present in the experimental values. This error was likely considerably less than 10% for the majority of the non-braced columns.
- An equivalent single-degree-of-freedom (SDOF) modeling approach can be used to predict the response of columns with and without retrofit. The accuracy of this model depends on the level of detail considered in the construction of the resistance curves and the load-mass factors for conversion to the SDOF system.
 - The comparisons of experimental and analytical response, conducted in the current research program, shows that a reasonable accuracy can be achieved through the use of SDOF dynamic inelastic blast analysis. The average ratio of experimental displacements to predicted displacements and mid-span over the 62 tests reported in this thesis was 0.93 with a standard deviation of 0.42. The small displacement responses (less than 10mm) had the highest standard deviation (0.76) for the 14 tests in that sample. This is due to measuring and testing errors with shock tube movement, gauge movement, and gaps between the load transferring device and the specimen having high influence on the recorded results. Also, since the ratio of displacements is reported, a small relative discrepancy between predicted and experimental values results in a large change in the displacement ratio. The columns with displacements between 10mm and 40mm as well as 40mm and 80mm had higher accuracy and consistency in displacement predictions with average displacement ratios of 0.91 and 0.76 and standard deviations of 0.22 and 0.16 respectively. The columns with large displacements were fairly accurately predicted with average displacement ratios of 0.89 and a standard deviation of 0.21.

- The effects of confinement on concrete and compression reinforcing steel properties must be considered in the development of resistance curves.
- Axial load variation in resistance curves can be modeled by piecing together a series of resistance curves developed under different axial load levels.
 - This composite curve shifts from one axial load level to another depending on vertical displacement of the column ends. The composite curve has a non-obvious yield point as the curve shifts from different axial load levels with different stiffness. The yield point may not coincide with the onset of non-linearity of the resistance curve. The ultimate response of the curve is pure beam bending which tends to have high displacement ductility capacity for flexure dominant columns.
- Design procedures were developed for all the blast retrofit procedures presented in this thesis. These procedures can be used to design blast resistant columns.
 - The procedures for the use of FRP expanded on existing standards in CSA S806-12 as well as in CSA S850-12 and the knowledge available in the literature. These procedures allow the user to design the required number of layers of FRP, select ultimate FRP strain limits, and design the FRP with the appropriate column ductility and strength behaviour taken into consideration. Distinct procedures have been developed for the design and implementation of compression brace and tension brace retrofits. These include design concepts about brace ductility, connection detailing and selection of compression or tension retrofits. In addition to these concepts, specific simplified design procedures are given for both compression and tension brace retrofits. These procedures allow a designer to quickly select brace area and geometry for any column under any blast threat level.

7.4. Recommendations for Future Research

Based on the research described in this thesis, the following areas are recommended for further study:

- Investigate the effectiveness of FRP jacketing to increase bond capacity of longitudinal FRP on square columns of varying dimensions.
- Investigate the use of transverse confinement to increase axial load carrying capacity for columns retrofitted with tension braces.

- Investigate the use of external compression chords in addition to the use of tension braces in order to reduce compression demand on the column.
- Experimentally validate alternate mechanisms of compression brace buckling prevention beyond transverse prestressing.
- Conduct experimental research program to determine ultimate strain limits for surface bonded FRP for reinforced concrete under high strain rates.

References

- Ahmed, E., Soubuz, H., & Sutan, N. (2011). Flexural performance of CFRP strengthened RC beams with different degrees of strengthening schemes. *International Journal of the Physical Sciences*, 2229-2238.
- Arduini, M., & Nanni, A. (1997). Behavior of Precracked RC Beams Strengthened with Carbon FRP Sheets. *Journal of Composites for Construction*, 63-70.
- ASTM. (2012). *ASTM A1008/A1008M - 12a Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low Alloy with Improved Formability, Solution Hardened, and Bake Hardenable*. Technical.
- ASTM. (2012). *ASTM A1020/A1020M-02 Standard Specifications for Steel Tubes, Carbon and Carbon Manganese, Fusion Welded, for Boiler, Superheater, Heat Exchanger and Condenser Applications*. ASTM.
- Bangash, M., & Bangash, T. (2006). *Explosion-Resistant Buildings*. New York: Springer Berlin Heidelberg.
- Beausejour, P. (2000). *Seismic retrofitting of existing concrete columns with prestressing strands*. Department of Civil Engineering. Ottawa: University of Ottawa.
- Bekaert. (2012). *Dramix 3D Material Product Sheet*. N.V. Bekaert S.A.
- Berger, J., Heffernan, P., & Wight, R. (2013). Blast testing of CFRP and SRP strengthened RC columns. In S. Syngellakis (Ed.), *Design Against Blast: Load definition and structural response* (pp. 189-198). Ashurst Lodge, Southampton, UK: WIT Press.
- Bonacci, J., & Maalej, M. (2000). Externally Bonded Fiber-Reinforced Polymer for Rehabilitation of Corrosion Damaged Concrete Beams. *ACI Structural Journal*, 703-711.
- Buchan, P., & Chen, J. (2007). Blast resistance of FRP composites and polymer strengthened concrete and masonry structures - A state-of-the-art review. *Composites Part B: Engineering*, 509-522.

- Burrell, R. (2012). *Performance of Steel Fibre Reinforced Concrete Columns under Shock Tube Induced Shock Wave Loading*. Ottawa: University of Ottawa.
- Carroere, M., Wight, R., Heffernan, P., & Braimah, A. (2007). Effect of Blast Loads on Reinforced Concrete Strengthened with Steel Reinforced Polymer Sheets. *FRPRCS-8*. Patras, Greece.
- Chajes, M., Thomson Jr, T., Januszka, T., & Finch Jr, W. (1994). Flexural strengthening of concrete beams using externally bonded composite materials. *Construction and Building Materials*, 8(3), 191-201.
- Chiew, S., Sun, Q., & Yu, Y. (2007). Flexural Strength of RC Beams with GFRP Laminates. *Journal of Composites for Construction*, 497-506.
- Clough, R. W. (1966). *Effects of stiffness degradation on earthquake ductility requirement*. Struct. and Mat. Res. Berkeley: University of California.
- Crawford, J., Malvar, J., Morrill, K., & Ferritto, J. (2001). Composite Retrofits to Increase the Blast Resistance of Reinforced Concrete Buildings. *Tenth International Symposium on Interaction of the Effects of Munitions with Structures*. San Diego.
- CSA Standard A23.3. (2004). *Design of Concrete Structures*. Mississauga, Ontario, Canada: Canadian Standards Association.
- CSA Standard S806-12. (2012). *Design and Construction of Building Components with Fibre-Reinforced Polymers*. Mississauga, Ontario, Canada: Canadian Standards Association.
- CSA Standard S850-12. (2012). *Design and assessment of buildings subjected to blast loads*. Mississauga, Ontario, Canada: Canadian Standards Association.
- Davis, C., Williams, G., Williamson, E., Marchand, K., McKay, A., & Bayrak, O. (2009, April 30). Design and Detailing Guidelines for Bridge Columns Subjected to Blast and Other Extreme Loads. *Structures Congress 2009: Don't Mess with Structural Engineers*, pp. 2143-2152.
- Department of Defense. (2008). *Unified Facilities Code (UFC) 03-340-02: Structures to Resist the Effects of Accidental Explosions*. Washington, DC: United States Department of Defense.
- Fyfe Co. LLC. (2009). *Tyfo SCH-41A-1 Composite Product Data*. Fyfe Co.

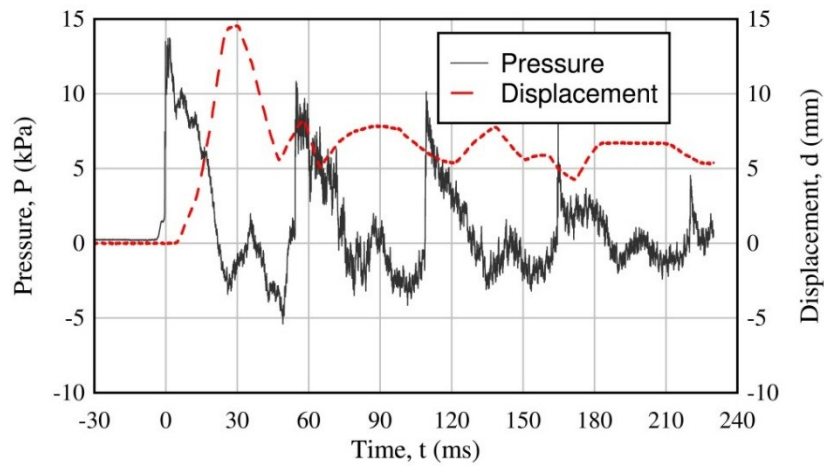
- Fyke Corporation. (2005). *Conventional Bolted Type Rupture Disc Holder Data Sheet*.
- Harding, J., & Welsh, L. M. (1983). A tensile testing technique for fibre-reinforced composites at impact rates of strain. *Journal of Materials Science*, 18(6), 1810-1826.
- Hognestad, E. (1951). Study of combined bending and axial load in reinforced concrete members. *University of Illinois - Engineering Experiment Station - Bulletin Series*, 399, 128p.
- Hyde, D. (1992). *conWep - Application of TM 5-1300*. Vicksburg: US Army Engineer Waterways Experiment Station.
- Jacques, E. (2011). *Blast Retrofit of Reinforced Concrete Walls and Slabs*. Ottawa: University of Ottawa.
- Jacques, E. (2014, December 14). *RCBLAST*. Retrieved from rcblast.ca
- Jacques, E., Lloyd, A., & Saatcuiglu, M. (2013). Predicting reinforced concrete response to blast loads. *Canadian Journal of Civil Engineering*, 40(5), 427-444.
- Kotynia, R., Baky, H., Neale, K., & Ebead, U. (2008). Flexural Strengthening of RC Beams with Externally Bonded CFRP Systems: Test Results and 3D Nonlinear FE Analysis. *Journal of Composites for Construcion*, 190-201.
- Lee, S., & Moy, S. (2007). A Method for Predicting the Flexural Strength of RC Beams Strengthened with Carbon Fiber Reinforced Polymer. *Journal of Reinforced Plastics and Composites*, 26(14), 1383-1401.
- Lloyd, A. (2010). *Performance of reinforced concrete columns under shock tube induced shock wave loading*. Ottawa: University of Ottawa.
- Lloyd, A., Jacques, E., Saatcioglu, M., Palermo, D., Nistor, I., & Tikka, T. (2010). Capabilities and Effectiveness of using a Shock Tube to Simulate Blast Loading on Structures and Structural Components. *ACI Special Publication on Blast*.
- Maalej, M., & Leong, K. (2005). Effect of beam size and FRP thickness on interfacial shear stress concentration and failure mode of FRP-strengthened beams. *Composites Science and Technology*, 1148-1158.

- Magnusson, J. (2007). *Structural Concrete Elements Subjected to Air Blast Loading*. Stockholm: Division of Concrete Structures, Royal Institute of Technology.
- Mess, D. (1999). *Seismic retrofitting of concrete bridge columns by external prestressing*. Ottawa: University of Ottawa.
- Muszynski, L., & Purcell, M. (2003). Composite Reinforcement to Strengthen Existing Concrete Structures Against Air Blast. *Journal of Composites for Construction*, 93-97.
- Niagara Lasalle Corporation. (2012). *StressProof Steel Bars*. Product Material Properties Report, Hammond, IN.
- Niu, H., Vasquez, A., & Karbhari, V. (2006). Effect of material configuration on strengthening of concrete slabs by CFRP composites. *Composites Part B: Engineering*, 37, 213-226.
- NRCC. (2010). *National Building Code of Canada*. Ottawa: National Research Council Canada.
- Pan, J., Chung, T., & Leung, C. (2009). FRP Debonding from Concrete Beams under Various Load Uniformities. *Advances in Structural Engineering*, 12(6), 807-819.
- Pan, J., Leung, C., & Luo, M. (2010). Effect of multiple secondary cracks on FRP debonding from the substrate of reinforced concrete beams. *Construction and Building Materials*, 24, 2507-2516.
- Pantelides, C., Garfield, T., Richins, W., Larson, T., & Blakeley, J. (2012). Behavior of Concrete Panels Reinforced with Synthetic Fibers, Mild Steel and GFRP Composites Subjected to Blasts. *Structures Congress 2012*, pp. 23-34.
- Pham, H., & Al-Mahaidi, R. (2006). Prediction Models for Debonding Failure Loads of Carbon Fiber Reinforced Polymer Retrofitted Reinforced Concrete Beams. *Journal of Composites for Construction*, 48-59.
- Razaqpur, G., Tolba, A., & Contestabile, E. (2006). Blast loading response of reinforced concrete panels with externally bonded GFRP laminates. *Composites Part B: Engineering*, 535-546.
- Rodriguez-Nikl, T., Kobayashi, T., Oesterle, M., Lan, S., Morrillm, K., Hegemier, G., & Seible, F. (2009, April 30). Carbon Fiber Composite Jackets to Protect Reinforced Concrete Columns against Blast Damage. *Structures Congress 2009: Don't Mess with Structural Engineers*, pp. 2775-2783.

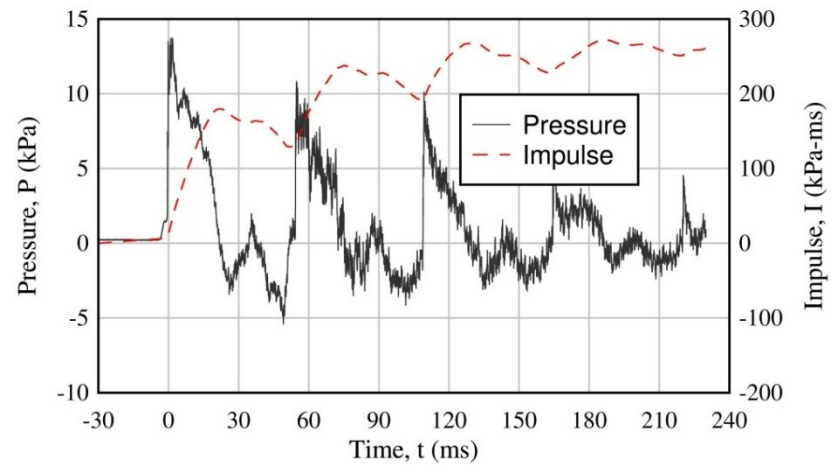
- Ross, C., Purcell, M., & Jerome, E. (1997). Blast response of concrete beams and slabs externally reinforced with fiber reinforced plastics (FRP). *Structures Congress - Proceedings: Building to Last*, (pp. 673-677).
- Saatcioglu, M. (2012). A displacement-based design procedure for FRP wrapping of seismically deficient concrete columns. *ACMBS-VI*. Kingston, ON.
- Saatcioglu, M. B. (1999). Circular high-strength concrete columns under simulated seismic loading. *Journal of Structural Engineering*, 272-280.
- Saatcioglu, M. R. (2002). Displacement based design of reinforced concrete columns for confinement. *ACI Structural Journal*, 90(1), 3-11.
- Saatcioglu, M., & Gira, M. (1999). Confinement of reinforced concrete columns with welded reinforcement grids. *ACI Structural Journal*, 96(1), 29-39.
- Saatcioglu, M., & Razvi, S. (1992). Strength and ductility of confined concrete. *Journal of Structural Engineering*, 118(6), 1590-1607.
- Saatcioglu, M., & Yalcin, C. (2003). External prestressing concrete columns for improved shear resistance. *Journal of Structural Engineering*, 129(8), 1057-1070.
- Saatcioglu, M., Chakrabarti, S., Selbuy, R., & Mes, D. (2002). Improving Ductility and Shear Capacity of Reinforced Concrete Columns with the Retro-Belt Retrofitting System. *Proceedings of the 7th U.S. National Conference on Earthquake Engineering*. Earthquake Engineering Research Institute.
- Sawyer, H. A. (1964). Design of concrete frames for two failure stages. *Proceedings International Symposium of the Flexural Mechanics of Reinforced Concrete*, SP-12, 405-431.
- Schenker, A., Anteby, I., Gal, E., Kivity, Y., Nizri, E., Sadot, O., . . . Ben-Dor, G. (2008). Full-scale field tests of concrete slabs subjected to blast loads. *International Journal of Impact Engineering*, 184-198.
- Seim, W. H., Karbhari, V., & Seible, F. (2001). External FRP Poststrengthening of Scaled Concrete Slabs. *Journal of Composites for Construction*, 67-75.

- Silva, P., & Lu, B. (2007). Improving the blast resistance capacity of RC slabs with innovative materials. *Composites: Part B*, 523-534.
- Smith, P. D., & Hetherington, J. G. (1994). *Blast and ballistic loading of structures*. Oxford, UK: Butterworth-Heinemann.
- Spadea, G., Bencardino, F., & Swamy, R. (1998). Structural Behavior of Composite RC Beams with Externally Bonded CFRP. *Journal of Composites for Construction*, 132-137.
- Toutlemonde, F., Rossi, P., Boulay, C., Gourraud, C., & Guedon, D. (1995). Dynamic behaviour of concrete: Tests of slabs with a shock tube. *Materials and Structures*, 293-298.
- Woodson, S., & Baylot, J. (1999). *Structural Collapse: Quarter-Scale Model Experiments*. Vicksburg: US Army Corps of Engineers.
- Wu, C., Oehlers, D., Rebentrost, M., Leach, J., & Whittaker, A. (2009). Blast testing of ultra-high performance fibre and FRP-retrofitted concrete slabs. *Engineering Solutions*, 2060-2069.
- Yalcin, C. (1998). *Seismic evaluation and retrofit of existing reinforced concrete bridge columns*. Department of Civil Engineering. Ottawa: University of Ottawa.
- Yalcin, C., & Saatcioglu, M. (2000). Inelastic analysis of reinforced concrete columns. *Computers and Structures*, 77, 539-555.

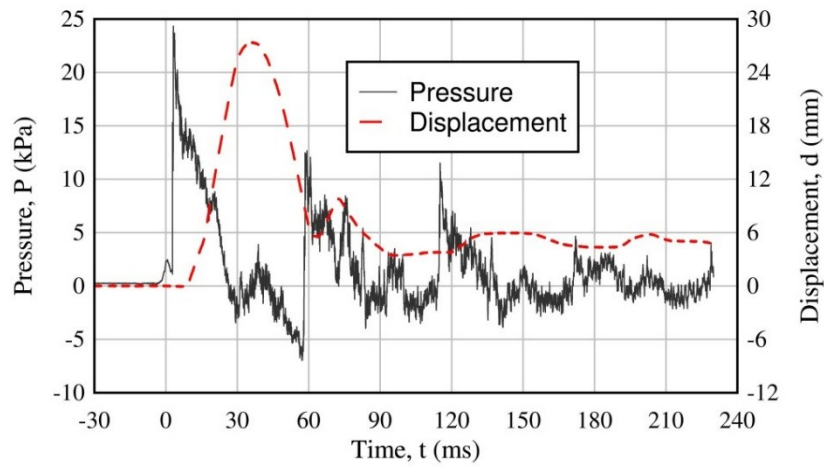
Appendix A - Pressure and Displacement Time Histories



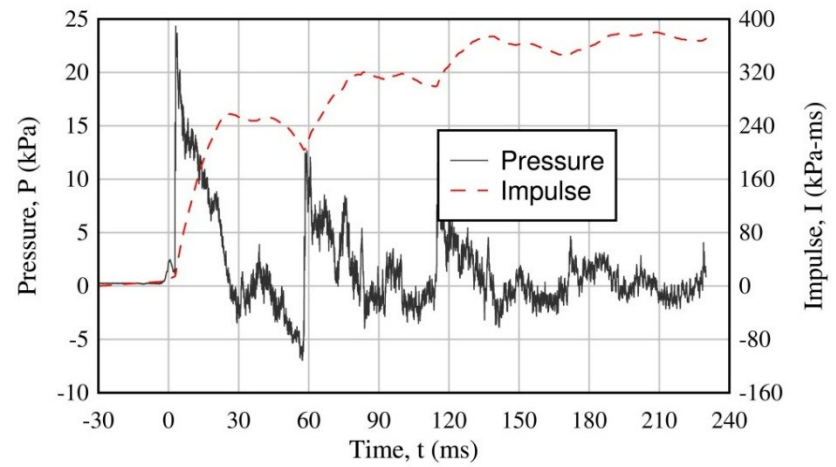
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

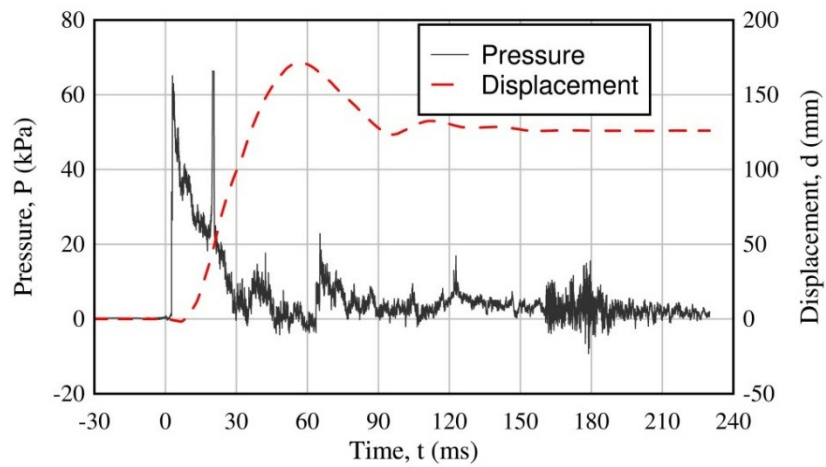


c) Shot 2 Pressure and Displacement

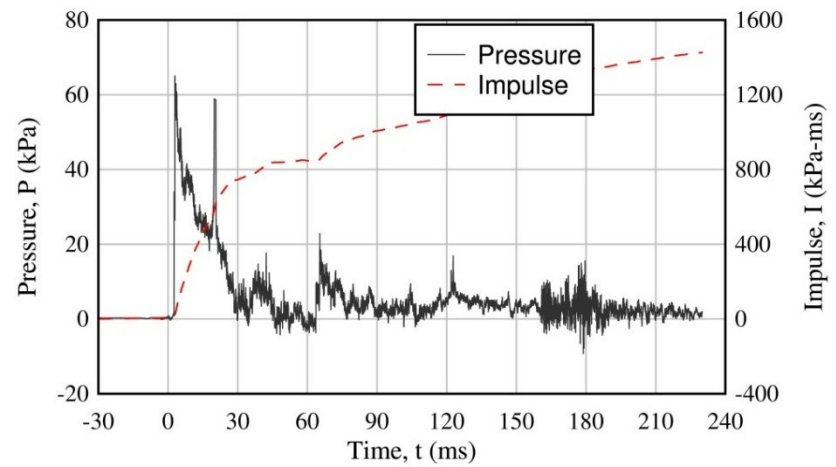


d) Shot 2 Pressure and Impulse

Figure A-1: Pressure, Impulse, and Displacement Time History for Column 1NS-C Shot 1 and Shot 2

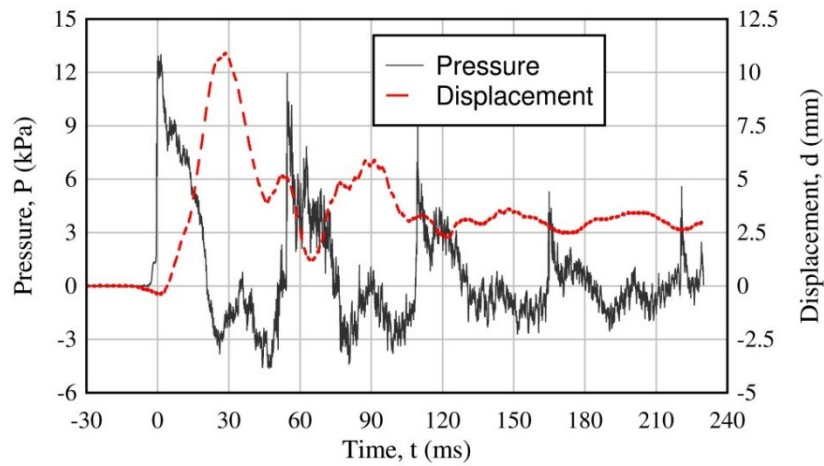


a) Shot 3 Pressure and Displacement

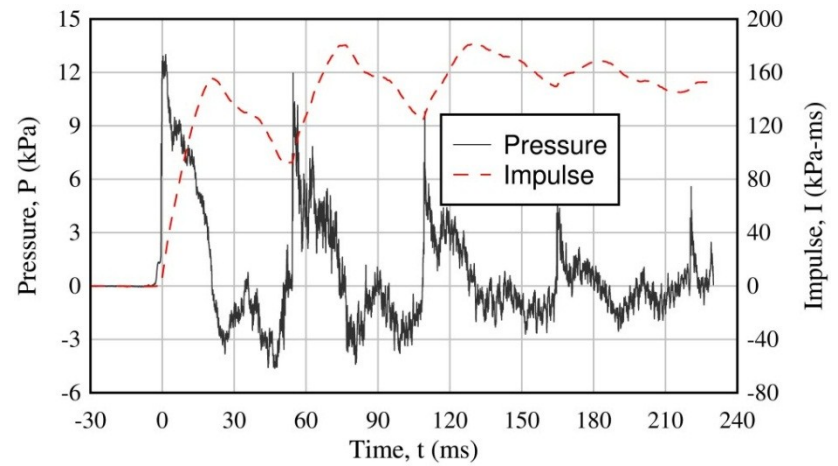


b) Shot 3 Pressure and Impulse

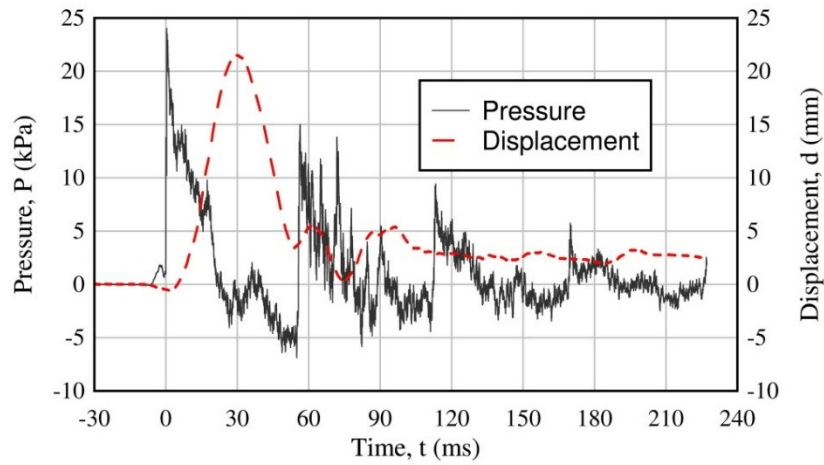
Figure A-2: Pressure, Impulse, and Displacement Time History for Column 1NS-C Shot 3



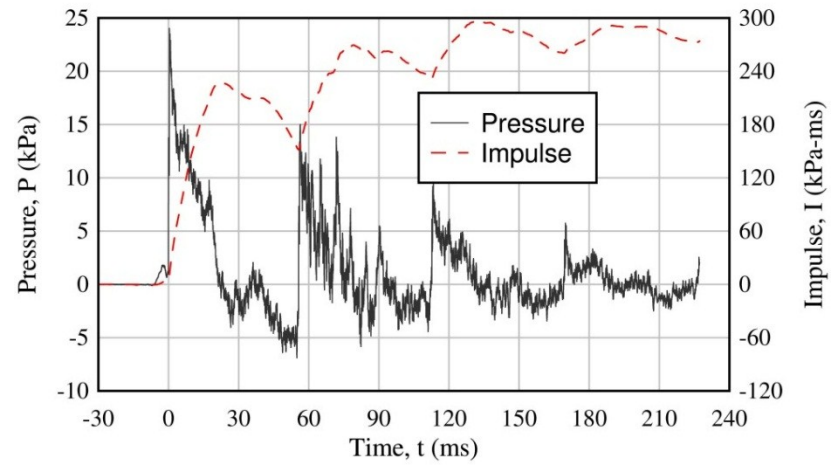
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

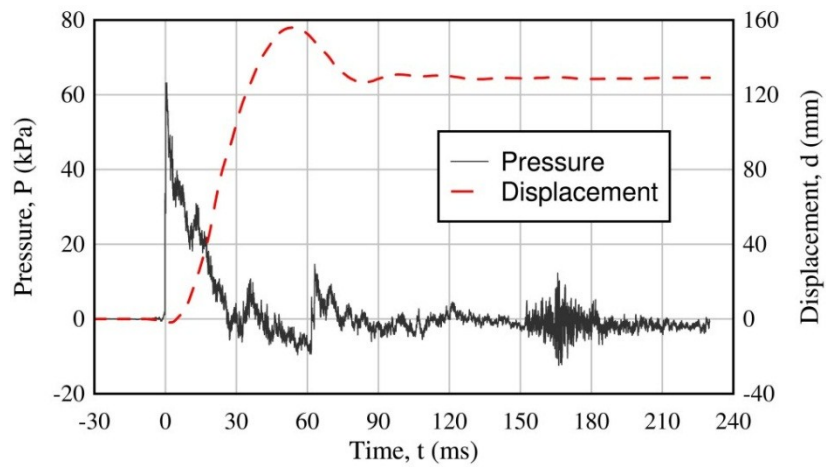


c) Shot 2 Pressure and Displacement

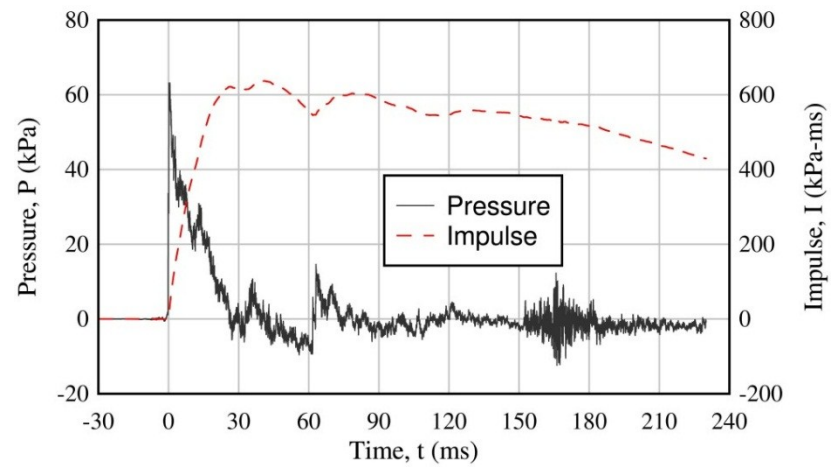


d) Shot 2 Pressure and Impulse

Figure A-3: Pressure, Impulse, and Displacement Time History for Column 1S-C Shot 1 and Shot 2

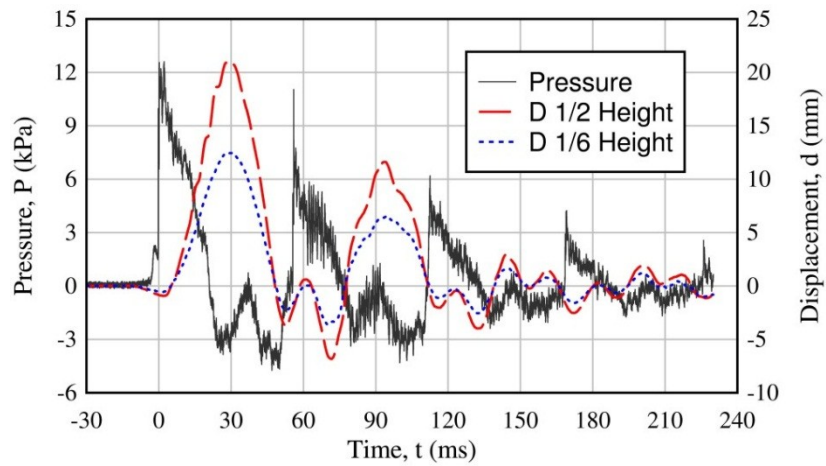


a) Shot 3 Pressure and Displacement

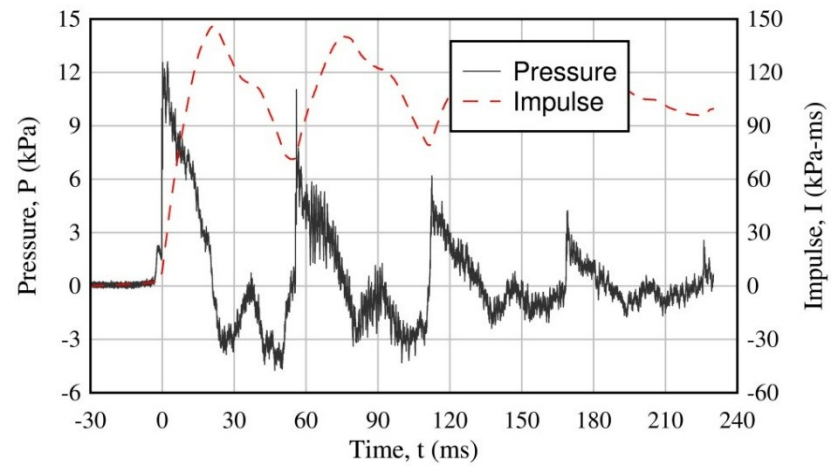


b) Shot 3 Pressure and Impulse

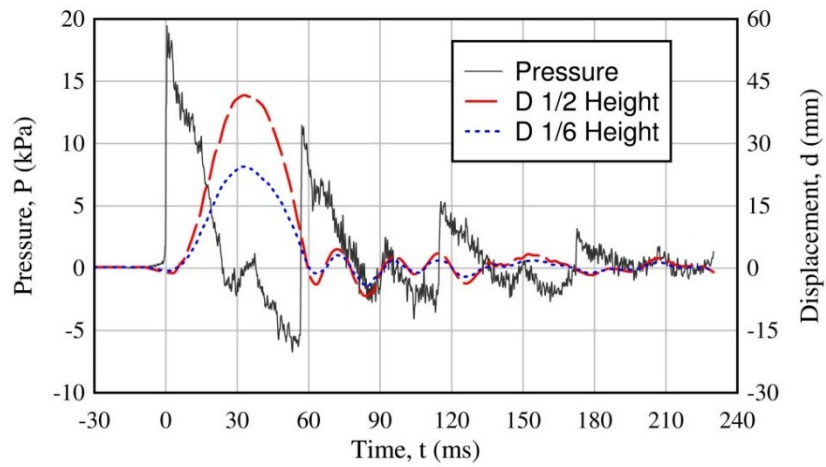
Figure A-4: Pressure, Impulse, and Displacement Time History for Column 1S-C Shot 3



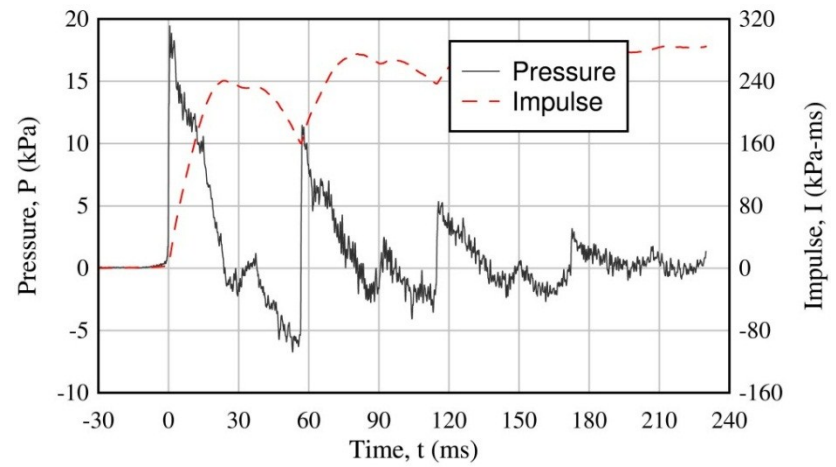
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

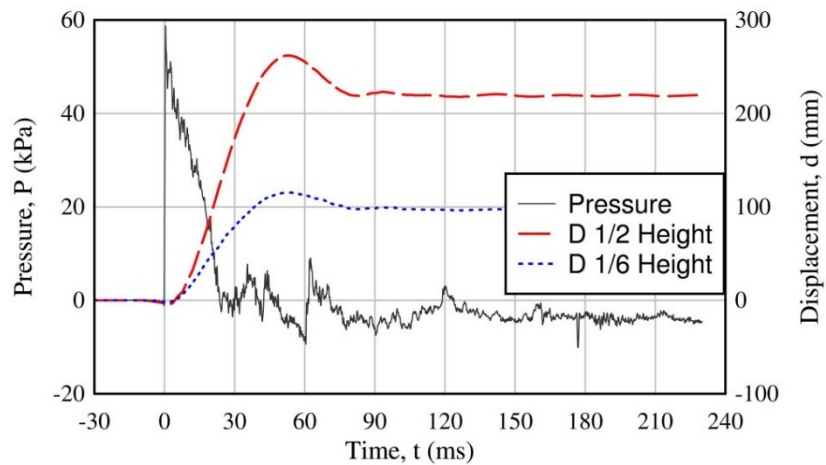


c) Shot 2 Pressure and Displacement

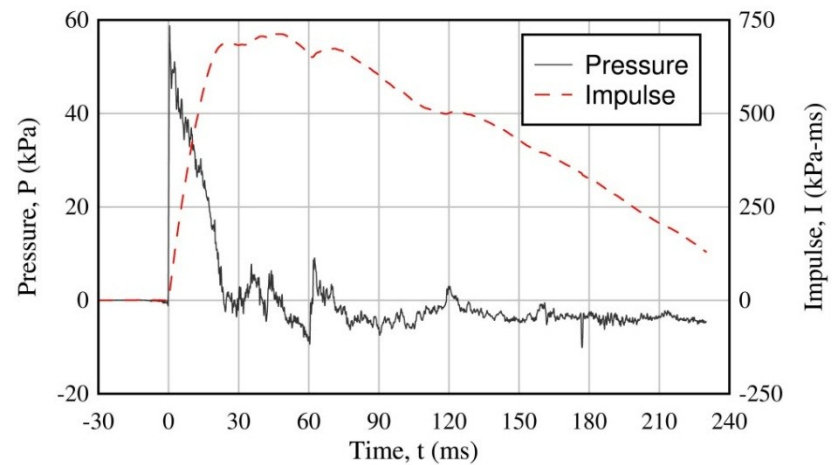


d) Shot 2 Pressure and Impulse

Figure A-5: Pressure, Impulse, and Displacement Time History for Column 1NS-J Shot 1 and Shot 2

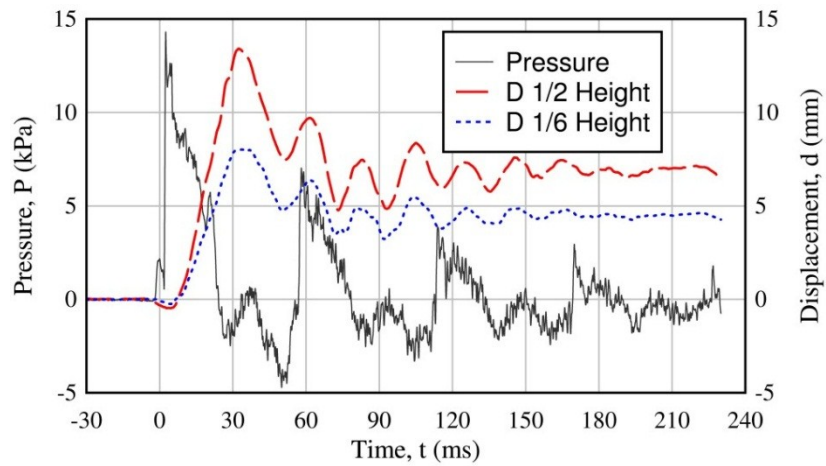


a) Shot 3 Pressure and Displacement

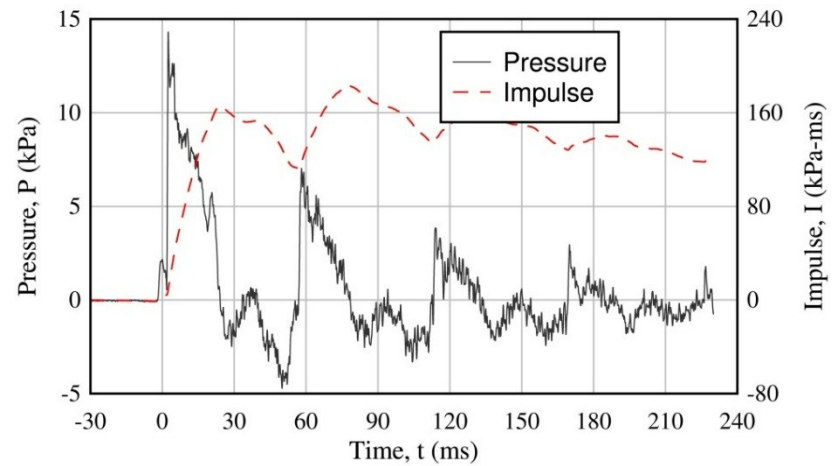


b) Shot 3 Pressure and Impulse

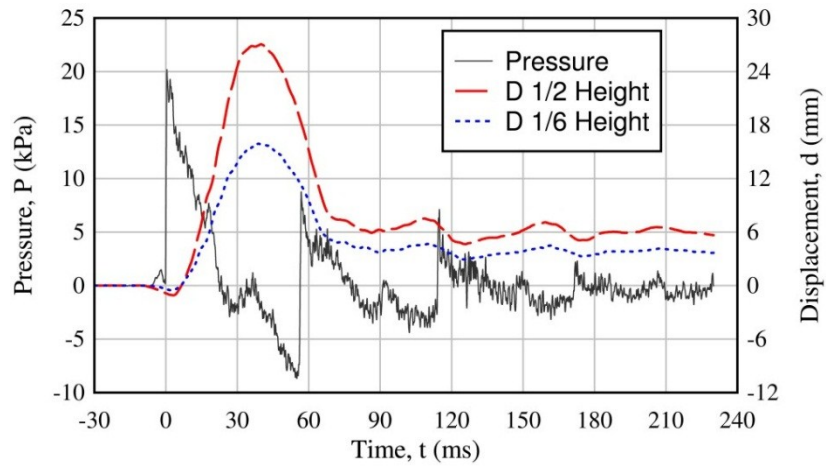
Figure A-6: Pressure, Impulse, and Displacement Time History for Column 1NS-J Shot 3



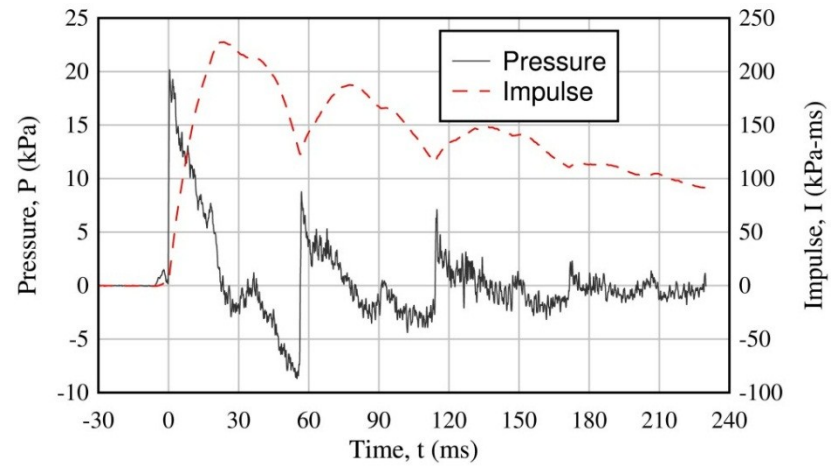
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

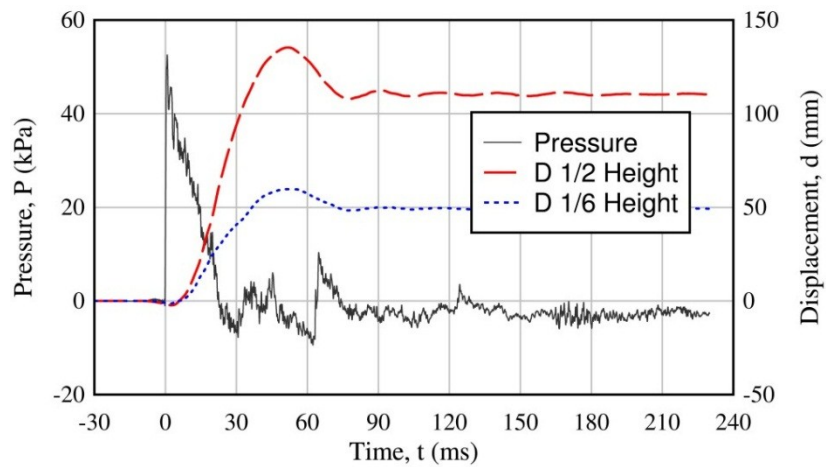


c) Shot 2 Pressure and Displacement

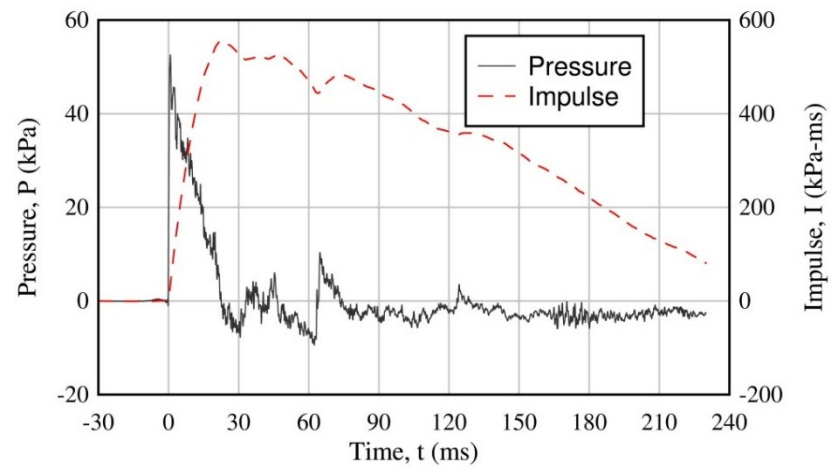


d) Shot 2 Pressure and Impulse

Figure A-7: Pressure, Impulse, and Displacement Time History for Column 1S-J Shot 1 and Shot 2

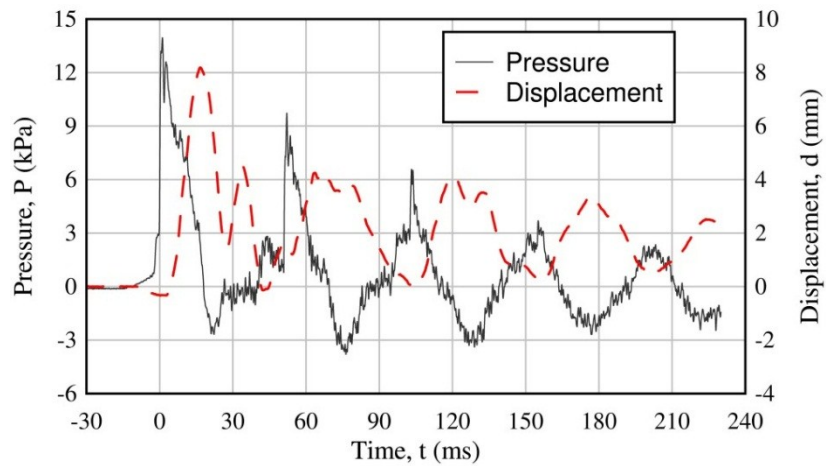


a) Shot 3 Pressure and Displacement

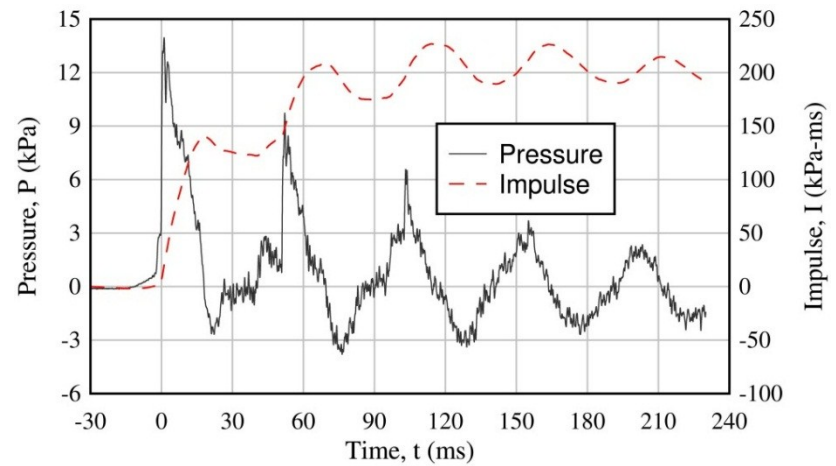


b) Shot 3 Pressure and Impulse

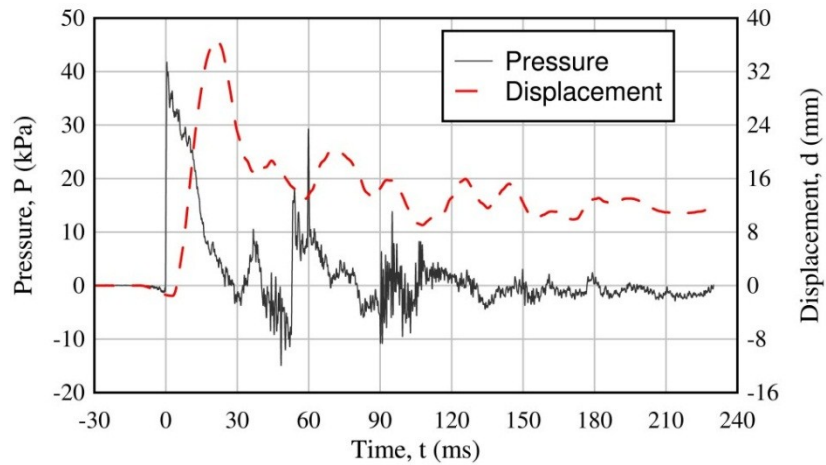
Figure A-8: Pressure, Impulse, and Displacement Time History for Column 1S-J Shot 3



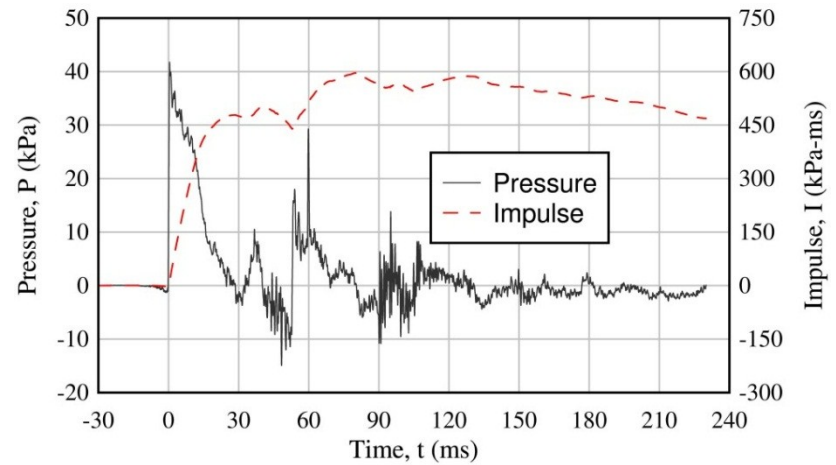
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

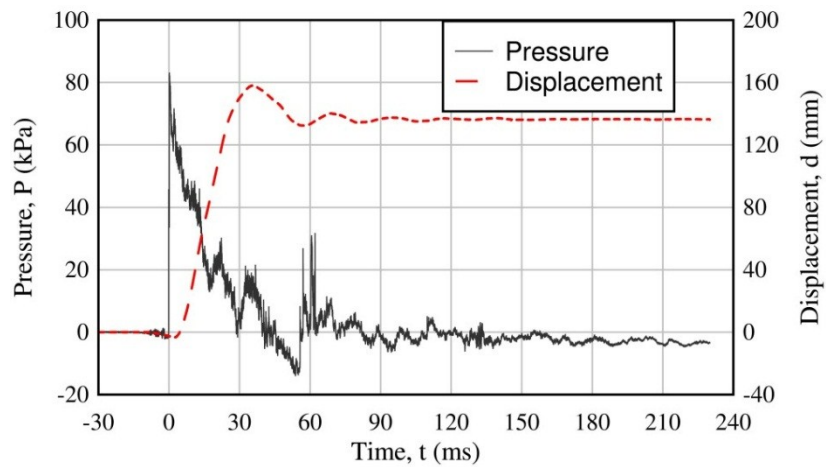


c) Shot 2 Pressure and Displacement

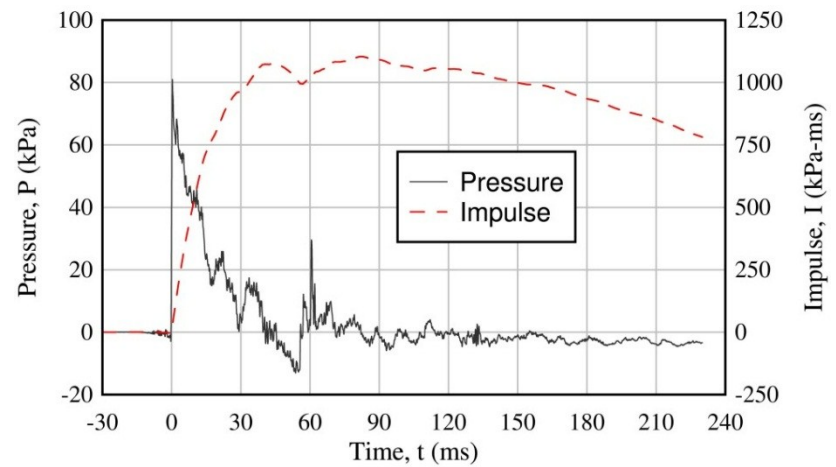


d) Shot 2 Pressure and Impulse

Figure A-9: Pressure, Impulse, and Displacement Time History for Column 2NS-C Shot 1 and Shot 2

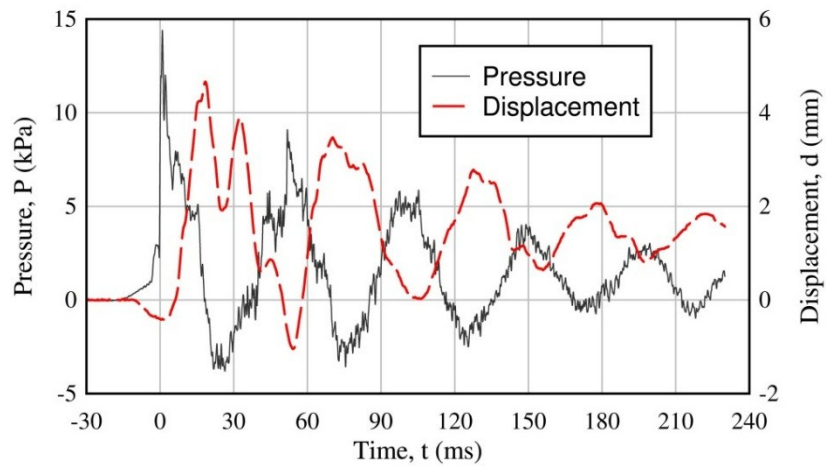


a) Shot 3 Pressure and Displacement

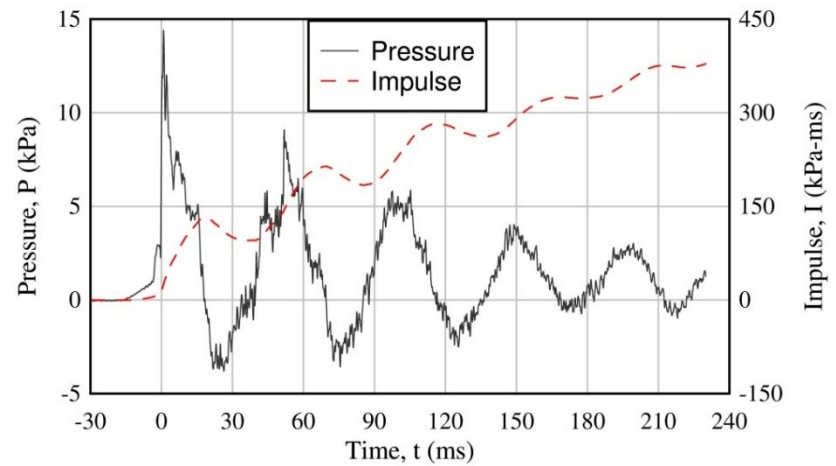


b) Shot 3 Pressure and Impulse

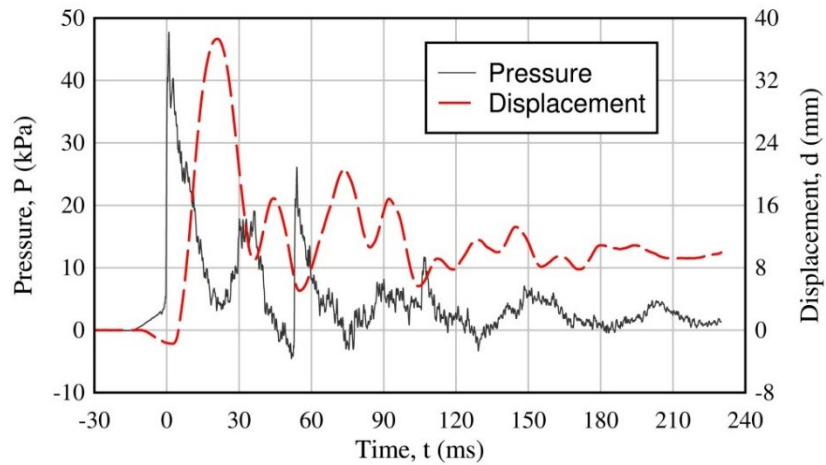
Figure A-10: Pressure, Impulse, and Displacement Time History for Column 2NS-C Shot 3



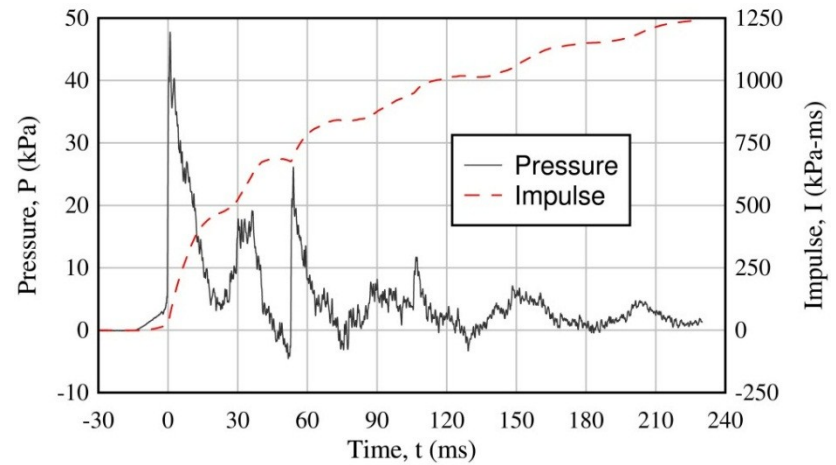
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

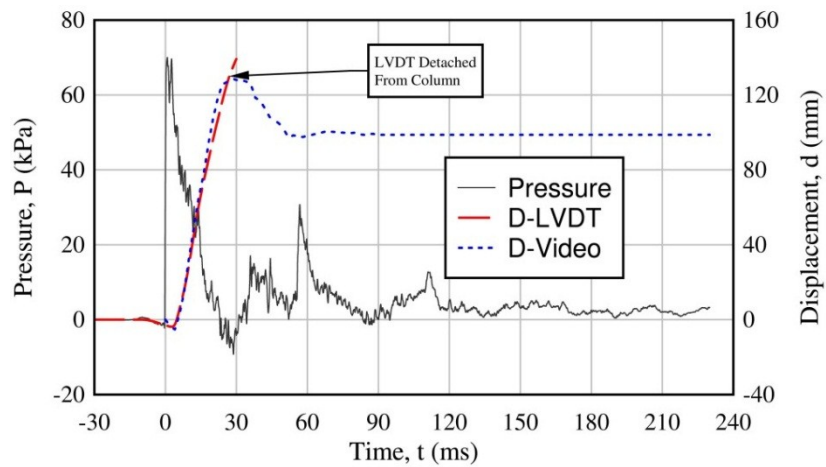


c) Shot 2 Pressure and Displacement

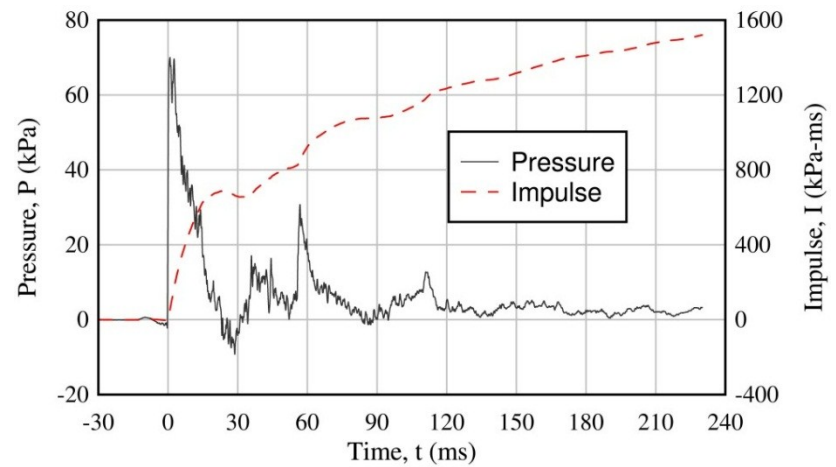


d) Shot 2 Pressure and Impulse

Figure A-11: Pressure, Impulse, and Displacement Time History for Column 2S-C Shot 1 and Shot 2

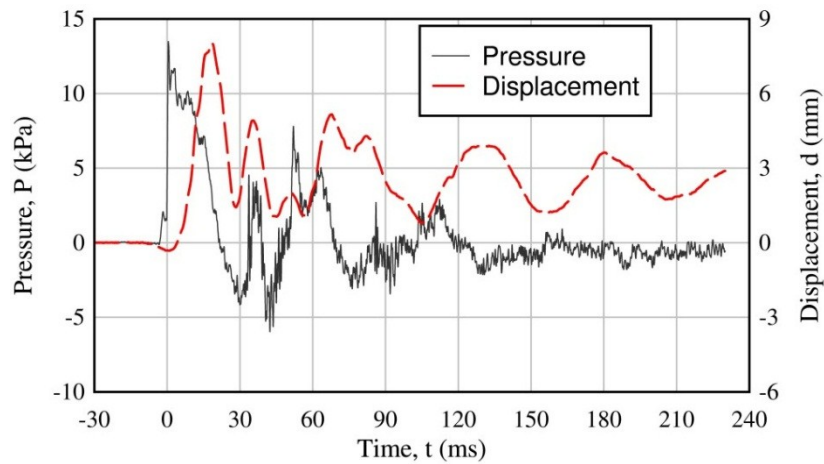


a) Shot 3 Pressure and Displacement

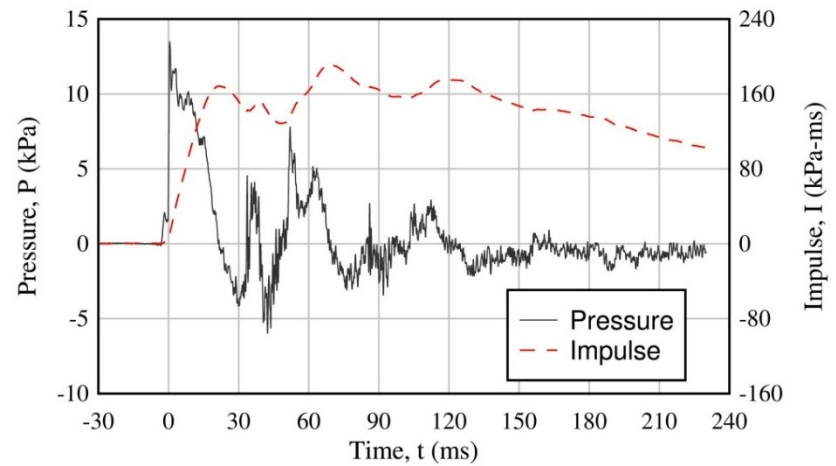


b) Shot 3 Pressure and Impulse

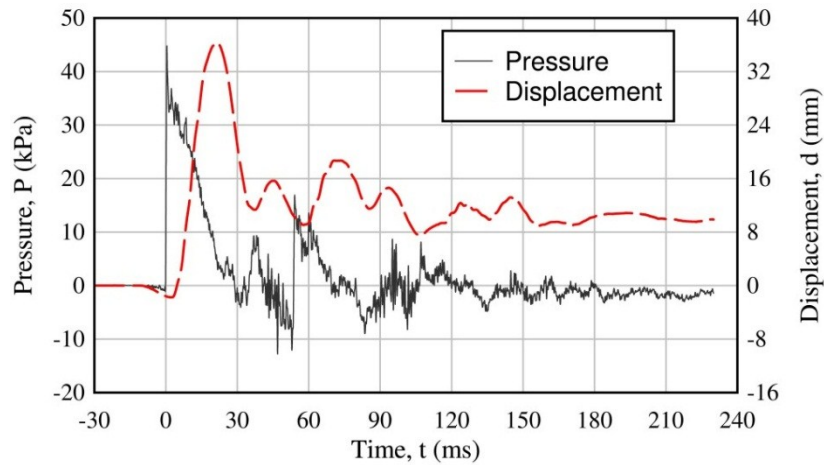
Figure A-12: Pressure, Impulse, and Displacement Time History for Column 2S-C Shot 3



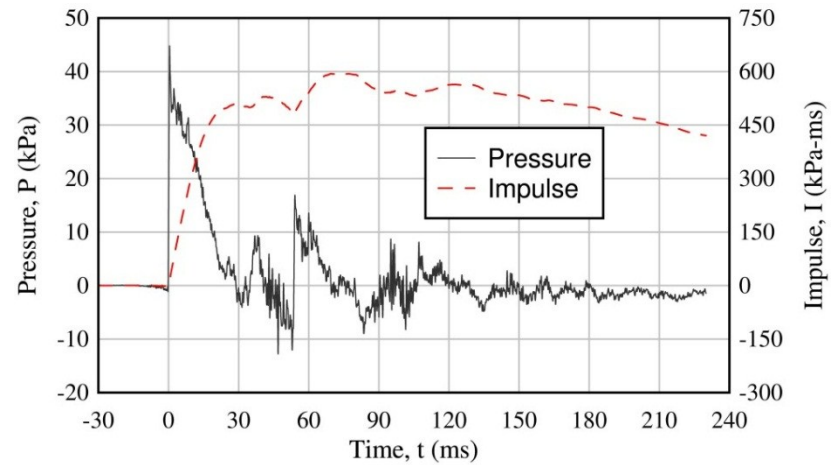
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

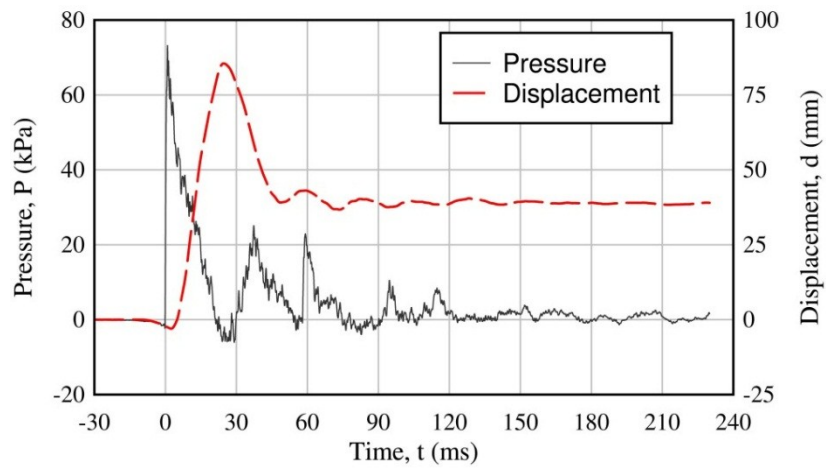


c) Shot 2 Pressure and Displacement

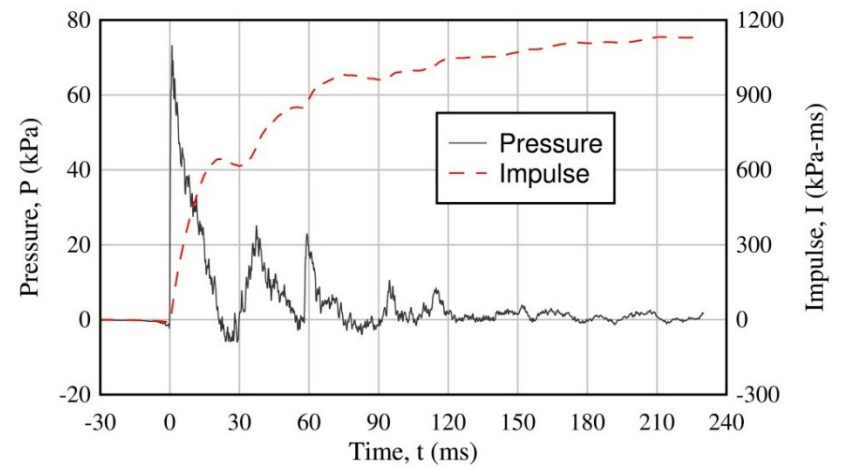


d) Shot 2 Pressure and Impulse

Figure A-13: Pressure, Impulse, and Displacement Time History for Column 2NS-J Shot 1 and Shot 2

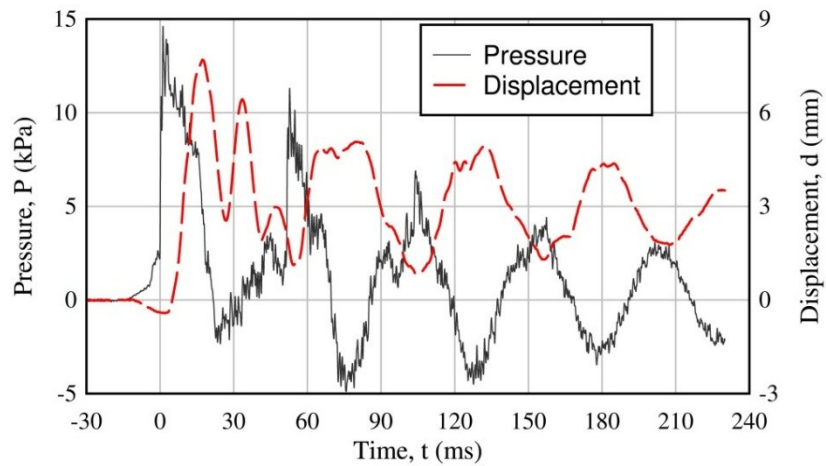


a) Shot 3 Pressure and Displacement

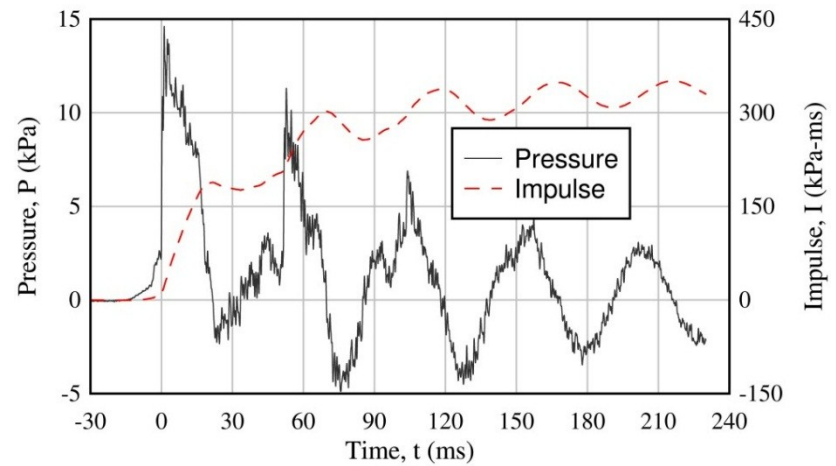


b) Shot 3 Pressure and Impulse

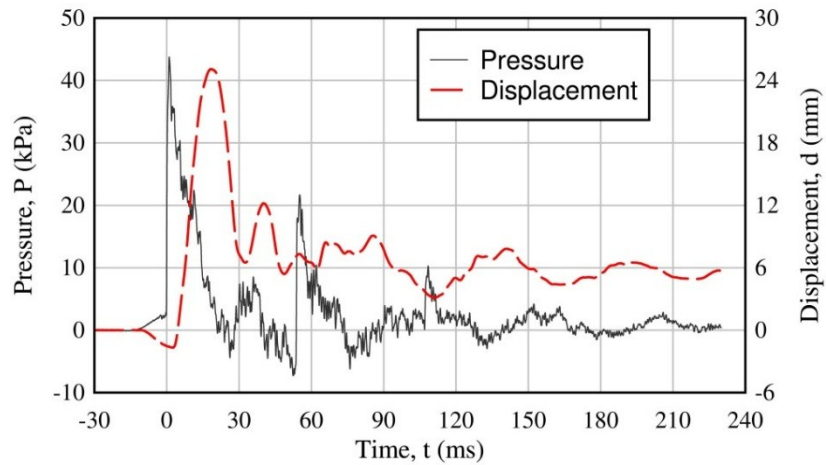
Figure A-14 : Pressure, Impulse, and Displacement Time History for Column 2NS-J Shot 3



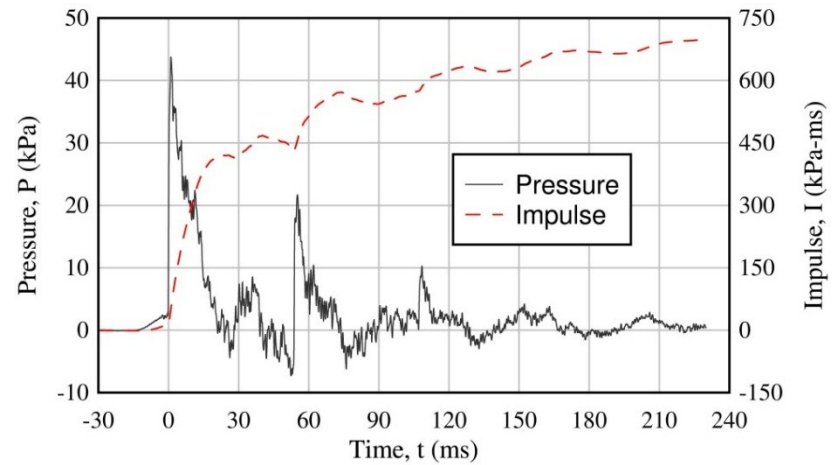
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

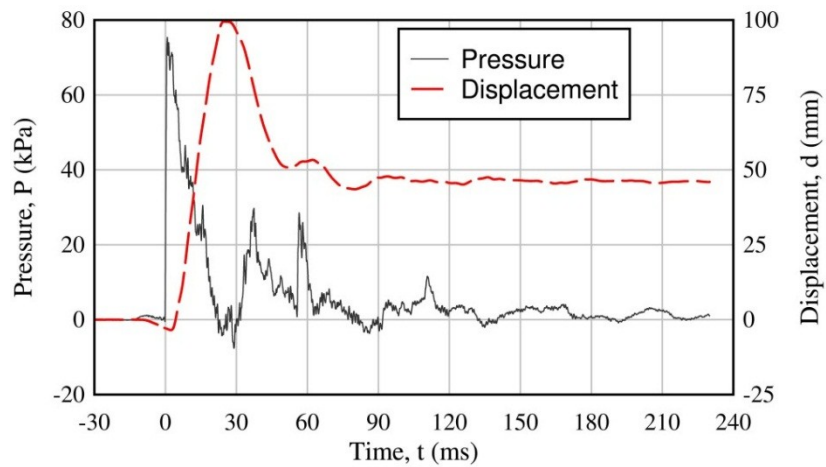


c) Shot 2 Pressure and Displacement

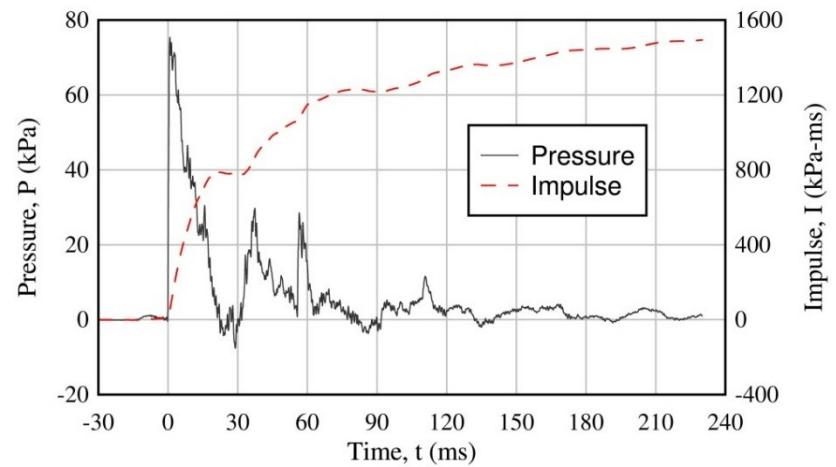


d) Shot 2 Pressure and Impulse

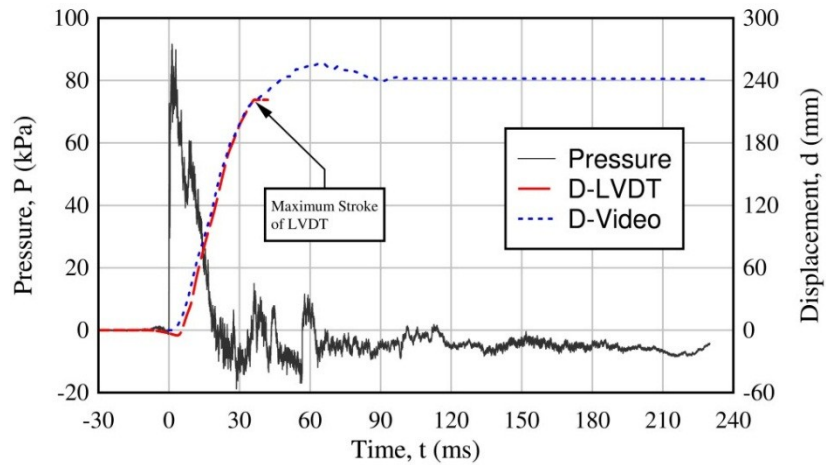
Figure A-15: Pressure, Impulse, and Displacement Time History for Column 2S-J Shot 1 and Shot 2



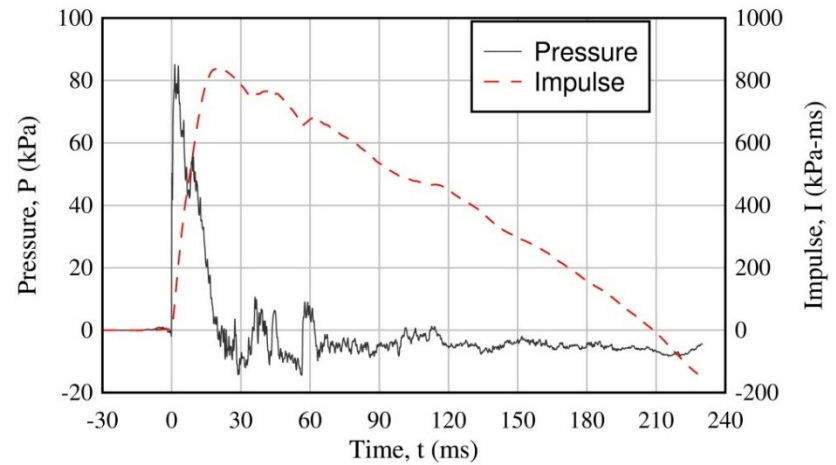
a) Shot 3 Pressure and Displacement



b) Shot 3 Pressure and Impulse

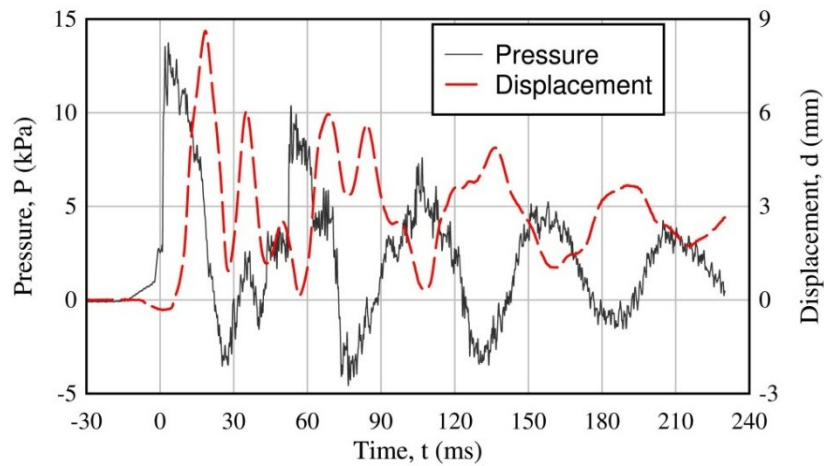


c) Shot 4 Pressure and Displacement

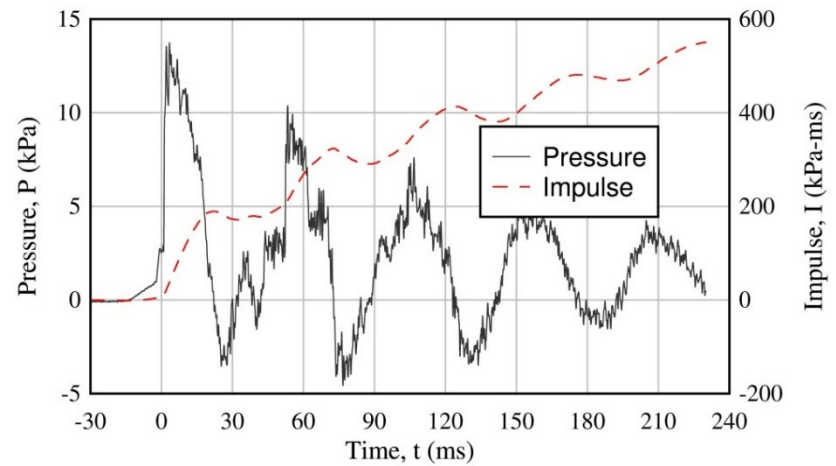


d) Shot 4 Pressure and Impulse

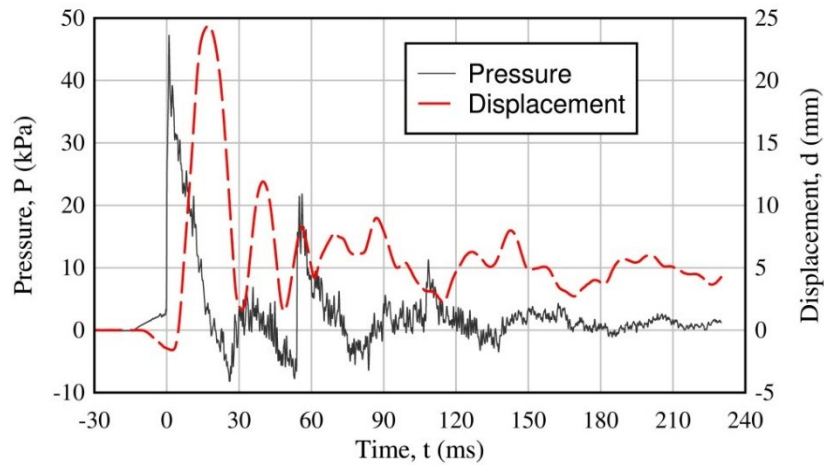
Figure A-16: Pressure, Impulse, and Displacement Time History for Column 2S-J Shot 3 and Shot 4



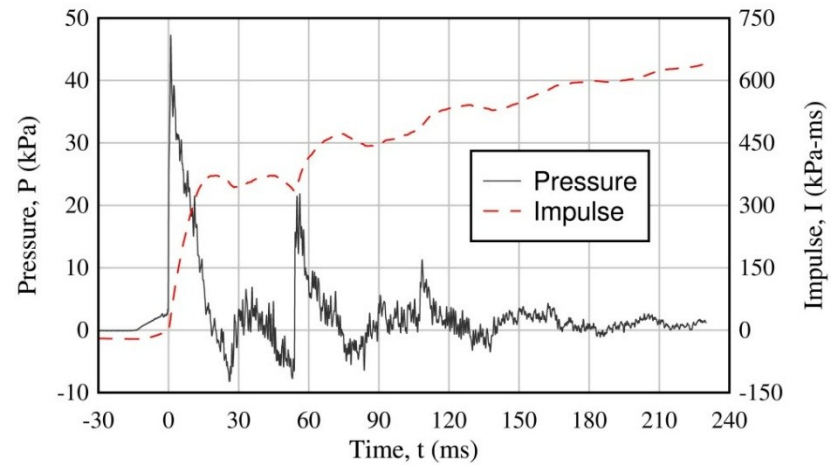
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

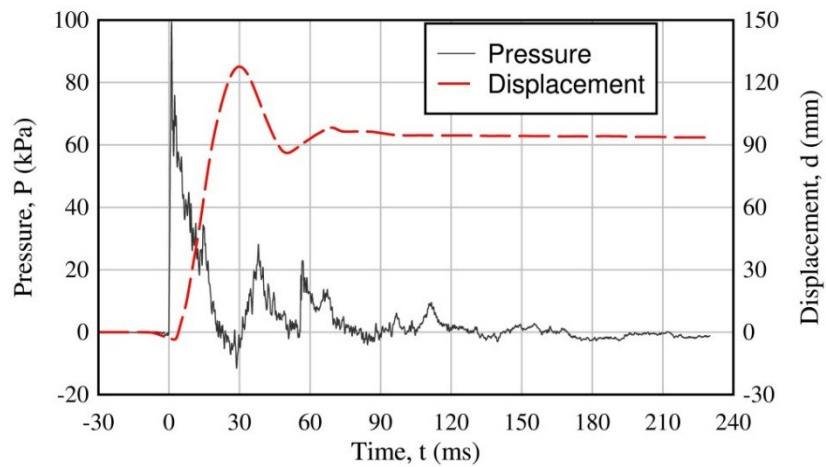


c) Shot 2 Pressure and Displacement

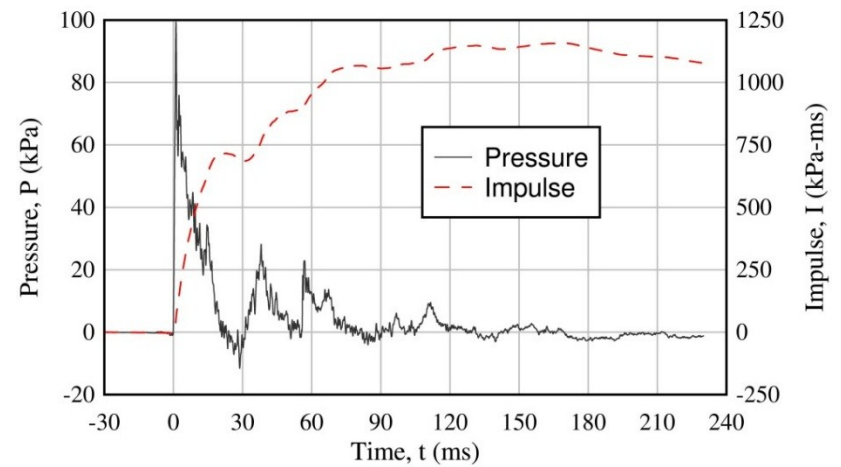


d) Shot 2 Pressure and Impulse

Figure A-17: Pressure, Impulse, and Displacement Time History for Column 2NS-R Shot 1 and Shot 2

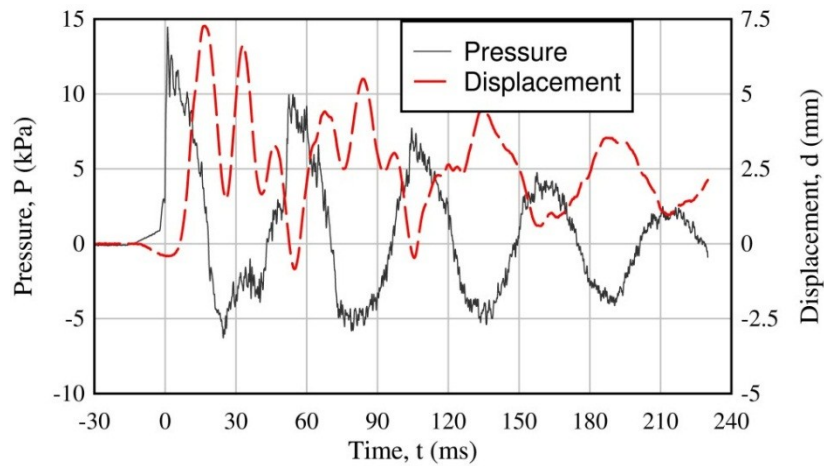


a) Shot 3 Pressure and Displacement

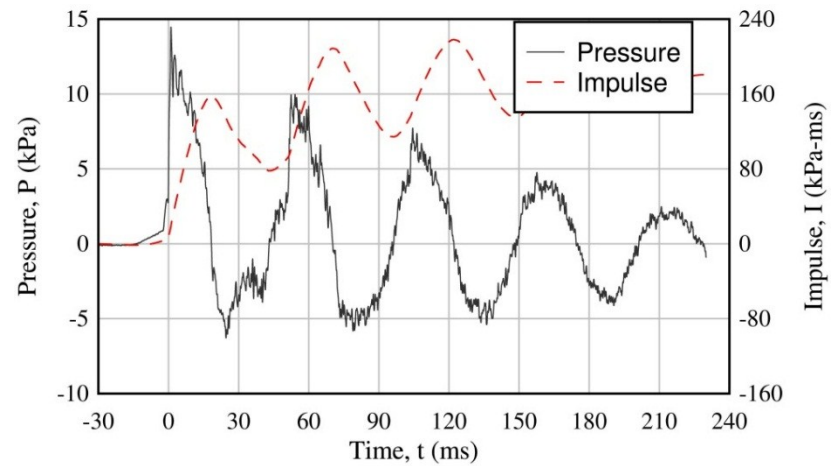


b) Shot 3 Pressure and Impulse

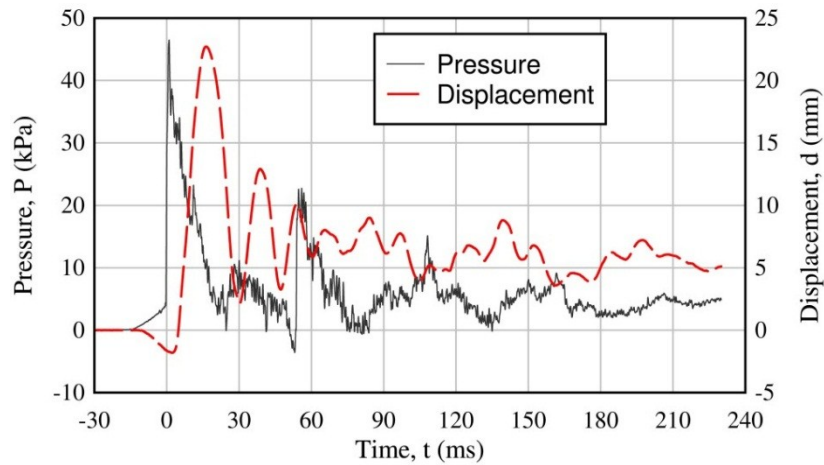
Figure A-18 : Pressure, Impulse, and Displacement Time History for Column 2NS-R Shot 3



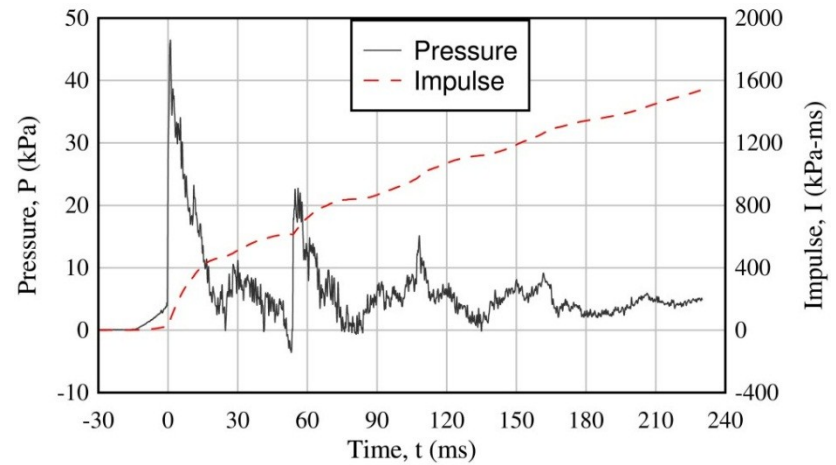
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

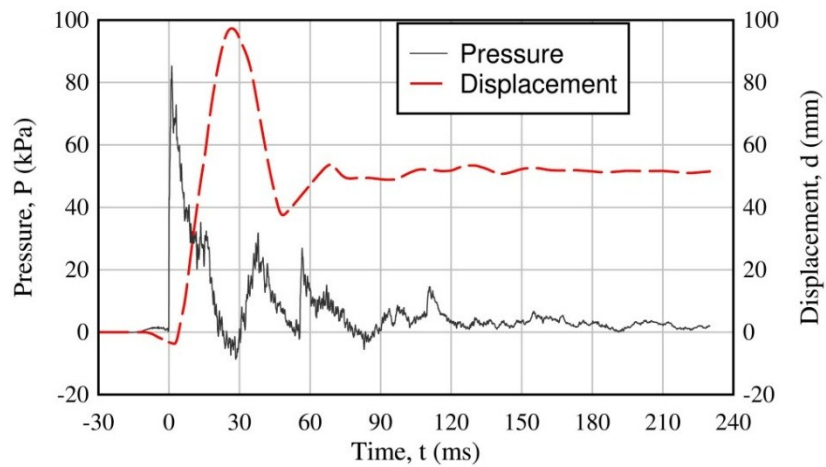


c) Shot 2 Pressure and Displacement

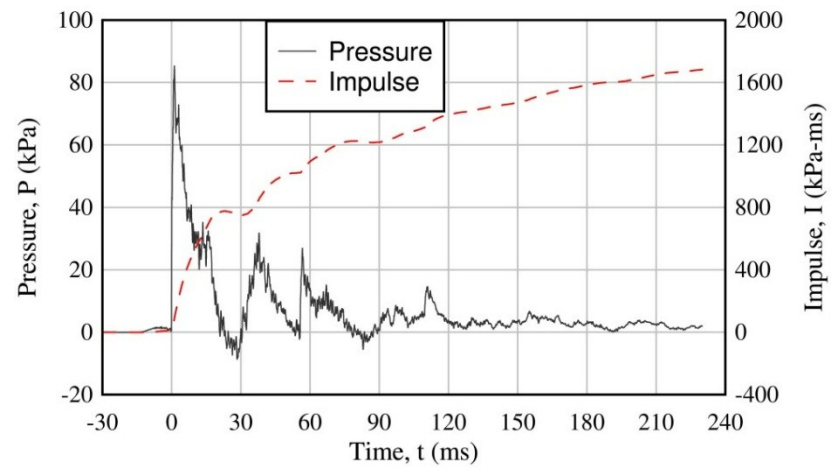


d) Shot 2 Pressure and Impulse

Figure A-19: Pressure, Impulse, and Displacement Time History for Column 2S-R Shot 1 and Shot 2

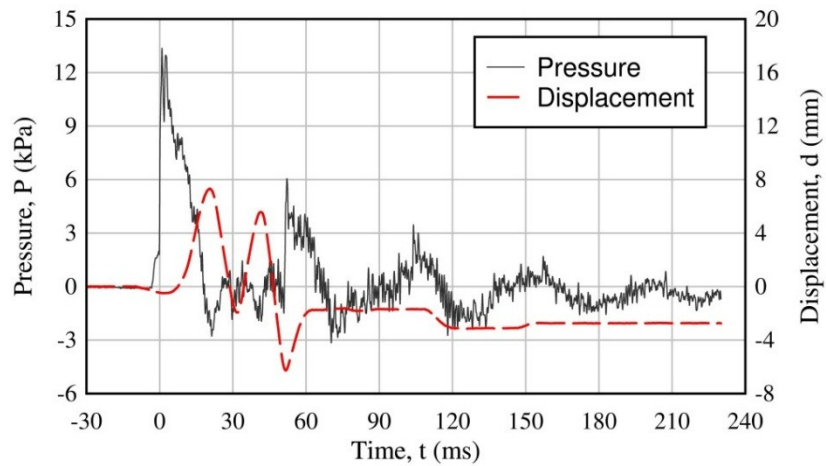


a) Shot 3 Pressure and Displacement

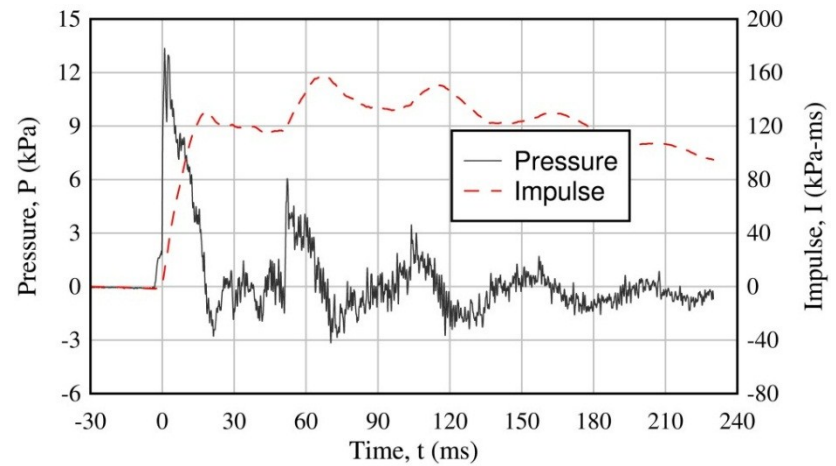


b) Shot 3 Pressure and Impulse

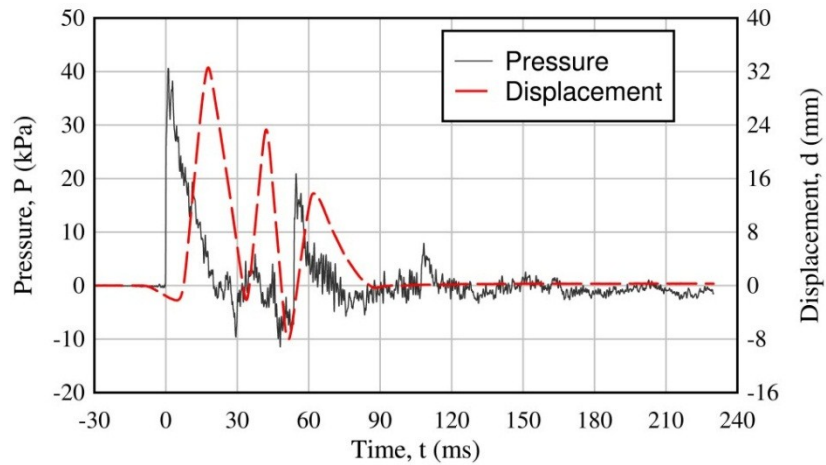
Figure A-20 : Pressure, Impulse, and Displacement Time History for Column 2S-R Shot 3



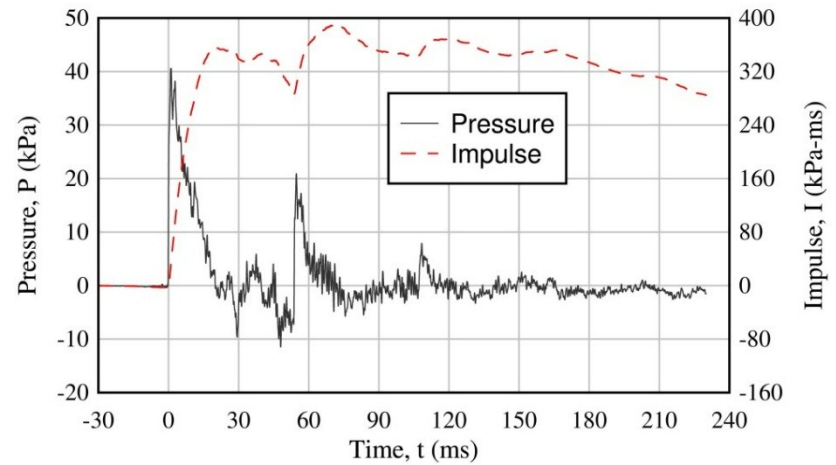
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

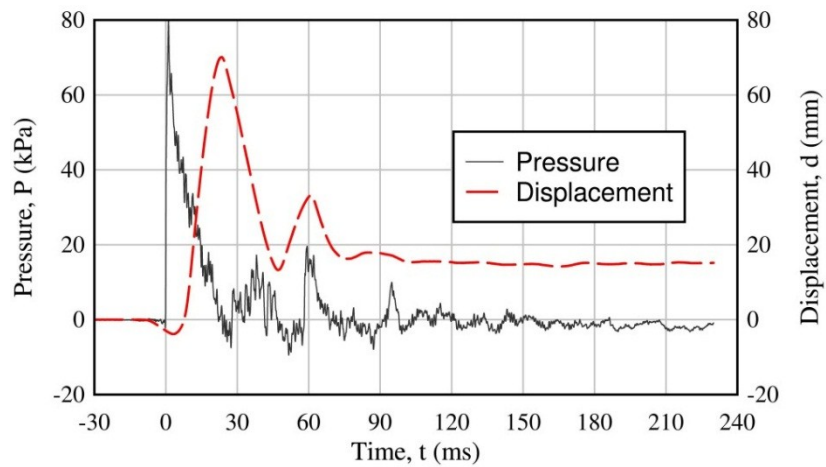


c) Shot 2 Pressure and Displacement

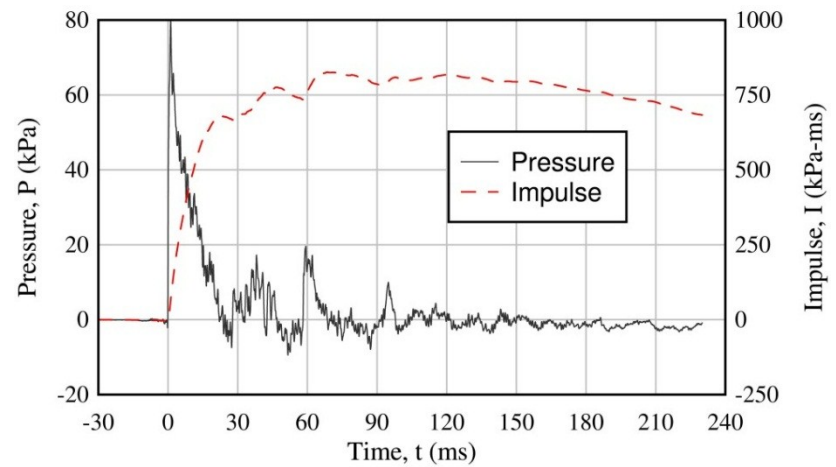


d) Shot 2 Pressure and Impulse

Figure A-21: Pressure, Impulse, and Displacement Time History for Column 2NS-JR Shot 1 and Shot 2

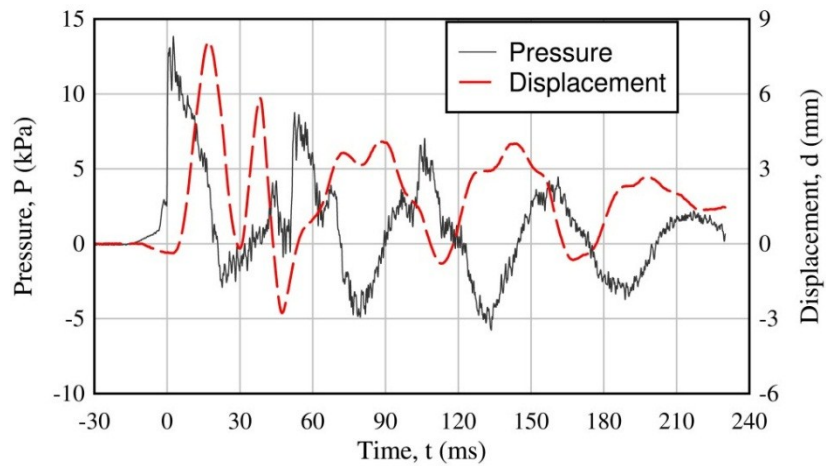


a) Shot 3 Pressure and Displacement

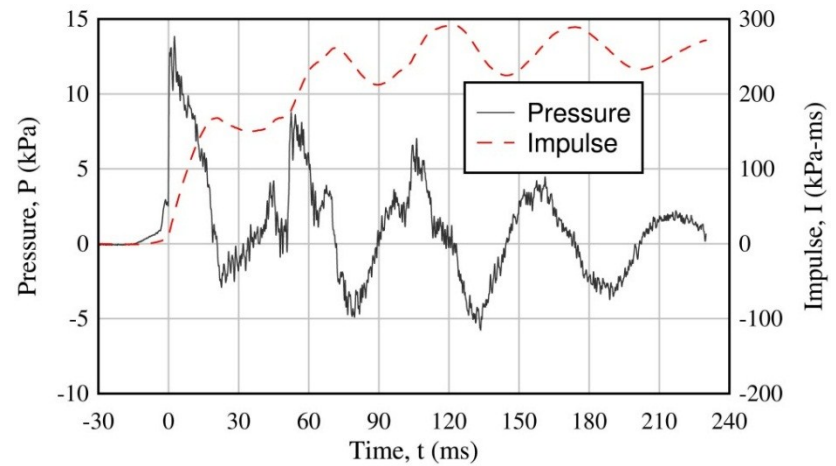


b) Shot 3 Pressure and Impulse

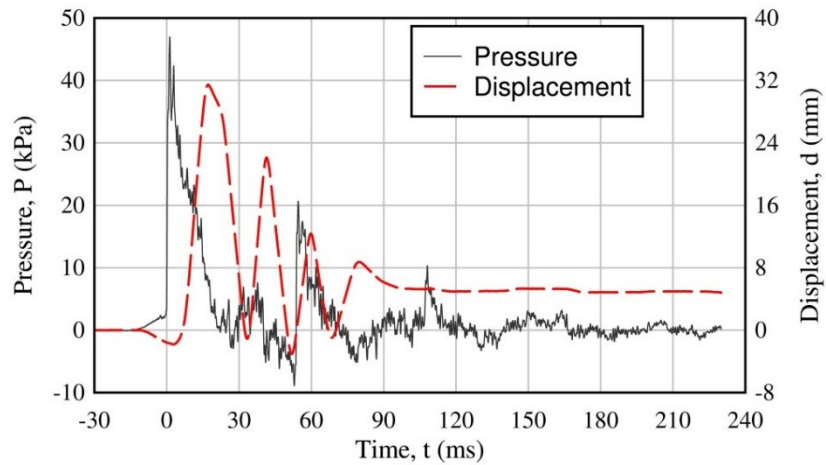
Figure A-22 : Pressure, Impulse, and Displacement Time History for Column 2NS-JR Shot 3



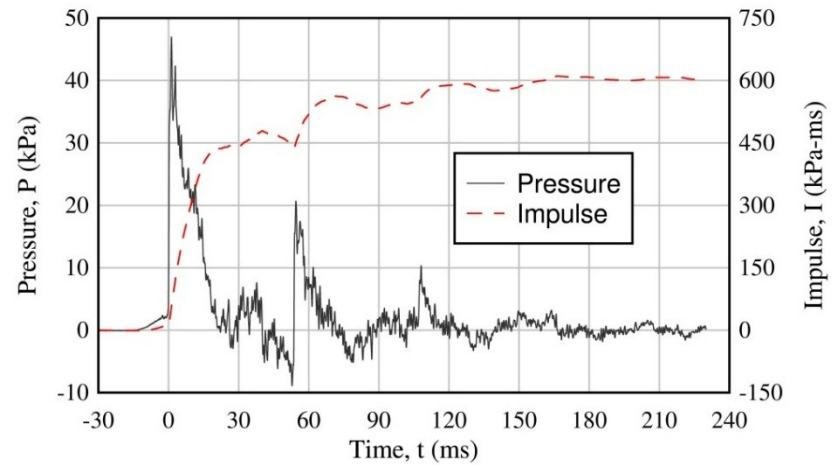
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

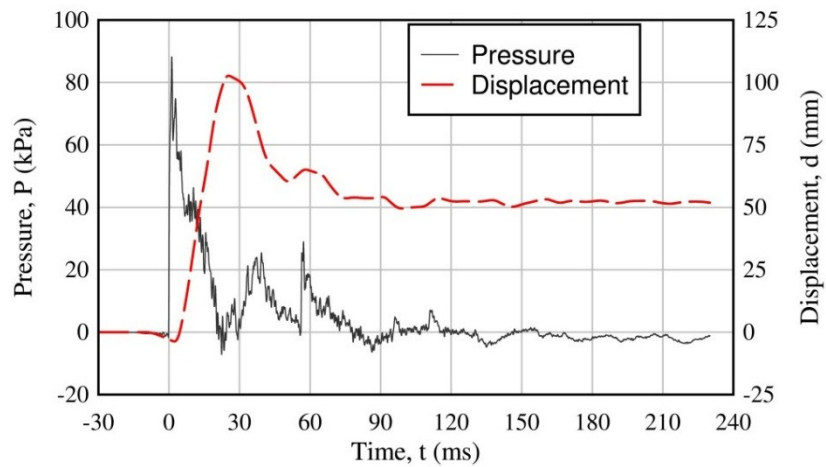


c) Shot 2 Pressure and Displacement

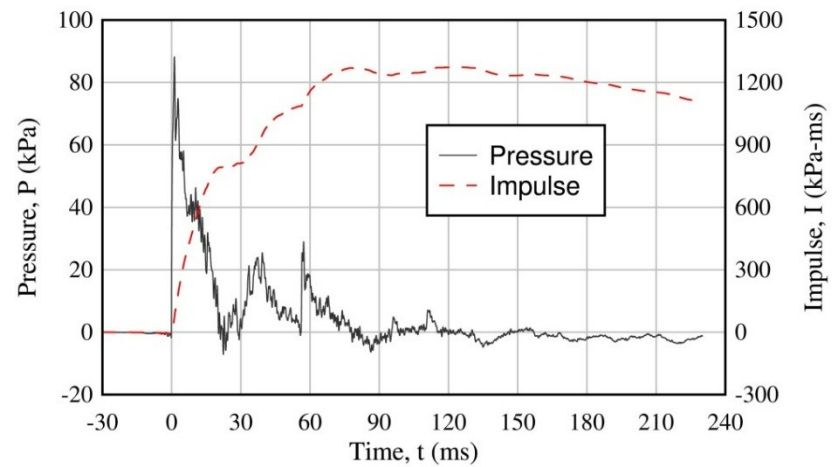


d) Shot 2 Pressure and Impulse

Figure A-23: Pressure, Impulse, and Displacement Time History for Column 2S-JR Shot 1 and Shot 2

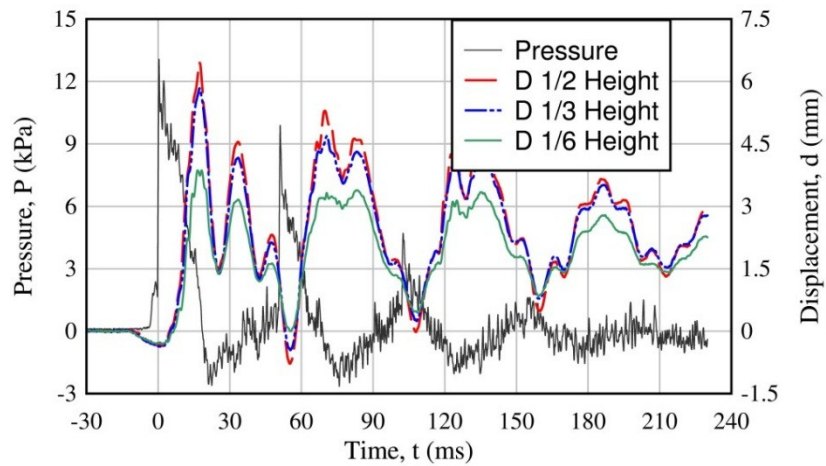


a) Shot 3 Pressure and Displacement

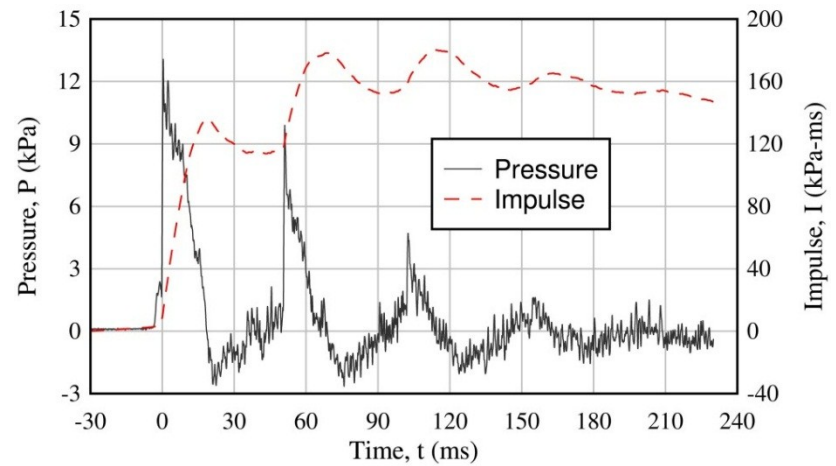


b) Shot 3 Pressure and Impulse

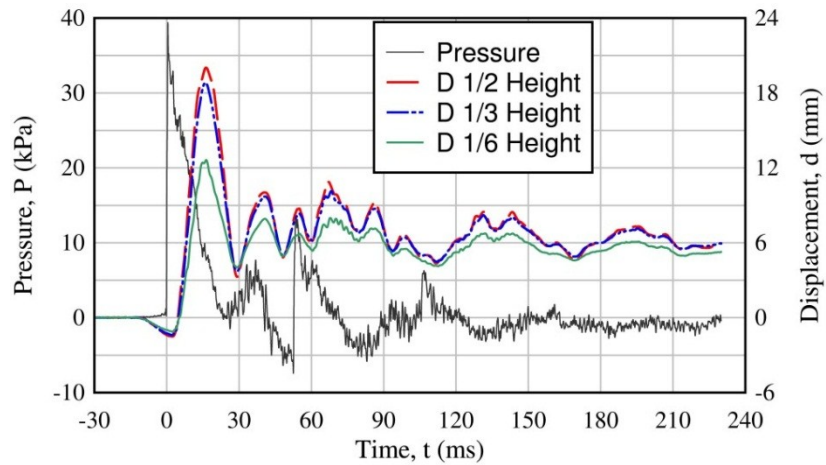
Figure A-24 : Pressure, Impulse, and Displacement Time History for Column 2S-JR Shot 3



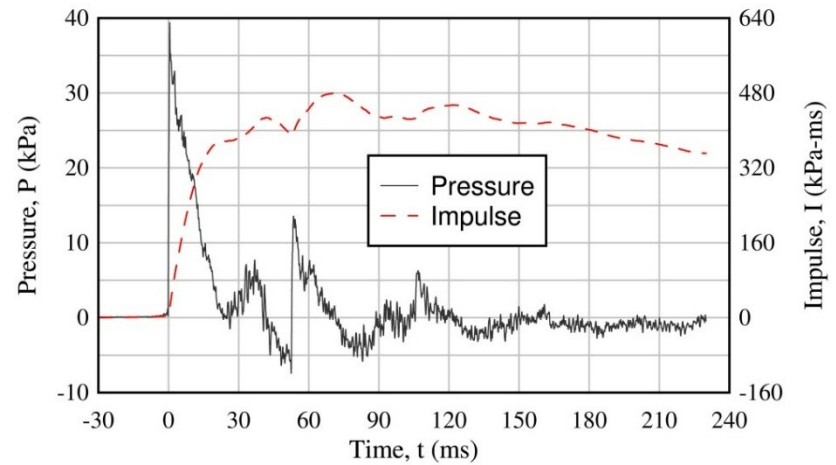
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

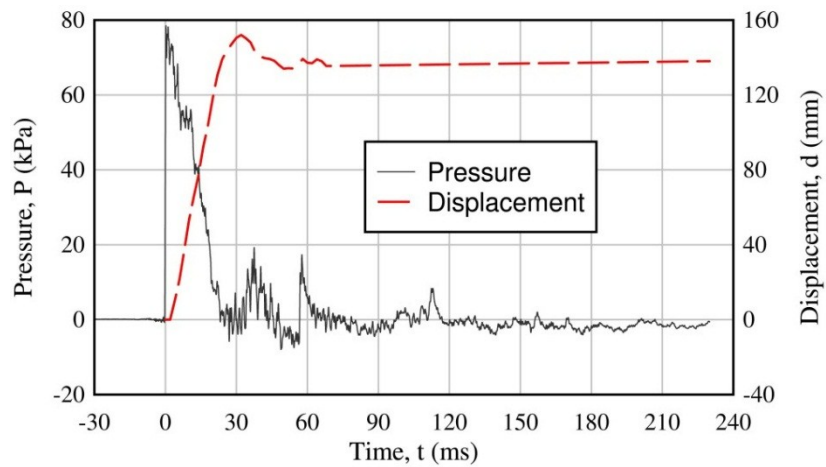


c) Shot 2 Pressure and Displacement

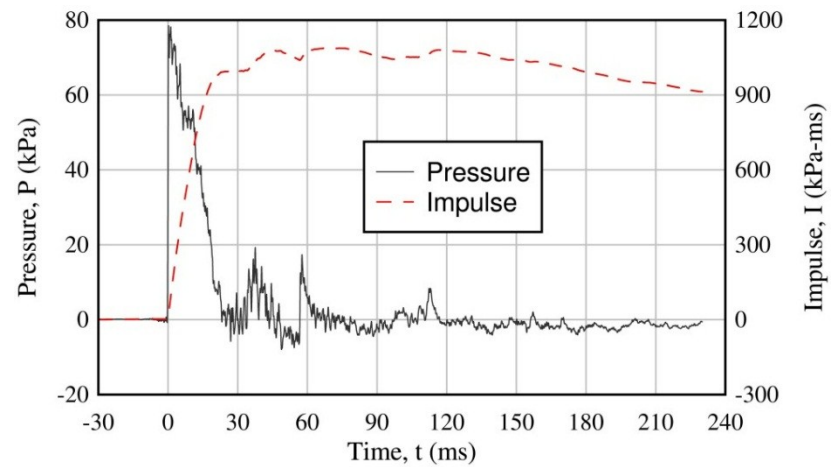


d) Shot 2 Pressure and Impulse

Figure A-25: Pressure, Impulse, and Displacement Time History for Column 3NS-C Shot 1 and Shot 2

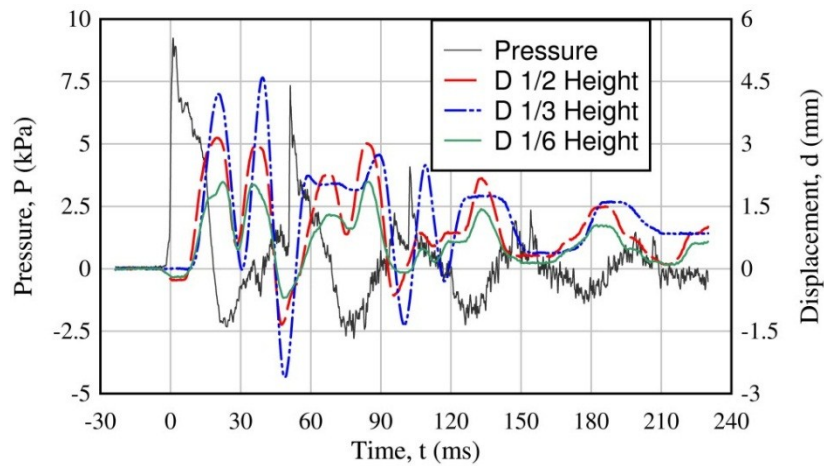


a) Shot 3 Pressure and Displacement

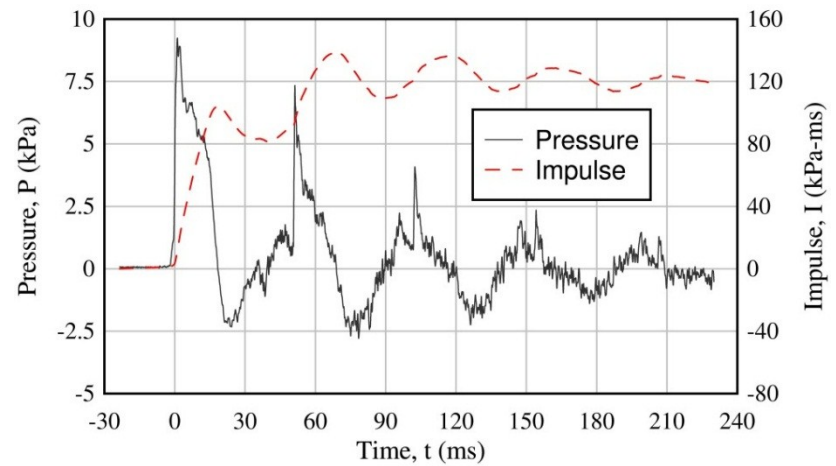


b) Shot 3 Pressure and Impulse

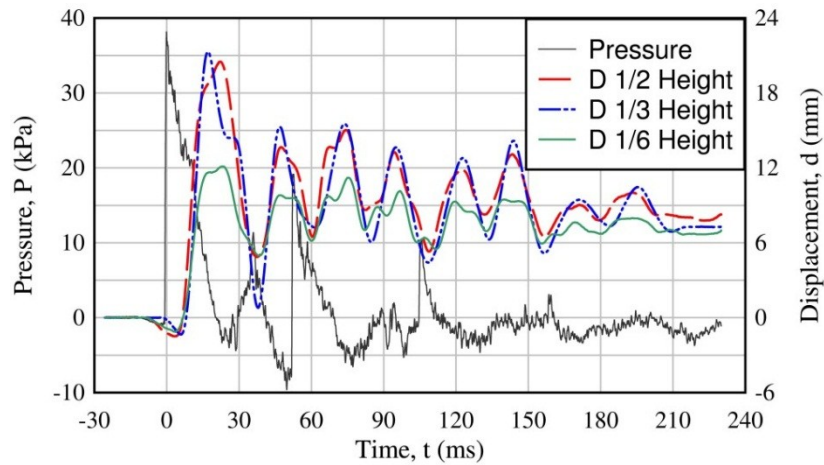
Figure A-26 : Pressure, Impulse, and Displacement Time History for Column 3NS-C Shot 3



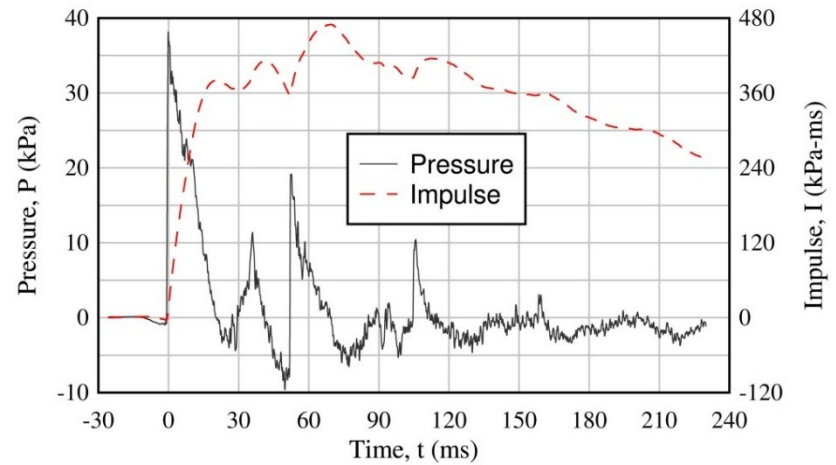
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

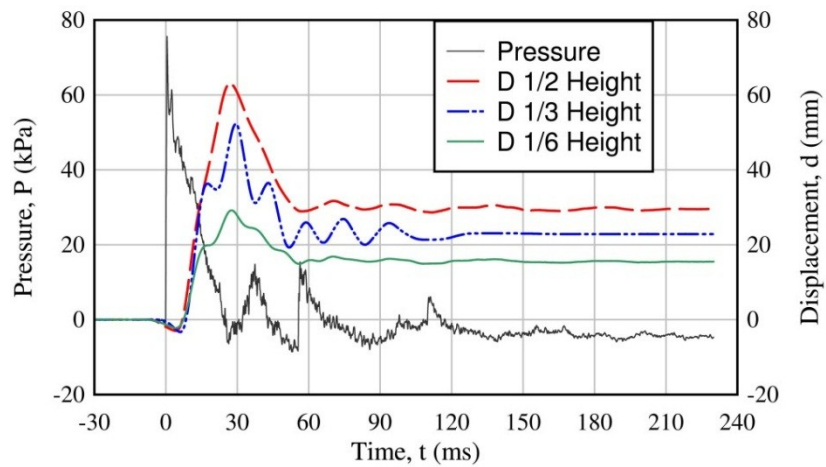


c) Shot 2 Pressure and Displacement

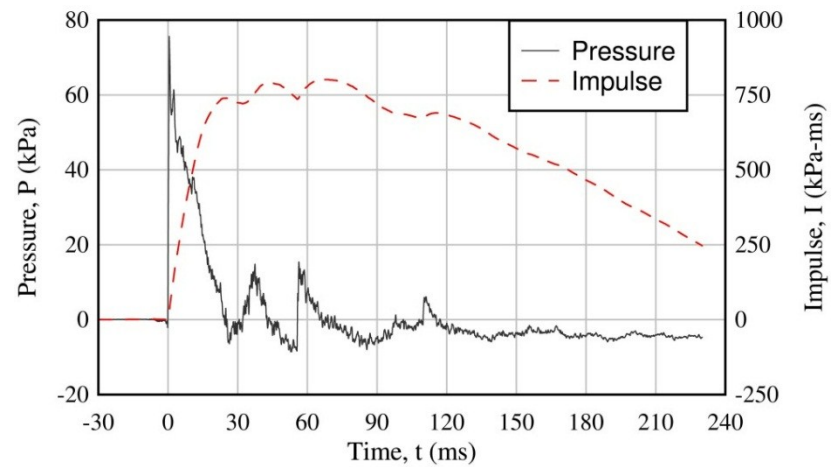


d) Shot 2 Pressure and Impulse

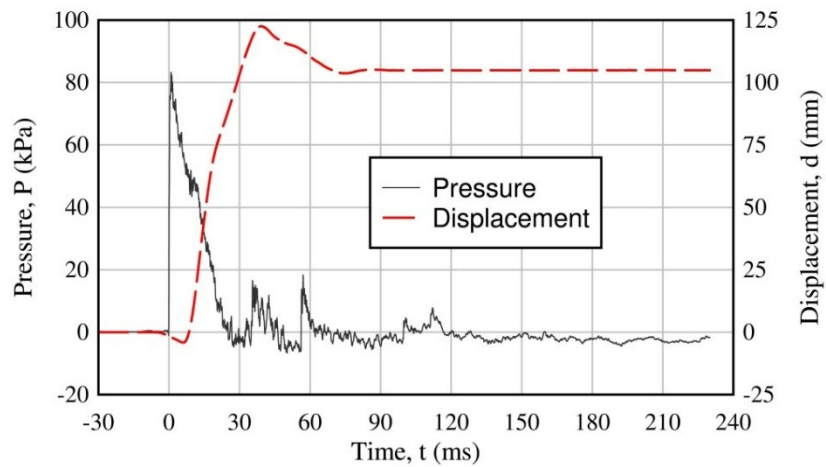
Figure A-27: Pressure, Impulse, and Displacement Time History for Column 3NS-W Shot 1 and Shot 2



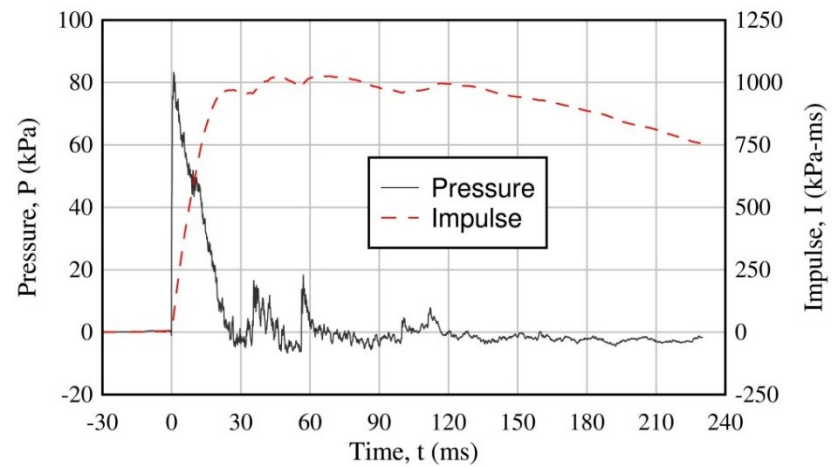
a) Shot 3 Pressure and Displacement



b) Shot 3 Pressure and Impulse

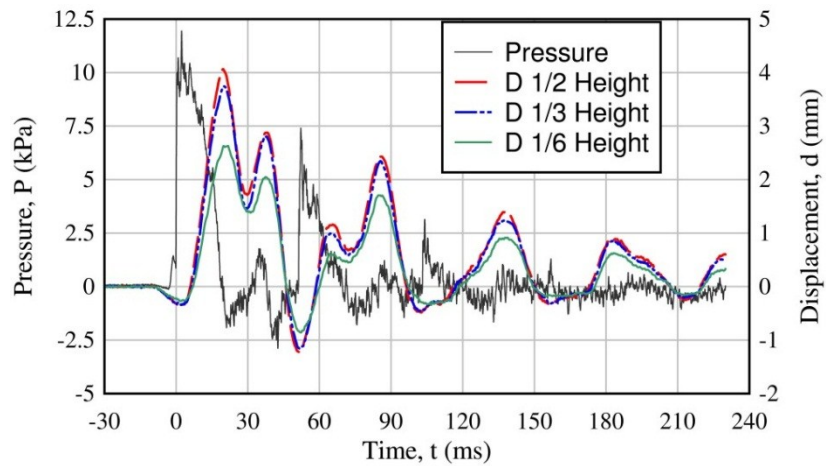


c) Shot 4 Pressure and Displacement

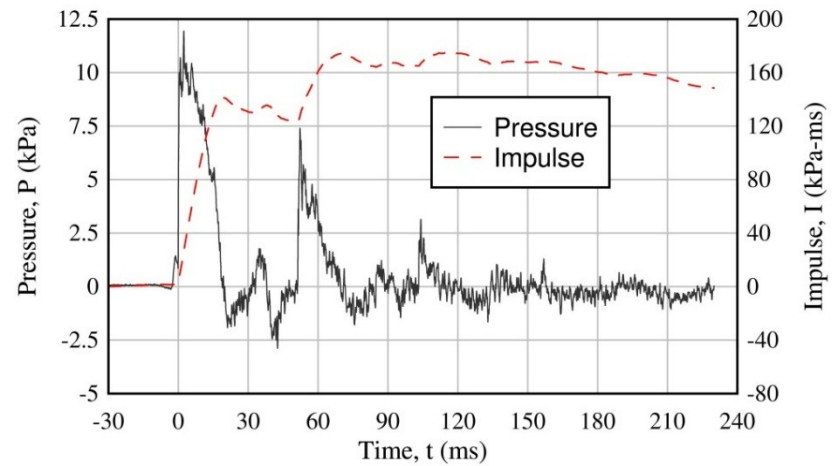


d) Shot 4 Pressure and Impulse

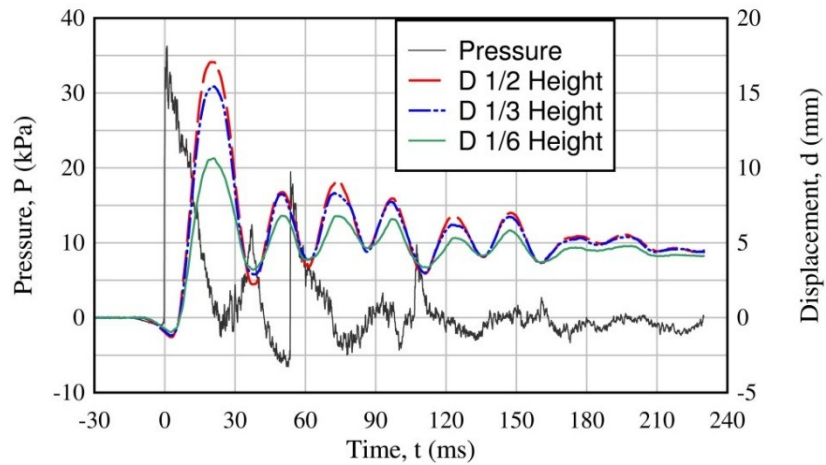
Figure A-28: Pressure, Impulse, and Displacement Time History for Column 3NS-W Shot 3 and Shot 4



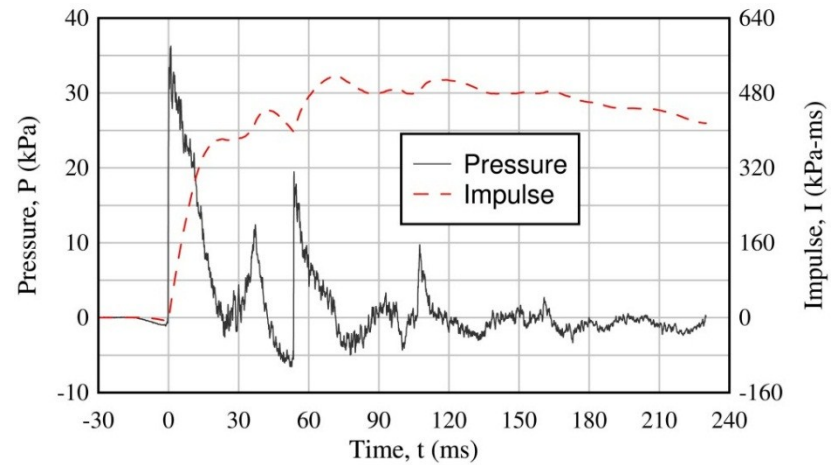
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

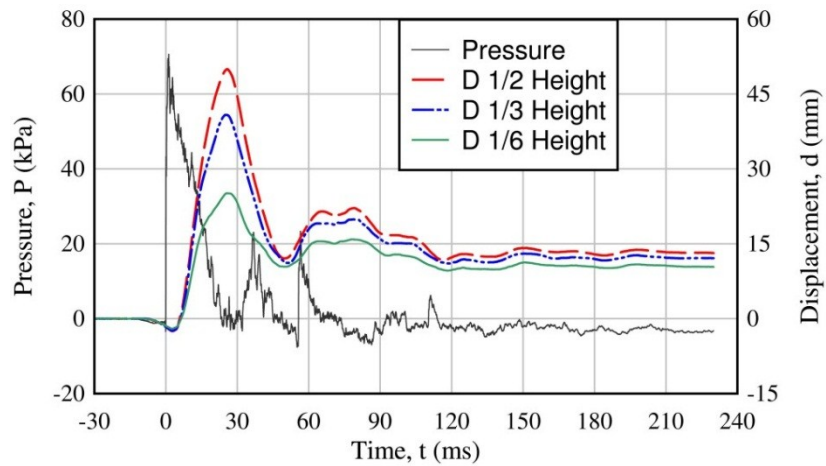


c) Shot 2 Pressure and Displacement

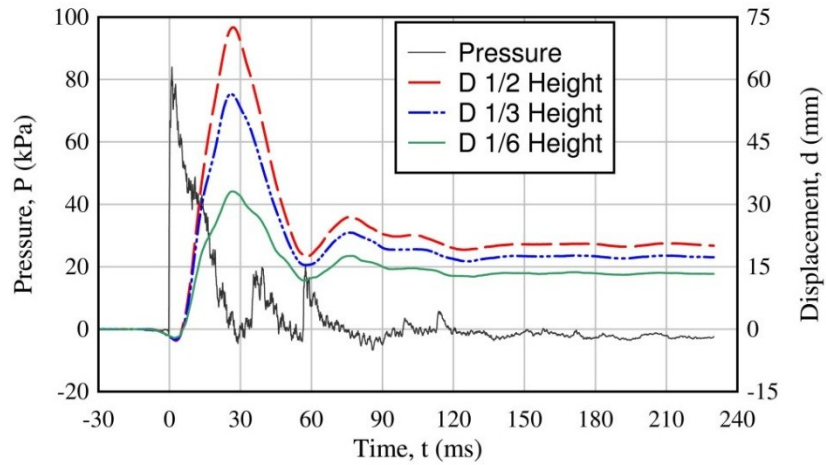


d) Shot 2 Pressure and Impulse

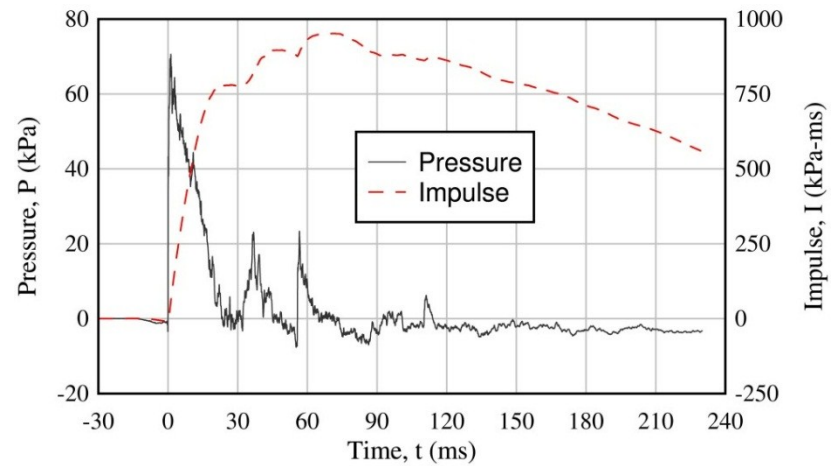
Figure A-29: Pressure, Impulse, and Displacement Time History for Column 3NS-CB Shot 1 and Shot 2



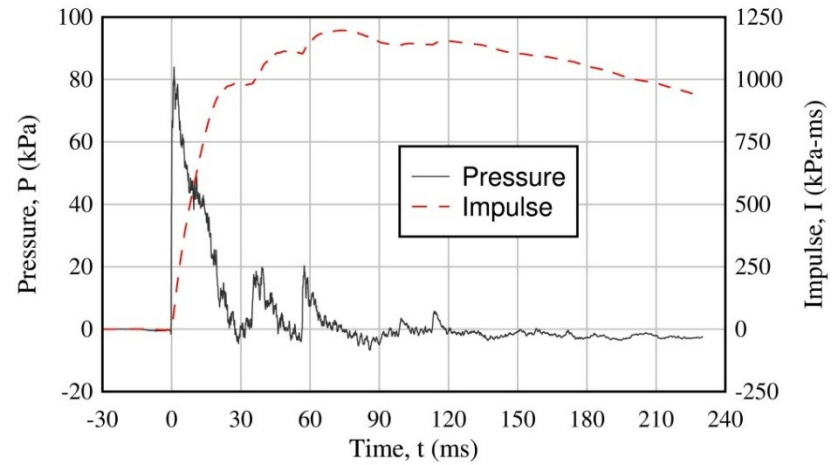
a) Shot 3 Pressure and Displacement



c) Shot 4 Pressure and Displacement

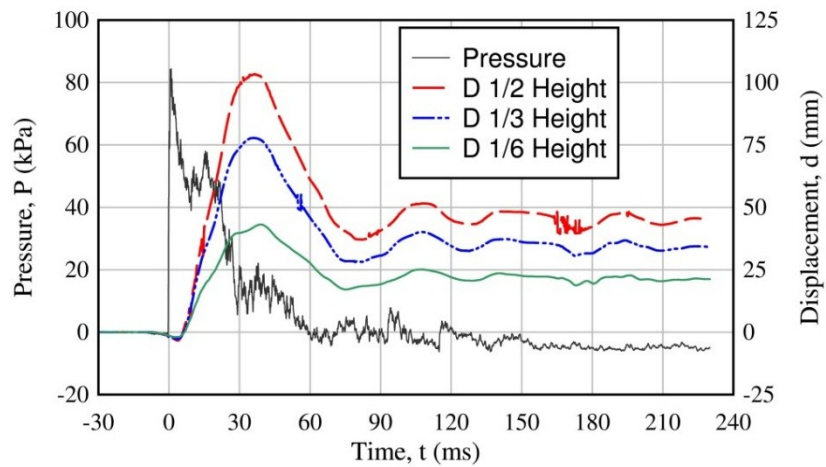


b) Shot 3 Pressure and Impulse

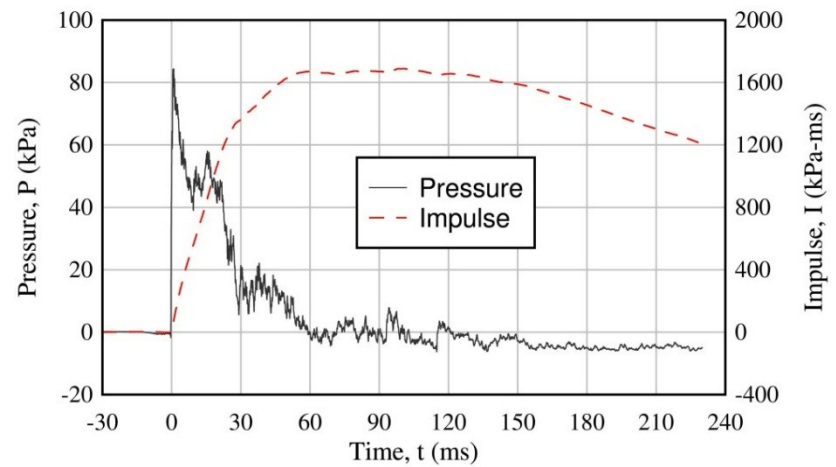


d) Shot 4 Pressure and Impulse

Figure A-30: Pressure, Impulse, and Displacement Time History for Column 3NS-CB Shot 3 and Shot 4

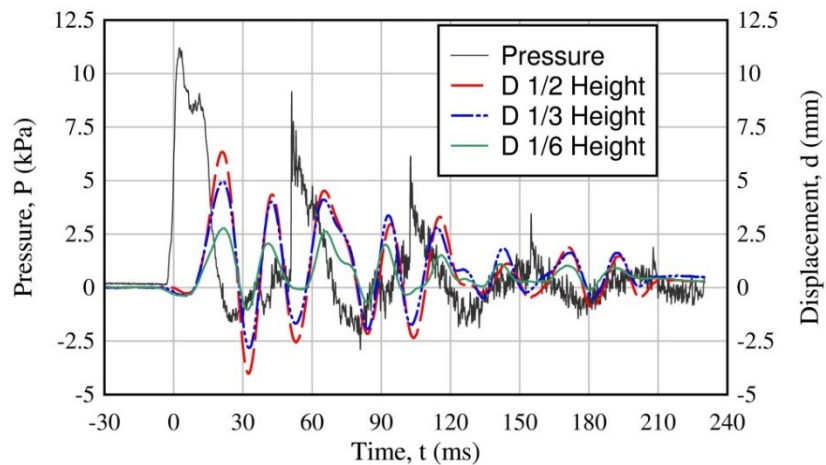


a) Shot 5 Pressure and Displacement

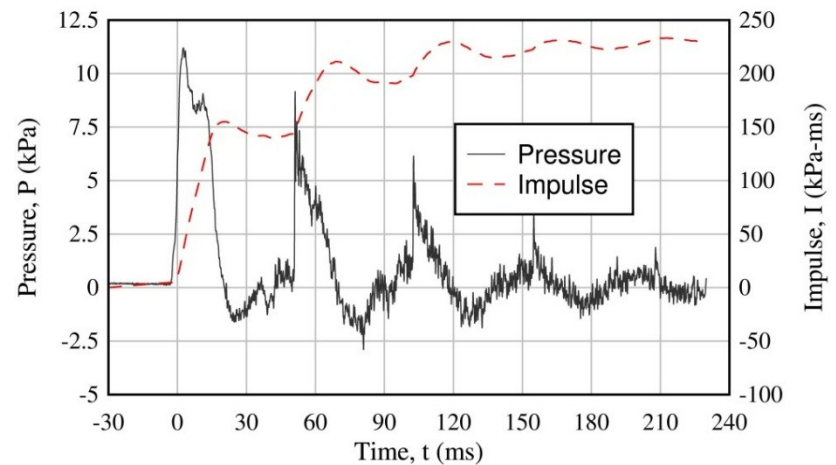


b) Shot 5 Pressure and Impulse

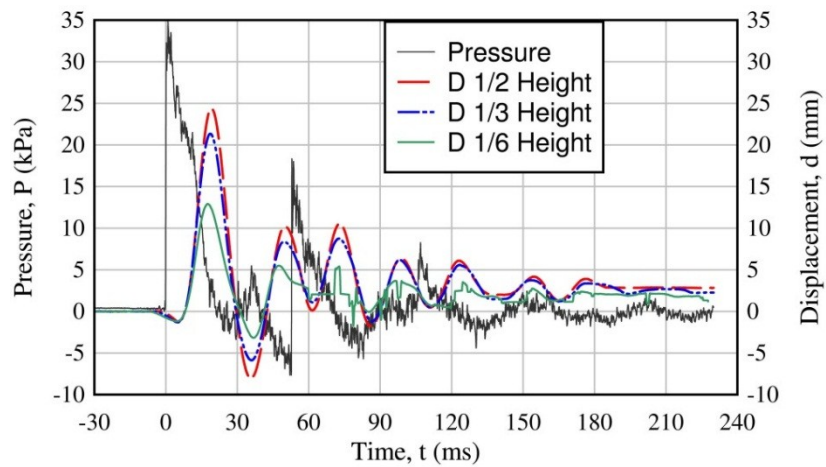
Figure A-31: Pressure, Impulse, and Displacement Time History for Column 3NS-CB Shot 5



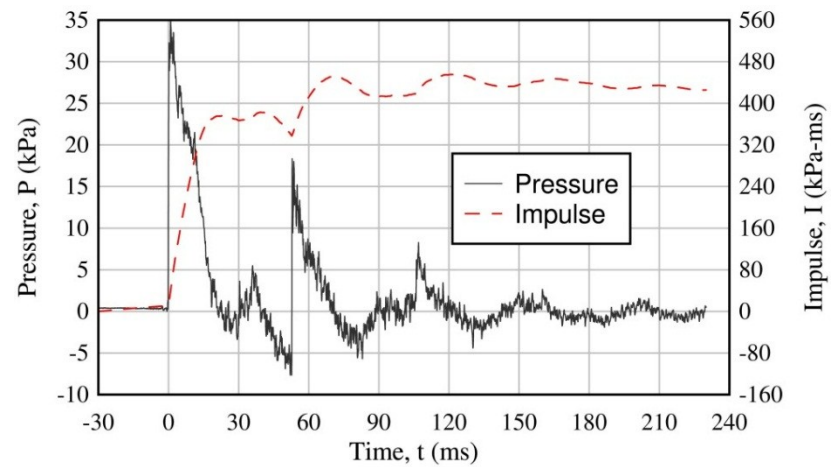
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

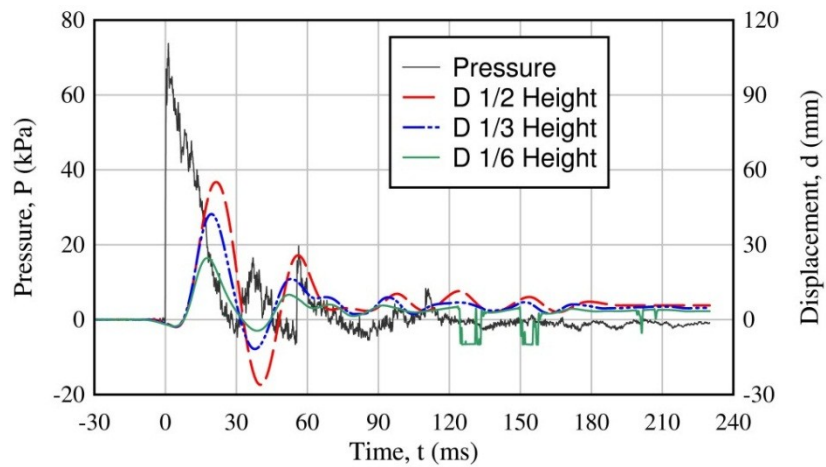


c) Shot 2 Pressure and Displacement

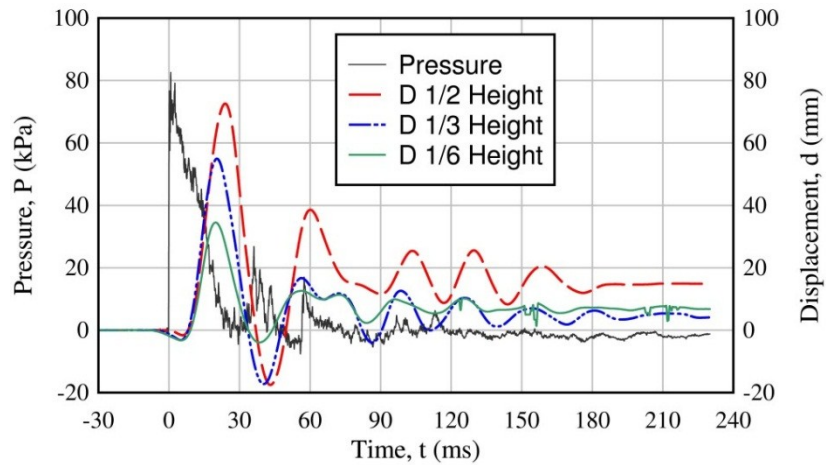


d) Shot 2 Pressure and Impulse

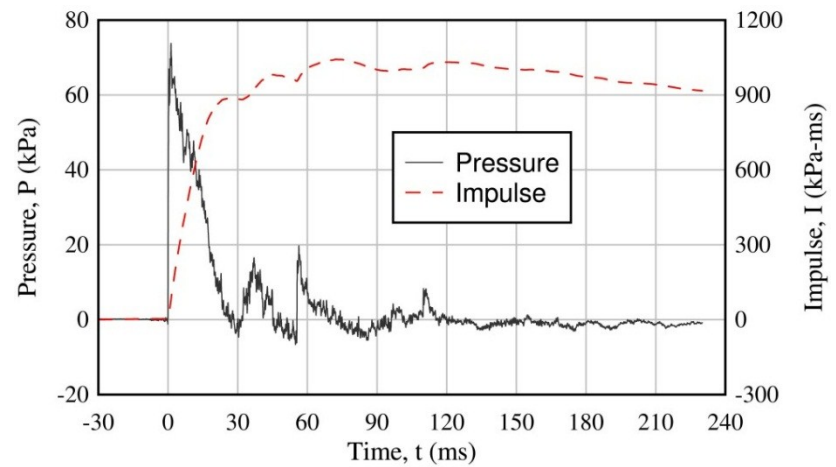
Figure A-32: Pressure, Impulse, and Displacement Time History for Column 3NS-TB1 Shot 1 and Shot 2



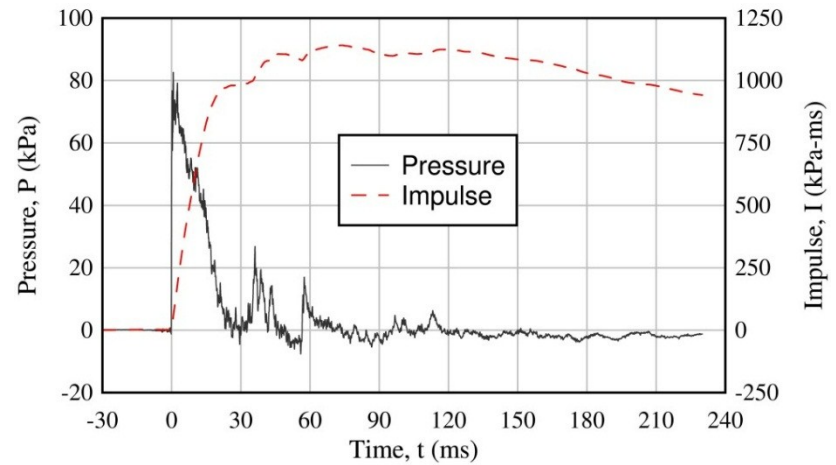
a) Shot 3 Pressure and Displacement



c) Shot 4 Pressure and Displacement

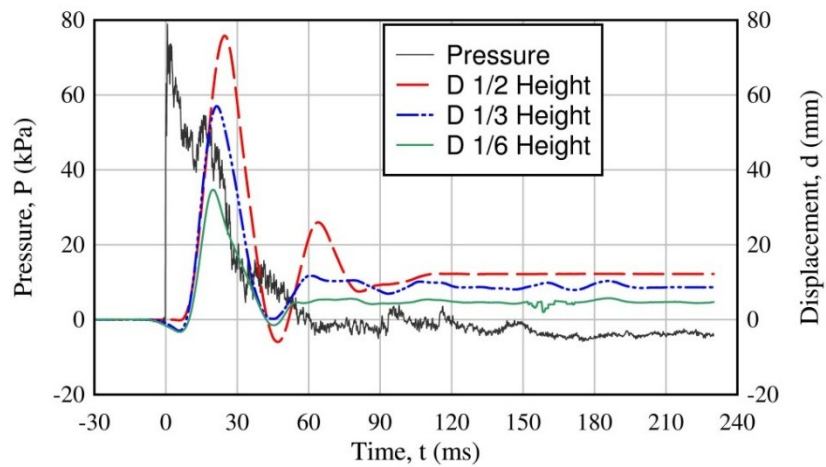


b) Shot 3 Pressure and Impulse

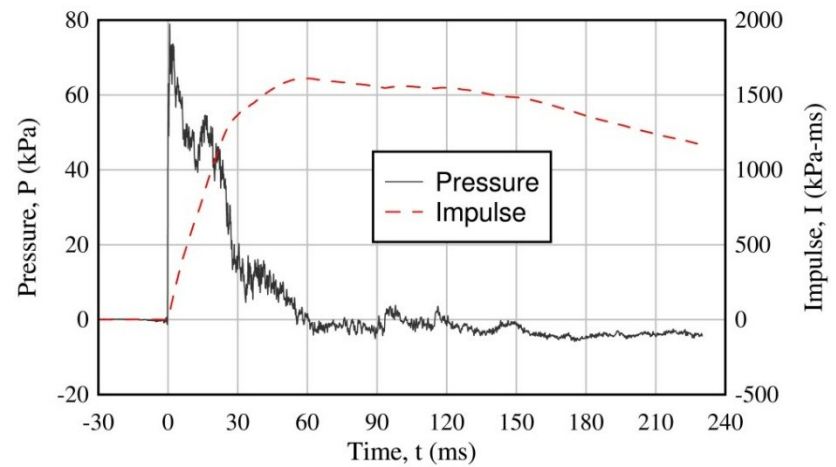


d) Shot 4 Pressure and Impulse

Figure A-33: Pressure, Impulse, and Displacement Time History for Column 3NS-TB1 Shot 3 and Shot 4

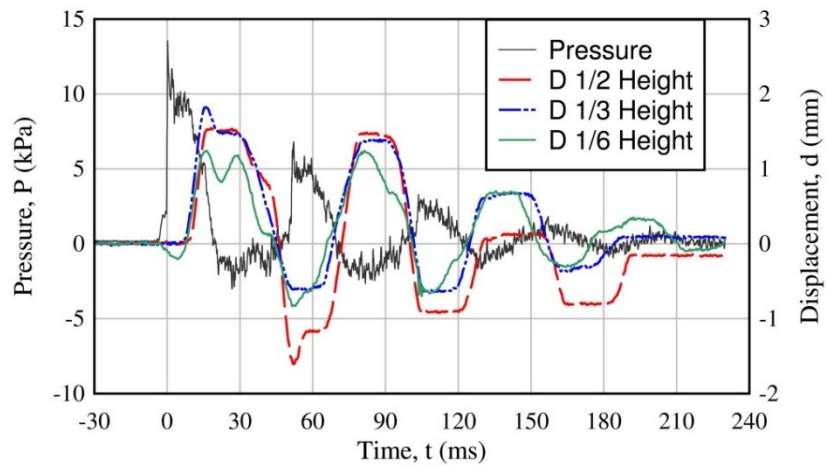


a) Shot 5 Pressure and Displacement

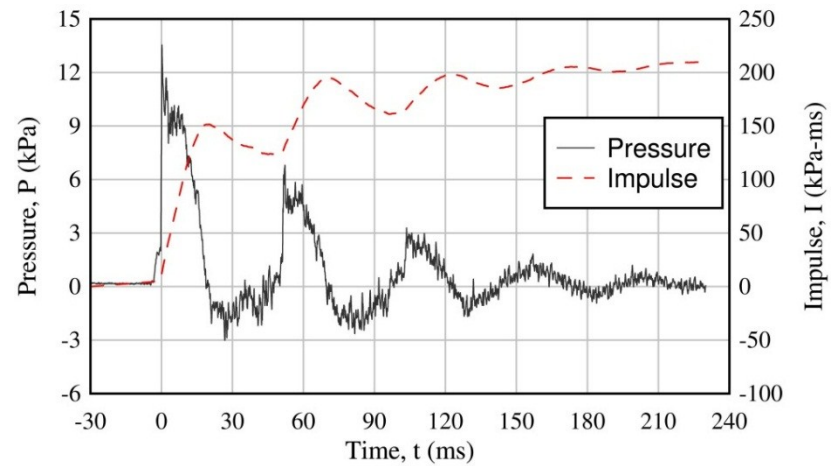


b) Shot 5 Pressure and Impulse

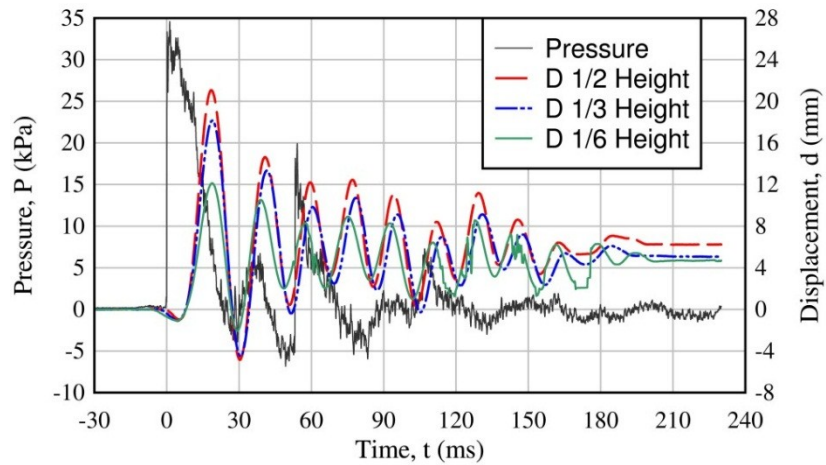
Figure A-34: Pressure, Impulse, and Displacement Time History for Column 3NS-TB1 Shot 5



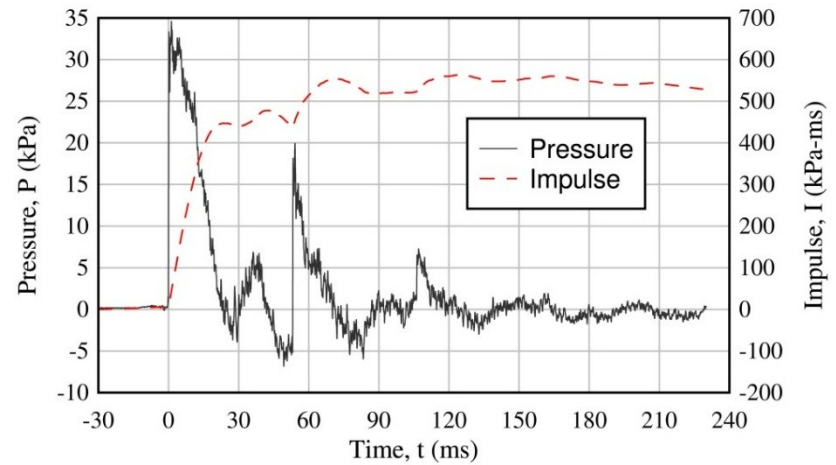
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

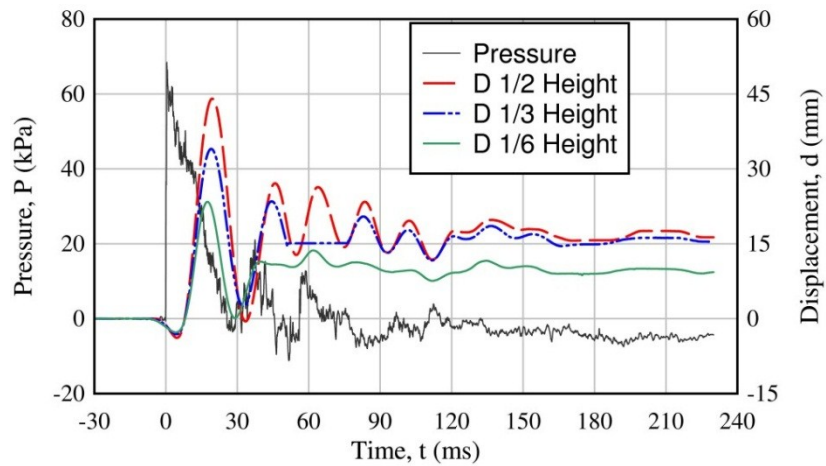


c) Shot 2 Pressure and Displacement

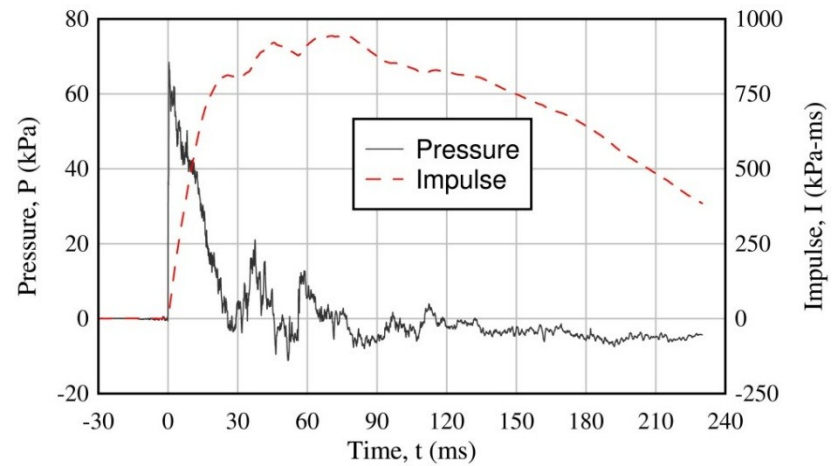


d) Shot 2 Pressure and Impulse

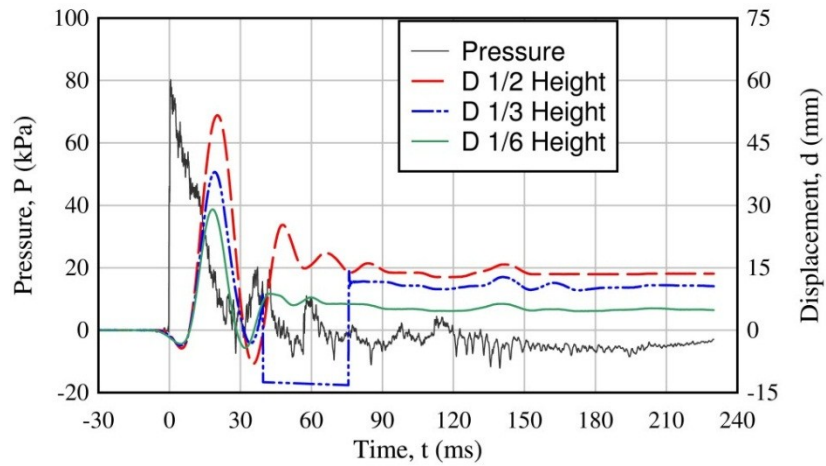
Figure A-35: Pressure, Impulse, and Displacement Time History for Column 3NS-TB2 Shot 1 and Shot 2



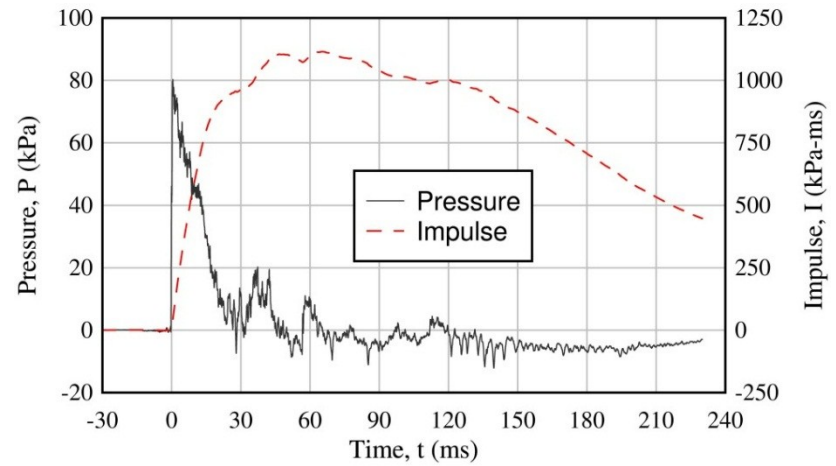
a) Shot 3 Pressure and Displacement



b) Shot 3 Pressure and Impulse

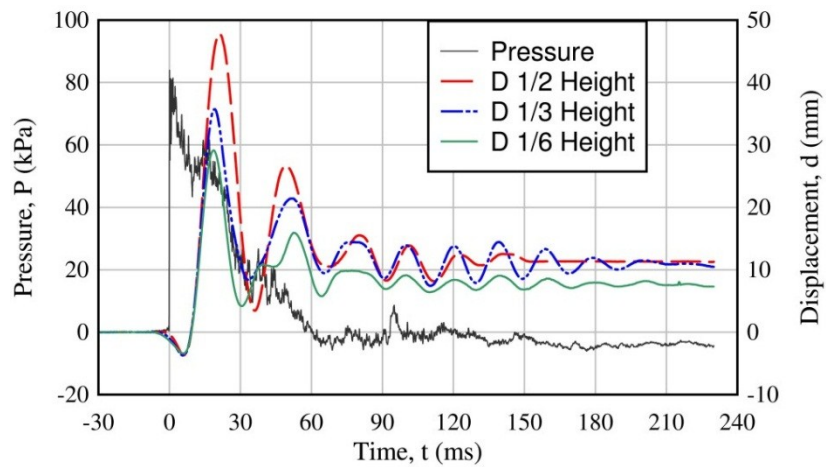


c) Shot 4 Pressure and Displacement

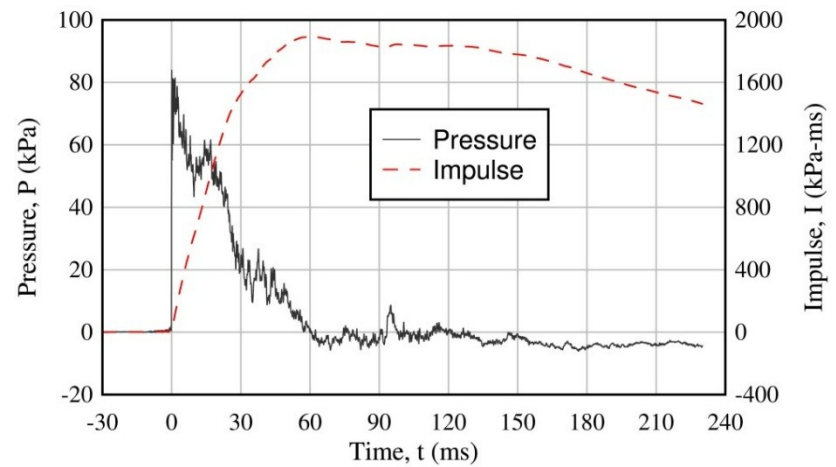


d) Shot 4 Pressure and Impulse

Figure A-36: Pressure, Impulse, and Displacement Time History for Column 3NS-TB2 Shot 3 and Shot 4

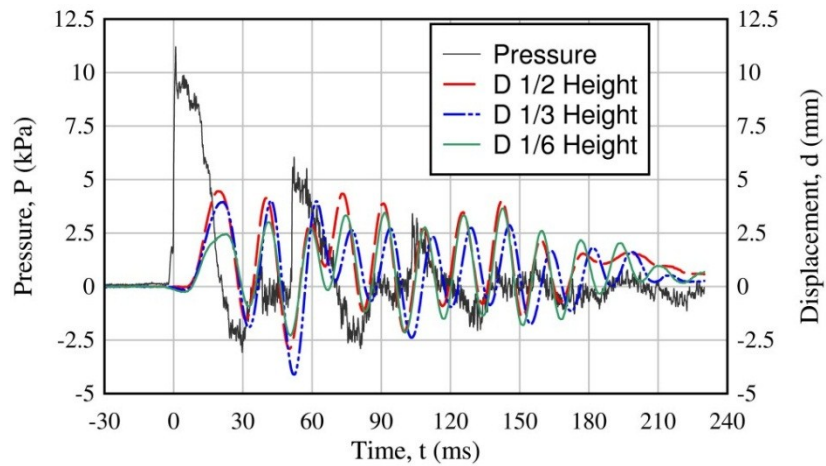


a) Shot 5 Pressure and Displacement

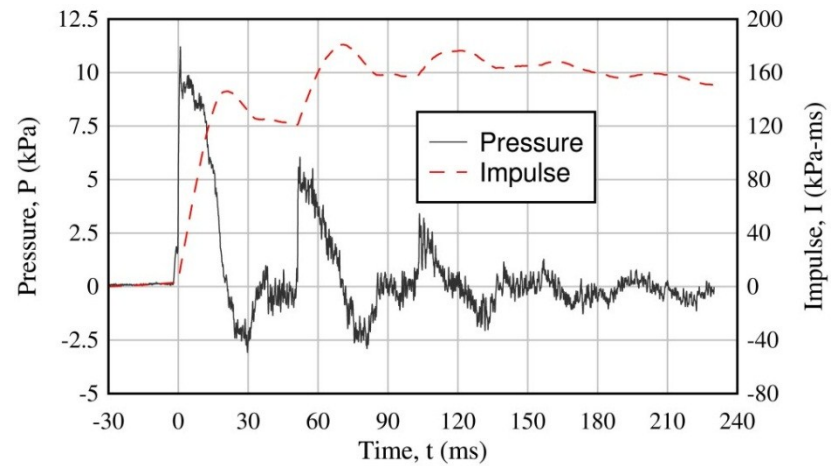


b) Shot 5 Pressure and Impulse

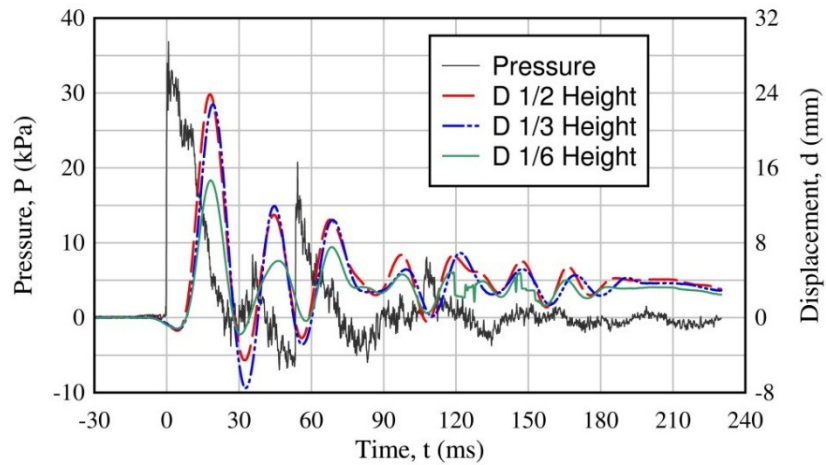
Figure A-37: Pressure, Impulse, and Displacement Time History for Column 3NS-TB2 Shot 5



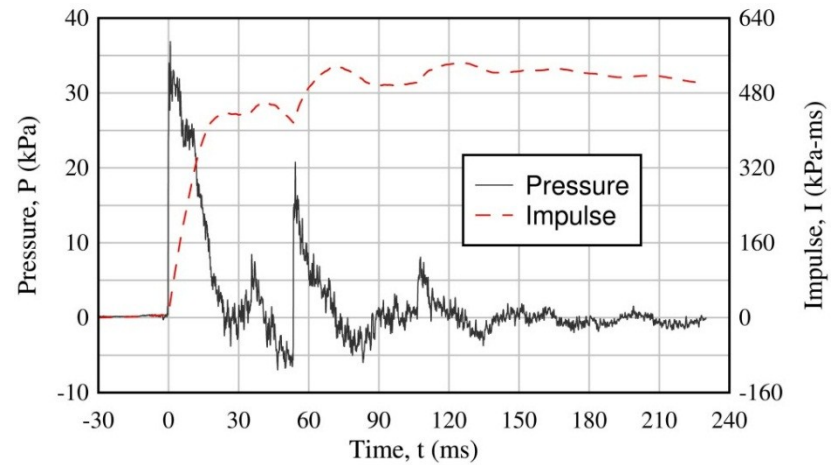
a) Shot 1 Pressure and Displacement



b) Shot 1 Pressure and Impulse

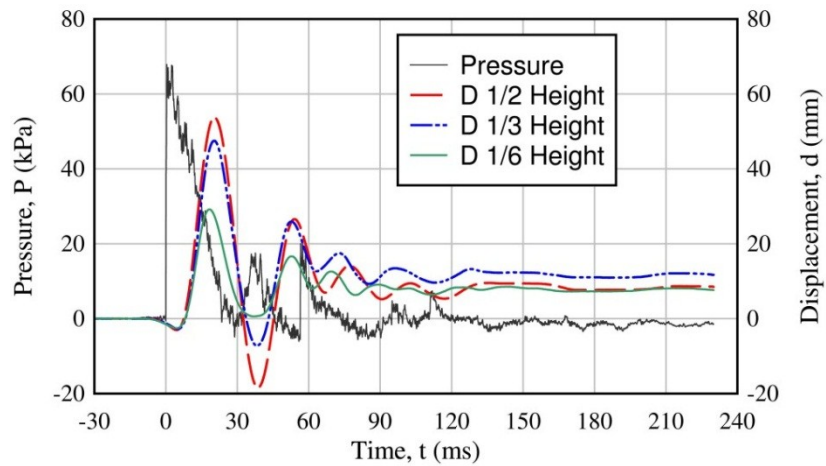


c) Shot 2 Pressure and Displacement

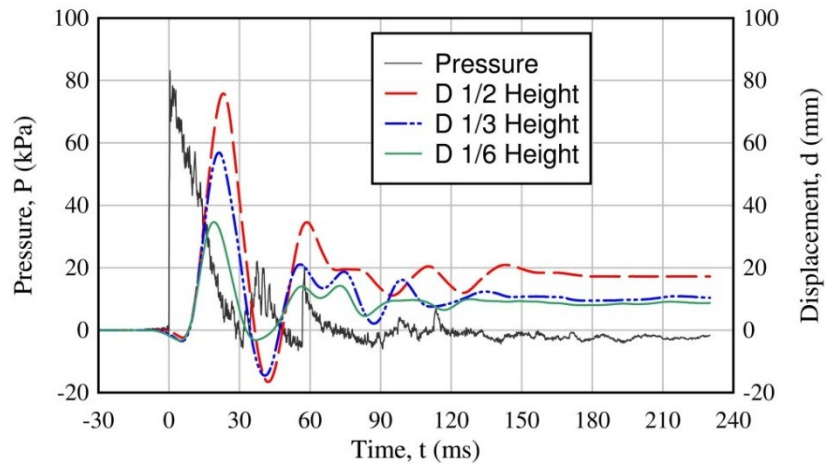


d) Shot 2 Pressure and Impulse

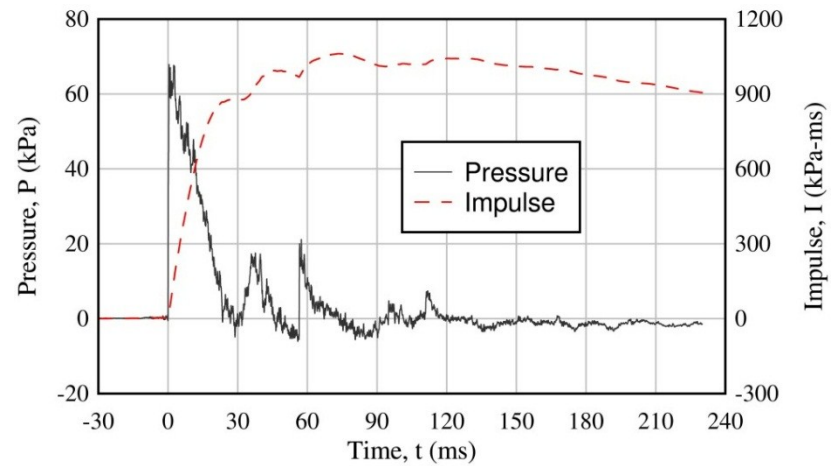
Figure A-38: Pressure, Impulse, and Displacement Time History for Column 3NS-TB3 Shot 1 and Shot 2



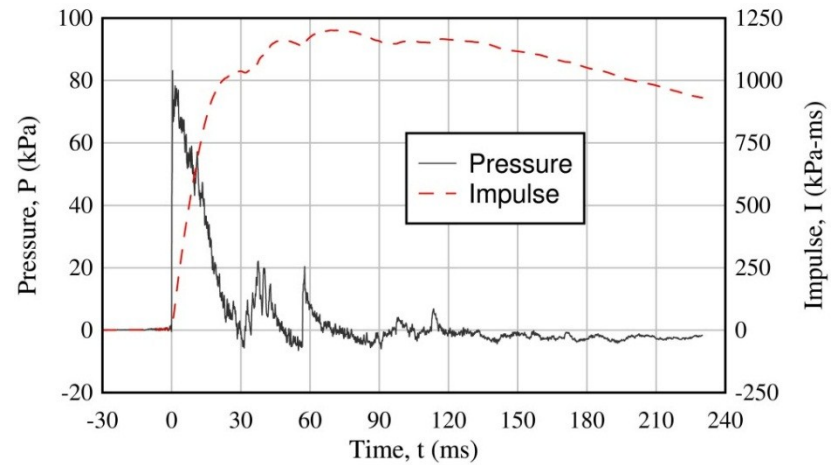
a) Shot 3 Pressure and Displacement



c) Shot 4 Pressure and Displacement

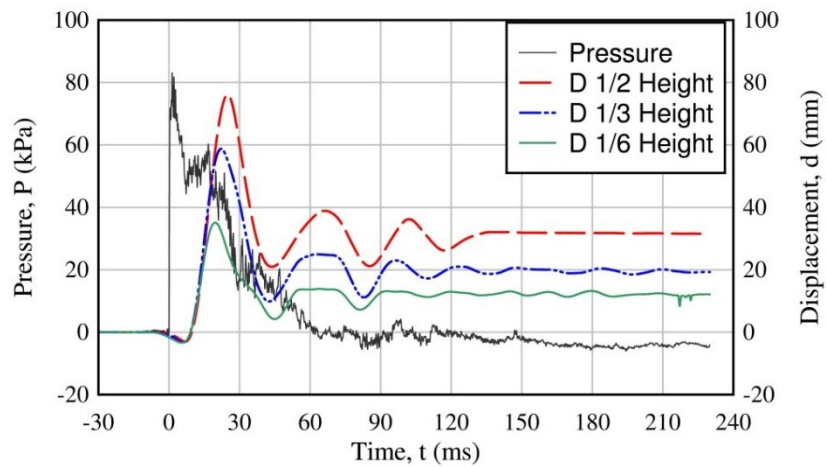


b) Shot 3 Pressure and Impulse

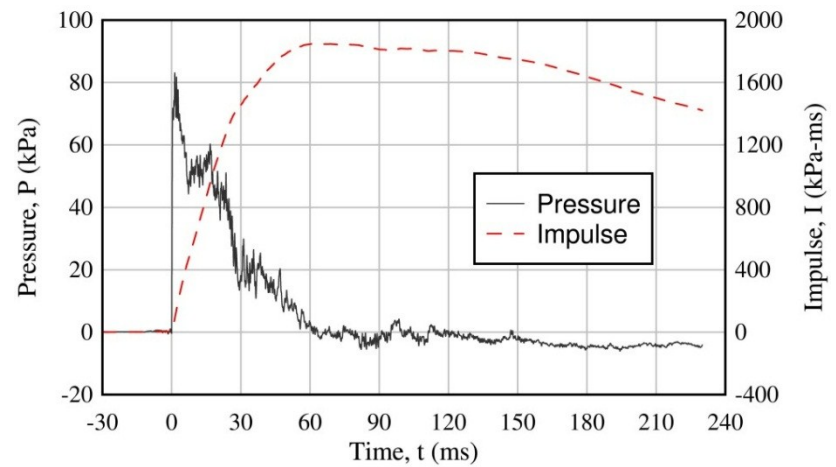


d) Shot 4 Pressure and Impulse

Figure A-39: Pressure, Impulse, and Displacement Time History for Column 3NS-TB3 Shot 3 and Shot 4



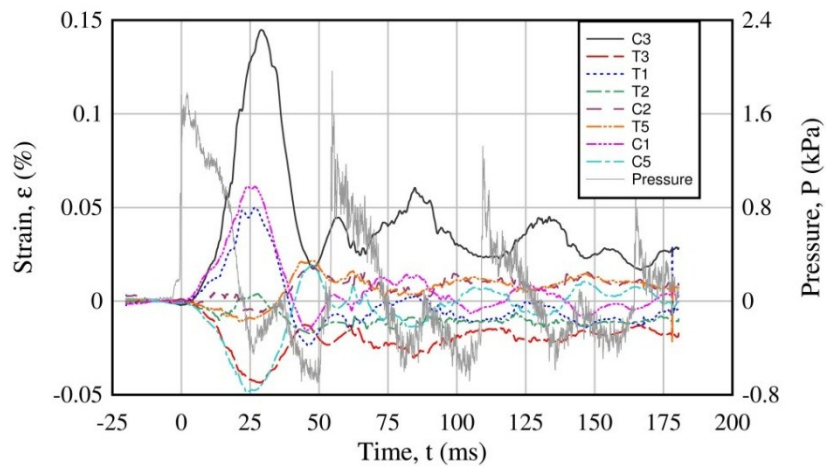
a) Shot 5 Pressure and Displacement



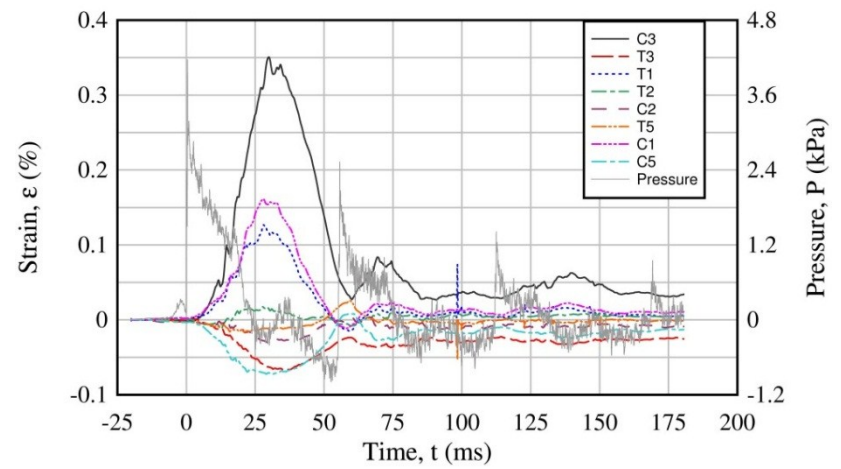
b) Shot 5 Pressure and Impulse

Figure A-40: Pressure, Impulse, and Displacement Time History for Column 3NS-TB3 Shot 5

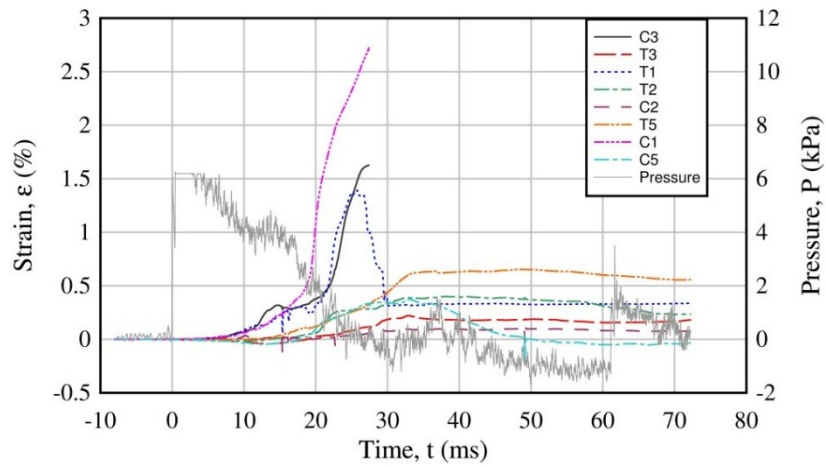
Appendix B - Strain Time Histories



a) Shot 1

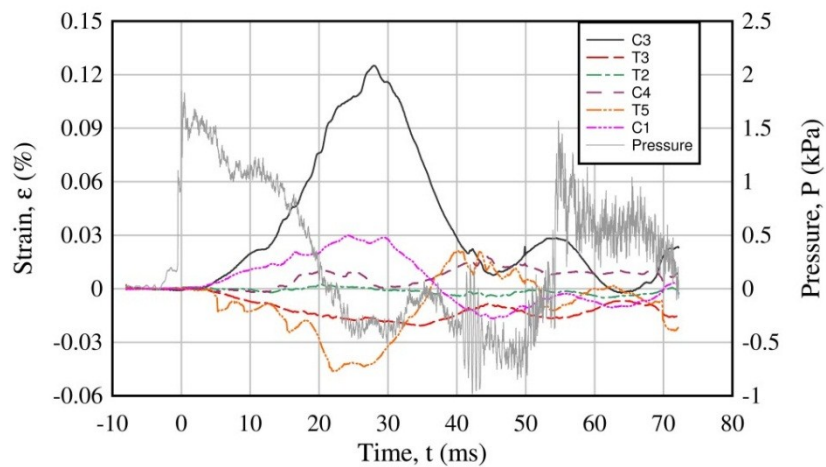


b) Shot 2

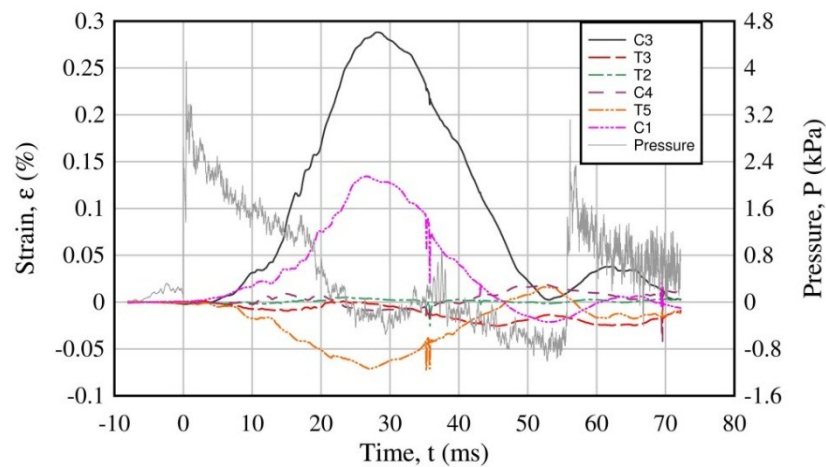


c) Shot 3

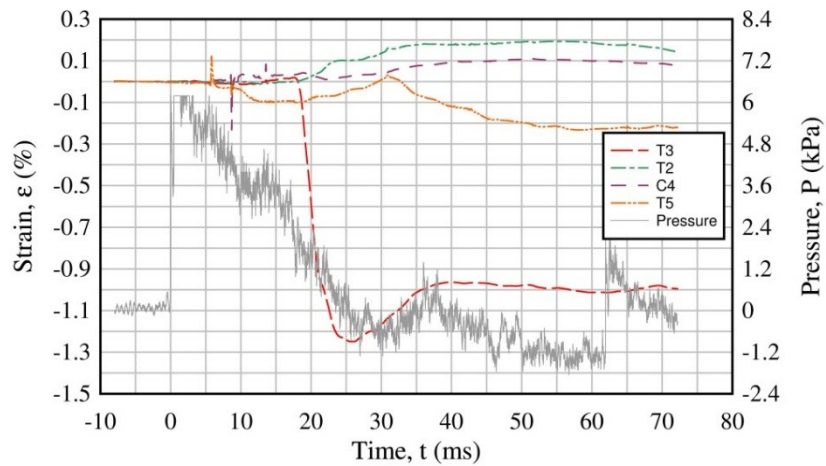
Figure B-1: Column 1NS-C Strain-Time Histories



a) Shot 1

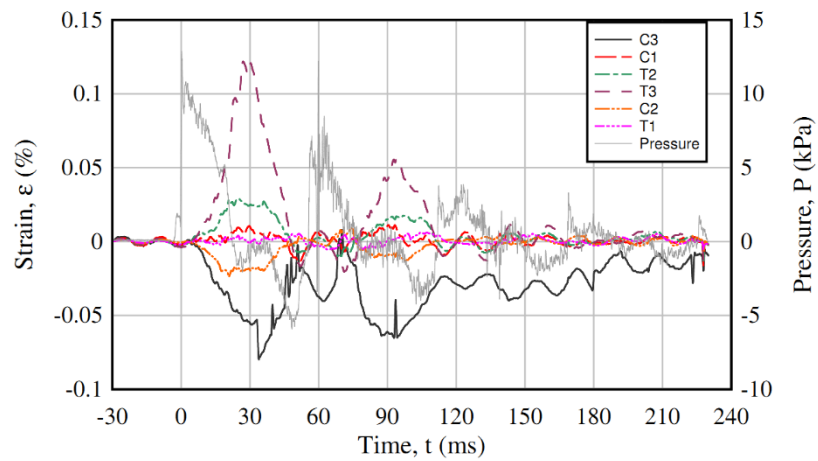


b) Shot 2

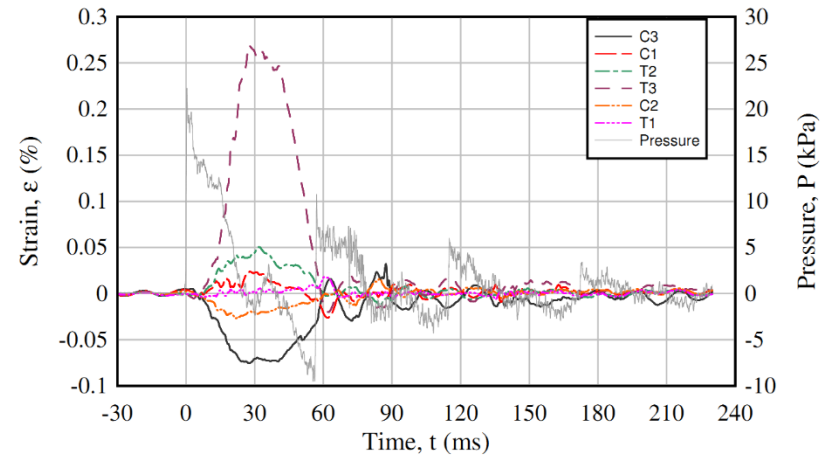


c) Shot 3

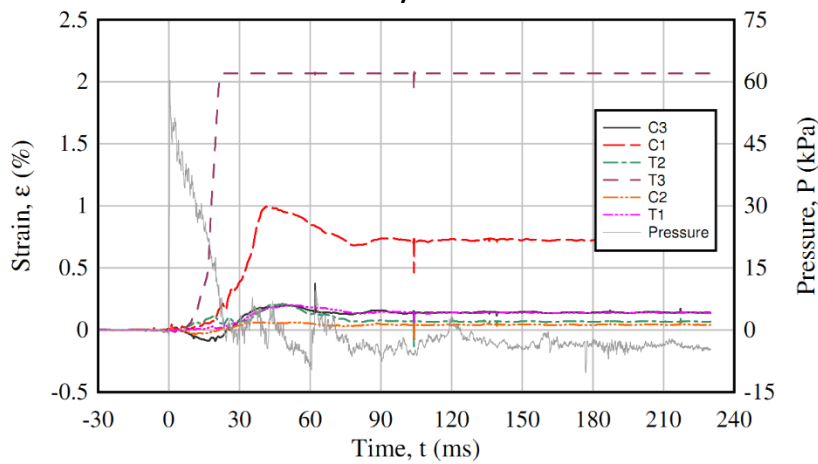
Figure B-2: Column 1S-C Strain-Time Histories



a) Shot 1

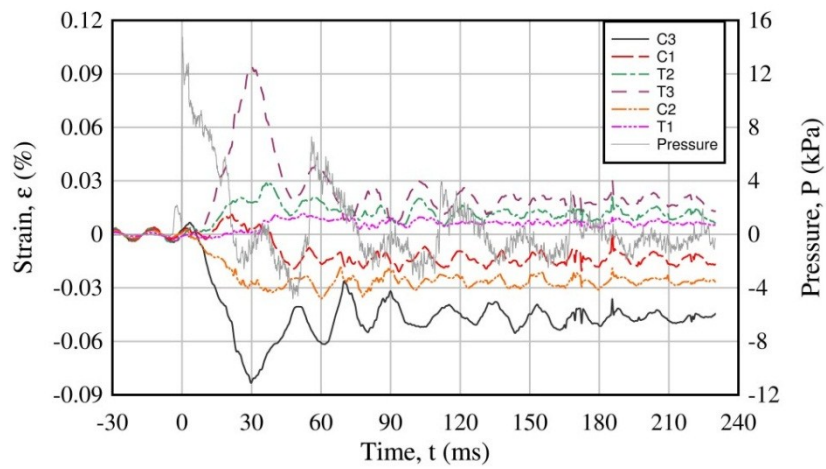


b) Shot 2

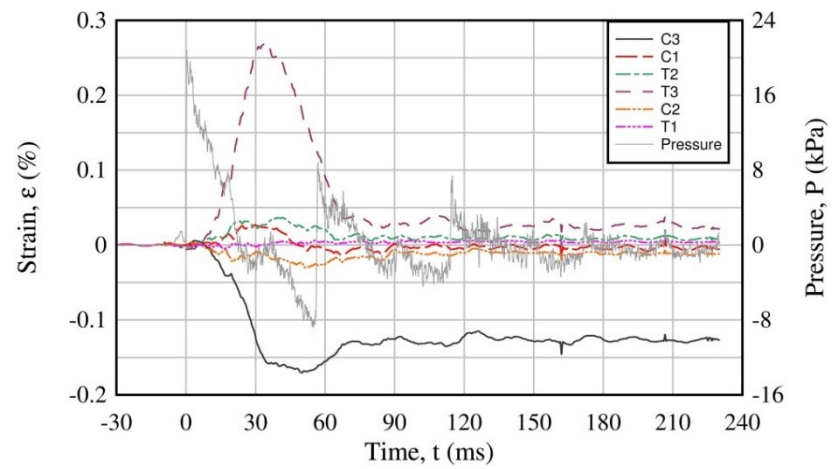


c) Shot 3

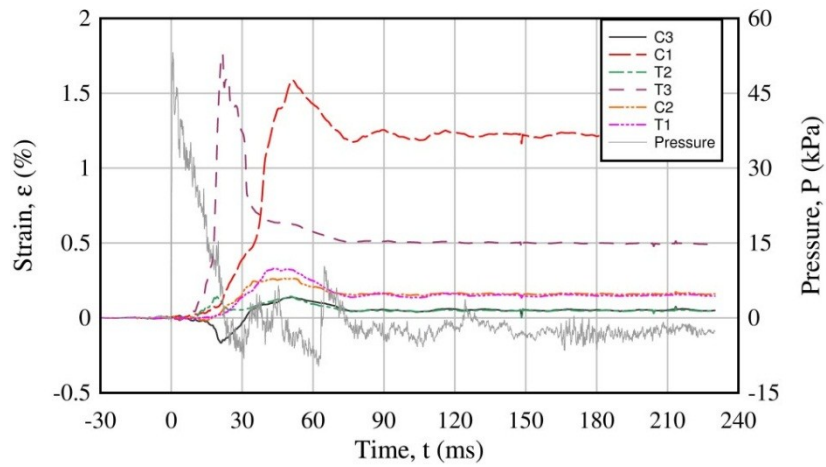
Figure B-3: Column 1NS-J Strain-Time Histories



a) Shot 1

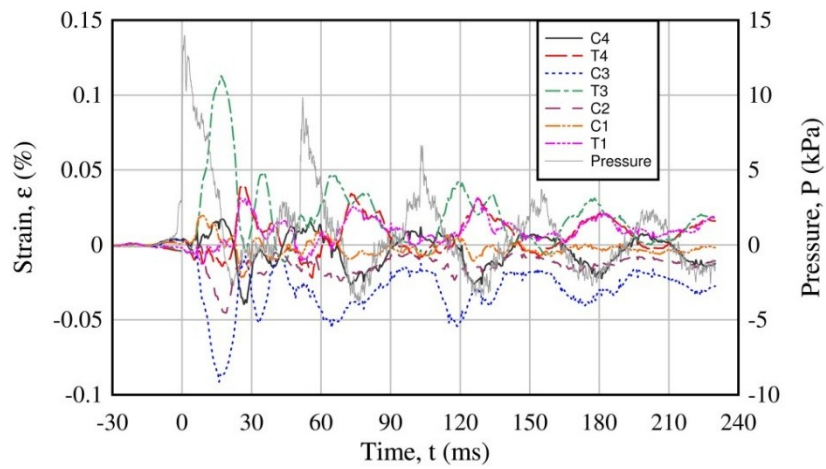


b) Shot 2

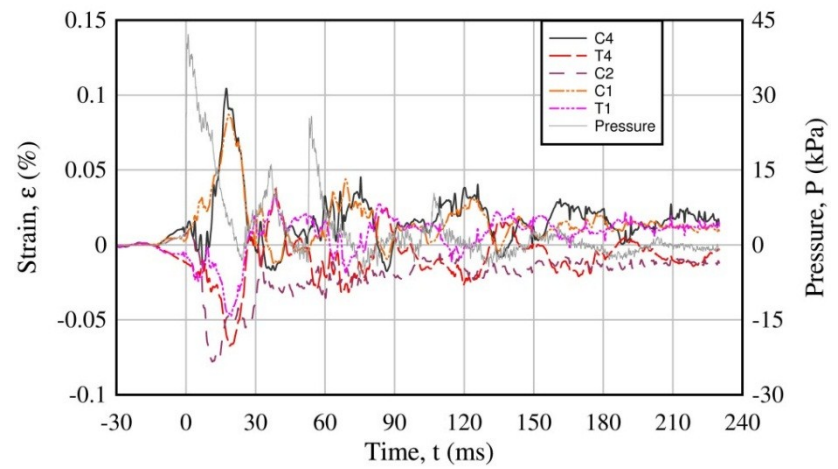


c) Shot 3

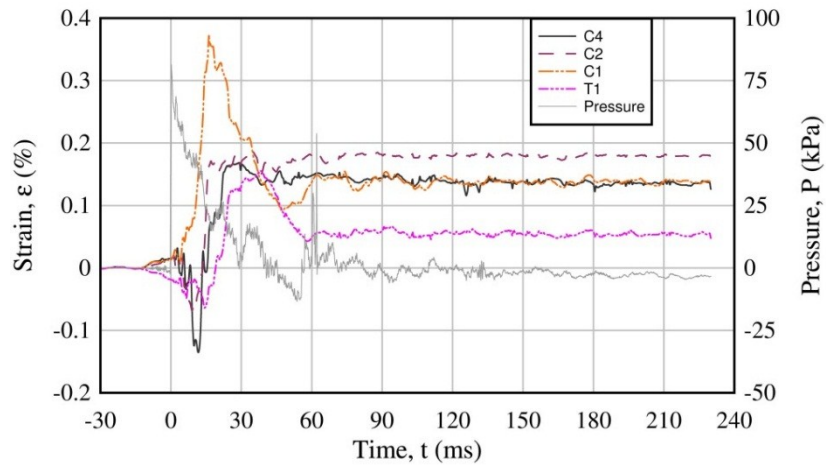
Figure B-4: Column 1S-J Strain-Time Histories



a) Shot 1

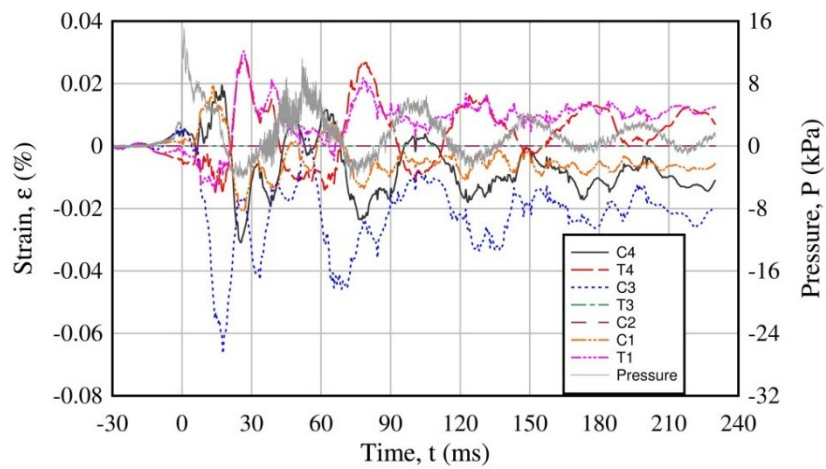


b) Shot 2

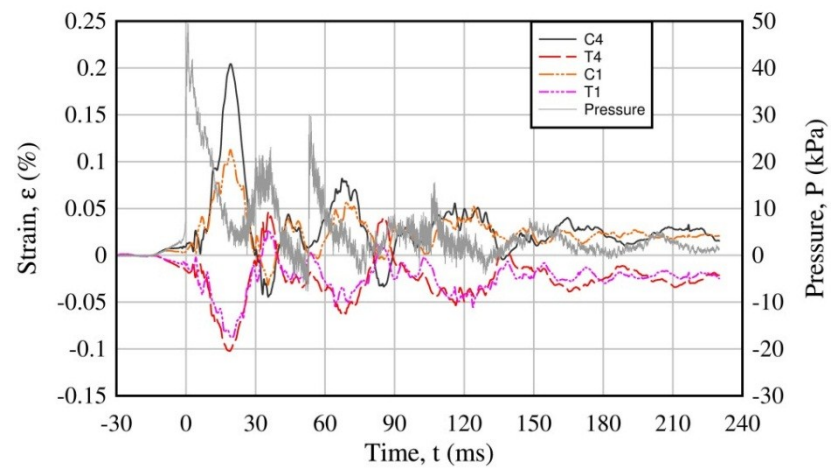


c) Shot 3

Figure B-5: Column 2NS-C Strain-Time Histories

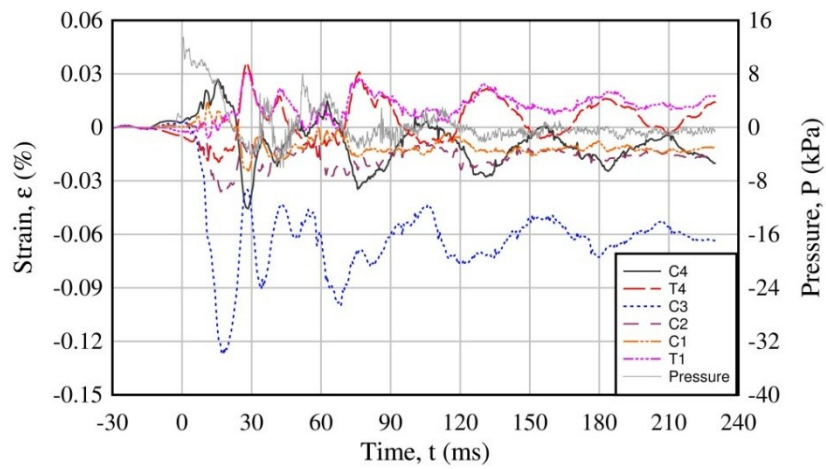


a) Shot 1

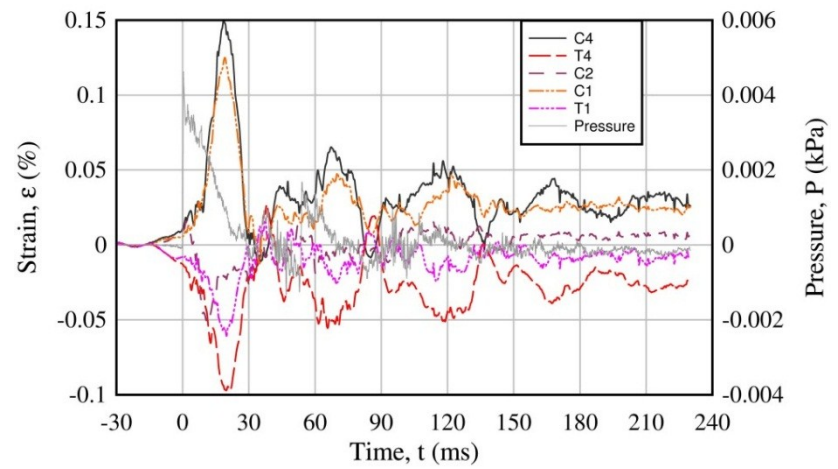


b) Shot 2

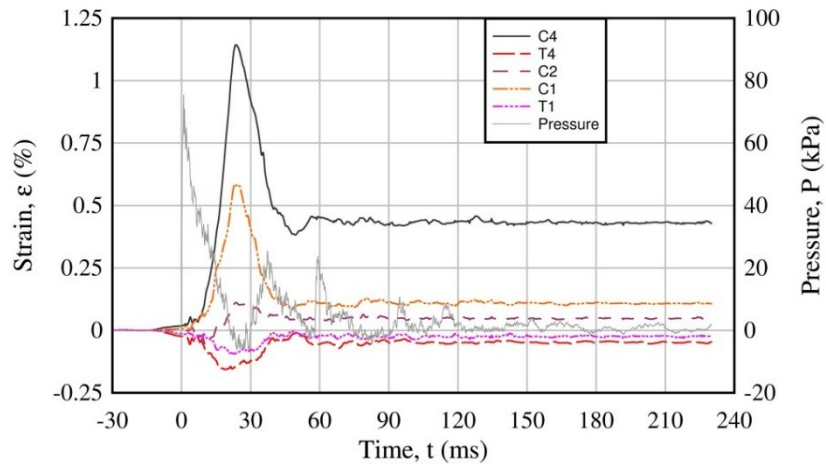
Figure B-6: Column 2S-C Strain-Time Histories



a) Shot 1

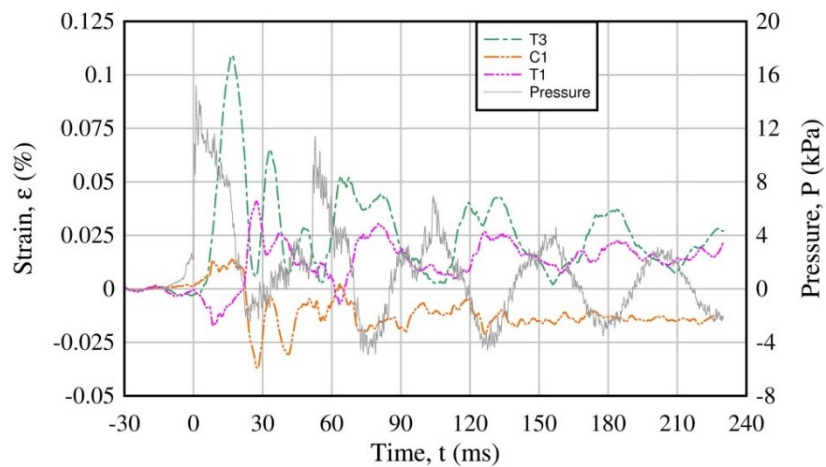


b) Shot 2

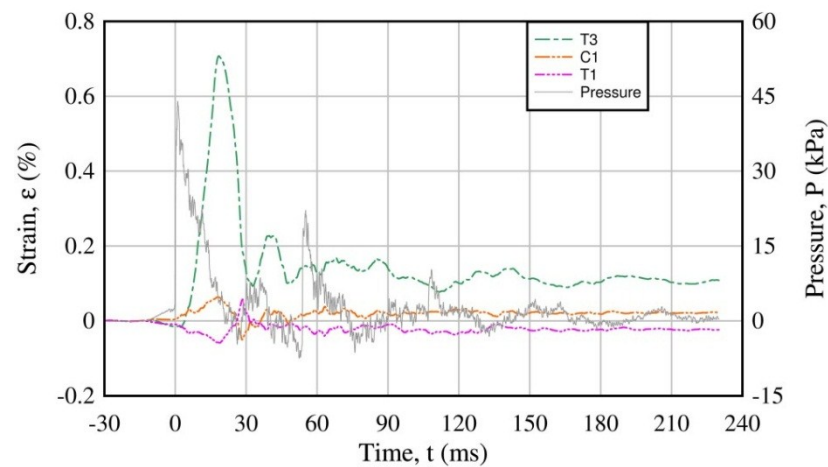


c) Shot 3

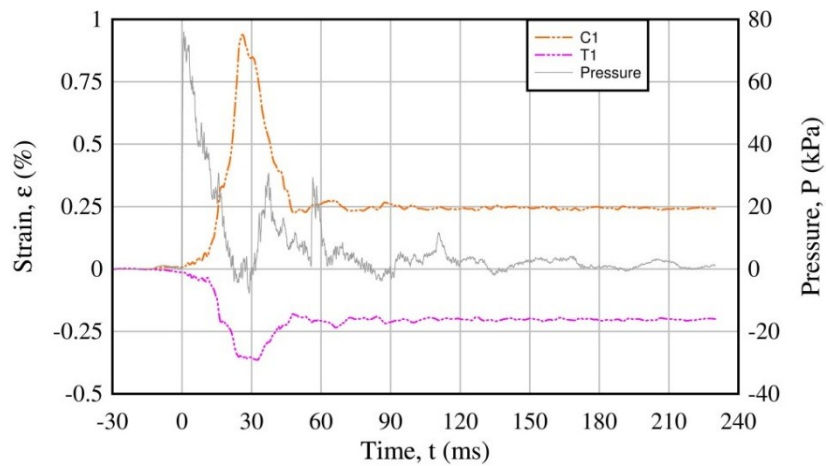
Figure B-7: Column 2NS-J Strain-Time Histories



a) Shot 1

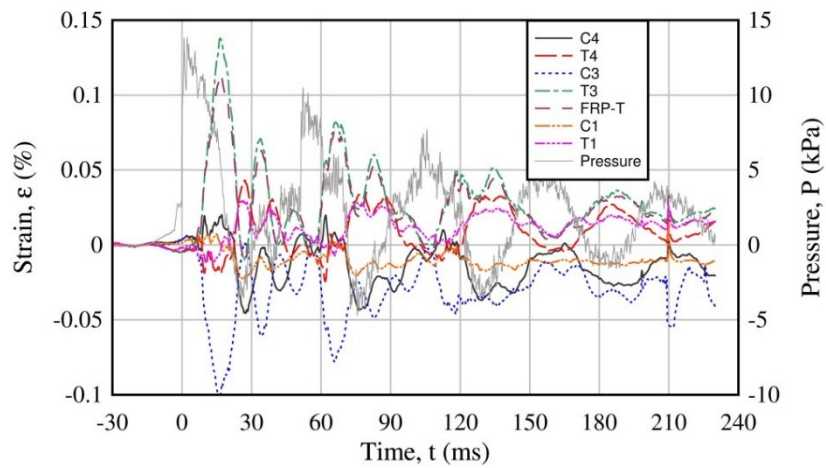


b) Shot 2

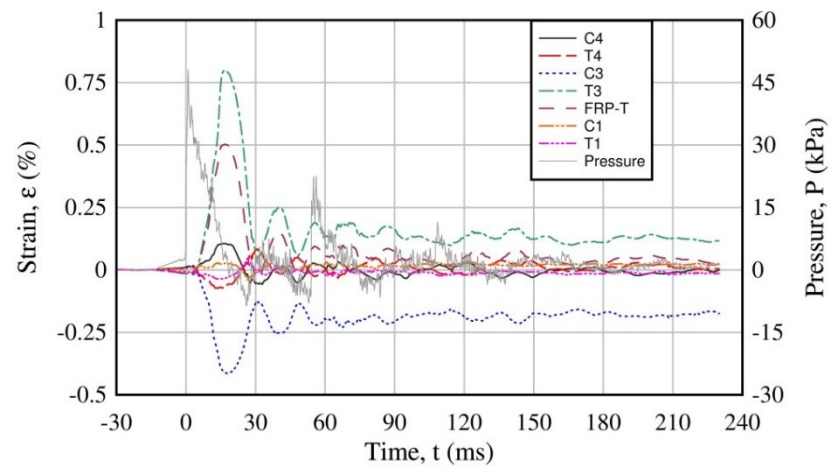


c) Shot 3

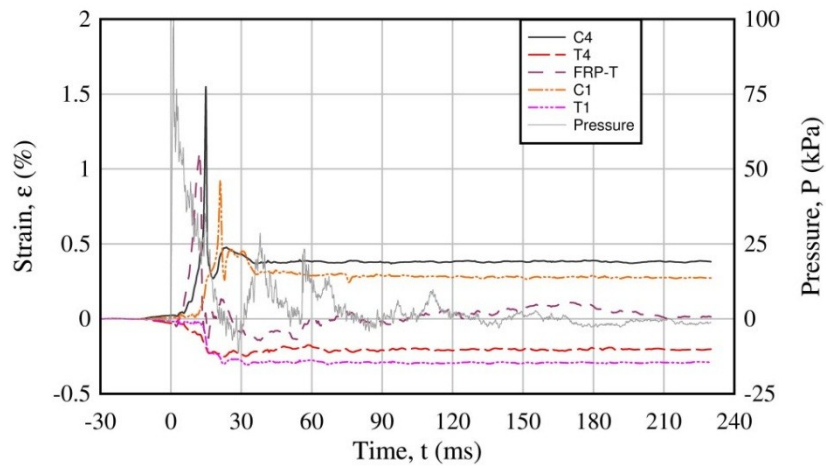
Figure B-8: Column 2S-J Strain-Time Histories



a) Shot 1

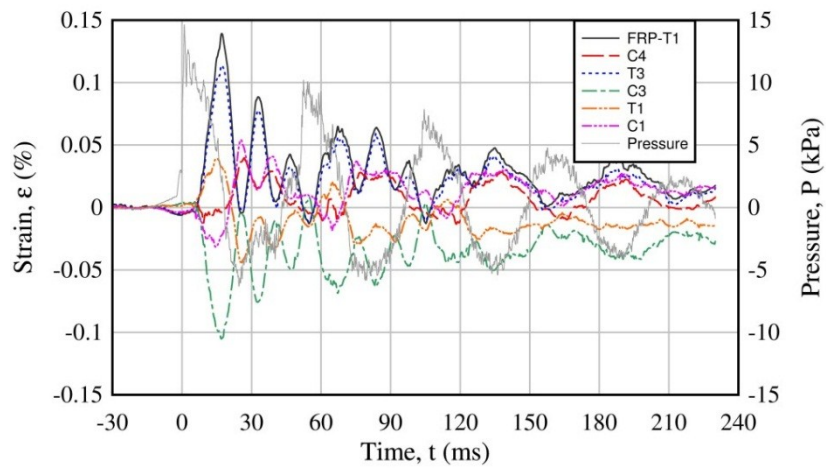


b) Shot 2

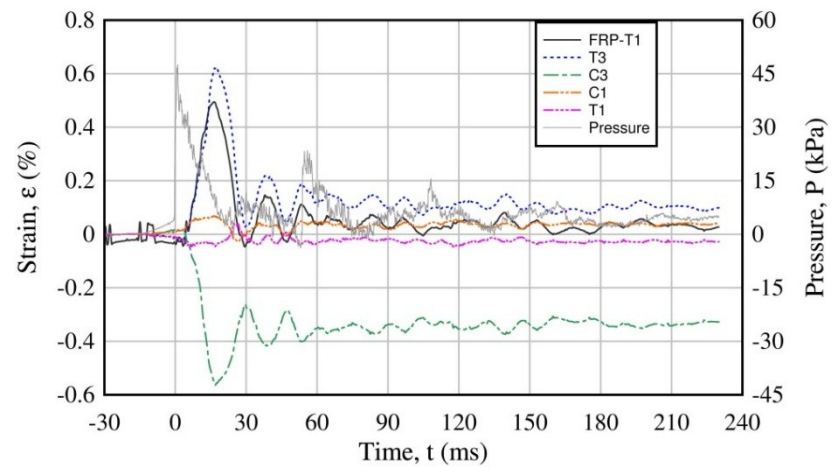


c) Shot 3

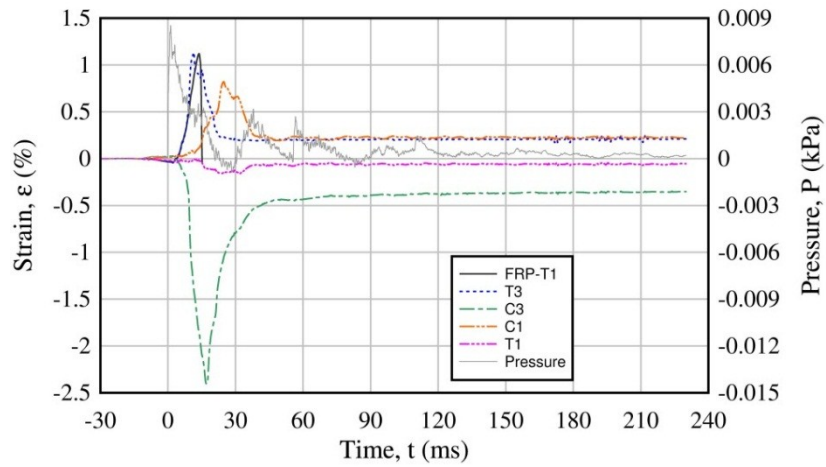
Figure B-9: Column 2NS-R Strain-Time Histories



a) Shot 1

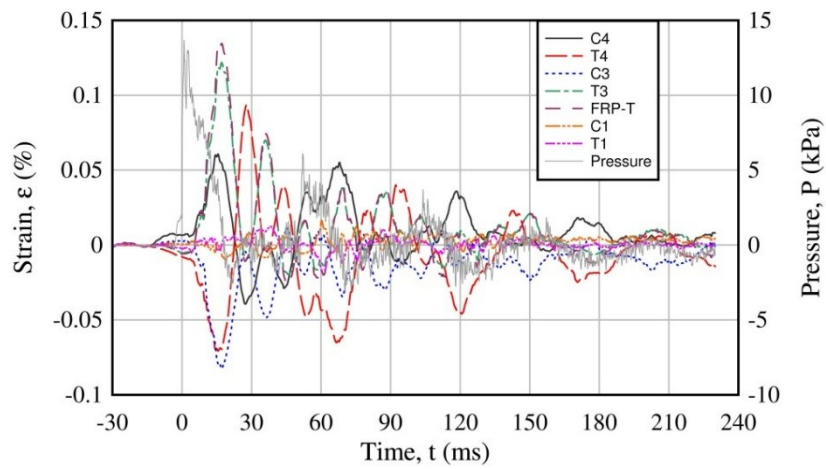


b) Shot 2

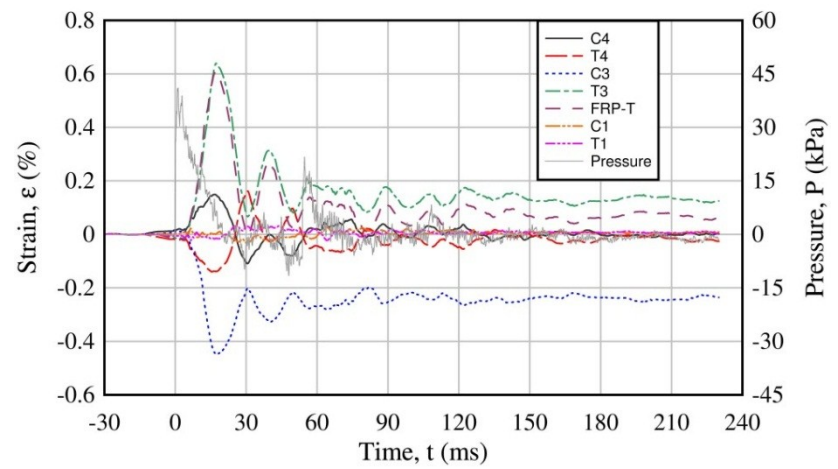


c) Shot 3

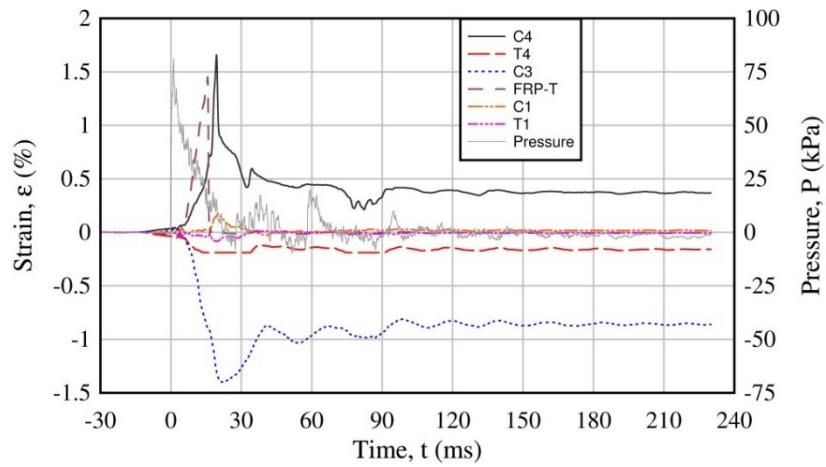
Figure B-10: Column 2S-R Strain-Time Histories



a) Shot 1

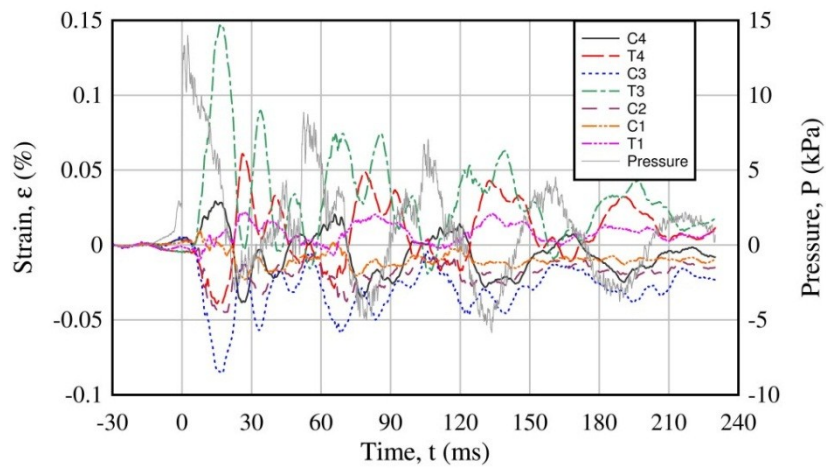


b) Shot 2

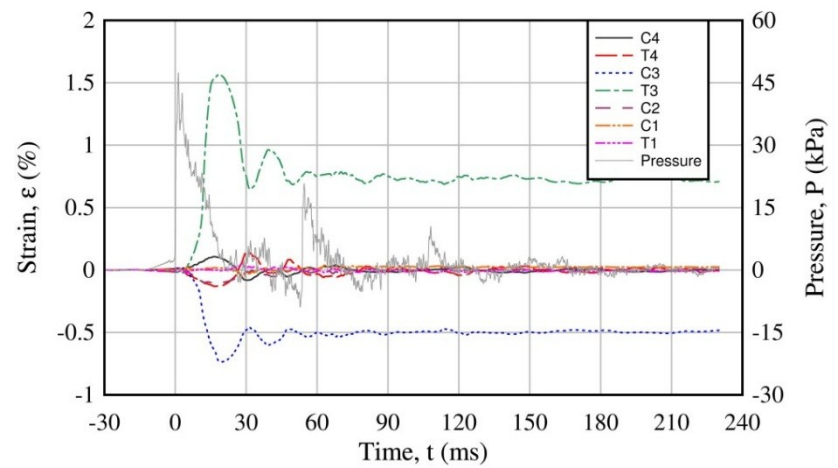


c) Shot 3

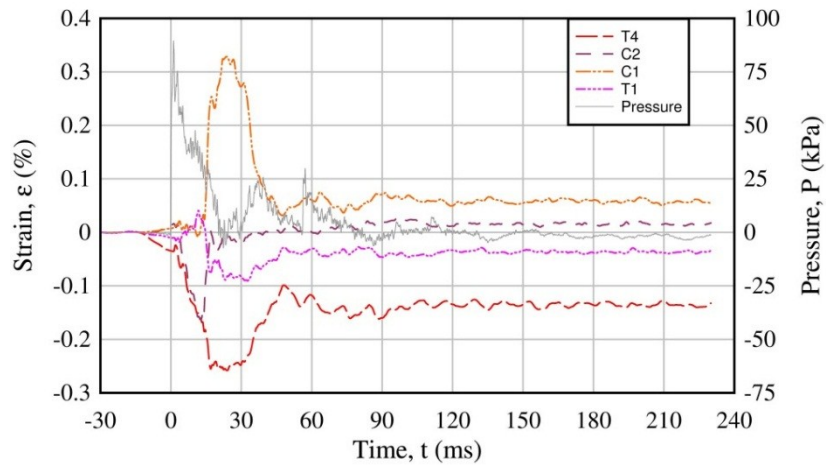
Figure B-11: Column 2NS-JR Strain-Time Histories



a) Shot 1

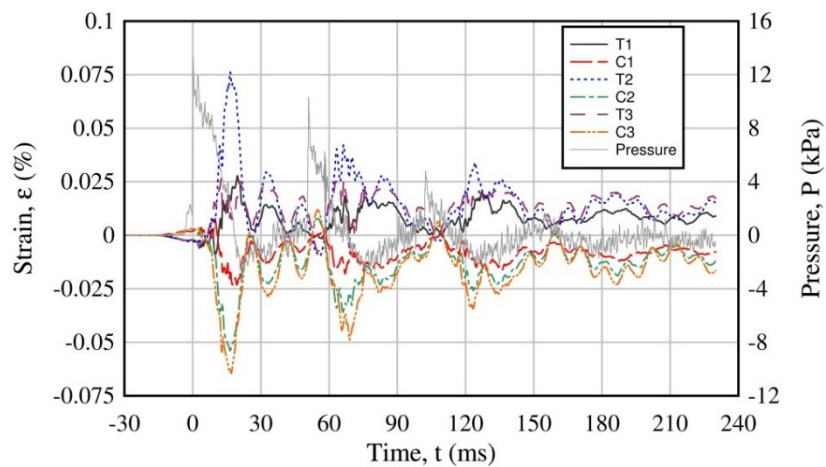


b) Shot 2

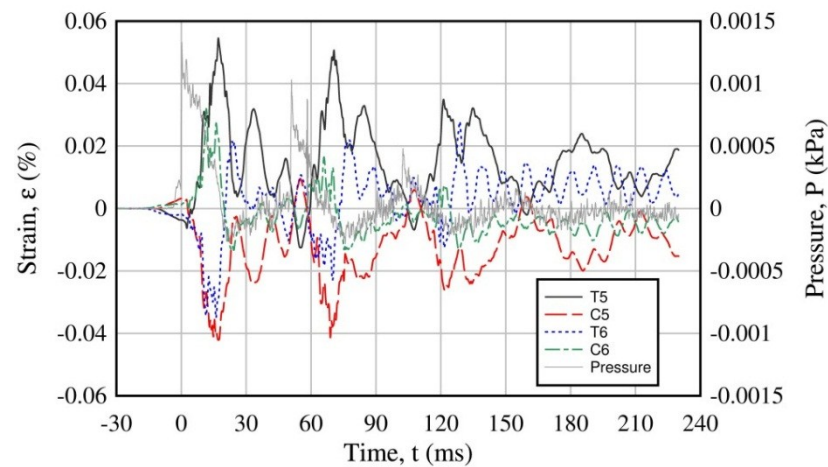


c) Shot 3

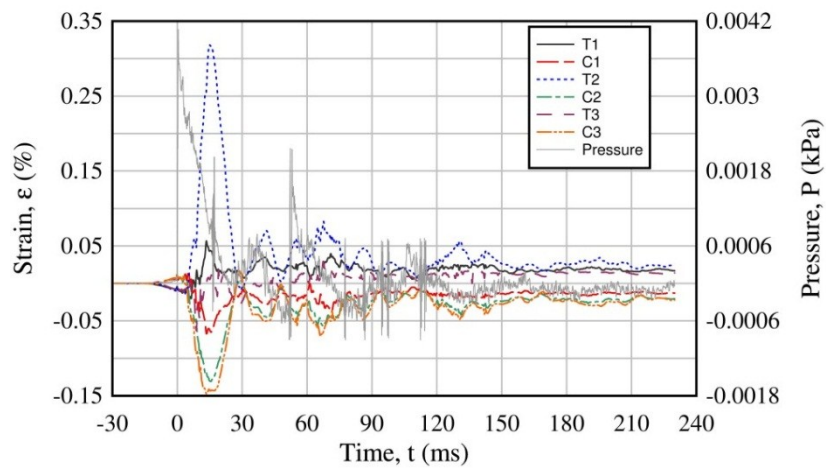
Figure B-12: Column 2S-JR Strain-Time Histories



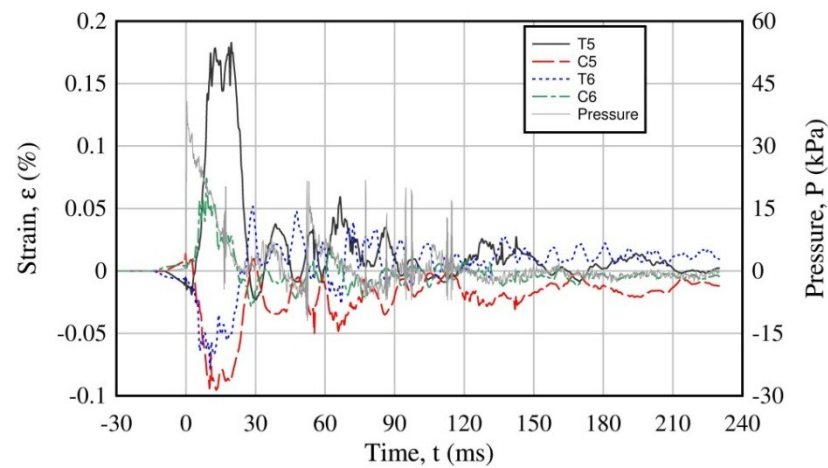
a) Shot 1



b) Shot 1 - Continued



c) Shot 2



d) Shot 2 - Continued

Figure B-13: Column 3NS-C Strain-Time Histories Shot 1 and Shot 2

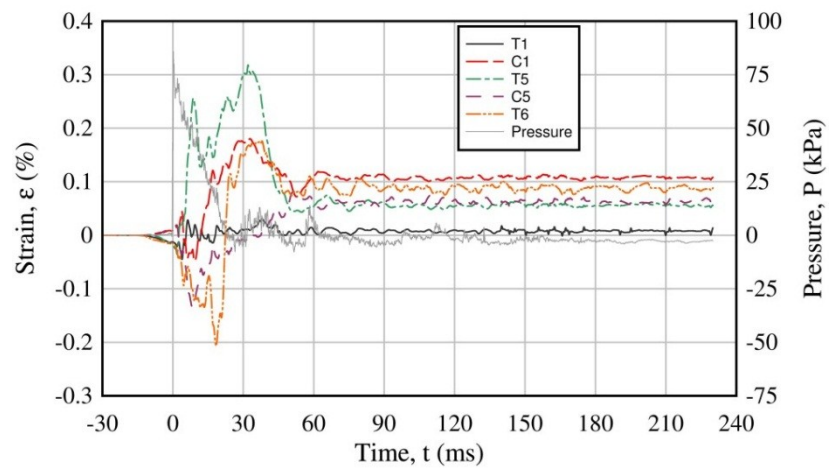
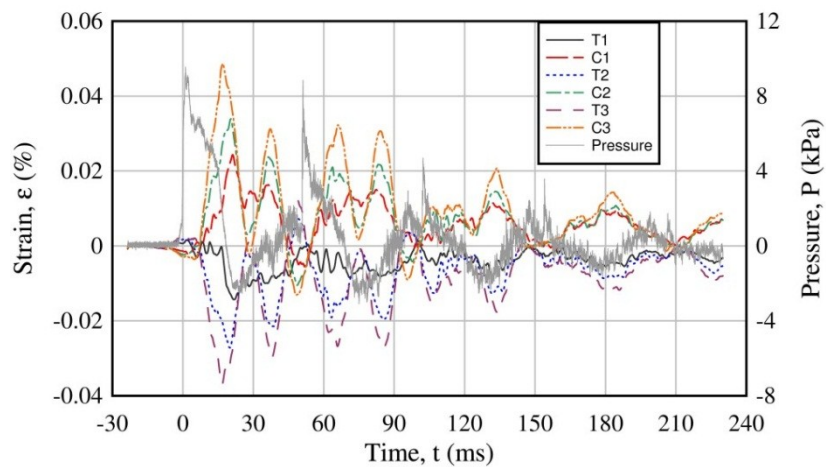
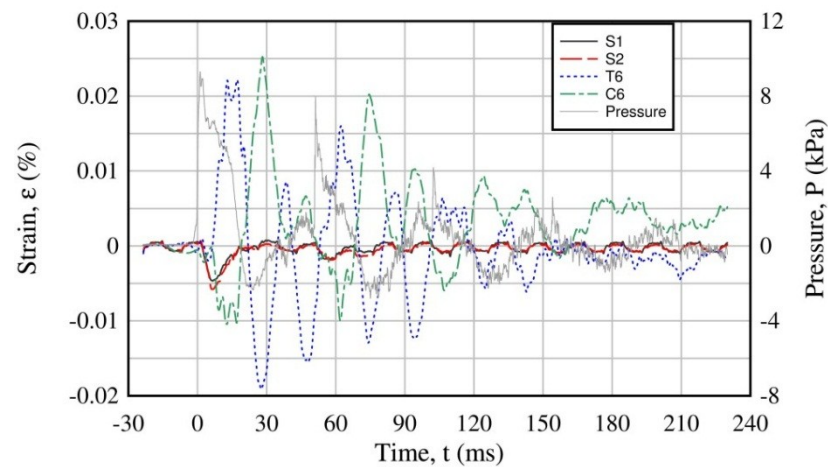


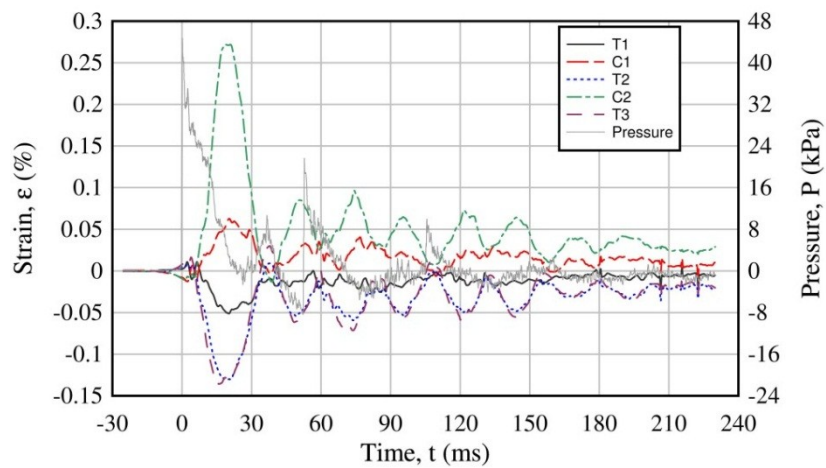
Figure B-14: Column 3NS-C Strain-Time Histories Shot 3



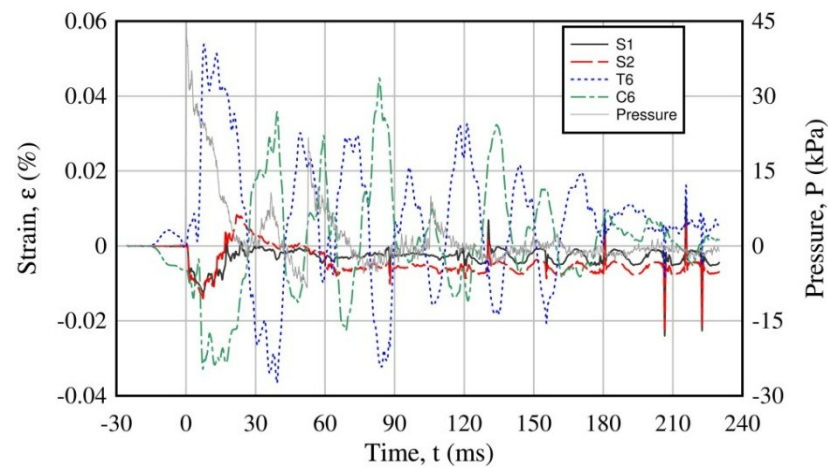
a) Shot 1



b) Shot 1 - Continued

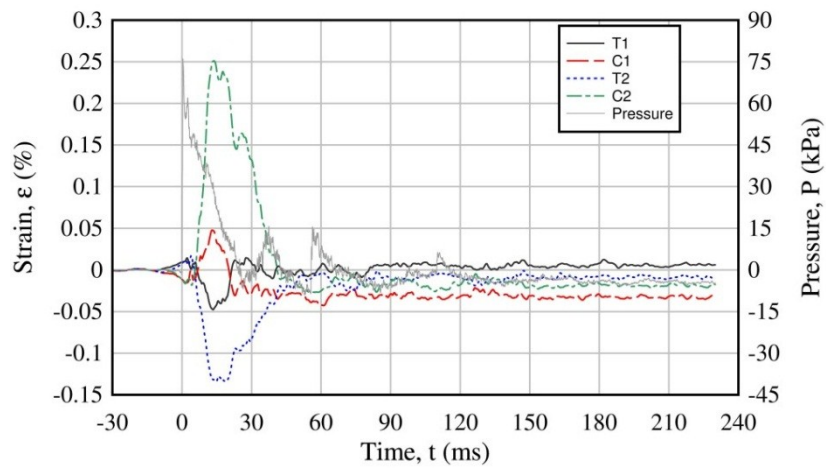


c) Shot 2

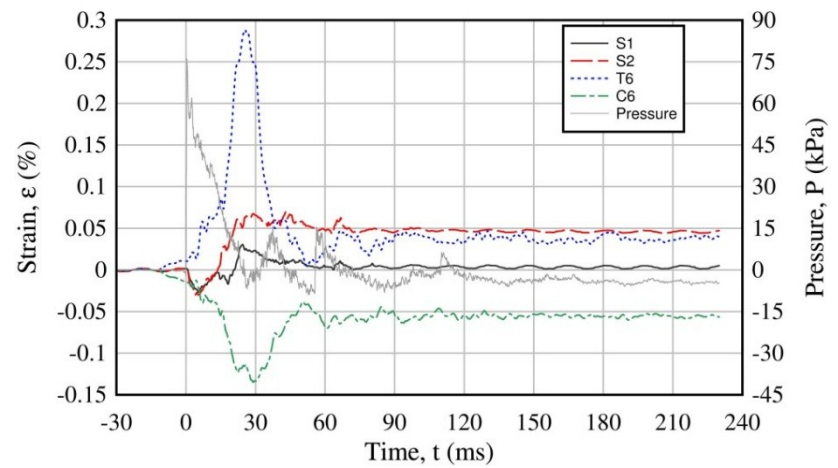


d) Shot 2 - Continued

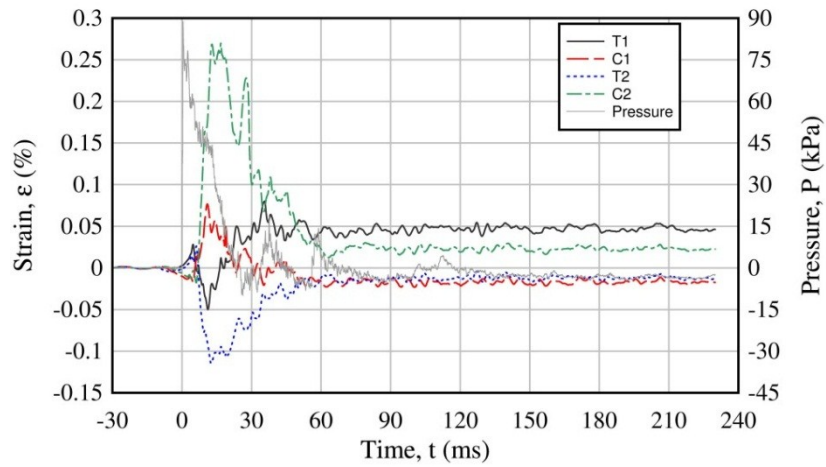
Figure B-15: Column 3NS-W Strain-Time Histories Shot 1 and Shot 2



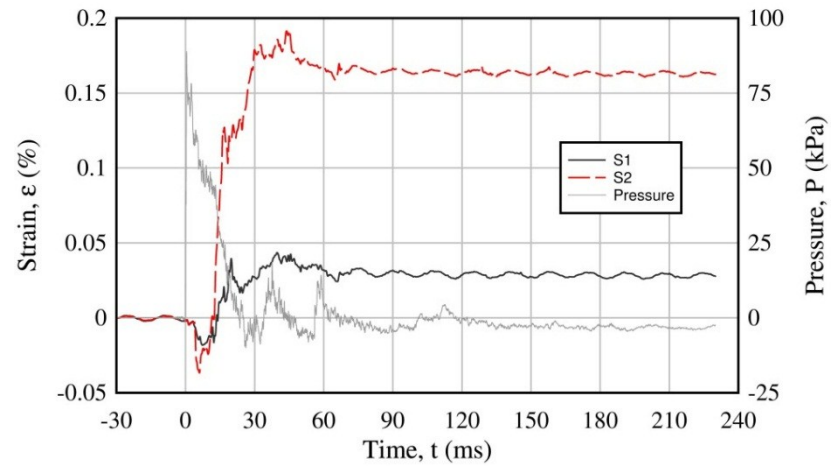
a) Shot 3



b) Shot 3 - Continued

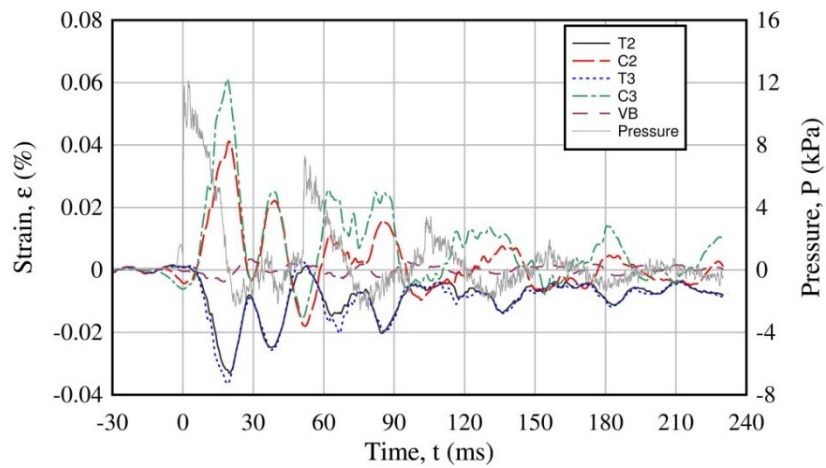


c) Shot 4

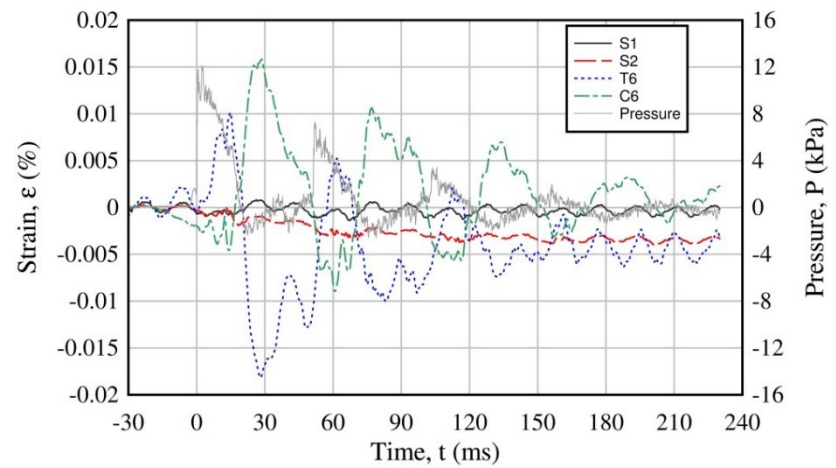


d) Shot 4 - Continued

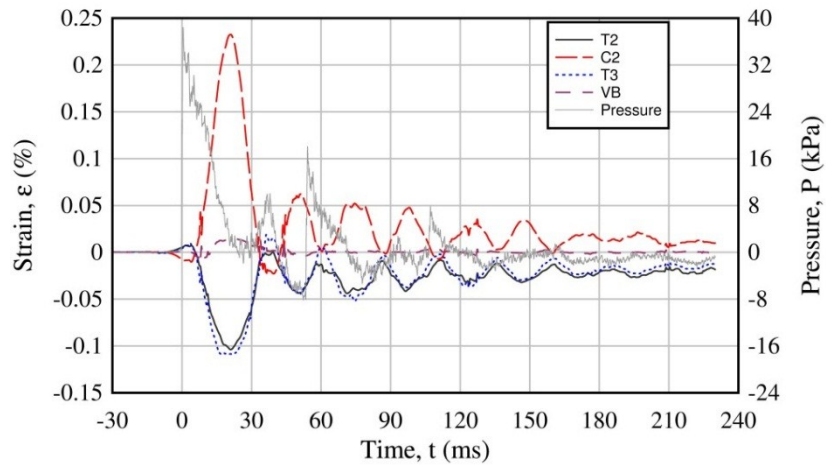
Figure B-16: Column 3NS-W Strain-Time Histories Shot 3 and Shot 4



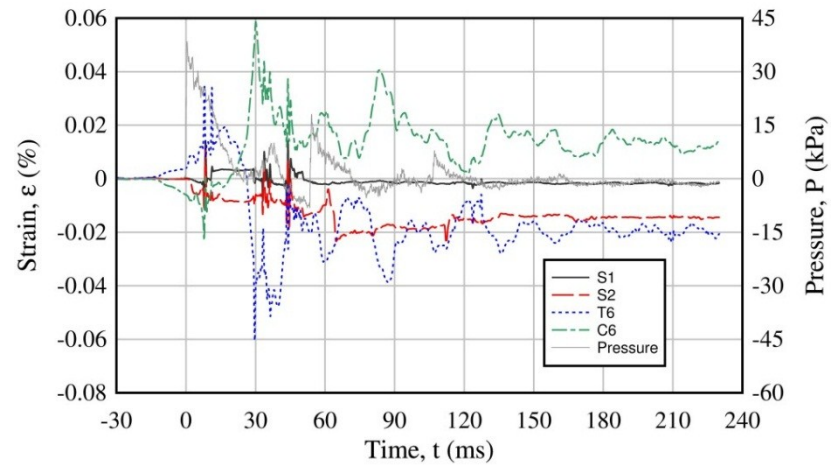
a) Shot 1



b) Shot 1 - Continued

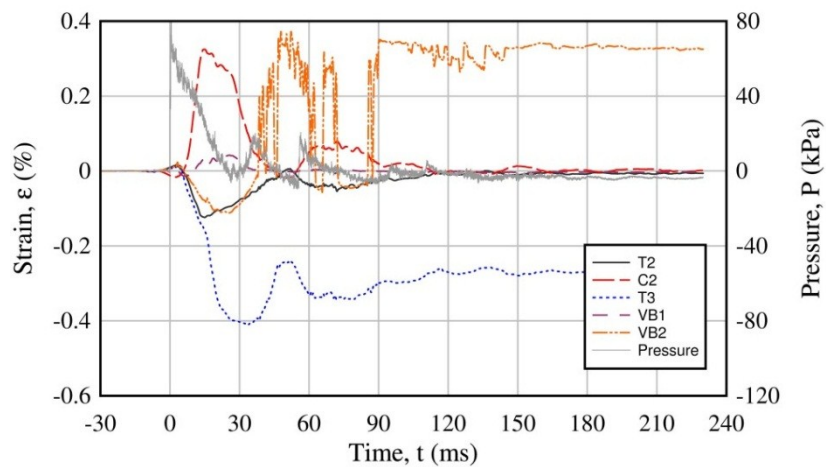


c) Shot 2

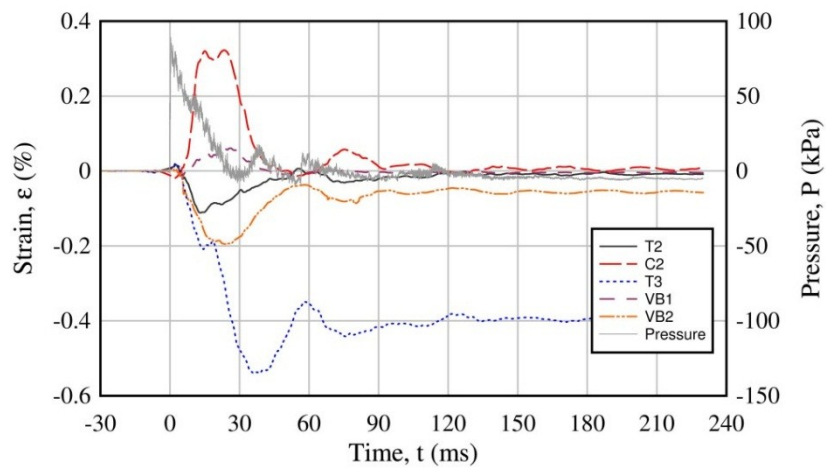


d) Shot 2 - Continued

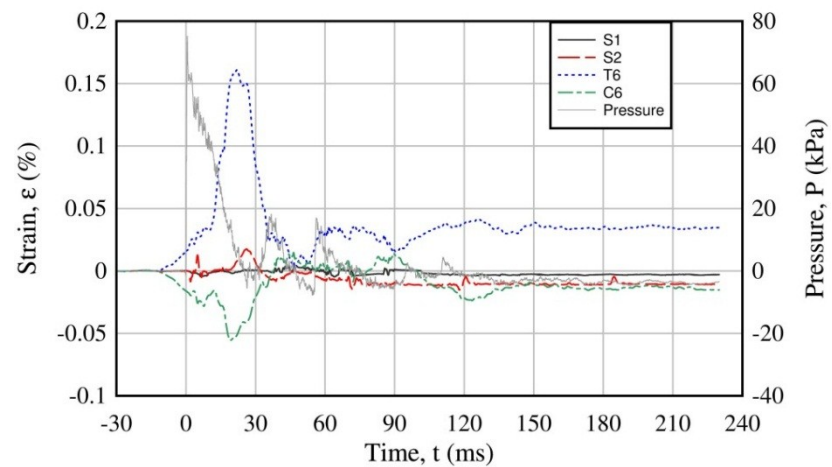
Figure B-17: Column 3NS-CB Strain-Time Histories Shot 1 and Shot 2



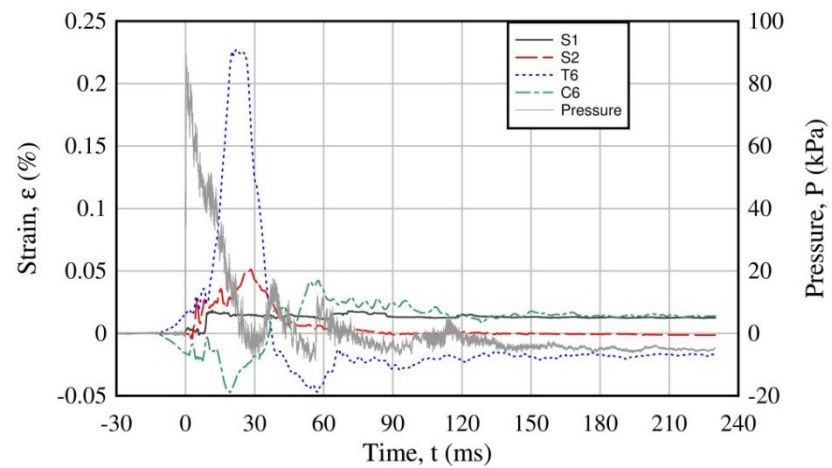
a) Shot 3



c) Shot 4

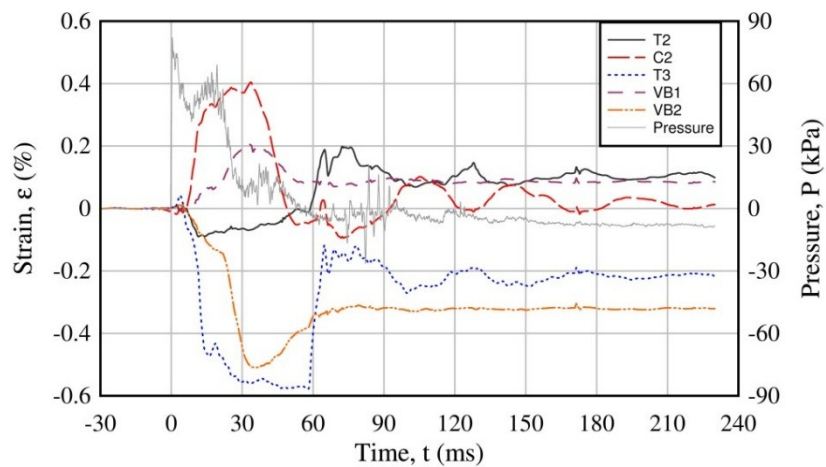


b) Shot 3 - Continued

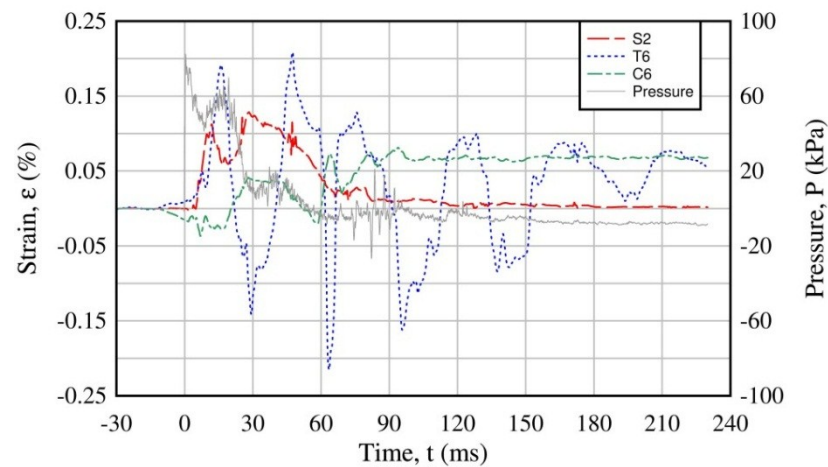


d) Shot 4 - Continued

Figure B-18: Column 3NS-CB Strain-Time Histories Shot 3 and Shot 4

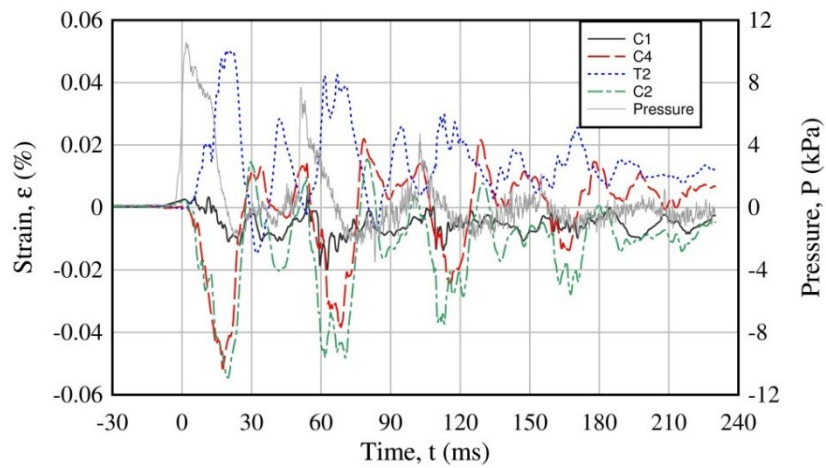


a) Shot 5

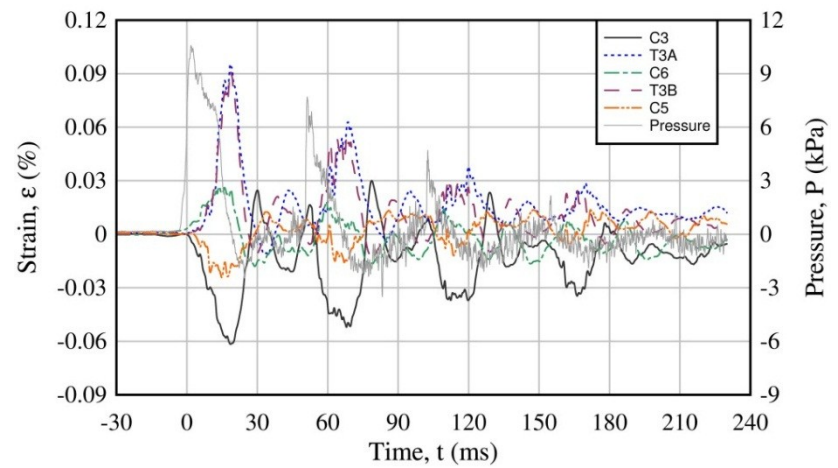


b) Shot 5 - Continued

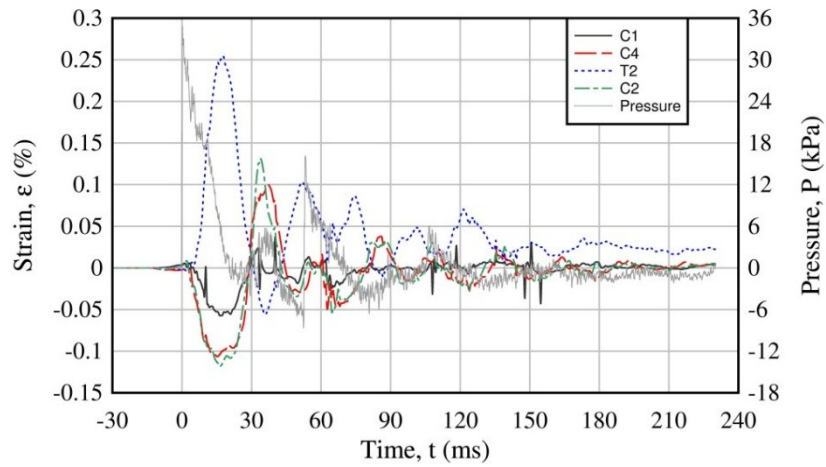
Figure B-19: Column 3NS-CB Strain-Time Histories Shot 5



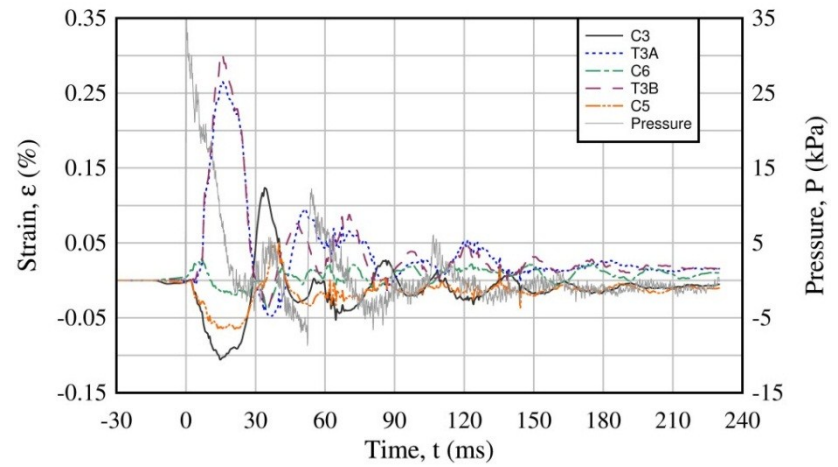
a) Shot 1



b) Shot 1 - Continued

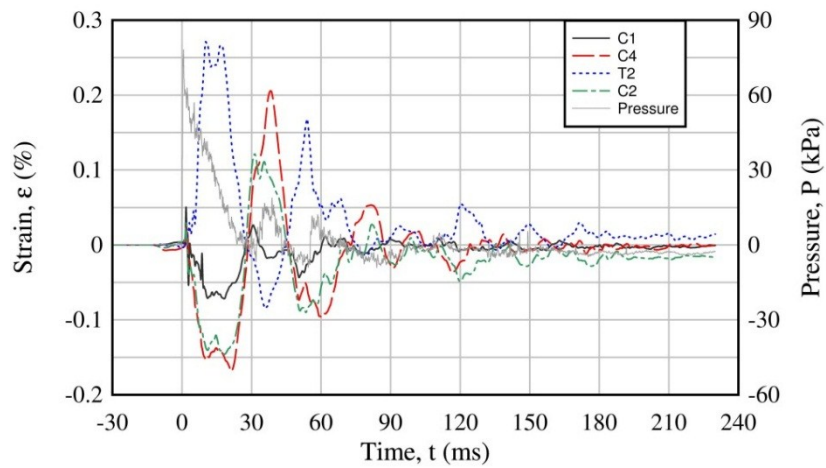


c) Shot 2

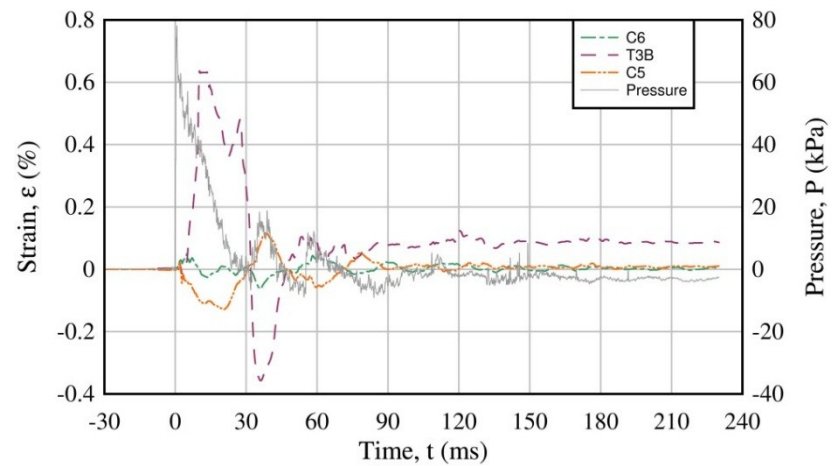


d) Shot 2 - Continued

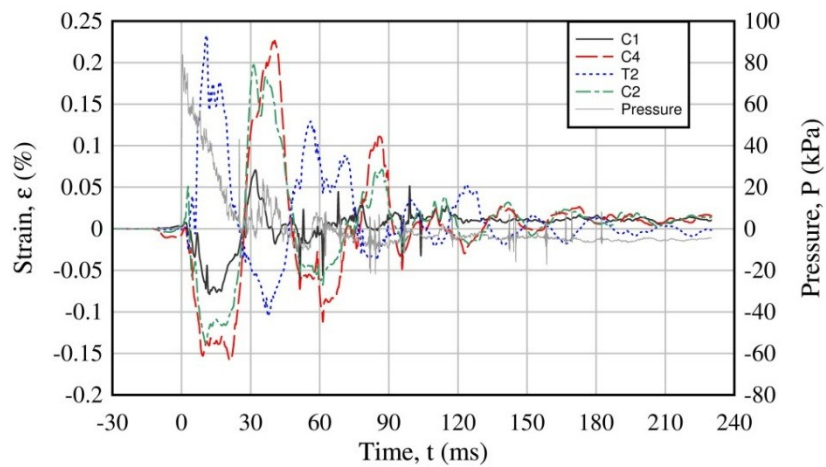
Figure B-20: Column 3NS-TB1 Strain-Time Histories Shot 1 and Shot 2



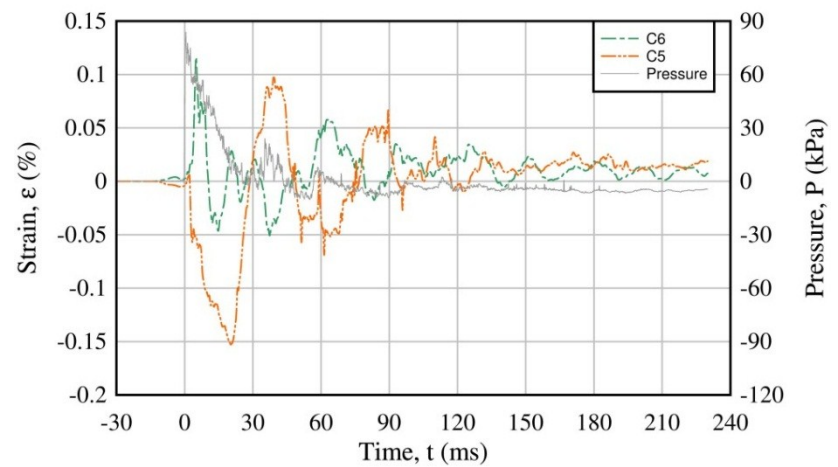
a) Shot 3



b) Shot 3 - Continued



c) Shot 4



d) Shot 4 - Continued

Figure B-21: Column 3NS-TB1 Strain-Time Histories Shot 3 and Shot 4

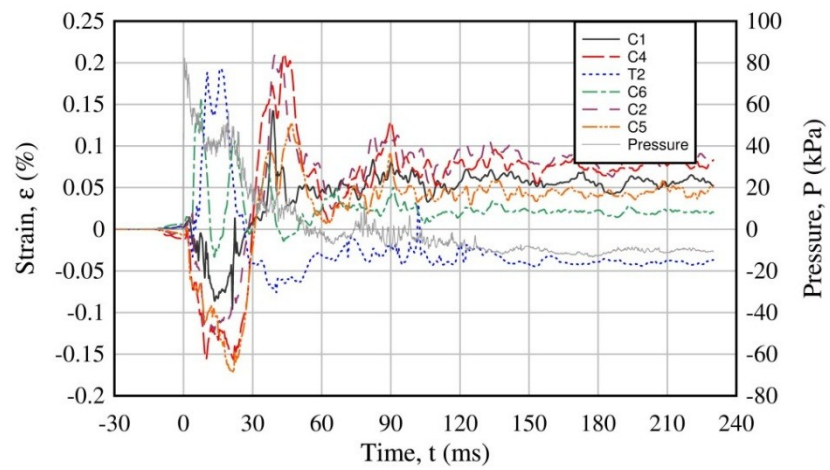
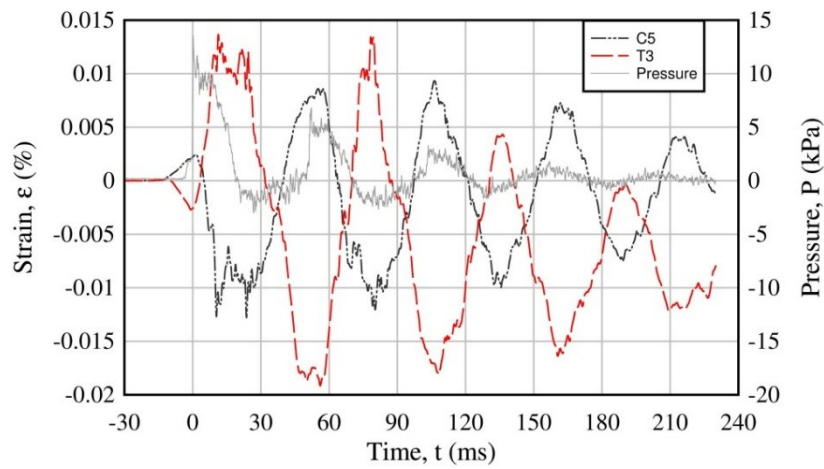
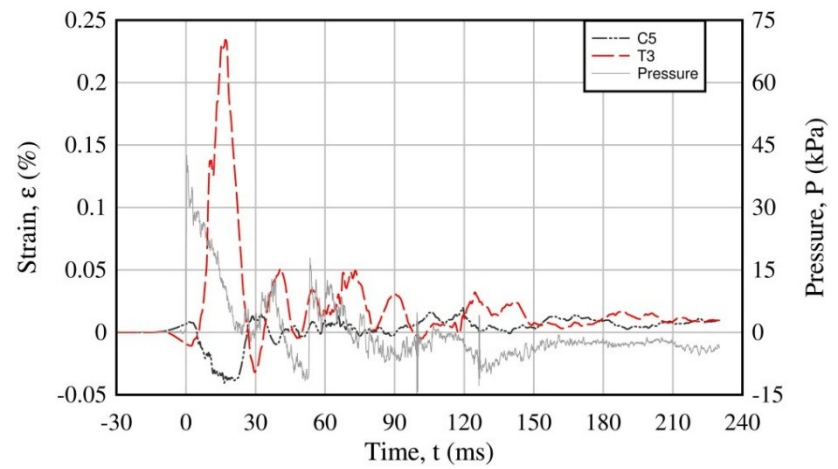


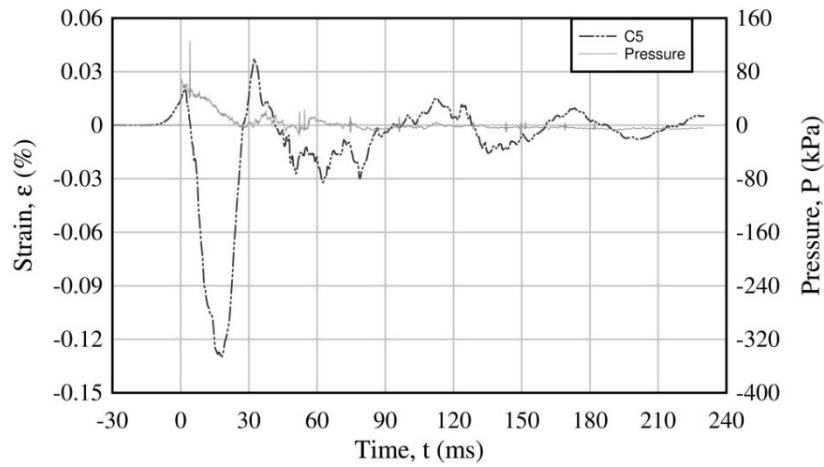
Figure B-22: Column 3NS-TB1 Strain-Time Histories Shot 5



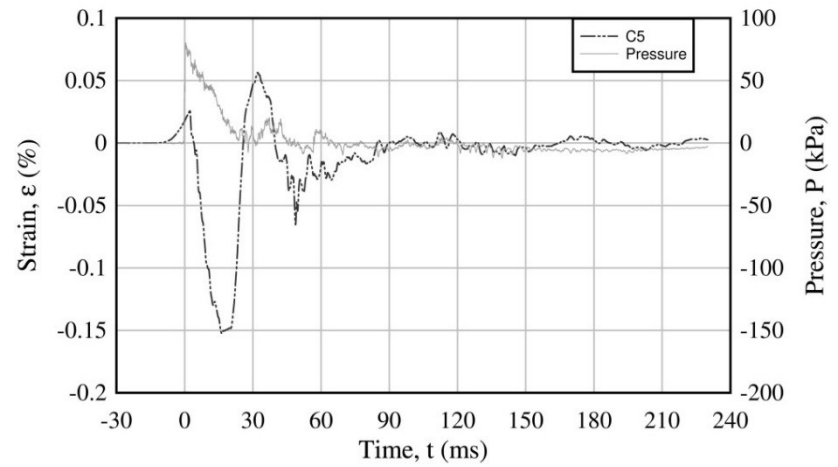
a) Shot 1



b) Shot 2



c) Shot 3



d) Shot 4

Figure B-23: Column 3NS-TB2 Strain-Time Histories Shot 1, Shot 2, Shot 3 and Shot 4

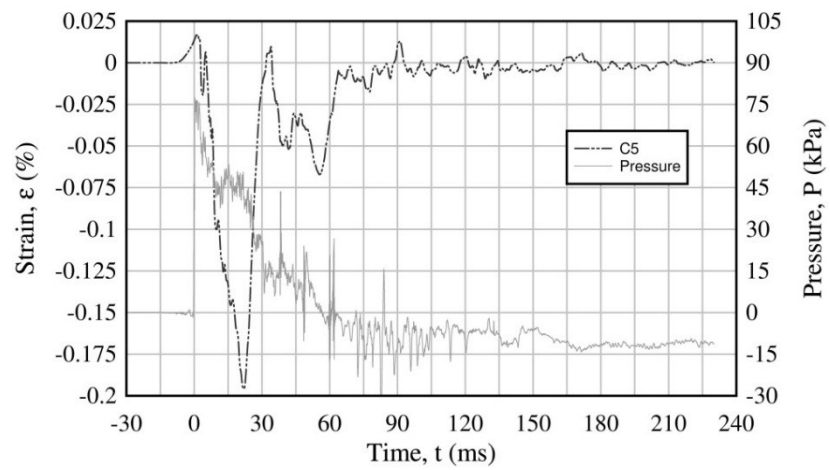
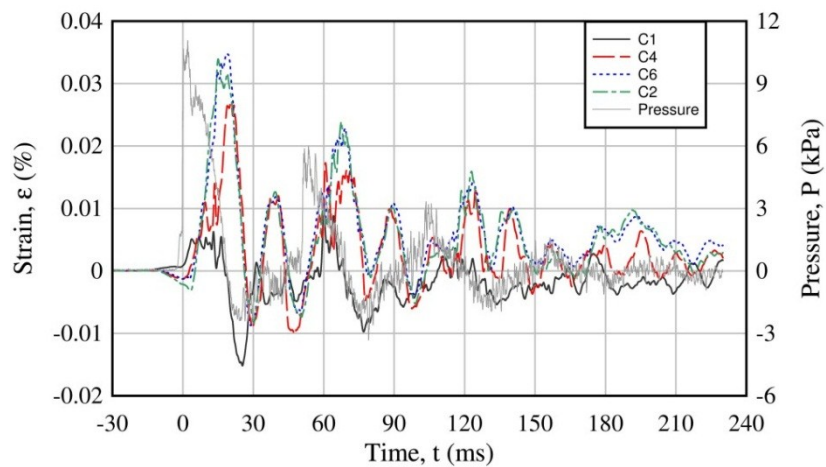
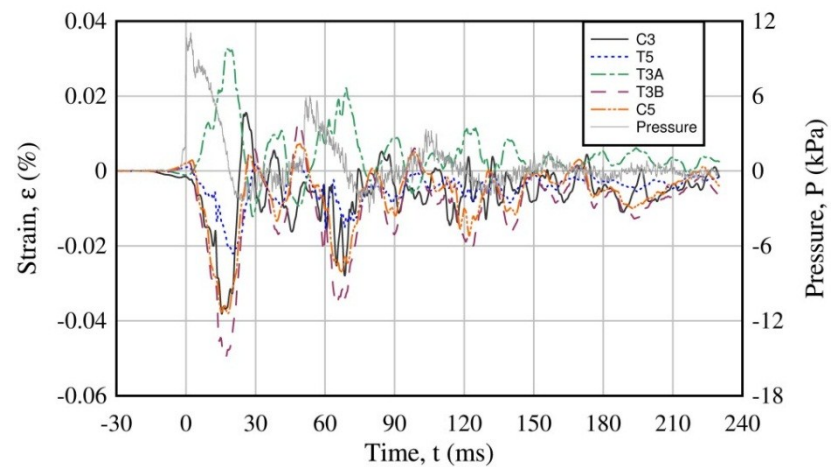


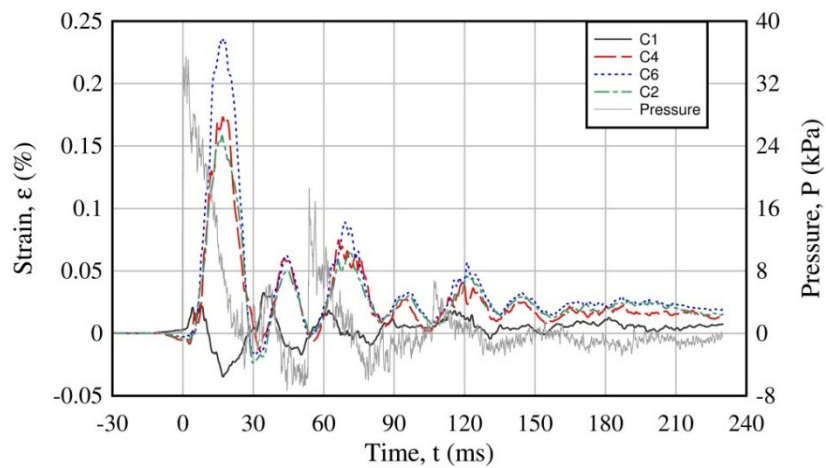
Figure B-24: Column 3NS-TB2 Strain-Time Histories Shot 5



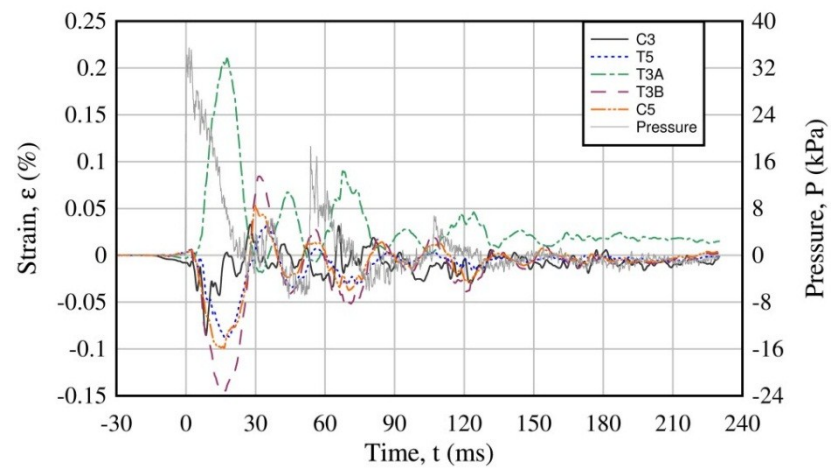
a) Shot 1



b) Shot 1 - Continued

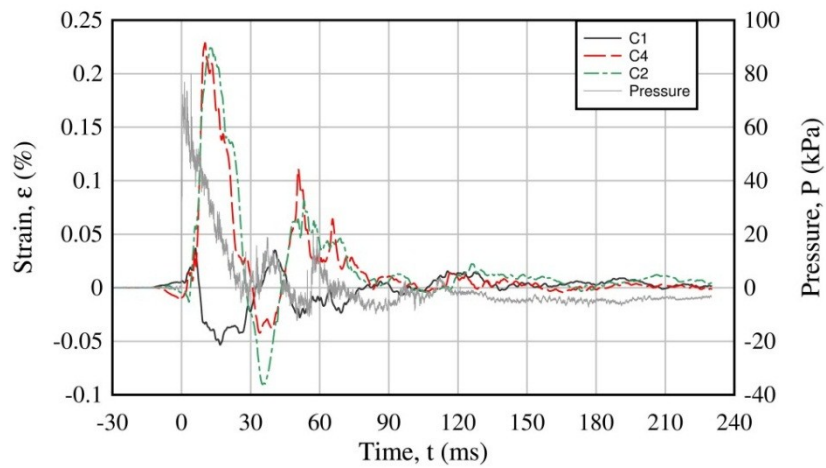


c) Shot 2

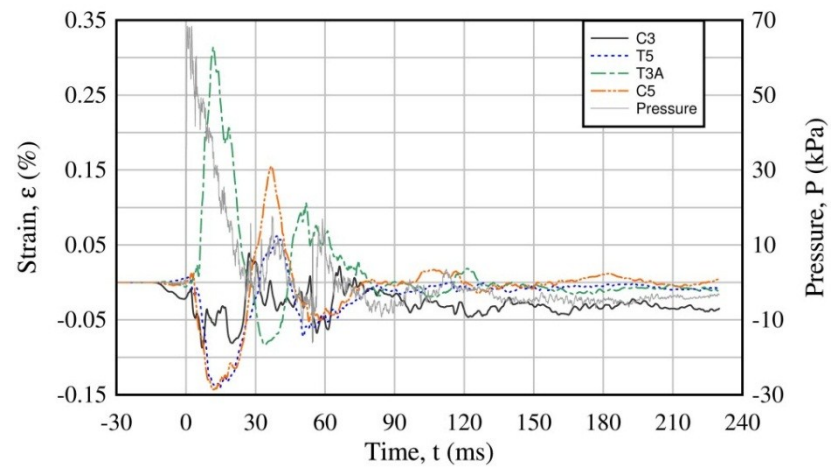


d) Shot 2 - Continued

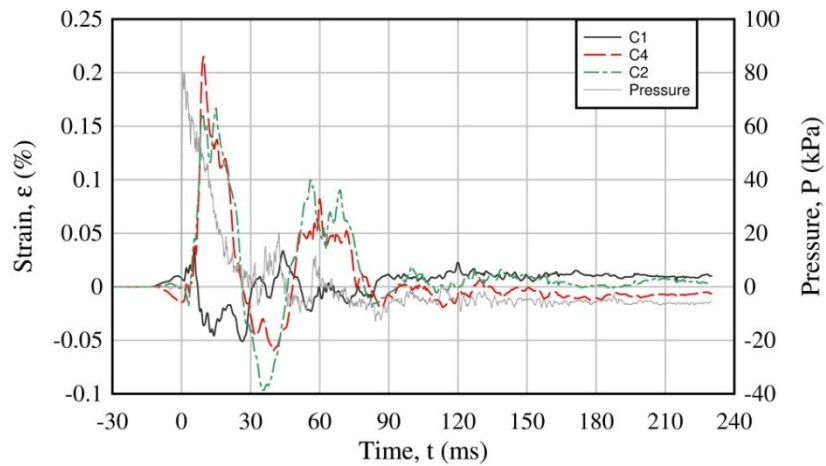
Figure B-25: Column 3NS-TB3 Strain-Time Histories Shot 1 and Shot 2



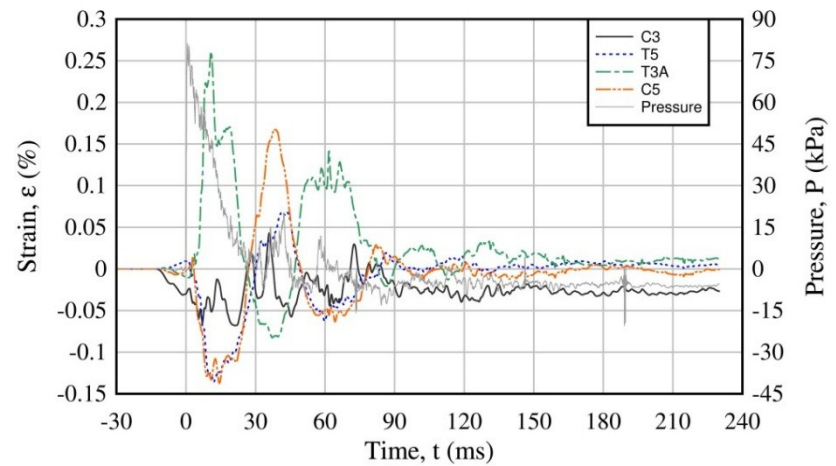
a) Shot 3



b) Shot 3 - Continued



c) Shot 4



d) Shot 4 - Continued

Figure B-26: Column 3NS-TB3 Strain-Time Histories Shot 3 and Shot 4

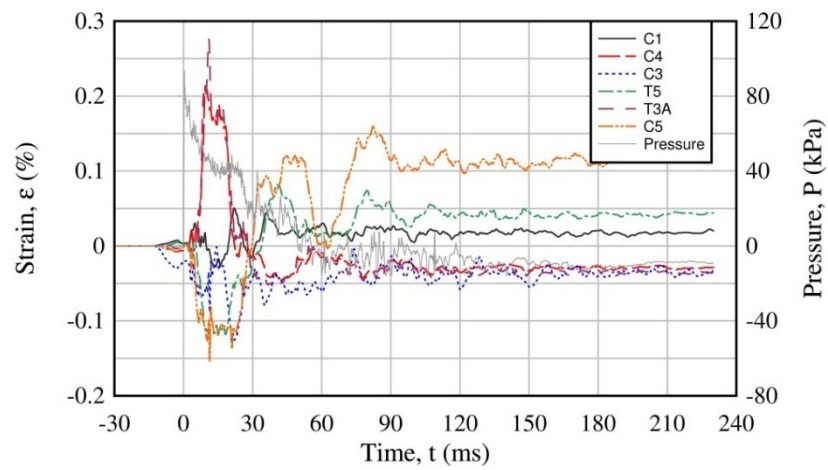
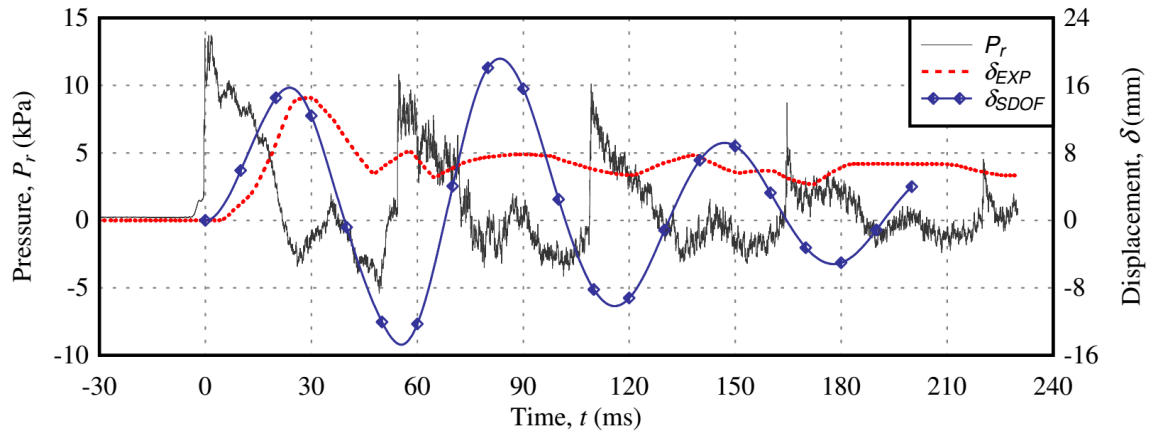
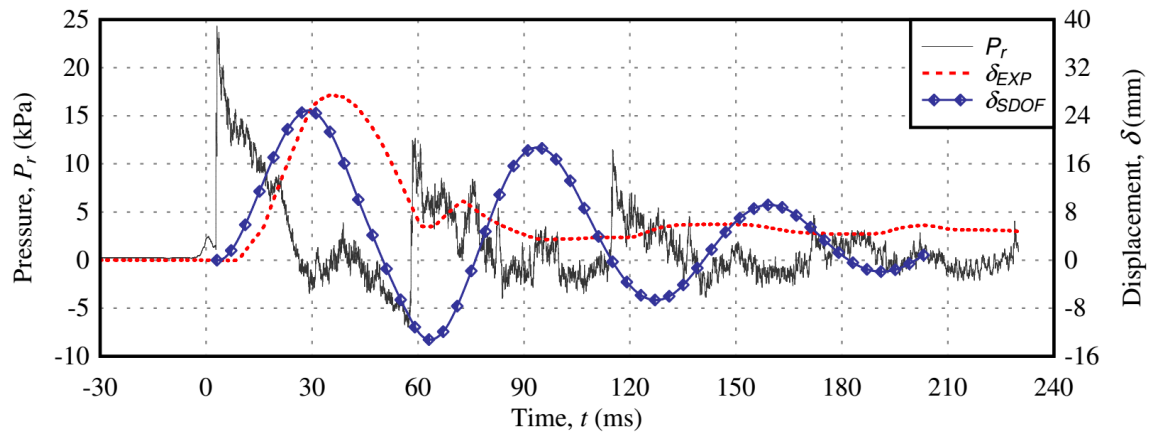


Figure B-27: Column 3NS-TB3 Strain-Time Histories Shot 5

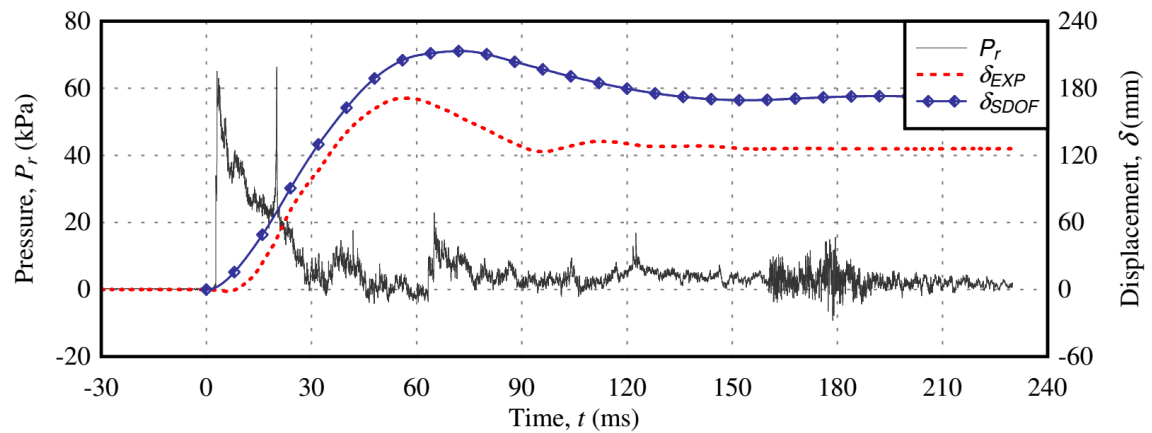
Appendix C - SDOF Displacement Predictions



a) Shot 1

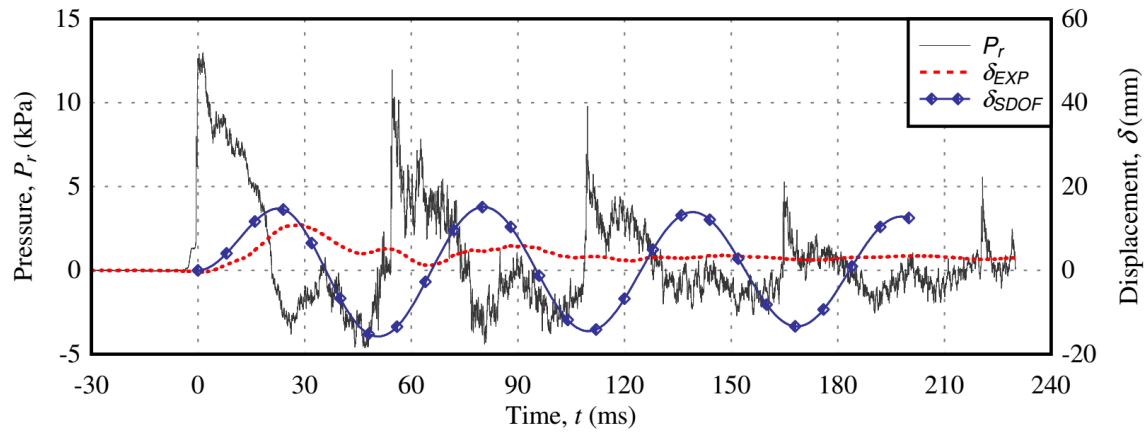


b) Shot 2

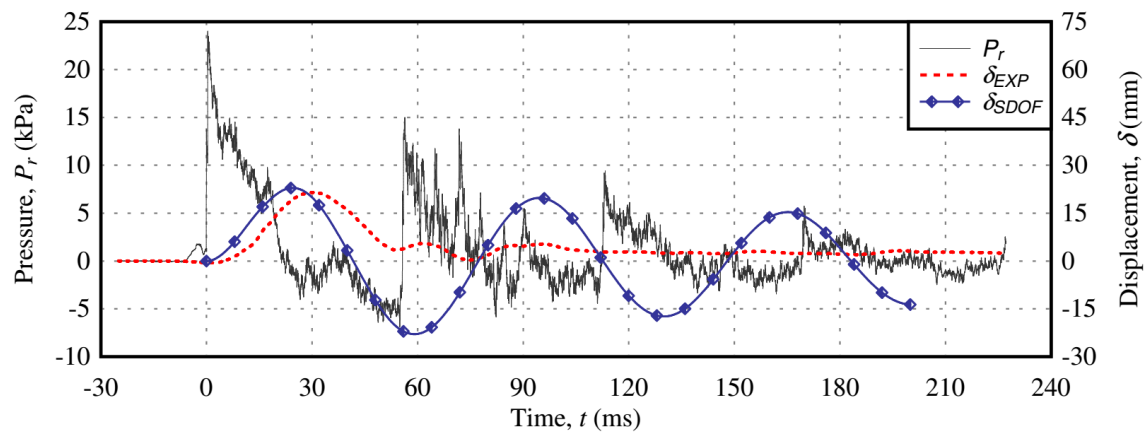


c) Shot 3

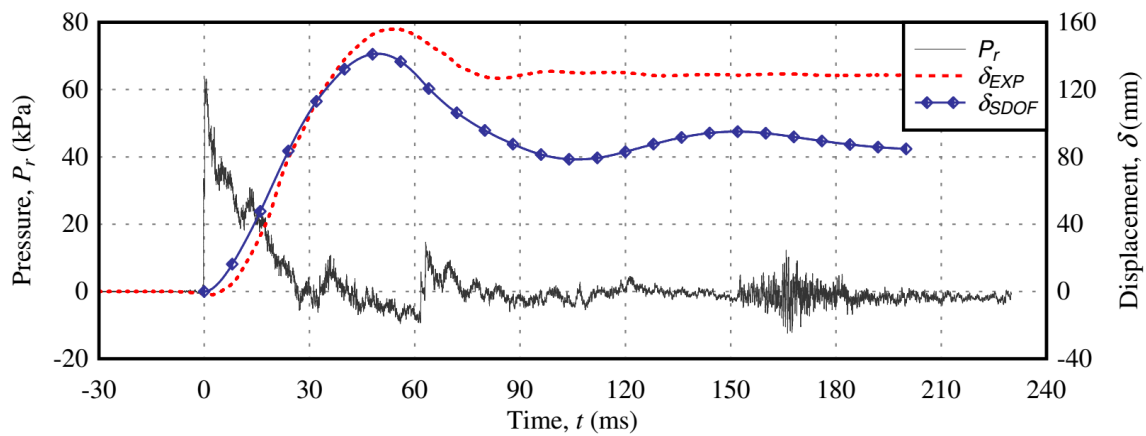
Figure C-1: Column 1NS-C Experimental and Predicted Displacement Time Histories



a) Shot 1

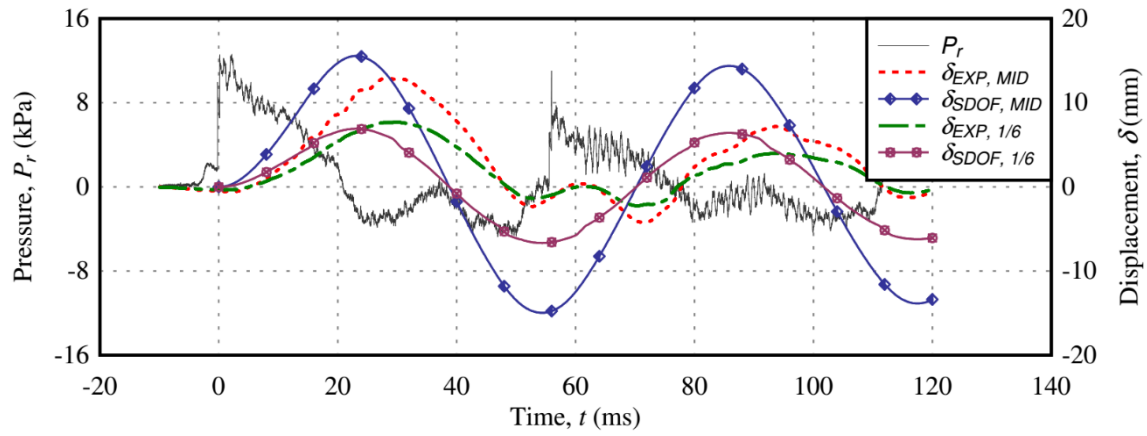


b) Shot 2

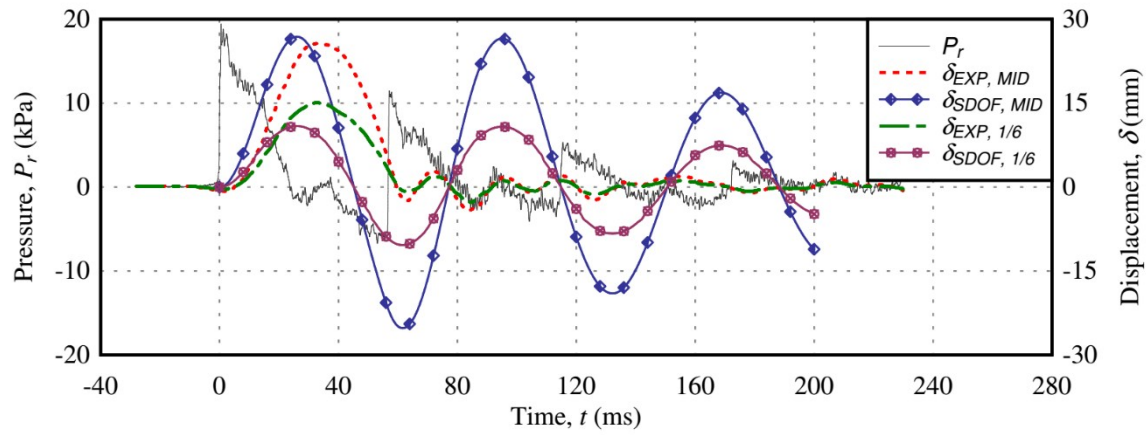


c) Shot 3

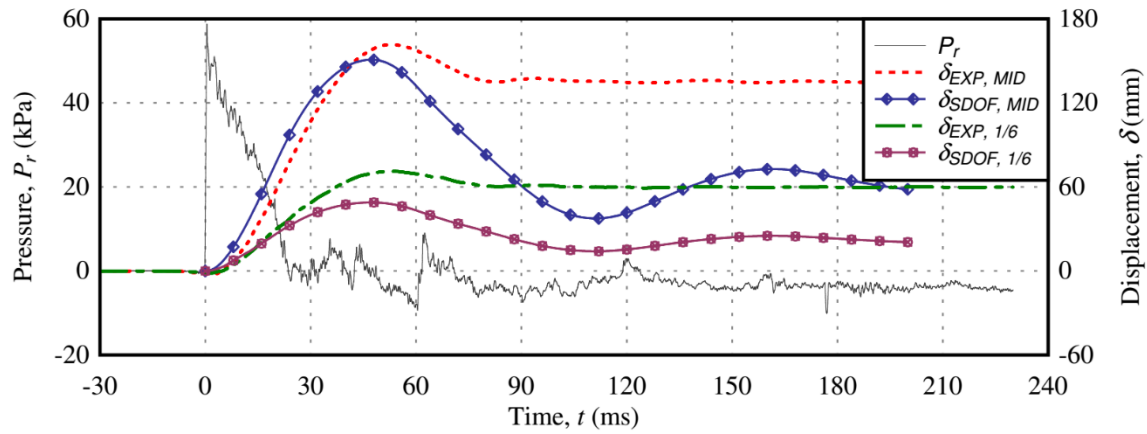
Figure C-2: Column 1S-C Experimental and Predicted Displacement Time Histories



a) Shot 1

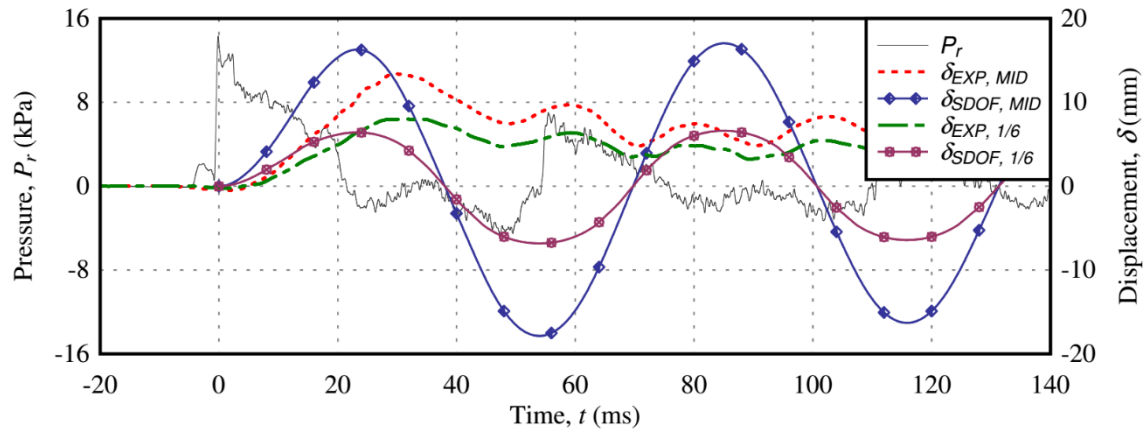


b) Shot 2

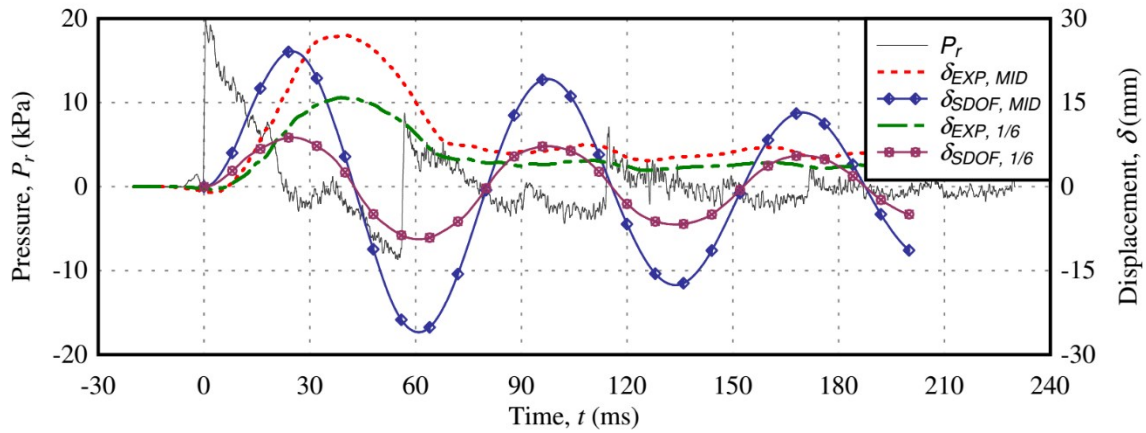


c) Shot 3

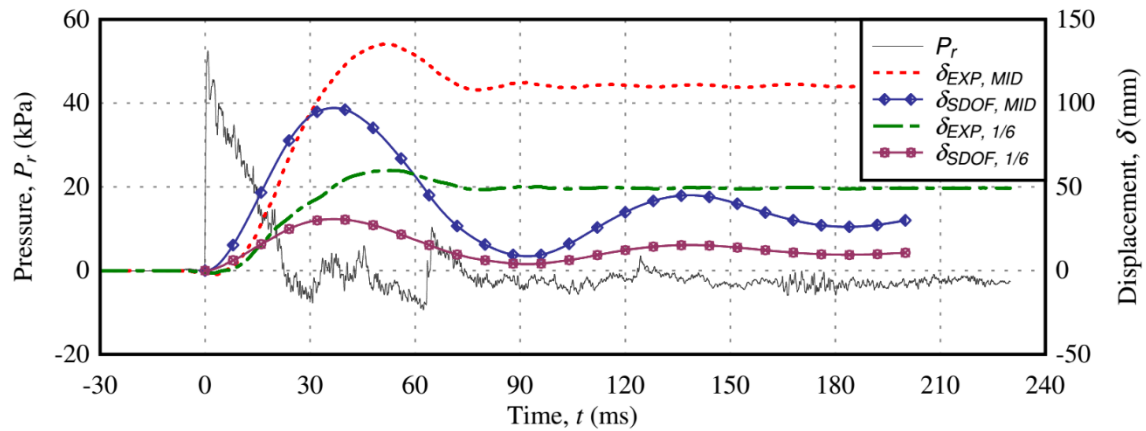
Figure C-3: Column 1NS-J Experimental and Predicted Displacement Time Histories



a) Shot 1

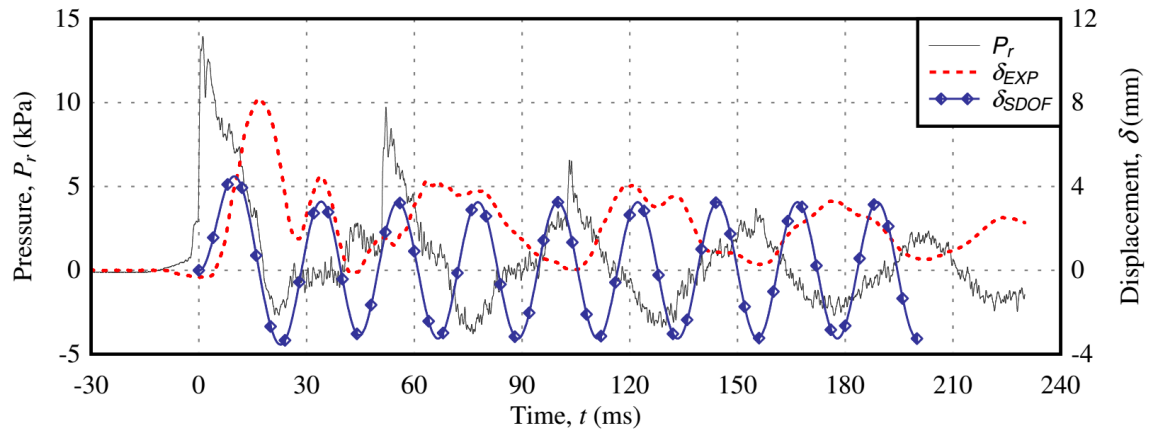


b) Shot 2

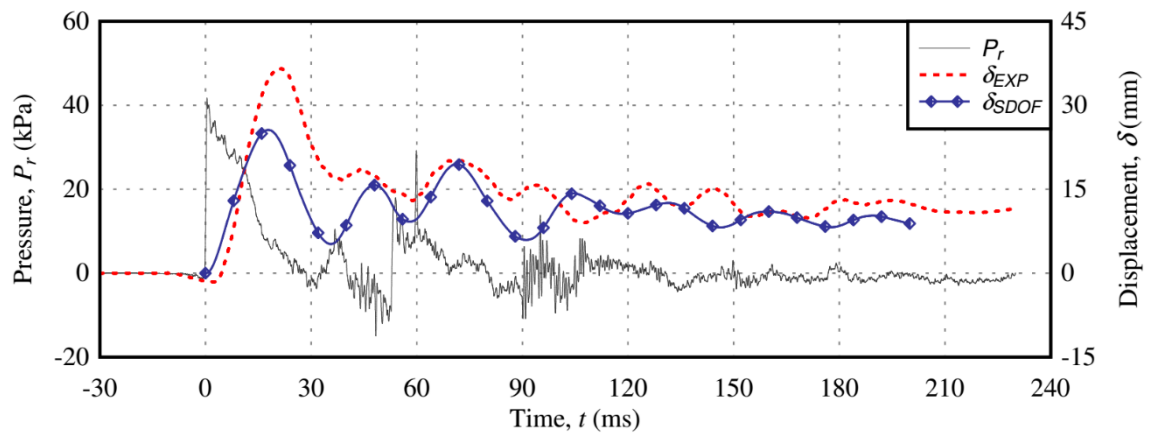


c) Shot 3

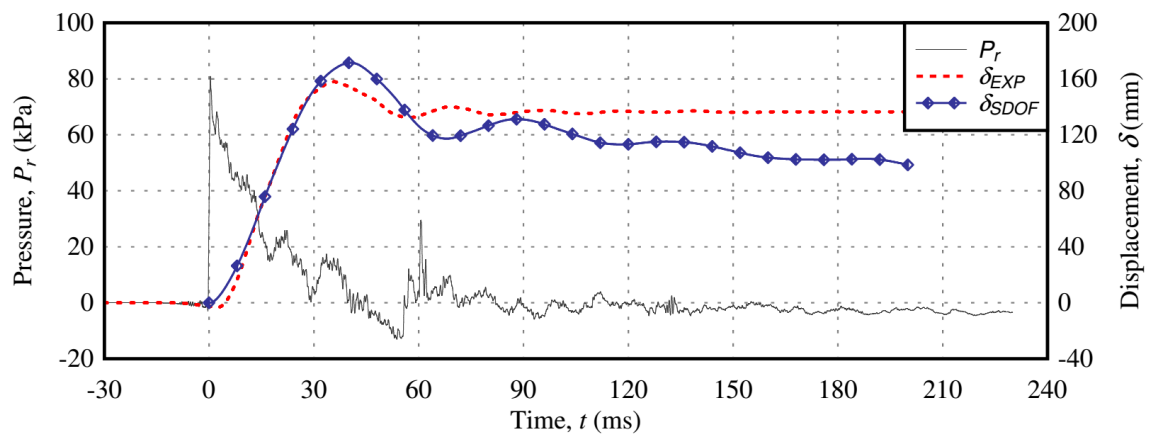
Figure C-4: Column 1S-J Experimental and Predicted Displacement Time Histories



a) Shot 1

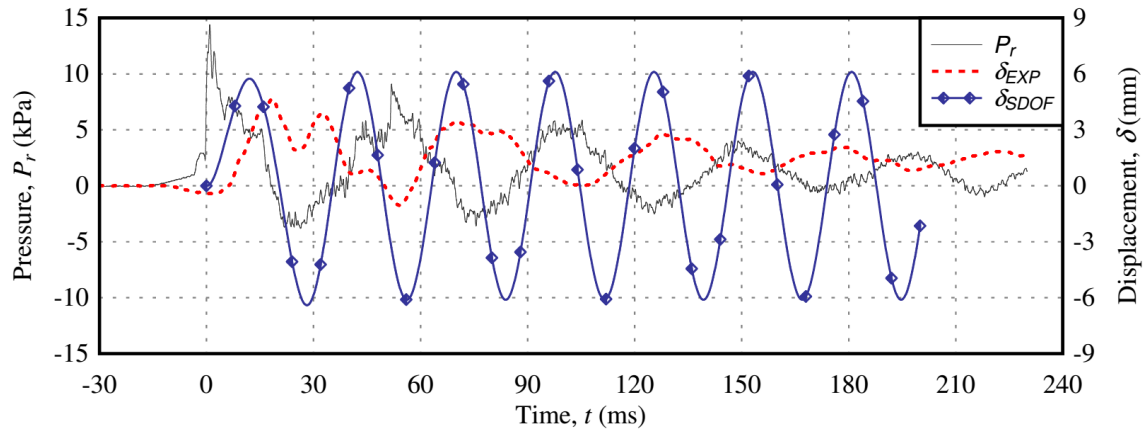


b) Shot 2

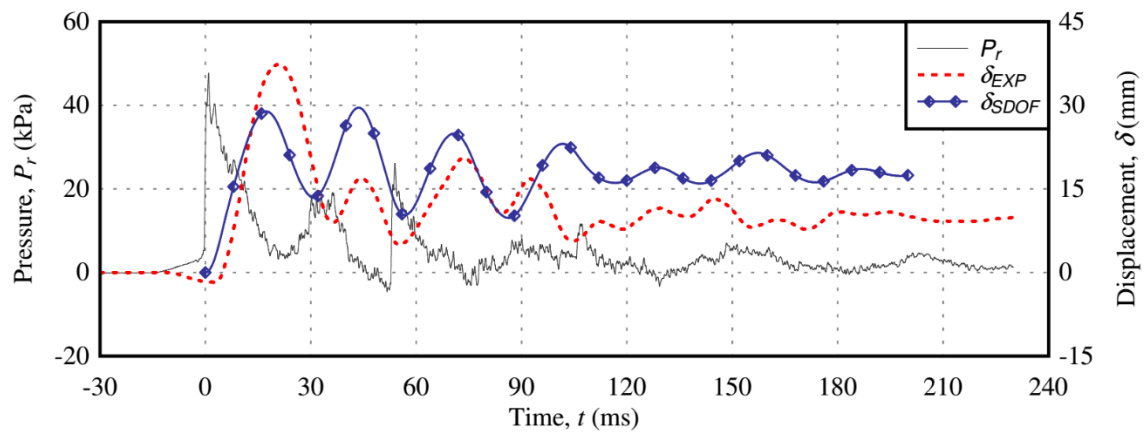


c) Shot 3

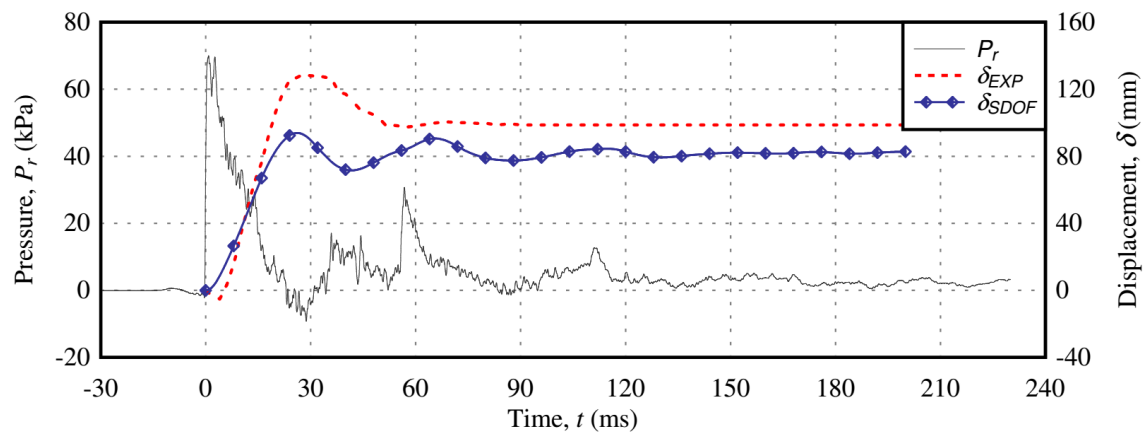
Figure C-5: Column 2NS-C Experimental and Predicted Displacement Time Histories



a) Shot 1

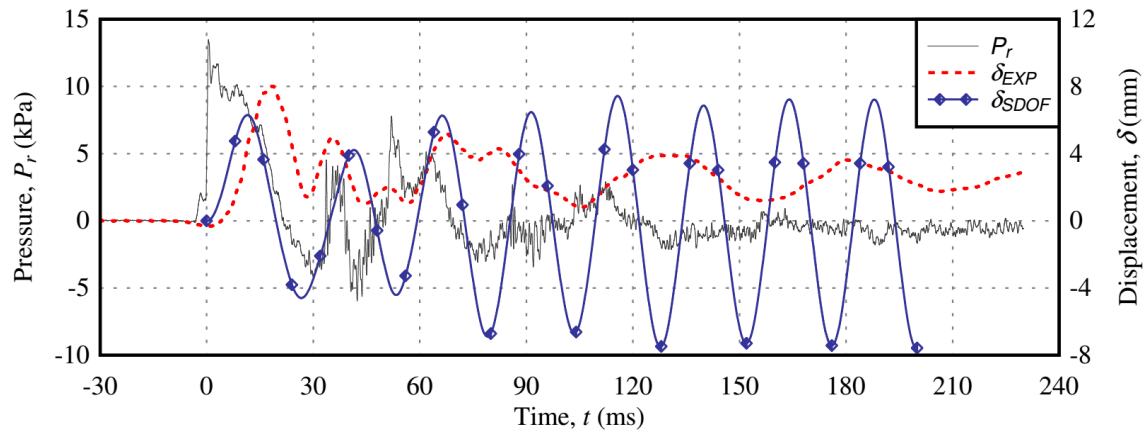


b) Shot 2

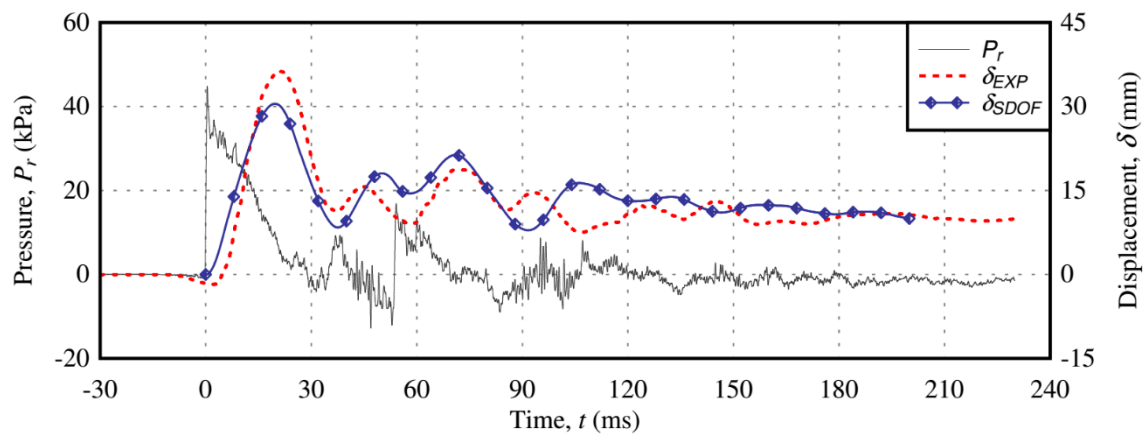


c) Shot 3

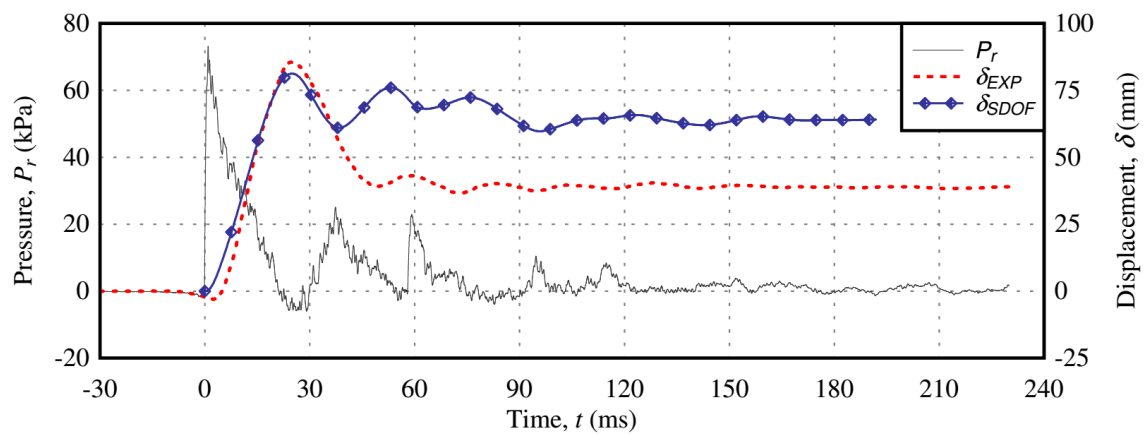
Figure C-6: Column 2S-C Experimental and Predicted Displacement Time Histories



a) Shot 1

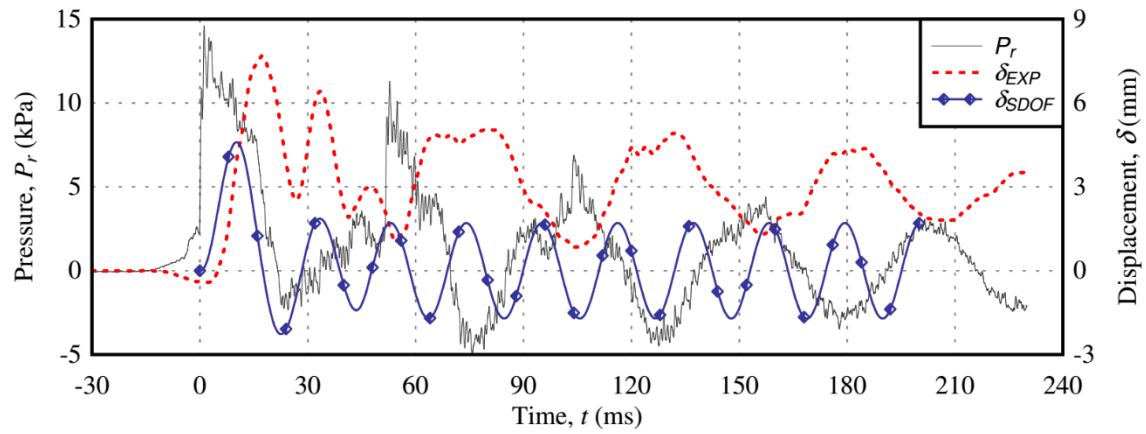


b) Shot 2

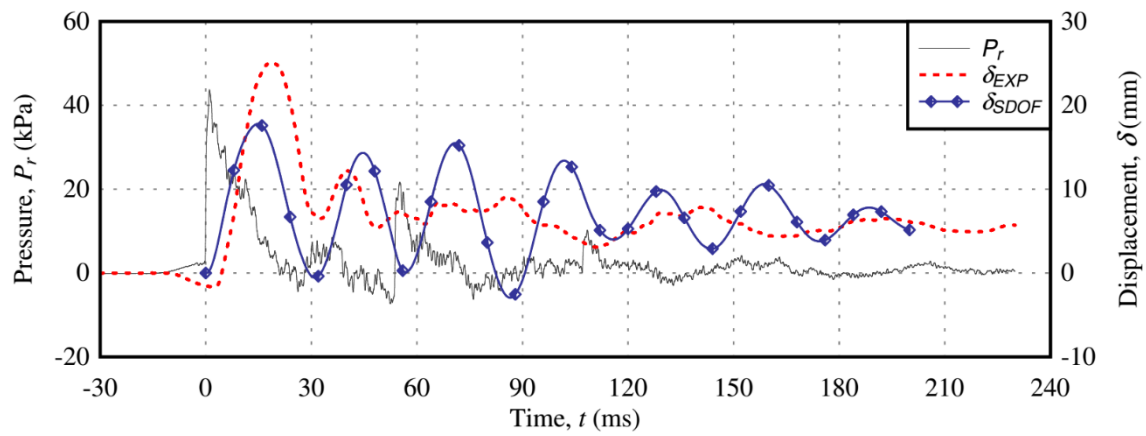


c) Shot 3

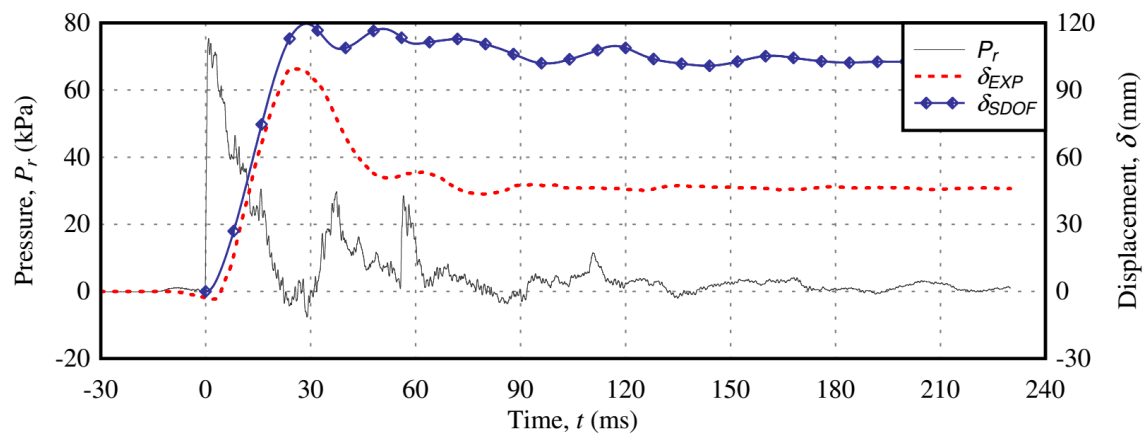
Figure C-7: Column 2NS-J Experimental and Predicted Displacement Time Histories



a) Shot1

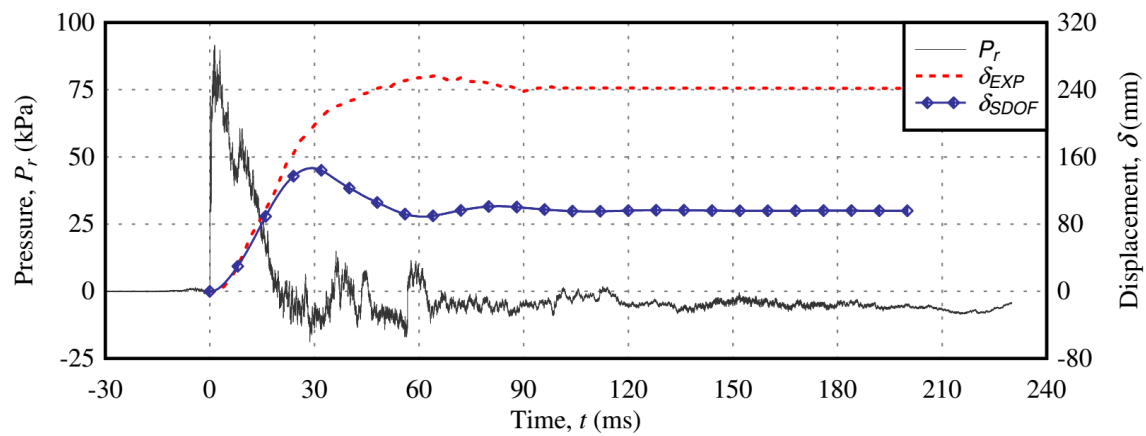


b) Shot 2



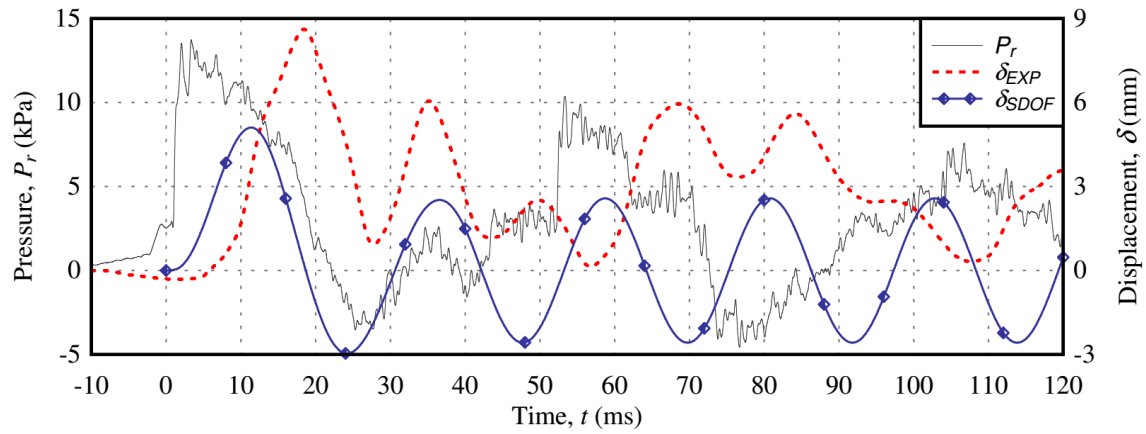
c) Shot 3

Figure C-8: Column 2S-J Experimental and Predicted Displacement Time Histories

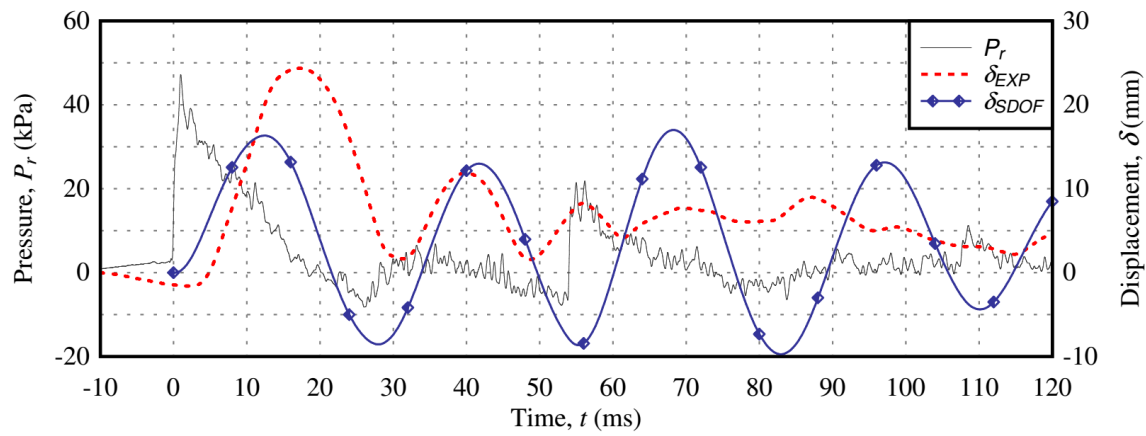


d) Shot 4

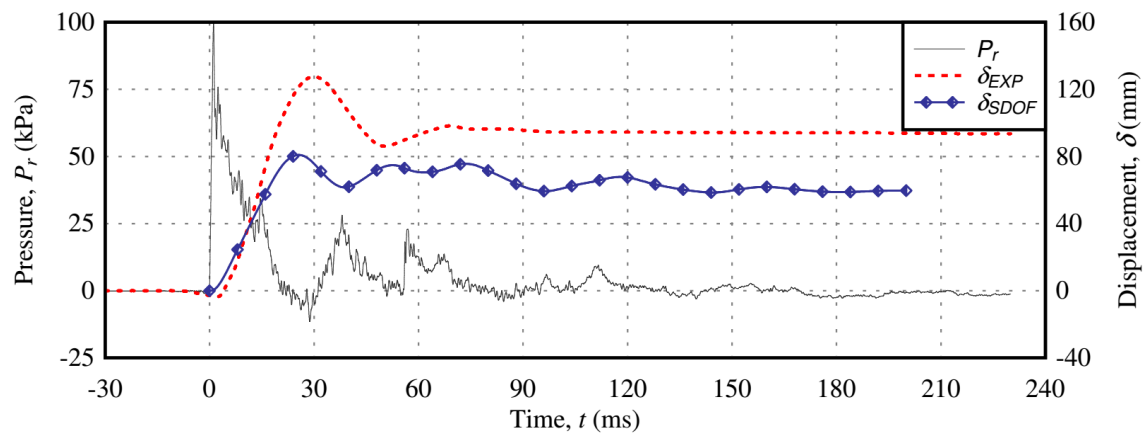
Figure C-8: Column 2S-J Experimental and Predicted Displacement Time Histories - Continued



a) Shot 1

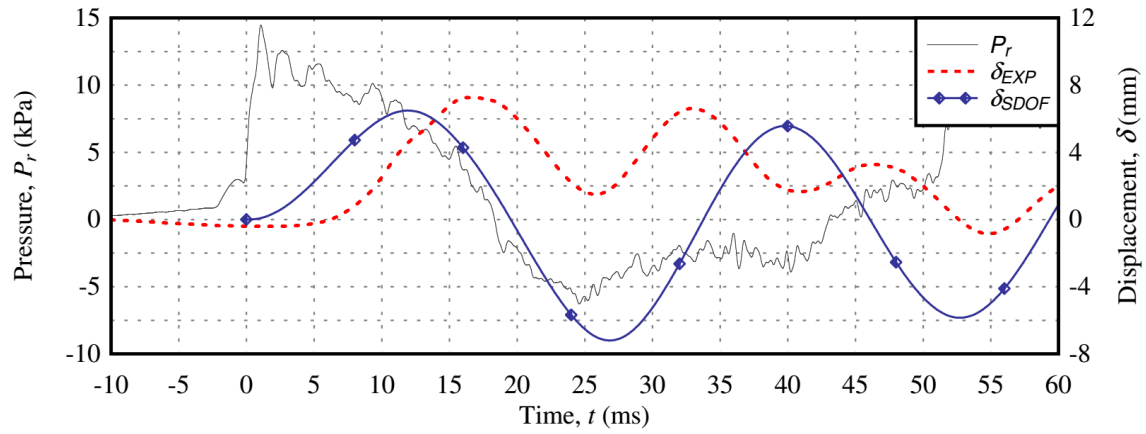


b) Shot 2

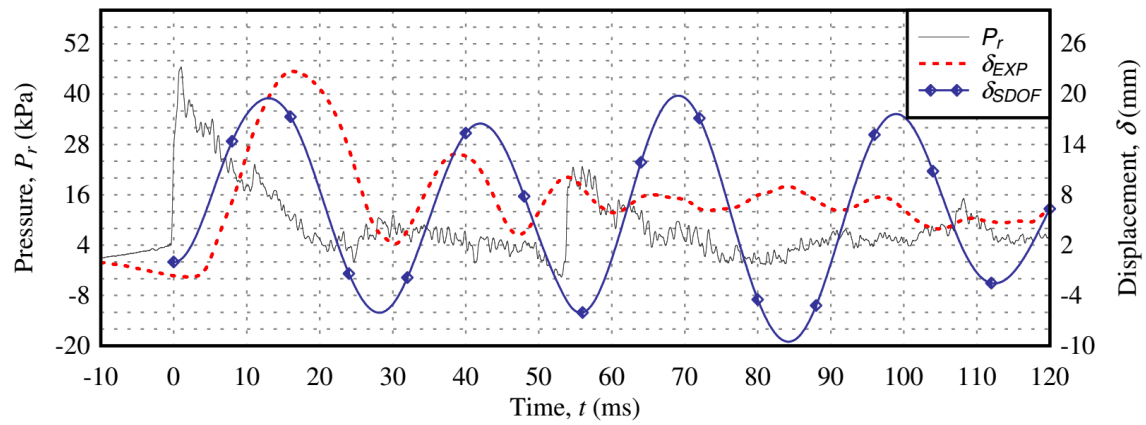


c) Shot 3

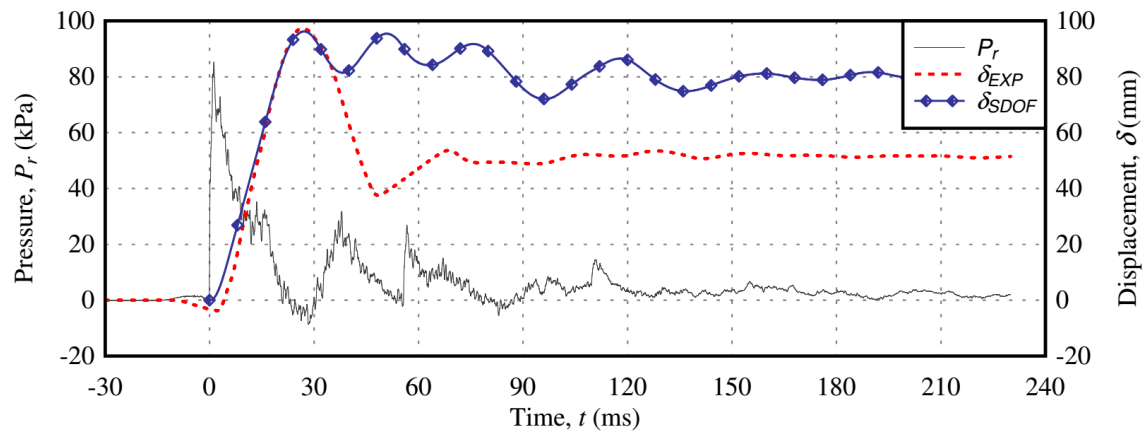
Figure C-9: Column 2NS-R Experimental and Predicted Displacement Time Histories



a) Shot 1

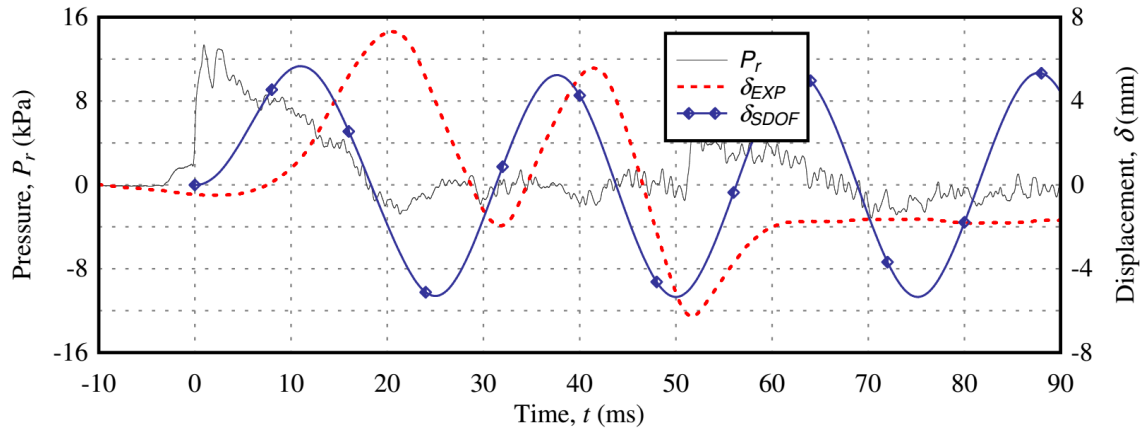


b) Shot 2

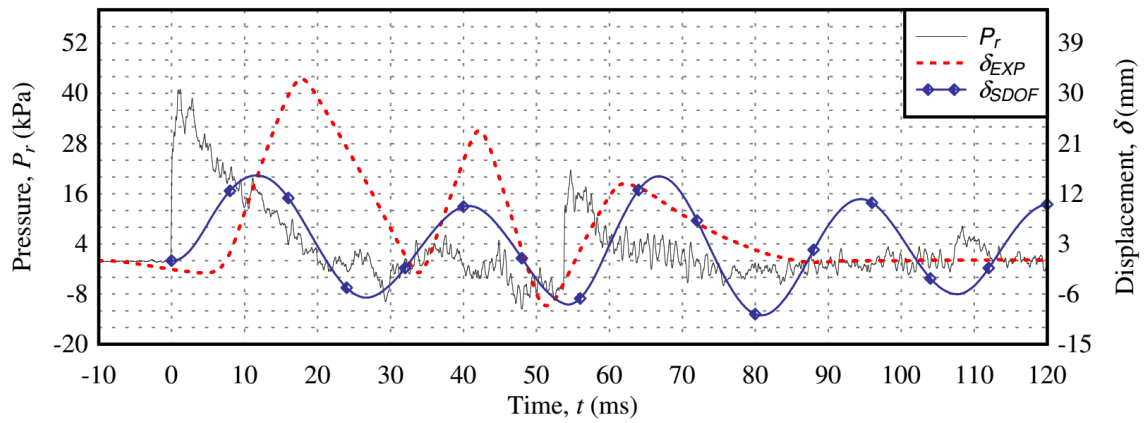


c) Shot 3

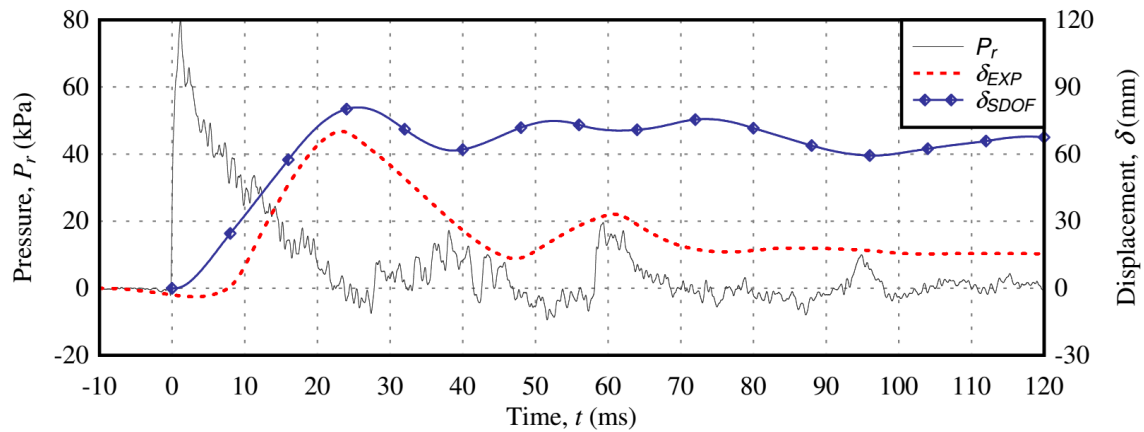
Figure C-10: Column 2S-R Experimental and Predicted Displacement Time Histories



a) Shot 1

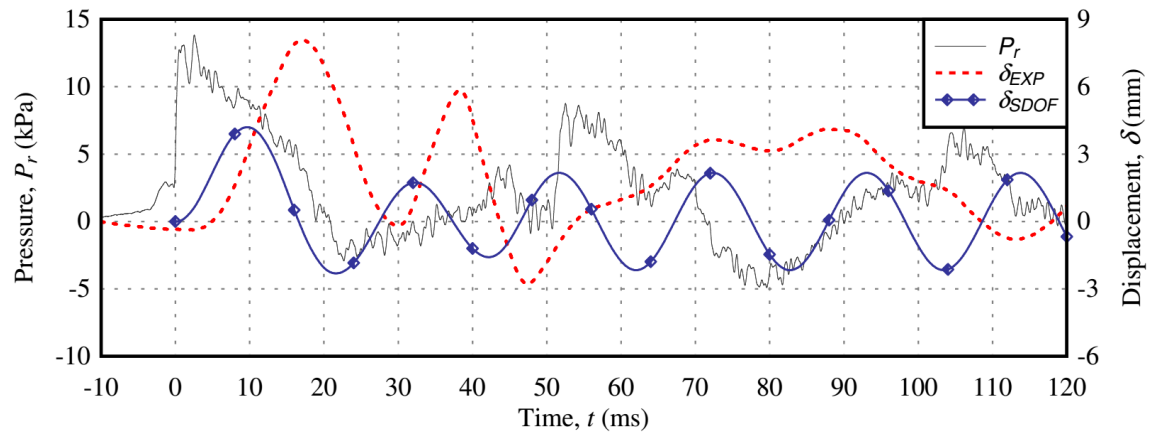


b) Shot 2

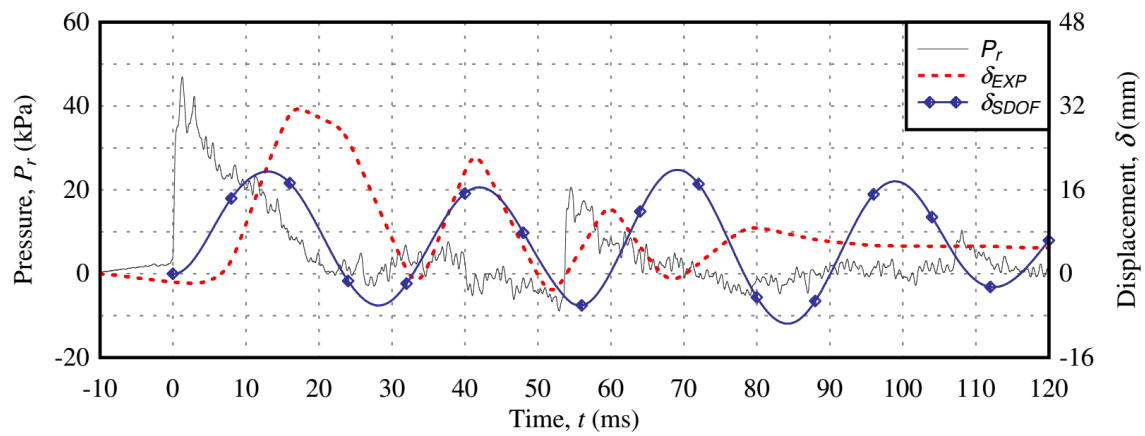


c) Shot 3

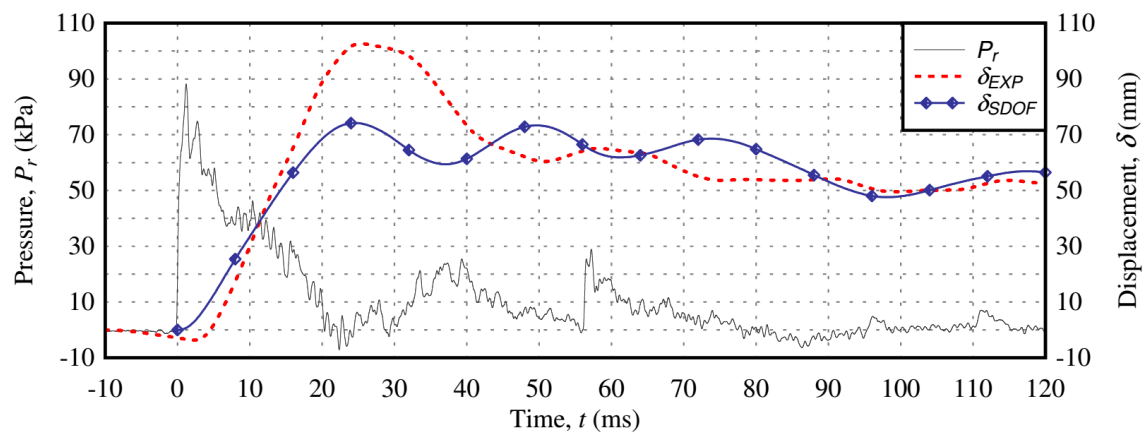
Figure C-11: Column 2NS-JR Experimental and Predicted Displacement Time Histories



a) Shot 1

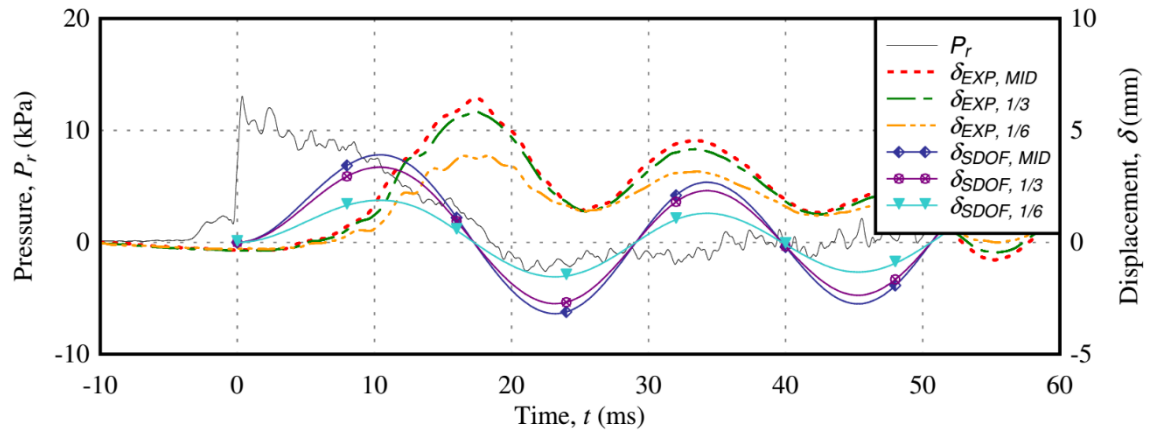


b) Shot 2

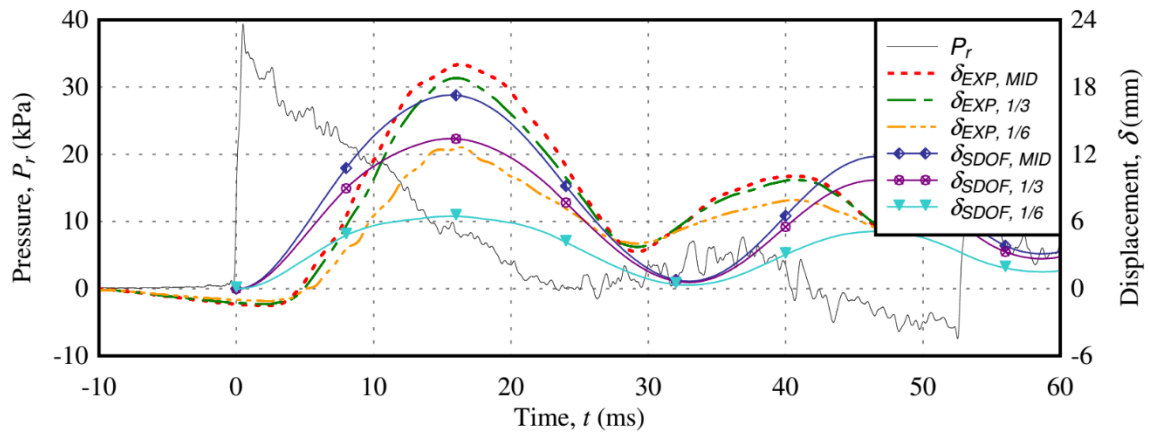


c) Shot 3

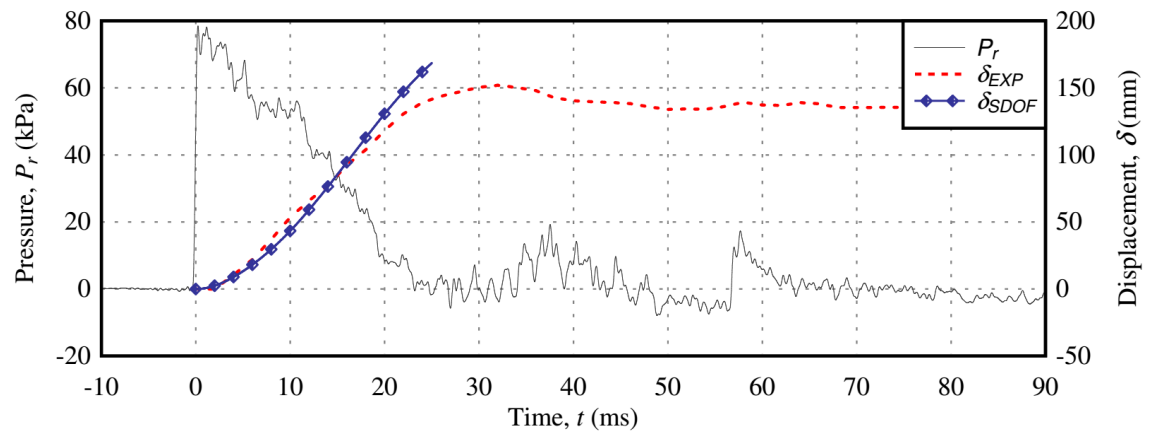
Figure C-12: Column 2S-JR Experimental and Predicted Displacement Time Histories



a) Shot 1

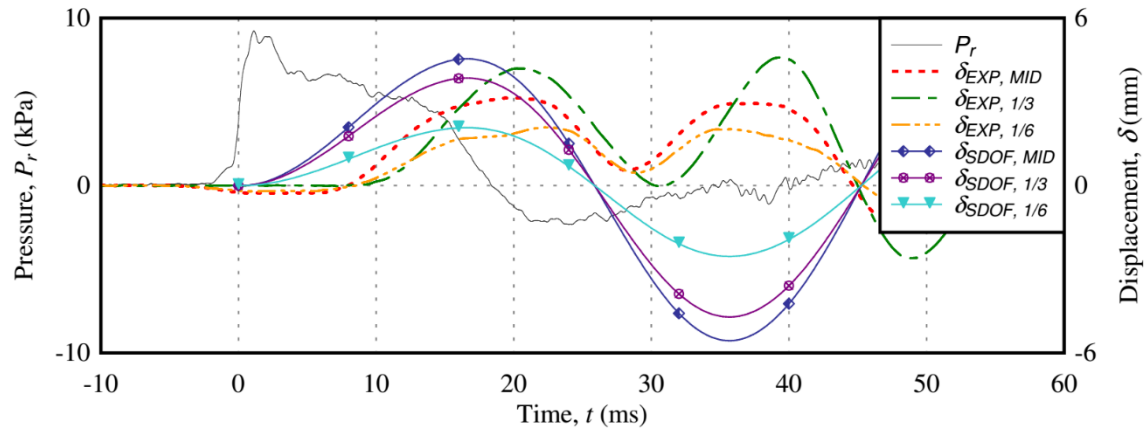


b) Shot 2

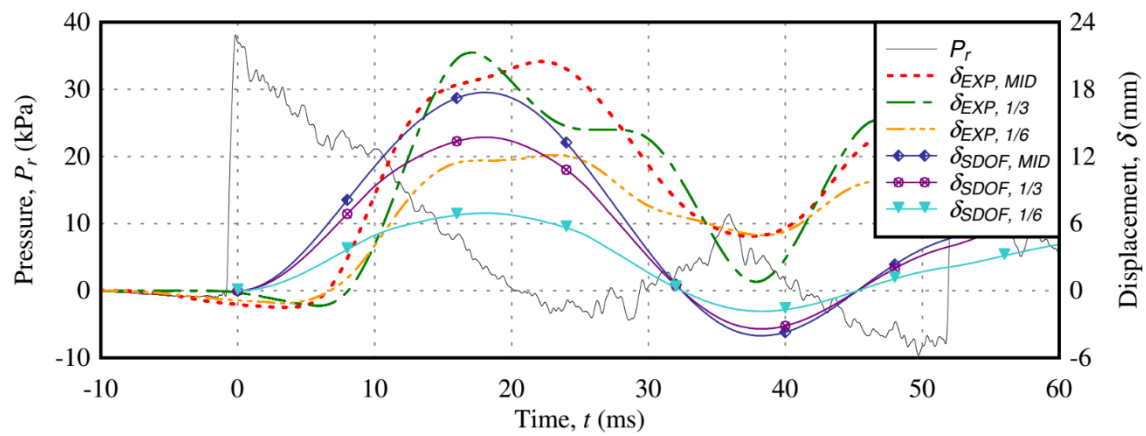


c) Shot 3

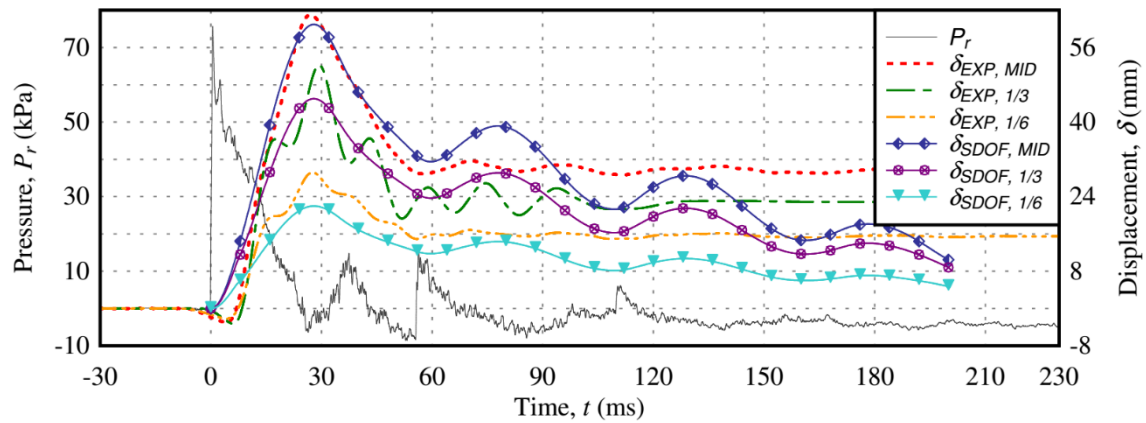
Figure C-13: Column 3NS-C Experimental and Predicted Displacement Time Histories



a) Shot 1

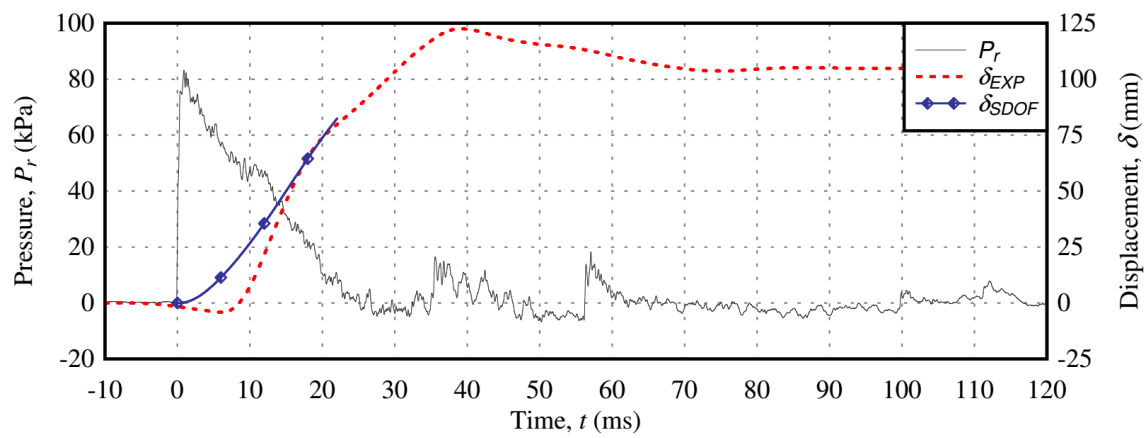


b) Shot 2



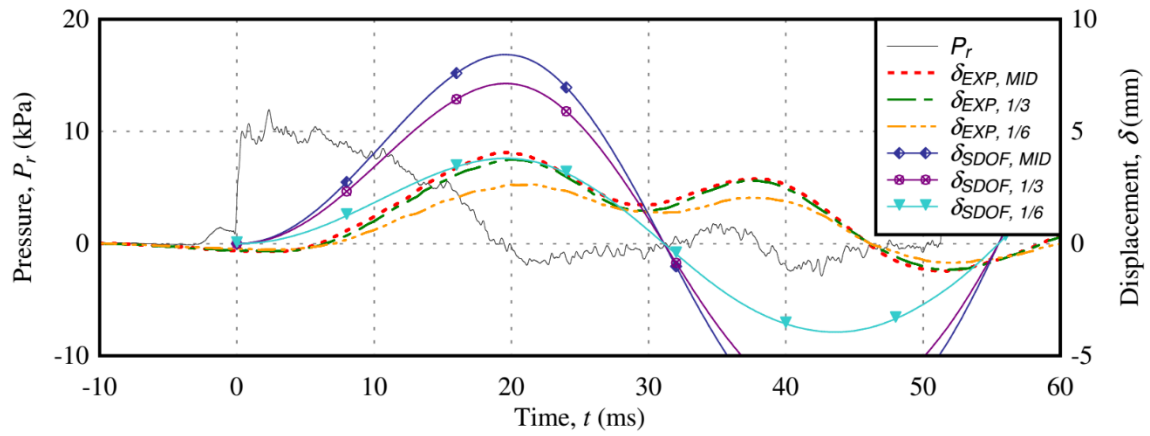
c) Shot 3

Figure C-14: Column 3NS-W Experimental and Predicted Displacement Time Histories

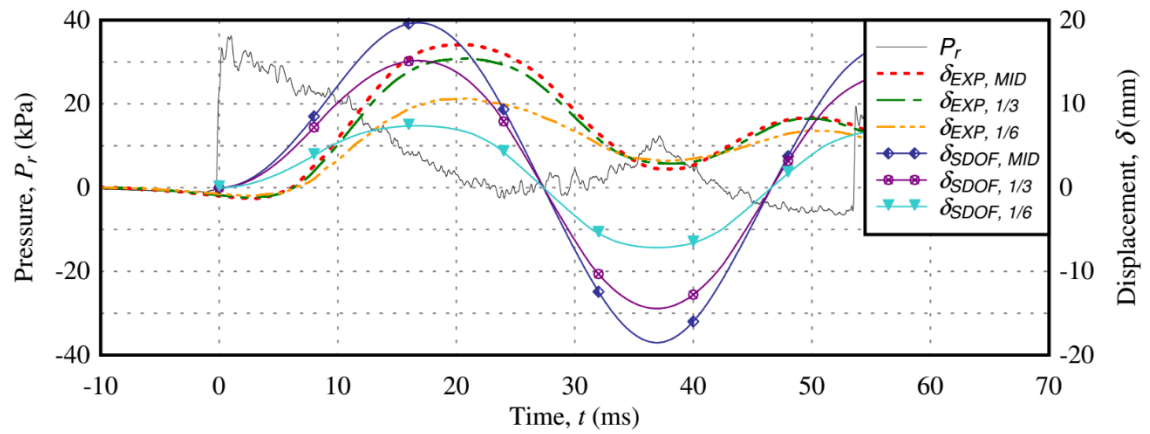


d) Shot 4

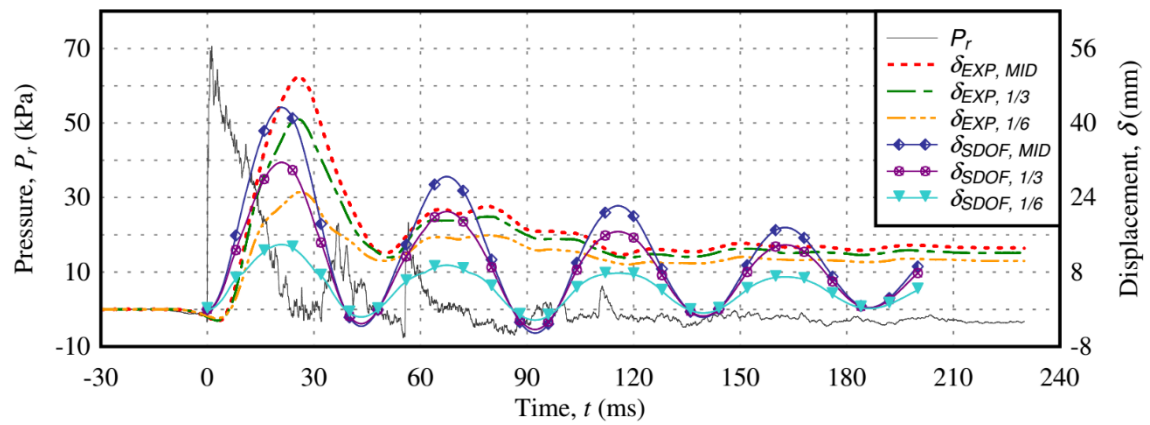
Figure C-14: Column 3NS-W Experimental and Predicted Displacement Time Histories - Continued



a) Shot 1

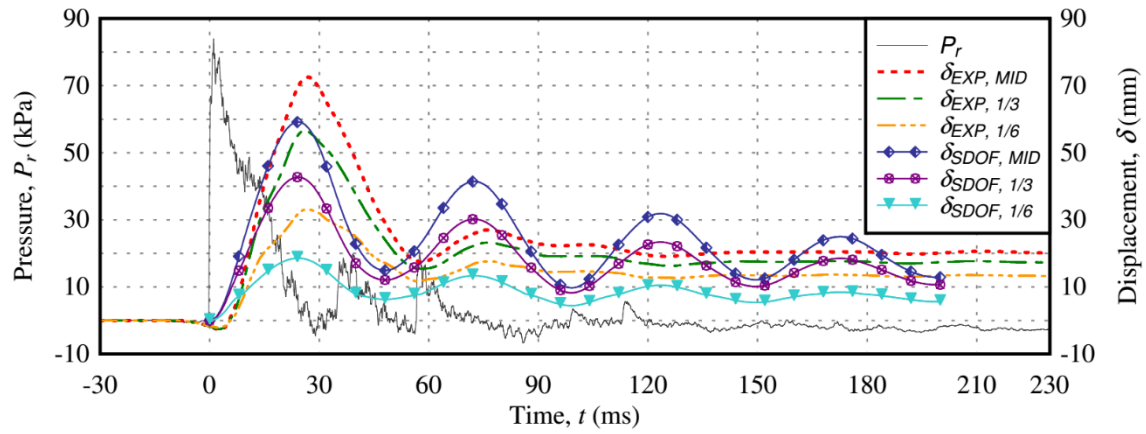


b) Shot 2

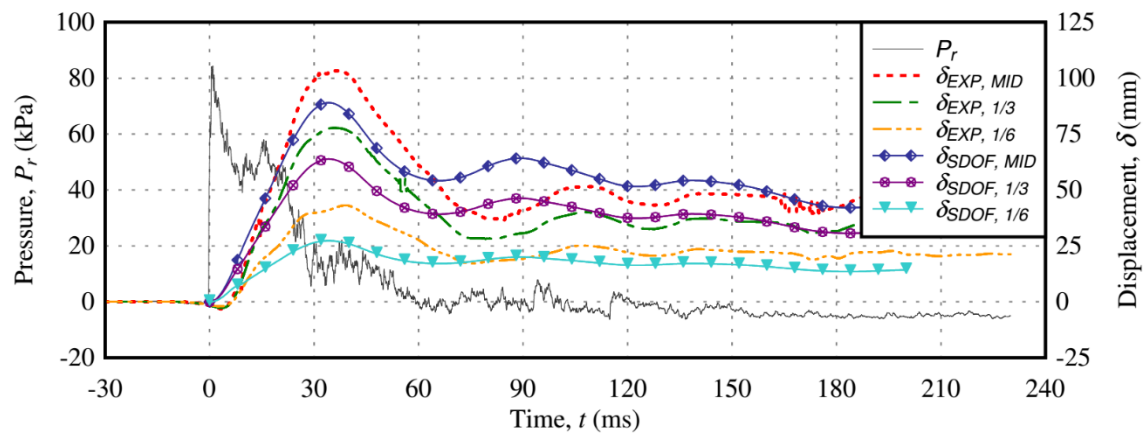


c) Shot 3

Figure C-15: Column 3NS-CB Experimental and Predicted Displacement Time Histories

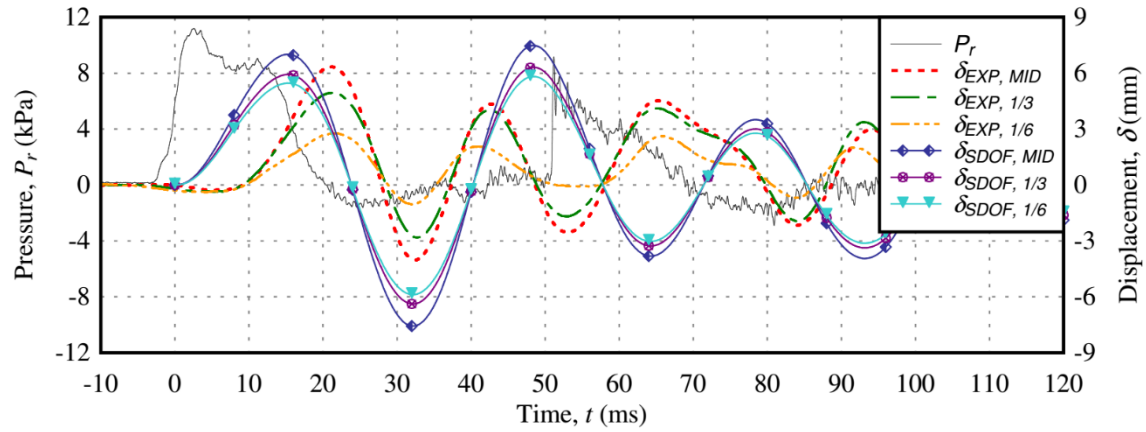


d) Shot 4

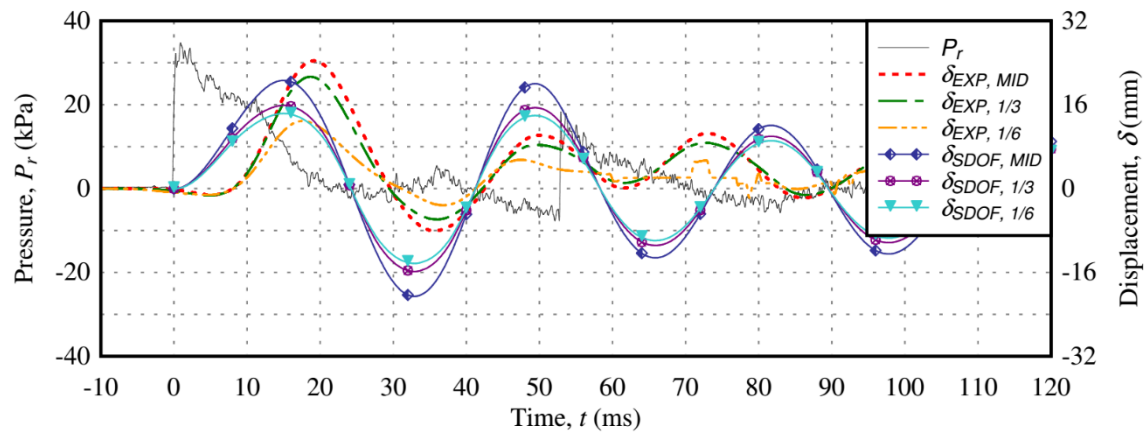


e) Shot 5

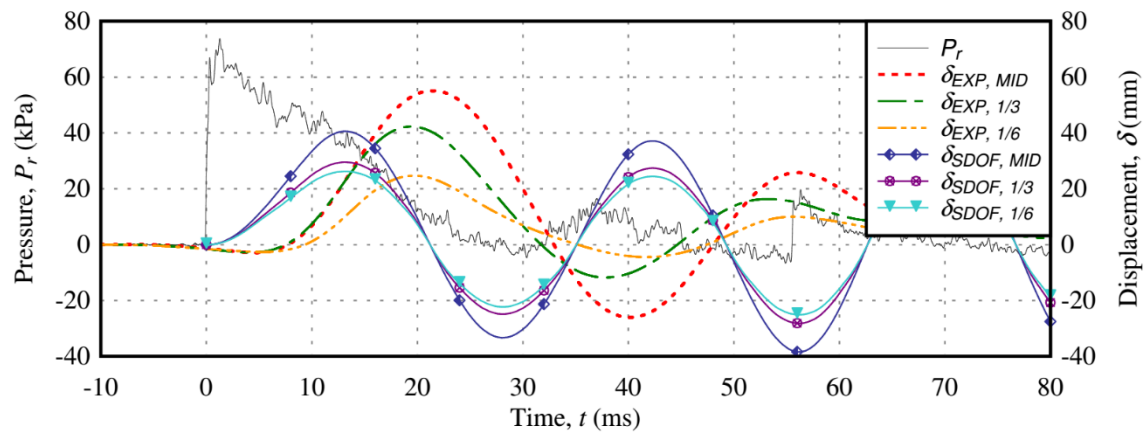
Figure C-15: Column 3NS-CB Experimental and Predicted Displacement Time Histories – Continued



a) Shot 1

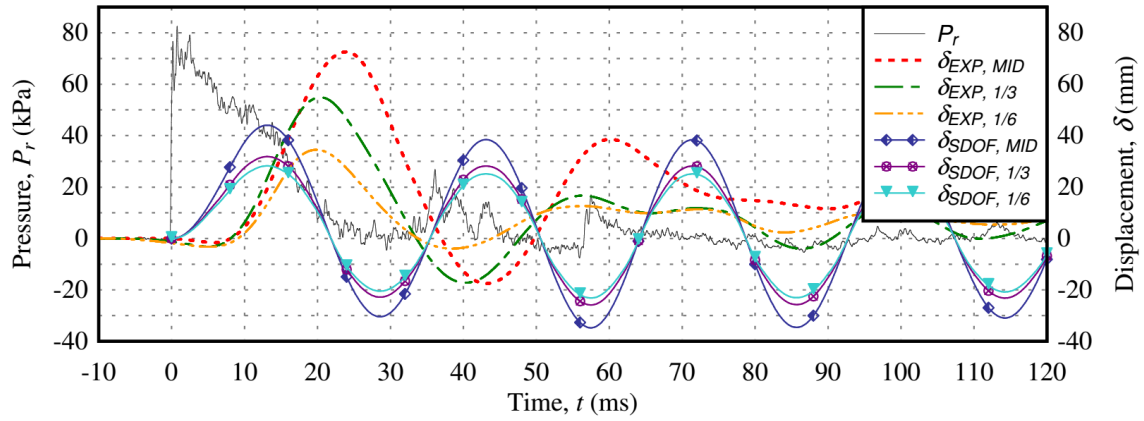


b) Shot 2

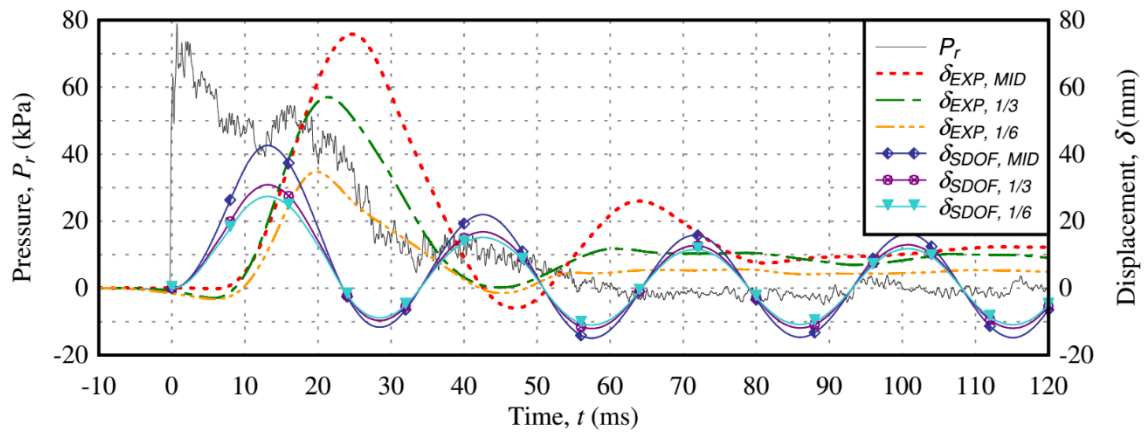


c) Shot 3

Figure C-16: Column 3NS-TB1 Experimental and Predicted Displacement Time Histories

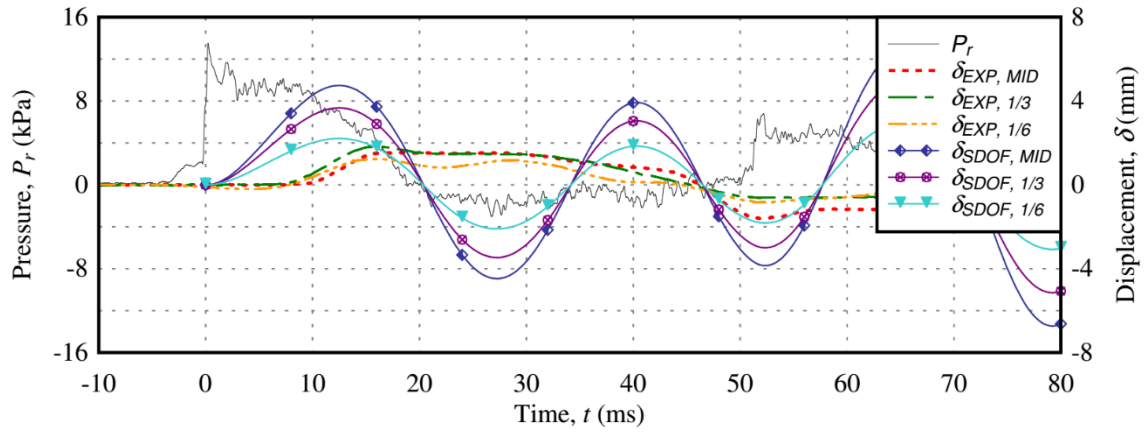


d) Shot 4

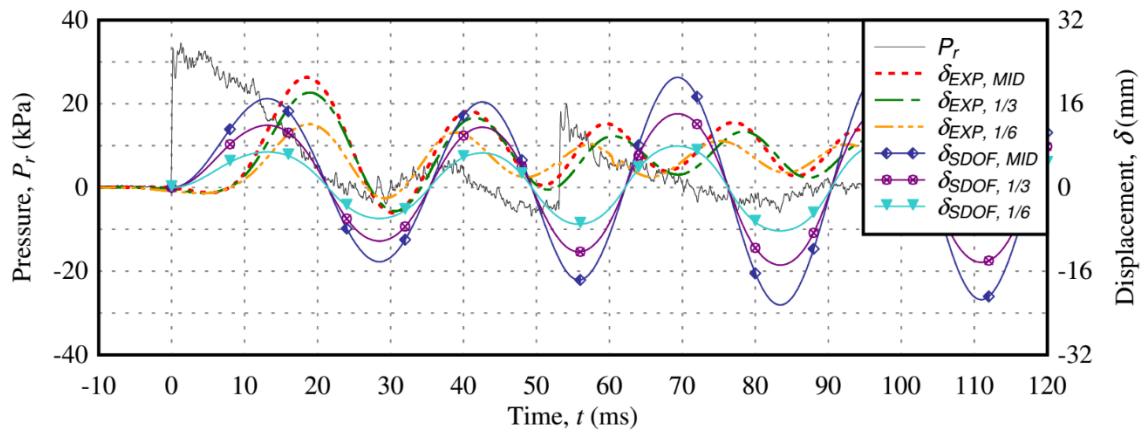


e) Shot 5

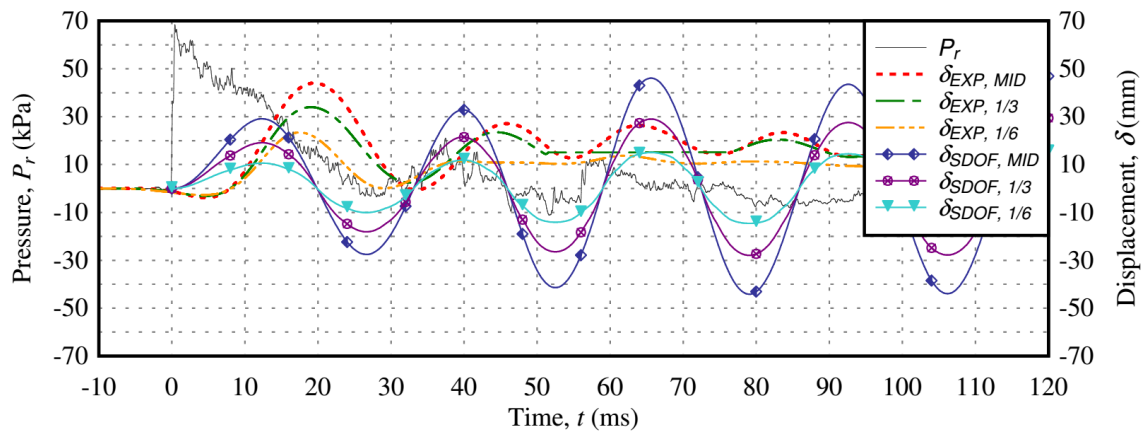
Figure C-16: Column 3NS-TB1 Experimental and Predicted Displacement Time Histories – Continued



a) Shot 1

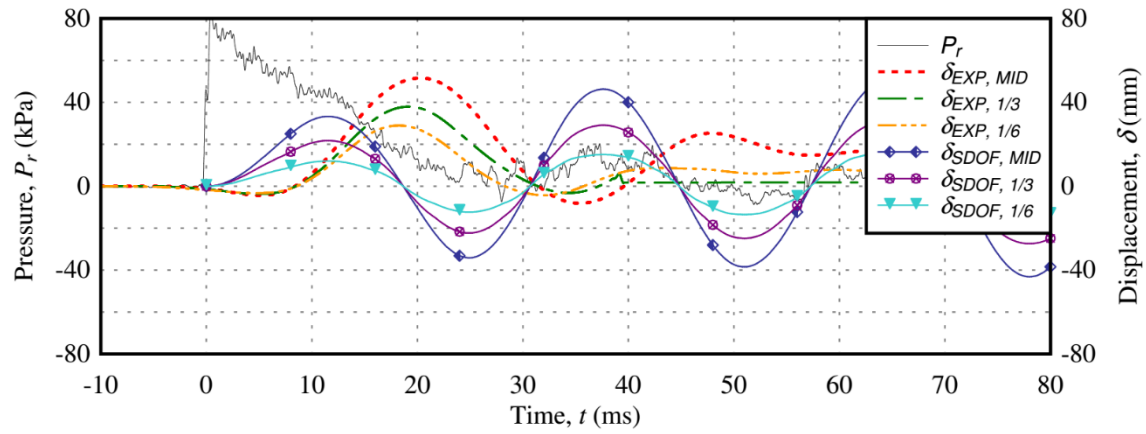


b) Shot 2

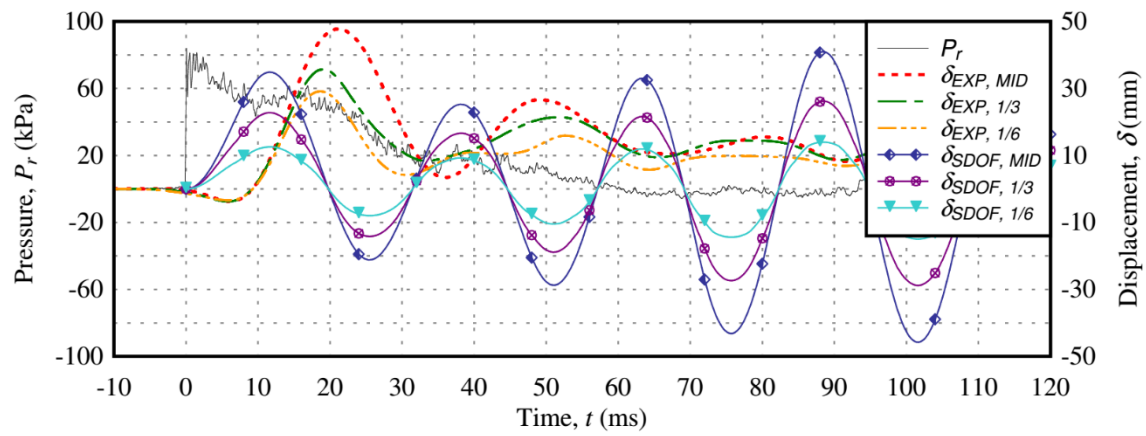


c) Shot 3

Figure C-17: Column 3NS-TB2 Experimental and Predicted Displacement Time Histories

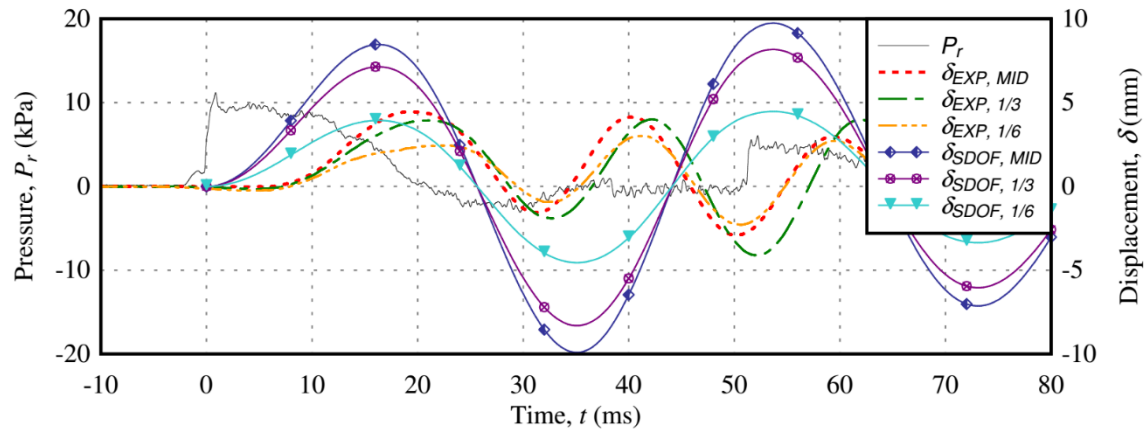


d) Shot 4

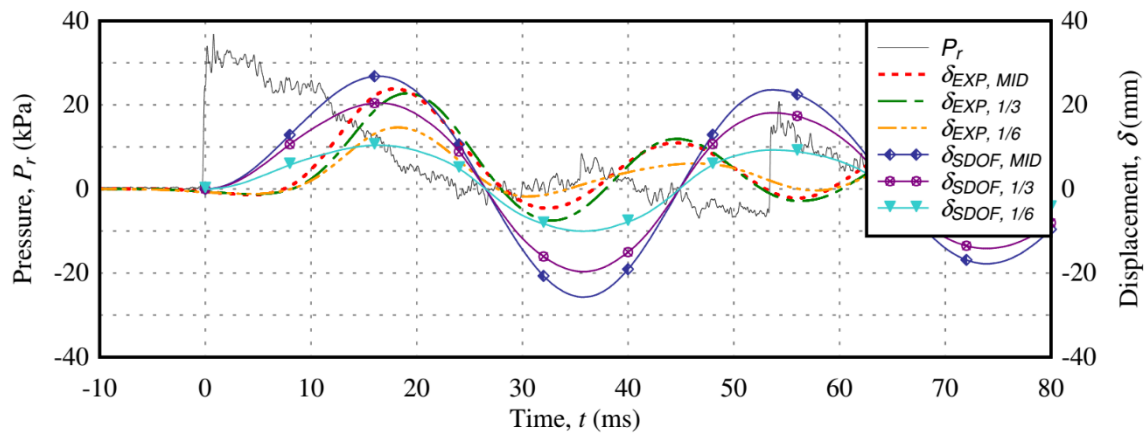


e) Shot 5

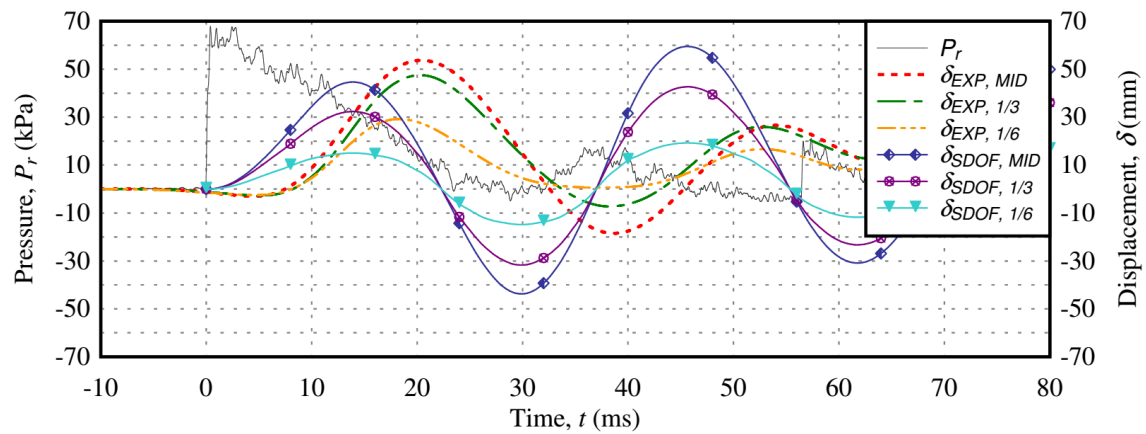
Figure C-17: Column 3NS-TB2 Experimental and Predicted Displacement Time Histories – Continued



a) Shot 1

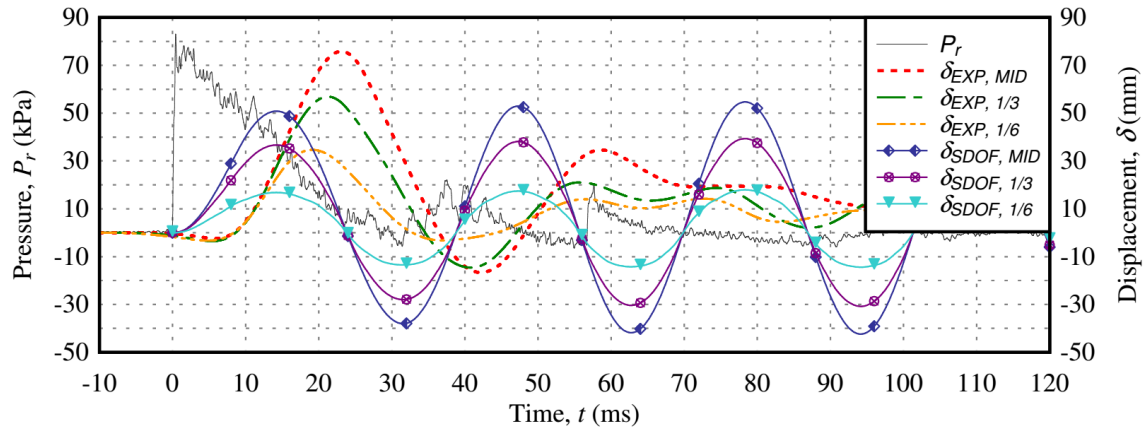


b) Shot 2

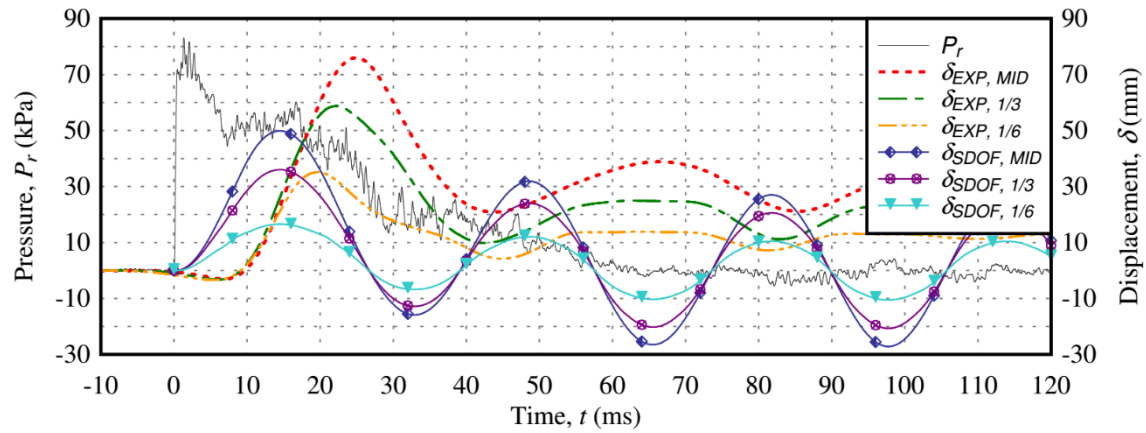


c) Shot 3

Figure C-18: Column 3NS-TB3 Experimental and Predicted Displacement Time Histories



d) Shot 4



e) Shot 5

Figure C-18: Column 3NS-TB3 Experimental and Predicted Displacement Time Histories – Continued