

Investigating the Behaviour of Glulam Beams and Columns Subjected to Simulated Blast Loading

by

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Abstract

The advancement in manufacturing technologies to produce high-performing engineered wood products (EWP) has allowed wood to be utilized beyond the traditional low-rise light-frame structures and to become a viable material option for much larger structures. Although glued-laminated timber (glulam) is included as a material option in the current blast code (CSA, 2012), its response to blast loading is not yet well documented.

An experimental program investigating the behaviour of seventy glulam beams and columns was developed with focus on establishing the dynamic characteristics of glulam beams and columns with and without the effect of FRP reinforcement. A shock tube capable of simulating high strain rates similar to those experienced during blast was used. Thirty-eight beams with three different cross-sections were tested statically and dynamically to establish the high strain rate effects (dynamic increase factor). Six columns were also tested dynamically with axial load levels ranging from 15 to 75 % of the columns' compression design capacity. Different retrofit configurations varying from simple tension reinforcement to U-shaped tension reinforcement with confinement using both unidirectional and bi-directional FRP were investigated on a total of twenty-six beams.

A procedure capturing the strain-rate effects, variable axial load and FRP, was developed and found to be capable of predicting the flexural behaviour of the beams up to maximum resistance with reasonable accuracy when compared to experimentally obtained static and dynamic resistance curves. Implications on the design of both retrofitted and unretrofitted specimens are also discussed.

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Table of Contents

Abstract	ii
Acknowledgements	iii
Table of Contents	iv
List of Tables	ix
List of Figures	xii
CHAPTER 1 - Introduction	1
1.1 Research needs	1
1.2 Research objectives	3
1.3 Scope	4
1.4 Structure of thesis.....	5
1.5 Original Contributions	6
CHAPTER 2 - Background and Literature Review	8
2.1 General	8
2.2 Wood as a construction material	8
2.2.1 Physical properties of wood.....	8
2.2.2 Flexural resistance of wood	10
2.2.3 Engineered wood products.....	11
2.3 FRP as a strengthening material.....	13
2.3.1 Overview of FRP materials.....	13

2.4	Blast wave characteristics and effects on structures	15
2.4.1	Blast loading	15
2.4.2	Dynamic analysis.....	17
2.4.3	Blast testing methods	18
2.5	Previous research	19
2.5.1	High strain-rate effects on wood properties.....	19
2.5.2	Response of structural elements subjected to combined transverse and axial loading	21
2.5.3	FRP as a retrofit option for flexural strengthening.....	25
CHAPTER 3 - Experimental Program.....		29
3.1	General	29
3.2	Description of the unretrofitted specimens	29
3.3	Description of retrofitted specimens	31
3.3.1	Retrofit configurations – Phase 1.....	31
3.3.2	Retrofit – Phase 2.....	33
3.4	Summary of test matrix.....	36
3.5	Test setups.....	37
3.5.1	Components	37
3.5.2	Full-scale static beam tests	39
3.5.3	Full-scale dynamic beams and columns tests	40

CHAPTER 4 - Experimental Results.....	46
4.1 General	46
4.2 Component test results	46
4.2.1 Wood coupons	46
4.2.2 FRP coupons	48
4.3 Characterization of static and dynamic failure modes	50
4.3.1 General terminology	50
4.3.2 Unretrofitted beams	52
4.3.3 Columns	54
4.3.4 Retrofitted beams	55
4.4 Strain-rate effects	65
4.4.1 Flexural strength of unretrofitted beams.....	65
4.4.2 Effect of axial load.....	75
4.4.3 Effect of FRP	79
CHAPTER 5 - Analytical Model	86
5.1 General	86
5.2 Constitutive Material Relationship	86
5.3 Moment Curvature Analysis	89
5.3.1 Unretrofitted beams and columns	89
5.3.2 FRP retrofitted beams	92

5.4	Development of Resistance Curves	96
5.4.1	Constant or no axial load	96
5.4.2	Variable axial load	97
CHAPTER 6 - Discussion.....		101
6.1	General	101
6.2	Dynamic increase factor for glulam beams.....	101
6.2.1	Failure modes and determining the DIF	101
6.2.2	Predicting the behaviour of the unretrofitted beams.....	105
6.2.3	Implications on design of beams	109
6.3	Effect of axial load on the response of glulam columns	111
6.3.1	Behaviour of glulam columns subjected to combined axial and out-of-plane simulated blast loading	111
6.3.2	Predicting the behaviour of glulam columns	112
6.4	Behaviour of FRP reinforced glulam beams.....	114
6.4.1	Effect of unidirectional FRP reinforcement configurations	114
6.4.2	Effect of multi-directional FRP reinforcement configurations.....	118
6.4.3	Effect of unidirectional FRP reinforcement on previously damaged beams	122
6.4.4	Effect of FRP reinforcement on the strain distribution at failure	122
6.4.5	Predicting the behaviour of FRP reinforced glulam beams	124
CHAPTER 7 - Conclusions		127

7.1 Recommendations for Future Research	129
References	131
Appendix A – Static Test Results of Wood Coupons.....	139
Appendix B – Static Test Results of FRP Coupons.....	143
Appendix C – Test Results for Unretrofitted Glulam Beams	160
Appendix D – Test Results for Unretrofitted Glulam Columns	210
Appendix E – Test Results for Retrofitted Glulam Beams	217
Appendix F – Derivation of Dynamic Resistance Based on SDOF Analysis	258
Appendix G – Process Details of Moment-Curvature and Resistance Curves Analysis	263
Appendix H – Effects of Material Properties on the Column Behaviour as Established by the Moment-Curvature Analysis.....	266

List of Tables

Table 3.1: Summary of test matrix	36
Table 4.1: Static tension and compression coupon test results	47
Table 4.2: Static tension GFRP and FRP coupon test results	50
Table 4.3: Summary of static test results	66
Table 4.4: Summary of non-destructive dynamic beam test results	69
Table 4.5: Summary of destructive dynamic beam test results	70
Table 4.6: Summary of experimental dynamic glulam column test results	75
Table 4.7: Summary of static test results for retrofitted beams	79
Table 4.8: Dynamic test results for Retrofits 1 to 6	81
Table 4.9: Dynamic test results for B-[86] retrofitted beams	84
Table 4.10: Dynamic test results for the restored and retrofitted beams	85
Table 5.1: Model input strengths adjusted for size effects	88
Table 6.1: Groups for t-Test analysis	102
Table 6.2: Results for DIF t-Test analysis	102
Table 6.3: Results for MOE and failure strain t-Tests	104
Table 6.4: Wood tensile failure strain	123
Table 6.5: Comparison of model's predictions to experimental results for retrofitted beams	125
Table A.1: Individual static tension coupon tests results for B-[86] beams	140

Table A.2: Individual static compression coupon tests results for B-[86] beams	140
Table A.3: Individual static tension coupon tests results for B-[86] beams	142
Table A.4: Individual static compression coupon tests results for B-[86] beams	142
Table B.1: Summary of results for GFRP [0] ₂ (FyfeCo)	143
Table B.2: Summary of results for CFRP [0] ₂ (FyfeCo)	145
Table B.3: Summary of results for GFRP [0] ₂	146
Table B.4: Summary of results for GFRP [±45] ₂	148
Table B.5: Summary of results for GFRP [0/90] ₂	149
Table B.6: Summary of results for GFRP [0/90,±45]	151
Table B.7: Summary of results for GFRP [±45] ₄	152
Table B.8: Summary of results for GFRP [0][0/90] ₂	154
Table B.9: Summary of results for GFRP [0] ₃ [0/90] ₂	155
Table B.10: Summary of results for GFRP [0] ₂ [±45] ₂	157
Table B.11: Summary of results for GFRP [0,0/90,±45]	158
Table C.1: Static test results	160
Table C.1: Static test results (CTN)	161
Table C.2: Dynamic beams elastic tests	162
Table C.3: Dynamic beams destructive test results	163
Table C.3: Dynamic beams destructive test results (CTN)	164
Table D.1: Columns dynamic test results	210

Table E.1: Static test results of Retrofits 1 through 4.....	217
Table E.2: Dynamic beams destructive test results for Retrofits 1 through 9 (Part I).....	218
Table E.3: Dynamic beams destructive test results for Retrofits 1 through 9 (Part II)	219

List of Figures

Figure 2.1: Effect of load duration on maximum bending stress.....	9
Figure 2.2: Types of failures in static bending	10
Figure 2.3: Model proposed by Buchanan (1990)	11
Figure 2.4: Richmond Oval	12
Figure 2.5: Examples of finger joints	12
Figure 2.6: Typical stress-strain curves for common FRP and mild steel.....	14
Figure 2.7: Typical blast wave history.....	15
Figure 2.8: Moment curvature curves for constant axial load for a rectangular wood section	22
Figure 2.9: Moment curvature curves for various constant axial load for a.....	24
seismic concrete reinforced column	24
Figure 2.10: FRP sheets used as tension reinforcement in glulam beams	26
Figure 2.11: Previous FRP wrap configurations investigated	27
Figure 2.12: Typical load-deflection curves for retrofitted wood beams	28
Figure 3.1: Representative static failure modes of generic glulam beams	30
Figure 3.2: Description of Retrofits 1 through 4	32
Figure 3.3: Description of Retrofits 5 and 6	34
Figure 3.4: Description of Retrofits 7 through 9	35
Figure 3.5: Wood coupon static tests.....	38

Figure 3.6: FRP coupon static tests	38
Figure 3.7: Static test setup.....	39
Figure 3.8: University of Ottawa’s Shock Tube	40
Figure 3.9: Dynamic test setup for beams	42
Figure 3.10: Dynamic experimental setup for columns.....	44
Figure 4.1: Coupon static test results.....	47
Figure 4.2: FRP coupon test results representative failure modes.....	48
Figure 4.3: Representative FRP coupon stress-strain relationship	49
Figure 4.4: Terminology used for describing resistance curves	51
Figure 4.5: Representative static failure modes of glulam beams	52
Figure 4.6: Representative dynamic failure modes of beams with single laminate	53
Figure 4.7: Representative dynamic failure modes of beams with multiple laminates	54
Figure 4.8: Comparison of static and dynamic failure modes of beams and columns	55
Figure 4.9: Representative static and dynamic failure modes for Retrofit 1	56
Figure 4.10: Representative static and dynamic failure modes for Retrofit 2	57
Figure 4.11: Static and dynamic representative failure modes for Retrofits 3 and 4	58
Figure 4.12: Failure modes of Retrofits 5A and 5B	59
Figure 4.13: Failure modes of Retrofits 6A and 6B	60
Figure 4.14: Failure modes of Retrofits 7A and 7B	61
Figure 4.15: Failure modes of Retrofits 8A and 8B	62

Figure 4.16: Failure modes of Retrofits 9A and 9B	64
Figure 4.17: Dynamic representative failure mode for restored beams.....	65
Figure 4.18: Static resistance curves of glulam beam specimens.....	68
Figure 4.19: Experimental dynamic time varying functions for B8.2-[137]	72
Figure 4.20: Damage progression in beam B8.2-[137]	73
Figure 4.21: Dynamic resistance curves of glulam beam specimens	74
Figure 4.22: Representative experimental data for C4.1-[137]	76
Figure 4.23: Comparison of displacement- and resistance-time histories for columns.....	78
Figure 4.24: Damage progression in beam C4.1-[137]	78
Figure 4.25: Resistance curves of Retrofits 1 to 4.....	80
Figure 4.26: Dynamic resistance curves for Retrofits 1 through 4.....	82
Figure 4.27: Dynamic resistance curves of Retrofits 5 and 6.....	83
Figure 4.28: Dynamic resistance curves of Retrofits 7, 8, and 9.....	84
Figure 4.29: Comparison of resistance curve of restored beams to unretrofitted beam	85
Figure 5.1: Wood input stress-strain relationship.....	88
Figure 5.2: FRP input stress-strain relationship.....	89
Figure 5.3: Strain, stress, and force diagrams for a rectangular wood cross-section.....	90
Figure 5.4: Representative moment-curvature from program for specimen B-[137].....	92
Figure 5.5: Strain, stress, and force diagrams for a rectangular wood cross-section with ..	93
U-shaped FRP	93

Figure 5.6: Representative moment-curvature from program for FRP reinforced specimen	96
Figure 5.7: Typical resistance curve for unretrofitted specimen	97
Figure 5.8: Details of projected shortening and elongation	98
Figure 5.9: Example of composite resistance curve	100
Figure 6.1: Dynamic increase factors on the resistance of glulam beams	103
Figure 6.2: Comparison of MOE and tensile failure strain for unretrofitted beams	105
Figure 6.3: Wood input stress-strain relationship	106
Figure 6.4: Prediction of unretrofitted beams' resistance curves	107
Figure 6.5: Representative predicted displacement-time histories for the unretrofitted beams	108
Figure 6.6: SDOF predicted to experimental results	109
Figure 6.7: Predicted resistance curve and axial load comparison to experimental results	112
Figure 6.8: Representative SDOF predicted displacement-time history for columns	114
Figure 6.9: Static resistance curve comparison of retrofitted to unretrofitted beams	116
Figure 6.10: Retrofits' dynamic resistance curve comparison to static counterpart	117
Figure 6.11: Retrofits' dynamic resistance curve comparison to static counterpart	119
Figure 6.12: Retrofits' dynamic resistance curve comparison to static counterpart	121
Figure 6.13: Comparison of resistance curve of restored beams to unretrofitted beam	122
Figure 6.14: Prediction of unretrofitted beams' resistance curves	126
Figure A.1: Experimental stress-strain curves for B-[86] coupons	139

Figure A.2: Representative failure modes for B-[86] coupons.....	139
Figure A.3: Experimental stress-strain curves for B-[137] coupons	141
Figure A.4: Representative failure modes for B-[86] coupons.....	141
Figure B.1: Stress-strain relationship for GFRP [0] ₂ (FyfeCo)	143
Figure B.2: Representative failure mode for GFRP [0] ₂ (FyfeCo).....	144
Figure B.3: Stress-strain relationship for CFRP [0] ₂ (FyfeCo)	144
Figure B.4: Representative failure modes for CFRP [0] ₂ (FyfeCo)	145
Figure B.5: Stress-strain relationship for GFRP [0] ₂	146
Figure B.6: Representative failure modes for GFRP [0] ₂	147
Figure B.7: Stress-strain relationship for GFRP [± 45] ₂	147
Figure B.8: Representative failure modes for GFRP [± 45] ₂	148
Figure B.9: Stress-strain relationship for GFRP [0/90] ₂	149
Figure B.10: Representative failure modes for GFRP [0/90] ₂	150
Figure B.11: Stress-strain relationship for GFRP [0/90, ± 45].....	150
Figure B.12: Representative failure modes for GFRP [0/90, ± 45].....	151
Figure B.13: Stress-strain relationship for GFRP [± 45] ₄	152
Figure B.14: Representative failure modes for GFRP [± 45] ₄	153
Figure B.15: Stress-strain relationship for GFRP [0][0/90] ₂	153
Figure B.16: Representative failure modes for GFRP [0][0/90] ₂	154
Figure B.17: Stress-strain relationship for GFRP [0] ₃ [0/90] ₂	155

Figure B.18: Representative failure modes for GFRP $[0]_3[0/90]_2$	156
Figure B.19: Stress-strain relationship for GFRP $[0]_2[\pm 45]_2$	156
Figure B.20: Representative failure modes for GFRP $[0]_2[\pm 45]_2$	157
Figure B.21: Stress-strain relationship for GFRP $[0,0/90,\pm 45]$	158
Figure B.22: Representative failure modes for GFRP $[0,0/90,\pm 45]$	159
Figure C.1: Static test results for B1-[80] through B4-[80] beams	165
Figure C.2: Static test results for B5-[80] through B8-[80] beams	166
Figure C.3: Static test results for B9-[80] through B11-[80] beams	167
Figure C.4: Static failure modes of B-[80] beams (Tension side facing down)	168
Figure C.5: Static test results for B-[86] beams.....	169
Figure C.7: Static test results for B-[137] beams.....	171
Figure C.9: Dynamic test results for B12.1-[80]	173
Figure C.10: Dynamic test results for B12.2-[80]	174
Figure C.11: Dynamic test results for B13.1-[80]	175
Figure C.12: Dynamic test results for B13.2-[80]	176
Figure C.13: Dynamic test results for B14.1-[80]	177
Figure C.14: Dynamic test results for B14.2-[80]	178
Figure C.15: Dynamic test results for B15.1-[80]	179
Figure C.16: Dynamic test results for B15.2-[80]	180
Figure C.17: Dynamic test results for B16.1-[80]	181

Figure C.18: Dynamic test results for B17.1-[80]	182
Figure C.19: Dynamic test results for B17.2-[80]	183
Figure C.20: Dynamic test results for B18.1-[80]	184
Figure C.21: Dynamic test results for B19.1-[80]	185
Figure C.22: Dynamic test results for B20.1-[80]	186
Figure C.23: Dynamic test results for B21.1-[80]	187
Figure C.24: Dynamic failure modes of B-[80] beams	188
Figure C.25: Dynamic test results for B5.1-[86]	189
Figure C.26: Dynamic test results for B5.2-[86]	190
Figure C.27: Dynamic test results for B5.3-[86]	191
Figure C.28: Dynamic test results for B6.1-[86]	192
Figure C.29: Dynamic test results for B6.2-[86]	193
Figure C.30: Dynamic test results for B7.1-[86]	194
Figure C.31: Dynamic test results for B7.2-[86]	195
Figure C.32: Dynamic test results for B8.1-[86]	196
Figure C.33: Dynamic test results for B8.2-[86]	197
Figure C.34: Dynamic failure modes of B-[86] beams	198
Figure C.35: Dynamic test results for B5.1-[137]	199
Figure C.36: Dynamic test results for B5.2-[137]	200
Figure C.37: Dynamic test results for B6.1-[137]	201

Figure C.38: Dynamic test results for B6.2-[137]	202
Figure C.39: Dynamic test results for B7.1-[137]	203
Figure C.40: Dynamic test results for B7.2-[137]	204
Figure C.41: Dynamic test results for B8.1-[137]	205
Figure C.42: Dynamic test results for B8.2-[137]	206
Figure C.43: Dynamic test results for B9.1-[137]	207
Figure C.44: Dynamic test results for B9.2-[137]	208
Figure C.45: Dynamic failure modes of B-[137] beams	209
Figure D.1: Test results for C1.1-[137]	211
Figure D.2: Test results for C2.1-[137]	212
Figure D.3: Test results for C3.1-[137]	213
Figure D.4: Test results for C4.1-[137]	214
Figure D.5: Test results for C5.1-[137]	215
Figure D.6: Test results for C6.1-[137]	216
Figure E.1: Static test results for Retrofits 1 and 2	220
Figure E.2: Static failure modes of Retrofits 1 and 2	221
Figure E.3: Static test results for Retrofits 3 and 4	222
Figure E.4: Static failure modes of Retrofits 3 and 4	223
Figure E.5: Test results for R1-3.1-[137]	224
Figure E.6: Test results for R1-4.1-[137]	225

Figure E.7: Dynamic failure modes of Retrofit 1	226
Figure E.8: Test results for R2-3.1-[137]	227
Figure E.9: Test results for R2-4.1-[137]	228
Figure E.10: Dynamic failure modes of Retrofit 2	229
Figure E.11: Test results for R3-3.1-[137]	230
Figure E.12: Test results for R3-3.2-[137]	231
Figure E.13: Test results for R3-4.1-[137]	232
Figure E.14: Test results for R3-4.2-[137]	233
Figure E.15: Dynamic failure modes of Retrofit 3	234
Figure E.16: Test results for R4-3.1-[137]	235
Figure E.17: Test results for R4-3.2-[137]	236
Figure E.18: Test results for R4-4.1-[137]	237
Figure E.19: Dynamic failure modes of Retrofit 4	238
Figure E.20: Test results for R5-A-[137].....	239
Figure E.21: Test results for R5-B-[137].....	240
Figure E.22: Dynamic failure modes of Retrofits 5.....	241
Figure E.23: Test results for R6-A-[137].....	242
Figure E.24: Test results for R6-B-[137].....	243
Figure E.25: Dynamic failure modes of Retrofits 6.....	244
Figure E.26: Test results for R7-A-[137].....	245

Figure E.27: Test results for R7-B-[137].....	246
Figure E.28: Dynamic failure modes of Retrofits 7.....	247
Figure E.29: Test results for R8-A-[137].....	248
Figure E.30: Test results for R8-B-[137].....	249
Figure E.31: Dynamic failure modes of Retrofits 8.....	250
Figure E.32: Test results for R9-A-[137].....	251
Figure E.33: Test results for R9-B-[137].....	252
Figure E.34: Dynamic failure modes of Retrofits 9.....	253
Figure E.35: Test results for D1.1-[137].....	254
Figure E.36: Test results for D2.1-[137].....	255
Figure E.37: Test results for D3.1-[137].....	256
Figure E.38: Dynamic failure modes of damaged beams	257
Figure F.1: Idealized system with external and internal forces for the elastic range	258
Figure G.1: Flowchart for moment-curvature process.....	264
Figure H.2: Flowchart for resistance curve process	265
Figure H.1: Case 1 – $E_{wt} = E_{wc}$; $m = 0.01$	267
Figure H.2: Case 2 – $E_{wt} = E_{wc}$; $m = 0.2$	268
Figure H.3: Case 3 – $E_{wt} = 2E_{wc}$; $m = 0.01$	269
Figure H.4: Case 4 – $E_{wt} = 2E_{wc}$; $m = 0.2$	270
Figure H.5: Case 5 – $E_{wt} = 0.5E_{wc}$; $m = 0.01$	271

Figure H.6: Case 6 – $E_{wt} = 0.5E_{wc}$; $m = 0.2$	272
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CHAPTER 1 - Introduction

1.1 Research needs

The need for sustainable construction has led to more focus being directed towards renewable building materials, such as wood. Glued-laminated timber (glulam) is an engineered wood product (EWP) that has been used for decades in low-rise constructions and recently also in mid- to high-rise buildings in North America and around the world. Examples of such buildings includes the Brock Commons building in B.C Canada, the Integrated Design Building of the University of Massachusetts in Amherst U.S., and the headquarters of the Swiss media corporation (Tamedia) in Zurich.

Glulam consists of two or more layers of dimensional lumber joined together with glue in the parallel-to-grain direction. The individual pieces of lumber used in the assembly of the section are stress-rated, thereby allowing for tighter control on the individual lamination's properties to achieve configurations with more predictable characteristics compared to those found in sawn timber. To obtain greater dimensions, the different lumber pieces are connected using finger-joints, allowing for glulam members of significant dimensions.

Significant work has been done to establish the material properties (e.g. Fox, 1978; Moody et al., 1983; Xiong, 1985; Plevris and Triantafillou, 1992; Plevris and Triantafillou, 1995; Lee and Kim, 2000; Davids et al., 2008) as well as the structural behaviour under environmental loads such as wind and earthquake (e.g. Buchanan and Fairweather, 1993; Bjertnaes and Malo, 2014) for glulam structural elements. An area in which there is clear lack of research is the behaviour of glulam structural members under blast loading. The increased presence of glulam in high-profile structures may put them at risk for deliberate attacks or accidental explosions. Protecting these structures can be achieved by developing accurate evaluation strategies, as well as economically viable and structurally sound retrofit techniques.

Blast loads involve a load duration in the order of milliseconds in comparison to earthquake loads, where the duration is typically measured in seconds. Therefore, under blast loading, the structural elements' response is generally affected by the high strain rates generated

during loading. Materials typically experience an increase in the apparent strength under high strain rates, a phenomenon that is addressed through the dynamic increase factor (DIF). The DIF is used in analysis and design to modify the static strength of a material in order to account for the dynamic effects experienced under blast loading (e.g. Department of Defense, 2008; ASCE/SEI 59-11, 2011; CSA, 2012).

Investigation of material behaviour and structural response under blast loading is currently of great interest to researchers and structural engineers. As a result, the structural performance of reinforced concrete members has been well documented through experimental air blast loading (Woodson and Baylot, 1999; Muszynski and Purcell, 2003; Ghani Razaqpur et al., 2007; Magnusson, 2007; Carriere et al., 2009; Siba, 2014), shock tube induced blast loading (Lloyd, 2010; Jacques, 2011; Burrell, 2012; Toikka, 2012; Jacques et al., 2015; Lloyd, 2015; Jacques, 2016), as well as analytically through high fidelity modelling (Malvar et al., 1997; Abladey, 2013; Crawford, 2013; Kyei, 2014) and simplified single-degree-of-freedom modelling (Nassr et al., 2012b; Jacques et al., 2013; Burrell et al., 2015; Kadhom, 2015; Lacroix and Doudak, 2015; Lloyd, 2015; Jacques, 2016; Viau, 2016). Similarly, research has been conducted on the performance of structural steel (Nabil Bassim and Panic, 1999; Salim et al., 2003; Magallanes et al., 2006; Jama et al., 2009; Nassr et al., 2012a; Nassr et al., 2013; Nassr et al., 2014) and masonry members (Myers et al., 2004; Crawford et al., 2008; Tan and Patoary, 2009; Urgessa and Maji, 2010; Abou-Zeid et al., 2011; Ciornei, 2012) through experimental tests and analytical work. The extensive research available on those materials has allowed for detailed guidelines and development of response limits that can currently be found in various blast design standards (e.g. Department of Defense, 2008; ASCE/SEI 59-11, 2011; CSA, 2012).

Contrarily, little research has been undertaken to establish the behaviour of wood structural elements when subjected to blast loading despite their inclusion in some blast design standards (e.g. CSA 2012). For example, glulam is assigned a dynamic increase factor (DIF) identical to that of light-frame structures although no experimental tests have been conducted to support such decision. Also, the Canadian blast standard (CSA, 2012) provides design guidelines on the strengthening and retrofitting of structural elements using fibre reinforced composite materials for reinforced concrete and masonry elements, however, the standard

includes no provisions for wood elements. This is primarily due to the lack of research in this area. Particularly, there has been no experimental investigations on the behaviour of fibre-reinforced glulam members when subjected to blast loading. There is, however, an extensive knowledge on the behaviour of fibre reinforced polymers (FRP) used to strengthen timber beams under static loads. The behaviour of clear wood reinforced with FRP bonded sheets has been investigated by Plevris and Triantafillou (1992), who showed that reinforcement ratios above 3% provided no significant increase in capacity. Lindyberg and Dagher (2012) developed a non-linear probabilistic model for analyzing FRP tension reinforced glulam beams in bending using moment-curvature analysis. Raftery and Harte (2013) developed a non-linear finite element modelling (FEM) that adequately captured the flexural behaviour of FRP reinforced glulam members.

Several studies have shown that the main drawback in using simple tension reinforcement is the accompanied partial- or full-length de-bonding of the reinforcement when the outer wood tension layers fail (Dorey and Cheng, 1996; Sonti et al., 1996; Hernandez et al., 1997). Buell and Saadatmanesh (2005) investigated the effect of wrapping timber beams with carbon FRP (CFRP) and Johns and Lacroix (2000) investigated the effect of a U-shaped tension reinforcement on sawn lumber, which was found to provide confinement to the wood, and thereby reducing the effect of defects. Even with the significant body of work on FRP retrofitted timber elements, to the author's knowledge no studies have investigated the behaviour of the FRP-wood composite elements under blast-like loading.

1.2 Research objectives

The overarching aim of the research is to investigate the behaviour of glulam and glulam-FRP composite elements subjected to blast loading. More specifically, the goals are to:

1. Investigate the ability of a shock tube used in conjunction with a load transfer device to generate high strain rates flexural response in glulam elements;
2. Investigate the flexural response and failure mode of glulam beams under static and dynamic loads for the purpose of establishing an appropriate dynamic increase factor;

3. Investigate the response of glulam columns (including axial loads) subjected to dynamic loads and their associated failure modes;
4. Develop a procedure incorporating the strain-rate effects, which uses tension and compression strengths as input, to predict the flexural resistance-deflection relationships for the case of zero, constant, and variable axial load;
5. Investigate the static and dynamic flexural response of GFRP and CFRP retrofitted beams and their associated failure modes and extend the proposed strain-rate model to include FRP;
6. Evaluate the validity current blast design requirements for glulam elements and provide, where appropriate, recommendations for analysis and design approaches.

1.3 Scope

The above stated objectives are achieved through:

- Detailed literature review on the behaviour of wood subjected to high strain rates and the use of FRP as a retrofit for wood elements;
- Review of blast wave characteristics, dynamic analysis procedures, and current blast design guidelines for wood;
- Testing of thirty-eight glulam beams statically and dynamically to establish the DIF, resistance curve, and failure modes;
- Testing of six glulam columns dynamically to establish the dynamic behaviour, resistance curves, and failure modes;
- Testing of sixteen FRP retrofitted glulam beams using uniaxial fabric both statically and dynamically;
- Investigating the effect of multi-directional FRP fabric on the dynamic response of ten FRP retrofitted glulam beams;
- Testing of wood and FRP coupons to use as input in the material model;
- Discussing the results by comparing the analytical and experimental results and proposing design recommendations.

The scope of this project is limited to idealized boundary conditions in order to limit variables affecting the flexural response. Specific limitations of the research are further discussed throughout the thesis as well as in the future recommendation section of Chapter 7.

1.4 Structure of thesis

The experimental component of the research can be divided into three distinct phases: investigation of the DIF and behaviour of glulam elements with no axial load; effect of different levels of axial load on the dynamic response; and effect of different FRP configurations on the dynamic behaviour of glulam. A total of seventy (70) beams and columns were tested, and hence a large portion of the thesis is dedicated to the experimental program and the reporting and discussion of the results. Key results are presented within the body of the thesis, whereas information tables for each phase are presented in appendices. The following provides a brief description of each chapter in this thesis:

Chapter 1 introduces the subject and provides research needs, key objectives and scope.

Chapter 2 presents a detailed literature review comprised of studies investigating the behaviour of wood under the effects of dynamic loading, the effect of axial load on wood elements, and the use of FRP as a reinforcing material for wood flexural elements.

Chapter 3 presents the experimental methodology employed in the research program, and provides a detailed description of the specimens tested and the test setups used.

Chapter 4 presents the static and dynamic test results for both the retrofitted and unretrofitted beams and columns. It also provides the data obtained from the coupon tests to be used in the material model. Failure modes are also reported in this chapter.

Chapter 5 provides a description of the modelling approach and proposed material model.

Chapter 6 discusses the results from both the experimental and analytical program. The derivation of the DIF, effect of axial load, FRP as a retrofit option, and implications on current code provisions, where appropriate.

Chapter 7 summarizes key findings and proposes potential future work.

Appendix A provides individual test results for the wood coupons and their failure mode.

Appendix B includes stress-strain relationship and failure mode for all FRP coupons matrix investigated.

Appendix C provides detailed test results for the unretrofitted beams tested statically and dynamically.

Appendix D provides results for the columns' dynamic tests.

Appendix E provides the results for both the static and dynamic testing of the retrofitted beams.

Appendix F presents the derivation of the equation for the dynamic resistance for glulam beams using an equivalent single-degree-of-freedom (SDOF) approach.

Appendix G provides details on the procedure used to derive moment-curvature and resistance deflection relationships for the glulam beams and columns.

Appendix H provides the results for a sensitivity analysis in which the stiffness properties used as input to the material model were investigated and discussed.

1.5 Original Contributions

The current research is expected to advance the knowledge base on the behaviour of wood (specifically glulam) under high strain rates effects. Whereas some studies have investigated the static flexural response of glulam beams as well as the strengthening of FRP, there has been no research investigating the dynamic response of unretrofitted and FRP retrofitted glulam members under high strain rates. The experimental program generates an extensive database with a total of seventy glulam specimens tested destructively with the aim to provide

valuable information for future researchers and structural engineers on the analysis and design of unretrofitted or FRP retrofitted glulam members.

Being the first study of its kind, fundamental concepts such as material behaviour (DIF), failure modes under static and dynamic loading and effect of axial load on the response were necessary for the development of a basic understanding of the behaviour. Investigating different FRP configurations provided important knowledge on the suitability of various retrofits for the purpose of damage prevention specifically for blast design considerations with emphasis on enhancing post-peak behaviour and limiting debris.

CHAPTER 2 - Background and Literature Review

2.1 General

Investigation of the material behaviour and structural response under blast loading is a topic of great interest to researchers and structural engineers, mainly due to the devastating consequences associated with blast events (e.g. Alfred P. Murrah Building, Oklahoma City, 1995). The increased interest in the field has led to the development of blast standards (e.g. Department of Defense, 2008; ASCE/SEI 59-11, 2011; CSA, 2012) giving designers provisions to perform risk assessment for a given blast threat as well as response limits to guide the analysis. These response limits were derived for materials such as concrete, steel, and masonry, and while available for timber and engineered wood products (EWPs), they were not determined based on adequate experimental investigation. The following provides background knowledge on the material properties of wood and FRP. Also, the characteristics of blast loading along with common analysis and blast testing methods are presented. A detailed literature review on previous studies for wood under high strain-rates, combined loading, and FRP as a retrofit option is presented. While the focus of this study is on the behaviour of wood and wood-FRP composite, findings of selected studies dealing with the behaviour of other materials under the effect of blast loading are discussed.

2.2 Wood as a construction material

2.2.1 Physical properties of wood

Wood is a renewable building material providing clear advantages when considering sustainable design. Being also a natural material, the tree growth ultimately affects the properties of the transformed products. For example, trees that grow fast will have large annual rings, lower density, and higher taper and grain deviation (Breyer et al., 2007).

Wood can be considered as an orthotropic material with distinct mechanical properties in the longitudinal, radial, and tangential directions (Ross, 2010).

Wood is also a viscoelastic material, with mechanical properties that are time-dependent and affected by rate of loading, load duration, creep, and fatigue. Figure 2.1 shows the relative

bending stress as a function of load duration. The figure clearly shows that an increase in strength is observed in very short load durations (less than 5 minutes) compared to that found during standard static testing (approximately 10 minutes). Creep effects also play a significant role for long duration loads (e.g. years), where the wood strength approaches 50-60% of its ultimate short-term strength (e.g. Breyer et al., 2007; Ross, 2010). As can be seen in Figure 2.1, the graph starts at 1 s, which represents the knowledge base at the time it was developed. The current study and contemporary research effort aim to shed more light on the behaviour of various wood elements in the range of very short load durations (e.g. milliseconds).

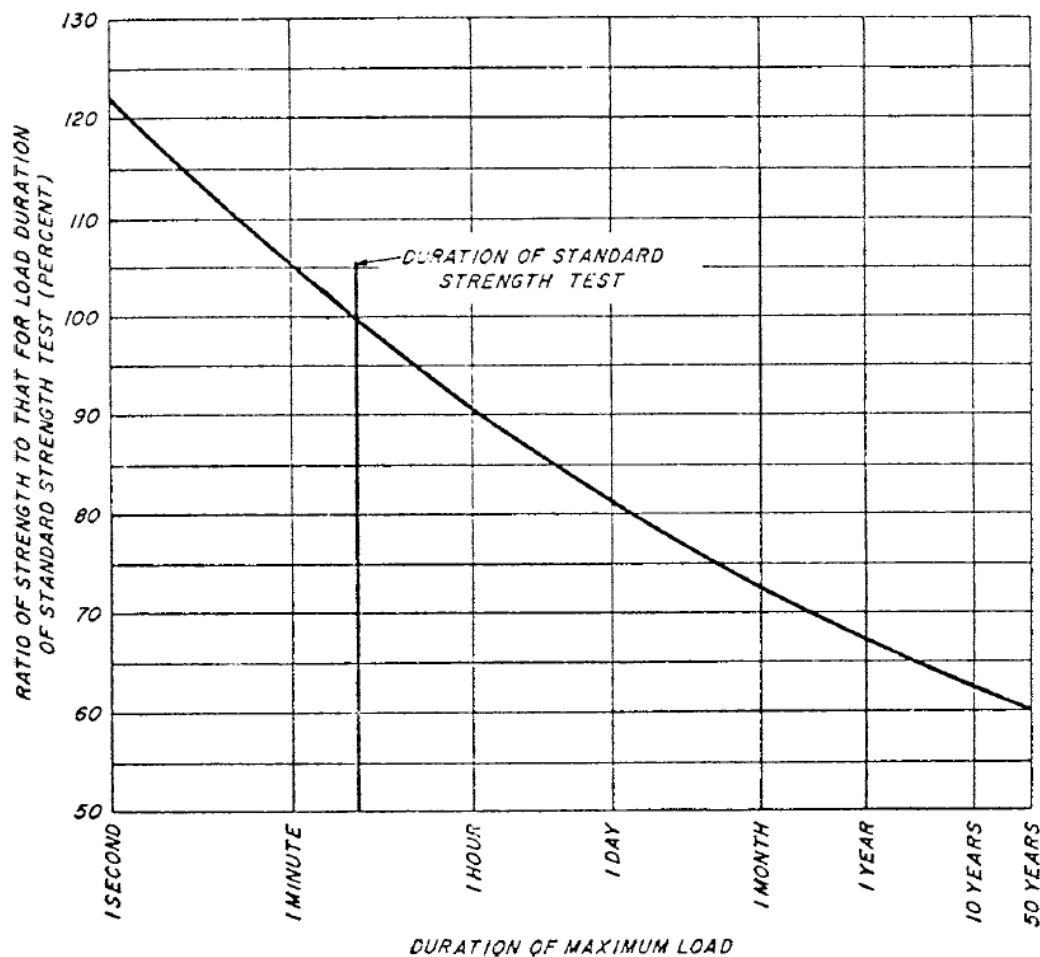


Figure 2.1: Effect of load duration on maximum bending stress

*Reproduced from ASTM (2011)

Adding to the complexity of wood is the fact that clear wood (i.e. wood with no defects) behaves differently than lumber and timber members typically used in construction (Barrett and Lau, 1994; Breyer et al., 2007; Ross, 2010). A flexural failure of a clear wood member will typically be governed by a compression failure with wrinkling of the fibres on the compression side whereas lumber or timber with defects will be governed by failure on the tension side as shown in Figures 2.2a to 2.2d (ASTM, 2014a). Although less common, other types of failure for lumber and timber will include compression (Fig. 2.2e) and horizontal shear (Fig. 2.2f) (ASTM, 2014a).

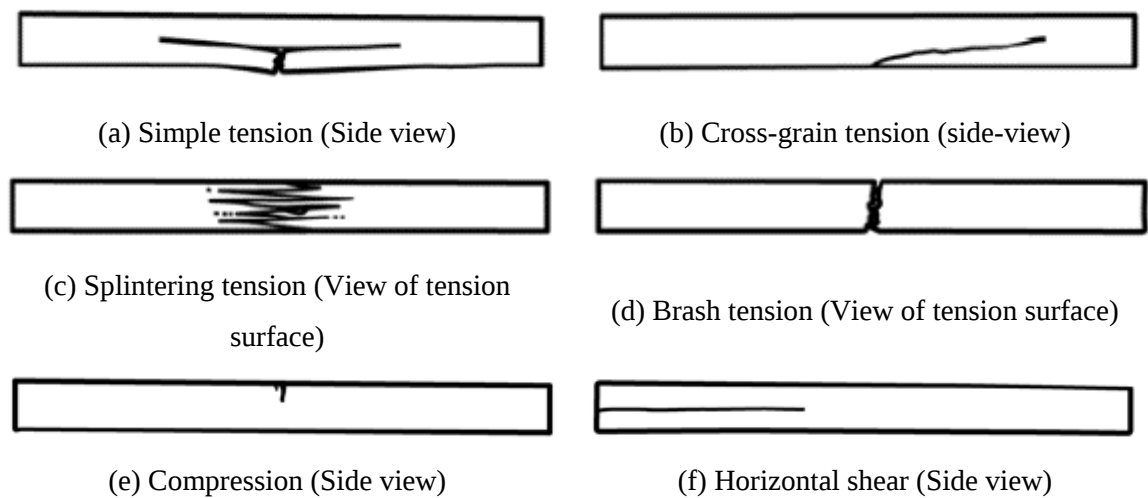
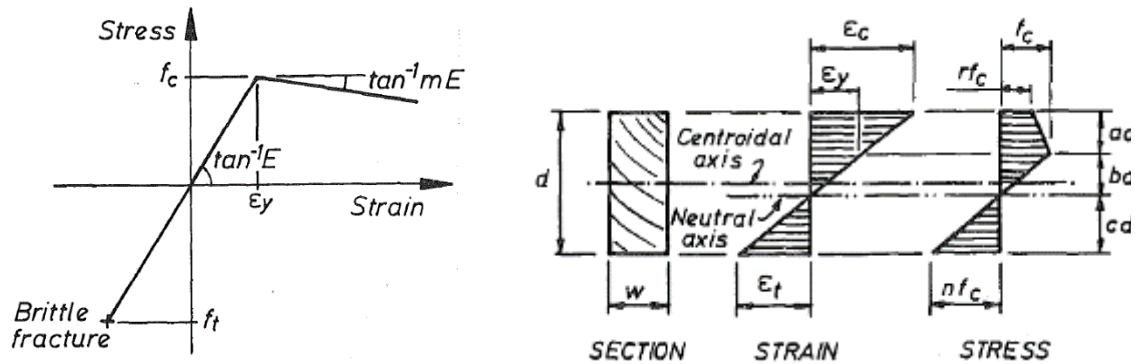


Figure 2.2: Types of failures in static bending

*Reproduced from ASTM (2014a)

2.2.2 Flexural resistance of wood

The moment-curvature relationship of a wooden member can be described using tension and compression strengths parallel to the grain along with their corresponding stress-strain relationships as shown in Figure 2.3 (Buchanan, 1986; Buchanan, 1990).



(a) Stress-strain relationship

(b) Distribution of stress and strain

Figure 2.3: Model proposed by Buchanan (1990)

*Reproduced from Buchanan (1990)

Originally developed by Bazan (1980) for clear wood, the stress-strain relationship for tension and compression was later modified by Buchanan (1990) to reflect in-grade lumber. The tension behaviour of wood is assumed to be linear elastic with failure occurring at a stress, f_t , corresponding to strain, ϵ_t . The behaviour in compression can be represented with a bi-linear curve where the first portion is defined by compression yielding at a stress f_c , corresponding to a strain, ϵ_{cy} , followed by a linear falling branch. Buchanan (1990) defined the sloped of the falling branch to be a constant ratio, m , of the modulus of elasticity, E . Buchanan (1990) further developed the methodology to include the effects of in-grade strength of commercial quality lumber throughout the distribution of properties. Size effects and stress distribution effects were considered and their implications will further be discussed in Chapter 5.

2.2.3 Engineered wood products

The advancement in manufacturing technologies to produce engineered wood products (EWP) has allowed for wood to be used beyond traditional low-rise light-frame structures and to become a viable material option for much larger structures. Such EWP include, but are not limited to, glued-laminated timber (glulam), cross-laminated timber (CLT), parallel strand lumber (PSL), and laminated veneer lumber (LVL).

In the current study, glulam was selected primarily because it is generic and widely used in the construction industry. Figure 2.4 shows some of the advantages of glulam as a structural

material where it can be seen that in addition to achieving large spans, it is also possible to construct curved and tapered members.



Figure 2.4: Richmond Oval

*Reproduced from Canadian Wood Council (2010)

As discussed in Chapter 1, glulam consists of two or more layers of dimensional lumber joined together with glue in the parallel-to-grain direction and to obtain greater dimensions, the different lumber pieces are connected using finger-joints (Fig. 2.5).

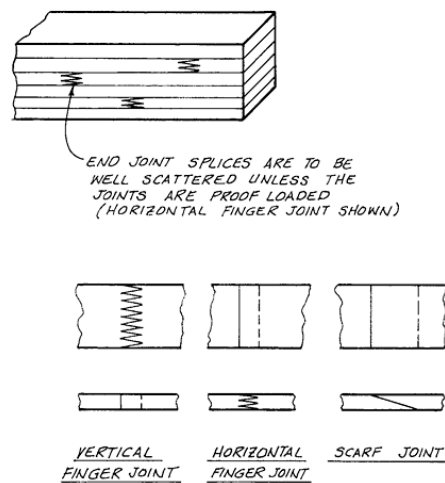


Figure 2.5: Examples of finger joints

*Reproduced from Breyer et al. (2007)

2.3 FRP as a strengthening material

2.3.1 Overview of FRP materials

Fibre reinforced polymers (FRP) consists of fibres (main load carrying component) embedded in resin. FRP has been used in the construction industry primarily for retrofitting and rehabilitating existing structural elements as well as for new constructions. The most common type of fibres in civil engineering applications are carbon FRP (CFRP), glass FRP (GFRP), and aramid FRP (AFRP). Aramid FRP has very low density and high specific tensile strength in comparison to other reinforcing fibres. AFRP is light weight and has a high impact damage tolerance, which is why its best-known usage is in bullet proof vests. However, it is extremely sensitive to environmental conditions and is not always suitable for structural applications. Glass FRP is widely used due to its lower cost, high strength, low weight relative to steel, and availability. However, GFRP also has disadvantages with its lower stiffness and fatigue resistance relative to other reinforcing fibres, and is subject to creep under high sustained loading. Carbon FRP has been increasingly used in structural applications due to its high stiffness and strength as well as high fatigue resistance. The main disadvantage of CFRP is its high cost (McDaniel and Knight, 2014).

Figure 2.6 shows the stress-strain behaviour of the three types of FRP discussed compared to that of mild steel. The modulus and strength used for the FRP were the average typical values provided by the American Concrete Institute “Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures” (American Concrete Institute, 2008). From Figure 2.6, it can be seen that while FRP has an ultimate tensile strength that is much higher than that of mild steel, its behaviour is characterized by a linear-elastic response and brittle failure.

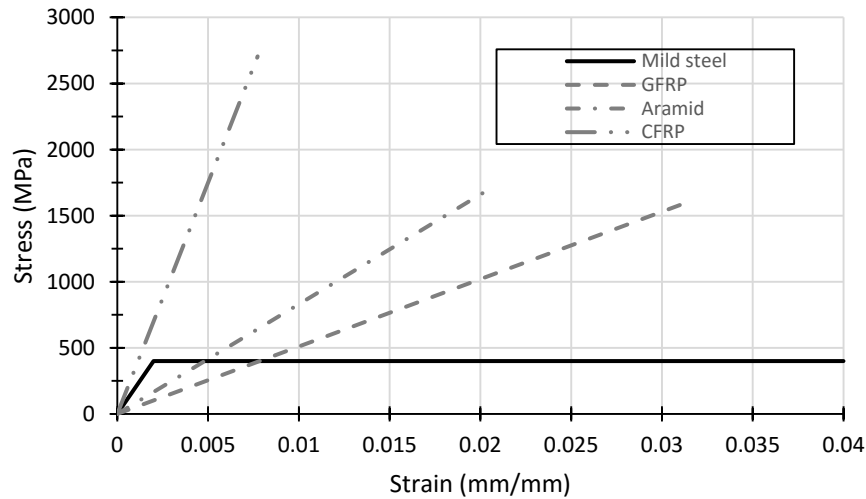


Figure 2.6: Typical stress-strain curves for common FRP and mild steel

American Concrete Institute (2008)

Thermoset and thermoplastic are the two categories of resin available, however, they differ significantly from one another. Thermoset resins are most common for structural uses. They are liquid at room temperature and become solid during the curing process. The curing process in thermoset resins cannot be reversed unlike for thermoplastic resins. The latter is solid at room temperature, and heated to a liquid state prior to being pressurized to impregnate the reinforcing fibres (McDaniel and Knight, 2014). The most common types of resin are: polyester, vinylester, polyurethane, and epoxy. Polyesters are easy to use and are less expensive, however, only moderate mechanical properties can be obtained. Vinyl esters offer higher mechanical properties than polyesters but are sensitive to heat and are more expensive. Similarly, polyurethanes offer higher strength, flexibility, chemical resistance, and mechanical properties than vinyl esters but at a higher cost. Finally, epoxies are the most expensive, however, they offer high mechanical and thermal properties as well as long working time. The downside, apart from the cost, is that the mixing of the quantities is critical for optimal results (McDaniel and Knight, 2014).

FRP fabrics are also produced with fibres in different orientation including bi-directional, triaxial, and quadriaxial. For the bi-directional fabrics, the fibres could be oriented at right angles relative to the specimens (0° and 90°) or at an angle (typically 45° - 45°). In such cases,

less fibres are oriented in the direction in which the load is applied and the response of the FRP may become non-linear. Currently, there is lack of research investigating the behaviour of multi-directional fabrics due to the higher cost associated with such fabrics.

2.4 Blast wave characteristics and effects on structures

2.4.1 Blast loading

In blast engineering, the benchmark comparison for high explosives is trinitrotoluene (TNT) since its blast wave parameters are well known due to the relative ease of commercial transportation, pureness, and availability of the explosive. The sudden release of energy associated with the detonation of a high explosive forms a shock front, which is propelled by the exothermic reaction, travelling at a speed faster than sound. A typical pressure-time history profile of a shock wave is shown in Figure 2.7 where it can be seen that at the time of arrival of the shock front, t_a , there is an immediate rise in atmospheric pressure, referred to as the peak incident pressure, P_s .

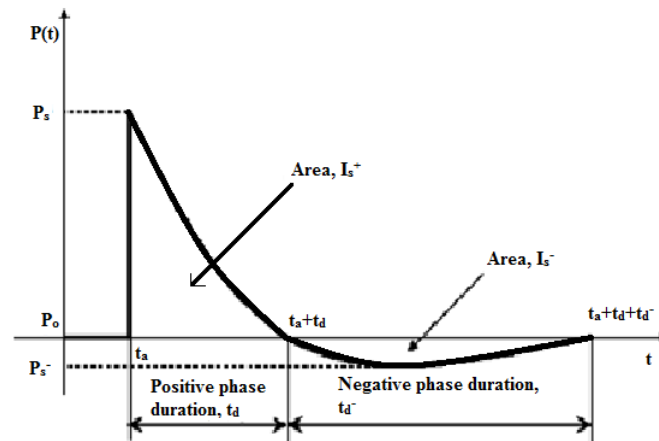


Figure 2.7: Typical blast wave history

*Reproduced from Dusenberry (2010)

In an ideal free-air explosion the shock front propagates spherically without any amplification, and thus the rise in pressure is solely due to the initial energy released. The velocity and temperature of the reaction decrease along with the peak incident pressure as the shock front propagates, up to ambient pressure, thereby defining the positive phase, t_d^+ ,

of the blast wave. The pressure then starts decaying below ambient pressure, which denotes the start of the negative phase, t_d^- , at time $t_a + t_d^+ + t_d^-$. The negative phase is caused by the rapid return to ambient pressure causing the air particles to rush back to the centre of the explosion. While the negative phase duration is significantly longer than the positive phase duration, the peak pressure observed in the negative phase is only a fraction of that observed in the positive phase. The negative phase is often neglected during the analysis of main structural elements such as beams and columns (Department of Defense, 2008; Krauthammer, 2008), however, it could have significant implications on design of for example connections between window frames and structural elements.

Equation [2.1] describes the pressure-time history, $P(t)$, using the Friedlander equation, while the positive, I^+ , and negative, I^- , impulses can be determined using Equations [2.2] and [2.3], respectively.

$$P(t) = P_s \left(1 - \frac{t}{t_d}\right) e^{-\frac{b \cdot t}{t_d}} \quad [2.1]$$

$$I^+ = \int_{t_a}^{t_a + t_d} P(t) dt \quad [2.2]$$

$$I^- = \int_{t_a + t_d}^{t_a + t_d + t_d^-} P(t) dt \quad [2.3]$$

where P_s is the incident peak pressure, t is the time, t_d is the positive phase duration, and b is the blast waveform parameter.

In reality, the shock wave will interact with reflecting surfaces (i.e. a structure) losing its spherical form while experiencing an amplification in magnitude. Therefore, the pressure acting on an object will not simply be the peak incident pressure shown in Figure 2.8 but rather a combination of the incident pressure, stagnation pressure, and dynamic pressure that results in a reflected pressure greater than the incident pressure. Concurrently, the reflected impulse is also greater. When interacting with a structure of finite dimensions and geometry, diffraction will occur in the blast wave along with drag forces. Therefore, the actual pressure-time history needs to be determined based on the geometry of the entire structure.

Given the high-cost associated with conducting field tests, the Hopkinson and Cranz scaling law, also known as the cube-root scaling law, was developed to compare the effect of different explosions detonated at different distances for similar atmospheric conditions, explosive geometry, and materials. The scaled distance, Z , relates the standoff distances, R , to the charge weights, W , as shown in Equation [2.4].

$$Z = \frac{R}{W^{1/3}} \quad [2.4]$$

Therefore, for the same scaled distance, two different charges with different stand-off distances can have the same reflected pressure, and thus the same response for the element under consideration. The scaled distance can be used to obtain different parameters such as the impulse and time of arrival by extending the concept of the cube root scaling (Krauthammer, 2008)

2.4.2 Dynamic analysis

The dynamic nature of blast loading involves a loading duration that is much smaller than that of earthquakes and wind loading, and as such the analysis of the structural response under such short loading needs to incorporate the inertia forces and kinetic energy.

Sophisticated modelling techniques, such as finite element analysis, have been employed to capture the dynamic response of structural elements subjected to blast loading (e.g. Abladey, 2013; Crawford, 2013; Kyei, 2014), however, given the uncertainties associated with blast loads, equivalent single-degree-of-freedom (SDOF) analysis technique has been proven to be adequate in capturing the dynamic response of various structural elements (e.g. Biggs, 1964; PDC-TR 06-01 Revision 1, 2008). Closed-form solutions are only practical in simple linear elastic systems subjected to forcing functions that can be described by simple mathematical formulations. The equivalent SDOF approach consists of replacing a real structural system by an equivalent spring-mass system that has the same properties as the real system through a load-mass factor, K_{LM} , which accounts for the load distribution, boundary conditions, and deflected shape. The equation of motion for an equivalent SDOF in its classical form is widely available in the literature (Biggs, 1964; Department of Defense, 2008; PDC-TR 06-01 Revision 1, 2008; Nassr et al., 2013). Equation [2.5] presents the

equation of motion for a SDOF system subjected to combined lateral and axial loading (Nassr et al., 2013; Dragos and Wu, 2015).

$$K_{LM}m\ddot{y}(t) + R(y, t) = P_R(t)A + \eta(y, t) \quad [2.5]$$

where K_{LM} is the load-mass factor, m is the total mass of the system, $R(y, t)$ is the resistance function, $P_R(t)$ is the reflected pressure-time history, A is the loaded area, $\eta(y, t)$ is the equivalent lateral load (ELL), and $\ddot{y}(t)$ is the acceleration of the system.

The equation is also valid for the case of no axial load by equating the equivalent lateral load factor, $\eta(y, t)$, to zero. It should be noted that the damping component is typically omitted since its effect on the first peak displacement is negligible (Biggs, 1964; Department of Defense, 2008).

2.4.3 Blast testing methods

Blast load effects on structures are commonly recreated by conducting live testing on full-scale structures or structural elements. Detonation of high explosives (e.g. TNT) at a stand-off distance corresponding to a specific blast pressure profile recreates the effects expected during an actual explosion. While this method is the most accurate in terms of applied pressure-time history, diffraction, and drag forces, conducting live explosive testing is expensive. Furthermore, the fire ball from the explosion may cause damage the instrumentation. An alternative to live explosive testing is the use of a Shock Tube, where high strain-rates similar to those experienced during blast loading can be generated. The Shock Tube is capable of producing a uniform pressure over the reflecting surface through the quick release of compressed air, thereby simulating a wide range of pressure and impulse combinations. While blast loads phenomena such as the negative phase, drag forces, and diffraction are not replicated, the advantages of using a Shock Tube over live explosive are the relatively low cost associated with such testing and the fact that a large number of tests can be conducted allowing to investigate the repeatability of the results. The Shock Tube facilitates the development of material characteristics and element responses under high strain-rates. This can help validate material models, which in turn can be used in, for

example, high fidelity modelling (e.g. LS-DYNA) in which blast load effects such as drag forces can be included.

2.5 Previous research

2.5.1 High strain-rate effects on wood properties

The existing body of knowledge has focused primarily on impact loading tests conducted on defect-free wood specimens. Increases in the range of 10-40% on the apparent compressive and flexural strengths have been reported (Liska, 1950; Spencer, 1978). Nadeau et al. (1982) conducted impact tests on clear specimens in order to determine the strain-rate effects and investigated the effects of initial cracks on the response. As a result, half of the specimens were intentionally notched on the tension face and compared to the remaining specimens that were not notched. The study reported that the specimens that were intentionally notched lacked the increase in strength observed in specimens without a notch. Mindess and Madsen (1986) conducted similar tests and also reported a lack of increase in the notched specimens. It should be noted that the aforementioned studies by Nadeau et al. (1982) and Mindess and Madsen (1986) may not be representative of the behaviour of timber elements used in construction as the testing was conducted on clear timber. The behaviour of small clear wood specimens has been shown not to be representative of the behaviour of full-size elements used in construction (Barrett and Lau, 1994). Jansson (1992) also found that the failure mode of timber elements, representative of those used in construction, was highly dependent on the quality of the tested specimens and that a higher strength increase was typically observed in the higher-quality lumber.

Limited field blast tests on full-scale light-frame wood structures have indicated that the overall damage to the structures is highly influenced by the damage inflicted to the rafters, studs, joists, and windows (Kimbell and Fies, 1953; Randall, 1955; AFH 10-2401, 2006). These studies focused on the overall behaviour of the structures rather than the quantification of the strain-rate effects.

More recently, research effort at the University of Ottawa has aimed at investigating the behaviour of light-frame wood stud walls subjected to simulated blast loading. Jacques et al.

(2014) conducted static and dynamic tests on thirty full-scale individual Spruce-Pine-Fir (SPF) studs (38 x 140 x 2,440 mm³) reporting dynamic increase factors of 1.41, 1.14, and 1.18 on the modulus of rupture (*MOR*), modulus of elasticity (*MOE*), and rupture strain (ϵ_d), respectively. However, following statistical analysis, it was determined that only the *MOR* was significantly affected by high strain-rate effects. Lacroix and Doudak (2015) conducted static and dynamic tests on twenty full-scale light-frames representative of typical Canadian construction practice and reported dynamic increase factors on the resistance and stiffness of 1.40 and 1.18, respectively. It was observed by Lacroix and Doudak (2015) that the sheathing, which acts as a load distributor to the load-bearing elements, failed prior to the studs in certain configurations and prompted a new study that investigated possible retrofit options for light-frame wood stud walls (Lacroix et al., 2014). Retrofits varied from replacing the existing OSB sheathing with thicker plywood sheets, adding plywood atop of the existing OSB with screws, adding a stud flat-wise to the interior face of the wall studs, and adding a corrugated metal panel on the wall exterior. The retrofits improved the performance by reducing the displacement, damage, and nail withdrawal. Similarly, Viau and Doudak (2016a) investigated the behaviour of both typical and retrofitted light-frame stud walls in a region corresponding to hazardous-blowout damage levels. The typical construction walls with OSB showed that premature sheathing failure could occur prior to the studs' failure resulting in sheathing debris. The use of a thicker plywood shifted the failure to the studs while also decreasing the amount of debris. A novel retrofit method for light-frame wood stud walls was investigated where a welded wire mesh (WWM) was used as sheathing reinforcement or as a catcher system, both of which showed promising results. When used as a catcher system, the sheathing was allowed to fail, with minimal damage to the studs, and the debris was retained.

Although all the aforementioned studies on light-frame wood walls provided valuable information, all specimens had idealized simply supported boundary conditions. Viau and Doudak (2016b) investigated whether prescriptive code requirements for connection detailing for low and high seismic and wind regions was capable of resisting a given blast loading. Retrofitted connections consisting of joist hangers, hurricane twist straps, and steel angles were also investigated. The authors concluded that the typical nailed connections,

including those designed for high seismic regions, did not allow the studs to develop their full flexural capacity due to a premature failure, and thus resulting in hazardous debris. The joist hangers, when having a capacity less than the maximum dynamic reaction experienced significant withdrawal and damage to the stud ends whereas less damage was observed with connection capacity greater than the maximum dynamic reaction. In both cases, however, the studs were able to attain their ultimate flexural resistance. Viau and Doudak (2016b) concluded that simply basing the connection design on the capacity may not be adequate and that additional considerations for connection detailing should be taken into account.

Combining the data from their previous studies, Viau et al. (2016) proposed response limits for light-frame wood stud walls subjected to blast loading. Detailed documentation of the walls' damage allowed the authors to propose evaluation criteria developed based on the overall performance of the wall system and its estimated post-blast axial residual capacity. Using a normalized approach, new response limits that are more representative of the response of light-frame walls were proposed.

2.5.2 Response of structural elements subjected to combined transverse and axial loading

To the author's knowledge, no dynamic tests have been conducted on the response of wood structural elements subjected to combined blast and axial load. There are, however, several studies that investigated the effect of combined loading under static test regimes for which models were developed. Buchanan (1986) investigated the behaviour of combined bending and axial loading in lumber and verified the proposed model and variations of the stress-strain relationship against tests conducted by Buchanan (1984) and Bleau (1984). Buchanan (1986) showed that for increasing levels of axial loads that there is decrease in ultimate moment capacity as shown in Figure 2.8. No initial increase in stiffness due to increasing axial load level was observed. It should be noted that the curves in Figure 2.8 were shifted from the origin for clarity.

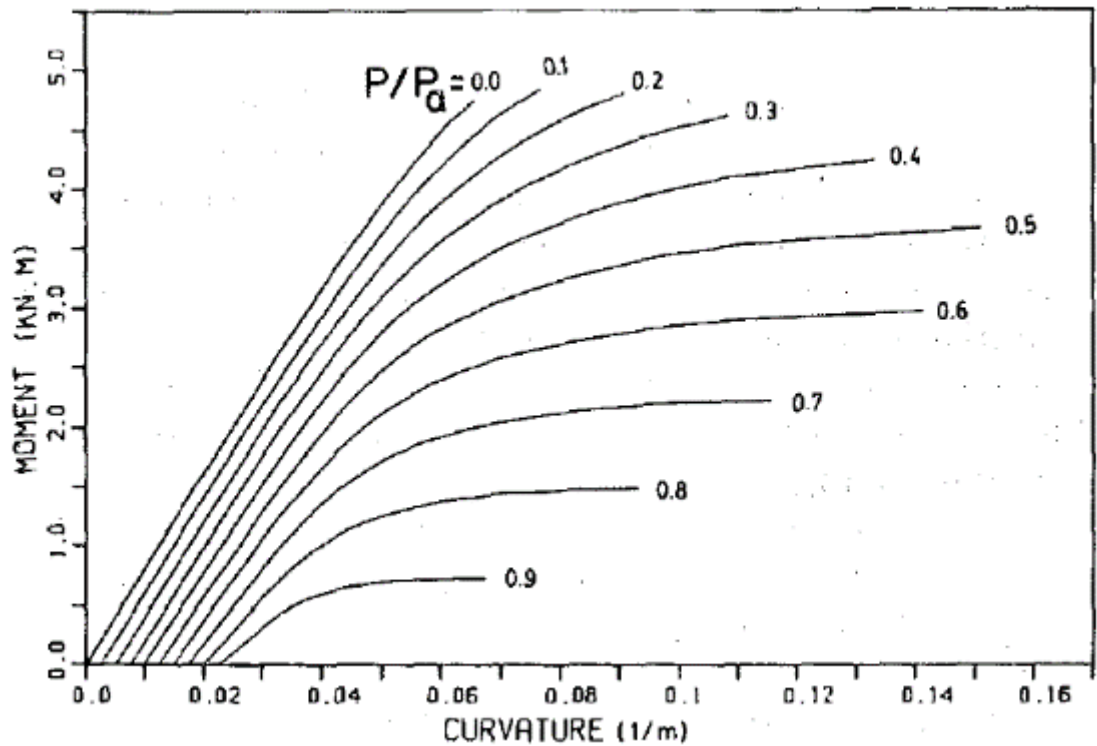


Figure 2.8: Moment curvature curves for constant axial load for a rectangular wood section

*Reproduced from Buchanan (1986)

Buchanan (1990) further extended this work by adapting the stress-strain relationship proposed by Bazan (1980) relating the tension, compression and bending strength of clear wood to that of commercial quality lumber that contains strength-reducing defects. The bilinear stress-strain relationship and the distribution of stress and strain for a rectangular section was developed and proposed by Buchanan (1990) (shown in Figures 2.3a and 2.3b). This model was also adapted for a hybrid FRP and wood member by Plevris and Triantafillou (1992) for the case of no axial and constant axial load. The model was developed for FRP tension reinforcement laminates where the bending strength of the member can be determined on the basis of four cases, where it was assumed that: i) both wood and FRP are linear-elastic; ii) wood is linear elastic and FRP had ruptured; iii) wood had yielded and FRP is linear-elastic iv) wood had yielded and FRP had ruptured.

Bulleit et al. (2005) also modelled the behaviour of wood stud wall systems subjected to combined transverse and axial loading. The model used an approximate deflection magnifier to account for second-order effects. The model output compared well with test results conducted previously (Gromala, 1983). A sensitivity analysis was conducted by the authors in which the effect of axial load among other variables was investigated and one of the outcome of interest to the current study was that the failure stress (i.e. MOR) was attained at a lower transverse load level for cases with axial load than without axial load.

There are design guidelines on how to account for the axial load in SDOF analyses for steel, concrete, and masonry due to the larger amount of research available on the subject (Department of Defense, 2008; PDC-TR 06-01 Revision 1, 2008). Of particular interest to this research are the thirteen steel columns tested by Nassr et al. (2014) under combined blast and axial loading. The columns were tested under constant axial load corresponding to 25% of their capacity simulated by using pre-stressing strands. The columns were tested simultaneously with companion beams (i.e. no axial load) in the same shots (Nassr et al., 2012a). Comparing the deformations of the columns to those of the beams it was found that for the elastic shots the deflection decreased. The axial load also had the effect of increasing the natural period, and thereby reducing the ratio of loading duration to natural period (t_d/T) by 4%. The behaviour of the column was different in the plastic range where the $P-\Delta$ had a tendency to dominate the overall response. The authors concluded that the axial-bending interaction may be neglected for axial load below 25 % of the column axial capacity if the columns remains elastic during its response. However, in the plastic deformation range the maximum lateral deformation was increased by 158% due to the $P-\Delta$ effects dominating the response. Nassr et al. (2013) reported that using a SDOF approach with an equivalent lateral load (ELL), which accounts for the secondary moments caused by the axial load, was appropriate to predict the displacement-time history of the column as well as pressure and impulse (P-I) diagrams by comparing it to that generated by finite element modelling.

Dynamic testing on reinforced concrete columns subjected to combined transverse blast and axial loading was conducted by Lloyd (2010; 2015). The axial load was applied by means of a hydraulic jack placed between the laboratory floor and column end, while the other end was shored between the laboratory strong floor and top of the column. Lloyd (2010) reported

that there was a considerable variation in the axial load due to the ends of the column moving in the direction of the column length as the column displaced laterally. Lloyd (2010; 2015) noticed little difference between the response of seismic and non-seismic specimens and attributed it to the fact that the loss of axial load resulted in a more beam like behaviour than column. Lloyd (2015) accounted for the axial load variation by forming a composite resistance curve which consisted of resistance points (deflection, resistance) that varied from one axial load level to another as the column displaced laterally. For a constant axial load level, resistance curves derived from moment-curvature analysis showed increases in stiffness and resistance as shown in Figure 2.9 (Lloyd, 2015).

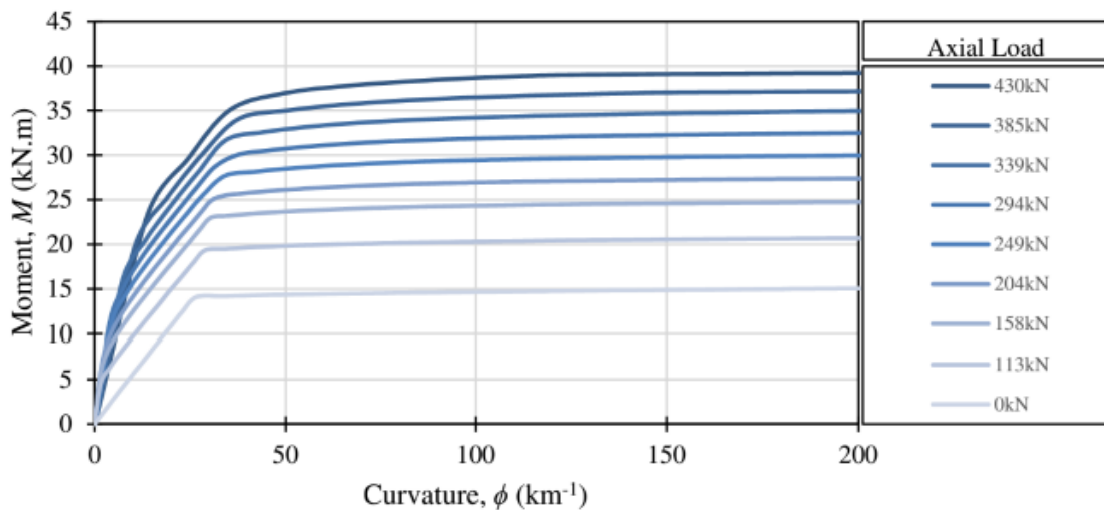


Figure 2.9: Moment curvature curves for various constant axial load for a seismic concrete reinforced column

*Reproduced from Lloyd (2015)

Lloyd (2015) reported that the loss in axial load observed in laboratory conditions occurred at a rate comparable to that of a mass in free-fall. For structures without significant continuity, it was recommended that a partial or complete loss of axial load should be considered. However, the study, also noted that the loss in axial level in structures with significant continuity may not be as significant and recommended that the designer evaluate the appropriate level of applied axial load and incorporate it into the analysis. Kadhon (2015) also conducted tests on reinforced concrete columns subjected to combined blast and axial

loading and observed a loss in axial load as the column displaced laterally. In addition to testing seismic and non-seismic columns under combined loading, Kadhon (2015) investigated the response of columns with CFRP jackets with different laminate design. The author reported that the jackets having fibres in the $\pm 45^\circ$ direction increased the columns' ductility and the plastic hinge lengths, thus allowing for more dissipation of energy during a blast.

2.5.3 FRP as a retrofit option for flexural strengthening

Enhancing the performance of existing buildings to meet contemporary code design requirements through various retrofitting techniques is now common practice in structural engineering. In the wood research community, improving the performance of wood-based materials goes as far back as the 1940s. The reinforcement of various wood products (e.g. joists, timber and laminated wood beams, plywood, particle board) was done using metal reinforcement (e.g. aluminium plates, steel plates, and steel bars), however, such retrofits have not necessarily been commercialized successfully (Bulleit, 1984).

Due to the wide array of literature available on reinforcement of wood members, emphasis is directed towards the techniques used for flexural reinforcement of wood beams. Such techniques include reinforcing outer tension laminates with steel and glass fibre reinforced polymer (GFRP) bars (Bulleit et al., 1989; Gentile et al., 2002; Yang et al., 2016), addition of steel plates (Mark, 1961), and prestressing or post-tensioning (De Luca and Marano, 2012; Al-Hayek and Svecova, 2014). Advancements in manufacturing and reduction in cost of fibre reinforced polymers (FRP) have meant that its application to enhance the performance of timber members is now viable and as a result has garnered a lot of attention from researchers within the wood community. An application in which FRP and wood are commonly used together is in the rehabilitation of timber bridges (Gentile et al., 2002; Buell and Saadatmanesh, 2005; Hay et al., 2006; CSA, 2014).

Retrofit configurations consisting mainly of FRP sheets on the outer tension laminate (Plevris and Triantafillou, 1992; Johns and Lacroix, 2000; Raftery and Harte, 2011; Yang et al., 2016), as shown in Figure 2.10a, and behind a bumper layer serving as protection layer

(Dagher et al., 1996; Raftery and Harte, 2011), as shown in Figure 2.10b, are examples of what have been previously investigated.

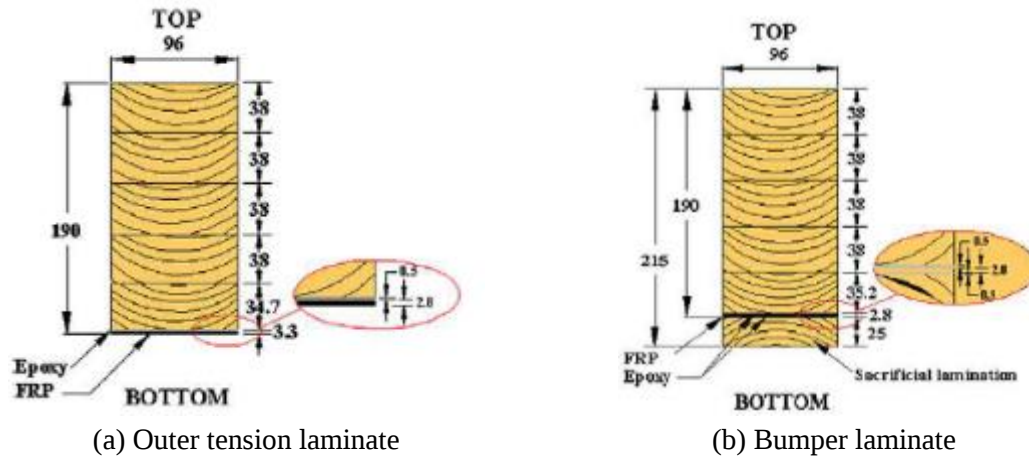


Figure 2.10: FRP sheets used as tension reinforcement in glulam beams

*Reproduced from Raftery and Harte (2011)

The addition of FRP strips to the tension face has shown to reduce the variability in wood properties and increase the ultimate failure strength (Johns and Lacroix, 2000; Gentile et al., 2002). Typically, increases in resistance and stiffness for wood reinforced with simple FRP tension reinforcement of up to 50% and 20%, respectively, can be attained. It is important to note that the strength and stiffness increase are highly dependent on FRP's properties as well as the quantity used to reinforce the specimens. Plevris and Triantafillou (1992) conducted experimental and analytical work on clear wood specimens reinforced with FRP sheets as external reinforcement at the outer tension layer and reported that above reinforcement ratios of 3% the benefit of adding more FRP is not justified as the ultimate capacity levels off. Plevris and Triantafillou (1992) established that the critical FRP area fraction depends on three parameters only: i) the ratio of tensile strain to compressive yield strain; ii) the ratio of FRP ultimate strain to compressive yield strain; iii) the ratio of the elastic modulus of elasticity of FRP to that of wood. In their study, the reinforcement ratio was defined as the area of FRP reinforcement divided by the area of the wooden cross-section. While this is appropriate for the simple cases studied, it becomes ambiguous to talk about a reinforcement ratio when transverse FRP is added around the beam. Only few studies have investigated the

effect of wrapping wood beams using FRP (Sonti et al., 1996; Johns and Lacroix, 2000; Buell and Saadatmanesh, 2005), however no such investigation has been done on glulam and FRP. Johns and Lacroix (2000) investigated the effect of a unidirectional U-shaped GFRP wrap on 38 x 89 mm² studs, as shown in Figure 2.11a, whereas Buell and Saadatmanesh (2005) investigated the effect of partial and full bidirectional CFRP wraps with and without tension reinforcement on timber beams as shown in Figures 2.11b and 2.11c, respectively.

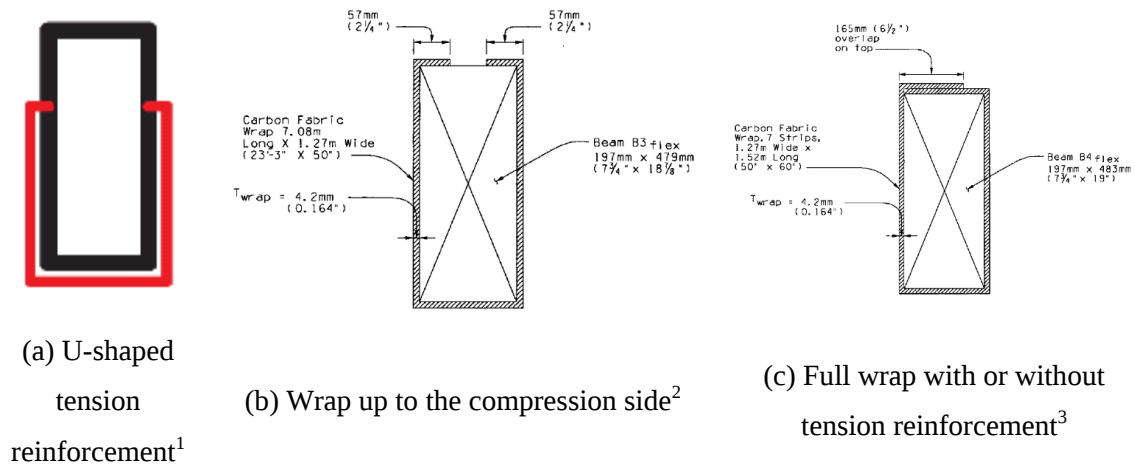


Figure 2.11: Previous FRP wrap configurations investigated

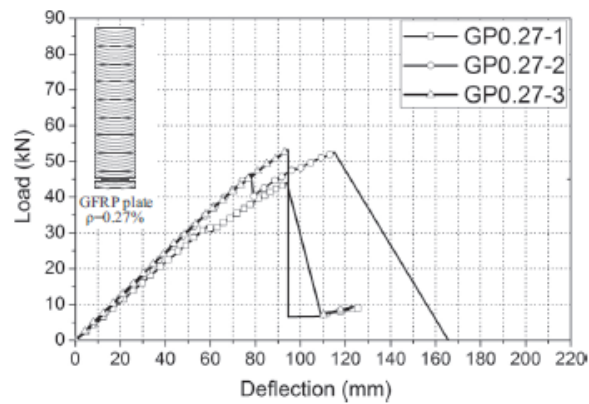
¹Reproduced from Johns and Lacroix (2000)

²Reproduced from Buell and Saadatmanesh (2005)

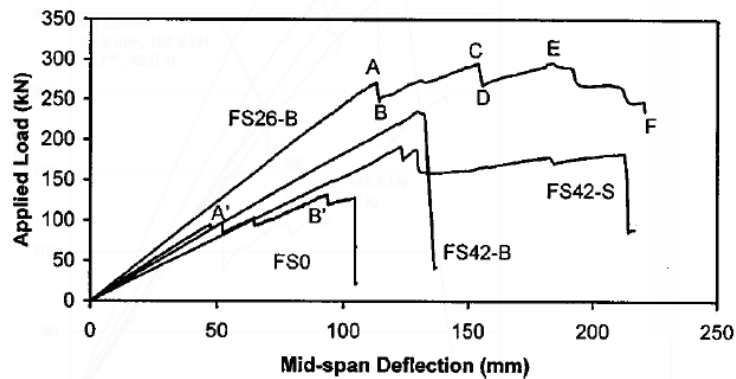
Buell and Saadatmanesh (2005) reported increases in resistance (40 - 53%) and stiffness (17 - 27%) for the various configurations investigated. The CFRP was found to provide confinement to the wood, thereby reducing the effects of defects and allowing the wood to approach the ultimate strength reach of members without defects. Sonti et al. (1996) investigated the effect of partial-length reinforcement and found that providing such confinement enhanced the strength and stiffness. Several studies have reported on the behaviour of wood beams reinforced with simple FRP tension laminates and it was found that no significant post-peak resistance was observed (Plevris and Triantafillou, 1992; Sonti et al., 1996; Buell and Saadatmanesh, 2005; Yang et al., 2016).

Gentile et al. (2002) investigated timber beams reinforced with GFRP bars and found an increase of 64 % in the average extreme fiber tensile strain of the reinforced beams compared to unreinforced ones. In order to account for the enhancement in bending strength of the

reinforced beams, a constant modification factor, α_m , was introduced. The modification factor was developed based on calibrating the model to the experimental data. The factor found by Gentile et al. (2002) was used in a study by Yang et al. (2016) to predict the strength of FRP and steel reinforced glulam beams. Figures 2.12a and 2.12b show examples of typical load-deflection curves achieved for simple tension reinforcement obtained using FRP and GFRP bars, respectively. Although post-peak resistance was attained in some of the tested configurations using GFRP bars, the appeal of investigating FRP fabric in blast is that the FRP confinement has the potential to reduce the debris by containing the failure of the specimen.



(a) FRP tension laminate¹



(b) GFRP bars²

Figure 2.12: Typical load-deflection curves for retrofitted wood beams

¹Reproduced from Yang et al. (2016)

²Reproduced from Gentile et al. (2002)

CHAPTER 3 - Experimental Program

3.1 General

An experimental program was developed to investigate the response of unreinforced and FRP reinforced glulam beams and columns subjected to simulated blast loading. The experimental program can be subdivided into three distinct phases investigating:

1. The dynamic increase factor (DIF);
2. Effect of axial load;
3. Effect of uni- and multi-directional FRP reinforcement;

The experimental program involves the testing of seventy glulam specimens, statically and dynamically, under four point bending. In addition to the static and dynamic full-scale testing, component tests were conducted to determine the properties of the wood and FRP matrices. The following sections describes the test specimens and experimental setups.

3.2 Description of the unretrofitted specimens

The glulam specimens investigated in this research program were obtained from two different manufacturers in which three different cross-sections were considered. The specimens' dimensions and grade consisted of 80 x 228 x 2,500 mm³ 20f-E Spruce-Pine (SP), 86 x 318 x 2,500 mm³ 24f-ES SP, and 137 x 222 x 2,500 mm³ 24f-ES SP. Two cross sections (i.e. 80 x 228 mm² and 86 x 318 mm²) were chosen to be more slender with depth to width aspect ratios exceeding 2.85:1 and thus representative of typical beams aspect ratios, while the third cross section (i.e. 137 x 222 mm²) was chosen to be stockier with a depth to width ratio of 1.6:1 to represent a column size. The nomenclature of the specimens is such that the letters (B or C) refer to the member type (beam or column) followed by the specimen number. The number in square brackets indicates the width of the narrow face of the cross-section. For example, B2-[86] refers to beam specimen number 2 with dimensions 86 x 318 x 2,500 mm³ and grade 24f-ES. As seen in Figure 3.1, the specimen's width either consisted of a single laminate (Fig. 3.1a) or multiple laminates (Figs. 3.1b and 3.1c). This means that for beams with single laminate, the finger joint was continuous across the member width

(Fig. 3.1d). In the case of multiple laminates, the joints could either be aligned across the member width (Fig. 3.1e) or be staggered (Fig. 3.1f), which as seen later will have a significant impact on the behaviour of the members.

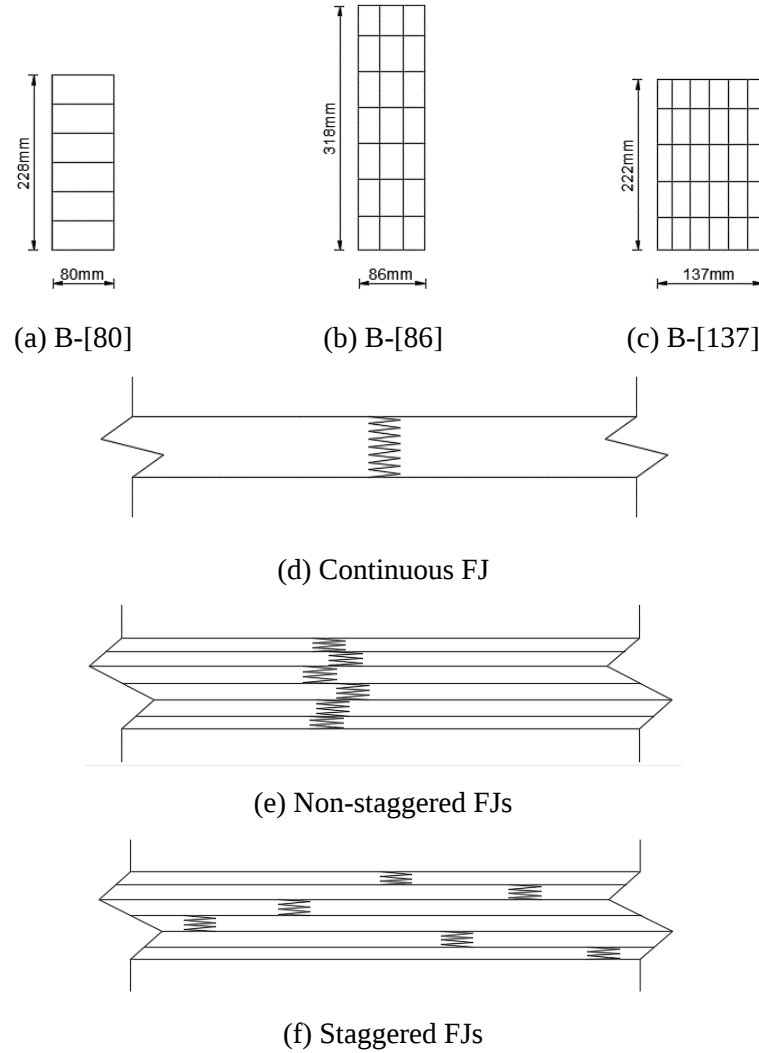


Figure 3.1: Representative static failure modes of generic glulam beams

The average density of members with single laminate (i.e. B-[80]) and multiple laminates (i.e. B-[86] and B-[137]) were determined to be 489 kg/m^3 and 551 kg/m^3 , with coefficients of variation of 0.02 and 0.03, respectively. The specimens were conditioned prior to testing to an average moisture content of 13 % at the time of testing. Throughout the thesis, the term “beam” is used to describe specimens that have no axial load while being subjected to out-of-plane loading, whereas specimens subjected to combined axial and out-of-plane loads are referred to as “column”.

3.3 Description of retrofitted specimens

A total of nine different FRP retrofit configurations, used to enhance the flexural behaviour of twenty-six undamaged glulam beams, were investigated. The testing of the retrofitted specimens can be divided into two separate phases. The first phase had the objective to investigate the effect of unidirectional fabric, including the effects of simple versus U-shaped tension reinforcement, partial- versus full-length confinement, and the type of FRP used. Furthermore, the first phase compared the static and dynamic behaviour of the retrofitted beams by testing half of the specimens statically and the remainder dynamically. The second phase, was developed following the completion of the first phase and had the objective to investigate the effect of multi-directional fabrics on the post-peak behaviour of beam elements. The following describes the retrofit layouts and the instrumentation of the specimens.

3.3.1 Retrofit configurations – Phase 1

A total of sixteen undamaged 137 x 222 x 2,500 mm³ 24F-ES SP glulam beams were tested to failure under static and dynamic loads. Four different retrofit configurations were considered each with four replicas of which half was tested statically and the remainder dynamically. Additionally, three previously tested specimens were restored by confining the damaged regions using GFRP, and retested dynamically.

Figure 3.2 shows a cross-section and side view of Retrofits 1 through 4 as well as the restored beams. Retrofit 1 (Fig 3.2a), which consists of two layers of GFRP tension reinforcement, was investigated as a baseline reference. Retrofit 2 (Fig. 3.2b) expands on Retrofit 1 by adding one layer of unidirectional GFRP in the perpendicular to wood fibre direction to investigate the effect of confinement. Investigated in Retrofit 3 is the effect of a tension U-shaped GFRP reinforcement with one layer of partial confinement. Retrofit 4 is identical to Retrofit 3, but using CFRP.

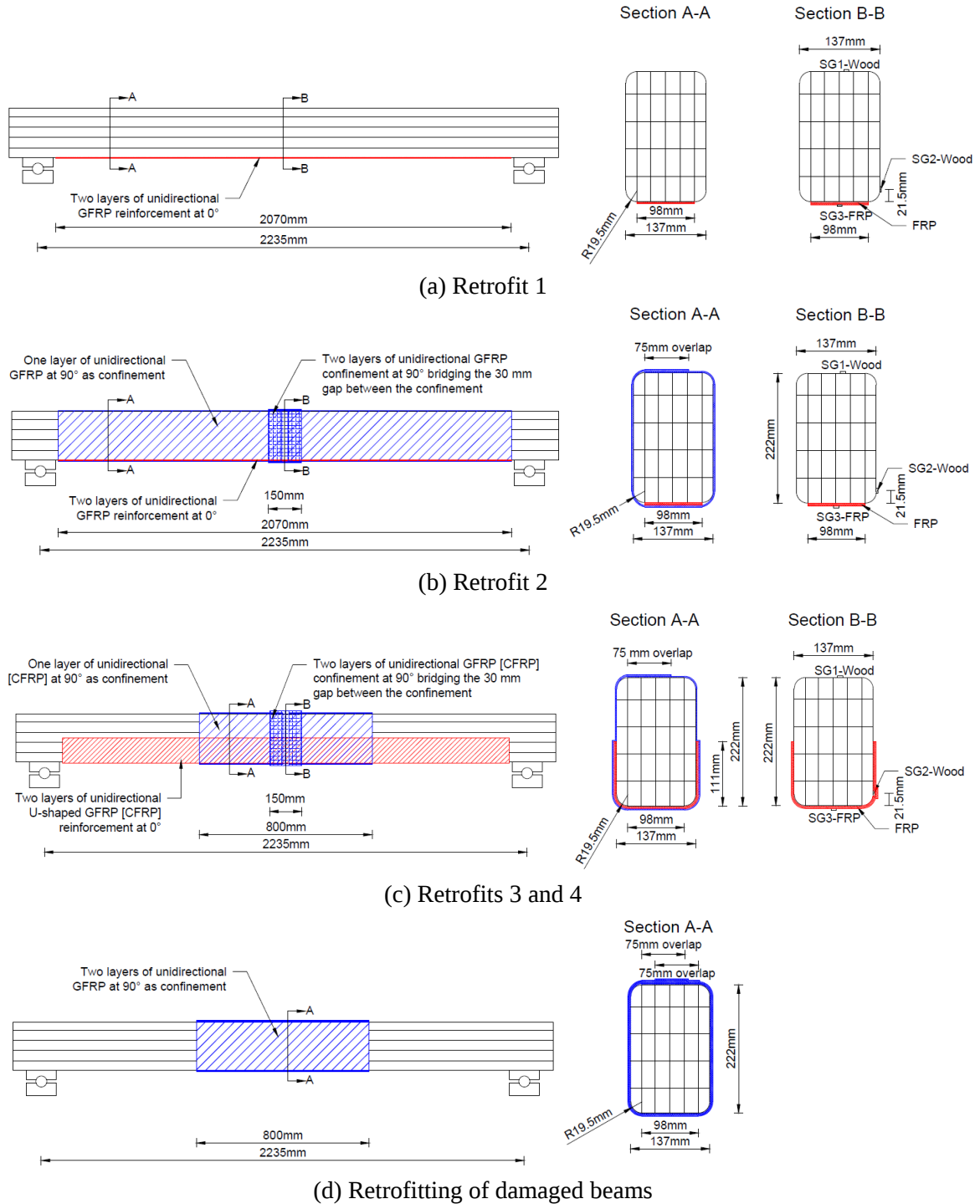


Figure 3.2: Description of Retrofits 1 through 4

Retrofits 3 and 4 are combined in Figure 3.2c since the only difference between them is the type of fabric. For the undamaged specimens, a 150 mm splice was placed at the member

mid-span (Figs. 3.2b and 3.2c) to bridge a 30 mm gap used for the placement of the strain gauges on the wood tension face. Each retrofitted specimen was instrumented with a total of three strain gauges at mid-span measuring the wood compressive strain, wood tensile strain, and FRP tensile strain as shown in Figure 3.2.

The retrofitting of the previously damaged beams consisted of applying two layers of GFRP confinement within the center-third span, where the damage was located. The damaged areas were smoothed and any loose or broken wood splinters were removed.

Unidirectional glass and carbon fabrics (Tyfo SEH-51A and Tyfo SCH-41) were used with a two-part epoxy (Tyfo S). Prior to the application of the FRP, the surfaces were lightly sandblasted in order to allow for better penetration of the epoxy and the beam corners were rounded to a 19.5 mm radius to properly apply the FRP confinement.

3.3.2 Retrofit – Phase 2

A total of ten undamaged specimens, each with a unique retrofit configuration, were tested dynamically to failure. Four 137 x 222 x 2,500 mm³ and six 86 x 318 x 2,500 mm³ 24f-ES SP glulam beams were retrofitted and tested. Three types of GFRP fabrics were included in this phase: unidirectional GFRP (CSS-CUGF27), 0/90 degree bidirectional GFRP (CSS-BGF018), and ± 45 degree bidirectional GFRP (CSS-CBGF424).

Retrofit 5A (Fig. 3.3a) consisted of two unidirectional GFRP U-shaped layers along with two layers of 0/90 bidirectional GFRP. Retrofit 5B (Fig. 3.3b) consisted of four layers of 0/90 bidirectional GFRP. The application of the FRP differed slightly from the previous phase where a splice was placed at mid-span. In this phase, the reinforcement was applied in the direction parallel to the length of the beam such that the reinforcement on the tension side was continuous. Due to the limitation in the fabric width, a piece of FRP fabric 150 mm larger than the specimen width was placed on the compression face to ensure continuity of the confinement at every second layer of applied confinement. This created an overlap length of 75 mm on each side.

Retrofit 6A was identical to Retrofit 5A with the exception of the confinement fabric type which consisted of ± 45 bidirectional GFRP (Fig. 3.3a). Similarly, Retrofit 6B (Fig. 3.3b) was identical to Retrofit 5B but had ± 45 degree bidirectional GFRP as confinement.

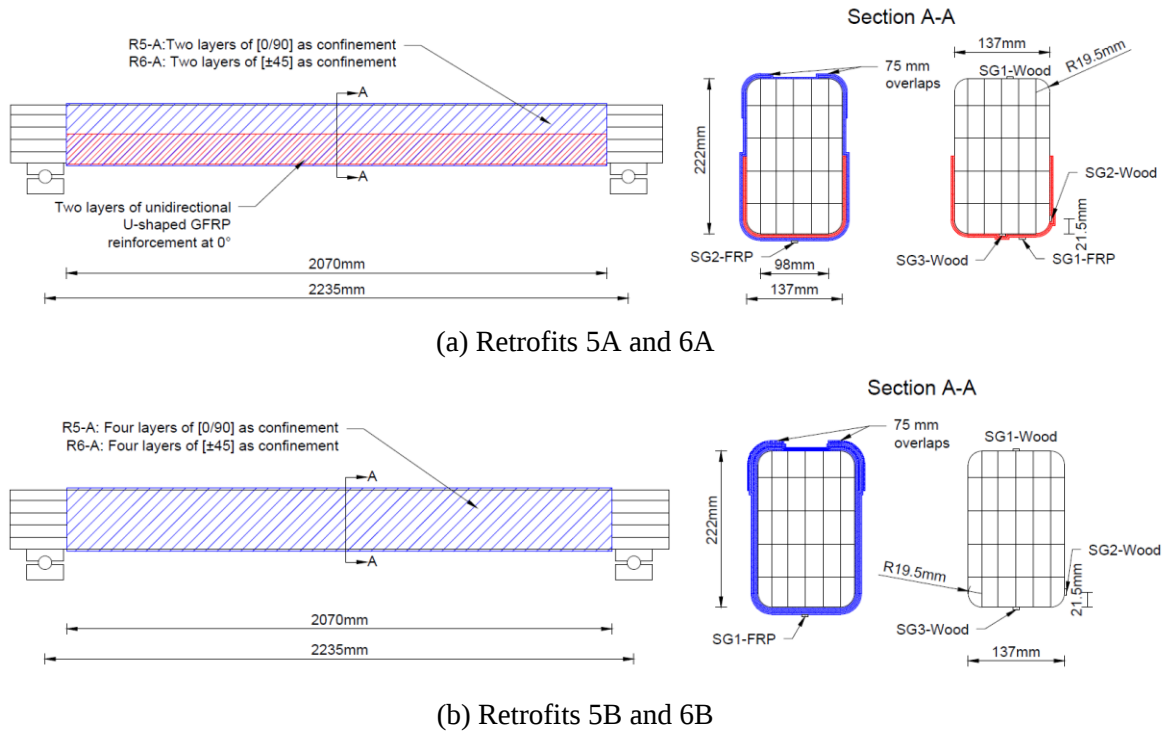
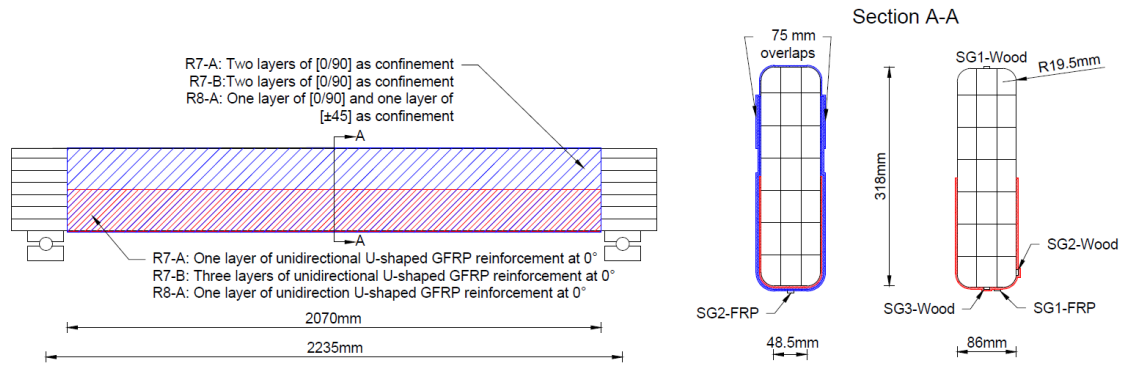


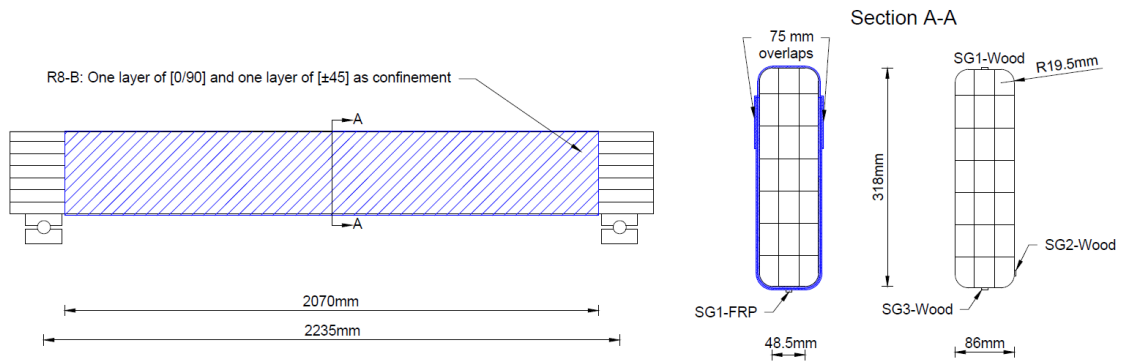
Figure 3.3: Description of Retrofits 5 and 6

Specimens of cross-section $86 \times 318 \text{ mm}^2$ were used for Retrofits 7 through 9. Retrofit 7A consisted of one U-shaped layer of unidirectional GFRP and two layers of 0/90 degree bidirectional GFRP confinement as shown in Figure 3.4a. Retrofit 7B has three U-shaped layers of unidirectional GFRP and two layers of 0/90 degree bidirectional GFRP confinement (Figure 3.4a).

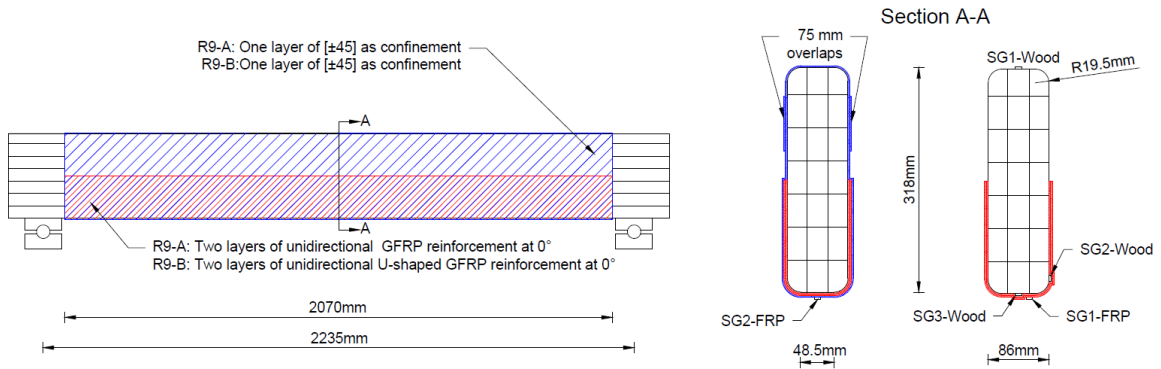
Retrofit 8A (Fig. 3.4b) had one U-shaped layer of unidirectional GFRP, one layer of 0/90 degree bidirectional GFRP confinement, and one layer of confinement consisted of ± 45 degree bidirectional GFRP. Retrofit 8B was identical to Retrofit 8A in terms of confinement but lacked the U-shaped unidirectional GFRP layer as tension reinforcement.



(a) Retrofits 7A, 7B and 8A



(b) Retrofit 8B



(c) Retrofits 9A and 9B

Figure 3.4: Description of Retrofits 7 through 9

Retrofit 9A (Fig. 3.4c) consists of two layers of unidirectional tension reinforcement with one layer of ± 45 degree bidirectional GFRP as confinement. Retrofit 9B (Fig. 3.4c) consists of two layers of unidirectional U-shaped tension reinforcement with one layer of ± 45 degree bidirectional GFRP as confinement.

The specimens were instrumented with strain gauges at mid-span to measure the wood tensile and compressive strain, strain in the unidirectional outer FRP layer if applicable, and finally the tension strain on the outer confinement.

3.4 Summary of test matrix

In order to provide an overview of the seventy glulam specimens tested, Table 3.1 summarizes the specimens' dimensions, FRP configurations (where applicable), applied axial load (where applicable), and loading type.

Table 3.1: Summary of test matrix

Specimen type	Loading type	Specimens	Axial load ratio	FRP retrofit configuration
Unretrofitted	Static	B1-B11; [80]	-	-
		B1-B4; [86]	-	-
		B1-B4; [137]	-	-
	Dynamic	B12-B21; [80]	-	-
		B5-B8; [86]	-	-
		B5-B9; [137]	-	-
		C1-C2; [137]	0.15	-
		C3-C4; [137]	0.40	-
		C5-C6; [137]	0.60	-
Retrofitted	Static	R1-1, R1-2; [137]	-	T [0] ₂
		R2-1, R2-2; [137]	-	T [0] ₂ C [90]
		R3-1, R3-2; [137]	-	U [0] ₂ C [90]
		R4-1, R4-2; [137]	-	U [0] ₂ C [90] (CFRP)
	Dynamic	R1-3, R1-4; [137]	-	T [0] ₂
		R2-3, R2-4; [137]	-	T [0] ₂ C [90]
		R3-3, R3-4; [137]	-	U [0] ₂ C [90]
		R4-3, R4-4; [137]	-	U [0] ₂ C [90] (CFRP)
		R5-A; [137]	-	U [0] ₂ C [0/90] ₂
		R5-B; [137]	-	C [0/90] ₄
		R6-A; [137]	-	U [0] ₂ C [± 45] ₂
		R6-B; [137]	-	C [± 45] ₄
		R7-A; [86]	-	U [0] C [± 45] ₂
		R7-B; [86]	-	U [0] ₃ C [± 45] ₂
		R8-A; [86]	-	U [0] C [0/90, ± 45]
		R8-B; [86]	-	C [0/90, ± 45]
		R9-A; [86]	-	T [0] ₂ C [± 45]
		R9-B; [86]	-	U [0] ₂ C [± 45]

For the retrofitted specimens, the stacking sequence is presented in terms of simple tension reinforcement (T) or U-shaped tension reinforcement (U) and confinement (C). Found in the square brackets is the fibre orientation of the fabric used while the subscript denotes the number of layers applied for multiple layers.

3.5 Test setups

3.5.1 Components

Material properties defining the wood uniaxial stress-strain relationship, to be used in the moment-curvature analysis, were obtained by conducting tests on coupons as shown in Figure 3.5. A total of twenty-four coupon tests were conducted, half of which were in tension and the other half in compression. The tests were done in accordance with ASTM D143 “Standard Test Methods for Small Clear Specimens of Timber” (ASTM, 2014a). Half of the coupons were obtained from an undamaged 86 x 318 x 2,500 mm³ specimen while the remainder were obtained from an undamaged 137 x 222 x 2,500 mm³. The tension specimens consisted of one lamination from within the cross-section that was further reduced to the dimensions required by the standard (ASTM, 2014a).

A Universal Testing Machine (UTM) recorded the applied force as a function of time which was then converted to stress by using the loaded area of the test coupon. For the tension tests, an extensometer with a gauge of 50 mm was used to measure the elongation of the specimen at mid-height (Fig. 3.5a). Similarly, a linear variable displacement transducer (LVDT) was used to measure the change in length of the compression coupon using the original specimen length as reference (Fig. 3.5b).

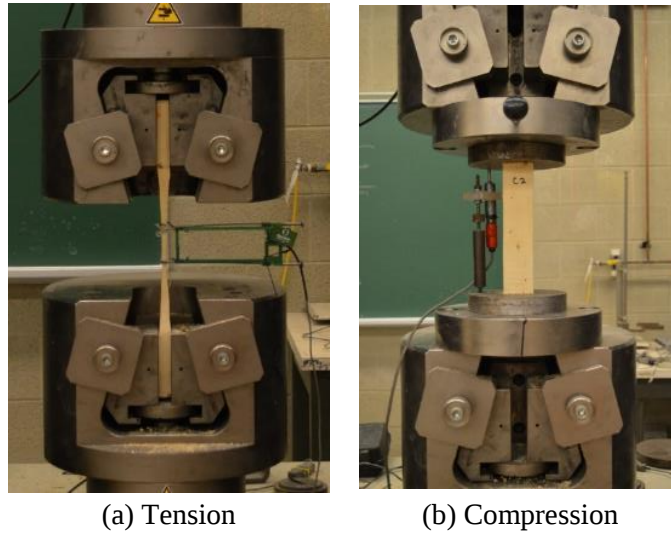


Figure 3.5: Wood coupon static tests

Material properties defining the FRP stress-strain relationship were obtained by conducting tests on coupons as shown in Figure 3.6. Five FRP coupons per lay-up combination were tested. The tests were conducted in accordance with ASTM D3039 “Standard Test Method for Tensile Properties of Polymer Matrix Composite Materials” (ASTM, 2014b). The coupons were held between aluminum tabs to avoid stress concentration at the gripping points when applying the tension force.

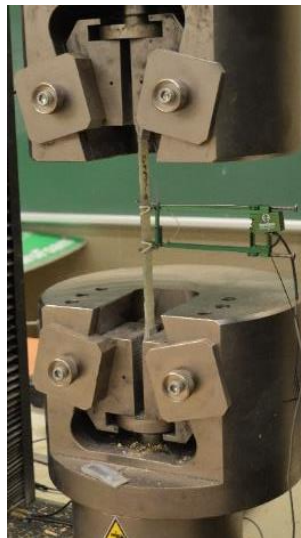


Figure 3.6: FRP coupon static tests

3.5.2 Full-scale static beam tests

A total of nineteen unretrofitted and eight retrofitted glulam members were tested statically under four point bending with simply supported boundary conditions as shown in Figure 3.7. The span-to-depth ratio was slightly less than that recommended by the ASTM D198 “Standard Test Method of Static Tests of Lumber in Structural Sizes” (ASTM, 2015) due to the size limitation imposed by the Shock Tube apparatus. For the static tests, a reaction frame with a steel I-beam was used to transfer the applied load at the third points of the span, as shown in Figure 3.7a.

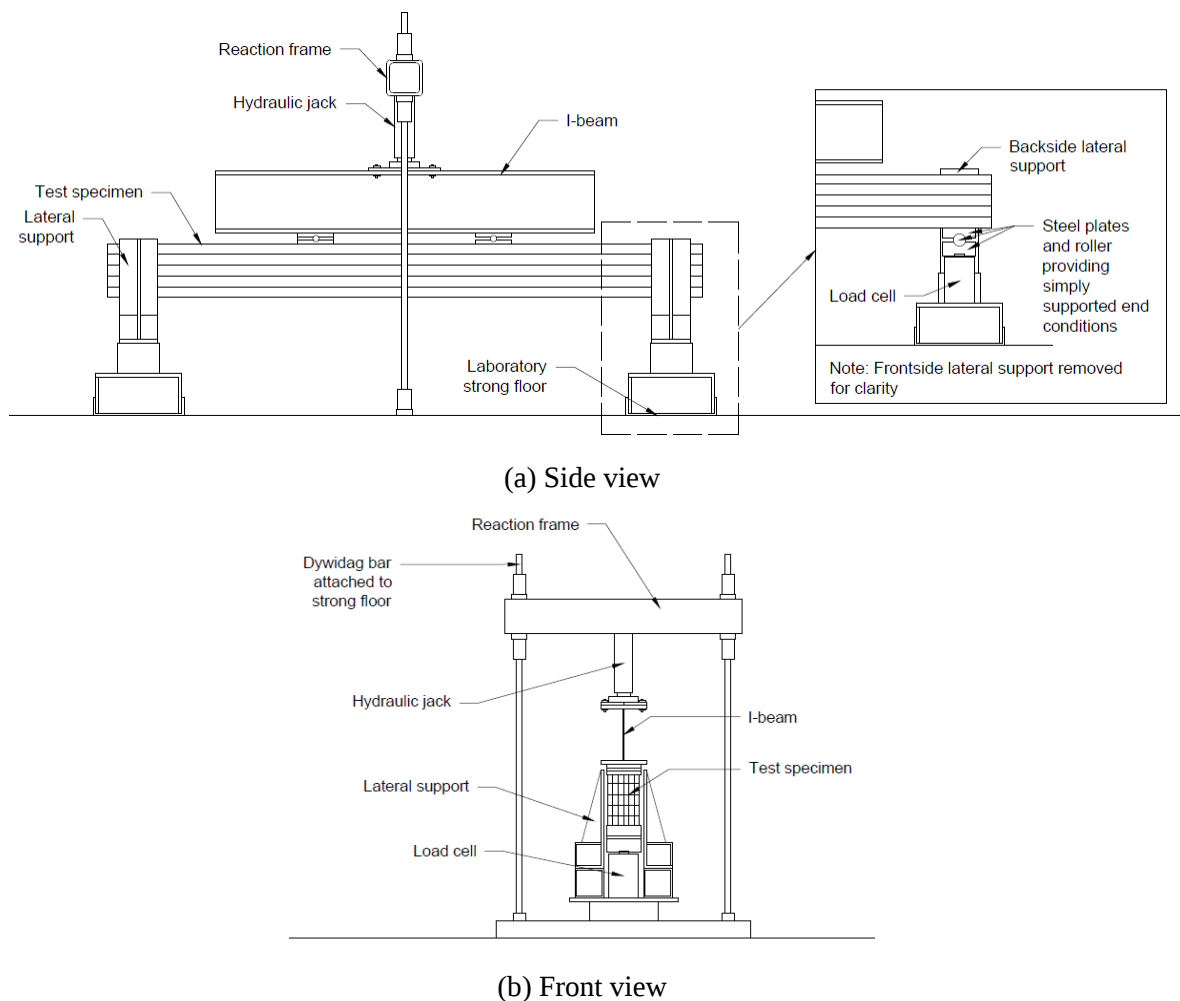


Figure 3.7: Static test setup

Load cells located at the member support points were used to measure the reactions. Idealized pin end conditions were provided by two steel plates and a roller on top of the load cells.

The specimens were tested to failure under static load using a combination of a hydraulic jack and a pump which increased the applied load at a constant rate until failure was obtained. During the static tests, a data acquisition system (DAS) was used to collect the data at a sampling rate of 10 samples per second. The mid-span displacement was measured using a wire-gauge. Strain gauges were used to record the strain-time history while the reactions were measured through the use of load cells at the member support points.

3.5.3 Full-scale dynamic beams and columns tests

High strain-rates similar to those experienced during blast loading were generated using the University of Ottawa's Shock Tube Facility. The Shock Tube, as shown in Figure 3.8, is a test apparatus capable of producing a uniform pressure over the reflecting surface without the use of live explosives, thereby simulating a wide range of pressures and impulses. The pneumatically driven Shock Tube can achieve maximum reflected pressure and reflected impulses of 100 kPa and 2,200 kPa-ms, respectively, and a range of time durations for the positive phase of 5 to 70 ms. A shock front, generated from the release of the compressed air in the driver section (Fig. 3.8a), travels along the 6,096 mm long expansion section (Fig. 3.8b) before interacting with the specimen or reflecting surface mounted to the end frame. The size of the opening at the end frame is 2,032 x 2,032 mm², which allows the testing of large or full-scale specimens (Fig. 3.8c).



(a) Driver section

(b) Expansion section

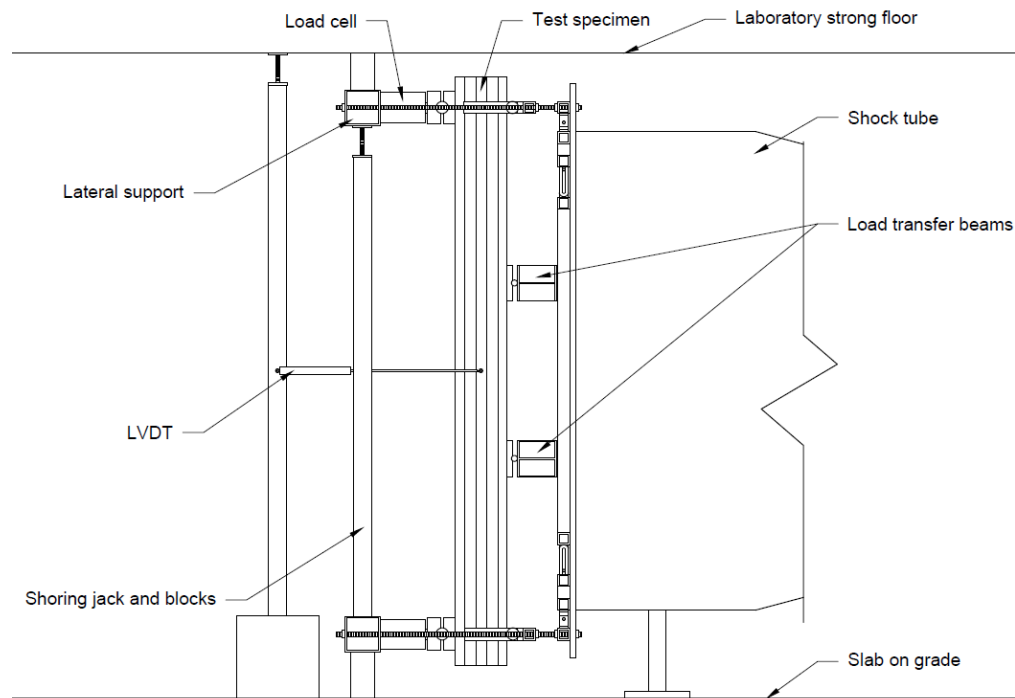
(c) End frame

Figure 3.8: University of Ottawa's Shock Tube

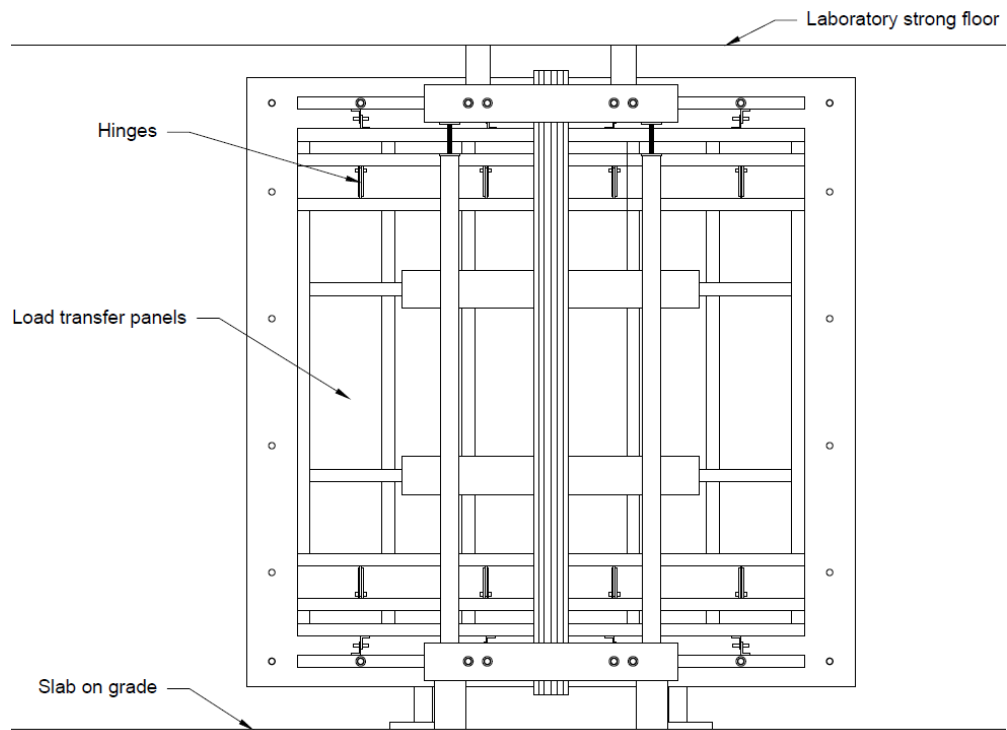
The maximum peak reflected pressure is controlled by the driver pressure while the reflected impulse, and hence the time duration of the positive phase, is controlled by varying the driver

length. The length of the driver section can be varied in 305 mm increments from 305 mm to 5,185 mm.

The dynamic setup for beams, as shown in Figure 3.9, used the same loading pattern, boundary conditions, and span as in the static tests. A load transfer device (LTD) was used to collect the pressure generated by the Shock Tube and transferred it to the third points of the test element (Fig. 3.9).



(a) Side view



(b) Front view

Figure 3.9: Dynamic test setup for beams

The LTD consisted of rigid steel panels covering the entire opening of the end frame and was capable of moving freely in the lateral direction up to deflections of 200 mm through the use of slotted hinges. Using the LTD provided an efficient mean of capturing the pressure generated in the Shock Tube and transferring it to the specimens as two point loads. The LTD has been successfully used to test dimensional lumber and reinforced concrete beams (e.g. Jacques, 2016).

For the columns, the axial load was applied using two hydraulic jacks located between the floor slab and the bottom face of the specimen as shown in Figure 3.10. Two load cells were mounted at the top end of the specimen to measure the applied axial load-time history. To allow free rotation when the column displaced laterally, two notched steel plates and a roller, similar to that provided at the member support points, were installed at the top and bottom of the member.

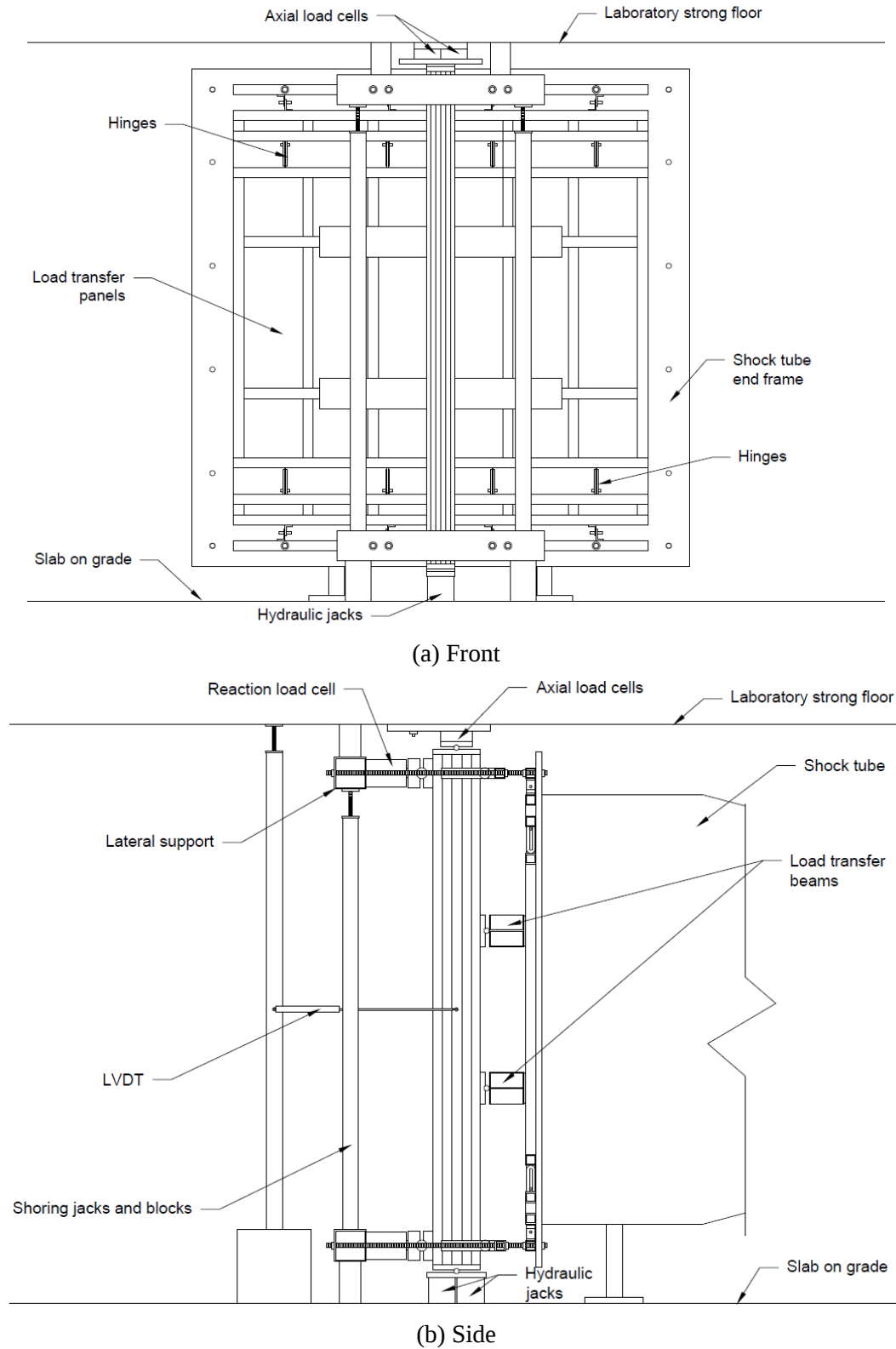


Figure 3.10: Dynamic experimental setup for columns

The axial load was applied prior to the tightening of the lateral supports, thereby allowing the system to settle without adding any unwanted forces. Six B-[137] specimens were used to investigate three different axial load ratios (ALR) corresponding to approximately 0.15, 0.45 and 0.75 of the column axial design capacity. For each ALR, two columns were tested in order to ensure replicability of the results. These two columns, tested with the same ALR, are also referred to as a set. Each column was subjected to a single shot causing ultimate failure with axial load.

The dynamic tests used a DAS with a sampling rate of 100,000 samples per second and recorded the mid-span deflection, strain at mid-span on the wood and FRP, axial load, and dynamic reactions. Additionally, the reflected pressure-time history of the shock wave was recorded using two dynamic piezoelectric sensors (i.e. pressure gauges) located at the bottom and side walls near the end frame, while the failure was captured through the use of a high-speed camera at a rate of 500 frames per second.

CHAPTER 4 - Experimental Results

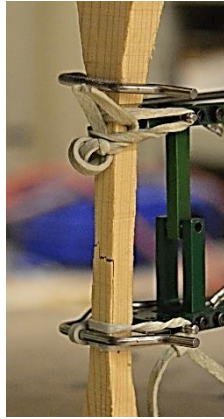
4.1 General

The experimental results from the static and dynamic tests are presented in this section. This includes the properties of the wood and FRP coupons as well as full-scale tests conducted on unretrofitted beams and columns and retrofitted beams.

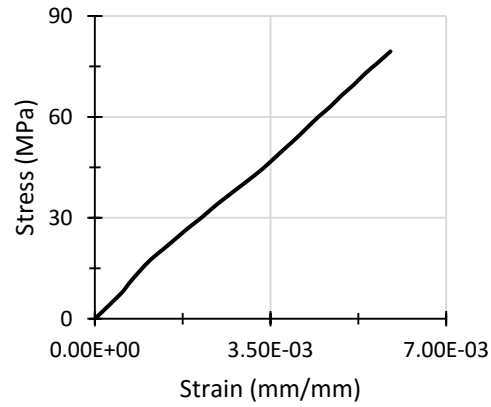
4.2 Component test results

4.2.1 Wood coupons

A total of twenty-four tension and compression coupon tests were conducted. Figure 4.1 shows representative failure modes and corresponding stress-strain curves obtained experimentally for both the tension and compression coupon tests. All tension specimens failed in the reduced cross-section area (Fig. 4.1a). The compression specimens showed characteristic wrinkling failure on the surface of the specimens (Fig. 4.1c). From Figure 4.1b, it can be seen that in tension the wood behaved linearly, however, in compression a non-linear behaviour is observed once the fibres start crushing (Fig. 4.1d). The compressive stress-strain relationship can be idealized with a bi-linear curve for the purpose of modelling as will further be discussed in Chapter 5.



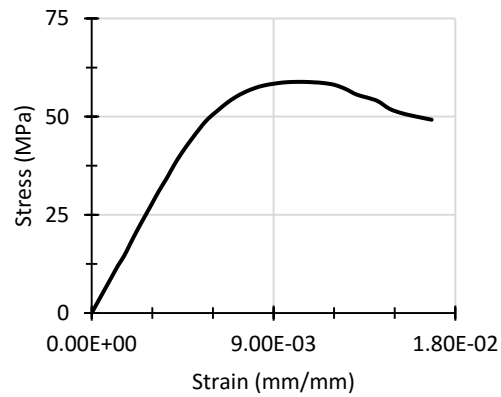
(a) Tension failure



(b) Tension stress-strain response



(c) Compression failure



(d) Compression stress-strain response

Figure 4.1: Coupon static test results

The average properties defining the tension and compression stress-strain relationships are presented in Table 4.1. The stress-strain curves for all wood coupons used to obtain the average values presented in Table 4.1 can be found in Appendix A.

Table 4.1: Static tension and compression coupon test results

Cross-section		Tension			Compression		
		E_{wt}^a (MPa)	$f_{t,exp}^b$ (MPa)	$\epsilon_t^c \times 10^4$ (mm/mm)	E_{wc}^d (MPa)	$f_{c,exp}^e$ (MPa)	$\epsilon_{cu}^g \times 10^4$ (mm/mm)
B-[86]	Average	8,289	63.2	83.8	7,752	40.6	146.9
	Std. Dev	883	11	11.0	952	2.5	38.24
	CV	0.11	0.18	0.13	0.12	0.06	0.26
B-[137]	Average	13,290	79.5	66.3	7,102	48.6	156.0
	Std. Dev	2,916	21.2	27.4	1,482	6.0	32.4
	CV	0.22	0.27	0.41	0.21	0.12	0.21

^a Tension modulus of elasticity

^b Ultimate tensile strength

^c Tensile failure strain

^d Compression modulus of elasticity

^e Compressive yield strength

^f Ultimate compressive strength

^g Ultimate compressive strain

4.2.2 FRP coupons

Material properties of the FRP lay-ups investigated were obtained by testing five specimens for each configuration. Figure 4.2 shows representative failure modes for the different layups tested.

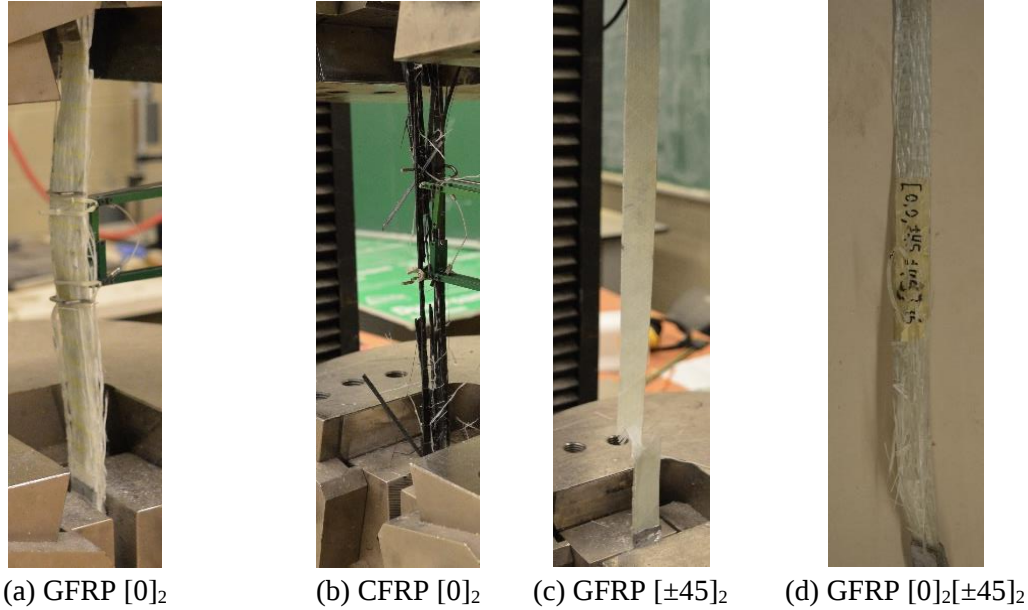


Figure 4.2: FRP coupon test results representative failure modes

Figure 4.3 shows representative stress-strain curves obtained experimentally for the lay-ups shown in Figure 4.2. It can be seen that the behaviour of the various layups is significantly different. In particular, it can be observed that when there is misalignment between the direction of the load and that of the fabric (i.e. ± 45), the behaviour of the FRP is non-linear.

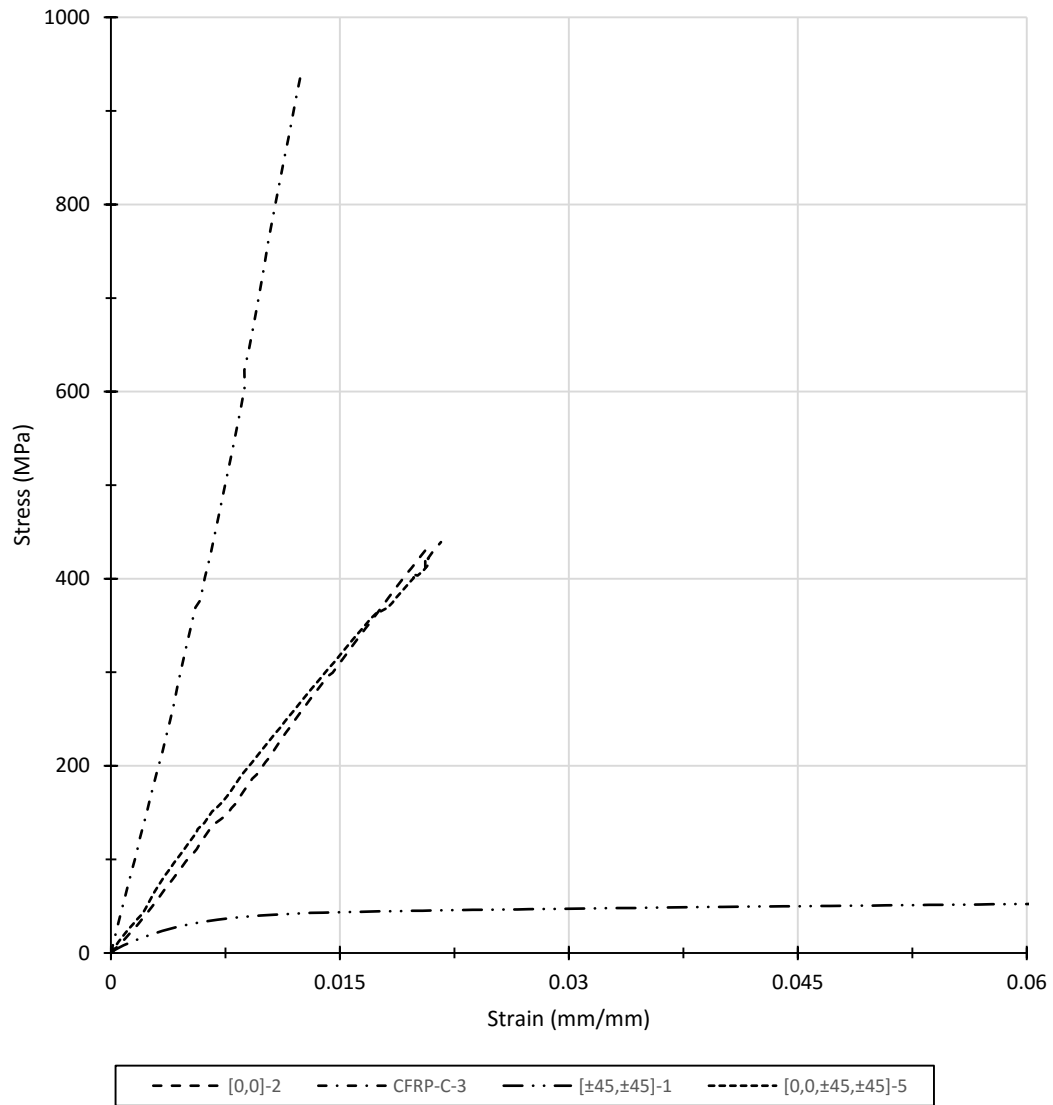


Figure 4.3: Representative FRP coupon stress-strain relationship

The average value, standard deviation, and coefficient of variation for the modulus of elasticity (E_{FRP}), ultimate strength (f_{FRP}), and failure strain ($\epsilon_{f,FRP}$) for each lay-up is presented in Table 4.2. More details on the test results for all FRP coupons are provided in Appendix B.

Table 4.2: Static tension GFRP and FRP coupon test results

Lay-up		E_{FRP} (MPa)	f_{FRP} (MPa)	$\epsilon_t \times 10^{-4}$ (mm/mm)	Lay-up		E_{FRP} (MPa)	f_{FRP} (MPa)	$\epsilon_t \times 10^{-4}$ (mm/mm)
[0] ₂ ^a	Average	21,701	448.3	211.6	[±45] ₄	Average	7,919	46.4	288.0
	Std. Dev	2,304	74	20.7		Std. Dev	1,228	2.9	82.7
	CV	0.11	0.17	0.10		CV	0.16	0.06	0.29
[0] ₂ ^b	Average	85,297	904.3	112.9	[0][0/90] ₂	Average	18,203	368.4	214.1
	Std. Dev	12,787	51.0	21.9		Std. Dev	3,408	15.9	38.0
	CV	0.15	0.06	0.19		CV	0.19	0.04	0.18
[0] ₂	Average	24,423	462.8	199.3	[0] ₃ [0/90] ₂	Average	22,974	516.8	242.7
	Std. Dev	5,876	100.3	34.4		Std. Dev	4,490	11.3	46.8
	CV	0.24	0.22	0.17		CV	0.20	0.02	0.19
[±45] ₂	Average	6,902	60.5	615.5	[0] ₂ [±45] ₂	Average	19,065	412.9	233.1
	Std. Dev	376	12.1	240.0		Std. Dev	3,182	20.9	39.3
	CV	0.05	0.20	0.17		CV	0.17	0.05	0.17
[0/90] ₂	Average	11,850	254.3	233.5	[0,0/90,±45]	Average	16,554	286.1	180.1
	Std. Dev	721	9.6	22.9		Std. Dev	1,299	20.5	22.9
	CV	0.06	0.04	0.10		CV	0.08	0.07	0.13
[0/90,±45]	Average	6,104	163.8	323.9					
	Std. Dev	1,881	8.7	90.4					
	CV	0.31	0.05	0.28					

^aUnidirectional GFRP used in Retrofits 1 through 3^bUnidirectional CFRP used Retrofit 4

4.3 Characterization of static and dynamic failure modes

4.3.1 General terminology

The following sections document the behaviour of the full-scale test specimens and describe their failure modes. Due to the relatively complex behaviour observed, this section attempts to clarify the terminology used. A drop in the flexural resistance is observed once the peak resistance is reached, and although a plateau is maintained beyond peak resistance, when debonding of the FRP occurs, or when a significant drop in resistance is attained, the member is assumed not to possess any resistance of significance. To illustrate this, an idealized resistance curve representative of typical observed behaviour is presented in Figure 4.4. As seen in the figure, the maximum (or peak) resistance relates to the maximum load resisted by the specimen (R_{max}). The deflection associated with the maximum resistance is denoted Δ_{Rmax} . A drop in resistance is observed once maximum value is reached. Currently, there is no established methodology or consensus on what constitutes an acceptable level of post-peak resistance for the purpose of blast design. The threshold at which the post-peak resistance was deemed no longer “significant” was selected by the author to be 50 % of maximum resistance. The ultimate (or failure) resistance of the specimen (R_f) corresponds to

the point at which the resistance is below 50 % of peak, or where de-bonding of the FRP occurred. The selected criterion is believed by the author to be a suitable threshold at which residual resistance can still significantly contribute to dissipating the energy and which could help prevent progressive collapse. The deflection associated with the ultimate resistance is denoted ultimate (or failure) deflection Δ_{Rf} . Defining a failure criterion at which the flexural resistance is no longer significant means that in some cases, especially in the dynamic tests, the maximum recorded displacement may be greater than that at ultimate deflection (shown by the dashed line in Fig. 4.4). Therefore, the term Δ_{max} is introduced and corresponds to the maximum displacement recorded. For the dynamic tests where the specimen remained in the elastic range, Δ_{max} represents the maximum recorded displacement, which obviously is of smaller magnitude than that of the ultimate deflection for the specimen considered. Defining the variables at this point is also important to understanding subsequent sections that include result tables as well as modelling inputs. It is important to emphasize that the majority of the failure pictures presented in this thesis are taken at a point exceeding that representing the maximum or even ultimate resistance as defined in this section.

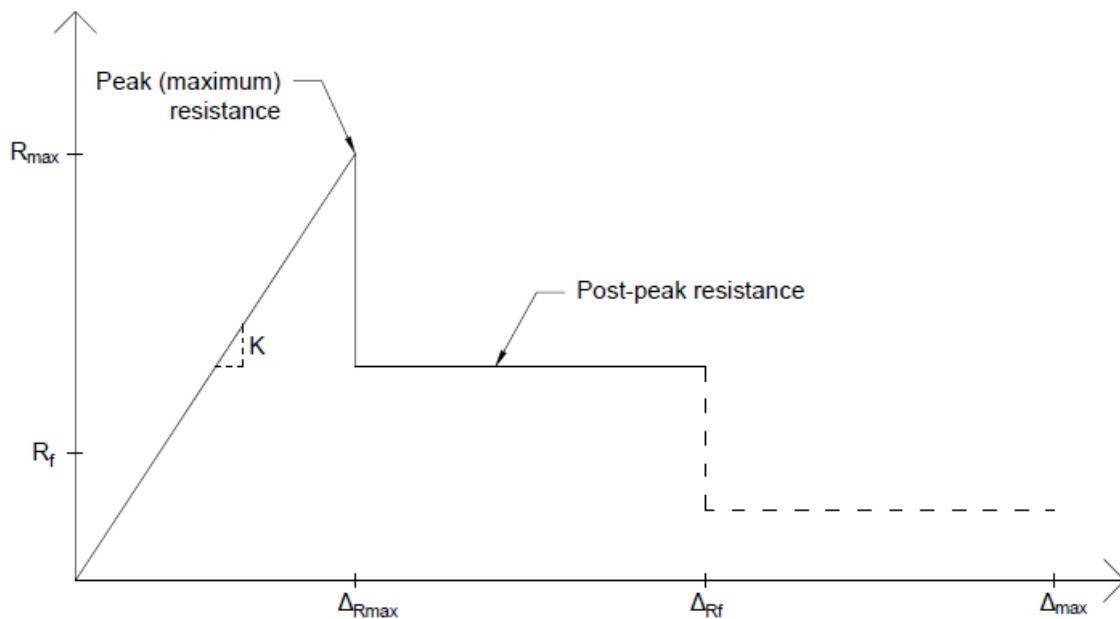


Figure 4.4: Terminology used for describing resistance curves

4.3.2 Unretrofitted beams

During the static testing, the failure modes were limited to simple or splintering tension failure as shown in Figure 4.5. Figure 4.5a shows the failure of the outer tension laminate followed by cracks propagating along the glulam layers. Failures either occurred at a knot, as seen in Figure 4.5b, or at a FJ, as seen in Figure 4.5c. For beams with multiple FJs that were not aligned, the failure appears staggered (Fig. 4.5d) in comparison to the straight cut across the member width for beams with single laminate or continuous FJ across the beam width (Figs. 4.5b and 4.5c).



Figure 4.5: Representative static failure modes of glulam beams

Contrary to the simple or splintering tension failure documented in the static tests, a brash tension failure was observed under dynamic loading for beams with single laminate (Fig. 4.6). Specimens that had the presence of a FJ on the outer tension laminate within the two

load application points consistently had the failure initiated at the FJ location, as shown in Figures 4.6b and 4.6c.

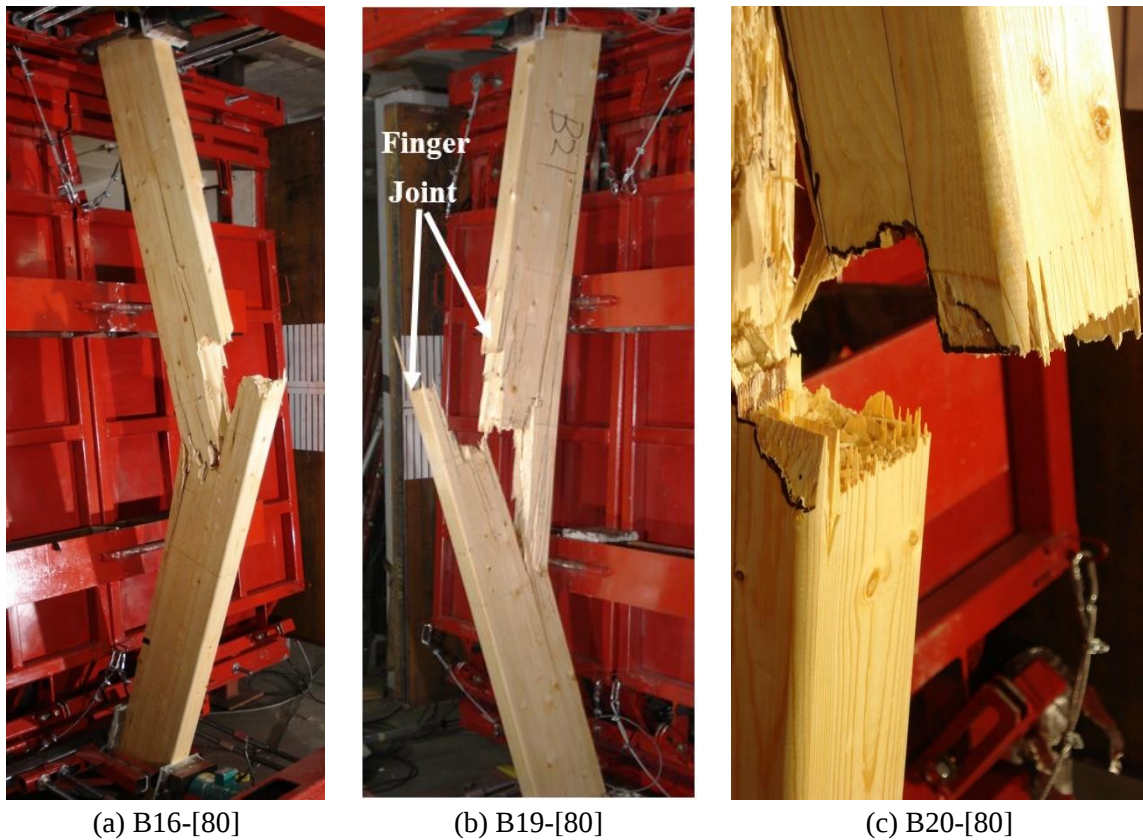


Figure 4.6: Representative dynamic failure modes of beams with single laminate

Beams with multiple laminates across the member width generally experienced significantly more crack propagation and splintering (Fig. 4.7). FJs aligned across the width of the beams had a significant effect on the failure mode and, as described in Chapter 6, on the DIF associated with such failure. An example of a failure where FJs were aligned across the width of the member can be seen in Figure 4.7a. Beams with staggered FJs (Figure 4.7b) followed a failure path that was similar to that observed under static loading (Fig. 4.5d). More details on the unretrofitted beams' behaviour can be found in Appendix C.



(a) B8-[86]



(b) B7-[137]

Figure 4.7: Representative dynamic failure modes of beams with multiple laminates

4.3.3 Columns

For column specimens, the damage through the depth of the section was observed to be focused near the member mid-span. A comparison between typical dynamic failure modes for beams and columns is shown in Figure 4.8. Figure 4.8a presents a typical beam failure, whereas Figure 4.8b is representative of the observed failure mode for specimens with axial load. Emphasized in Figure 4.8c (enlarged mid-span region of Figure 4.8b) is the compression failure observed in the column specimens as demonstrated by the “wrinkling” of the fibres. More details on the columns’ behaviour can be found in Appendix D.



(a) B5.2-[137]



(b) C2.3-[137]



(c) Close-up of mid-span damage for C2.3-[137]

Figure 4.8: Comparison of static and dynamic failure modes of beams and columns

4.3.4 Retrofitted beams

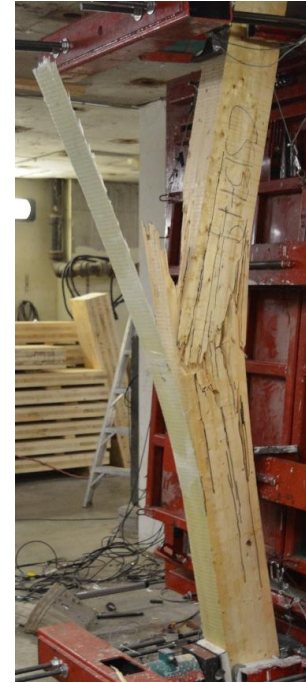
Both the static and dynamic failure modes of Retrofit 1 were characterized by failure in the outer wood tension laminates with damage propagating through the entire depth of the cross-section (Fig. 4.9). Following the wood failure, the outer tension laminates pushed outwards on the GFRP causing separation between the wood and reinforcement. The failure of the wood was dominated by simple or splintering tension failure that was influenced by the presence of knots or finger joints (FJ).



(a) Static – R1-1



(b) Static – R1-2



(c) Dynamic - R1-4

Figure 4.9: Representative static and dynamic failure modes for Retrofit 1

Representative failure modes for Retrofit 2 are shown in Figure 4.10. No separation was observed between the wood tension face and the GFRP laminates (Fig. 4.10a), which can be attributed to the confinement. The addition of confinement also led to a brash tension failure resulting in a straight cut through the beam width (Fig. 4.10b). For both the static and dynamic failures, the damage to the wood specimens was limited to a small region and involved significant compression failure (Fig. 4.10c). Tearing in the FRP confinement perpendicular to the fibre direction was observed after peak resistance as anticipated with unidirectional fabric used at ninety degrees to the wood fibres (Figs. 4.10a, 4.10b, and 4.10d).

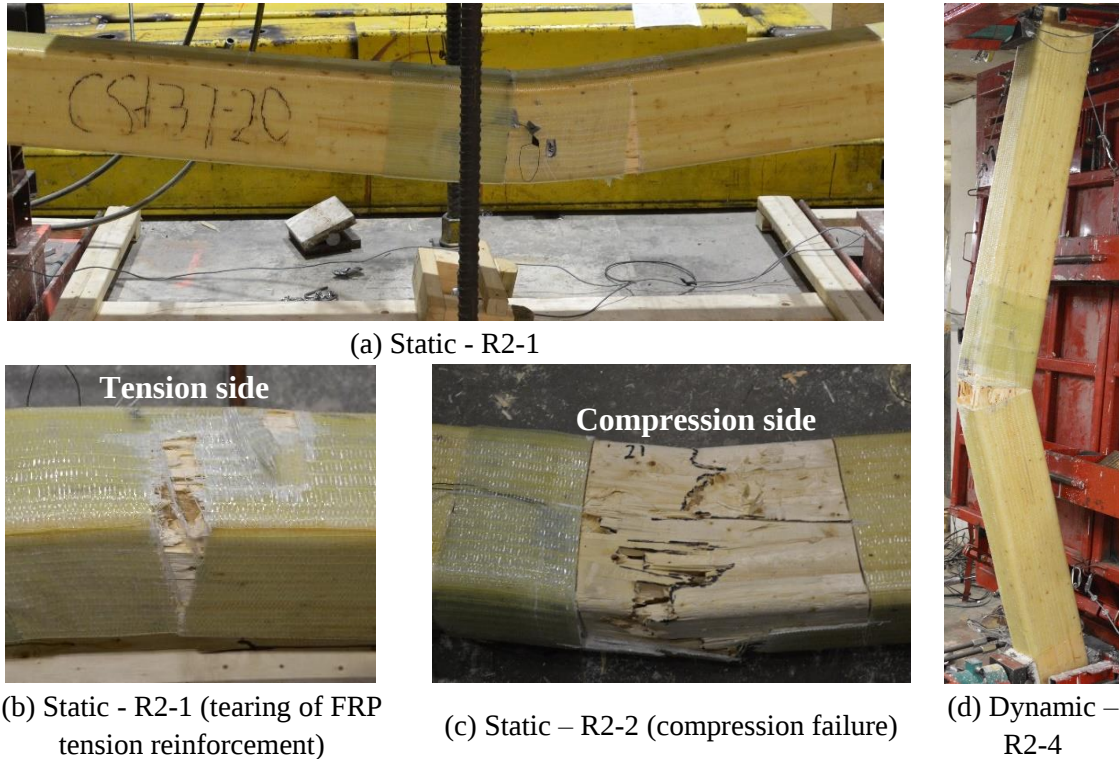


Figure 4.10: Representative static and dynamic failure modes for Retrofit 2

Figure 4.11 shows representative static and dynamic failures for both Retrofits 3 and 4. No noticeable differences between the static and dynamic failure modes were observed. While some separation of the FRP tension reinforcement was observed (Figs. 4.11a, 4.11b, and 4.11c), it occurred at a deflection level greater than that corresponding to the ultimate deflection. An example is shown in Figure 4.11d for specimen R3-3-[137], where the beam deflected 1.6 times more than the deflection corresponding to maximum resistance without de-bonding of the FRP. This indicates that using partial-length confinement can be effective in maintaining the integrity of the tension FRP laminates and preventing premature de-bonding.

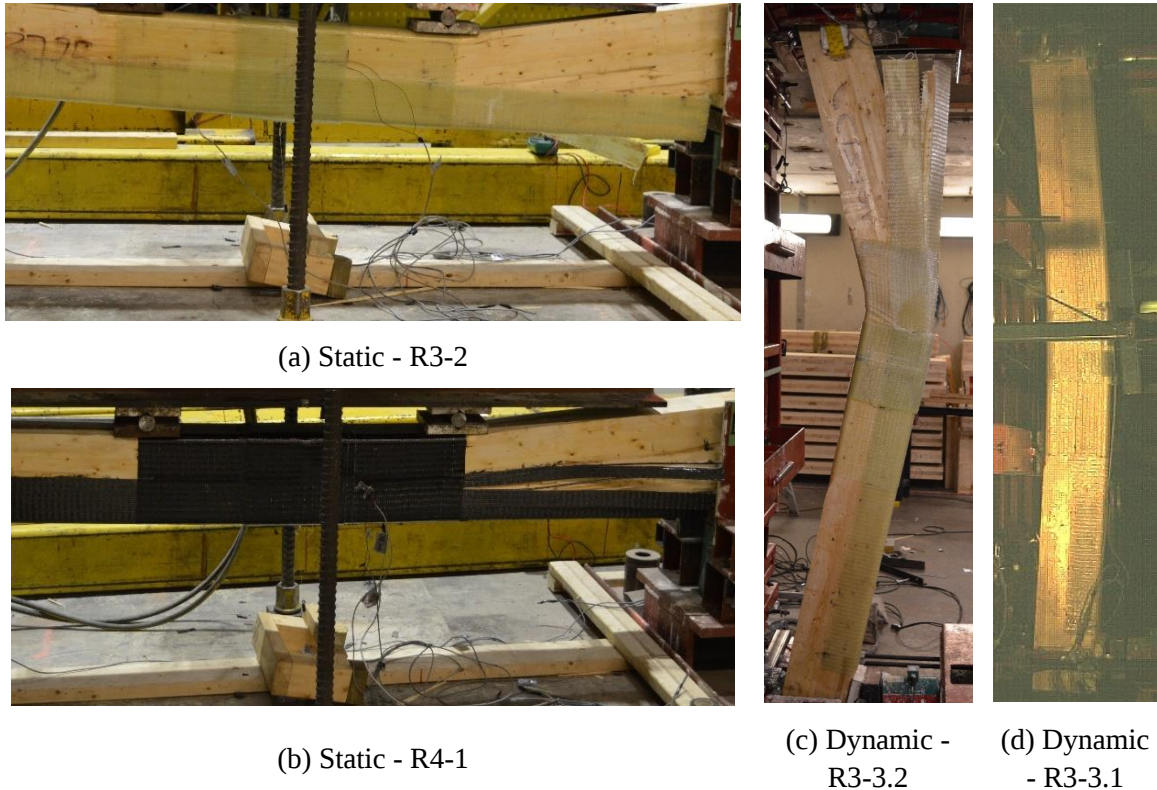


Figure 4.11: Static and dynamic representative failure modes for Retrofits 3 and 4

Figure 4.12 shows the observed failure modes for specimens R5-A and R5B. Figure 4.12a shows specimen R5-A at ultimate resistance with a displacement of 4.35 times that corresponding to the deflection at maximum resistance. The final deflected shape of the specimen is shown in Figure 4.12b where the de-bonding occurred past the ultimate resistance. Specimen R5-B is shown in Figure 4.12c at a displacement corresponding to 4.15 times that recorded at maximum resistance without any de-bonding. While specimen R5-B did not reach ultimate failure and recovered part of its displacement as shown in in Figure 4.12d, local “buckling” of the FRP around the mid-span region was observed. For both specimens (R5-A and R5-B), the reinforcement was able to limit the failure to a small region (Figs. 4.12e and 4.12f) and induce compression failure (Fig. 4.12f).

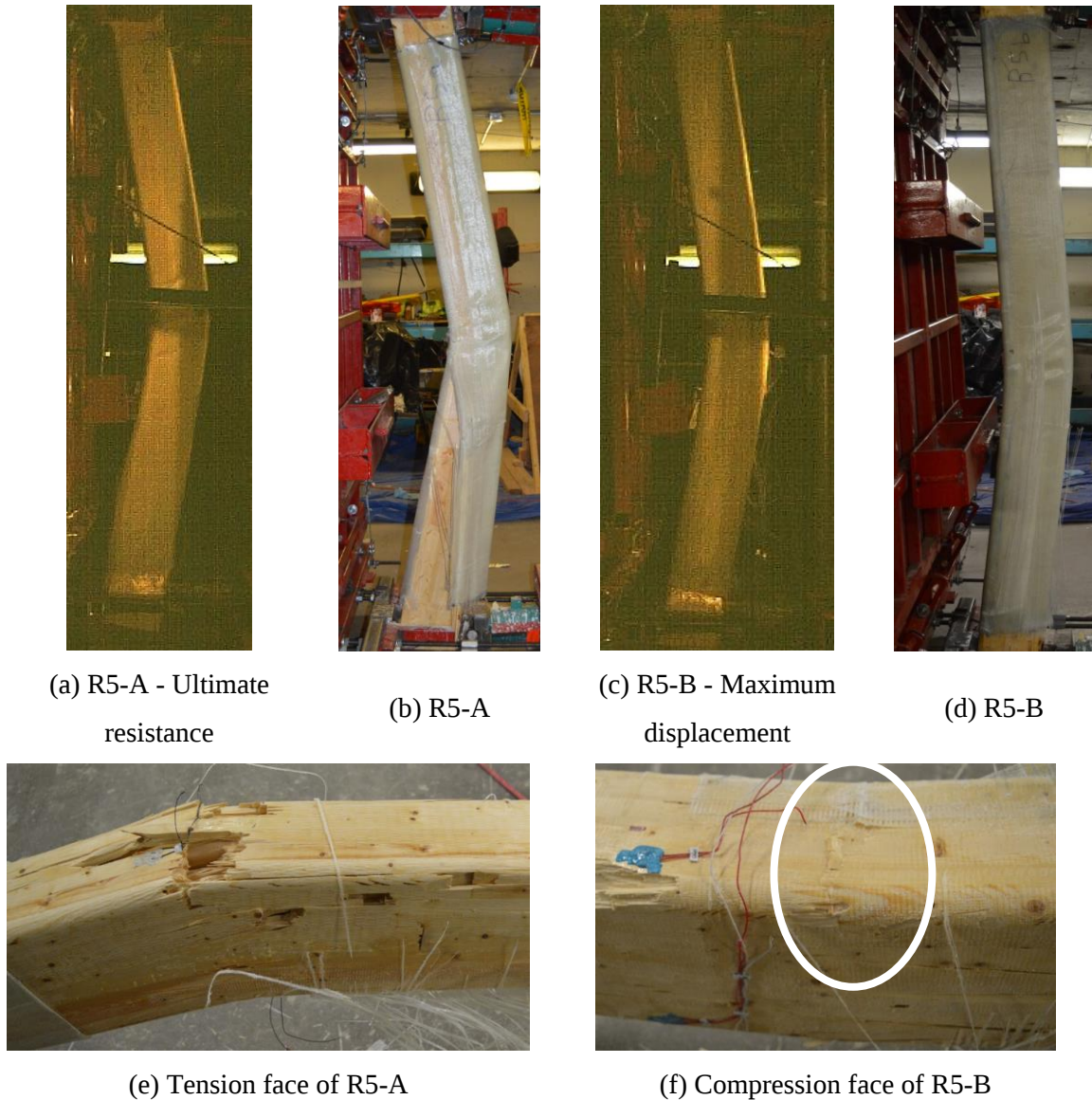


Figure 4.12: Failure modes of Retrofits 5A and 5B

Representative failure modes for Retrofit 6 are shown in Figure 4.13. Figure 4.13a shows specimen R6-A at a displacement corresponding to approximately 3.2 times that observed at maximum resistance without de-bonding. Specimen R6-A did not reach ultimate failure and recovered as shown in Figure 4.13b with a permanent deflection corresponding to sixty percent of the maximum displacement. Although specimen R6-A was subjected to a pressure-impulse combination similar to that of specimen R5-A, it can be seen that the deflection as well as the associated damage in R6-A is significantly less.

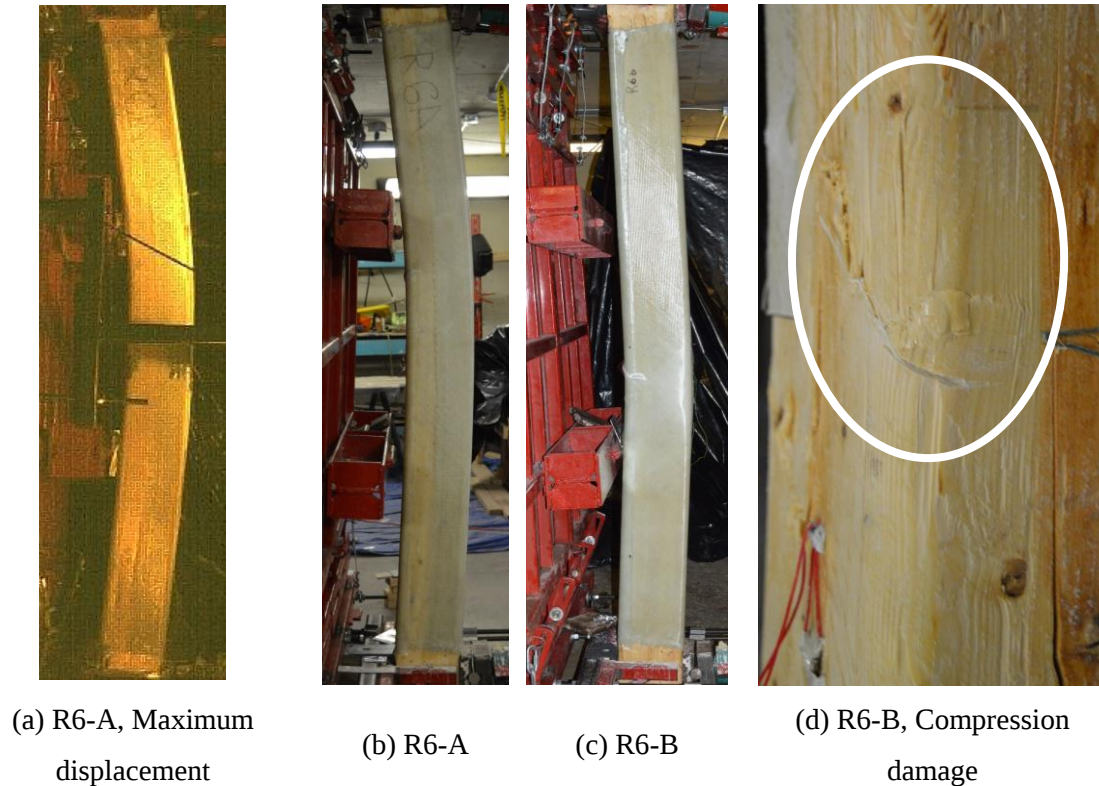


Figure 4.13: Failure modes of Retrofits 6A and 6B

Figure 4.13c shows specimen R6-B after being subjected to a similar pressure and impulse combination as for R6-A. While no de-bonding was observed, yielding of the FRP was deduced by the change in colour of the FRP throughout specimen (Fig. 4.13c). R6-B did not reach ultimate resistance, but deflected 3.6 times that recorded at maximum resistance. Compression damage, as evident by the “wrinkling” of the fibres, was observed for both specimens (Fig. 4.13d).

Figure 4.14 shows representative failure modes for specimens R7-A-[86] (Figs. 4.14a, 4.14b, 4.14e) and R7-B-[86] (Figs. 4.14c, 4.14d). The two specimens were only different in that specimen R7-A had one layer of unidirectional U-shaped GFRP compared to three layers in specimen R7-B. Figure 4.14a shows specimen R7-A at ultimate resistance corresponding to a displacement of 4.01 times that at maximum resistance without any de-bonding. Tearing of the FRP as shown in Figure 4.14b occurred past the point of ultimate resistance. Figure 4.14e clearly shows that specimen R7-A was subjected to a pressure and impulse combination exceeding its ability to resist the load as evidenced by the clear cut through the

section. Also in this case, the confinement was effective in limiting the damage to a small region.

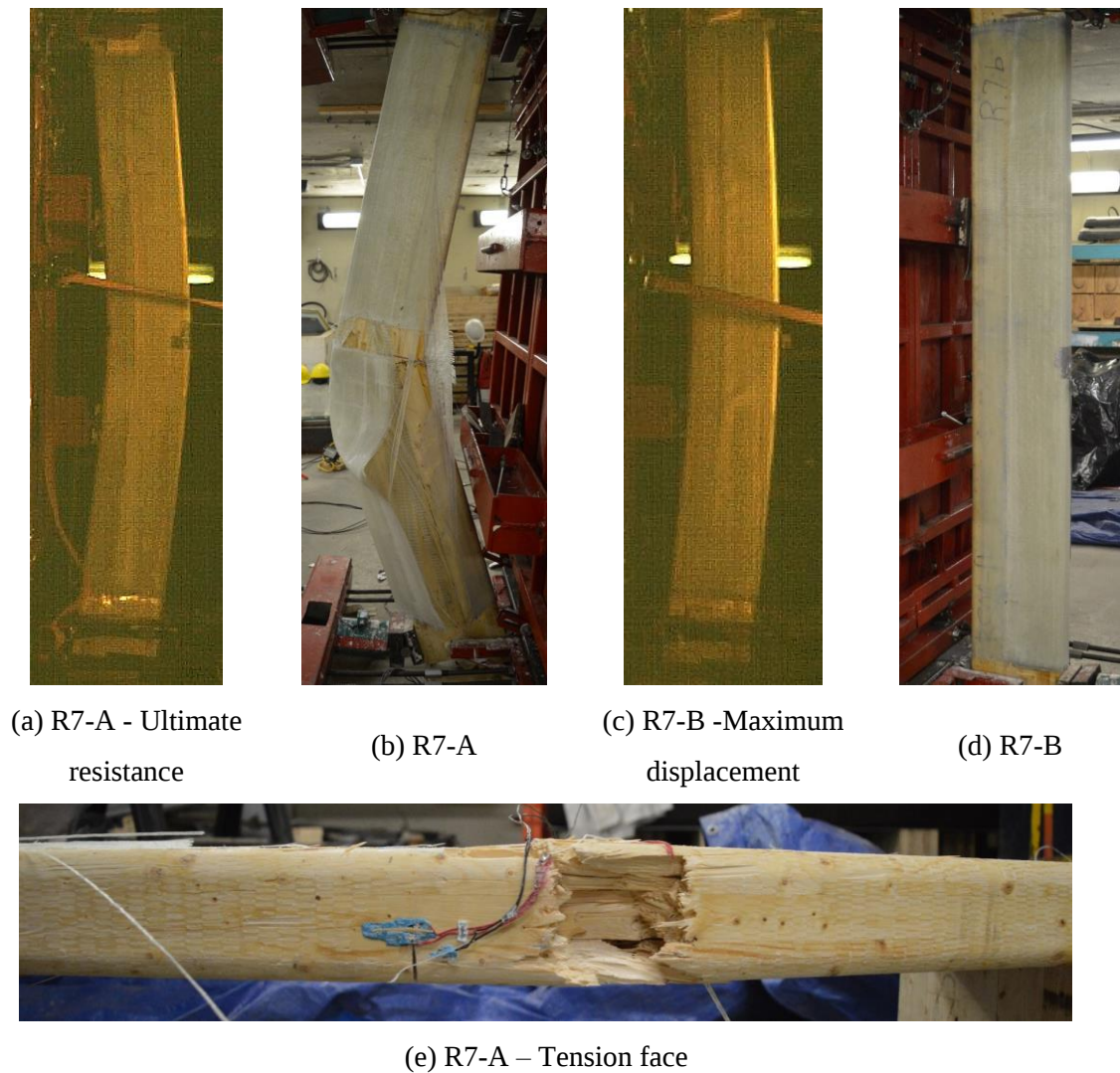


Figure 4.14: Failure modes of Retrofits 7A and 7B

Specimen R7-B was subjected to a slightly higher pressure and impulse combination compared to specimen R7-A since it had two more U-shaped layers of unidirectional GFRP. No de-bonding was observed for specimen R7-B. The specimen did not reach ultimate failure, however it reached a displacement corresponding to 2.7 times that recorded at maximum resistance after which it almost completely recovered to its initial position with minimal visible damage (Figs. 4.14c and 4.14d).

For Retrofits R8-A and R8-B, the effect of combining bidirectional fabrics on top of one another was investigated. Figure 4.15 shows representative failure modes, where it can be seen that the failures are significantly different. Figure 4.15a shows specimen R8-A at ultimate resistance corresponding to a displacement 2.77 times that recorded at maximum resistance without significant de-bonding, but with some localized FRP buckling.

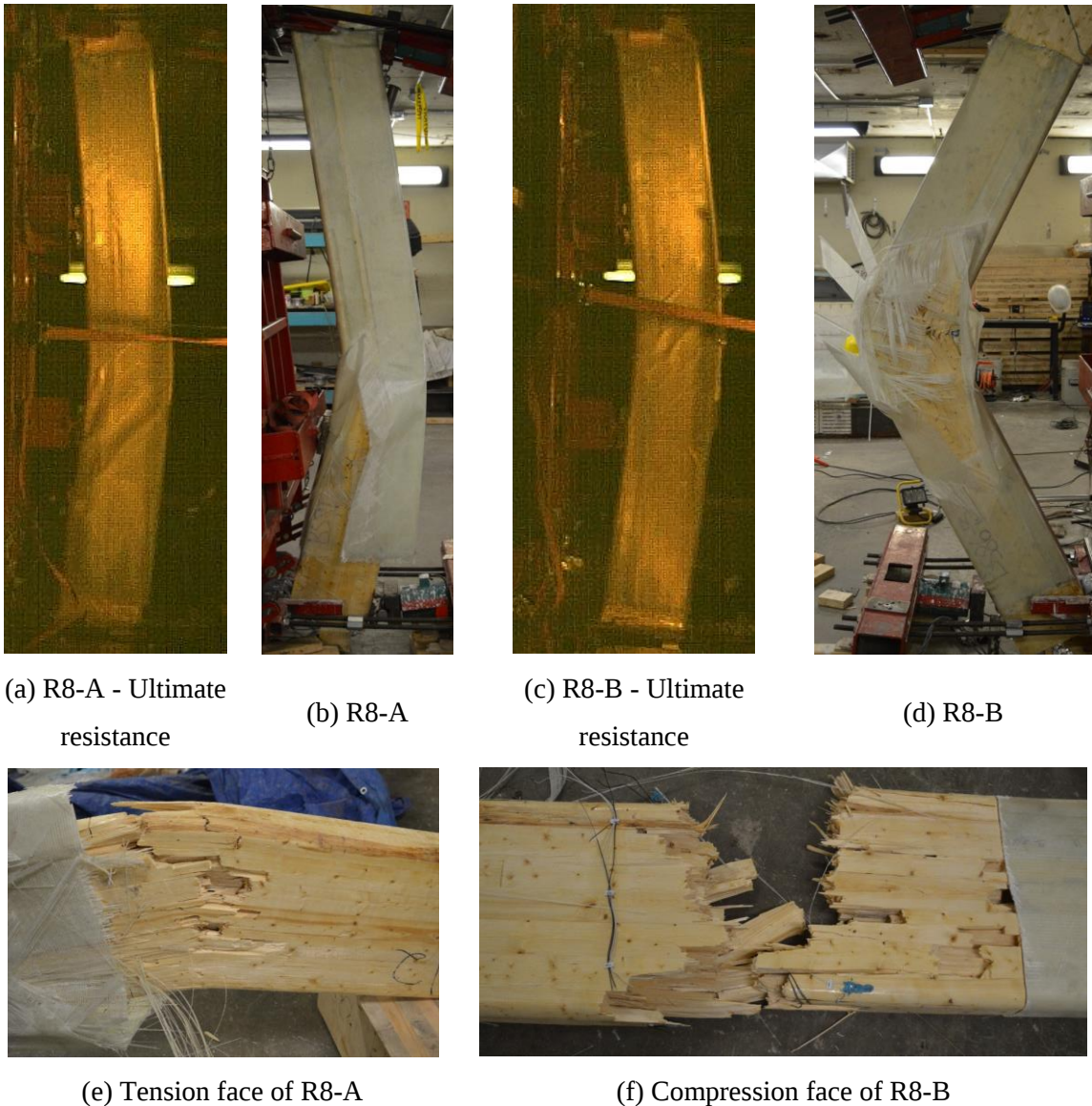


Figure 4.15: Failure modes of Retrofits 8A and 8B

Additionally, some damage along the splice placed on the compression side was observed at ultimate resistance. Figure 4.15b shows specimen R8-A following the test. The specimen was able to recover some displacement despite having a de-bonding of the FRP.

Figure 4.15c shows specimen R8-B at ultimate resistance, which corresponded to a displacement of 3.33 times that recorded at maximum resistance. Tearing of the FRP as shown in Figure 4.15d depicting the final deflected shape was observed to occur past the point of ultimate resistance.

Figure 4.16 shows representative failure modes for Retrofits 9-A and 9-B. Figure 4.16a shows specimen R9-A at a displacement corresponding to 4.00 times that recorded at maximum resistance while Figure 4.16b shows the specimen's final deflected shape. While it can be seen that some splintering of the FRP confinement occurred along the direction of the reinforcing fibres, specimen R9-A did not reach its ultimate resistance. Furthermore, damage to the FRP is evident by the change in colour near the mid-span of the beam. Since R9-B had a layer of U-shaped reinforcement, it was subjected to a higher pressure and impulse combination than R9-A. Higher damage level was observed as a result of the higher shot, as shown in Figures 4.16c and 4.16d. The de-bonding of the FRP along the splice on the compression face (Fig. 4.16d) was observed to occur after the point of ultimate resistance (Fig. 4.16c), which corresponded to a displacement of 2.30 times of that recorded at maximum resistance. Figures 4.16e and 4.16f show the failure on the tension and compression face, respectively, for specimen R9-A. Again, the FRP was effective in limiting the damage to a small region. It should also be noted that no significant debris throw was observed with any of the retrofits considered.

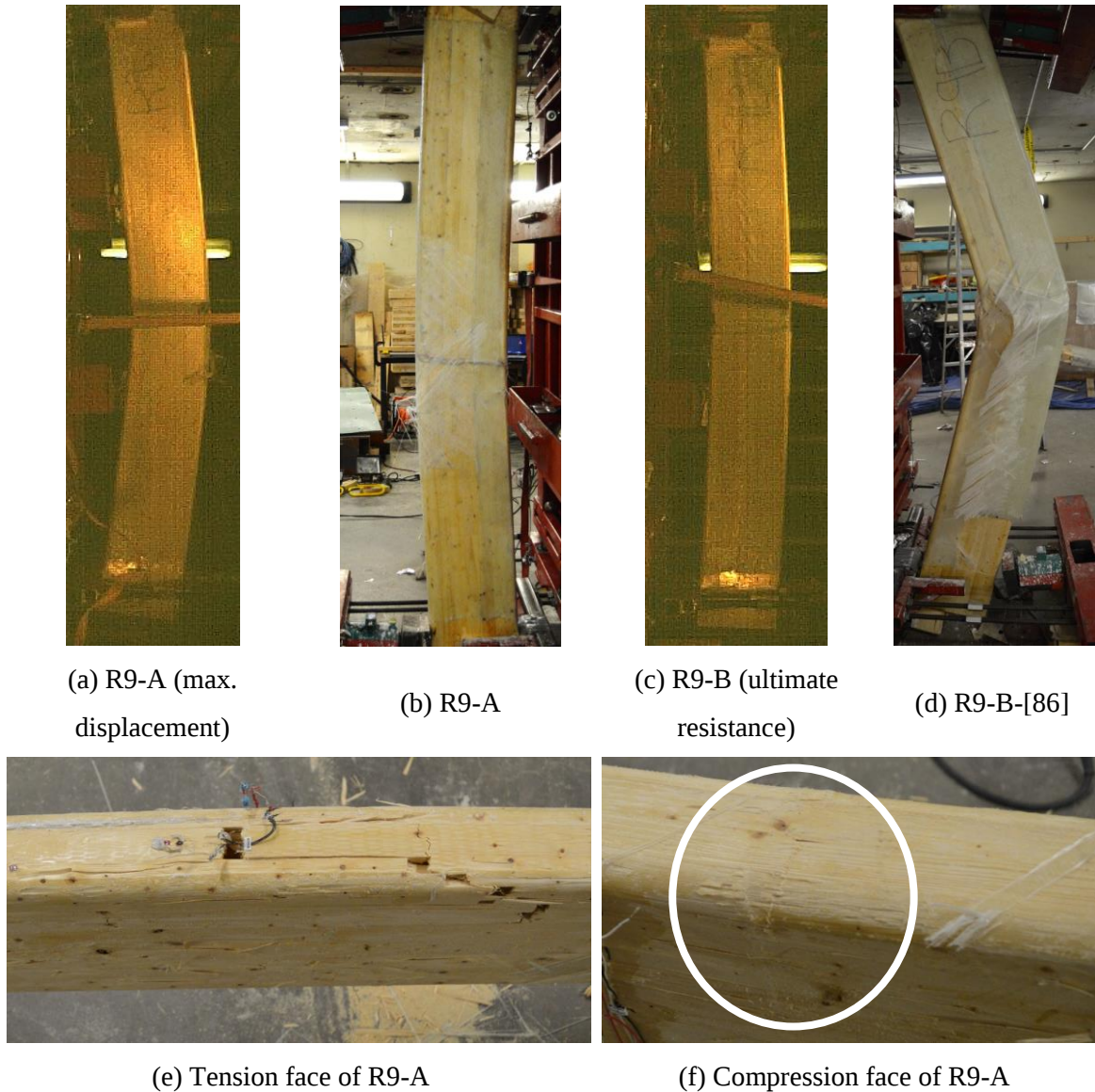


Figure 4.16: Failure modes of Retrofits 9A and 9B

Previously tested specimens (C-[137]) were restored and retrofitted using unidirectional FRP acting as confinement at ninety degrees relative to the wood fibres. The restored and retrofitted beams sustained significant residual displacements after testing (Fig. 4.17a). A closer look at the failure shows that the damage was confined to a limited region (Fig. 4.17b), compared to that incurred during the previous shots (denoted by the black permanent marker). Some compression failure (e.g. wrinkling of the fibres) was also observed. More details on the retrofitted beams' behaviour can be found in Appendix E.



(a) D3.1



(b) Close-up of damage for D3.1

Figure 4.17: Dynamic representative failure mode for restored beams

4.4 Strain-rate effects

4.4.1 Flexural strength of unretrofitted beams

Table 4.3 summarizes the static test results for the nineteen beams tested destructively including key information describing the beams' behaviour and whether or not failure was dominated by continuous FJs across the member width. Cases with continuous FJs are denoted with subscript C, whereas staggered FJs are denoted with subscript S. For the unretrofitted beams, and due to the brittle nature of the behaviour, the ultimate failure deflection is equal to that at maximum resistance. For clarity, only values corresponding to the maximum resistance and displacement at maximum resistance are reported in Table 4.3. The complete test results can be found in Appendix C.

Table 4.3: Summary of static test results

Specimen	R_{max}^a (kN)	ΔR_{max}^b (mm)	$\epsilon_{f-T}^c \times 10^{-4}$ (mm/mm)	$\epsilon_{f-C}^d \times 10^{-4}$ (mm/mm)	MOR ^e (MPa)	MOE ^f (MPa)	FJ failure
B1-[80]	104.8	27.2			50.6	8,535	N
B2-[80]	124.1	34.8	No data ¹		59.9	9,090	N
B3-[80]	123.0	32.8			59.4	8,511	N
B4-[80]	102.6	25.6	44.8	-145.7	49.5	7,953	Y _C
B5-[80]	92.7	22.1	39.4	-22.4	44.8	8,256	Y _C
B6-[80]	98.6	24.3	49.0	-39.4	47.6	7,785	N
B7-[80]	99.4	27.5	No data ¹		48.0	8,455	N
B8-[80]	106.9	25.9	52.1	-35.7	51.6	8,827	N
B9-[80]	102.3	28.5			49.4	8,031	N
B10-[80]	111.5	33.5	No data ¹		53.8	7,986	N
B11-[80]	98.1	27.3			48.5	8,623	N
	Average	28.1	46.3	-60.8	51.2	8,368	
	Std. Dev.	3.65	4.77	49.43	4.35	369	
	CV	0.130	0.103	-0.813	0.085	0.044	
B1-[86]	174.3	19.2	39.1	-47.7	43.6	8,275	Y _C
B2-[86]	162.3	20.0	35.1	-52.8	40.7	8,079	Y _S
B3-[86]	159.9	17.5	35.7	-31.7	40.1	8,296	Y _S
B4-[86]	166.1	18.8	34.1	-62.9	41.6	7,451	N
	Average	18.9	36.0	-48.8	41.5	8,025	
	Std. Dev.	0.92	1.88	11.27	1.31	342	
	CV	0.048	0.052	-0.231	0.032	0.043	
B1-[137]	131.2	22.89	32.3	-44.9	42.0	9,362	Y _S
B2-[137]	161.5	25.17	38.2	-53.8	52.0	10,966	Y _S
B3-[137]	132.1	21.36	38.5	-41.5	42.8	9,739	Y _S
B4-[137]	141.7	25.51	54.3	-69.2	45.8	10,226	Y _S
	Average	23.7	40.8	-52.4	45.6	10,073	
	Std. Dev.	1.70	8.15	9.59	3.49	536	
	CV	0.072	0.200	-0.183	0.076	0.053	

^aMaximum resistance^bDisplacement at maximum resistance^cStrain at tensile rupture^dMaximum compressive strain^eModulus of rupture^fModulus of elasticity¹Bond incompatibility between strain gauges and wood prevented the recording of meaningful data

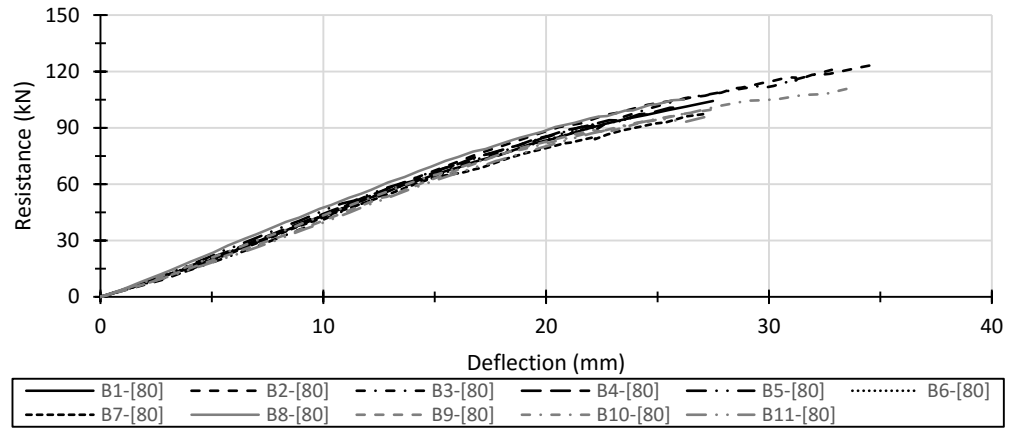
The MOE and MOR presented in Table 4.3 are “apparent material properties”, which means that they are based on the average global response of the beams. Equations [4.1] and [4.2] were used to obtain the MOE and MOR, respectively, under four-point bending using the experimental data.

$$MOE = \frac{23 * K * L^3}{1296 * I} \quad [4.1]$$

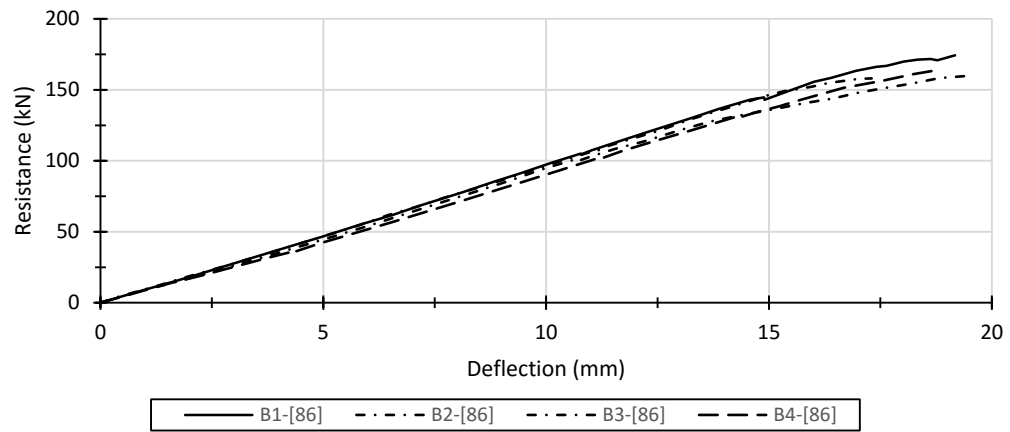
$$MOR = \frac{R_{max} * L}{6 * S} \quad [4.2]$$

where K is the stiffness obtained from 10 % to 40 % of the experimental load deflection relationship, L is the clear span length, I is the moment of inertia of the beam, R_{max} is the maximum force resisted by the member obtained from the static reactions, and S is the section modulus.

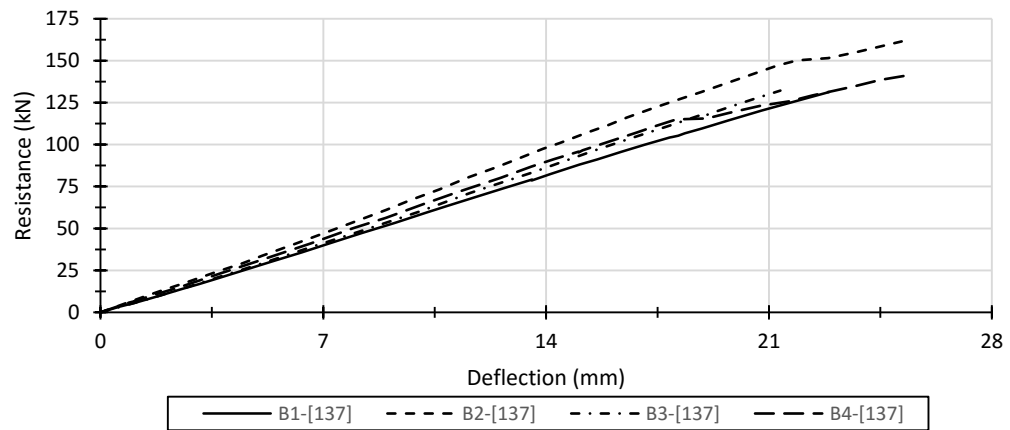
Figures 4.18a, 4.18b, and 4.18c show the static resistance curves for the B-[80], B-[86], and B-[137] beams, respectively. In all cases, the beams behaved linearly up to maximum resistance with no significant post-peak resistance.



(a) B-[80]



(b) B-[86]



(c) B-[137]

Figure 4.18: Static resistance curves of glulam beam specimens

Nineteen beams were tested destructively under dynamic loading. Tables 4.4 and 4.5 present a summary of the dynamic elastic and destructive tests, respectively.

Table 4.4: Summary of non-destructive dynamic beam test results

Specimen	P_R^a (kPa)	I_R^b (kPa-ms)	Δ_{max}^c (mm)	$\epsilon_T^d \times 10^{-4}$ (mm/mm)	$\dot{\epsilon}^e$ (s ⁻¹)
B12.1-[80]	14.2	158.2	15.6	24.9	0.10
B13.1-[80]	10.7	122.9	11.2	15.2	0.06
B14.1-[80]	9.2	60.1	6.4	7.9	0.03
B15.1-[80]	18.9	188.1	18.5	31.6	0.13
B17.1-[80]	28.3	231.8	26.1	35.0	0.17
B5.1-[86]	14.9	126.0	6.2	8.0	0.03
B5.2-[86]	45.9	448.6	21.7	37.8	0.22
B6.1-[86]	13.2	140.5	6.2	8.2	0.04
B7.1-[86]	16.8	155.0	7.4	10.8	0.05
B8.1-[86]	6.9	36.4		No data ¹	
B5.1-[137]	11.0	124.1	7.6	9.3	0.04
B6.1-[137]	13.1	121.3	7.0	10.7	0.04
B7.1-[137]	11.7	125.7	8.4	10.4	0.05
B8.1-[137]	6.2	60.8		No data ¹	
B9.1-[137]	10.9	105.5	4.2	No data ¹	

^aReflected pressure

^bReflected impulse

^cMaximum recorded displacement

^dMaximum tensile strain

^eStrain rate

¹Noise interference causing experimental time history to be unusable

In Table 4.5, the average for each beam size is presented for the maximum recorded displacement, maximum dynamic resistance, tensile failure strain, maximum compressive strain, and dynamic modulus of rupture and modulus of elasticity. Due to the brittle nature of the unretrofitted beams, only the maximum resistance and deflection at maximum resistance are reported in Table 4.5, which also correspond to the ultimate resistance and deflection.

Table 4.5: Summary of destructive dynamic beam test results

Specimen	P _R ^a	I _R ^b	Δ _{max} ^c	R _{max-d} ^d	Δ _{Rmax} ^e	ε _{f-T} ^f x 10 ⁻⁴	ε̇ ^g	ε _{f-C} ^h x 10 ⁻⁴	MOR _d ⁱ	MOE _d ^j	FJ Failure
	(kPa)	(kPa-ms)	(mm)	(kN)	(mm)	(mm/mm)	(s ⁻¹)	(mm/mm)	(MPa)	(MPa)	
B12.2-[80]	40.7	369.5	132.7	109.8	34.4	47.9	0.30	-61.6	59.0	9,594	N
B13.2-[80]	42.7	397.1	72.6	107.0	29.7	42.0	0.28	-51.9	57.5	9,373	N
B14.2-[80]	41.4	333.8	37.8	107.1	32.5	45.6	0.23	-33.0	57.5	8,877	N
B15.2-[80]	40.0	372.6	115.5	98.5	31.6	57.0	0.38	-50.8	53.0	9,279	N
B16.1-[80]	52.1	390.5	183.9	104.9	44.6	28.5	0.25	-44.4	56.4	10,978	N
B17.2-[80]	44.7	328.4	137.9	111.5	30.5	28.9	0.17	-56.0	59.9	9,925	N
B18.1-[80]	49.3	337.0	111.2	101.8	35.7	59.4	0.39	-70.8	54.7	8,251	N
B19.1-[80]	54.5	353.4	94.4	88.5	21.9	34.6	0.31	-37.9	47.6	8,969	Y _C
B20.1-[80]	54.8	347.0	159.8	82.9	20.8	65.3	0.51	-42.4	44.6	12,161	Y _C
B21.1-[80]	58.2	292.6	99.7	83.2	27.4	42.5	0.28	-44.5	44.7	8,107	Y _C
		Average	114.5	99.5	30.9	45.2		-49.3	53.5	9,551	
		Std. Dev.	40.0	10.3	6.5	12.0		10.8	5.5	1,169	
		CV	0.350	0.103	0.210	0.265		-0.218	0.103	0.122	
B5.3-[86]	48.7	549.7	70.9	190.9	22.2	37.1	0.23	-	49.1	8,366	Y _S
B6.2-[86]	66.4	697.1	114.4	224.4	26.0	42.0	0.26	-44.9	57.7	7,863	Y _S
B7.2-[86]	56.7	672.2	92.9	190.9	23.5	39.9	0.28	-41.2	49.0	7,781	Y _S
B8.2-[86]	50.9	605.1	119.3	123.0	16.2	24.1	0.39	-50.6	31.6	7,216	Y _C
		Average	99.4	182.3	22.0	35.8		-45.6	46.8	7,807	
		Std. Dev.	19.2	36.9	3.6	7.0		3.9	9.5	408	
		CV	0.194	0.202	0.163	0.195		-0.085	0.202	0.052	
B5.2-[137]	54.3	668.5	200.5	137.6	24.5	30.9	0.24	-57.8	45.5	8,814	Y _S
B6.2-[137]	50.0	670.3	187.5	123.0	20.6	29.7	0.26	-65.2	40.7	9,217	Y _C
B7.2-[137]	54.4	536.5	176.6	153.6	29.1	41.8	0.29	-55.6	50.8	9,575	Y _S
B8.2-[137]	50.2	547.5	199.9	153.1	29.0	30.4	0.20	-51.4	50.7	9,511	Y _S
B9.2-[137]	53.2	491.9	115.6	170.1	30.9	47.2	0.33	-25.0	56.3	9,446	Y _S
		Average	176.0	147.5	26.8	36.0		-51.0	48.8	9,313	
		Std. Dev.	31.5	16.0	3.8	7.1		13.7	5.3	277	
		CV	0.179	0.108	0.140	0.198		-0.269	0.108	0.030	
^a Reflected pressure			^b Reflected impulse				^c Maximum recorded displacement				
^d Maximum dynamic resistance			^e Deflection at maximum resistance				^f Strain at tensile rupture				
^g Strain rate			^h Maximum compressive strain				ⁱ Dynamic modulus of rupture				
^j Dynamic modulus of elasticity											

Unlike in the case of static loading, where the sum of the measured reactions equals to the applied load, the measured dynamic reactions are not equal to the dynamic resistance of the

member (Biggs, 1964). The dynamic equilibrium of the system must therefore include the inertia forces, loading function and reactions. For single-degree-of-freedom (SDOF) models, the dynamic reactions derived from both the loading type and boundary conditions are widely available (e.g. Biggs, 1964; PDC-TR 06-01 Revision 1, 2008). However, since a load transfer device (LTD) with significant weight was used to transfer the pressure to the beams as two concentrated loads, its effect on the location of the resulting inertia forces of the beam must be considered in the analysis of the dynamic resistance. Equation [4.3] demonstrates how the dynamic resistance of the glulam beams was obtained from the measured dynamic reactions- and pressure-time histories.

$$R(t) = \left\{\frac{6}{L}\right\} \left\{V(t)x_{eq} + 0.5 \left[\frac{L}{3} - x_{eq}\right] F(t)\right\} \quad [4.3]$$

where $R(t)$ is the beam resistance, $V(t)$ is the dynamic reaction, $F(t)$ is the applied force, L is the clear span, and x_{eq} is the distance from the support to the point of application of the equivalent inertia force.

Equation [4.3] was obtained by considering the dynamic equilibrium of the idealized system at half-span and by taking the sum of moments about the point at which the equivalent inertia force acts. The inertia forces' distribution was assumed to follow that of the static deflected shape (Biggs, 1964). In the derivation, the specimen was assigned a distributed mass while half of the LTD mass was lumped at each load application point.

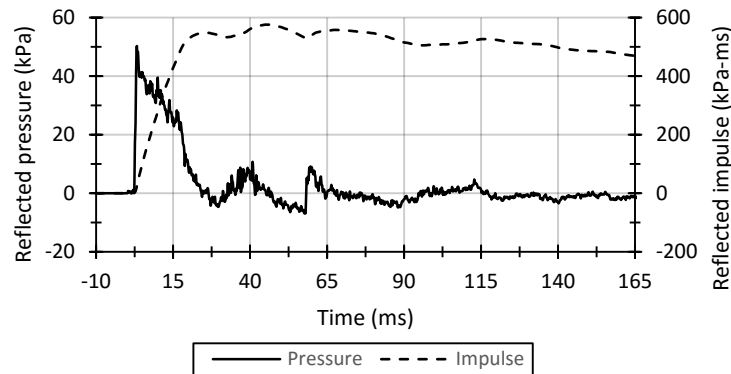
This idealized approach was also successfully used by Jacques (2016) for reinforced concrete beams. The location of the equivalent inertia force from the support of the equivalent SDOF system is shown in Equation [4.4].

$$x_{eq} = \frac{0.102\bar{m}L^2 + 0.290m_cL}{0.319\bar{m}L + 0.870m_c} \quad [4.4]$$

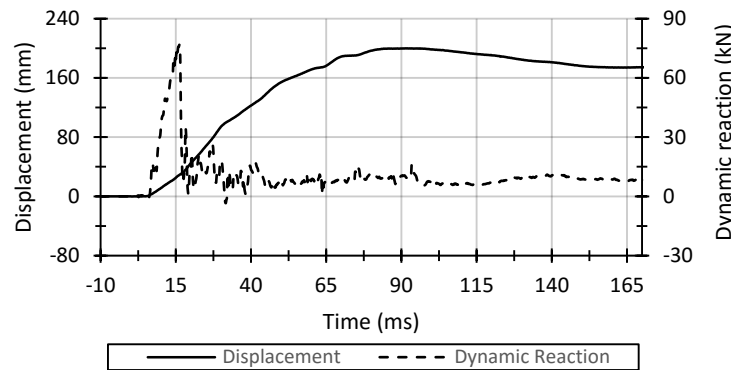
where $\bar{m}$ is the distributed mass of the beam, m_c is half of the mass of the LTD lumped at the load application point, and L is the span length. The maximum dynamic reaction, R_{max-d} , and dynamic stiffness, K_d , obtained from the dynamic resistance, can then be used in Equations [4.1] and [4.2] to obtain MOE_d and MOR_d , respectively.

The derivation of Equations [4.3] and [4.4] can be found in Appendix F.

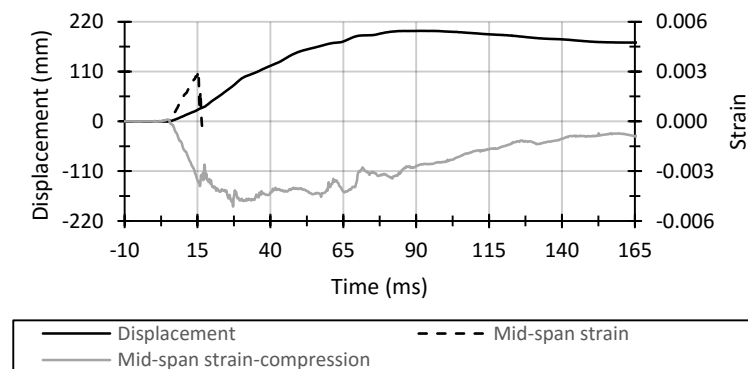
Figure 4.19 shows representative time-histories obtained from the dynamic test for specimen B8.2-[137], including reflected pressure, impulse, displacement, dynamic reaction, and strain profiles.



(a) Pressure and impulse



(b) Displacement and dynamic reaction



(c) Displacement and strain

Figure 4.19: Experimental dynamic time varying functions for B8.2-[137]

From Figure 4.19b, it can be seen that the maximum recorded displacement (Δ_{max}) as reported in Table 4.5 occurs at a deflection significantly greater than that recorded at maximum resistance for specimen B8.2-[137]. It can also be seen that the tensile rupture (Fig. 4.19c) occurs at approximately the same time the maximum reaction is recorded (Fig. 4.19b). Furthermore, the maximum compressive strain is observed to occur much later than that of tensile failure (Fig. 4.19c). Figure 4.20 illustrates the progression of damage for the different points such as initial tensile rupture (Fig. 4.20a), maximum resistance (Fig. 4.20b), maximum compressive strain (Fig. 4.20c), and maximum recorded deflection (Fig. 4.20d) observed in Figure 4.189 for specimen B8.2-[137].

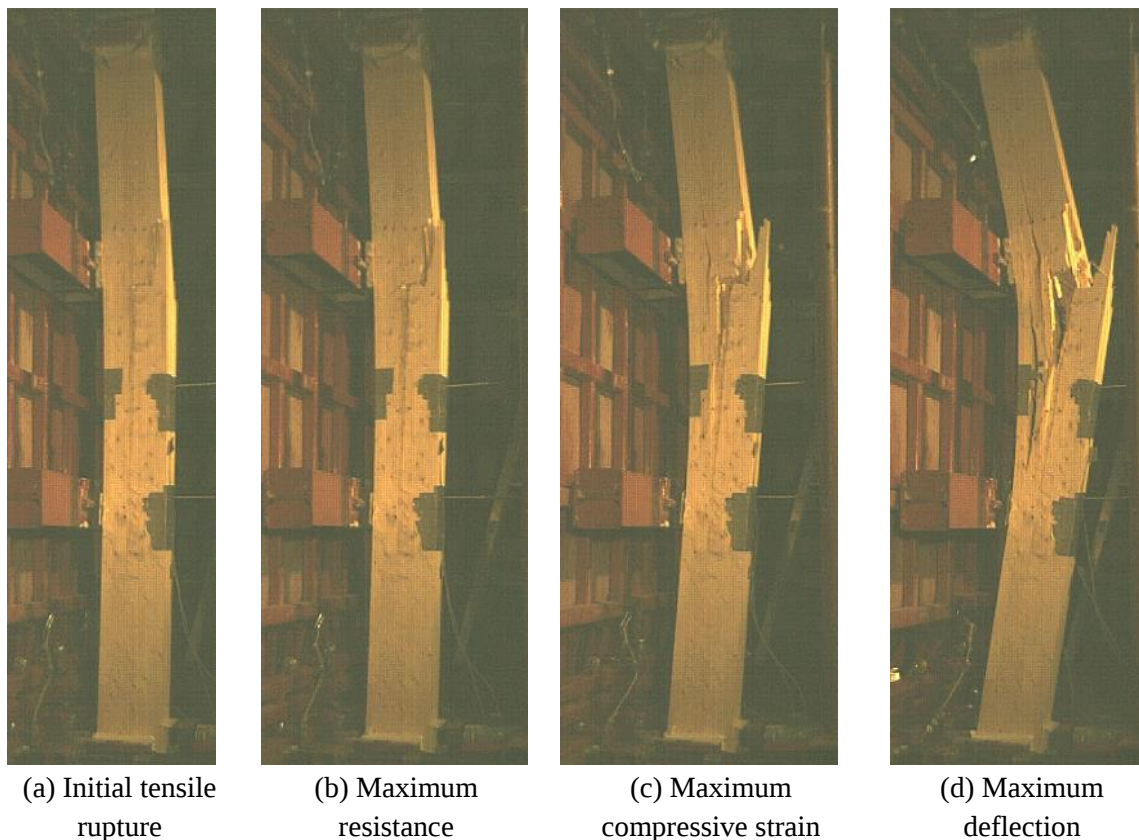
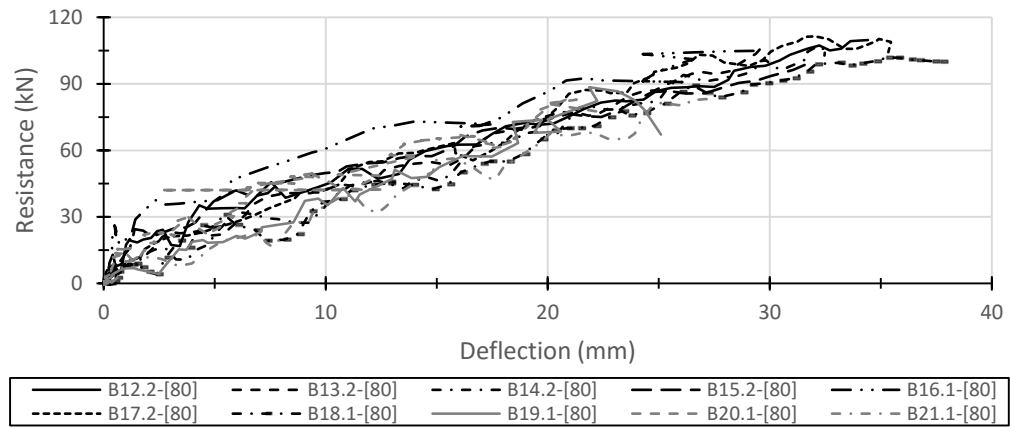
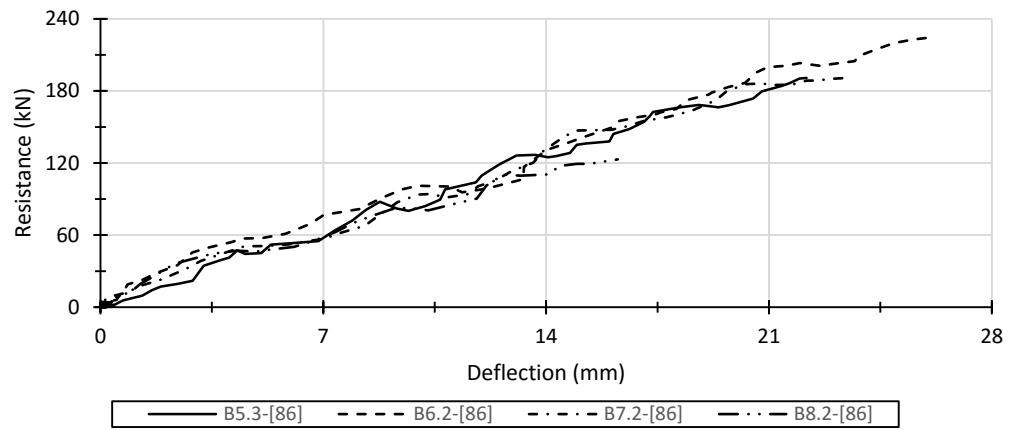


Figure 4.20: Damage progression in beam B8.2-[137]

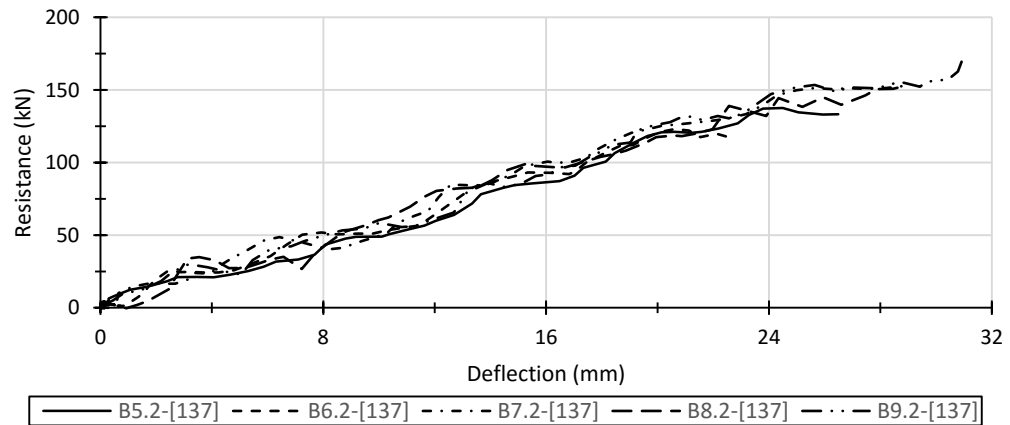
Figures 4.21a, 4.21b, and 4.21c show the dynamic resistance curves for the B-[80], B-[86], and B-[137] beams, respectively. Similar to the static case, the specimens tested dynamically behaved linearly up to maximum resistance with no significant post-peak resistance. More details on the static and dynamic test results for the unretrofitted beams can be found in Appendix C.



(a) B-[80]



(b) B-[86]



(c) B-[137]

Figure 4.21: Dynamic resistance curves of glulam beam specimens

4.4.2 Effect of axial load

Table 4.6 presents a summary of the results for the six glulam members that were tested dynamically to failure under combined axial and transverse four-point bending with simply supported boundary conditions.

Table 4.6: Summary of experimental dynamic glulam column test results

Specimen	P_R^a (kPa)	I_R^b (kPa-ms)	$R_{max,\Delta-d}^c$ (kN)	$\Delta R_{max,\Delta}^d$ (mm)	$\epsilon_{f-T}^e \times 10^{-4}$ (mm/mm)	$\dot{\epsilon}^f$ (s ⁻¹)	$P_{initial}^g$ (kN)	P_{final}^h (kN)
C1.1-[137]	47.3	458.0	119.6	27.6	45.7	0.34	100.0	50.0
C2.1-[137]	54.1	604.8	145.1	30.2	35.4	0.28	160.0	108.9
C3.1-[137]	44.0	466.9	112.6	27.6	48.4	0.33	338.0	281.5
C4.1-[137]	60.4	664.2	157.7	30.0	49.8	0.40	330.0	271.7
C5.1-[137]	56.9	513.5	145.7	23.7	38.9	0.26	540.2 ^{1,2}	531.7 ^{1,2}
C6.1-[137]	60.5	541.5	184.2	29.8	54.0	0.40	455.8 ^{1,2}	438.9 ^{1,2}

^aReflected pressure

^bReflected impulse

^cColumn dynamic resistance

^dDeflection at maximum resistance

^eStrain at tensile failure

^fStrain rate

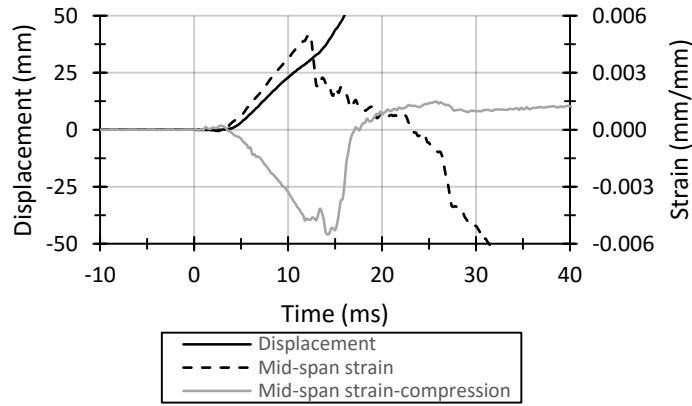
^gInitial axial load

^hAxial load at maximum resistance

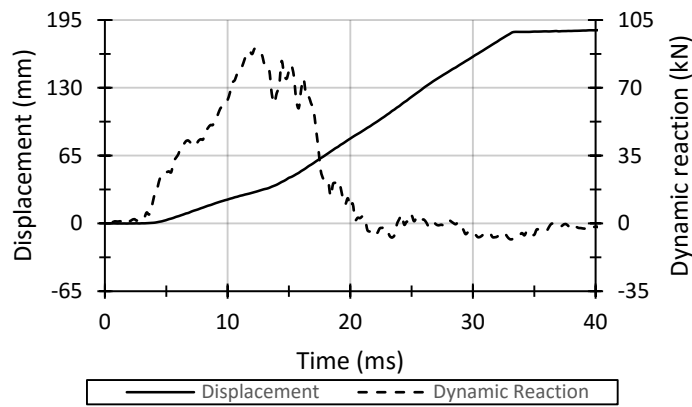
¹Noise interference causing experimental time history to be unusable

²Axial load interpolated linearly from hydraulic jack reading prior to and after the test

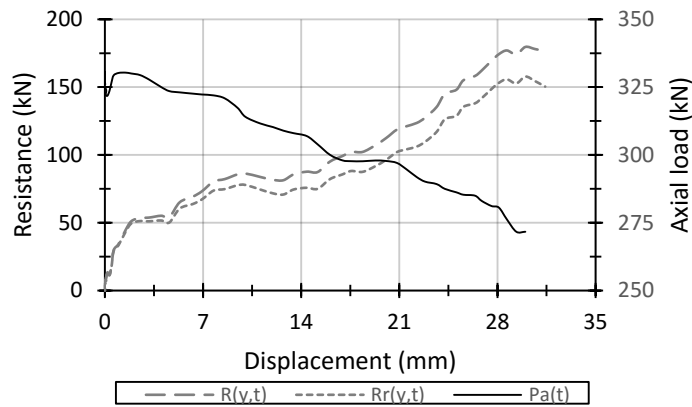
It should again be emphasized that the final state of damage (e.g. Fig. 4.8) should not be directly linked to tensile rupture or ultimate strength. For example, it can be seen in Figure 4.22 (C4.1-[137]) that both the tension and compression strain gauges indicate failure at approximately the same time, corresponding to a displacement that is significantly less than that shown by the displacement-time history, which never peaked but rather continued to deflect until the maximum stroke of the LVDT was reached (Fig. 4.22b).



(a) Displacement and strain



(b) Displacement and dynamic reaction



(c) Variation in axial load

Figure 4.22: Representative experimental data for C4.1-[137]

The dynamic reaction (Fig. 4.22b) peaked at approximately the same time complete failure of the specimen was attained. It can also be observed that the behaviour of the specimens could be approximated as linear with a significant loss in the resistance post-peak. From Table 4.6, it can be seen that for the ultimate shots the axial load was not constant throughout

the response of the column. Rather, the magnitude of axial load corresponding to peak resistance was observed to be significant (approximately 82 % of initial applied load, Fig. 4.22c).

The experimentally measured reactions were used to obtain the resistance of the specimens, as reported in Table 4.6, by considering the dynamic equilibrium of the system. Since Equation [4.3] does not account for the effect of axial load, the dynamic resistance of the columns was modified using Equation [4.5] which accounts for the secondary moment through an equivalent lateral load (ELL) (Nassr et al., 2013)

$$R_r(y, t) = R(y, t) - \eta(y, t) \quad [4.5]$$

where $R_r(y, t)$ is the dynamic resistance of the column, $R(y, t)$ is the resistance as a function of time if there were no axial load, $\eta(y, t)$ is the equivalent lateral load.

The ELL (last term in Equation [4.5]) reflects the spatial distribution of the blast load (e.g. four-point bending) and has the same magnitude as the secondary moment caused by the eccentricity of the applied axial load, as shown in Equation [4.6].

$$\eta(y, t) = \frac{6P_a(t)}{L} y(t) \quad [4.6]$$

where $P_a(t)$ is the experimentally measured axial load-time history, $y(t)$ is the mid-span deflection of the column as a function of time, and L is the clear span. Figure 4.22c shows the obtained resistance for column C4.1-[137].

The axial load-time histories recorded for all six specimens show that the axial load dropped at a consistent rate independently of axial load levels up to maximum resistance. However, it should be noted that past the peak resistance, there was a spike in axial load as shown in Figure 4.23 which was identified as incompatibility of boundary conditions at large displacements.

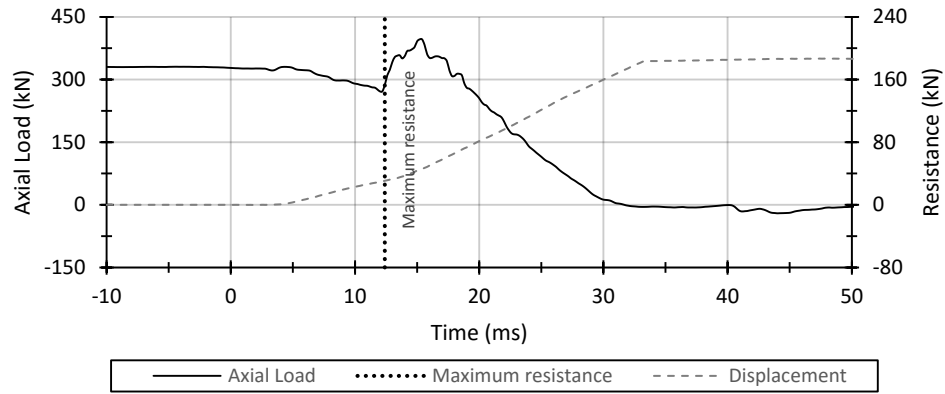


Figure 4.23: Comparison of displacement- and resistance-time histories for columns

Figure 4.24 shows the progression of damage in specimen C4.1-[137] for different stages of failure including: initial tensile rupture (Fig. 4.24a), maximum resistance (Fig. 4.24b), zero lateral resistance (Fig. 4.24c), and final state (Fig. 4.24d). It can be seen that the maximum resistance was attained at approximately the same time (14 ms) as the axial load started increasing. Since the point of interest is that of maximum resistance, there are no implications on the interpretation of the results.

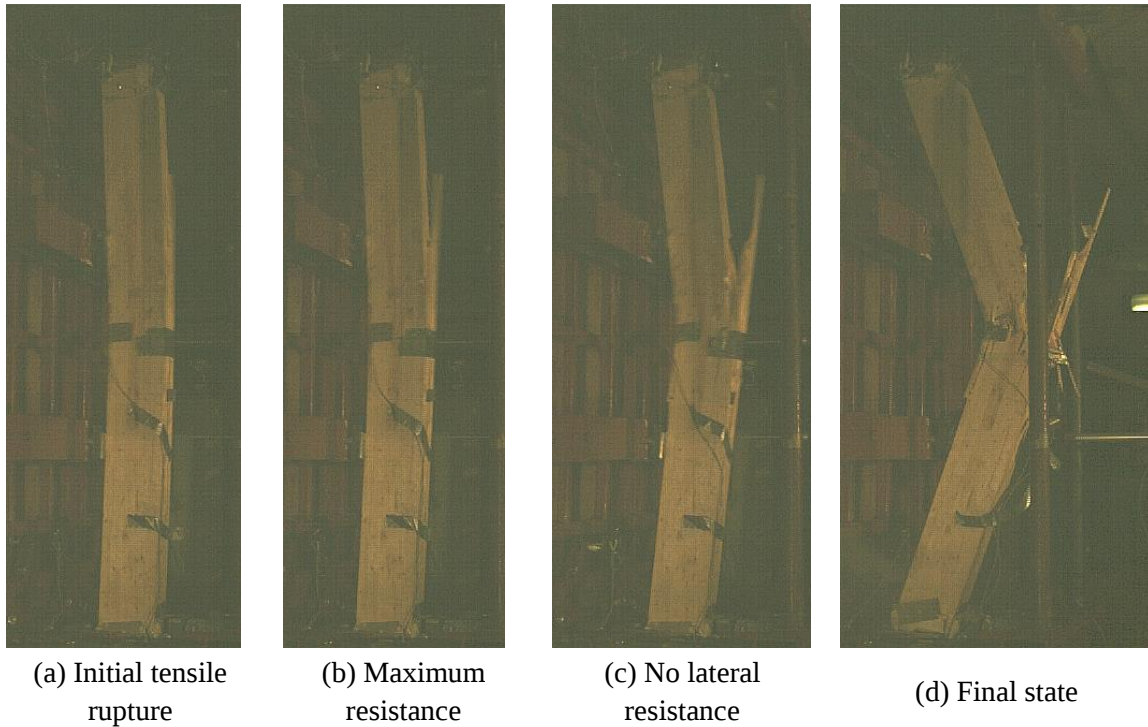


Figure 4.24: Damage progression in beam C4.1-[137]

4.4.3 Effect of FRP

A total of twenty-six retrofitted beams were tested destructively under static and dynamic loading. A summary of the static test results for Retrofits 1 through 4 is presented in Table 4.7. Also included in Table 4.7 is the average of the unretrofitted beams (B-[137]) to provide a reference case.

Table 4.7: Summary of static test results for retrofitted beams

Configuration	Specimen	R_{max}^a	ΔR_{max}^b	R_f^c	ΔR_f^d	K^e	Strain ($\times 10^{-4}$)		
		(kN)	(mm)	(kN)	(mm)	(kN/mm)	$\epsilon_{t,w-f}^f$	$\epsilon_{t,FRP-f}^g$	$\epsilon_{c,w-f}^h$
Unretrofitted	Average	141.6	23.7	141.6	23.7	6.0	40.8	-	-52.4
Retrofit 1	R1-1	196.7	32.9	196.7	32.9	6.4	39.2	40.2	-42.0
	R1-2	186.2	28.9	186.2	28.9	6.7	40.8	41.8	-37.8
	Average	191.5	30.9	191.5	30.9	6.6	40.0	41.0	-39.9
Retrofit 2	R2-1	206.3	32.6	71.2	71.0	6.7	44.1	45.1	-47.3
	R2-2	189.8	33.2	32.4	67.0	6.8	49.0	51.4	-94.5
	Average	198.0	32.9	51.8	69.0	6.8	46.5	48.2	-70.9
Retrofit 3	R3-1	214.3	38.5	207.0	53.5	6.6	43.6	94.2	-60.7
	R3-2	230.3	38.6	172.6	52.1	6.8	48.3	50.2	-
	Average	222.3	38.5	189.6	52.8	6.7	46.0	72.2	-60.7
Retrofit 4	R4-1	242.2	40.3	31.9	40.3	7.5	43.2	44.2	-59.8
	R4-2	226.5	33.8	151.0	64.3	7.4	40.0	48.9	-62.0
	Average	234.4	37.0	91.5	52.3	7.4	41.6	46.5	-60.9

^aMaximum resistance ^bDisplacement at maximum resistance ^cUltimate resistance
^dDisplacement at ultimate resistance ^eStiffness ^fStrain at wood tensile rupture
^gFRP tensile failure strain ^hMaximum wood compressive strain

From Table 4.7, it can be observed that, in general, the addition of FRP to glulam enhances the flexural resistance and stiffness relative to the unretrofitted beams. Figure 4.25 shows the static resistance curves for all eight retrofitted specimens tested statically.

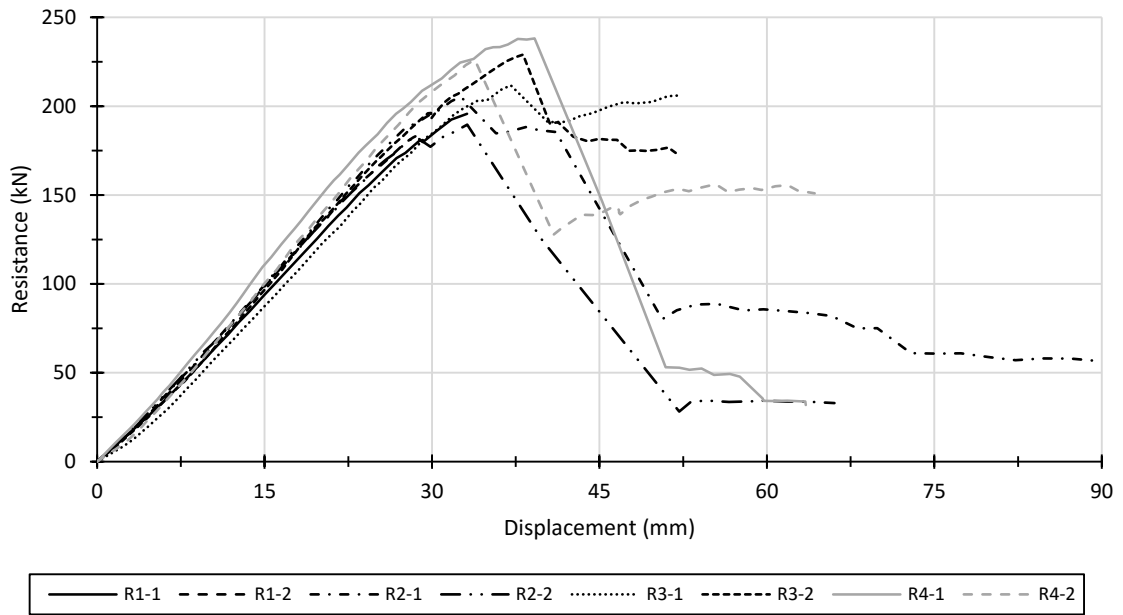


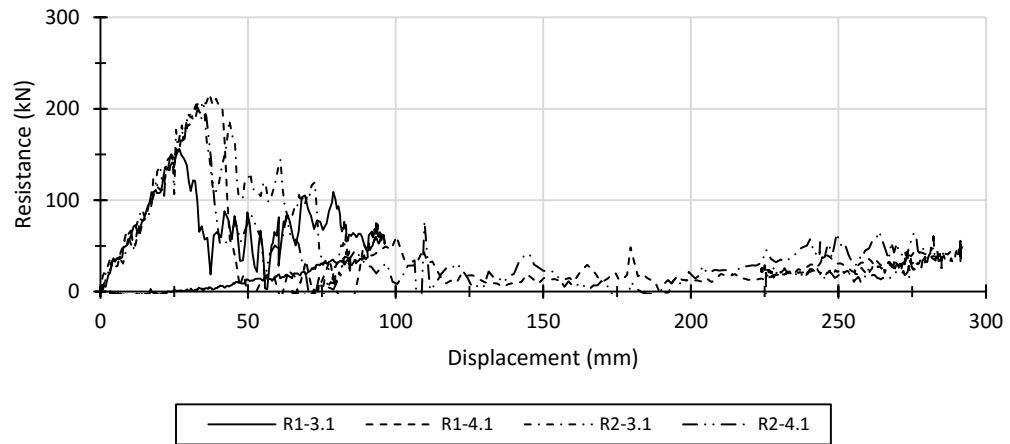
Figure 4.25: Resistance curves of Retrofits 1 to 4

The dynamic test results for Retrofits 1 through 6 are summarized in Table 4.8 to facilitate the comparison between the different retrofit configurations of the same cross-section. Also included in Table 4.8 is the average of the dynamically tested unretrofitted beams (B-[137]).

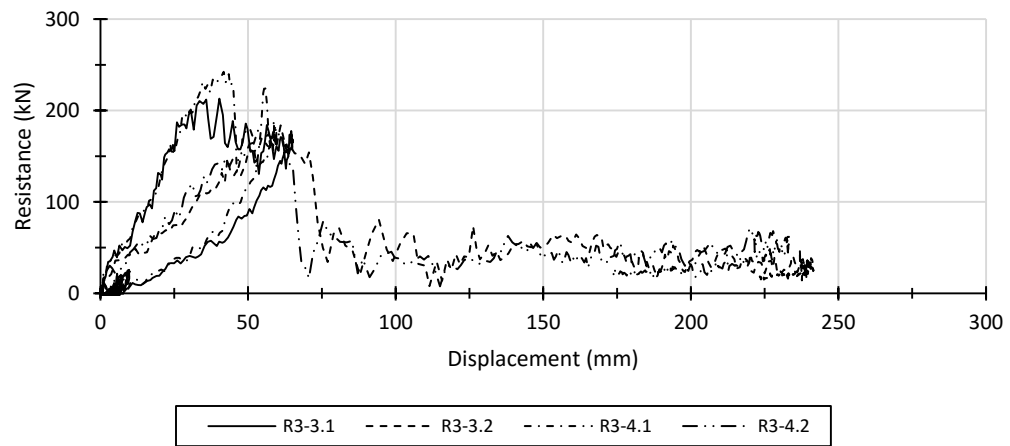
Table 4.8: Dynamic test results for Retrofits 1 to 6

Specimen	P_R^a	I_R^b	Δ_{max}^c	R_{max-d}^d	Δ_{Rmax}^e	R_{f-d}^f	Δ_{Rf-d}^g	K^h	Strain ($\times 10^{-4}$)			
									ϵ_{f-T}^i	$\epsilon_{f-FRP-UD}^j$	$\epsilon_{f-FRP-BD}^k$	ϵ_{f-C}^l
	(kPa)	(kPa-ms)	(mm)	(kN)	(mm)	(kN)	(mm)	(kN/mm)	(mm/mm)	(mm/mm)	(mm/mm)	(mm/mm)
B-[137], Avg.	53.0	561	173.2	153.6	28.4	153.6	28.4	5.9	37.6	-	-	-47.4
R1-3.1	52.2	577	96.3	156.1	26.7	53.8	96.3	6.9	39.5	88.9	-	-37.2
R1-4.1	68.7	727	36.9	216.1	36.9	216.1	36.9	6.0	49.1	51.9	-	-41.4
R2-3.1	62.3	718	84.7	198.0	32.7	119.4	72.6	6.0	42.8	61.1	-	-133.1
R2-4.1	68.8	757	285.6	208.0	34.6	52.3	67.1	6.0	-	-	-	-
R3-3.1	63.5	698	65.0	213.0	40.3	168.7	65.0	6.4	51.8	175.3	-	-113.9
R3-3.2	73.8	829	240.8	183.7	61.0	154.2	70.6	3.0	-	165.9	-	-73.6
R3-4.1	67.6	818	59.6	242.3	41.7	161.5	59.6	6.5	49.8	118.6	-	-
R3-4.2	74.4	780	233.4	182.9	58.9	182.9	58.9	3.5	-	113.4	-	-
R4-3.1	54.4	785	36.2	237.4	36.0	237.4	36.0	7.2	-	51.6	-	-222.3
R4-3.2	59.7	1,037	168.6	295.7	46.6	295.7	46.6	7.0	53.5	80.9	-	-118.9
R4-4.1	76.2	1,110	64.6	274.6	43.5	252.1	56.7	6.6	47.4	69.3	-	-
R5-A	76.3	1,091	218.0	241.2	38.2	120.6	60.0	6.3	42.8	145.0	143.9	-164.3
R5-B	71.5	1,092	156.5	242.9	37.7	121.5	128.0	6.5	45.5	-	207.8	-69.9
R6-A	74.6	1,153	115.5	246.0	36.0	123.0	115.5	7.4	43.2	153.7	152.7	-54.7
R6-B	65.7	1,170	138.8	234.3	38.9	117.2	122.0	6.4	47.7	-	251.0	-59.7
^a Reflected pressure				^b Reflected impulse				^c Maximum deflection				
^d Maximum dynamic resistance				^e Deflection at maximum resistance				^f Ultimate resistance				
^g Displacement at ultimate resistance				^h Stiffness				ⁱ Strain at wood tensile rupture				
^j Unidirectional FRP tensile failure strain				^k Bi-directional FRP tensile failure strain				^l Maximum wood compressive strain				

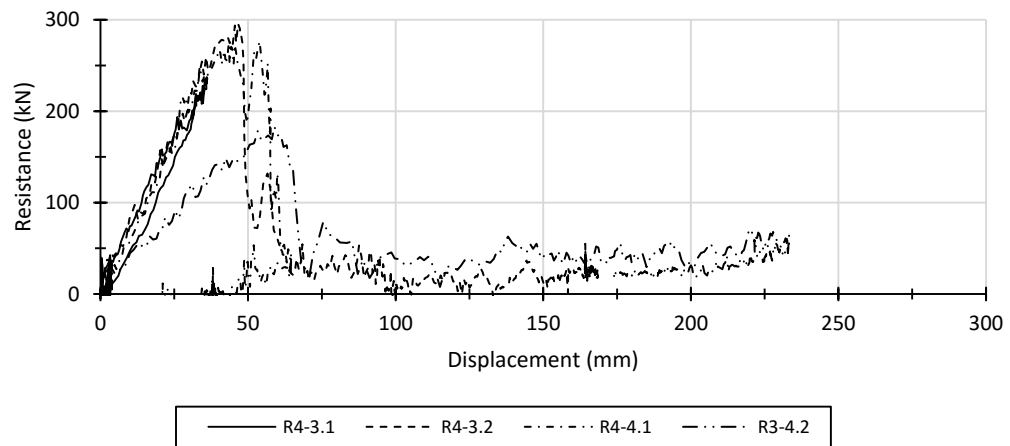
Figures 4.26 and 4.27 show the dynamic resistance curves obtained for Retrofits 1 through 4 and Retrofits 5 through 6, respectively. In general, it can be observed that significantly greater deflections were experienced by the specimens during the dynamic testing compared to the static testing. Due to the noise in the dynamic post-peak region, the determination of the ultimate failure point was based on average value.



(a) Retrofits 1 and 2



(b) Retrofit 3



(c) Retrofit 4

Figure 4.26: Dynamic resistance curves for Retrofits 1 through 4

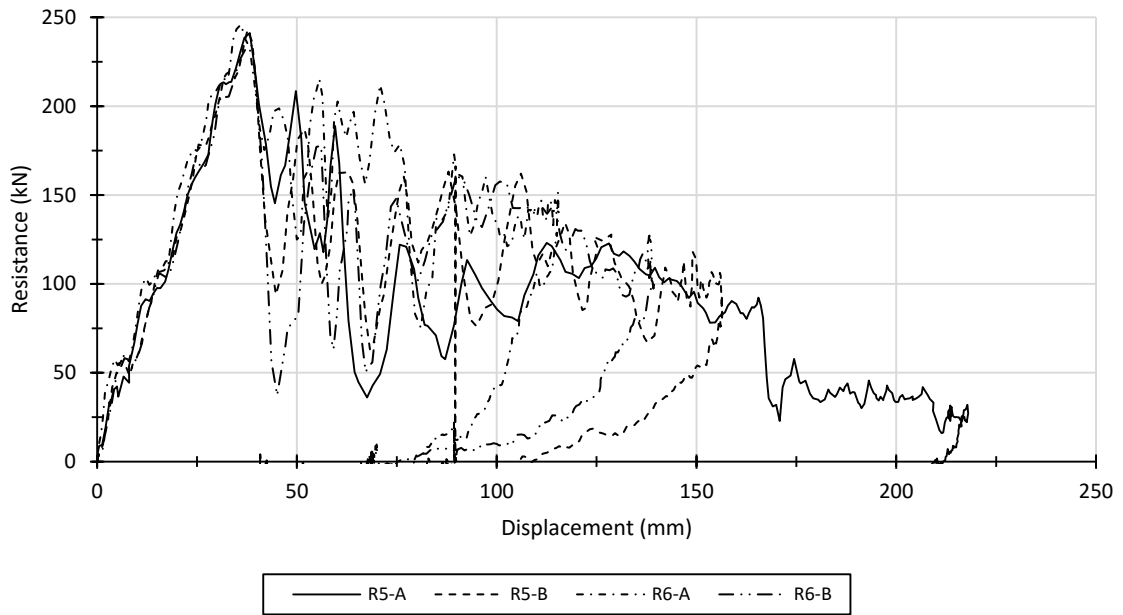


Figure 4.27: Dynamic resistance curves of Retrofits 5 and 6

The dynamic test results for Retrofits 7, 8, and 9 along with the average for the unretrofitted B-[86] beams tested dynamically are presented in Table 4.9. Figure 4.28 shows the dynamic resistance curves for Retrofits 7, 8, and 9.

Table 4.9: Dynamic test results for B-[86] retrofitted beams

Specimen	P_R^a	I_R^b	Δ_{max}^c	R_{max-d}^d	Δ_{Rmax}^e	R_{f-d}^f	Δ_{RF-d}^g	K^h	Strain ($\times 10^{-4}$)			
	(kPa)	(kPa-ms)	(mm)	(kN)	(mm)	(kN)	(mm)	(kN/mm)	ϵ_{f-T}^i	$\epsilon_{f-FRP-UD}^j$	$\epsilon_{f-FRP-BD}^k$	ϵ_{f-C}^l
B-[86], Avg.	55.7	631	22.0	182.3	22.0	182.3	22.0	8.3	35.8	-	-	-45.6
R7-A	68.3	1,029	283.5	270.6	21.3	109.1	121.7	9.5	37.8	250.7	50.9	-98.7
R7-B	70.6	1,166	64.6	234.2	30.4	181.6	64.5	10.8	53.3	124.1	130.1	-47.3
R8-A	67.8	1,157	176.2	274.2	26.3	124.3	83.9	8.8	37.2	205.6	150.2	-
R8-B	92.2	1,206	100.4	239.5	30.1	80.7	100.4	8.5	42.3	-	251.6	-62.1
R9-A	67.6	1,074	113.4	234.3	28.3	110.2	113.4	9.6	46.4	65.2	47.5	-146.6
R9-B	78.5	1,139	61.3	223.4	26.7	193.9	61.3	8.7	44.9	206.5	165.8	-182.8

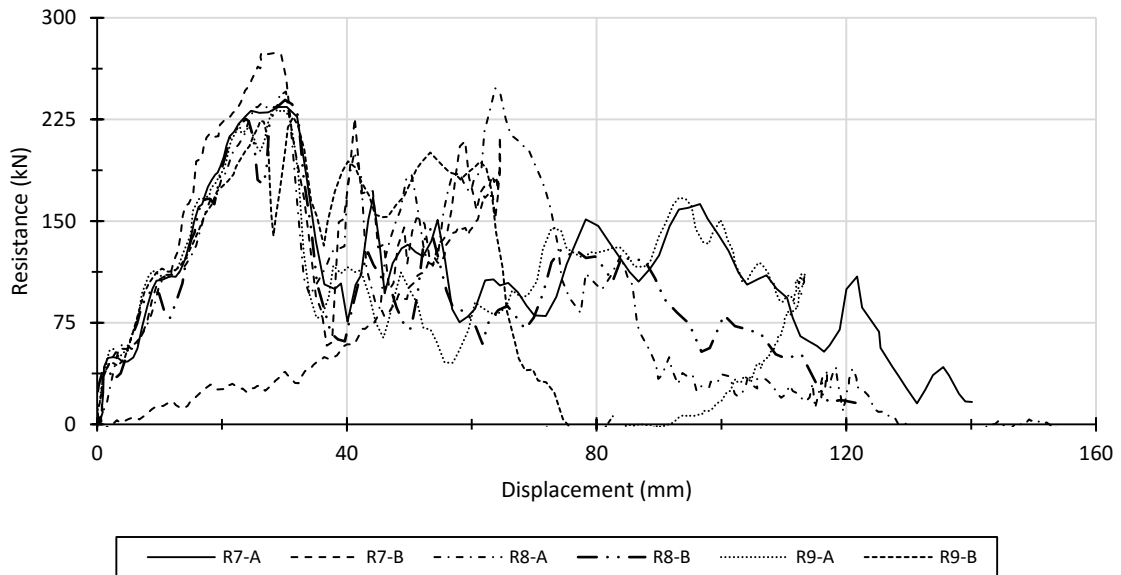
^aReflected pressure^bReflected impulse^cMaximum recorded deflection^dMaximum dynamic resistance^eDeflection at maximum resistance^fUltimate resistance^gDisplacement at ultimate resistance^hStiffnessⁱStrain at wood tensile rupture^jUnidirectional FRP tensile failure strain^kBi-directional FRP tensile failure strain^lMaximum wood compressive strain

Figure 4.28: Dynamic resistance curves of Retrofits 7, 8, and 9

The dynamic test results for the restored and retrofitted beams are presented in Table 4.10 along with the average of the unretrofitted B-[137] beams for comparison. The dynamic resistance curves of the restored and retrofitted beams are shown in Figure 4.29.

Table 4.10: Dynamic test results for the restored and retrofitted beams

Specimen	P_R (kPa)	I_R (kPa-ms)	Δ_{max} (mm)	R_{max-d} (kN)	Δ_{Rmax} (mm)	K (kN/mm)
B-[137], Avg.	53.0	561	173.2	153.6	28.4	5.9
D1.1	48.6	475	121.1	67.6	19.5	3.4
D2.1	41.2	419	76.3	77.0	23.7	4.1
D3.1	58.6	521	160.7	93.1	42.8	3.6

^aReflected pressure

^bReflected impulse

^cMaximum displacement

^dMaximum dynamic resistance

^eDeflection at maximum resistance

^fStiffness

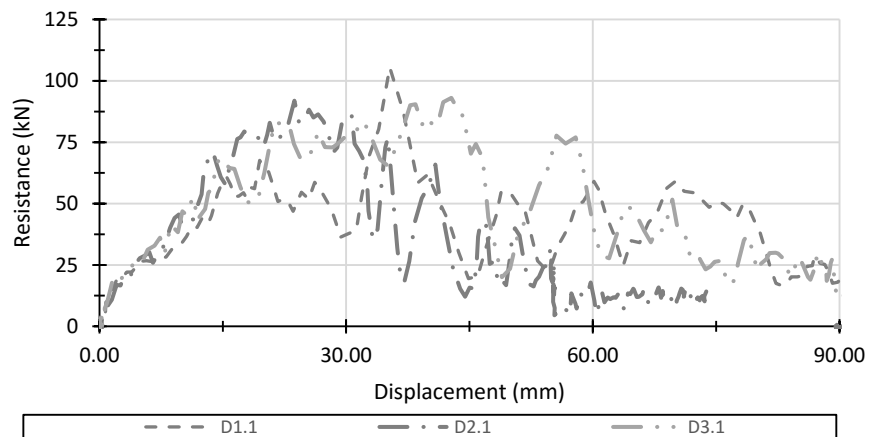


Figure 4.29: Comparison of resistance curve of restored beams to unretrofitted beam

More details on the static and dynamic test results for retrofitted beams can be found in Appendix E.

CHAPTER 5 - Analytical Model

5.1 General

The following sections describe the methodology used to develop the material predictive model. The material properties adjusted for size effects and their respective stress-strain profile used for moment-curvature analysis are presented and discussed in details. Finally, the methodologies used for the development of the predicted resistance curves for both unretrofitted and retrofitted beams and implementation of variable axial load are presented.

5.2 Constitutive Material Relationship

The idealized stress-strain relationship for clear wood was originally developed by Bazan (1980), and modified by Buchanan (1990) to reflect in-grade lumber which contains strength-reducing defects. The behaviour of wood can be assumed linear elastic in tension with failure occurring at corresponding stress and strain of f_t and ε_t . The behaviour in compression can be simplified with a linear response up to the compression yielding stress f_c , and corresponding strain, ε_{cy} , followed by a linear falling branch, defined by a slope $m \cdot E$, where E is the modulus of elasticity in the longitudinal direction (Buchanan, 1990). While the values defining the stress-strain relationship can be determined from the literature (Barrett and Lau, 1994) and provide reasonable results (Gentile, 2000; Lacroix and Doudak, 2016), these values can also be determined experimentally from coupon tests as was done in the current study.

Due to size effect dependency, the experimental compression and tension strength results obtained from the coupon tests (Section 4.2.1) were modified for length and depth effects using Equation [5.1] and [5.2], respectively (Madsen and Buchanan, 1986; Madsen and Tomoi, 1991; Barrett and Lau, 1994; Gentile, 2000). Although size effects are generally assumed to only relate to brittle failure, compression strength has also been shown to be affected by size effects, but with different size effect parameters (k) than for the tension strength (Buchanan, 1984; Buchanan, 1990; Barrett and Lau, 1994; Gentile et al., 2002). The theory is based on the presence of greater probability that a critical defect is found in a larger member resulting in a decrease in the member unit strength.

$$f_c = f_{c,exp} \left(\frac{L_{c,exp}}{L_{ec}} \right)^{1/k_1} \left(\frac{d_{c,exp}}{d} \right)^{1/k_2} \quad [5.1]$$

$$f_t = f_{t,exp} \left(\frac{L_{t,exp}}{L_{et}} \right)^{1/k_1} \left(\frac{d_{t,exp}}{d} \right)^{1/k_2} \quad [5.2]$$

where f_c is the adjusted compressive material model input yield strength, f_t is the ultimate tension strength, L_{ec} is the equivalent length in compression, L_{et} is the equivalent stressed length in tension, and d is the average depth of the layers composing the glulam section. $f_{c,exp}$ is the average compressive yield stress obtained from the coupon tests, $f_{t,exp}$ is the average ultimate tension stress obtained from the coupons, $L_{c,exp}$ is the length of the compression coupon specimens, $L_{t,exp}$ is the length of the tension coupon specimens, $d_{c,exp}$ is the depth of the compression coupon specimens, $d_{t,exp}$ is the depth of the tension coupon specimens, k_1 is the size effect parameter for length, and k_2 is the size effect parameter for depth.

Values for the length effect parameter k_1 of 10.00 and 5.88 were obtained from Barrett and Lau (1994) for compression and tension, respectively. The compressive material model yield strength input was not modified for depth effects since the coupons' cross-section consisted of actual laminates from the specimens that were not reduced in size. The ultimate tensile strength, f_t , was modified for depth effects through a factor k_2 of 4.19 for the B-[137] coupons and 4.76 for the B-[86] coupons. The equivalent stressed lengths for the tension and compression laminates were determined using Equation [5.3].

$$L_e = \frac{1 + \frac{a_1 k_1}{L}}{k_1 + 1} L \quad [5.3]$$

where L_e is the equivalent stressed length, L is the span of the beam, a_1 is the distance between the two point loads and k_1 is the length effect parameter.

Table 5.1 presents the model input parameters using values from the coupon tests. The resulting material stress-strain constitutive relationship for static loading can be developed as shown in Figure 5.1

Table 5.1: Model input strengths adjusted for size effects

Property	B-[86]		B-[137]	
	Compression	Tension	Compression	Tension
Exp. strength, $f_{c,exp} / f_{t,exp}$ (MPa)	40.6	63.0	48.6	79.6
Coefficient of variation	0.06	0.18	0.11	0.27
Exp. length, $L_{c,exp} / L_{t,exp}$ (mm)	200	254	200	254
Length effect parameter, k_1	10.00	5.88	10.00	5.88
Depth effect parameter, k_2	-	4.76	-	4.19
Equivalent stressed length, L_e (mm)	880.5	961.5	880.5	961.5
Average depth of layers, d (mm)	-	44.2	-	44.4
Input model strength (MPa)	35.0	35.6	41.9	43.3

The failure strain for the tension stress-strain relationship was determined by dividing the input tension strength by the average modulus of elasticity presented in Table 4.1. Furthermore, the compressive failure point was defined by using the resulting falling branch ratio from the average experimental compressive yield and ultimate strengths as well as the average failure strain.

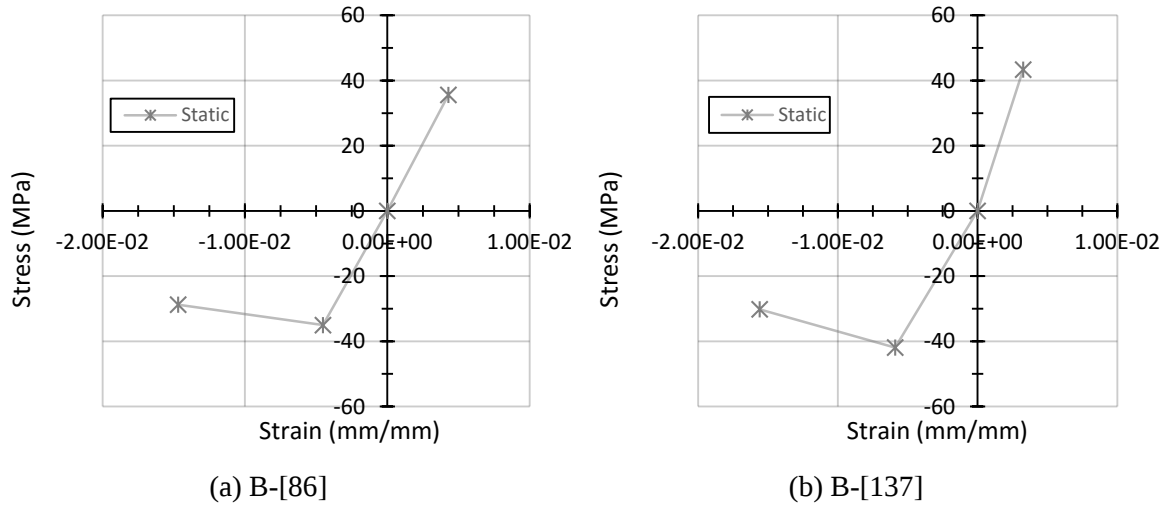


Figure 5.1: Wood input stress-strain relationship

The FRP stress-strain behaviours used as input in the modelling of the various layups were obtained experimentally as discussed in Chapter 4 and are shown in Figure 5.2. Figures 5.2a and 5.2b represent the FRP properties used for Retrofits 1 through 4 and 5 through 9, respectively.

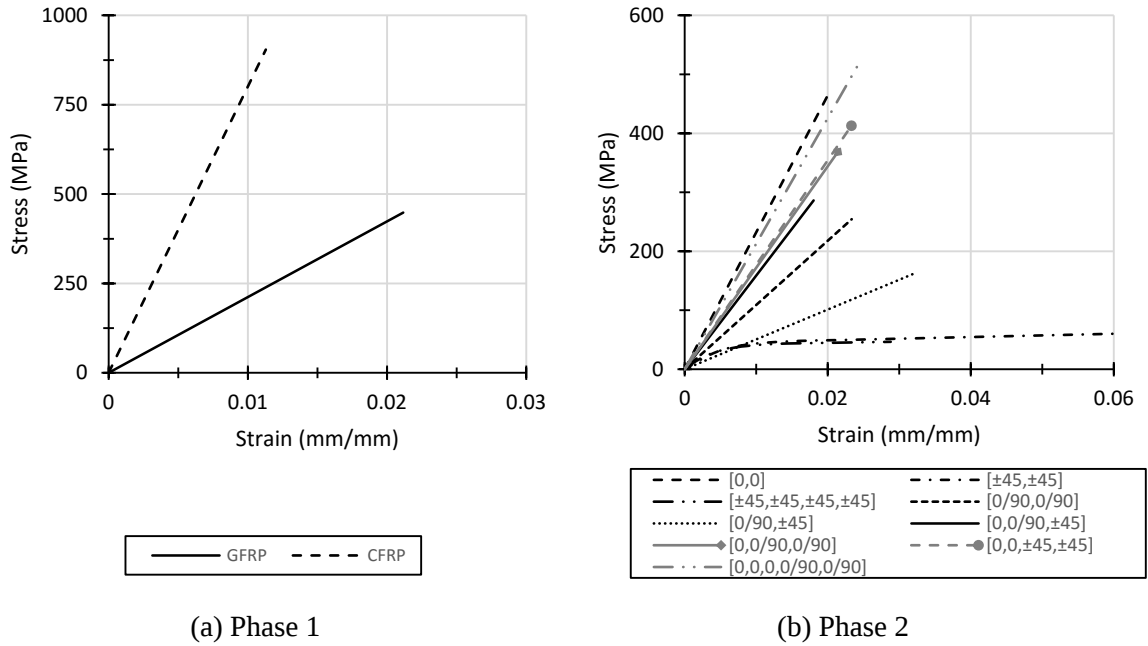


Figure 5.2: FRP input stress-strain relationship

5.3 Moment Curvature Analysis

5.3.1 Unretrofitted beams and columns

The bending behaviour of a timber beam-column can be described by conducting a moment-curvature analysis using the constitutive material relationship established in the previous section. The constitutive stress-strain relationship for wood originally developed by Buchanan (1990) has since been successfully used by several researchers to predict the flexural capacity for static applications (Plevris and Triantafillou, 1992; Gentile et al., 2002; Yang et al., 2016). Figure 5.3 shows the strain and stress profiles, and the resulting forces for a rectangular cross-section subjected to an axial force, P , applied concentrically in combination with bending for two separate cases: Case 1-Elastic, and Case 2-Inelastic.

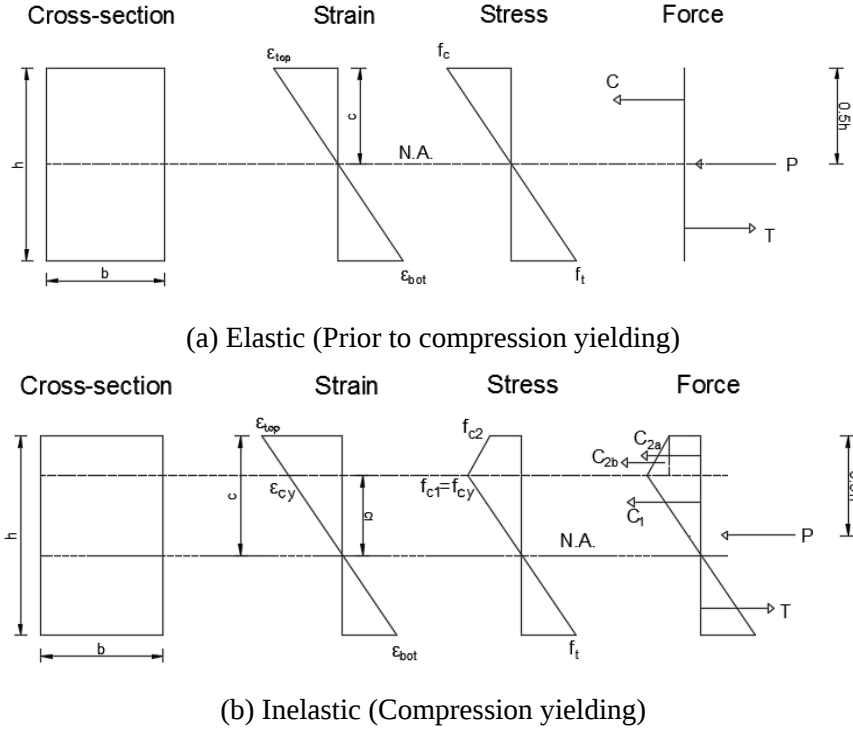


Figure 5.3: Strain, stress, and force diagrams for a rectangular wood cross-section

For Case 1 (Fig. 5.3a), where wood yielding in compression has not been reached, the compression and tension forces can be determined using Equations [5.4] and [5.5], respectively, and the resulting moment about the mid-height of the cross-section is determined using Equation [5.6].

$$C = 0.5E_{wc}\varepsilon_{top}cb \quad [5.4]$$

$$T = 0.5E_{wt}\varepsilon_{bot}(h - c)b \quad [5.5]$$

$$M_{h/2} = C_1 \left\{ \frac{h}{2} - \frac{c}{3} \right\} + T \left\{ \frac{h}{6} + \frac{c}{3} \right\} \quad [5.6]$$

where E_{wc} and E_{wt} are the modulus of elasticity of wood in compression and tension, ε_{top} and ε_{bot} are the top and bottom strain, c the compression zone depth, b the width of the section, and h is the height of the section.

Similarly, the forces acting on the cross-section and the resulting moment for Case 2 (Fig. 5.3b), where wood in compression has yielded, can be determined using Equations [5.7] to [5.11]

$$C_1 = 0.5f_{cy} \left(\frac{\varepsilon_{cy}}{\varepsilon_{top}} \right) cb \quad [5.7]$$

$$C_{2a} = [\varepsilon_{cy} - m(\varepsilon_{top} - \varepsilon_{cy})] \left[1 - \frac{\varepsilon_{cy}}{\varepsilon_{top}} \right] E_{wc} cb \quad [5.8]$$

$$C_{2b} = 0.5[mE_{wc}(\varepsilon_{top} - \varepsilon_{cy})] \left[1 - \frac{\varepsilon_{cy}}{\varepsilon_{top}} \right] cb \quad [5.9]$$

$$T = 0.5E_{wt}\varepsilon_{bot}(h - c)b \quad [5.10]$$

$$M_{h/2} = C_1 \left\{ \frac{h}{2} + \frac{2c_1}{3} - c \right\} + C_{2a} \left\{ \frac{h}{2} - \frac{c}{2} + \frac{c_1}{2} \right\} + C_{2b} \left\{ \frac{h}{2} - \frac{2c}{3} + \frac{2c_1}{3} \right\} + T \left\{ \frac{h}{6} + \frac{c}{3} \right\} \quad [5.11]$$

where C_1 , C_{2a} , and C_{2b} are the compression forces and T is the tension force acting over the section.

When deriving a moment-curvature relationship of a wood member, a phenomenon referred to as the stress distribution effect needs to be considered (Buchanan, 1984; Madsen and Buchanan, 1986; Buchanan, 1990). In bending, the stress at the extreme tension fibre is greater than that of axial tension alone. The extreme tension fibre stress, f_m , at which failure occurs in bending can be obtained using Equation [5.12].

$$f_m = \left[\frac{h - c}{h(1 + k_3)} \right]^{-1/k_3} f_t \quad [5.12]$$

where h is the height of the cross-section, c is the depth of the neutral axis from the compressive face, k_3 accounts for the variability in strength properties within the member's cross-section height, and f_t is the tension strength of the member.

Due to the failure stress being a function of the neutral axis (NA) position, a closed-form solution is not possible and a program was developed to generate the moment-curvature relationships for rectangular cross-sections. For a defined material stress-strain relationship and a given axial load, the bottom tensile strain is increased until failure is reached. Failure is defined as the level where a) the stress at the extreme tension fibre exceeds the maximum allowable stress as defined in Equation [5.12], b) the ultimate compression strain is exceeded, or c) the equilibrium of the section cannot be reached. Figures 5.4a and 5.4b show examples

of the output from the moment-curvature analysis routine for the case of no axial load and when a compressive axial load of 300 kN is applied. Also shown in the figures is the compression zone depth as a function of curvature.

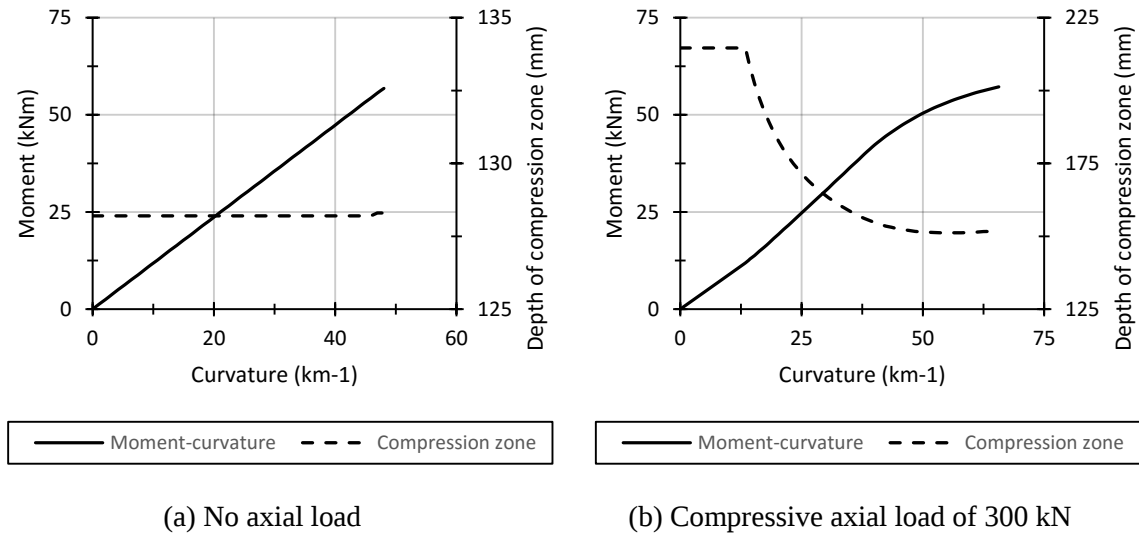


Figure 5.4: Representative moment-curvature from program for specimen B-[137]

5.3.2 FRP retrofitted beams

Similar to the case of the unretrofitted beams, a program was developed to generate the moment-curvature relationships of FRP reinforced glulam cross-sections. Figure 5.5 shows the strain, stress, and resulting forces for a FRP U-shaped reinforcement configuration of a rectangular cross-section for two separate cases: Case 1-Elastic, and Case 2-Inelastic.

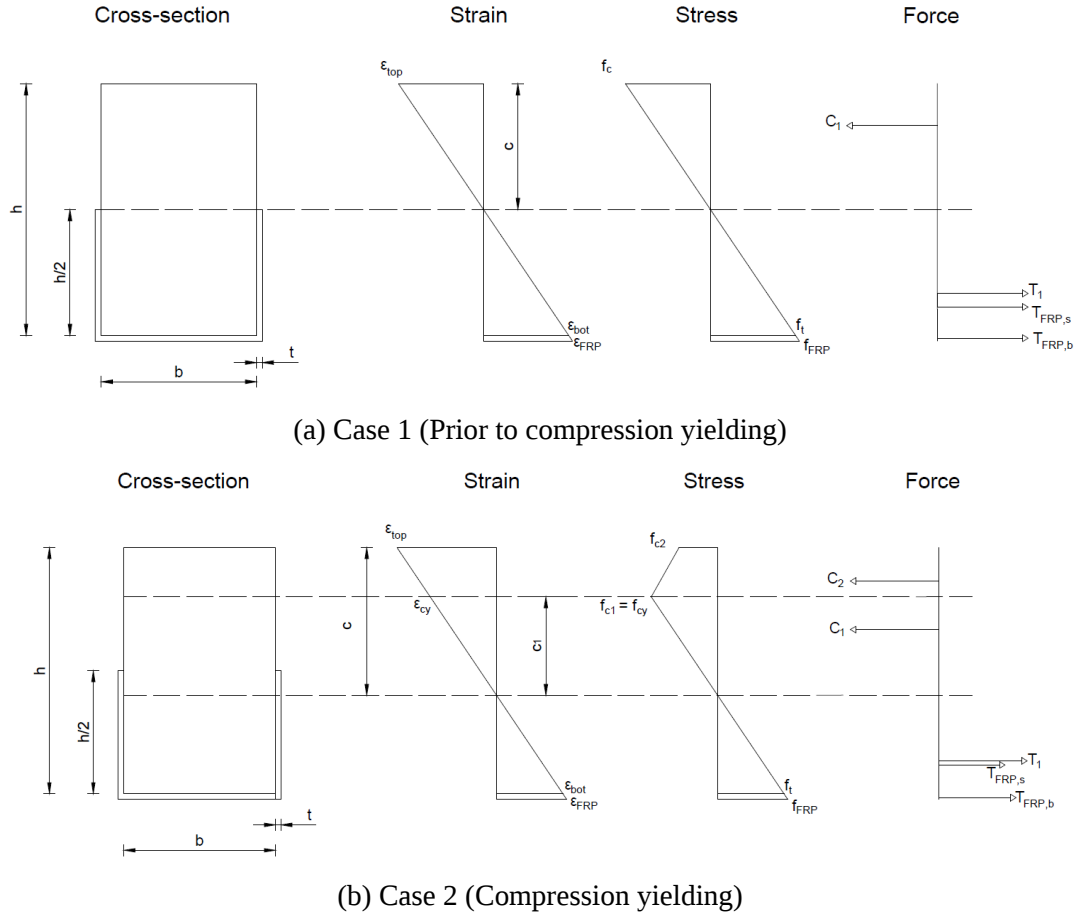


Figure 5.5: Strain, stress, and force diagrams for a rectangular wood cross-section with U-shaped FRP

For the elastic case, where wood in compression has not yet yielded (Fig. 5.5a), the wood tension and compression forces can be determined using Equations [5.13] and [5.14], respectively. The FRP tension forces can be determined using Equation [5.15], and the resulting moment about the mid-height of the cross-section is found using Equation [5.16]. These equations are developed based on static equilibrium for the convention presented in Figure 5.5.

$$C_1 = 0.5E_{wc}\epsilon_{top}cb \quad [5.13]$$

$$T_1 = 0.5E_{wt}\epsilon_{bot}(h - c)b \quad [5.14]$$

$$T_{FRP} = T_{FRP,b} + T_{FRP,s} \quad [5.15]$$

$$T_{FRP} = 0.5E_{FRP}(\epsilon_{bot} + \epsilon_{FRP})t_{FRP}b + E_{FRP}\epsilon_{FRP}t_{FRP}(h - c + t_{FRP})$$

$$M_{h/2} = C_1 \left\{ \frac{h}{2} - \frac{c}{3} \right\} + T \left\{ \frac{h}{6} + \frac{c}{3} \right\} + T_{FRP,b} \left\{ \frac{h + t_{FRP}}{2} \right\} + T_{FRP,s} \left\{ \frac{h}{6} + \frac{c}{3} \right\} \quad [5.16]$$

where E_{Wc} and E_{Wt} are the modulus of elasticity of wood in compression and tension, E_{FRP} is the FRP modulus of elasticity, $T_{FRP,b}$ is the force in the bottom tension portion of the FRP, $T_{FRP,s}$ is the force in the FRP from bottom to mid-height, ε_{top} and ε_{bot} are the top and bottom strain, ε_{FRP} is the strain in the outermost FRP layer, c the compression zone depth, t_{FRP} is the thickness of the FRP at both the bottom and side, b the width of the section, and h is the height of the section.

For the inelastic case, where wood in compression has yielded (Fig. 5.5b), the forces acting on the cross-section and the resulting moment can be determined using Equations [5.17] to [5.21].

$$C_1 = 0.5E_{Wc}\varepsilon_{top}cb \quad [5.17]$$

$$C_2 = \left[1 - \frac{\varepsilon_{cy}}{\varepsilon_{top}} \right] \left[\frac{1}{m} - \frac{\varepsilon_{top}}{2\varepsilon_{cy}} + \frac{1}{2} \right] m f_c cb \quad [5.18]$$

$$T_W = 0.5E_{Wt}\varepsilon_{bot}(h - c)b \quad [5.19]$$

$$T_{FRP} = T_{FRP,b} + T_{FRP,s} = 0.5E_{FRP}(\varepsilon_{bot} + \varepsilon_{bot})t_{FRP}b + E_{FRP}\varepsilon_{FRP}t_{FRP}(h - c + t_{FRP}) \quad [5.20]$$

$$M_{h/2} = C_1 \left\{ \frac{h}{2} + \frac{2c_1}{3} - c \right\} + C_2 \left\{ \frac{h}{2} - c \left[1 - \frac{\varepsilon_{cy}}{\varepsilon_{top}} \right] + \frac{c \left[1 - \varepsilon_{cy}/\varepsilon_{top} \right] [2f_{c2} + f_c]}{3[f_{c2} + f_c]} \right\} \quad [5.21]$$

$$+ T_{FRP,b} \left\{ \frac{h + t_{FRP}}{2} \right\} + T_{FRP,s} \left\{ \frac{h}{6} + \frac{c}{3} \right\}$$

where C_1 and C_2 are the compression forces and T is the tension force acting over the section.

For the case where a simple tension reinforcement scheme is provided, $T_{FRP,s}$ is set to 0.

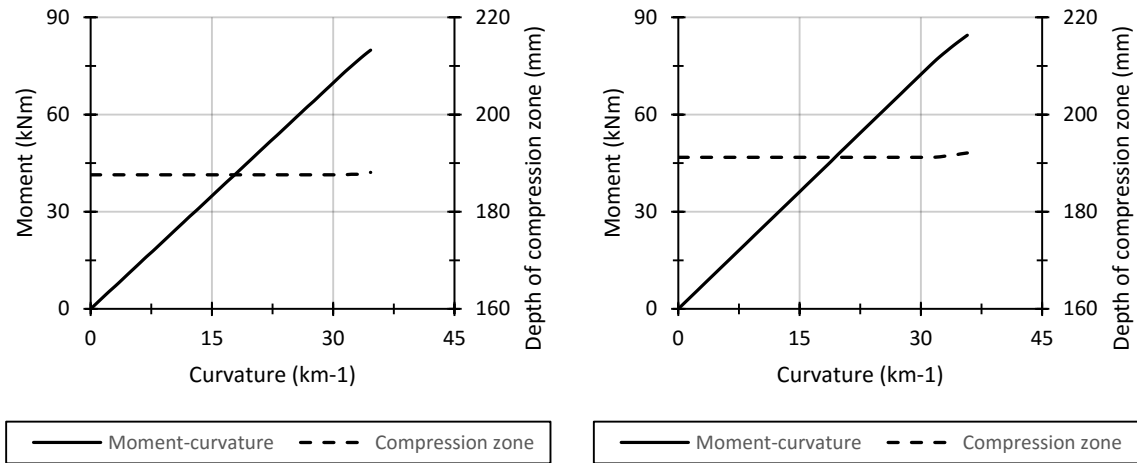
As mentioned in the previous section, the stress at the extreme tension fibres in bending is greater than that of axial tension alone (stress distribution effect). For the unretrofitted beams, this effect was accounted for through Equation [5.12]. Adapted from Buchanan (1990) for

the convention presented in Figure 5.5, the extreme tension fibre stress, $f_{m,r}$, at which failure of the FRP reinforced section occurs in bending can be obtained using Equation [5.22].

$$f_{m,r} = \alpha_m \left[\frac{h - c}{h(1 + k_3)} \right]^{-1/k_3} f_t \quad [5.22]$$

where α_m is the modification factor introduced by Gentile et al. (2002) to account for the enhancement of the bending strength due to the added FRP reinforcement, h is the height of the cross-section, c is the depth of the neutral axis from the compressive face, k_3 accounts for the variability in strength properties within the depth of a member, and f_t is the tension strength of the member.

The developed routine used for the unretrofitted beams and columns was modified to incorporate the effects of FRP. Failure in the moment-curvature analysis of the FRP reinforced specimens was defined as the level where a) the stress at the extreme tension fibre exceeds the maximum allowable stress (defined in Eq. [5.22]), b) the ultimate compression strain is exceeded, c) FRP has ruptured, or d) equilibrium of the section cannot be reached. Figures 5.6a and 5.6b show the resulting moment-curvature relationship obtained for specimen B-[137] with two layers of unidirectional GFRP reinforcement and two layers of U-shaped unidirectional GFRP reinforcement, respectively.



(a) Simple tension reinforcement

(b) U-shaped tension reinforcement

Figure 5.6: Representative moment-curvature from program for FRP reinforced specimen

5.4 Development of Resistance Curves

In blast engineering, the resistance of the component is required as a function of displacement, which is also referred to as resistance curve. The following sections describe the general methodology used to generate the resistance curves for beams and columns and include the effects of axial load and FRP reinforcement.

5.4.1 Constant or no axial load

A routine was developed to determine the resistance curves based on the previously generate moment-curvature relationship. Input parameters include: loading distribution, span, number of elements discretizing the specimen, and the curvature increment. The routine applies a mid-span curvature based on the specified increment and determines the resulting moment from the moment-curvature relationship. The curvature is then distributed along the discretized specimen's length using the moment distribution pertaining to the loading type (i.e. four point bending or uniformly distributed) based on the previously generated moment-curvature relationship. Curvatures are integrated twice to initially obtain rotation along the member's length and ultimately the displacement. In the presence of an axial load, a second-order analysis is performed to account for the P-delta effects until convergence is attained. Based on the displaced shape of the specimen, load-mass factors (K_{LM}) are calculated at each

lateral displacement and used in the SDOF analysis. Figure 5.7 shows an example of a typical output of the routine for an unretrofitted cross-section.

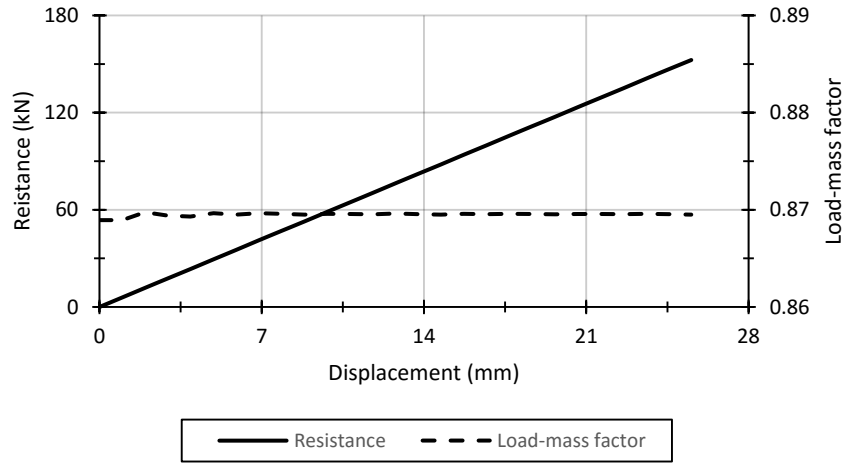


Figure 5.7: Typical resistance curve for unretrofitted specimen

5.4.2 Variable axial load

As discussed in Chapter 4, a loss in axial load was measured as the columns displaced laterally, and thus using a resistance curve with constant axial load is not representative of the actual behaviour observed in the laboratory conditions. A methodology was therefore developed to account for the loss of axial load during the out-of-plane response of the columns. From Figure 5.8a, it can be seen that the total length of the column, L_{column} , exceeds that of the vertical span, L_{span} , and that the load is applied concentrically (Figs. 5.8a and 5.8d). The axial load was applied prior to securing the lateral supports, thereby compressing the column by an initial deflection, Δ_{Pc0} (Fig. 5.8b).

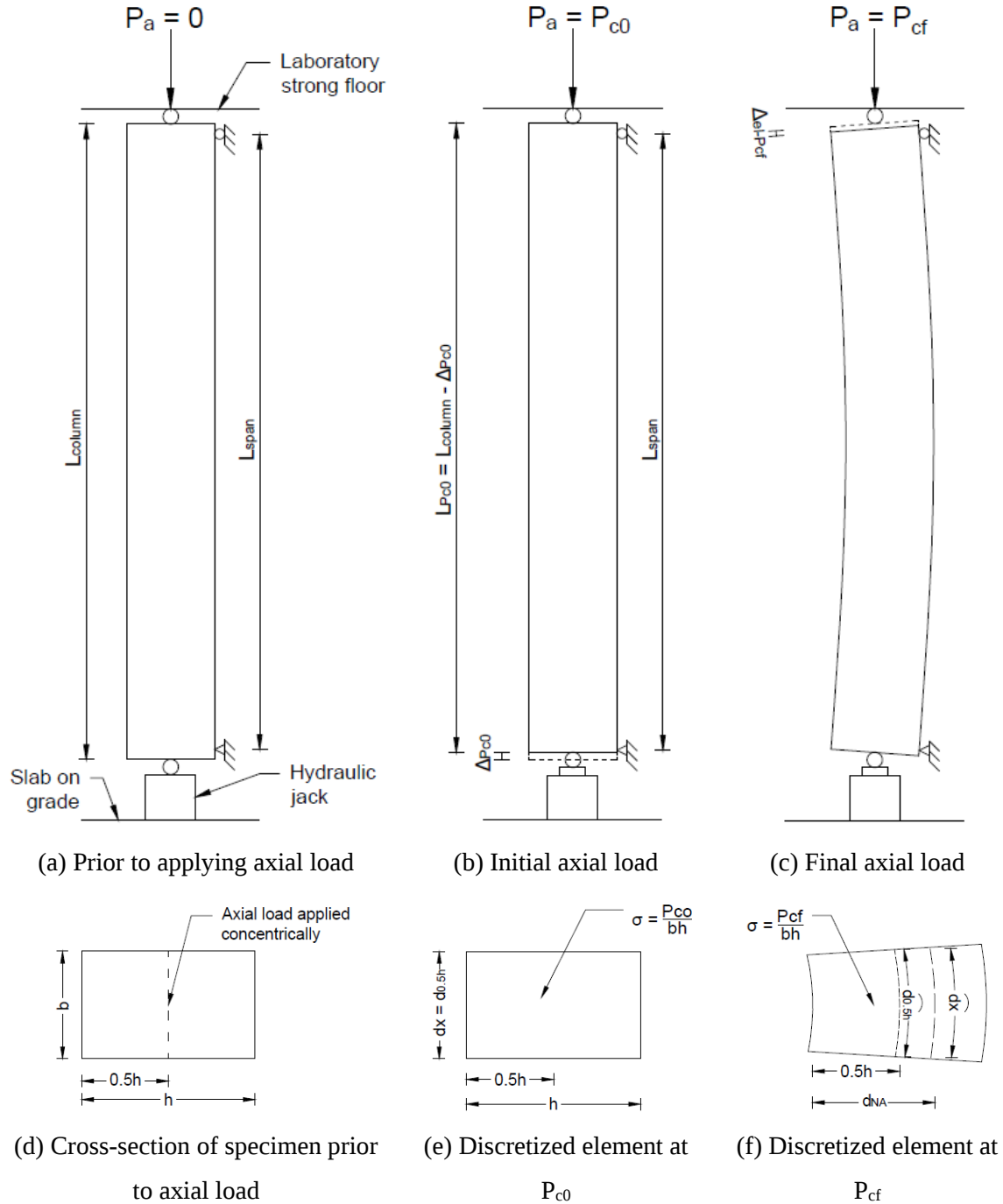


Figure 5.8: Details of projected shortening and elongation

When the column starts displacing laterally (Fig. 5.8d), the neutral axis shifts from the mid-height of the cross-section where the load is applied, resulting in projected shortening at the axial load application point. The arc length along the neutral axis will always remain equal to that of the vertical supported length (e.g. L_{span}), however, above and below the neutral

axis, shortening and elongation, respectively, will occur. Figures 5.8e and 5.8f show an element at mid-span of length dx used to discretize the column's total length. For this analysis, a total of 150 elements were used to discretize the column's length. It can be seen that as the column displaces laterally, there will be a progressive shortening at the axial load application point resulting in a loss of axial load. The applied axial load becomes zero when the column decompresses with an amount Δ_{el-Pf} equal to the initial compressed deflection, Δ_{Pc0} . Similar approaches have also been used in the literature (Saatcioglu et al., 2011; Burrell et al., 2015; Kadhon, 2015; Lloyd, 2015).

The initial compressed displacement, Δ_{Pc0} , can be approximated using Equation [5.23].

$$\Delta_{Pc0} = \frac{P_{c0} L_{total}}{EA} \quad [5.23]$$

where P_{c0} is the initial applied axial load, L_{total} is the length of the column, E_{wc} is the modulus of elasticity, and A is the area of the column's cross-section.

A composite resistance curve is generated by correlating the elongation to the resulting difference between the initial shortening and that of the axial load level under consideration. In order to generate a full composite resistance curve, several resistance curves with different constant axial load levels need to be generated with each resistance curve contributing to a single point on the composite curve. Failure was established at the point where the axial elongation equilibrium could no longer be satisfied for the next resistance curve point or when the current deflection exceeded that of the last deflection point of the subsequent axial load level resistance curve.

Figure 5.9 shows an example of a composite resistance curve for a B-[137] element using the stress-strain relationship in Figure 5.1b with an initial applied compressive axial load of 500 kN and an axial load at maximum resistance of 402 kN. In order to generate the full composite curve shown in Figure 5.9, several resistance curves with different constant axial load levels were generated. Each resistance curve contributed to one point on the composite curve.

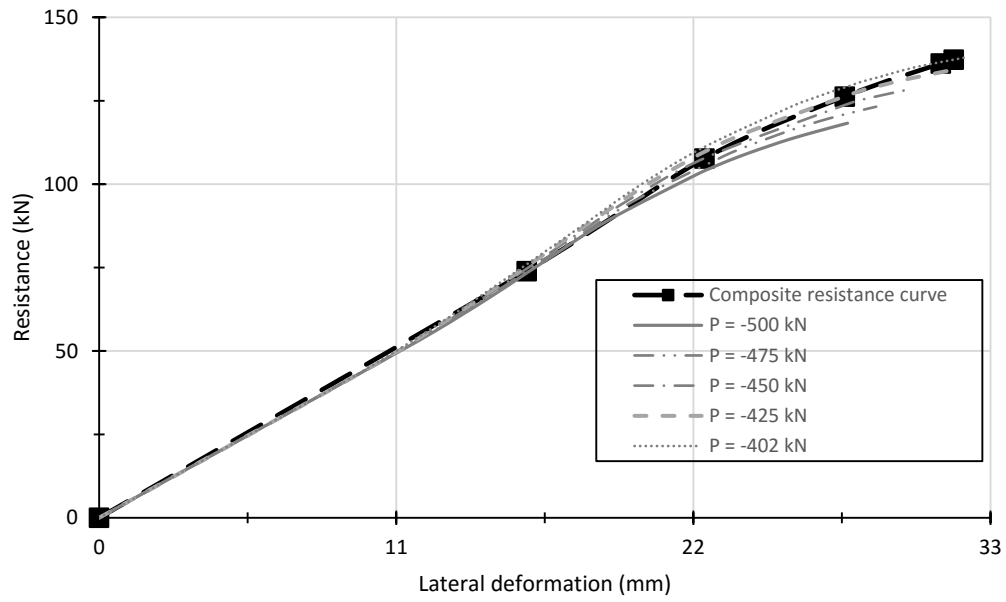


Figure 5.9: Example of composite resistance curve

CHAPTER 6 - Discussion

6.1 General

A total of seventy glulam beams and columns were tested to failure under static and dynamic loads. The experimental program was divided into four phases investigating the dynamic increase factor (DIF), effect of axial load, effect of unidirectional FRP reinforcement, as well as alternative configurations of FRP combinations. The following sections discuss the findings of these phases. The results from the proposed predictive procedure, which accounts for variables such as high strain-rates, axial load, and FRP are presented following each section.

6.2 Dynamic increase factor for glulam beams

6.2.1 Failure modes and determining the DIF

A total of thirty-eight glulam beams were tested destructively under both static and dynamic loading with the purpose of determining a dynamic increase factor (DIF). Since reactions were measured for all beams, determining a DIF for glulam beams could therefore simply be a matter of relating the dynamic modulus of rupture of each glulam beam type to their static average. However, differences in static and dynamic failure modes reported earlier had a significant impact on the individual specimens' DIF value.

Specimens with a single laminate across their width (e.g. B-[80]) and with a finger-joint (FJ) within the two load application points consistently had the failure initiated at that location under dynamic loading. More crack propagation was observed in beams with multiple FJs across their width (e.g. B-[86] and B-[137]).

The observed failures for the static and dynamic tests were grouped depending on whether they were dominated by a FJ failure and whether the FJs were continuous across the width of the member. Then, t-Tests were performed in order to establish whether there was significant difference between the various groups. Six different groups were identified, as shown in Table 6.1.

Table 6.1: Groups for t-Test analysis

Group	Description	Specimen number included in group:		
		B-[80]	B-[86]	B-[137]
1	Static tests with no continuous FJ failure	1-3; 6-11	2-4	1-4
2	Static tests with continuous FJ failure	4-5	1	-
3	Dynamic tests with no continuous FJ failure	12-18	5-7	5; 7-9
4	Dynamic tests with continuous FJ failure	19-21	8	6
5	All static tests	1-11	1-4	1-4
6	All dynamic tests	12-21	5-8	5-9

The confidence level for the two-tail t-Tests for two samples with unequal variance was chosen to be 95%. The null hypothesis (e.g. no difference between the mean of the two data sets) was rejected if the absolute value of t_{stat} was greater than the absolute value of t_{crit} . The t-Tests results are presented in Table 6.2 where bold numbers indicate that there is significant difference between the two data sets to consider them as separate groups.

Table 6.2: Results for DIF t-Test analysis

Group comparison	Dynamic increase factor	
	t_{stat}	t_{crit}
1-vs-2	0.82	± 3.18
1-vs-3	-3.99	± 2.06
2-vs-4	2.02	± 2.31
3-vs-4	7.03	± 2.26
3-vs-5	4.39	± 2.05
4-vs-5	-4.15	± 2.31
5-vs-6	-1.62	± 2.05

The results show that under static loading there was no significant difference between the resistances of the specimens that failed at a finger joint and those that did not (Group 1-vs-2). The data also indicates that under dynamic loading, failure of specimens with continuous finger joints were statistically indistinguishable from those that failed under static loading (Group 2-vs-4). This means that when the dynamic failure occurred at a finger joint, no increase in resistance was observed due to high strain-rate effects. This is consistent with finding by Nadeau et al. (1982) who reported that clear wood specimens, which were intentionally notched on the tension side, lacked the increase observed in specimens without a notch.

When considering the dynamic test results, it is clear that specimens with a continuous FJ failure had a significantly different and lower resistance than those that did not experience a continuous FJ failure (Group 3-vs-4). Finally, it is evident that specimens tested dynamically and where no continuous FJ failure was observed can be considered as a separate group when compared to the static results (Group 1-vs-3).

The first four groups of data from Table 6.1 are plotted in Figure 6.1, which shows the ratios of the beams' strength normalized to static strength of their respective group. As seen in Figure 6.1, the dynamic average excluding specimens with FJ dominated failure is 1.14 times greater than the static average of all tests. In contrast, the dynamic strength of specimens with continuous FJ failure did not have a significant increase in resistance relative to the average static resistance.

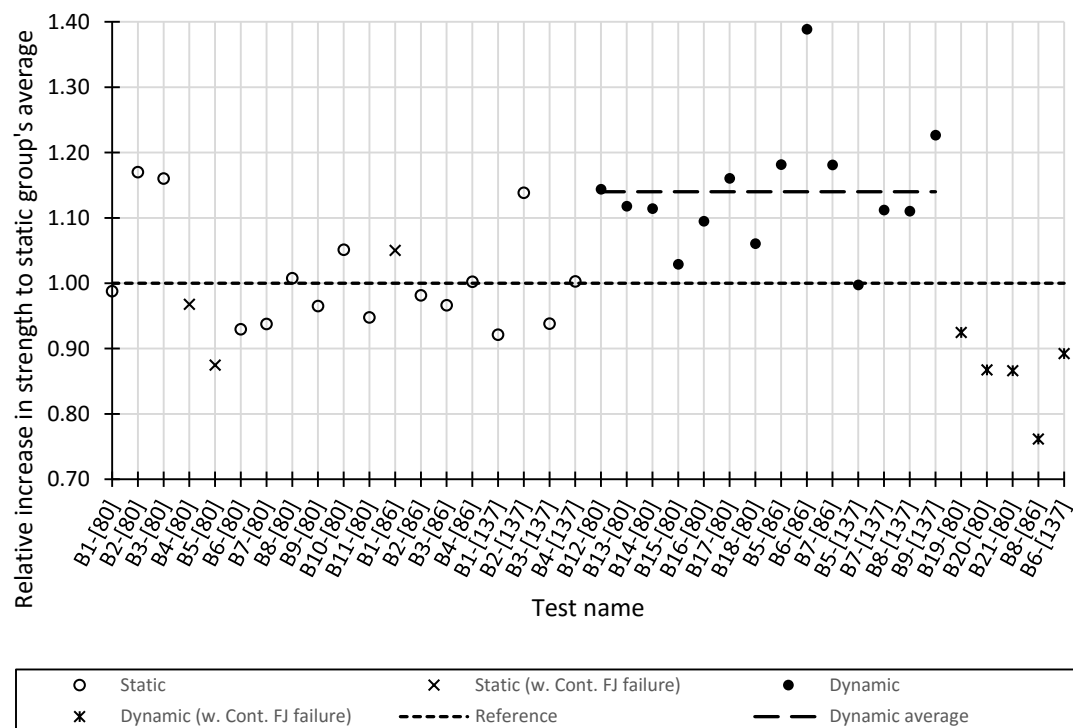


Figure 6.1: Dynamic increase factors on the resistance of glulam beams

Although an increase on the average resistance of all dynamic tests relative to the static resistance is 1.05, the t-Test results indicate that given the spread in the data there is no significant difference between the two groups (Group 5-vs-6) to justify such an increase based on the average value alone. This is further emphasized by fact that while the t-Tests

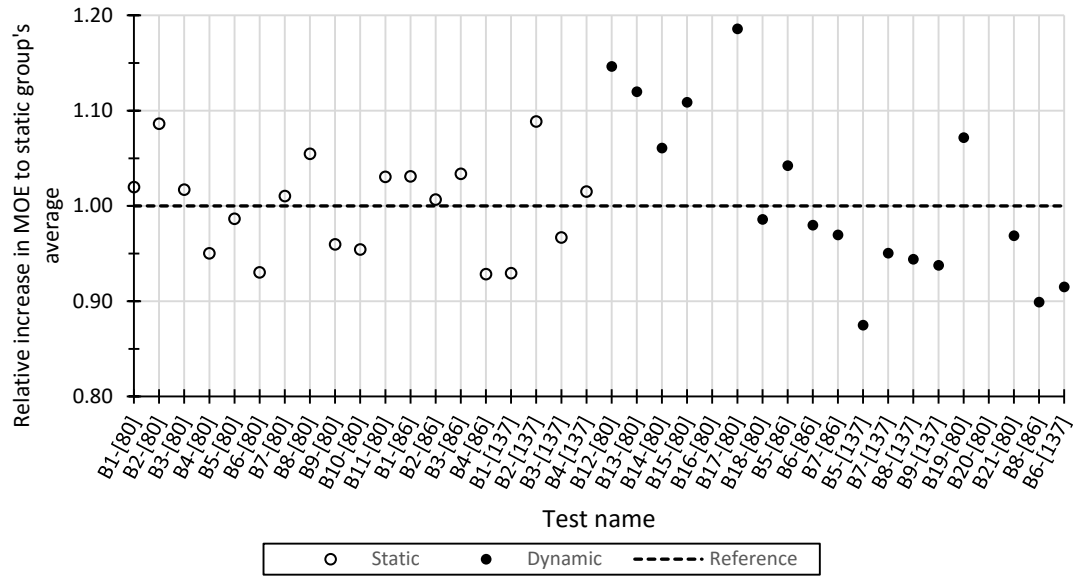
show that there is no statistical difference between the continuous and staggered static strengths (Group 1-vs-2), a comparison between the dynamic tests with continuous FJ failure and all the static tests (Group 4-vs-5) shows a significant statistical difference with a slight decrease in strength under dynamic loading (with an average dynamic to static strength ratio of 0.9). This decrease could possibly be attributed to glue in the FJ, however, this hypothesis cannot be substantiated due to the limited number of dynamic tests including continuous FJs. It is therefore suggested that more research be undertaken to investigate a larger sample size of glulam members with predetermined placement of FJs in the outer laminate.

Similar t-Tests were conducted for the stiffness and the results showed no evidence of a dynamic increase factor on the MOE and tensile failure strain as shown in Table 6.3.

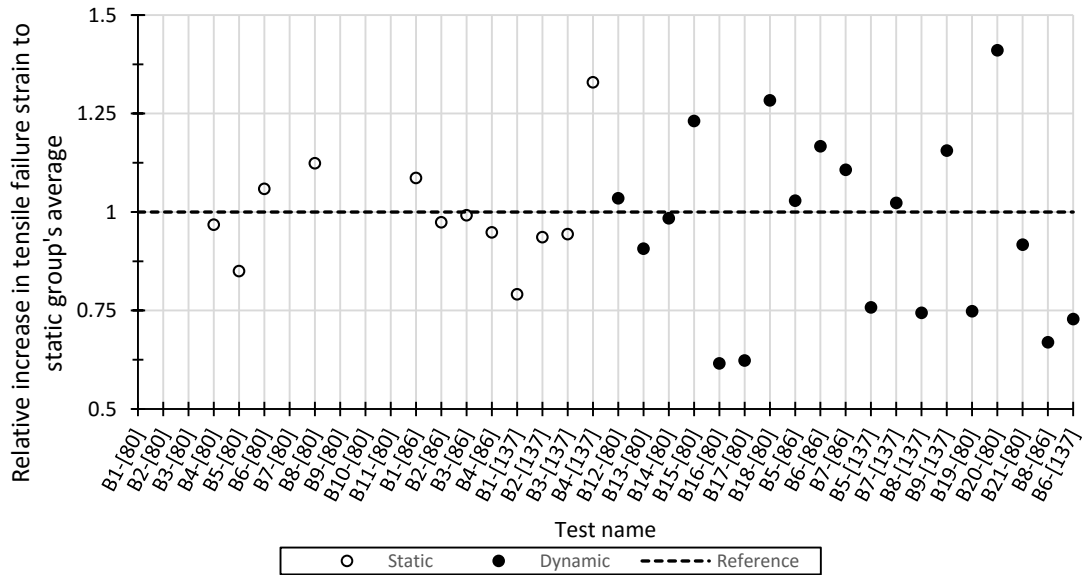
Table 6.3: Results for MOE and failure strain t-Tests

Group comparison	Modulus of elasticity		Tensile failure strain	
	t_{stat}	t_{crit}	t_{stat}	t_{crit}
1-vs-2	0.47	± 3.18	0.50	± 2.78
1-vs-3	-1.23	± 2.11	0.45	± 2.08
2-vs-4	-0.69	± 2.78	0.49	± 2.45
3-vs-4	-0.16	± 2.57	0.55	± 2.45
3-vs-5	1.31	± 2.12	-0.34	± 2.07
5-vs-6	-1.36	± 2.07	0.68	± 2.05

The lack of a clear trend is further shown in Figures 6.2a and 6.2b for the MOE and tensile failure strain, respectively.



(a) Modulus of elasticity



(b) Tensile failure strain

Figure 6.2: Comparison of MOE and tensile failure strain for unretrofitted beams

6.2.2 Predicting the behaviour of the unretrofitted beams

The static and dynamic flexural behaviour of the unretrofitted beams can be obtained by conducting moment-curvature analysis using the methodology described in Section 5.3.1 and in Appendix G. Once the moment-curvature relationship was established for the cross-

section, the flexural resistance curves of the beams were obtained using the methodology described in Section 5.4.1 and Appendix G. The dynamic resistance curves accounted for high strain rate effects described in the previous section, as shown in Figure 6.3.

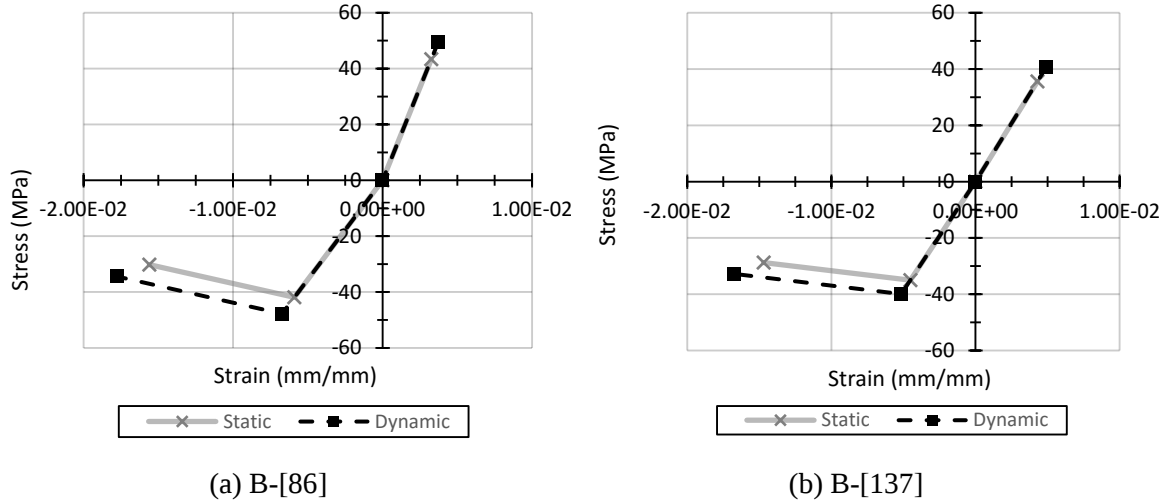
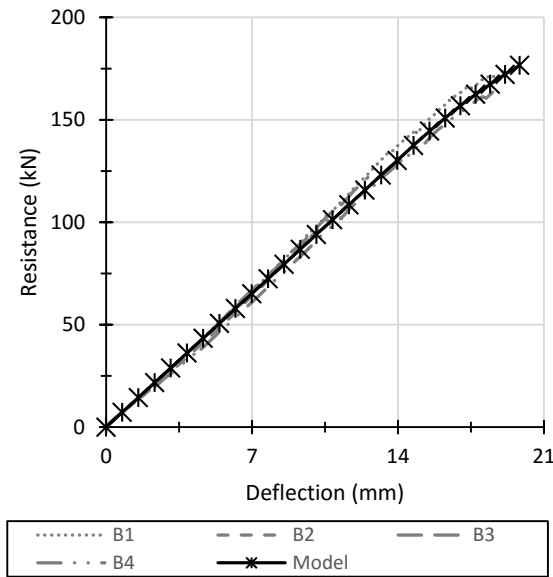


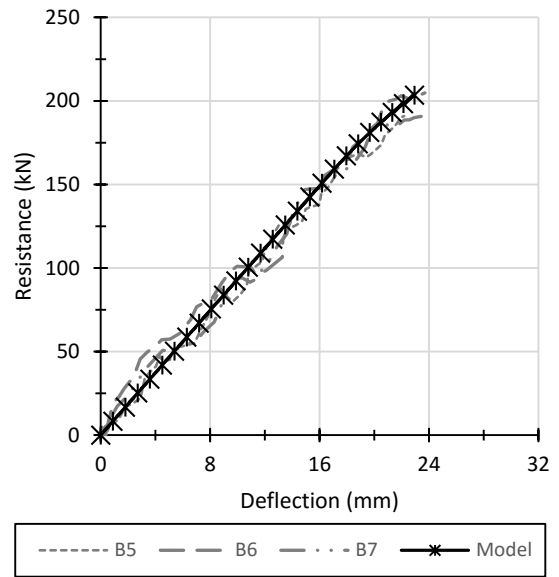
Figure 6.3: Wood input stress-strain relationship

Figure 6.4 compares the predicted static and dynamic resistance curves for both the B-[86] and B-[137] specimens to those obtained experimentally. For the B-[86] specimens, the static model (Fig. 6.4a) slightly over-predicts the maximum resistance and displacement at maximum resistance by ratios of 1.07 and 1.05, respectively. The dynamic model for the B-[86] beams over-predicts on average the resistance of the beams with no continuous FJ failure by a ratio of 1.01 and under-predicts the displacement at maximum resistance by a ratio of 0.96 (Fig. 6.4b).

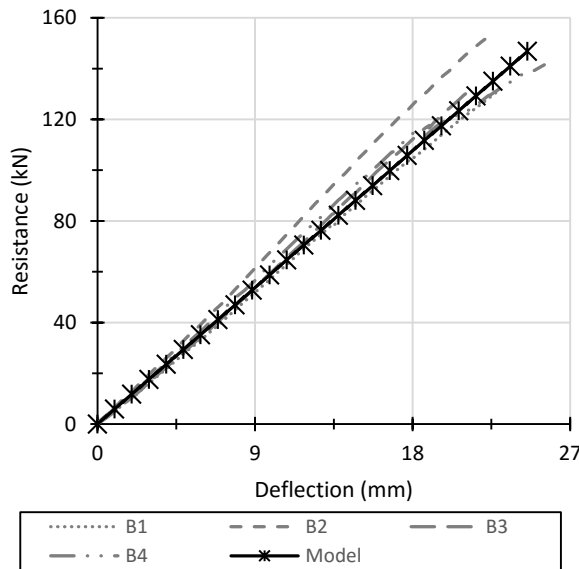
For the B-[137] beams, the static model (Fig. 6.4c) slightly over-predicts both the experimental peak resistance and displacement at maximum resistance by a ratio of 1.08, whereas the dynamic model (Fig. 6.4d) over-predicts the maximum resistance and displacement at maximum resistance by ratios of 1.13 and 1.02, respectively. In general, it can be seen that the proposed approach can capture the resistance curves with reasonable accuracy.



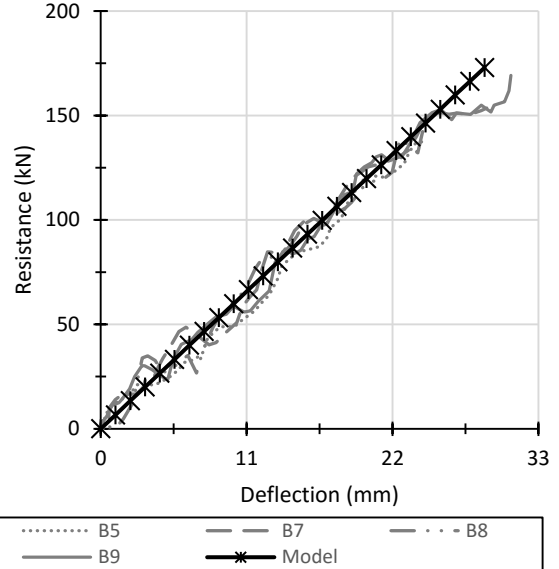
(a) Static – B-[86]



(b) Dynamic – B-[86]



(c) Static – B-[137]



(d) Dynamic – B-[137]

Figure 6.4: Prediction of unretrofitted beams' resistance curves

The equations used for the dynamic resistance were derived from the reactions of an equivalent single-degree-of-freedom (SDOF) system under four point bending with simply supported boundary conditions. The total system mass used in Equation [2.5] was 317.3 kg and 321.0 kg for the B-[86] and B-[137] systems, respectively. The loaded area was taken as 3.55m² which is equivalent to the effective tributary area of the LTD. The reflected pressure-

time histories were obtained from the actual test. Figure 6.5 shows the results of the predicted SDOF displacement compared to the measured displacement for four representative beams. As it can be seen, both the maximum predicted displacement and time to maximum displacement coincide reasonably well with the experimental results for both the elastic and destructive shots.

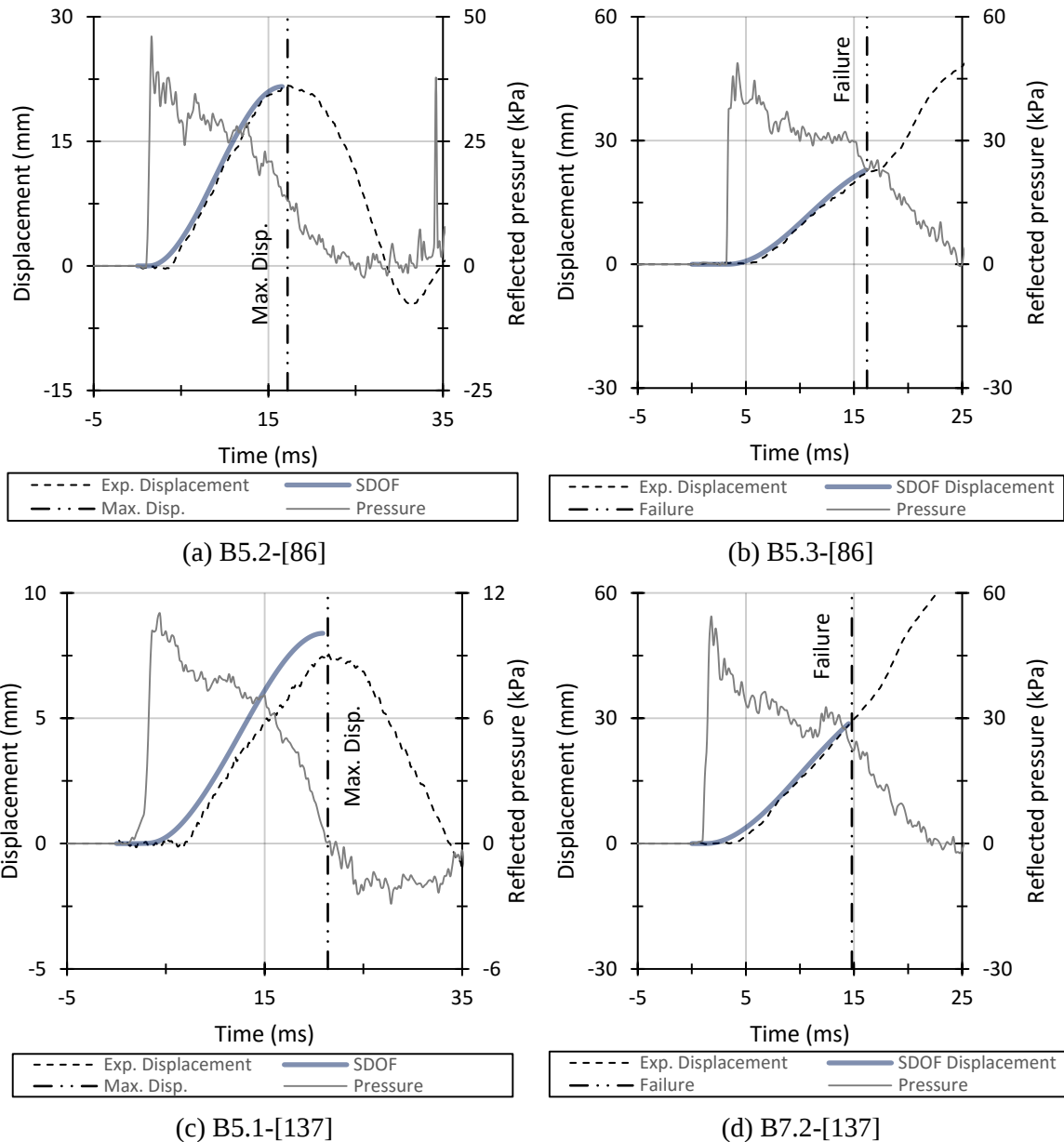


Figure 6.5: Representative predicted displacement-time histories for the unretrofitted beams

Figure 6.6 shows the predicted displacement and time to maximum displacement for all B-[86] and B-[137] specimens compared to those obtained experimentally. An overall average ratio of predicted displacement to experimental displacement of 1.10 with COV of 0.18 was obtained for all tests while the ratio of predicted to experimental time to maximum displacement was 0.96 with a COV of 0.09. The findings show that SDOF analysis with proper input accounting for the tensile and compressive stress-strain behaviour and the strain rate effects can adequately simulate the dynamic response of glulam beams subjected to blast loading.

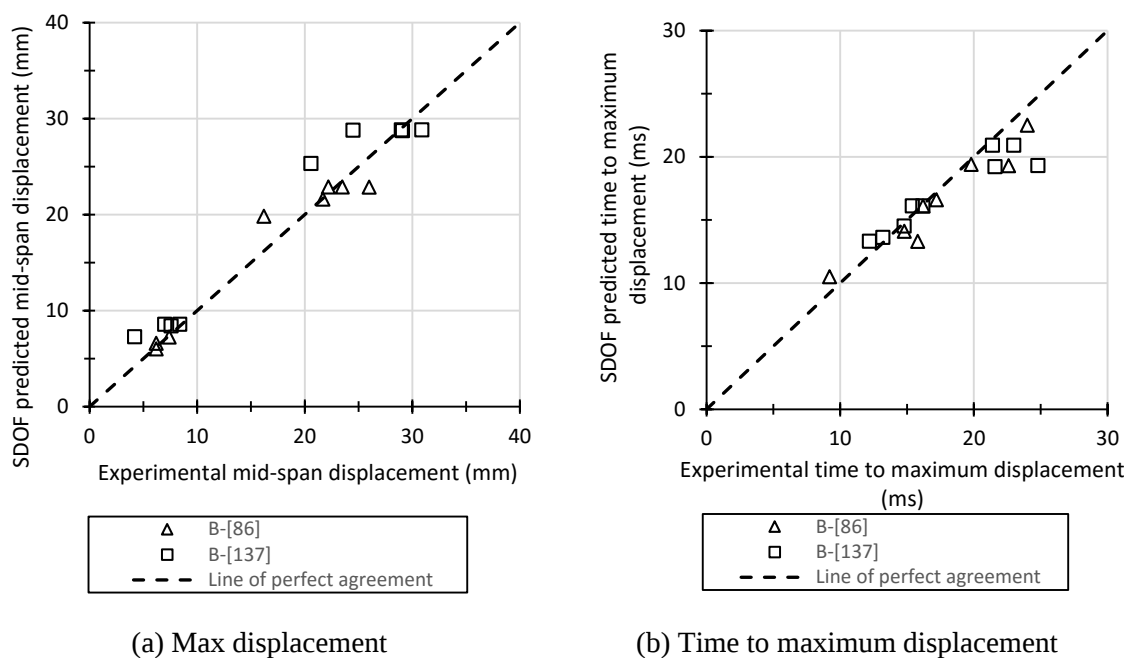


Figure 6.6: SDOF predicted to experimental results

6.2.3 Implications on design of beams

Design provisions have been enacted in the United Kingdom (Office of the Deputy Prime Minister, 2004), and North America (Department of Defense, 2008; ASCE/SEI 59-11, 2011; CSA, 2012) in which requirements covering blast loads and design of various structural elements. Although wood is included as a material option, research on glued laminated timber elements under blast loading has not been undertaken prior to the current study.

In design of structural members for blast loading, levels of protection (LOP) are used to correlate the expected damage level to the response limits of the structure. The component

damage level (i.e. wall, beam, and column) is associated with a non-dimensional response limit that is most often characterized by a support rotation or ductility ratio. Currently, the Canadian blast standard (CSA, 2012) utilizes ductility ratios (μ) for wood structural members to characterize the dynamic response. The ratio is obtained by dividing the maximum displacement of the component, x_{max} , by its elastic displacement, x_e . The current response limits for wood under blast loading indicates that a ductility ratio of 4 corresponds to the component being overwhelmed with significant debris. Viau et al. (2016) demonstrated that for light-frame wood stud walls a ductility ratio of 4 is non-conservative and that a more appropriate value for the maximum ductility at the damage level of blowout is 2. In the case of glulam beams, the specimens were observed not to have any significant post-peak resistance. Therefore, due to the brittle nature of the material any limit above the ductility ratio of unity would indicate ultimate failure. Future research may be able to describe the crack propagation and possibly establish response limits based on support rotations rather than ductility. Also, proper detailing of boundary connections could provide a source of energy dissipation in the system.

The Canadian blast standard (CSA, 2012) assigns a dynamic increase factor of 1.4 for visually graded lumber, machine stress rate (MSR) lumber, and glulam and engineered wood products (EWP). Previous research on studs (Jacques et al., 2014) and light-frame wood stud walls using visually graded lumber (Viau and Doudak, 2016b; Viau and Doudak, 2016a) and MSR (Lacroix, 2013; Lacroix and Doudak, 2015) have shown that a values of 1.4 is appropriate for such structural systems. However, based on the current study it is observed that including a DIF on the flexural strength for the design of glulam members is only appropriate if the glulam beam is manufactured specifically not to include any finger joints (FJs) in the outer laminations in the cases where the width is composed of a single laminate or to include only staggered FJs in the case of multiple laminates across the width of a glulam beam. In such cases, a DIF of 1.14 is deemed appropriate at strain rates in the range of 0.14 to 0.51 s⁻¹. If an existing glulam beam is evaluated or if laminates that are uninterrupted by continuous FJs cannot be produced, then a DIF equal to unity is cautiously suggested for the design of such glulam elements. Furthermore, it is recommended that a DIF equal to unity is used for stiffness when deriving the dynamic resistance curves.

6.3 Effect of axial load on the response of glulam columns

6.3.1 Behaviour of glulam columns subjected to combined axial and out-of-plane simulated blast loading

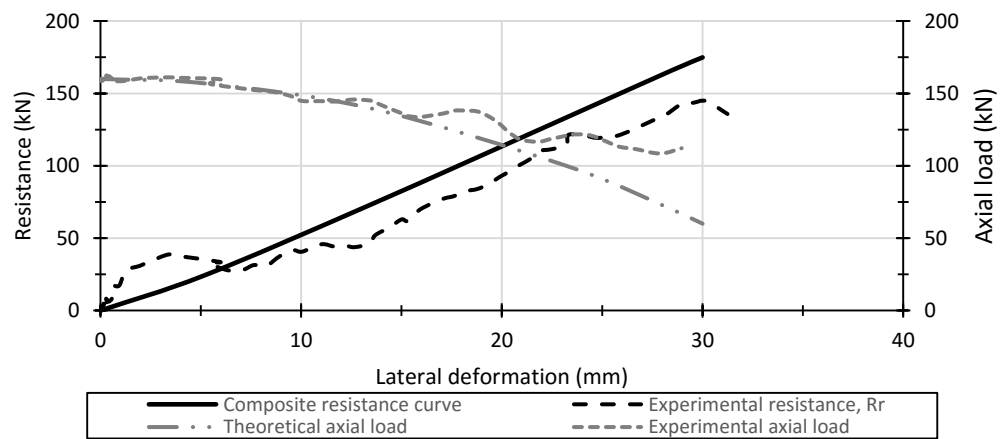
A total of six glulam columns were subjected to combined axial and out-of-plane simulated blast loading as part of the research program. A significant contribution of this study is the actual measurement of the applied axial load during the test. The experimentally measured axial load-time history is therefore used to confirm the theoretical loss in axial load predicted. The addition of an axial load was found to influence the failure mode, where post-test observations (Chapter 4) showed the damage in the column specimens to be concentrated near the member mid-span and highlighted the presence of compression failure. The addition of axial load has, on average, resulted in a displacement at maximum resistance and tensile failure strain that are 1.05 and 1.26 times, respectively, greater than those attained in the unretrofitted beams tested dynamically.

A loss in axial load was observed throughout the response of the columns, which is consistent with previous findings (Kadhom, 2015; Lloyd, 2015). However, the magnitude of the axial load corresponding to peak resistance was more significant (ranging from 50 to 83% of the initial load) than that reported elsewhere for reinforced concrete columns (e.g. Lloyd, 2015). Since the span was very similar to the aforementioned studies, the difference can be attributed to the fact that the compressive stiffness of the glulam columns is significantly less than that of concrete.

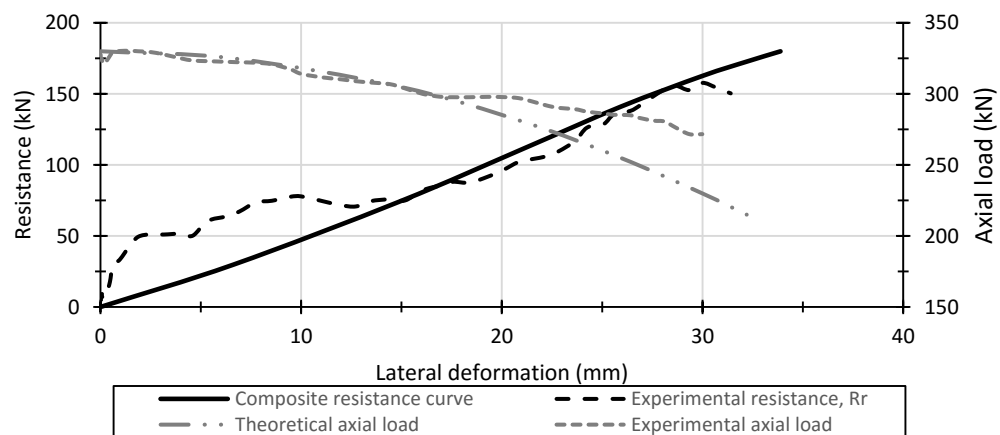
Also, the results in the current study showed that the axial load dropped at a consistent rate independent of axial load levels. Based on the proposed methodology for the variable axial load (Section 5.4.2), this observation was anticipated due to the columns having similar compressive stiffness. This partly corroborate observations made by Kadhom (2015) where specimens with different compressive stiffness were tested, and variable rates in loss in axial load were reported.

6.3.2 Predicting the behaviour of glulam columns

Composite resistance curves for each column were generated following the methodology described in Section 5.4.2 and accounted for the loss in axial load as the column displaced laterally. The material stress-strain relationship proposed in Figure 6.3b was used to generate the moment-curvature relationships as well as resistance curves with different axial load levels. Figure 6.7 shows the generated composite resistance curves compared to the experimental results obtained from the column specimens. Also included in Figure 6.7, is the predicted and experimentally obtained change in axial load.



(a) C2.1



(b) C4.1

Figure 6.7: Predicted resistance curve and axial load comparison to experimental results

In general, it can be seen that the proposed approach is capable of capturing the resistance curves and loss in axial load with reasonable accuracy. Particularly, the prediction of the

variation in axial load is of significance since, to the author's knowledge, no other research has attempted to measure or estimate the actual drop in axial load in timber columns subjected to out-of-plane loading. Although the effect of incompatibility in the boundary condition at relatively large displacements was deemed insignificant up to maximum resistance, it is suggested that future work addresses this issue especially when testing ductile structural elements. It can also be concluded that using the ELL approach together with experimentally determined inputs (i.e. dynamic reaction-, applied pressure-, and axial load-time histories) can adequately capture the behaviour of glulam columns.

The increase in levels of applied axial load has been reported to be accompanied with an increase in stiffness in reinforced concrete columns (Jacques et al., 2013; Burrell et al., 2015; Lloyd, 2015). However, this behaviour depends on the stress-strain relationship used as input in the moment-curvature analysis. For example, when the tension MOE is significantly higher than that of the compression MOE (as is the case in the current study), a slight reduction in stiffness is observed as the levels of applied axial load increases. Due to the discrepancy in behaviour between materials and the importance of this subject to the current study, a sensitivity analysis was undertaken, where the stiffness properties used as input in the model were investigated and discussed (Appendix H).

Figure 6.8 shows representative results from the SDOF analysis using the proposed model compared to the experimental results. The analysis used the actual recorded pressure-time histories for each shot, a system mass of 321.0 kg, and an effective loaded area of 3.55 m². It should be noted that the composite resistance functions used to obtain the predicted displacement-time histories shown in Figure 6.8 already incorporates the second order effects caused by the axial load in the program subroutine, therefore the ELL term in Equation [2.5] was set to zero. While developing the composite resistance curves, the actual load-mass factor (K_{LM}) was calculated at each point based on the actual deflected shape. As seen in Figure 6.8, both the maximum predicted displacement and time to maximum displacement coincide reasonably well with the experimental results for both elastic and destructive shots. An average ratio of predicted displacement to experimental displacement of 1.03 with COV of 0.06 was obtained for all six tests while the time to maximum displacement ratio was 1.02 with a COV of 0.10. It can be concluded that the proposed

material model can be used with reasonable accuracy, provided that the material stress-strain relationships are well defined.

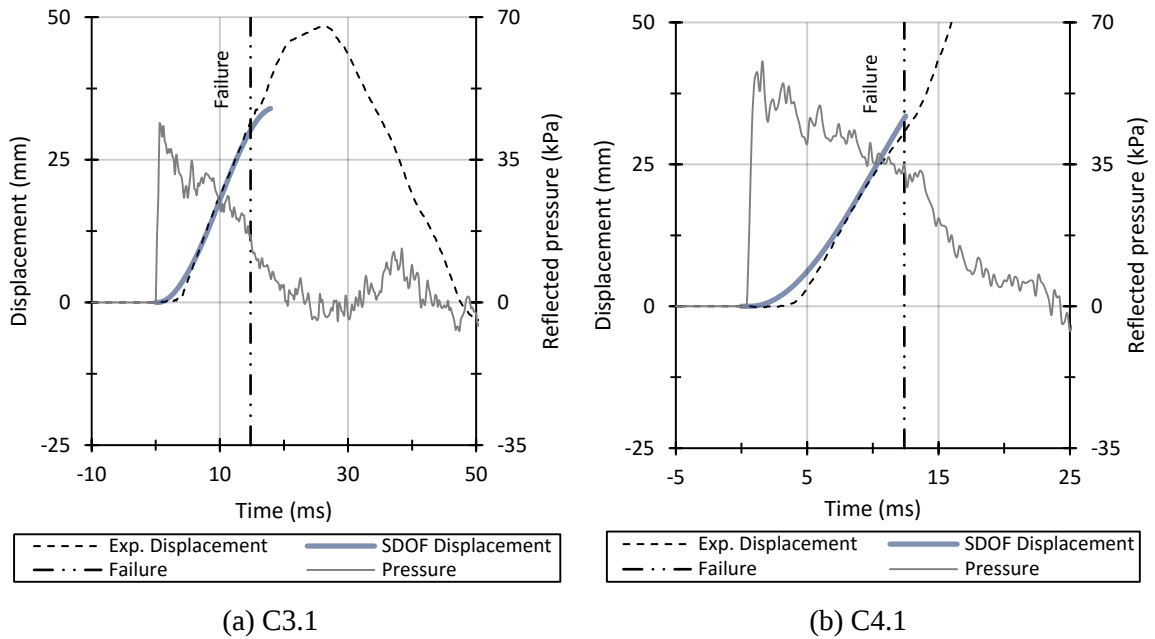


Figure 6.8: Representative SDOF predicted displacement-time history for columns

A methodology to predict the loss in axial load was developed and implemented into the composite resistance curves. To obtain the fit reported in this paper, material inputs for the tension and compression laminates were carefully obtained and the boundary conditions were well defined. It should be noted that the observed loss in axial load was unique to the laboratory conditions. Currently, there is no available literature documenting the effect of loss of axial load in structural elements with “realistic” boundary conditions, where factors such as continuity and connections details between the column and the floor beams are expected to influence the behaviour. More research is needed in this critically important area.

6.4 Behaviour of FRP reinforced glulam beams

6.4.1 Effect of unidirectional FRP reinforcement configurations

Twenty-six retrofitted beams were tested destructively under static and dynamic loading with the aim to investigate the effect of FRP on their flexural behaviour. The first phase investigated the effect of unidirectional fabric on the static and dynamic behaviour of glulam

beams while the effect of multi-directional fabric (i.e. $\pm 45^\circ$, $0^\circ/90^\circ$) was investigated in the second phase with the main objective of enhancing the post-peak capacity and ductility.

The addition of two layers of unidirectional GFRP to the tension laminates of the beams (i.e. Retrofit 1) increased the capacity by a factor of 1.35, deflection at ultimate resistance by a factor of 1.30, and stiffness by a factor of 1.10 compared to the unretrofitted beams. However, this retrofit configuration failed to produce any significant post-peak resistance (Fig. 6.9a), due to the de-bonding of the GFRP reinforcement. This observation is anticipated and consistent with findings reported by other researchers (Dorey and Cheng, 1996; Sonti et al., 1996; Hernandez et al., 1997) for this type of configuration, however, it was investigated to provide a baseline performance for the other retrofit configurations considered in the current study.

The addition of GFRP confinement to the two layers of unidirectional GFRP tension reinforcement (Retrofit 2) increased the ultimate resistance, deflection at ultimate resistance, and stiffness by factors of 1.40, 1.39, and 1.13, respectively, relative to the unretrofitted beams. The slight increase in capacity of Retrofit 2 relative to Retrofit 1 can be attributed to the addition of confinement, which helped prevent the de-bonding between the FRP and wood. The confinement also helped maintain a small level of post-peak capacity (Fig. 6.9b), however, a significant level of post-peak resistance was not attained. Furthermore, the addition of confinement contributed to limiting the damage of the wood specimen to a small region while causing compression failure.

Retrofit 3, with a unidirectional U-shaped GFRP tension reinforcement and one layer of partial-length confinement, had some post-peak resistance as seen in Figure 6.9c. The U-shaped tension reinforcement with partial-length confinement provided increases of 1.57, 1.62, and 1.12 in resistance, deflection at maximum resistance, and stiffness, respectively, when compared to the unretrofitted beams. The deflection at which ultimate failure of Retrofit 3 was attained represented average increases of 1.37 and 2.22 relative to the displacement at peak resistance and the average failure displacement of the unretrofitted beams, respectively. While the level of post-peak resistance was more significant than for

that of Retrofit 2, the lack of full-length confinement resulted in a de-bonding of the FRP, and the post peak resistance could not be further sustained.

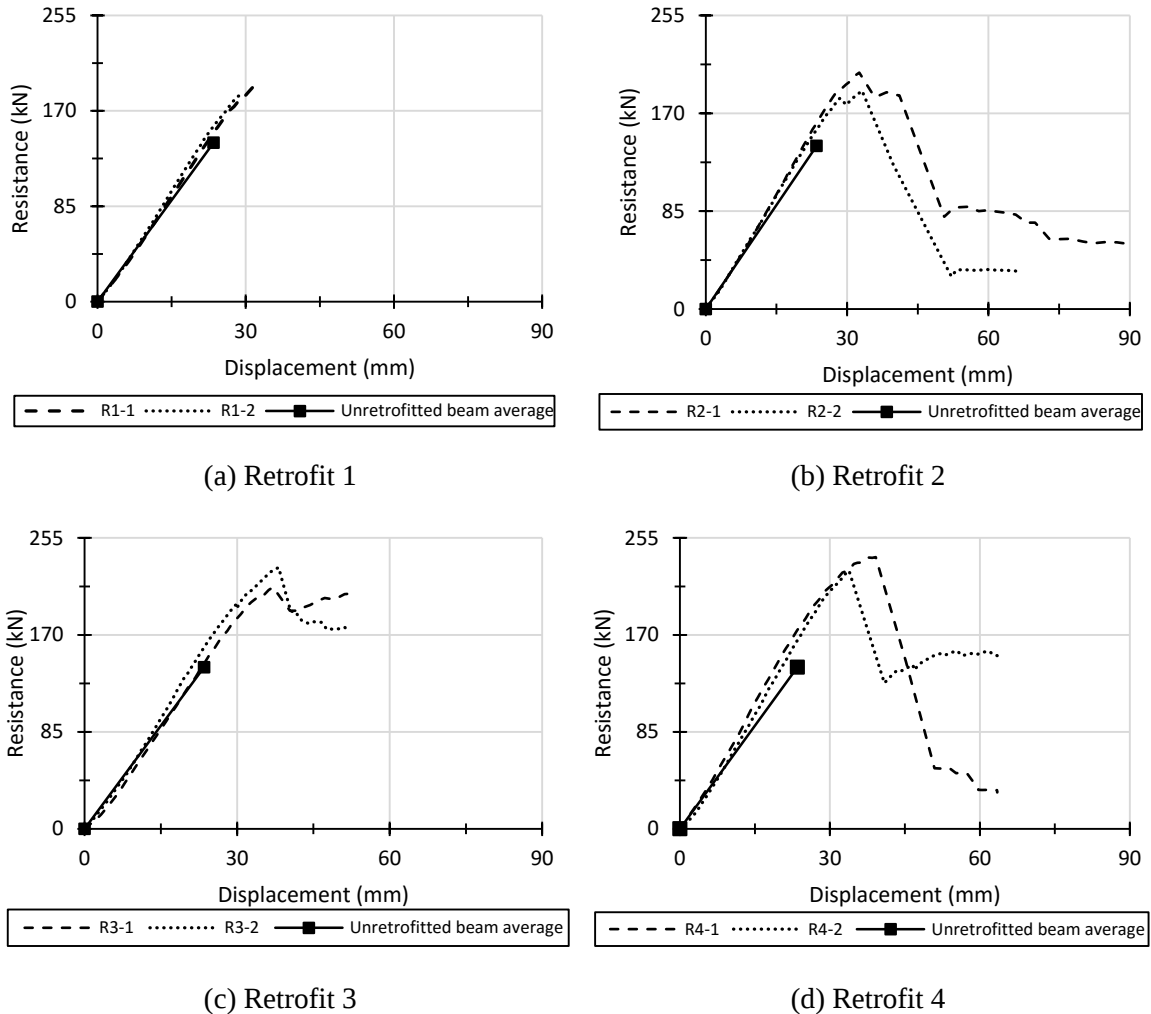


Figure 6.9: Static resistance curve comparison of retrofitted to unretrofitted beams

Retrofit 4 provided the most increase in resistance and stiffness due to the higher ultimate strength and tension modulus of elasticity of carbon relative to glass FRP (Fig. 6.9d). Increases of 1.65, 1.56, and 1.24 in resistance, deflection at maximum resistance, and stiffness, respectively, were obtained relative to the unretrofitted beams. However, no significant post peak resistance was attained.

The dynamic resistance curves of Retrofits 1 through 4 were observed to follow the same behaviour as their static counterparts as shown in Figure 6.10. This observation is consistent

with the fact that no discernable differences were observed between the static and dynamic failure modes. In general, it can be seen that similar levels of deflection were achieved dynamically when compared to the static specimens.

Since the reactions under both static and dynamic loading were measured directly for Retrofits 1 through 4, the DIF was calculated based on the experimental results. A factor of 1.10 was obtained, which compares well with the value of 1.14 found in section 6.2.1 for unretrofitted beams. Figure 6.10 corroborates the assumption that the dynamic resistance curves can be obtained by modifying the static curves with a DIF to account for the strain-rate effects.

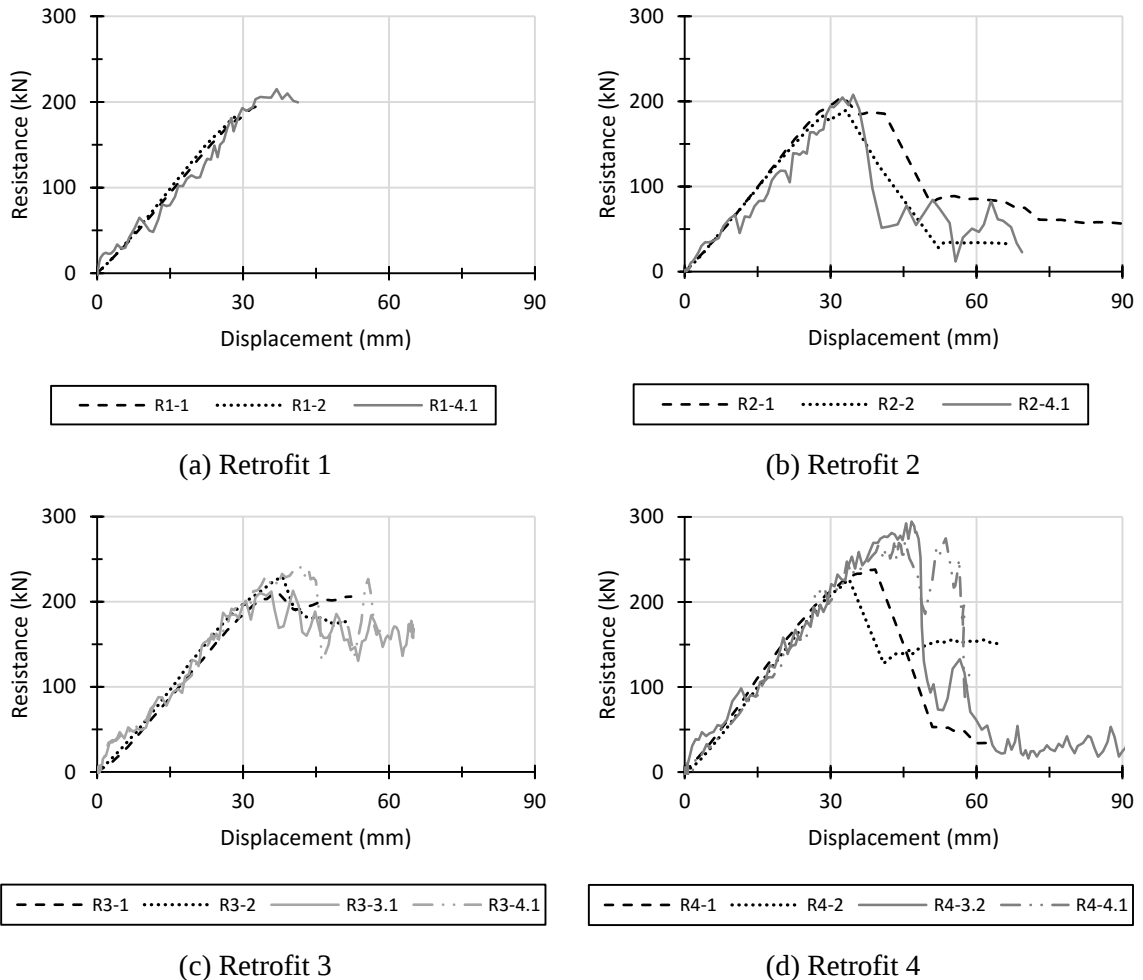
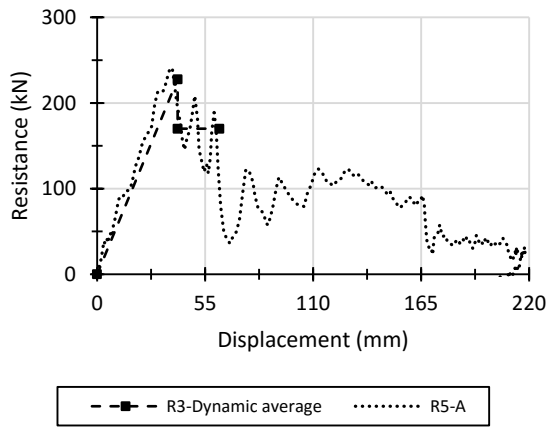


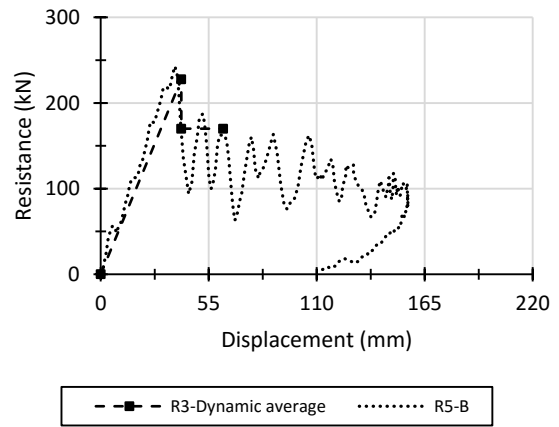
Figure 6.10: Retrofits' dynamic resistance curve comparison to static counterpart

6.4.2 Effect of multi-directional FRP reinforcement configurations

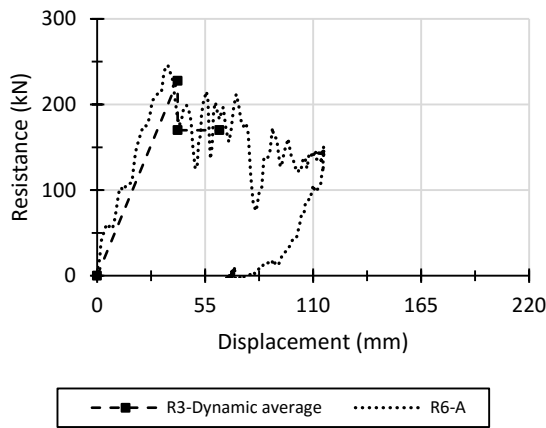
The second phase with FRP reinforcement, which involved multi-directional fabric, had for objective to improve the post-peak behaviour of the beams. The first four specimens tested (R5-A, R5-B, R6-A, and R6-B) consisted of the same cross-section size as the retrofits investigated in the first phase (i.e. $137 \times 222 \text{ mm}^2$). Retrofit 5 investigated the effect of 0/90 bi-directional fabric with (R5-A) and without (R5-B) unidirectional U-shaped fabric. Retrofit 6 investigated the behaviour of ± 45 bi-directional fabric as confinement also with (R6-A) and without (R6-B) unidirectional U-shaped fabric. No additional improvement in the resistance and stiffness was observed for Retrofits 5 and 6, however, the ductility ratio ranged between 1.57 and 3.40. These values constitute a significant improvement in the post-peak behaviour compared to the configurations tested in the first phase of the FRP study. For example, an average ductility ratio of 1.45 was obtained for Retrofit 3, which was the retrofit with the best post-peak performance in Phase 1. Figure 6.11 compares the individual dynamic resistance curves of Retrofits 5 and 6 to the dynamic average of Retrofit 3.



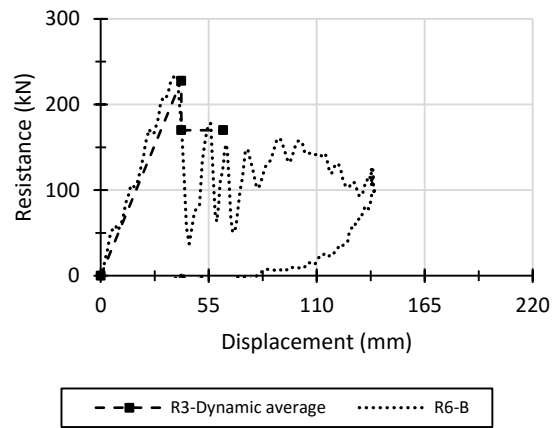
(a) Retrofit 5-A



(b) Retrofit 5-B



(c) Retrofit 6-A



(d) Retrofit 6-B

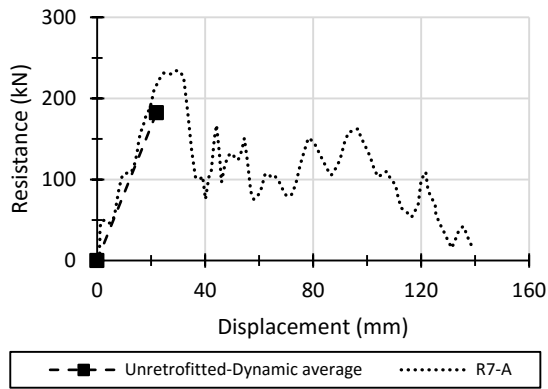
Figure 6.11: Retrofits' dynamic resistance curve comparison to static counterpart

It is clearly seen that the performance is significantly improved by using multi-directional FRP fabrics (i.e. $\pm 45^\circ$, $0^\circ/90^\circ$).

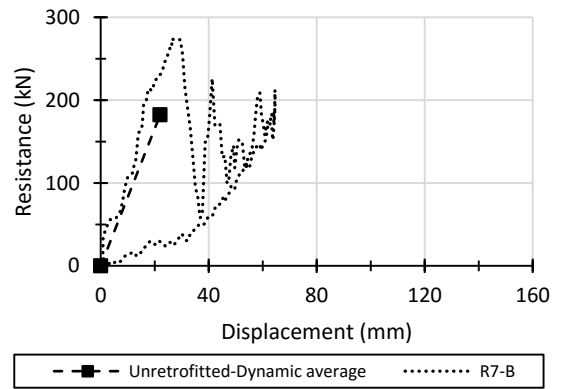
Retrofits 7 to 9 were significantly different than the other retrofits and therefore no direct comparison can be made. However, a comparison is made to the average of the unretrofitted (B-[86]) beams tested dynamically. As can be seen from Figure 6.12, the effect of Retrofits 7 to 9 on the post-peak behaviour is significant. Ductility ratios ranging from 2.30 to 3.61 for Retrofits 7 to 9 were achieved. This range is similar to that observed for Retrofits 5 and 6. The lower value in the range (2.3) may not be representative of the actual ductility of this group of retrofits because some of the beams did not reach their ultimate displacement, and

thus the ductility ratio was based on the maximum recorded displacement reached in those shots.

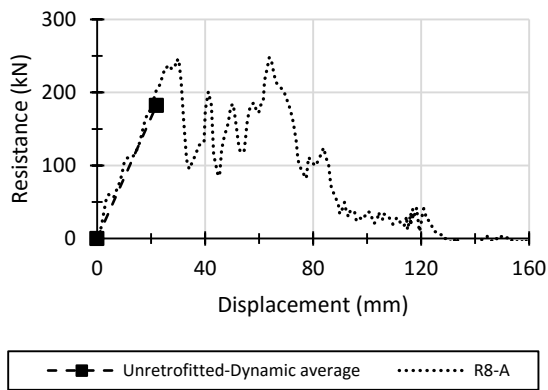
Although similar values of ductility ratios were obtained for Retrofits 8 and 9 for the configurations with (i.e. R8-A and R9-B) and without (R8-B and R9-A) U-shaped reinforcement, it can be observed that the level of sustained post-peak resistance is more significant for the specimens with the U-shaped FRP reinforcement.



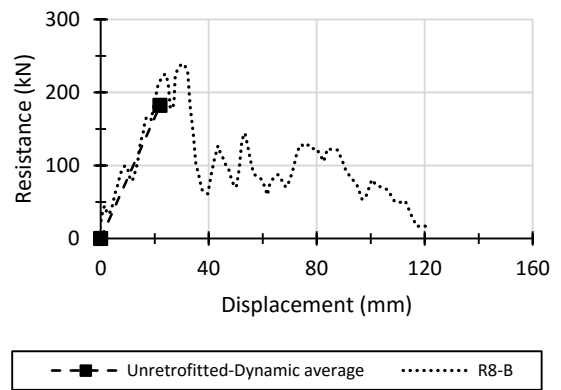
(a) Retrofit 7-A



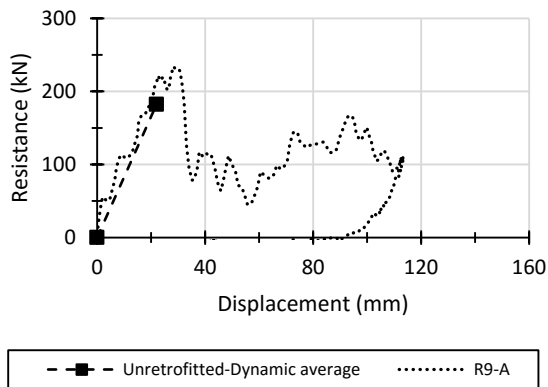
(b) Retrofit 7-B



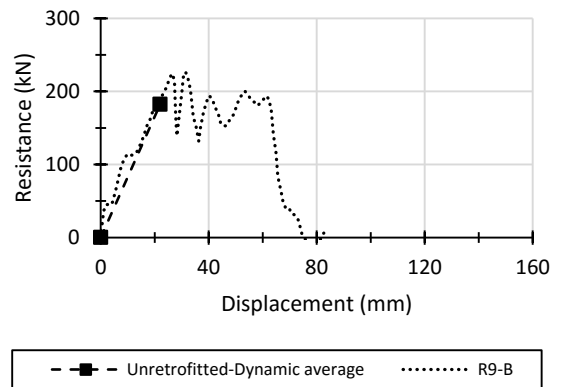
(c) Retrofit 8-A



(d) Retrofit 8-B



(e) Retrofit 9-A



(f) Retrofit 9-B

Figure 6.12: Retrofits' dynamic resistance curve comparison to static counterpart

6.4.3 Effect of unidirectional FRP reinforcement on previously damaged beams

In addition to the four retrofit configurations investigated in the first phase (e.g. R1, R2, R3, and R4), previously tested specimens were restored and retrofitted with two layers of unidirectional confinement at ninety degrees. At the time of retrofitting, this alternative was considered to investigate the potential for temporary support and to better understand the contribution of confinement on restoring the initial capacity of the beams. Figure 6.13 compares the dynamic resistance of the previously damaged and restored beams to the dynamic average of the unretrofitted beams. It can be seen that the FRP confinement was capable of restoring the initial stiffness of the beam, however, the stiffness started degrading at early stages of loading. Adding unidirectional FRP as confinement only at the perpendicular to grain direction did not allow the beam to regain its original resistance, however, it contributed to dissipating energy in the specimens as indicated by the non-linearity of the resistance curves of the restored beam relative to the unretrofitted one. The flexural resistance of the restored and retrofitted beams was estimated to be approximately 50% of that found for the unretrofitted beams (see Table 4.5).

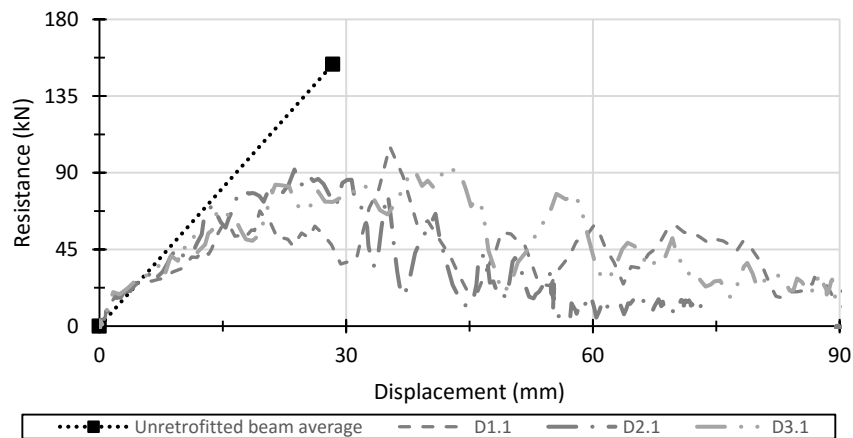


Figure 6.13: Comparison of resistance curve of restored beams to unretrofitted beam

6.4.4 Effect of FRP reinforcement on the strain distribution at failure

The static and dynamic results have shown that the addition of FRP contributes to a higher wood tensile failure strain (Tables 4.8, 4.9 and 4.10). No further increase in failure strain was observed due to the dynamic loading as established in section 6.2.1 where high strain rate

effects are only statistically significant on strength. Table 6.4 shows the increase in tensile failure strain for each retrofit on average. Average factors α_m of 1.17 and 1.25 were obtained for the B-[137] and B-[86] specimens, respectively.

Table 6.4: Wood tensile failure strain

Configuration	Wood tensile failure strain, $\epsilon_{f-T} \times 10^{-4}$			Retrofit / Unretrofitted
	Static	Dynamic	Average	
B-[137]	40.8	36.0	38.4	-
R1-[137]	40.0	44.3	42.2	1.10
R2-[137]	46.5	42.8	44.7	1.10
R3-[137]	46.0	50.8	48.4	1.26
R4-[137]	41.6	46.7	44.2	1.15
R5-A-[137]	-	42.8	42.8	1.11
R5-B-[137]	-	45.5	45.5	1.18
R6-A-[137]	-	43.2	43.2	1.13
R6-B-[137]	-	47.7	47.7	1.24
Unretrofitted-[86]	36.0	34.0	35.0	-
R7-A-[86]	-	37.8	37.8	1.08
R7-B-[86]	-	53.3	53.3	1.52
R8-A-[86]	-	37.2	37.2	1.06
R8-B-[86]	-	42.3	42.3	1.21
R9-A-[86]	-	46.4	46.4	1.33
R9-B-[86]	-	44.9	44.9	1.28

Johns and Lacroix (2000) attributed this phenomena to the presence of a composite layer with high strength and stiffness that arrests crack opening and bridges the defects. However, the authors did not report on a value for the increase. A value of 1.3 for α_m was selected by Gentile et al. (2002) to fit the model to the experimental results. Yang et al. (2016) have also used the value of 1.3 to model the flexural response of FRP and steel reinforced glulam beams. The difference between the value reported in this study and that used in the literature could also be attributed to the fact that FRP fabric was used on glulam in the current study, whereas GFRP bars was used in the study by Gentile et al. (2002). Also, higher variability is found in timber, and according to Johns and Lacroix (2000) and Plevris and Triantafillou (1992) the increase in resistance associated with lower grade timber is more significant than that found in higher grade wood (e.g. glulam). Defining the value of α_m by direct measurement of strain values is a significant contribution as the previous values used in the

literature were calibrated on half-scale specimens before being used for full-scale beams and then by other researchers.

6.4.5 Predicting the behaviour of FRP reinforced glulam beams

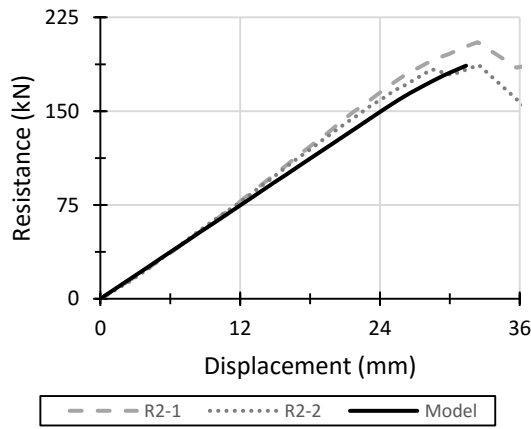
The static and dynamic flexural behaviour of the retrofitted beams can be obtained by conducting moment-curvature analysis (Section 5.3.2 and in Appendix G) and flexural resistance curves (Section 5.4.1 and Appendix G) including the effect of strain rate developed in Section 6.2.1. The static and dynamic stress-strain relationships for the wood member (Figure 6.3) and FRP (Fig. 5.2) were used in the analysis. The specific lay-up for each retrofit was used and no DIF was applied on the FRP (CSA, 2012). An average value of 1.20 for α_m as determined in the previous section was used in Equation [5.22] to modify the wood failure strength.

The results are presented in Table 6.5 and representative resistance curves are shown in Figure 6.18. A ratio of predicted maximum resistance and displacement at maximum resistance to those obtained experimentally of 0.97 and 0.99 with coefficients of variation (COV) of 0.07 and 0.08, respectively, were achieved. From Figures 6.14a through 6.14e, it can be seen that the model captures the overall behaviour of the retrofits tested up to maximum resistance including the non-linearity observed experimentally. This fit is predicated on appropriate model inputs obtained through experimental testing.

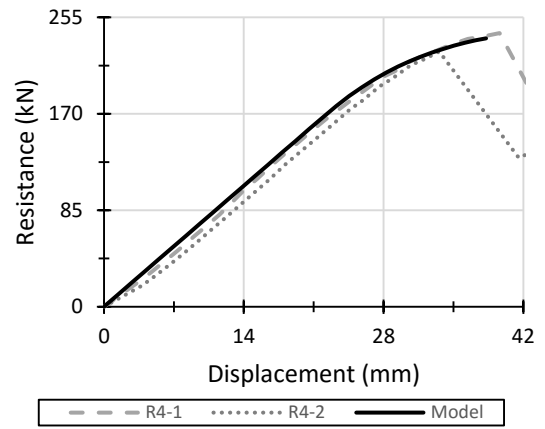
Table 6.5: Comparison of model's predictions to experimental results for retrofitted beams

Specimen	Experimental		Model		Model to experimental ratio	
	R ^a (kN)	Δ_{Rmax}^b (mm)	R ^a (kN)	Δ_{Rmax}^b (mm)	R	Δ_{Rmax}
R1-1	196.7	32.9			0.95	0.95
R1-2	186.2	28.9	186.3	31.4	1.00	1.08
R1-4.1	216.1	36.9	213.7	36.1	0.99	0.98
R2-1	206.3	32.6			0.90	0.96
R2-2	189.8	33.2	186.3	31.4	0.98	0.95
R2-4.1	208.0	34.6	213.7	36.1	1.03	1.04
R3-1	214.3	38.5			0.93	0.87
R3-2	230.3	38.6	199.2	33.3	0.86	0.86
R3-4.1	242.3	41.7	224.2	37.1	0.93	0.89
R4-1	242.2	40.3			0.98	0.95
R4-2	226.5	33.8	236.5	38.3	1.04	1.13
R4-3.2	295.7	46.6			0.92	0.94
R4-4.1	274.6	43.5	270.6	43.8	0.99	1.01
R5-A	241.2	38.2	219.3	36.7	0.91	0.96
R5-B	242.9	37.7	217.7	36.5	0.90	0.97
R6-A	246.0	36.0	223.9	37.1	0.91	1.03
R6-B	234.3	38.9	212.1	36.0	0.91	0.93
R7-A	234.2	30.4	257.1	31.0	1.10	1.02
R7-B	274.2	26.3	262.2	31.5	0.96	1.20
R8-A	245.5	30.2	252.6	30.0	1.03	0.99
R8-B	239.5	30.1	233.0	28.2	0.97	0.94
R9-A	234.3	28.3	259.9	30.7	1.11	1.08
R9-B	223.4	26.7	244.8	29.1	1.10	1.09
Avg.					0.97	0.99
St. Dev.					0.07	0.08
COV					0.07	0.08

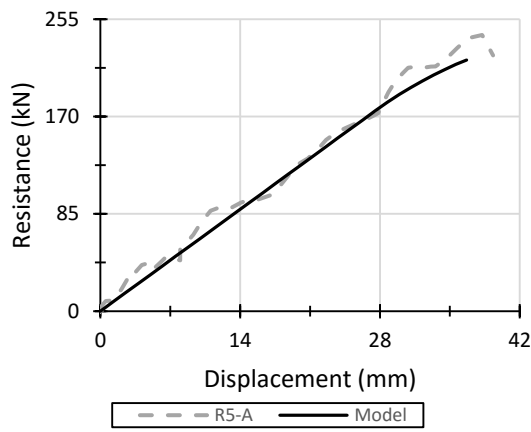
^aMaximum resistance^bDeflection at maximum resistance



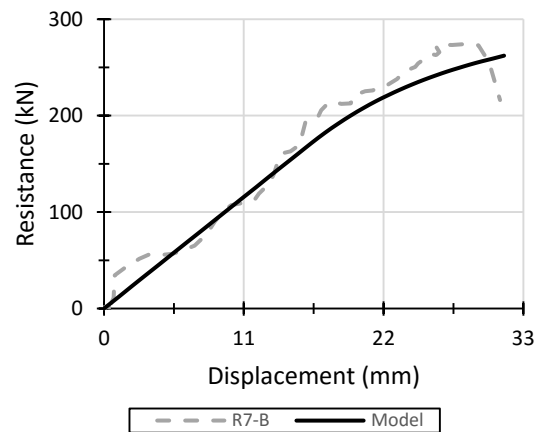
(a) Static - R2-[137]



(b) Static - R4-[137]



(c) Dynamic - R5-A



(d) Dynamic - R7-B

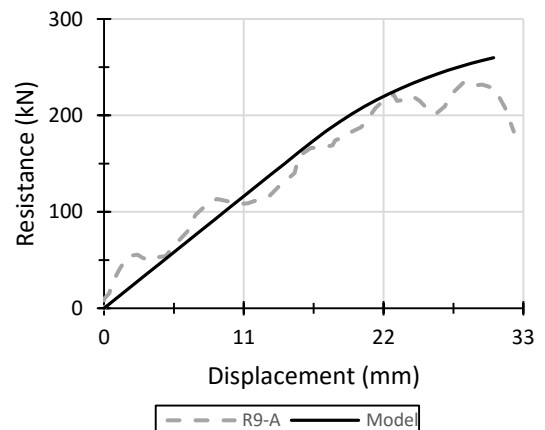
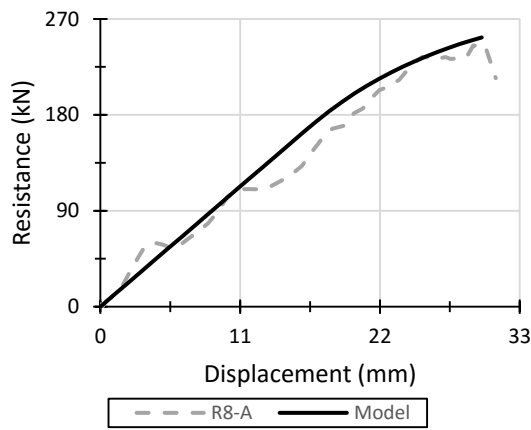


Figure 6.14: Prediction of unretrofitted beams' resistance curves

CHAPTER 7 - Conclusions

The following conclusions can be drawn from the current research study:

- A shock tube with a load transfer device (LTD) can effectively be used to generate high strain rates response in the range of 0.14 to 0.52 s^{-1} in the unretrofitted and FRP retrofitted glulam beam and column members
- Differences in failure modes were observed in unretrofitted glulam beams, primarily due to the presence and distribution of defects such as finger joints and knots. Simple or splintering tension failures were observed in the static tests for beams that consist of a single laminate in their width, whereas a brash tension failure was observed during under dynamic loading. It was also observed that beams with multiple laminates across their width generally experienced significantly more crack propagation and splintering compared to beams consisting of a single laminate when tested dynamically.
- A dynamic increase factor of 1.14 on the flexural strength was found to be appropriate for elements manufactured specifically not to include any FJ (for beams consisting of a single laminate in their width) or continuous FJs (for multiple laminates beams) in the outer laminations. If continuous laminates, uninterrupted by FJs cannot be guaranteed, a dynamic increase factor of unity is cautiously suggested for design. Furthermore, it is recommended that a DIF of unity be used for stiffness when deriving the dynamic resistance curves. Since the unretrofitted beams exhibited little to no ductility past their peak resistance, it is recommended that a linear-elastic resistance curve be used for purpose of flexural design of glulam beams under blast loads.
- The addition of an axial load was found to influence the failure mode where post-test observations showed the damage to be concentrated near the member mid-span. The applied axial load was measured throughout the response and the loss in axial load was documented. The magnitude of the axial load corresponding to peak resistance was significant, ranging from 50 to 83% of the initial load and the loss in load occurred at a constant rate independent of axial load levels.

- The potential of unidirectional and bi-directional FRP as strength enhancing options for glulam elements subjected to blast loading was investigated experimentally. It was found that the addition of FRP to glulam beams has great potential to improve their performance including significant increase in resistance and post-peak behaviour. While an increase in performance was demonstrated for tension-alone reinforcement, this retrofit resulted in premature de-bonding between the FRP and the wood. Adding confinement prevented premature de-bonding and significantly improved the behaviour. The addition of U-shaped tension reinforcement with partial- or full-length confinement delayed the de-bonding between the FRP and wood beyond peak-resistance.
- Bi-directional confinement (i.e. 0/90 and ± 45) with and without unidirectional reinforcement provided significantly better post-peak behaviour achieving ductility ratios ranging 1.57 – 3.61. The ductility ratios are based on a post-peak resistance equal to 50 % of maximum resistance for the glulam member.
- Maximum increases in resistance, stiffness, and deflection of 1.93, 1.56, and 1.24, respectively, were obtained for the retrofitted beams compared to their unretrofitted counterparts.
- The addition of FRP to glulam beams also contributed to limiting crack development and bridging the wood substance, thereby increasing the tensile strain associated with the failure. A factor accounting for the enhancement of the bending strength of 1.21 on the tensile failure strain was determined experimentally.
- Tearing in the FRP fabric in some configurations contributed to lower ductility ratios and lower post-peak resistance. Full-length confinement seemed to perform well to circumvent such shortcoming. The use of bi-directional fabric also helped improve the post-peak capacity due to having strength in multiple directions.
- The addition of confinement to previously damaged beams significantly altered the failure mode, and partially restored their original strength and stiffness, presenting a potential option for members with limited damage. Although no generalized statement can be made based on the limited test specimens presented in this study,

the results encourage further investigation in the area of using FRP to restore damaged wood members.

- A procedure capturing the strain-rate effects (DIF), variable axial load and FRP, was developed and found to be capable of predicting the flexural behaviour of the beams up to maximum resistance with reasonable accuracy when compared to experimentally obtained static and dynamic resistance curves. The implementation of the variable axial load into the composite resistance functions can accurately capture the behaviour of glulam columns. The predicted loss in axial load using the projected shortening analogy correlated well with the experimentally measured axial load up to the point of maximum resistance. An equivalent SDOF approach implementing the ELL approach can effectively be used to determine the column's resistance when the displacement-, pressure-, reaction-, and axial load-time histories are measured experimentally.

Despite the effort to provide a comprehensive understanding of the behaviour of glulam members subjected to blast loading, the current study has some limitations. The use of the proposed procedure to model the flexural response of glulam beams and columns, especially in design, should be cautioned. To obtain the fit reported in this paper, material inputs for the tension and compression laminates were carefully obtained. Furthermore, the results show that the model is sensitive to the MOE inputs. Also, the axial load was measured experimentally on a limited number of specimens. Finally, actual column conditions may be different than those used in the laboratory and it is therefore not yet feasible, based on these results alone, to generalize the findings to any design situation.

7.1 Recommendations for Future Research

Based on the research described in this thesis, it recommended to investigate:

- Different span to depth ratio, layups, and cross-section sizes to help validate or improve the DIF value, failure mode observations, and modelling techniques presented in the current study.
- The relationship between defect distribution (including FJ) and failure mode. This can help establish, in a quantitative manner, the effect of such defect on the DIF.

- The effect of realistic boundary conditions on the behaviour of glulam beams and columns. The behaviour using typical connections could be investigated and joints that are optimized to resist the blast effects could be developed.
- The response limits and development of appropriate damage levels descriptions.

More advanced modelling techniques better capable of predicting the post-peak behaviour of both retrofitted and unretrofitted beams and columns.

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Appendix A – Static Test Results of Wood Coupons

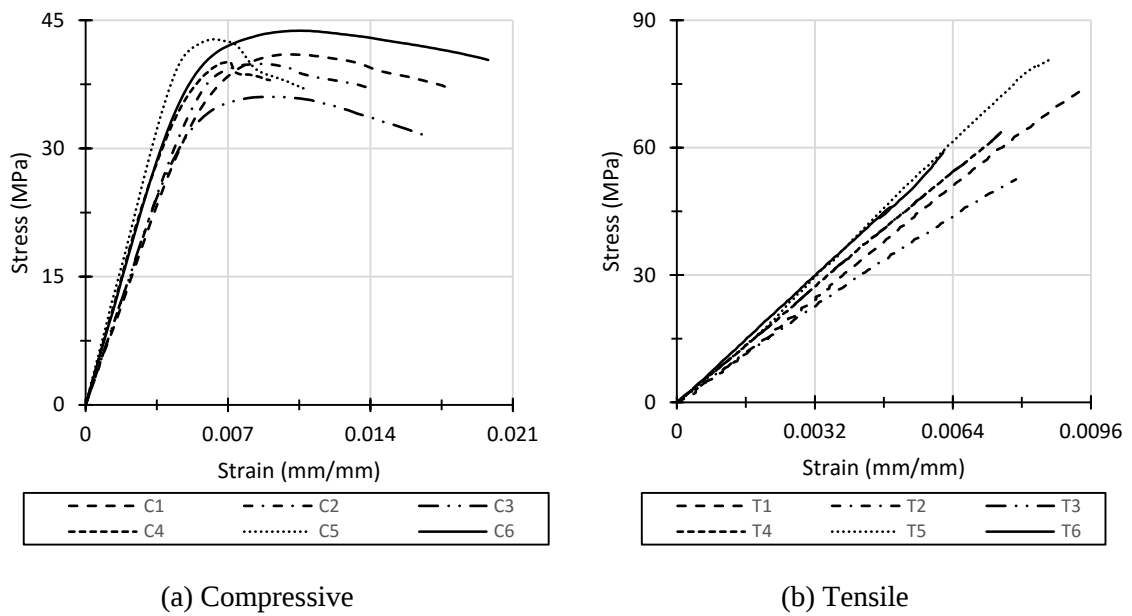
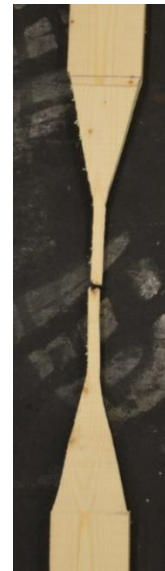


Figure A.1: Experimental stress-strain curves for B-[86] coupons



(a) Compression – C6



(b) Tension – T3

Figure A.2: Representative failure modes for B-[86] coupons

Table A.1: Individual static tension coupon tests results for B-[86] beams

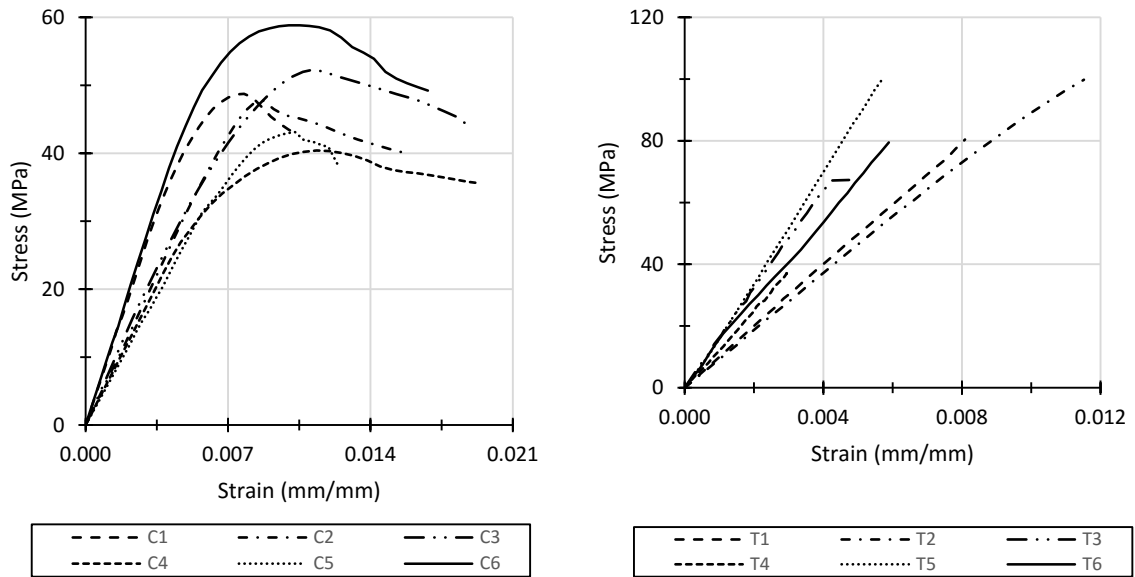
Coupon	E_{wt}^a (MPa)	$f_{t,exp}^b$ (MPa)	$\epsilon_t^c \times 10^{-4}$ (mm/mm)
T1	7,906	73.4	93.7
T2	7,030	52.5	78.6
T3	7,536	49.5	68.0
T4	8,522	63.6	75.2
T5	9,411	81.1	87.1
T6	9,324	59.1	61.8
<i>Avg.</i>	<i>8,289</i>	<i>63.2</i>	<i>83.8</i>
<i>St. Dev.</i>	<i>883</i>	<i>11</i>	<i>10.8</i>
<i>COV</i>	<i>0.11</i>	<i>0.18</i>	<i>0.13</i>

^aTension modulus of elasticity^bUltimate tensile strength^cTensile failure strain

Table A.2: Individual static compression coupon tests results for B-[86] beams

Coupon	E_{wc}^a (MPa)	$f_{c,exp}^b$ (MPa)	$f_{cu,exp}^c$ (MPa)	$\epsilon_{cu}^d \times 10^{-4}$ (mm/mm)
C1	6,638	41.0	36.9	179.1
C2	6,952	39.9	37.1	139.2
C3	6,963	36.1	31.5	166.6
C4	8,449	40.1	38.0	90.6
C5	9,220	42.8	37.0	108.0
C6	8,289	43.8	40.4	197.9
<i>Avg.</i>	<i>7,752</i>	<i>40.6</i>	<i>36.8</i>	<i>146.9</i>
<i>St. Dev.</i>	<i>952</i>	<i>2.5</i>	<i>2.7</i>	<i>38.2</i>
<i>COV</i>	<i>0.12</i>	<i>0.06</i>	<i>0.07</i>	<i>0.26</i>

^aCompression modulus of elasticity^bCompressive yield strength^cUltimate compressive strength^dUltimate compressive strain



(a) Compressive

(b) Tensile

Figure A.3: Experimental stress-strain curves for B-[137] coupons



(a) Compression – C2



(b) Tension – T3

Figure A.4: Representative failure modes for B-[86] coupons

Table A.3: Individual static tension coupon tests results for B-[86] beams

Coupon	E_{wt}^a (MPa)	$f_{t,exp}^b$ (MPa)	$\epsilon_t^c \times 10^{-4}$ (mm/mm)
T1	10,033	83.32	80.0
T2	9,334	101.48	118.0
T3	16,206	75.84	54.6
T4	12,221	37.33	29.4
T5	16,575	99.84	56.9
T6	15,372	79.48	58.9
<i>Avg.</i>	<i>13,290</i>	<i>79.5</i>	<i>66.3</i>
<i>St. Dev.</i>	<i>2,916</i>	<i>21.2</i>	<i>27.4</i>
<i>COV</i>	<i>0.22</i>	<i>0.27</i>	<i>0.41</i>

^aTension modulus of elasticity^bUltimate tensile strength^cTensile failure strain

Table A.4: Individual static compression coupon tests results for B-[86] beams

Coupon	E_{wc}^a (MPa)	$f_{c,exp}^b$ (MPa)	$f_{cu,exp}^c$ (MPa)	$\epsilon_{cu}^d \times 10^{-4}$ (mm/mm)
C1	8,933	48.8	42.8	104.3
C2	6,295	47.8	39.8	159.5
C3	6,778	52.3	44.1	187.9
C4	5,854	40.4	35.6	194.1
C5	5,458	43.2	38.2	124.2
C6	9,293	58.9	49.2	168.2
<i>Avg.</i>	<i>7,102</i>	<i>48.6</i>	<i>41.6</i>	<i>156.4</i>
<i>St. Dev.</i>	<i>1,482</i>	<i>6.0</i>	<i>4.4</i>	<i>32.4</i>
<i>COV</i>	<i>0.21</i>	<i>0.12</i>	<i>0.11</i>	<i>0.21</i>

^aCompression modulus of elasticity^bCompressive yield strength^cUltimate compressive strength^dUltimate compressive strain

Appendix B – Static Test Results of FRP Coupons

FyfeCo – GFRP - [0]₂

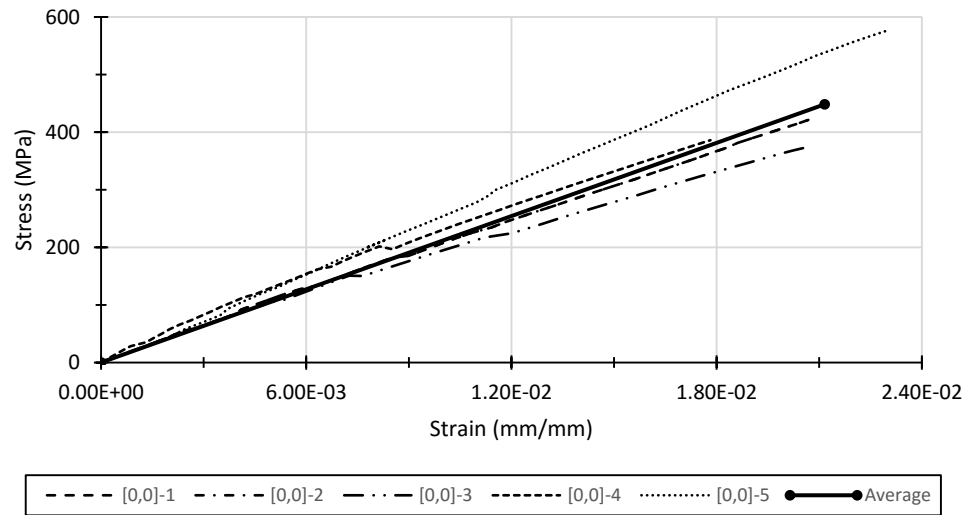


Figure B.1: Stress-strain relationship for GFRP [0]₂ (FyfeCo)

Table B.1: Summary of results for GFRP [0]₂ (FyfeCo)

Specimen	[0] ₂					Average	Std. Dev.	COV
	1	2	3	4	5			
MOE (MPa)	21,100	20,538	18,681	22,635	25,552	21,701	2,304	0.11
f_{FRP} (MPa)	484.6	421.2	373.7	386.0	575.7	448.3	74	0.17
ε_{FRP} (mm/mm)	2.37E-02	2.07E-02	2.06E-02	1.78E-02	2.30E-02	2.12E-02	2.07E-03	0.10
t (mm)	2.82	2.66	2.81	2.82	2.20	2.66	0.24	0.09

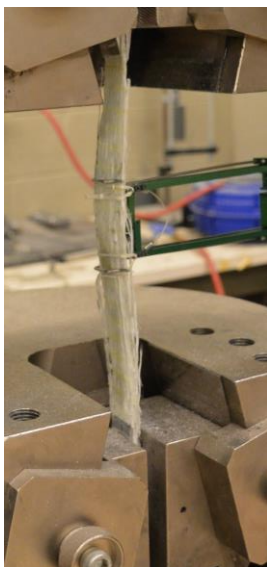


Figure B.2: Representative failure mode for GFRP [0]₂ (FyfeCo)

FyfeCo – CFRP - [0]₂

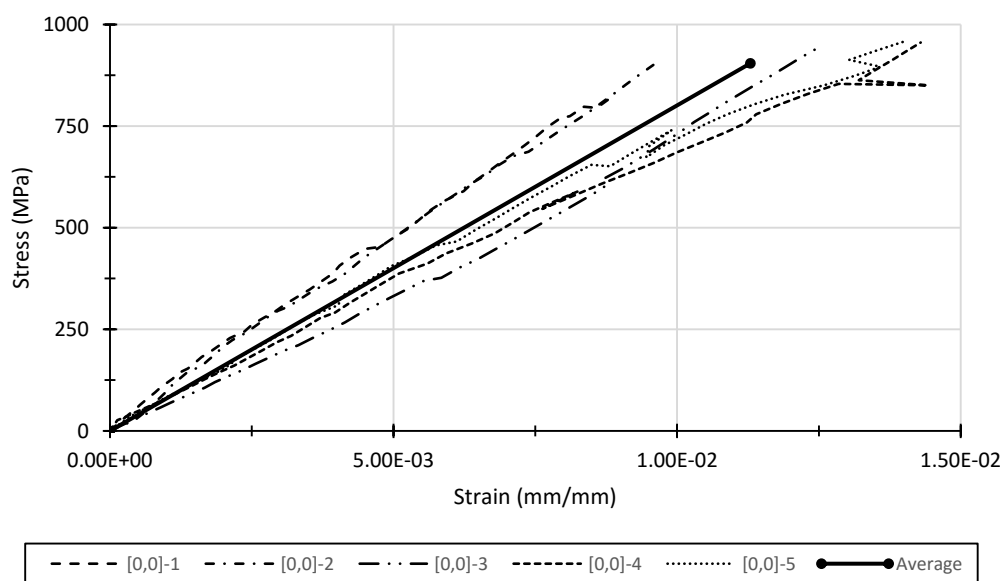


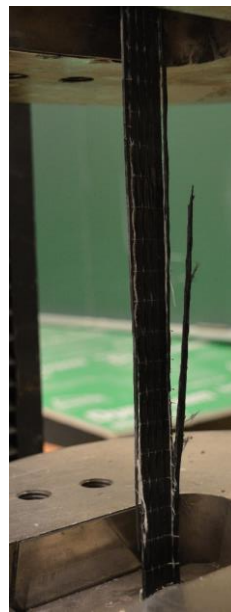
Figure B.3: Stress-strain relationship for CFRP [0]₂ (FyfeCo)

Table B.2: Summary of results for CFRP $[0]_2$ (FyfeCo)

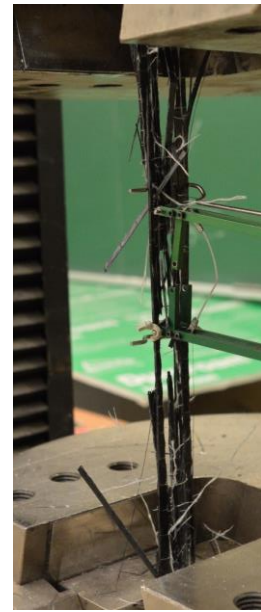
Specimen	$[0]_2$					Average	Std. Dev.	COV
	1	2	3	4	5			
MOE (MPa)	96,329	94,378	71,550	67,899	96,329	85,297	12,787	0.15
f_{FRP} (MPa)	823.5	900.3	937.7	955.5	823.5	904.3	51.0	0.06
ϵ_{FRP} (mm/mm)	8.87E-03	9.58E-03	1.24E-02	1.43E-02	8.87E-03	1.13E-02	2.19E-03	0.19
t (mm)	2.04	1.93	2.05	2.19	2.05	2.1	0.09	0.05



(a) $[0]_2$ -3



(b) $[0]_2$ -4



(c) $[0]_2$ -4

Figure B.4: Representative failure modes for CFRP $[0]_2$ (FyfeCo)

Simpson StrongTie – GFRP - $[0]_2$

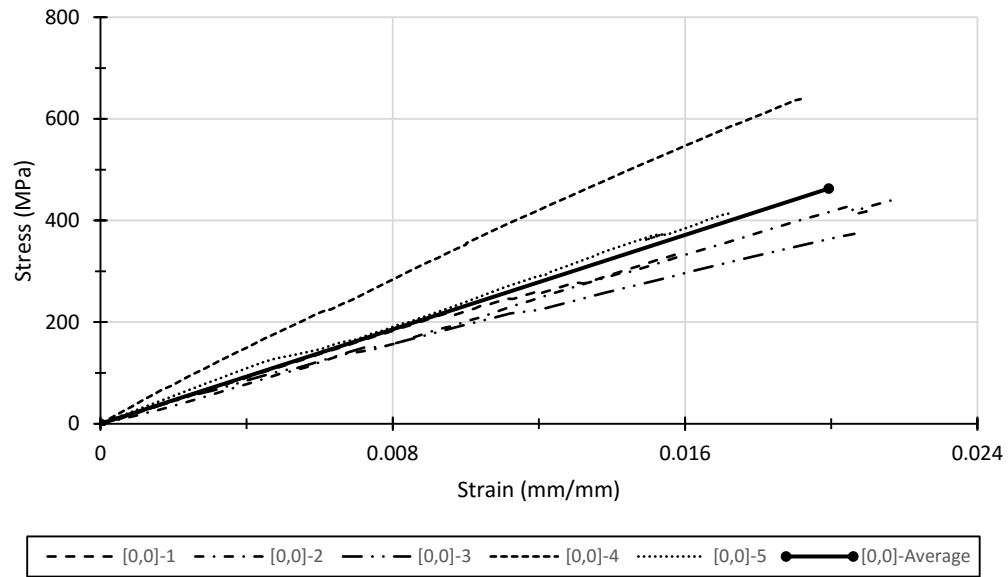


Figure B.5: Stress-strain relationship for GFRP $[0]_2$

Table B.3: Summary of results for GFRP $[0]_2$

Specimen	$[0]_2$					Average	Std. Dev.	COV
	1	2	3	4	5			
MOE (MPa)	22,593	20,642	18,972	35,615	24,294	24,423	5,876	0.24
f_{FRP} (MPa)	338.4	439.2	482.9	640.1	413.6	462.8	100.3	0.22
ϵ_{FRP} (mm/mm)	1.60E-02	2.16E-02	2.56E-02	1.93E-02	1.72E-02	1.99E-02	3.44E-03	0.17
t (mm)	2.14	2.28	2.21	2.21	2.50	2.27	0.12	0.05

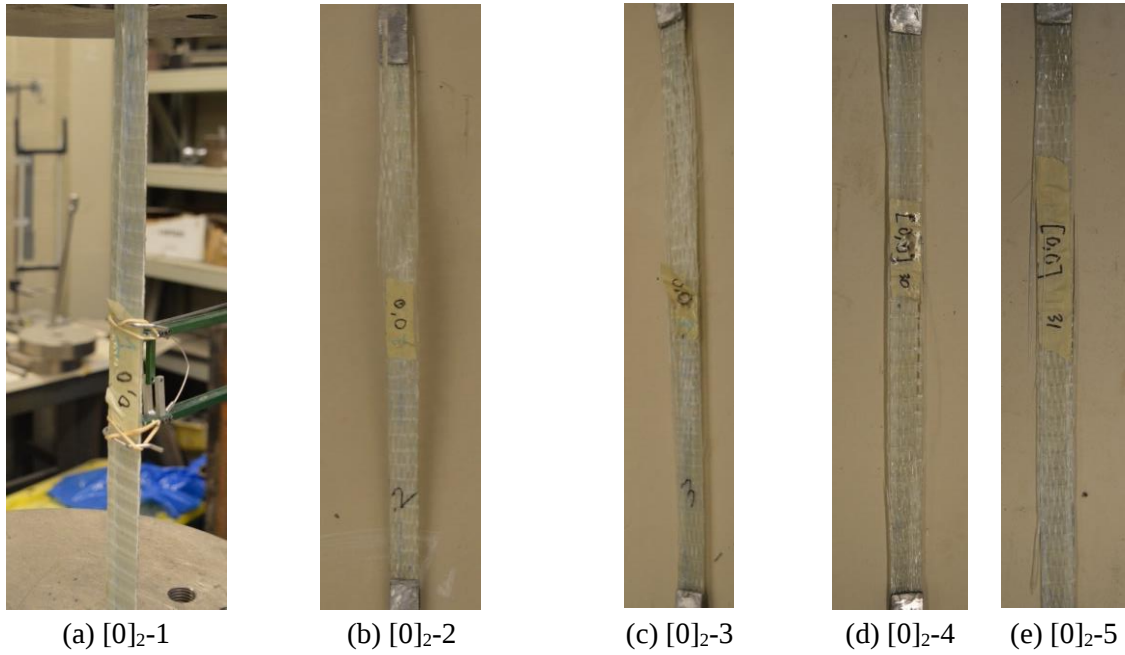


Figure B.6: Representative failure modes for GFRP $[0]_2$

Simpson StrongTie – GFRP - $[\pm 45]_2$

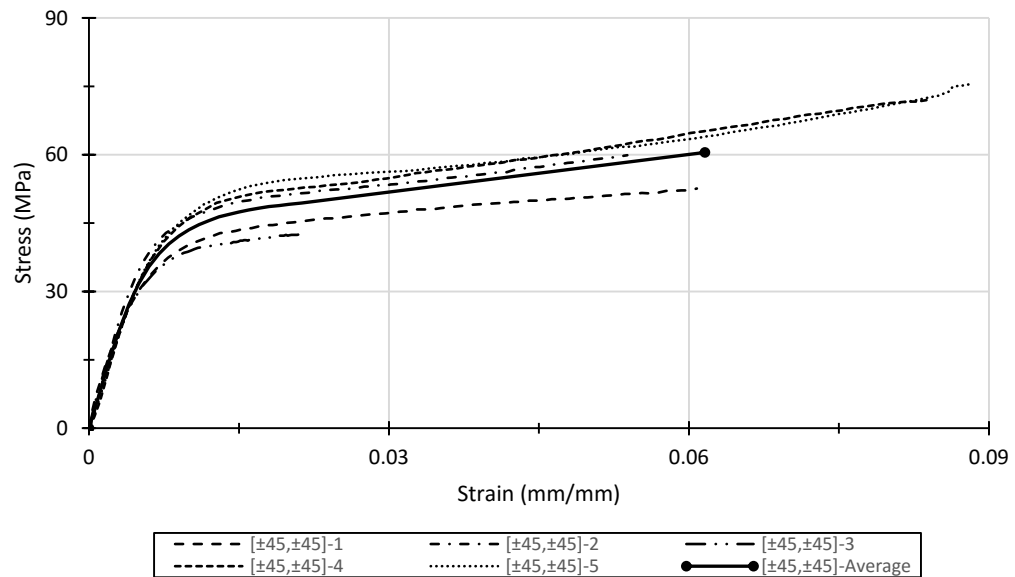


Figure B.7: Stress-strain relationship for GFRP $[\pm 45]_2$

Table B.4: Summary of results for GFRP $[\pm 45]_2$

Specimen	$[\pm 45]_2$					Average	Std. Dev.	COV
	1	2	3	4	5			
MOE (MPa)	6,654	7,427	7,276	6,487	6,666	6,902	376	0.05
f_{FRP} (MPa)	52.6	59.9	42.6	72.0	75.4	60.5	12.1	0.20
ϵ_{FRP} (mm/mm)	6.08E-02	5.38E-02	2.13E-02	8.39E-02	8.80E-02	6.16E-02	2.40E-02	0.39
t (mm)	1.67	1.76	1.65	1.77	1.77	1.72	0.05	0.03

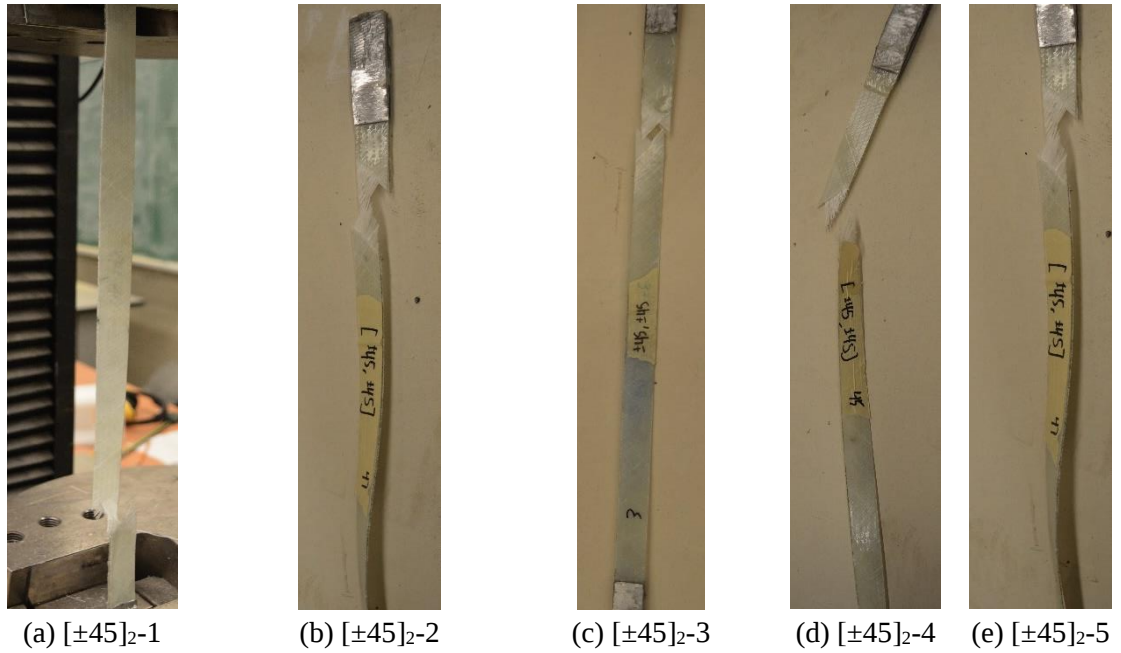


Figure B.8: Representative failure modes for GFRP $[\pm 45]_2$

Simpson StrongTie – GFRP - [0/90]₂

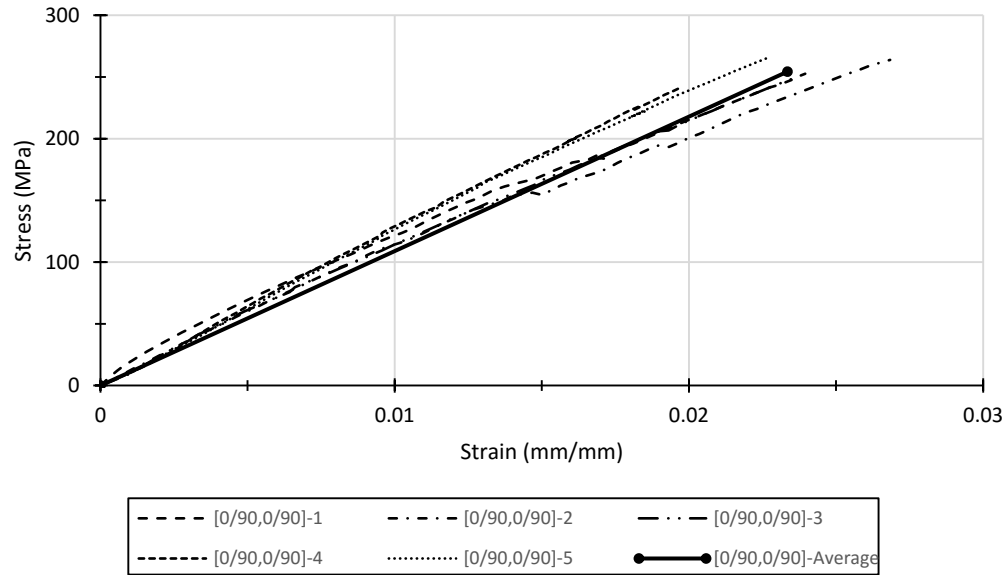


Figure B.9: Stress-strain relationship for GFRP [0/90]₂

Table B.5: Summary of results for GFRP [0/90]₂

Specimen	[0/90] ₂					Average	Std. Dev.	COV
	1	2	3	4	5			
MOE (MPa)	11,982	10,810	11,264	12,734	12,460	11,850	721	0.06
f_{FRP} (MPa)	247.6	263.8	252.6	241.1	266.4	254.3	9.6	0.04
ε_{FRP} (mm/mm)	2.35E-02	2.68E-02	2.40E-02	1.97E-02	2.28E-02	2.33E-02	2.29E-03	0.10
t (mm)	1.70	1.74	1.75	1.78	1.79	1.75	0.03	0.02

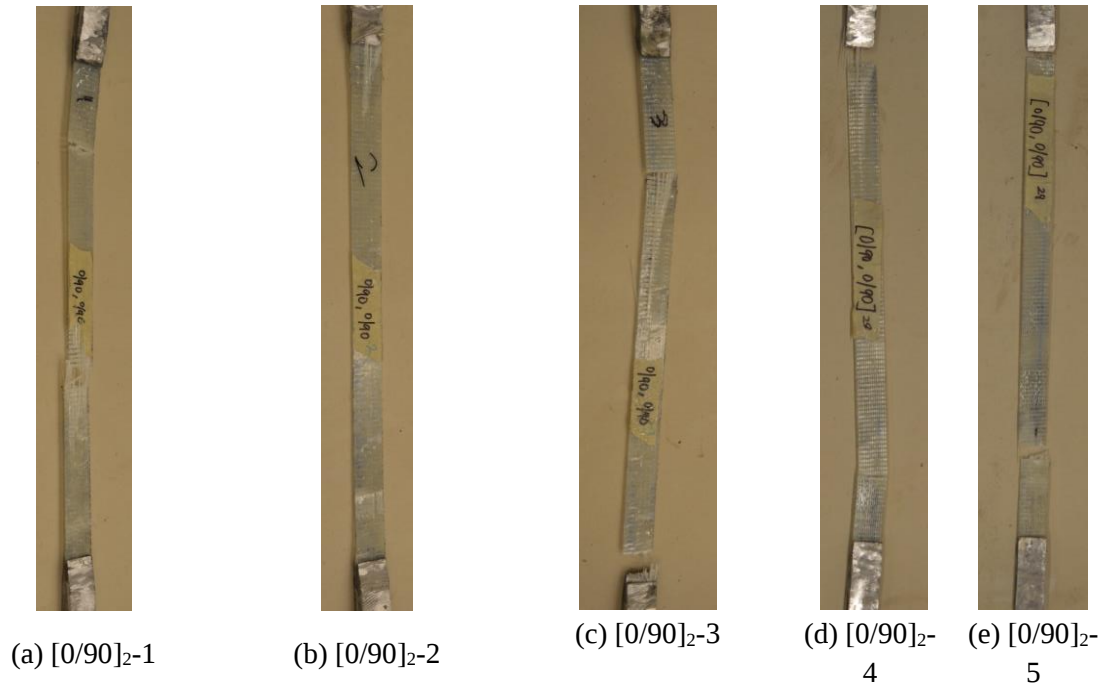


Figure B.10: Representative failure modes for GFRP $[0/90]_2$

Simpson StrongTie – GFRP - $[0/90, \pm 45]$

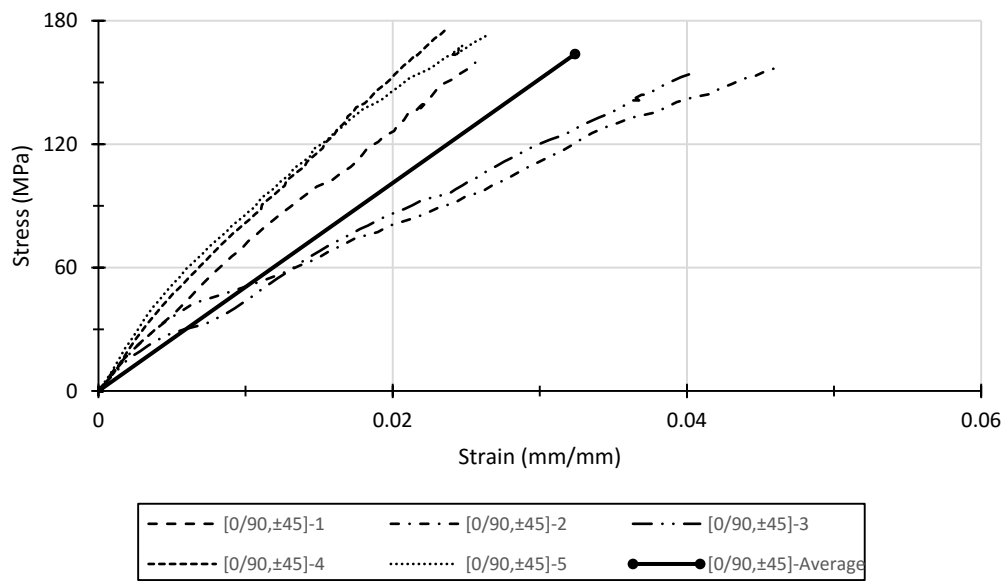
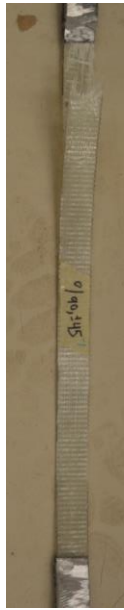


Figure B.11: Stress-strain relationship for GFRP $[0/90, \pm 45]$

Table B.6: Summary of results for GFRP [0/90,±45]

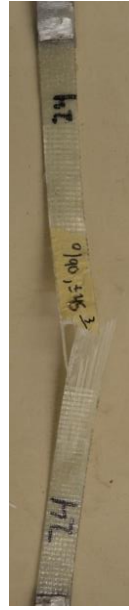
Specimen	[0/90,±45]					Average	Std. Dev.	COV
	1	2	3	4	5			
MOE (MPa)	6,439	3,659	4,213	8,016	8,192	6,104	1,881	0.31
f_{FRP} (MPa)	159.6	157.2	153.9	175.5	172.7	163.8	8.7	0.05
ε_{FRP} (mm/mm)	2.56E-02	4.62E-02	4.01E-02	2.36E-02	2.65E-02	3.24E-02	9.04E-03	0.28
t (mm)	1.67	1.65	1.65	1.62	1.70	1.66	0.03	0.02



(a) [0/90,±45]-1



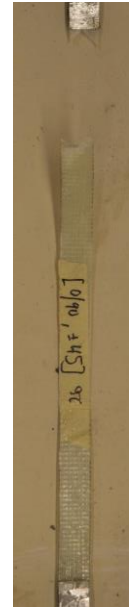
(b) [0/90,±45]-2



(c) [0/90,±45]-3



(d) [0/90,±45]-4



(e) [0/90,±45]-5

Figure B.12: Representative failure modes for GFRP [0/90,±45]

Simpson StrongTie – GFRP - $[\pm 45]_4$

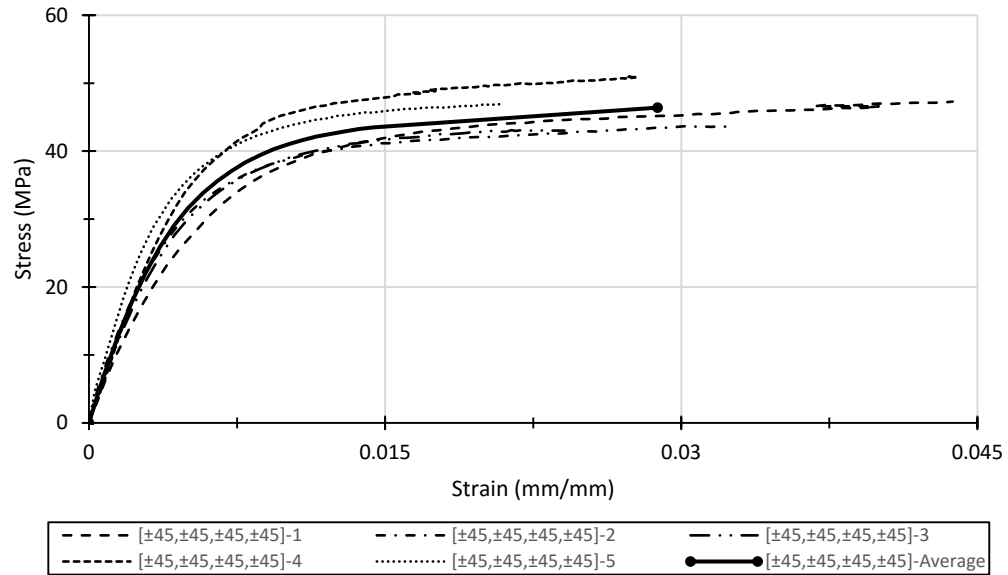


Figure B.13: Stress-strain relationship for GFRP $[\pm 45]_4$

Table B.7: Summary of results for GFRP $[\pm 45]_4$

Specimen	$[\pm 45]_4$					Average	Std. Dev.	COV
	1	2	3	4	5			
MOE (MPa)	6,268	7,645	7,616	7,994	10,073	7,919	1,228	0.16
f_{FRP} (MPa)	47.3	43.7	43.1	51.0	47.0	46.4	2.9	0.06
ϵ_{FRP} (mm/mm)	4.37E-02	3.01E-02	2.14E-02	2.77E-02	2.10E-02	0.0288	8.27E-03	0.29
t (mm)	2.92	3.03	2.91	2.91	2.85	2.92	0.06	0.02

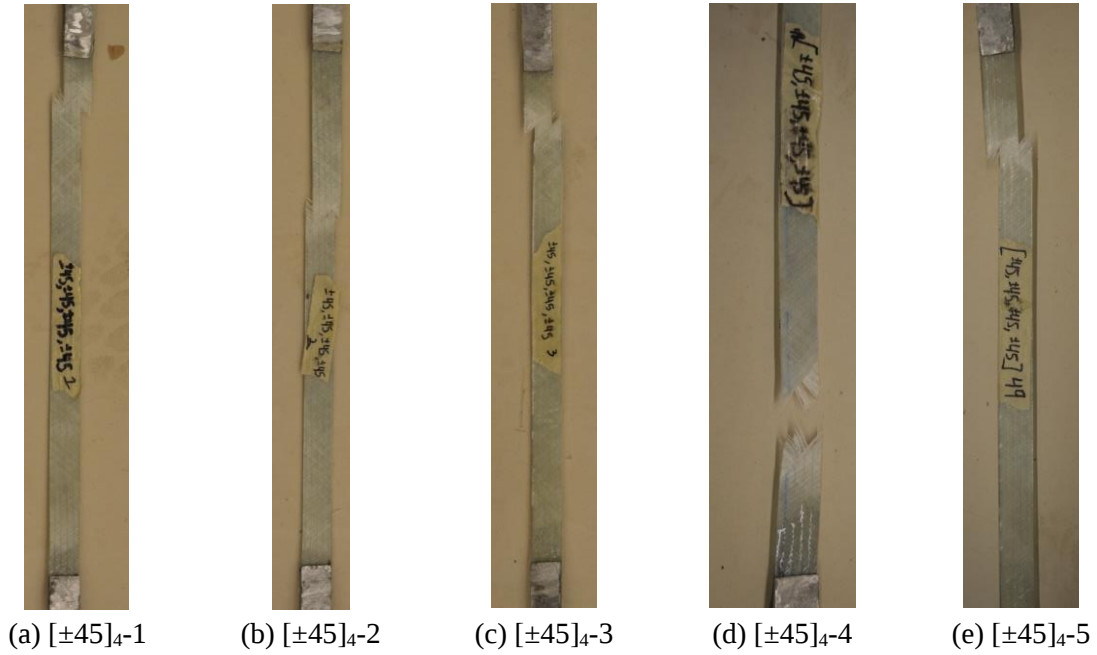


Figure B.14: Representative failure modes for GFRP $[\pm 45]_4$

Simpson StrongTie – GFRP – $[0][0/90]_2$

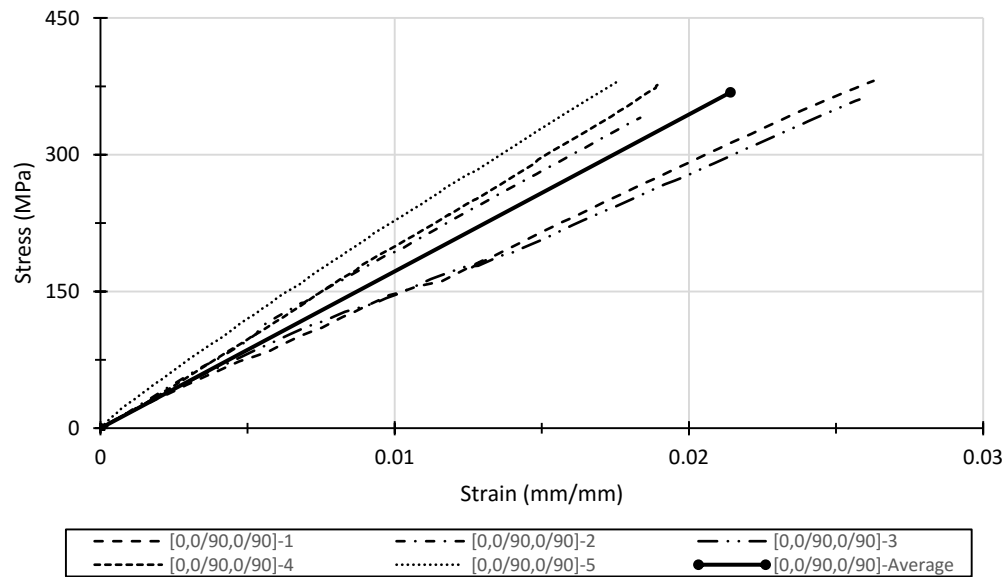


Figure B.15: Stress-strain relationship for GFRP $[0][0/90]_2$

Table B.8: Summary of results for GFRP [0][0/90]₂

Specimen	[0][0/90] ₂					Average	Std. Dev.	COV
	1	2	3	4	5			
MOE (MPa)	14,392	19,540	14,238	19,796	23,051	18,203	3,408	0.19
f_{FRP} (MPa)	381.0	340.5	360.6	378.4	381.6	368.4	15.9	0.04
ε_{FRP} (mm/mm)	2.63E-02	1.83E-02	2.58E-02	1.90E-02	1.76E-02	2.14E-02	3.80E-03	0.18
t (mm)	2.56	2.52	2.58	2.41	2.45	2.50	0.06	0.03

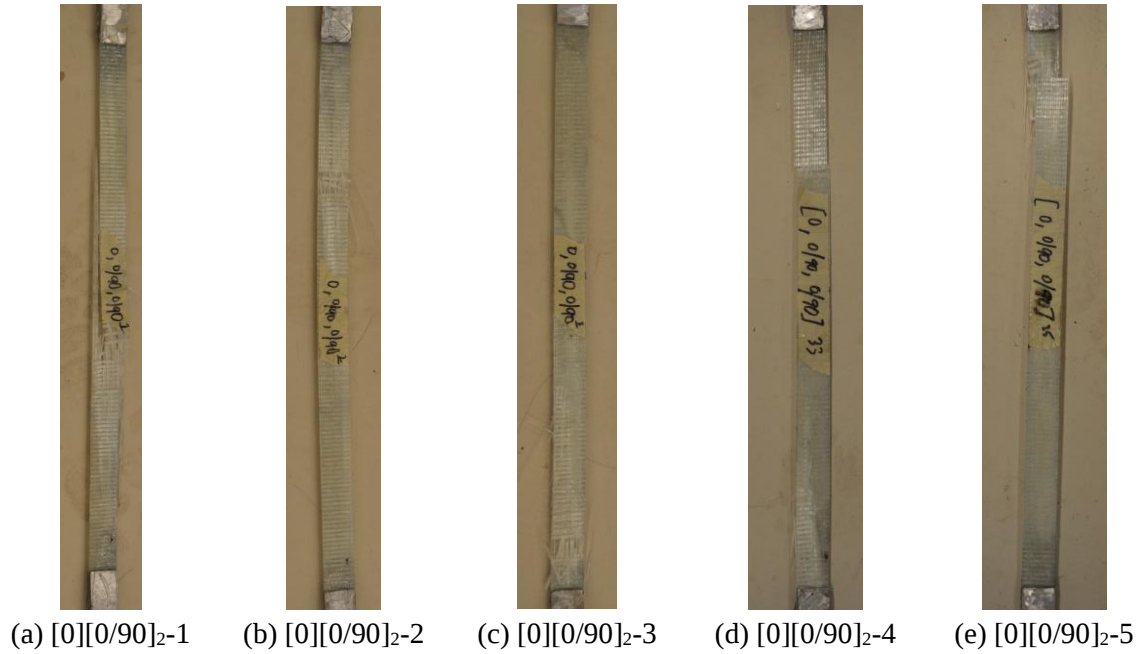


Figure B.16: Representative failure modes for GFRP [0][0/90]₂

Simpson StrongTie – GFRP – $[0]_3[0/90]_2$

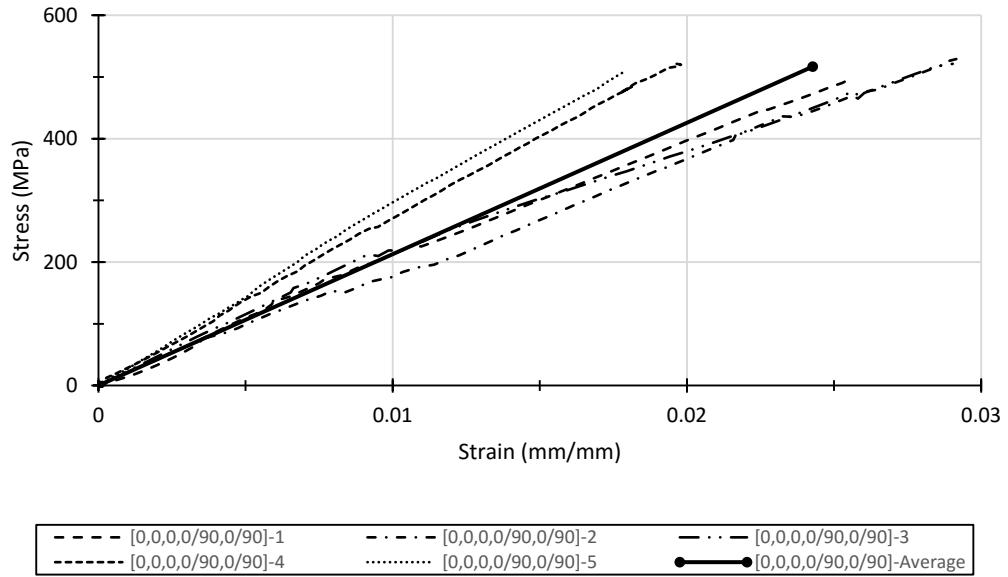


Figure B.17: Stress-strain relationship for GFRP $[0]_3[0/90]_2$

Table B.9: Summary of results for GFRP $[0]_3[0/90]_2$

Specimen	$[0]_3[0/90]_2$					Average	Std. Dev.	COV
	1	2	3	4	5			
MOE (MPa)	20,198	18,311	19,666	27,194	29,502	22,974	4,490	0.20
f_{FRP} (MPa)	497.4	529.0	521.2	524.8	511.3	516.8	11.3	0.02
ϵ_{FRP} (mm/mm)	2.56E-02	2.92E-02	2.90E-02	1.97E-02	1.79E-02	2.43E-02	4.68E-03	0.19
t (mm)	3.96	4.08	4.03	4.19	4.19	4.09	0.09	0.02



(a) $[0]_3[0/90]_2$ -1 (b) $[0]_3[0/90]_2$ -2 (c) $[0]_3[0/90]_2$ -3 (d) $[0]_3[0/90]_2$ -4 (e) $[0]_3[0/90]_2$ -5

Figure B.18: Representative failure modes for GFRP $[0]_3[0/90]_2$

Simpson StrongTie – GFRP – $[0]_3[0/90]_2$

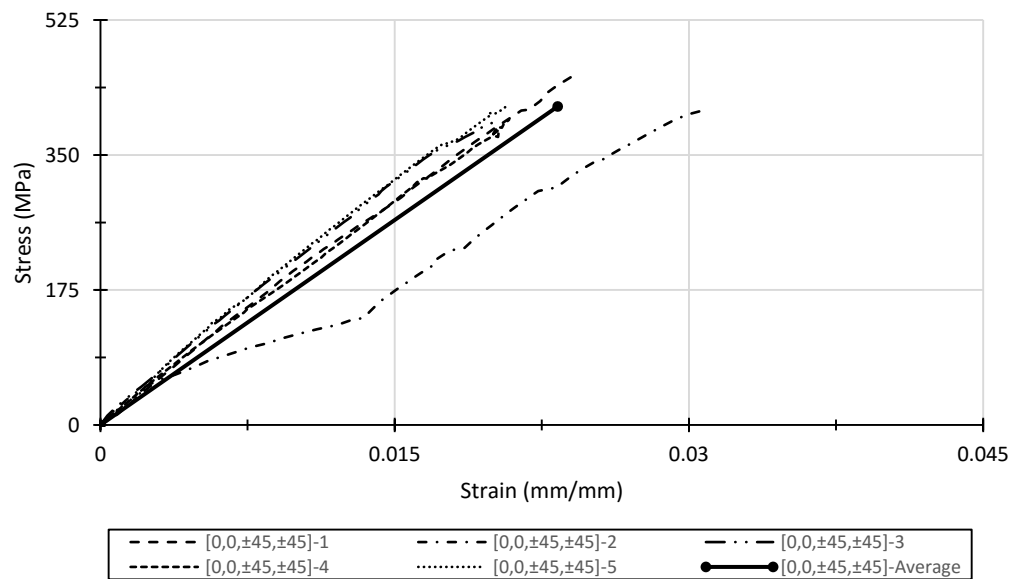


Figure B.19: Stress-strain relationship for GFRP $[0]_2[±45]_2$

Table B.10: Summary of results for GFRP $[0]_2[\pm 45]_2$

Specimen	$[0]_2[\pm 45]_2$					Average	Std. Dev.	COV
	1	2	3	4	5			
MOE (MPa)	19,235	13,074	21,615	19,506	21,897	19,065	3,182	0.17
f_{FRP} (MPa)	451.4	407.7	393.9	395.7	415.8	412.9	20.9	0.05
ϵ_{FRP} (mm/mm)	2.40E-02	3.07E-02	2.02E-02	2.08E-02	2.08E-02	2.33E-02	3.93E-03	0.17
t (mm)	2.92	3.08	3.24	3.16	3.14	3.1	0.11	0.04



Figure B.20: Representative failure modes for GFRP $[0]_2[\pm 45]_2$

Simpson StrongTie – GFRP – [0,0/90,±45]

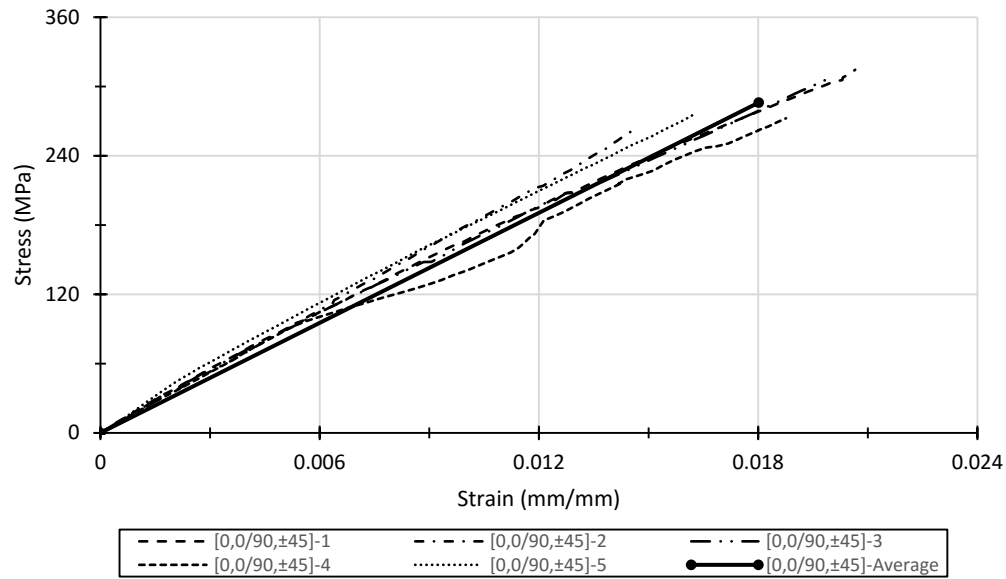


Figure B.21: Stress-strain relationship for GFRP [0,0/90,±45]

Table B.11: Summary of results for GFRP [0,0/90,±45]

Specimen	[0,0/90,±45]					Average	Std. Dev.	COV
	1	2	3	4	5			
MOE (MPa)	15,844	17,856	15,870	14,902	18,297	16,554	1,299	0.08
f_{FRP} (MPa)	314.5	261.3	306.0	272.5	276.2	286.1	20.5	0.07
ε_{FRP} (mm/mm)	2.07E-02	1.45E-02	1.99E-02	1.88E-02	1.63E-02	1.80E-02	2.29E-03	0.13
t (mm)	2.47	2.63	2.43	2.61	2.64	2.56	0.08	0.03

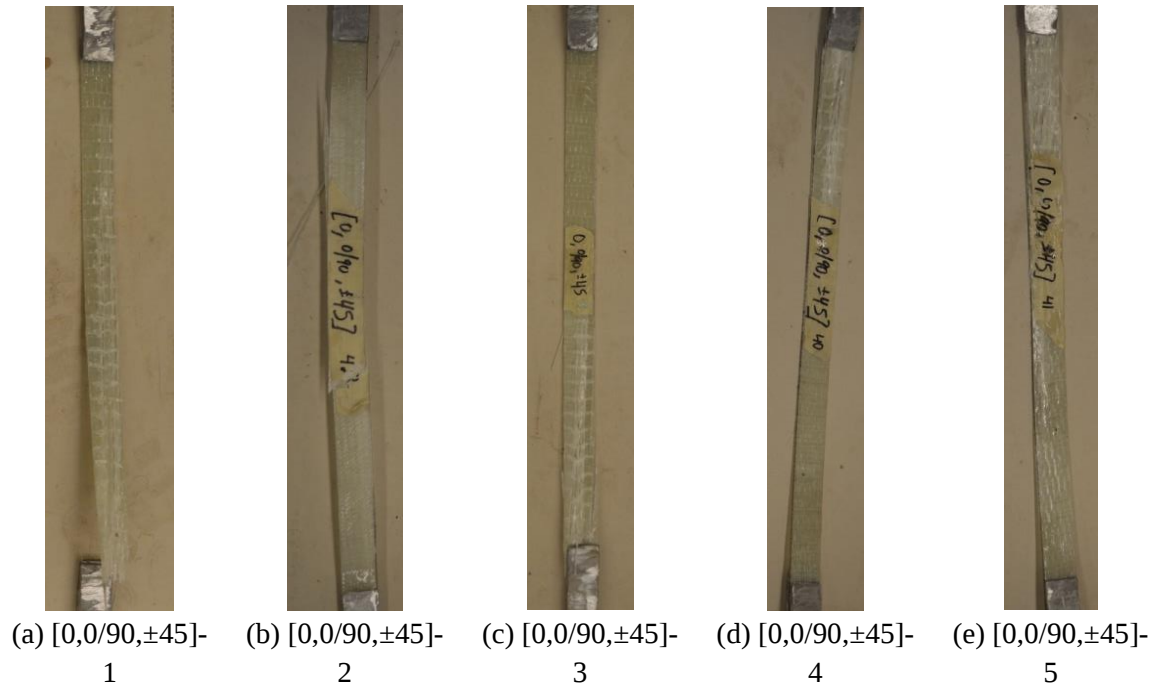


Figure B.22: Representative failure modes for GFRP [0,0/90,±45]

Appendix C – Test Results for Unretrofitted Glulam Beams

Table C.1: Static test results

Specimen	R_{max}^a (kN)	Δ_{Rmax}^b (mm)	t_{Rmax}^c (s)	Δ_{max}^d (mm)	$t_{\Delta-max}^e$ (s)	$\epsilon_{f-T}^f \times 10^{-4}$ (mm/mm)	t_{ff-T}^g (s)	$\dot{\epsilon}^h$	$\epsilon_{f-C}^i \times 10^{-4}$ (mm/mm)	t_{ff-C}^j (s)	MOR ^k (MPa)	MOE ^l (MPa)	FJ failure
B1-[80]	104.8	27.2	615.2	27.2	615.2						50.6	8,535	N
B2-[80]	124.1	34.8	374.5	34.8	374.5		No data ¹				59.9	9,090	N
B3-[80]	123.0	32.8	523.3	32.8	523.3						59.4	8,511	N
B4-[80]	102.6	25.6	146.9	25.6	146.9	44.8	149.8	2.99E-05	-145.7	150.1	49.5	7,953	Y _C
B5-[80]	92.7	22.1	149.3	22.1	149.3	39.4	152.2	2.59E-05	-22.4	123.7	44.8	8,256	Y _C
B6-[80]	98.6	24.3	177.0	24.3	177.0	49.0	179.9	2.73E-05	-39.4	179.9	47.6	7,785	N
B7-[80]	99.4	27.5	351.4	27.5	351.4		No data ¹				48.0	8,455	N
B8-[80]	106.9	25.9	175.8	25.9	175.8	52.1	178.7	2.91E-05	-35.7	178.7	51.6	8,827	N
B9-[80]	102.3	28.5	815.6	28.5	815.6						49.4	8,031	N
B10-[80]	111.5	33.5	316.2	33.5	316.2		No data ¹				53.8	7,986	N
B11-[80]	98.1	27.3	387.6	27.3	387.6						48.5	8,623	N
Average		28.1		28.1		46.3			-60.8		51.2	8,368	
Std. Dev.		3.65		3.65		4.77			49.43		4.35	369	
CV		0.13		0.13		0.10			-0.81		0.09	0.04	

^aMaximum resistance

^bDisplacement at maximum resistance

^cTime to maximum resistance

^dMaximum recorded displacement

^eTime to maximum recorded displacement

^fStrain at tensile rupture

^gTime to tensile rupture

^hStrain rate

ⁱMaximum compressive strain

^jTime to maximum compressive strain

^kModulus of rupture

^lModulus of elasticity

¹Bond incompatibility between strain gauges and wood prevented the recording of meaningful data

Table C.1: Static test results (CTN)

Specimen	R_{max}^a (kN)	ΔR_{max}^b (mm)	t_{Rmax}^c (s)	Δ_{max}^d (mm)	$t_{\Delta-max}^e$ (s)	$\epsilon_{f-T}^f \times 10^{-4}$ (mm/mm)	$t_{\epsilon f-T}^g$ (s)	ξ^h	$\epsilon_{f-C}^i \times 10^{-4}$ (mm/mm)	$t_{\epsilon f-C}^j$ (s)	MOR ^k (MPa)	MOE ^l (MPa)	FJ failure
B1-[86]	174.3	19.2	344.0	96.2	464.3	39.1	344.0	1.14E-05	-47.7	381.0	43.6	8,275	Y _C
B2-[86]	162.3	20.0	269.5	89.2	424.5	35.1	268.4	1.31E-05	-52.8	387.2	40.7	8,079	Y _S
B3-[86]	159.9	17.5	280.2	27.4	306.6	35.7	306.6	1.17E-05	-31.7	312.0	40.1	8,296	Y _S
B4-[86]	166.1	18.8	317.9	123.2	497.9	34.1	317.7	1.07E-05	-62.9	484.1	41.6	7,451	N
Average		18.9		84.0		36.0			-48.8		41.5	8,025	
Std. Dev.		0.92		38.23		1.88			11.27		1.31	342	
CV		0.05		0.46		0.05			-0.23		0.03	0.04	
B1-[137]	131.2	22.89	500.0	69.1	755.3	32.3	500.0	6.46E-06	-44.9	500.9	42.0	9,362	Y _S
B2-[137]	161.5	25.17	376.6	75.3	621.2	38.2	339.3	1.13E-05	-53.8	542.2	52.0	10,966	Y _S
B3-[137]	132.1	21.36	243.8	105.3	534.5	38.5	243.8	1.58E-05	-41.5	283.6	42.8	9,739	Y _S
B4-[137]	141.7	25.51	301.4	96.4	428.6	54.3	301.7	1.80E-05	-69.2	389.7	45.8	10,226	Y _S
Average		23.7		86.5		40.8			-52.4		45.6	10,073	
Std. Dev.		1.70		14.83		8.15			9.59		3.49	536	
CV		0.07		0.17		0.20			-0.18		0.08	0.05	

^aMaximum resistance^bDisplacement at maximum resistance^cTime to maximum resistance^dMaximum recorded displacement^eTime to maximum recorded displacement^fStrain at tensile rupture^gTime to tensile rupture^hStrain rateⁱMaximum compressive strain^jTime to maximum compressive strain^kModulus of rupture^lModulus of elasticity¹Bond incompatibility between strain gauges and wood prevented the recording of meaningful data

Table C.2: Dynamic beams elastic tests

Specimen	P _R ^a (kPa)	I _R ^b (kPa-ms)	Δ _{max} ^c (mm)	t _{f-max} ^d (ms)	ε _f ^e x 10 ⁻⁴ (mm/mm)	ε̇ ^f (s ⁻¹)	R _{max-d} ^g (kN)	MOR _d ^h (MPa)	MOE _d ⁱ (MPa)
B12.1-[80]	14.2	158.2	15.6	26	24.9	0.1	48.9	26.3	7,868
B13.1-[80]	10.7	122.9	11.2	25.2	15.2	0.06	34.3	18.4	7,894
B14.1-[80]	9.2	60.1	6.4	23.4	7.9	0.03	17.1	9.2	6,585
B15.1-[80]	18.9	188.1	18.5	24	31.6	0.13	55	29.5	7,724
B17.1-[80]	28.3	231.8	26.1	21.2	35	0.17	81.7	43.9	9,525
B5.1-[86]	14.9	126	6.2	24	8	0.03	53.3	13.7	8,123
B5.2-[86]	45.9	448.6	21.7	17.2	37.8	0.22	180.8	46.5	8,255
B6.1-[86]	13.2	140.5	6.2	20	8.2	0.04	55.3	14.2	8,581
B7.1-[86]	16.8	155	7.4	19.8	10.8	0.05	59.7	15.3	8,241
B8.1-[86]	6.9	36.4			No data ¹		32.2	8.3	7,216
B5.1-[137]	11	124.1	7.6	21.4	9.3	0.04	35.6	11.8	8,197
B6.1-[137]	13.1	121.3	7	24.8	10.7	0.04	41.5	13.7	8,906
B7.1-[137]	11.7	125.7	8.4	21.6	10.4	0.05	47.2	15.6	9,207
B8.1-[137]	6.2	60.8				No data ¹			
B9.1-[137]	10.9	105.5			No data ¹		44.1	14.6	9,511

^aMaximum reflected pressure^bReflected impulse^cMaximum recorded displacement^dTime to maximum displacement^eMaximum tensile strain^fStrain rate^gMaximum dynamic resistance^hDynamic modulus of ruptureⁱDynamic modulus of elasticity¹Noise interference causing experimental time history to be unusable

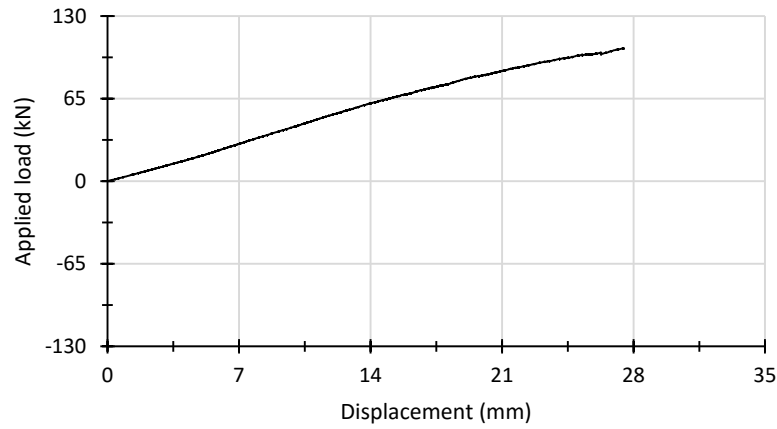
Table C.3: Dynamic beams destructive test results

Specimen	P _R ^a (kPa)	I _R ^b (kPa-ms)	Δ _{max} ^c (mm)	t _{Δ-max} ^d (ms)	R _{max-d} ^e (kN)	Δ _{Rmax} ^f (mm)	t _{Rmax-d} ^g (ms)	Δ _{ef-t} ^h (mm)	ε _{f-T} ⁱ x 10 ⁻⁴ (mm/mm)	t _{ef-T} ^j (ms)	ε̇ ^k (s ⁻¹)	ε _{f-C} ^l x 10 ⁻⁴ (mm/mm)	t _{ef-C} ^m (ms)	MOR _d ⁿ (MPa)	MOE _d ^o (MPa)	FJ Failure
B12.2-[80]	40.7	369.5	132.7	45.0	109.8	34.4	16.6	32.7	47.9	16.0	0.30	-61.6	43.6	59.0	9,594	N
B13.2-[80]	42.7	397.1	72.6	25.2	107.0	29.7	15.4	28.6	42.0	15.0	0.28	-51.9	33.4	57.5	9,373	N
B14.2-[80]	41.4	333.8	37.8	30.2	107.1	32.5	20.4	32.2	45.6	19.8	0.23	-33.0	20.4	57.5	8,877	N
B15.2-[80]	40.0	372.6	115.5	42.2	98.5	31.6	15.4	30.0	57.0	15.0	0.38	-50.8	29.0	53.0	9,279	N
B16.1-[80]	52.1	390.5	183.9	40.0	104.9	44.6	15.4	22.4	28.5	11.6	0.25	-44.4	12.4	56.4	10,978	N
B17.2-[80]	44.7	328.4	137.9	50.4	111.5	30.5	15.4	34.8	28.9	17.2	0.17	-56.0	12.8	59.9	9,925	N
B18.1-[80]	49.3	337.0	111.2	51.0	101.8	35.7	15.4	35.7	59.4	15.4	0.39	-70.8	112.0	54.7	8,251	N
B19.1-[80]	54.5	353.4	94.4	21.2	88.5	21.9	11.6	20.0	34.6	11.2	0.31	-37.9	12.4	47.6	8,969	Y _C
B20.1-[80]	54.8	347.0	159.8	46.2	82.9	20.8	11.9	23.2	65.3	12.8	0.51	-42.4	46.2	44.6	12,161	Y _C
B21.1-[80]	58.2	292.6	99.7	36.2	83.2	27.4	15.4	26.9	42.5	15.0	0.28	-44.5	24.6	44.7	8,107	Y _C
		Average	114.5		99.5	30.9		28.7	45.2			-49.3		53.5	9,551	
		Std. Dev.	40.0		10.3	6.5		5.2	12.0			10.8		5.5	1,169	
		CV	0.350		0.103	0.210		0.180	0.265			-0.218		0.103	0.122	
^a Reflected pressure				^b Reflected impulse				^c Maximum recorded displacement				^d Time to maximum recorded displacement				
^e Maximum dynamic resistance				^f Deflection at beam maximum resistance				^g Time to maximum resistance				^h Deflection at tensile rupture				
ⁱ Strain at tensile rupture				^j Time to tensile rupture				^k Strain rate				^l Maximum compressive strain				
^m Time to maximum compressive strain				ⁿ Dynamic modulus of rupture				^o Dynamic modulus of elasticity								

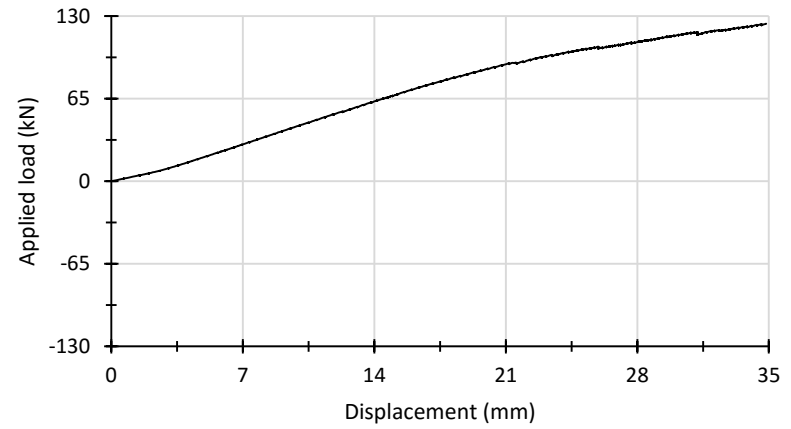
Table C.3: Dynamic beams destructive test results (CTN)

Specimen	P _R ^a (kPa)	I _R ^b (kPa-ms)	Δ _{max} ^c (mm)	t _{Δ-max} ^d (ms)	R _{max-d} ^e (kN)	Δ _{Rmax} ^f (mm)	t _{Rmax-d} ^g (ms)	Δ _{ef-t} ^h (mm)	ε _{f-T} ⁱ x 10 ⁻⁴ (mm/mm)	t _{ef-T} ^j (ms)	ε̇ ^k (s ⁻¹)	ε _{f-C} ^l x 10 ⁻⁴ (mm/mm)	t _{ef-C} ^m (ms)	MOR _d ⁿ (MPa)	MOE _d ^o (MPa)	FJ Failure
B5.3-[86]	48.7	549.7	70.9	46.0	190.9	22.2	16.2	21.7	37.1	15.8	0.23	-	-	49.1	8,366	Y _S
B6.2-[86]	66.4	697.1	114.4	51.6	224.4	26.0	15.8	26.8	42.0	16.2	0.26	-44.9	20.6	57.7	7,863	Y _S
B7.2-[86]	56.7	672.2	92.9	25.4	190.9	23.5	14.8	22.6	39.9	14.4	0.28	-41.2	25.0	49.0	7,781	Y _S
B8.2-[86]	50.9	605.1	119.3	24.2	123.0	16.2	9.2	25.6	24.1	11.8	0.14	-50.6	24.2	31.6	7,216	Y _C
		Average	99.4		182.3	22.0		24.2	35.8			-45.6		46.8	7,807	
		Std. Dev.	19.2		36.9	3.6		2.1	7.0			3.9		9.5	408	
		CV	0.194		0.202	0.163		0.088	0.20			-0.085		0.202	0.052	
B5.2-[137]	54.3	668.5	200.5	86.8	137.6	24.5	13.2	23.8	30.9	13.0	0.24	-57.8	80.0	45.5	8,814	Y _S
B6.2-[137]	50.0	670.3	187.5	83.0	123.0	20.6	12.2	18.7	29.7	11.4	0.26	-65.2	73.2	40.7	9,217	Y _C
B7.2-[137]	54.4	536.5	176.6	87.2	153.6	29.1	14.8	27.6	41.8	14.2	0.29	-55.6	85.4	50.8	9,575	Y _S
B8.2-[137]	50.2	547.5	199.9	92.0	153.1	29.0	16.2	25.9	30.4	15.2	0.20	-51.4	27.2	50.7	9,511	Y _S
B9.2-[137]	53.2	491.9	115.6	53.6	170.1	30.9	15.4	28.7	47.2	14.4	0.33	-25.0	11.6	56.3	9,446	Y _S
		Average	176.0		147.5	26.8		24.9	36.0			-51.0		48.8	9,313	
		Std. Dev.	31.5		16.0	3.8		3.5	7.1			13.7		5.3	277	
		CV	0.179		0.108	0.140		0.142	0.198			-0.269		0.108	0.030	

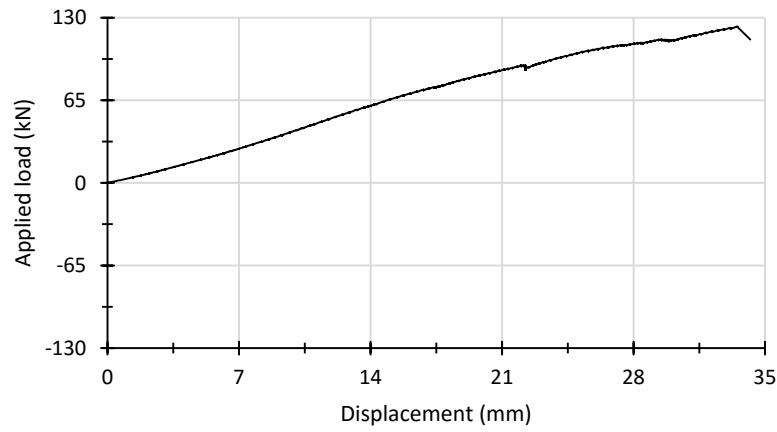
^aReflected pressure^bReflected impulse^cMaximum recorded displacement^dTime to maximum recorded displacement^eMaximum dynamic resistance^fDeflection at beam maximum resistance^gTime to maximum resistance^hDeflection at tensile ruptureⁱStrain at tensile rupture^jTime to tensile rupture^kStrain rate^lMaximum compressive strain^mTime to maximum compressive strainⁿDynamic modulus of rupture^oDynamic modulus of elasticity



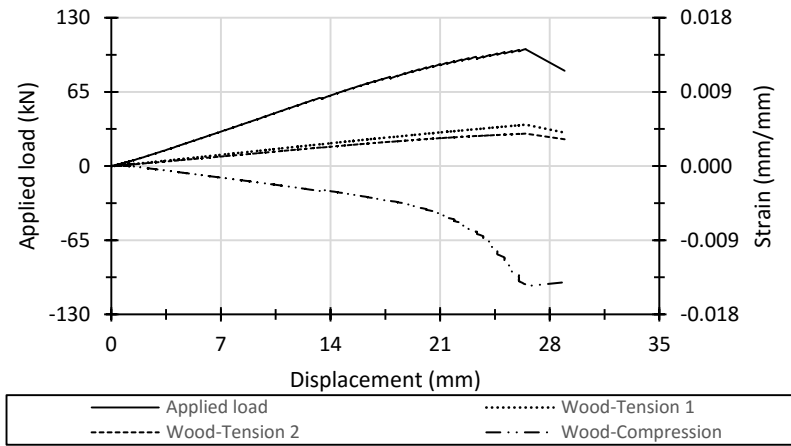
(a) B1-[80]



(b) B2-[80]

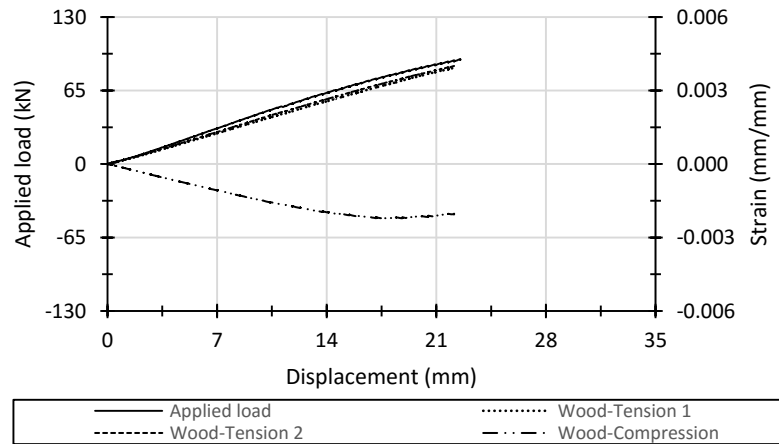


(c) B3-[80]

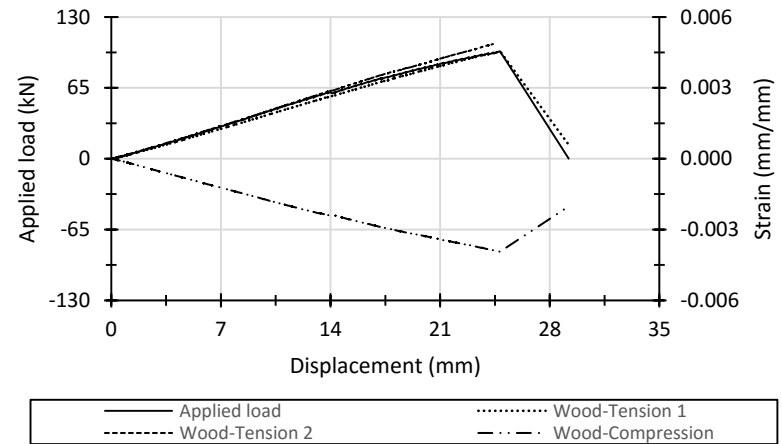


(d) B4-[80]

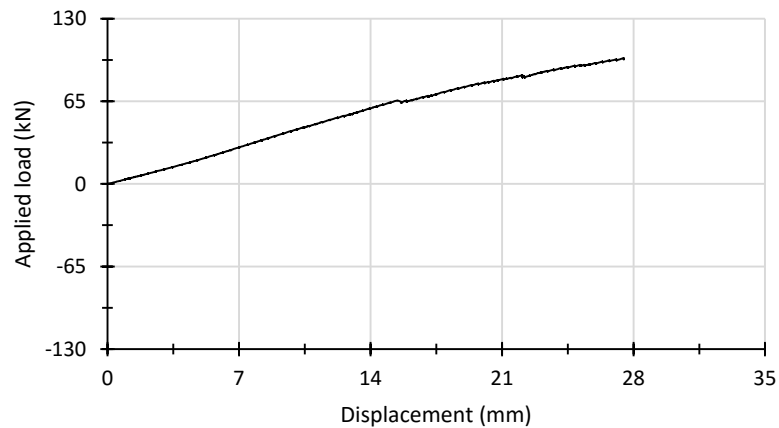
Figure C.1: Static test results for B1-[80] through B4-[80] beams



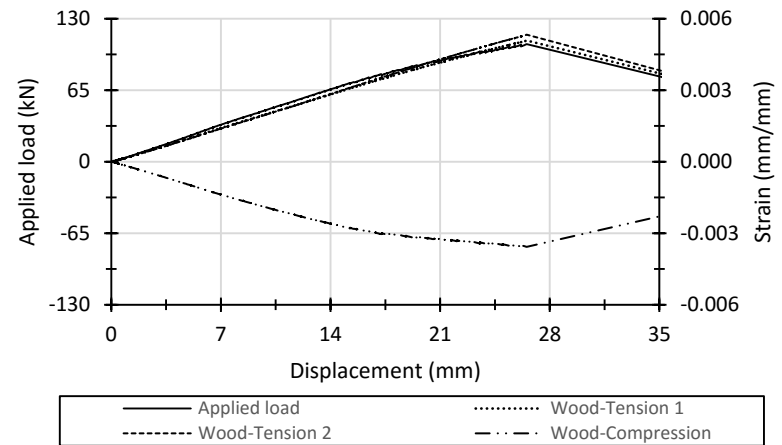
(a) B5-[80]



(b) B6-[80]

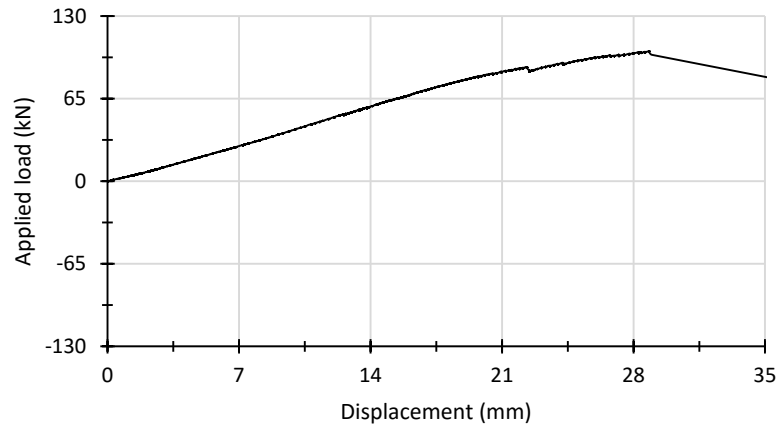


(c) B7-[80]

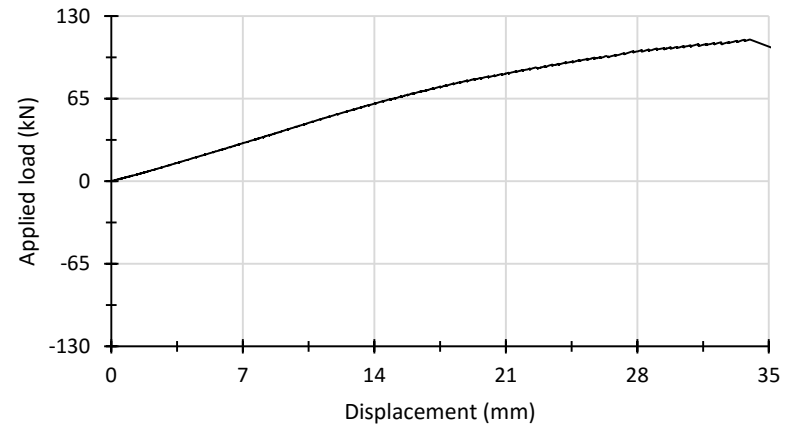


(d) B8-[80]

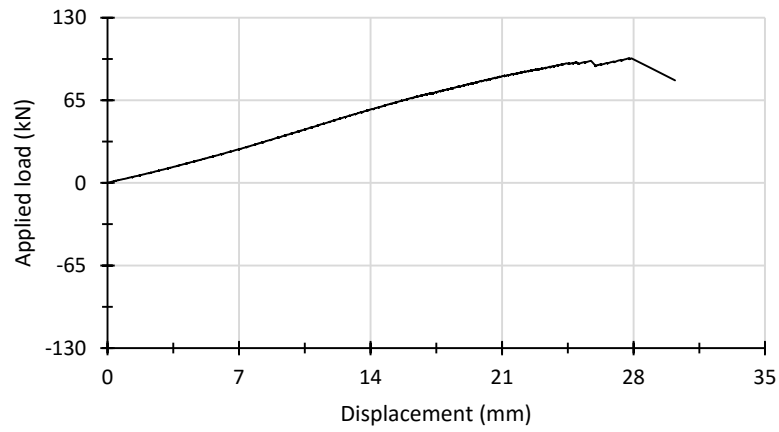
Figure C.2: Static test results for B5-[80] through B8-[80] beams



(a) B9-[80]



(b) B10-[80]



(c) B11-[80]

Figure C.3: Static test results for B9-[80] through B11-[80] beams



(a) B1-[80]



(b) B2-[80]



(c) B3-[80]



(d) B4-[80]



(e) B5-[80]



(f) B6-[80]



(g) B7-[80]



(h) B8-[80]



(i) B9-[80]

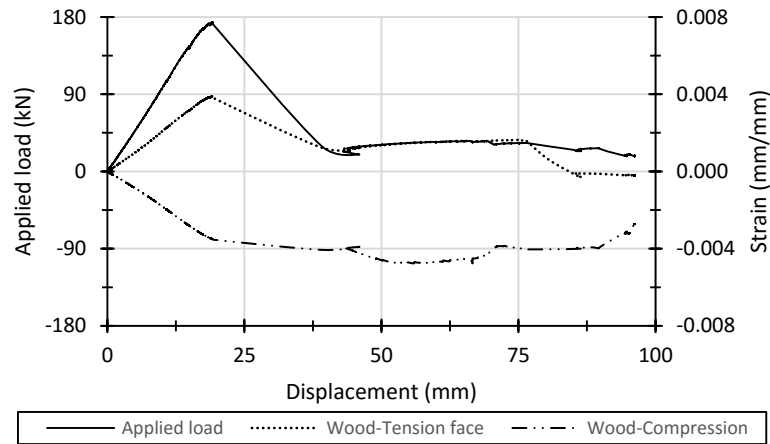


(j) B10-[80]

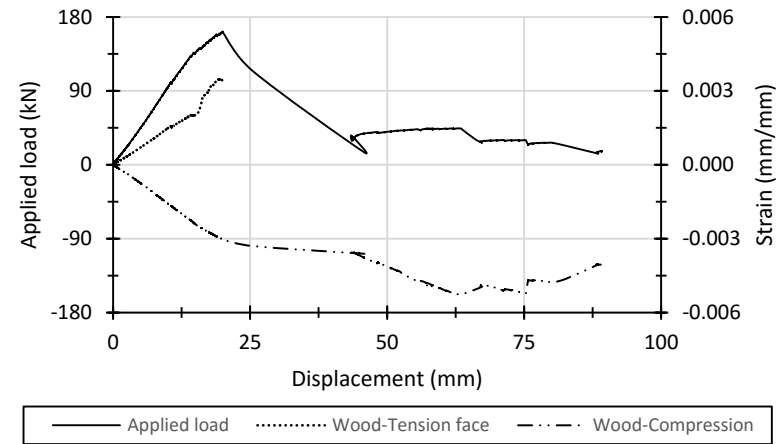


(k) B11-[80]

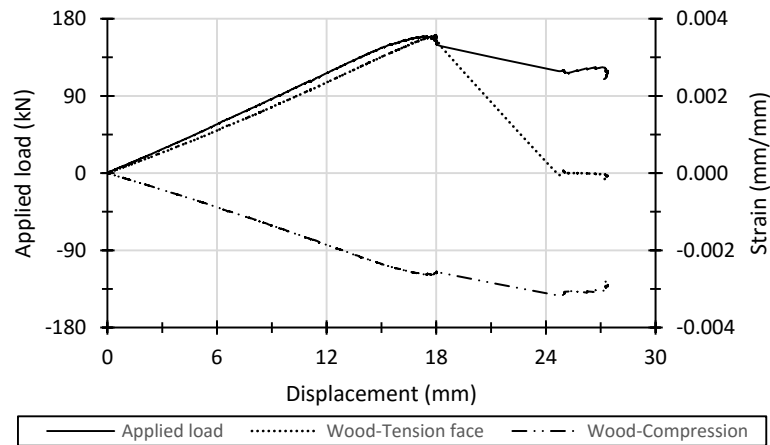
Figure C.4: Static failure modes of B-[80] beams (Tension side facing down)



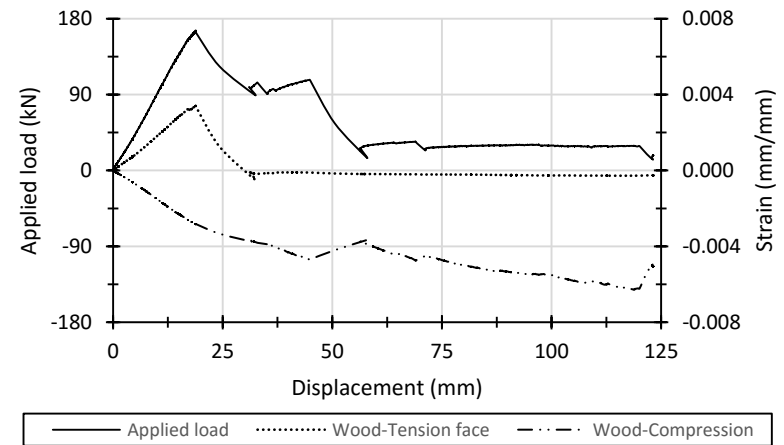
(a) B-1-[86]



(b) B-2-[86]



(c) B-3-[86]



(d) B-4-[86]

Figure C.5: Static test results for B-[86] beams



(a) B1-[86]



(b) B2-[86]

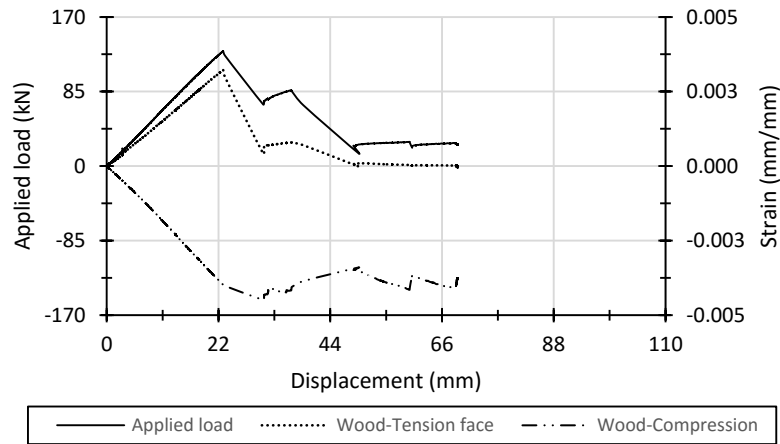


(c) B3-[86]

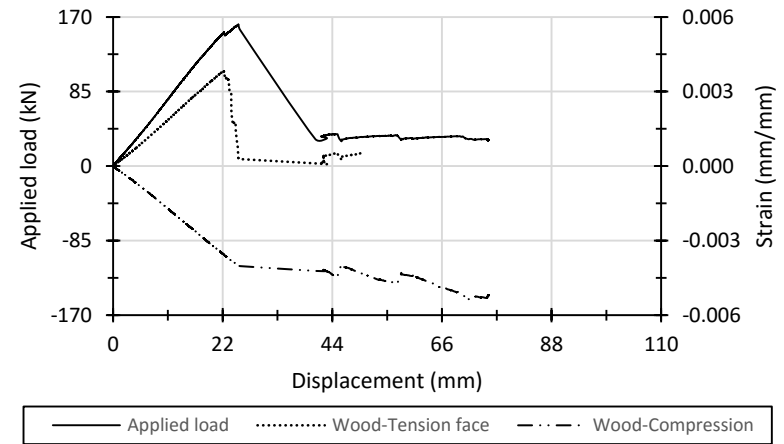


(d) B4-[86]

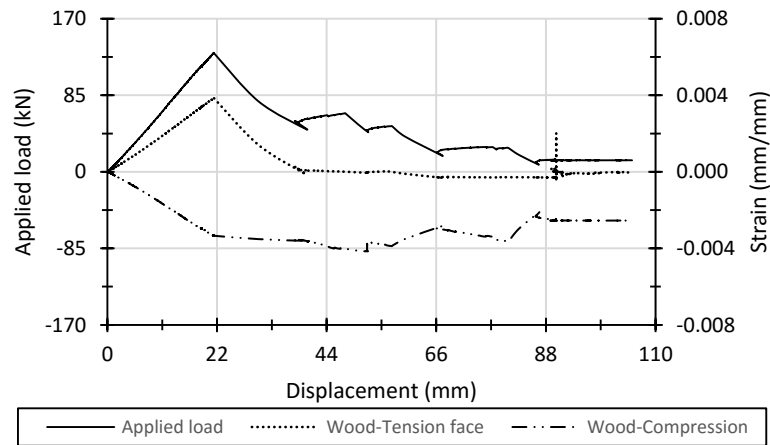
Figure C.6: Static failure modes of B-[86] beams



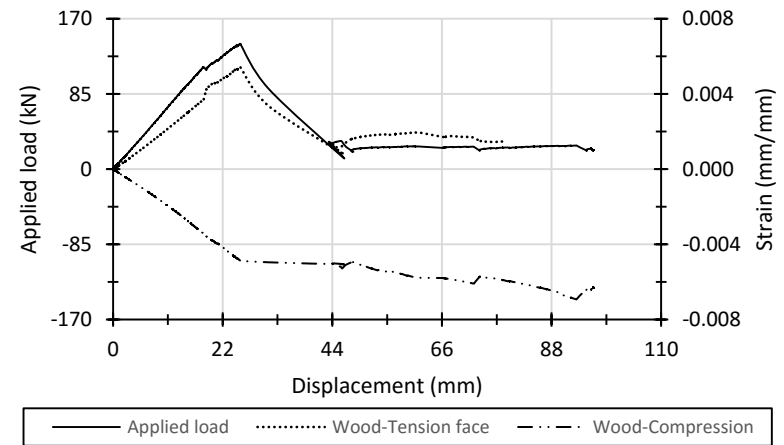
(a) B-1-[137]



(b) B-2-[137]



(c) B-3-[137]



(d) B-4-[137]

Figure C.7: Static test results for B-[137] beams



(a) B1-[137]



(b) B2-[137]

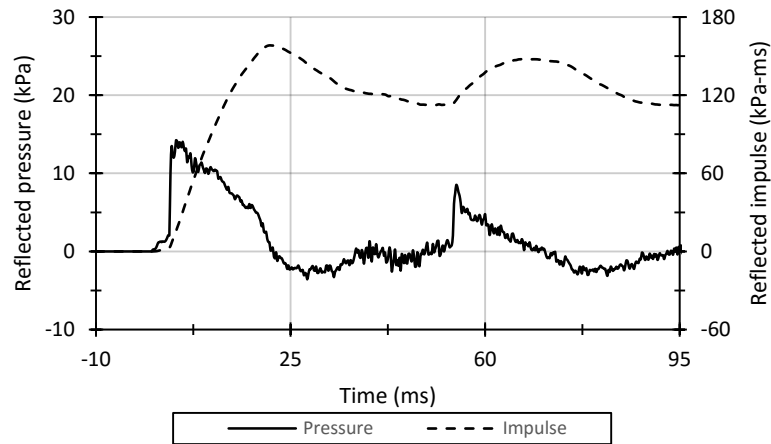


(c) B3-[137]

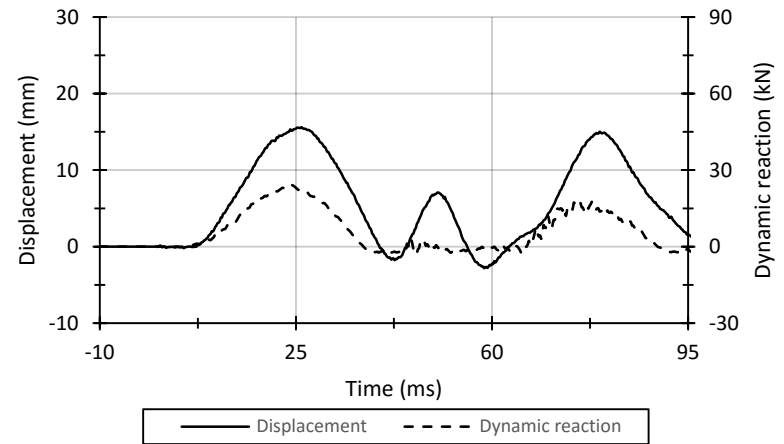


(d) B4-[137]

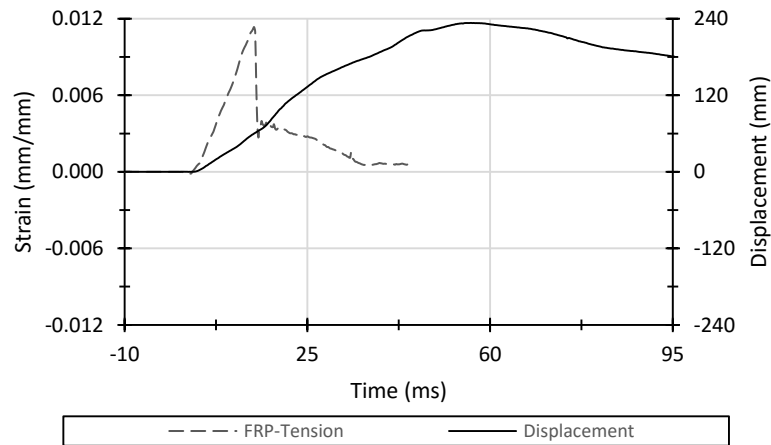
Figure C.8: Static failure modes of B-[137] beams



(a) Pressure and impulse

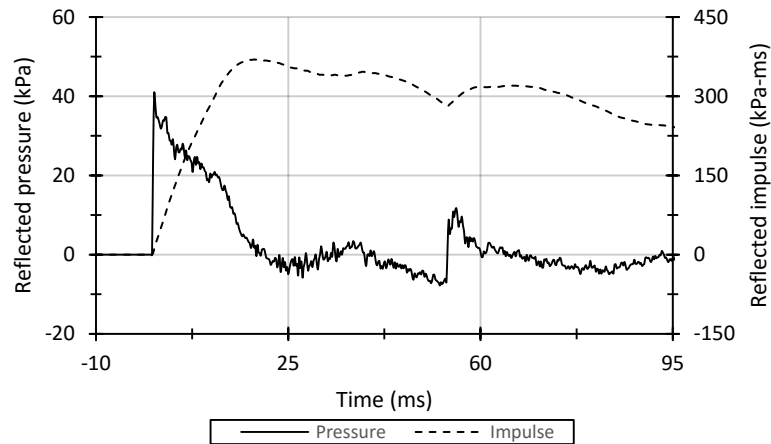


(b) Displacement and reaction

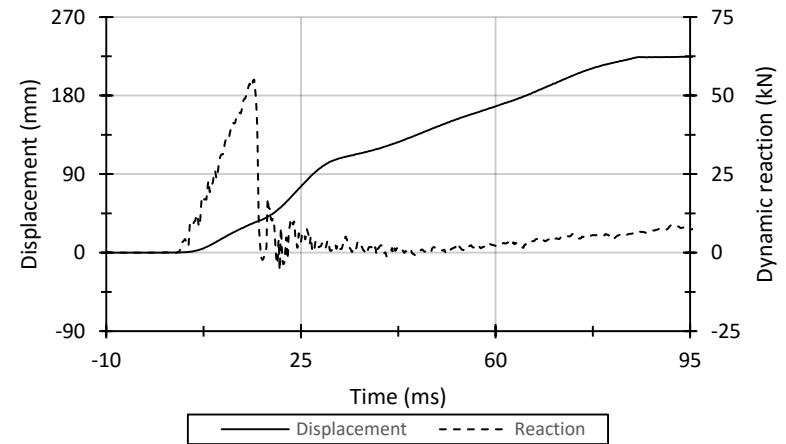


(c) Strain and displacement

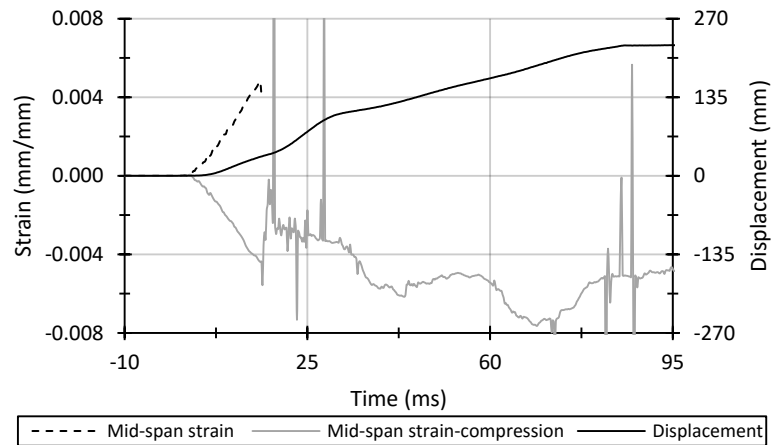
Figure C.9: Dynamic test results for B12.1-[80]



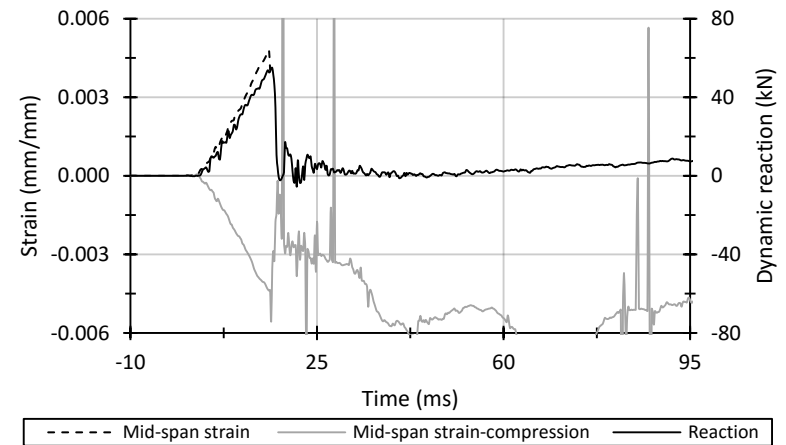
(a) Pressure and impulse



(b) Displacement and reaction

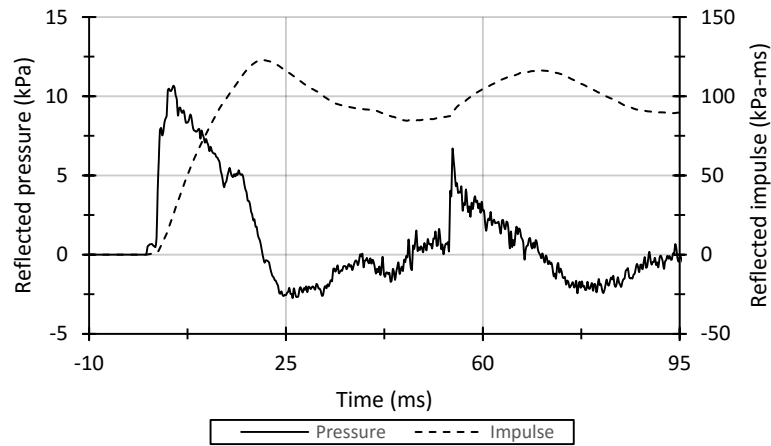


(c) Strain and displacement

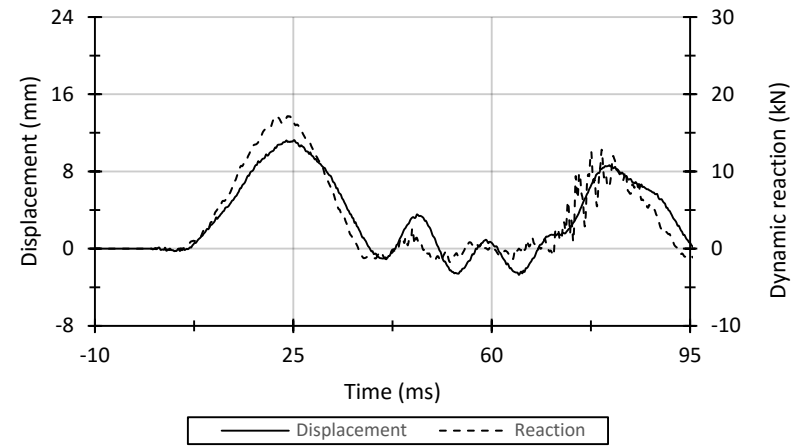


(d) Strain and reaction

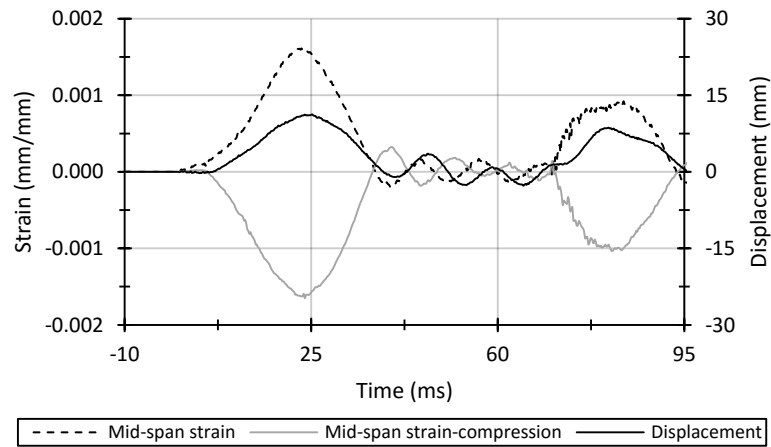
Figure C.10: Dynamic test results for B12.2-[80]



(a) Pressure and impulse

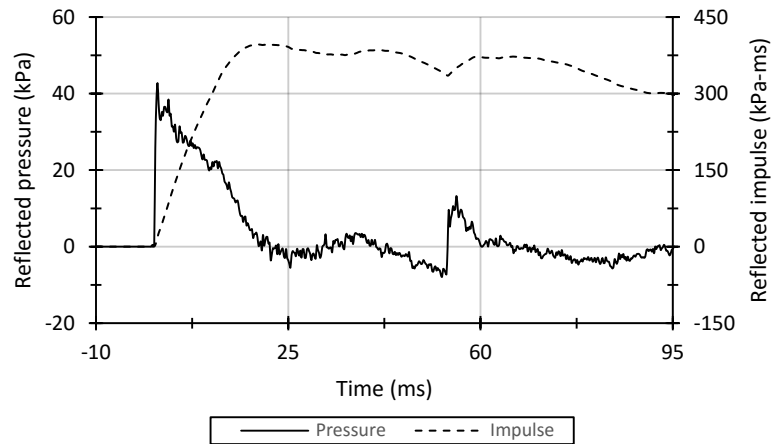


(b) Displacement and reaction

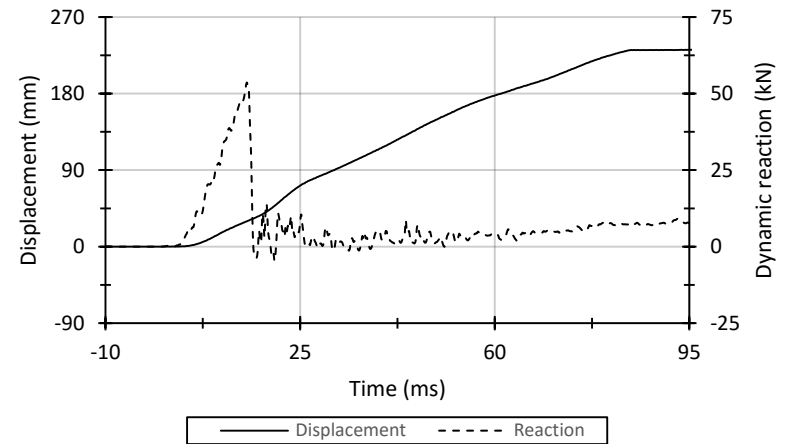


(c) Strain and displacement

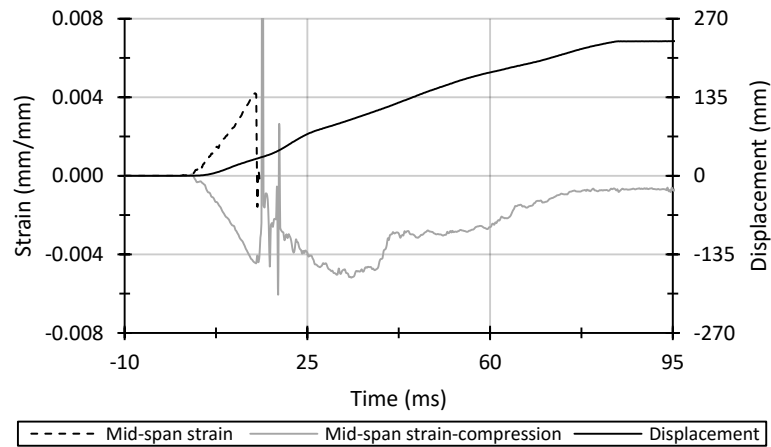
Figure C.11: Dynamic test results for B13.1-[80]



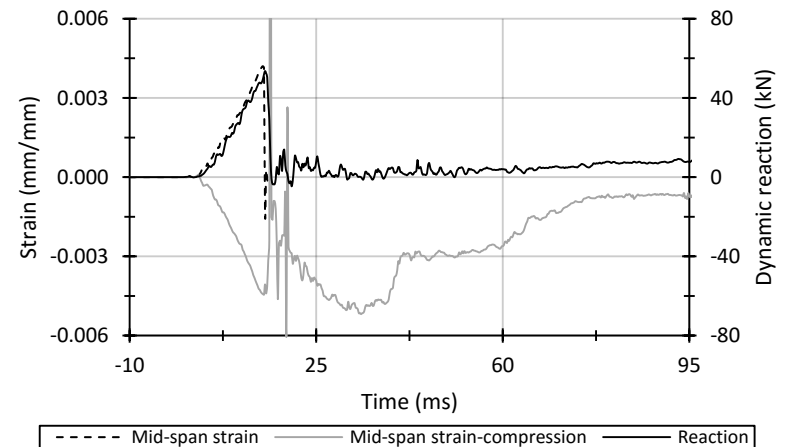
(a) Pressure and impulse



(b) Displacement and reaction

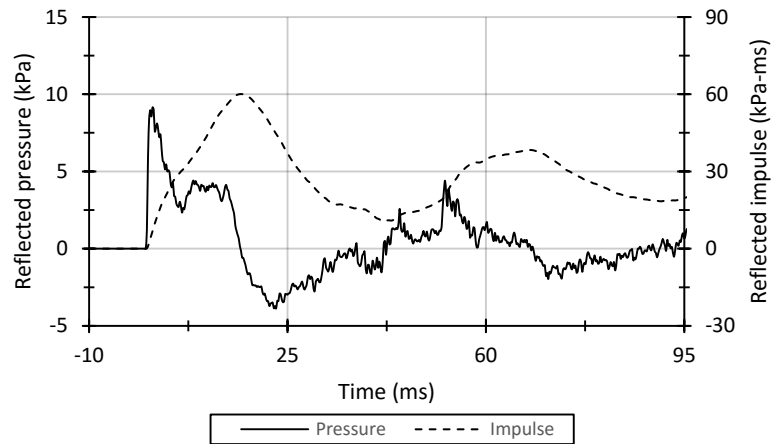


(c) Strain and displacement

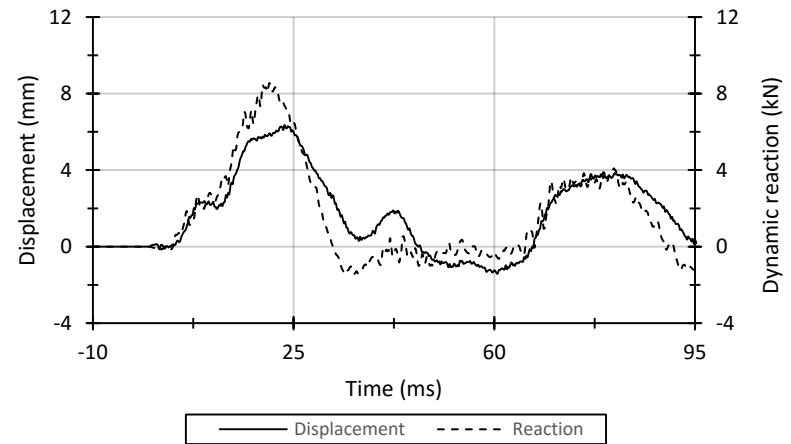


(d) Strain and reaction

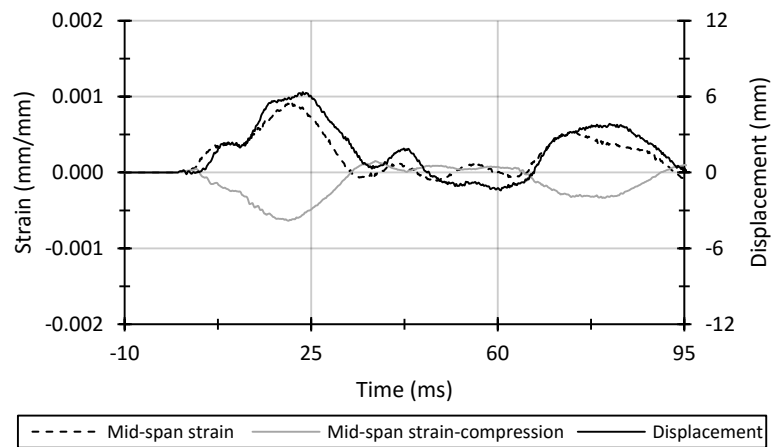
Figure C.12: Dynamic test results for B13.2-[80]



(a) Pressure and impulse

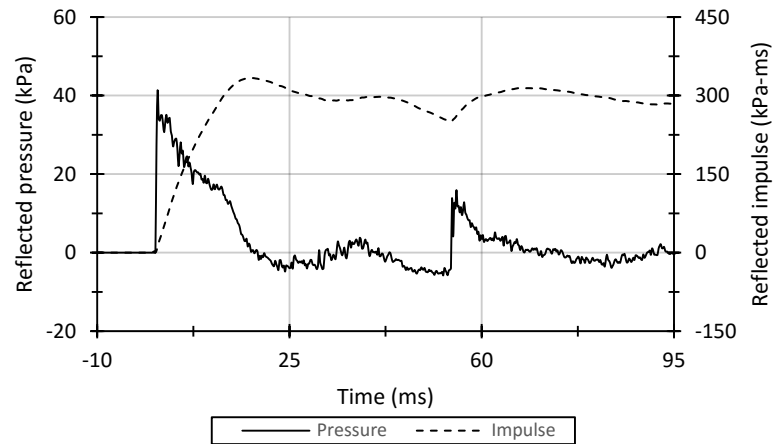


(b) Displacement and reaction

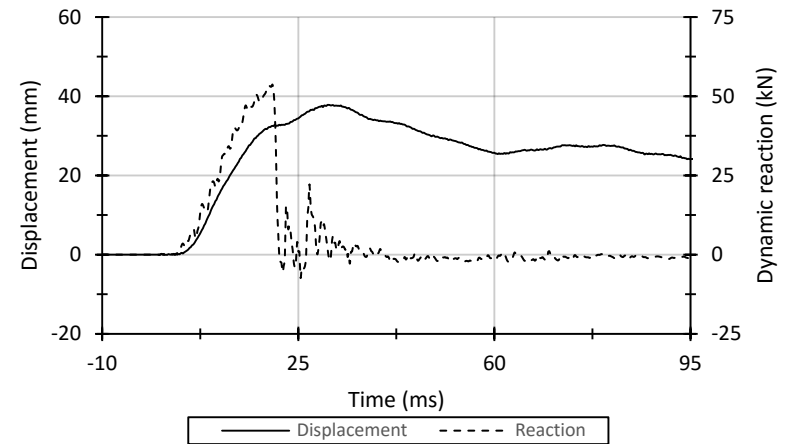


(c) Strain and displacement

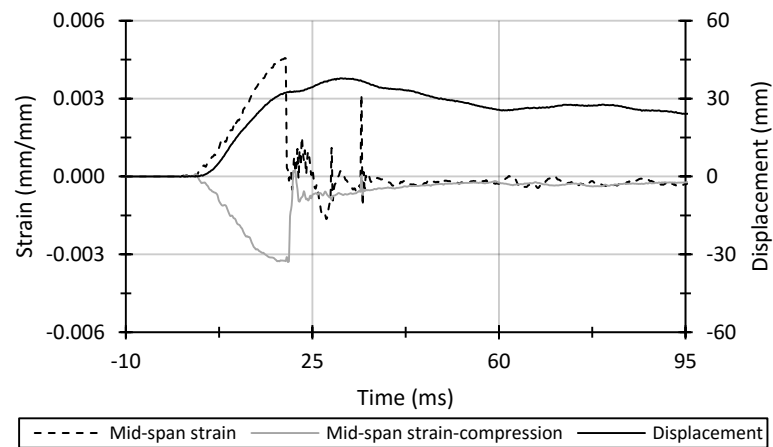
Figure C.13: Dynamic test results for B14.1-[80]



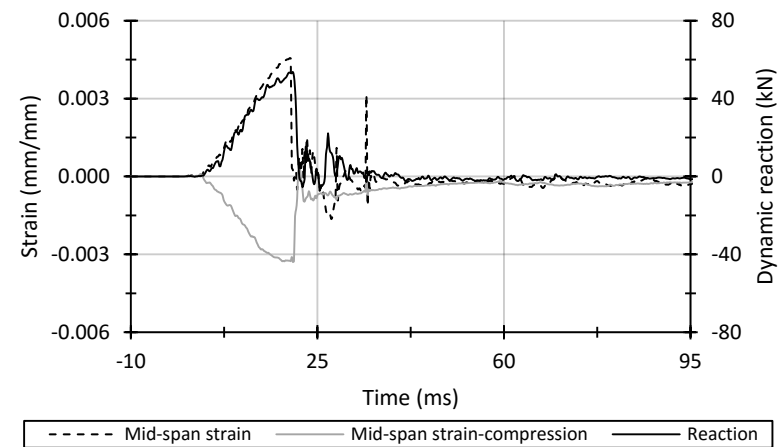
(a) Pressure and impulse



(b) Displacement and reaction

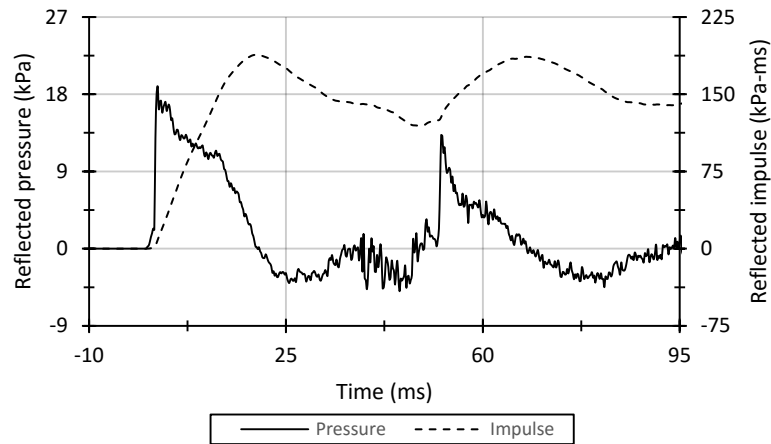


(c) Strain and displacement

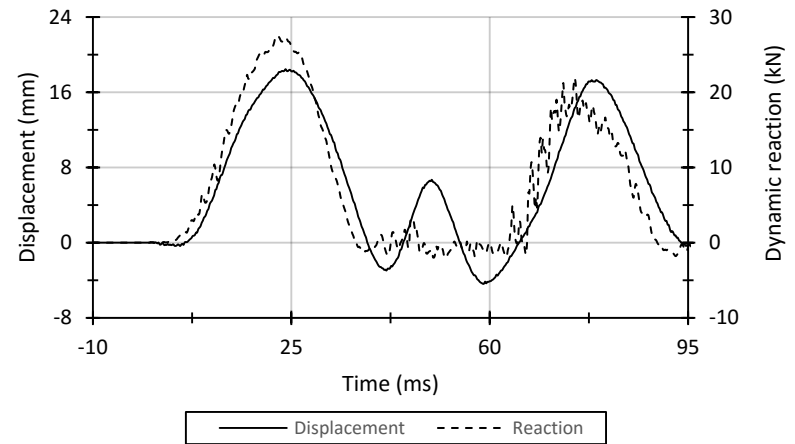


(d) Strain and reaction

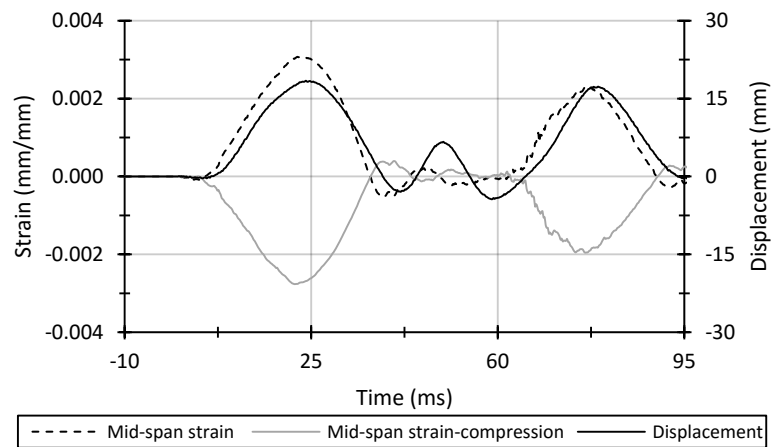
Figure C.14: Dynamic test results for B14.2-[80]



(a) Pressure and impulse

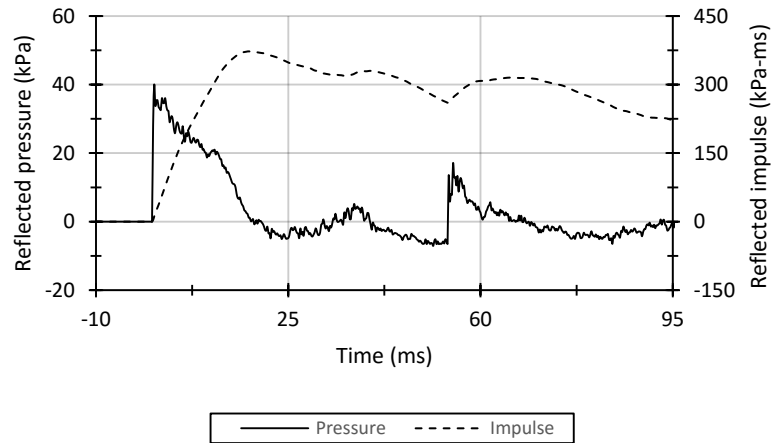


(b) Displacement and reaction

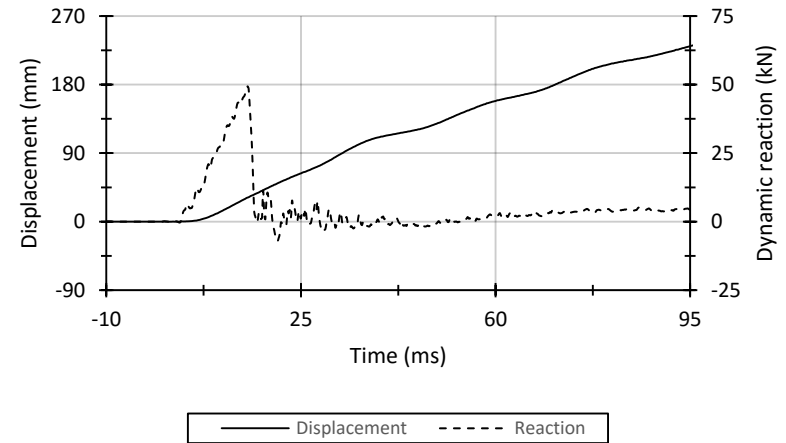


(c) Strain and displacement

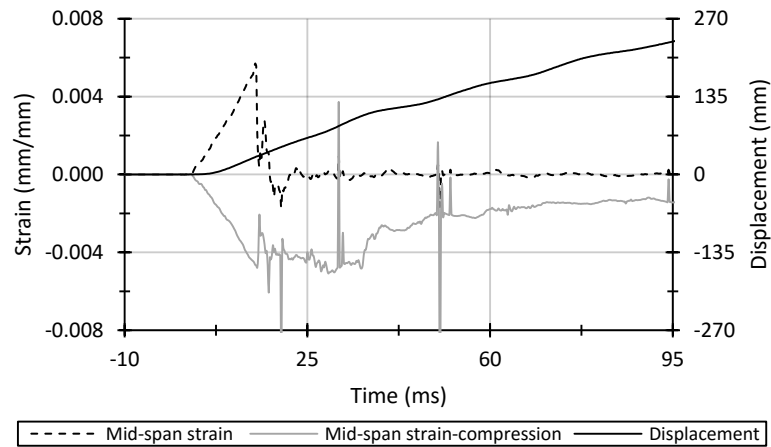
Figure C.15: Dynamic test results for B15.1-[80]



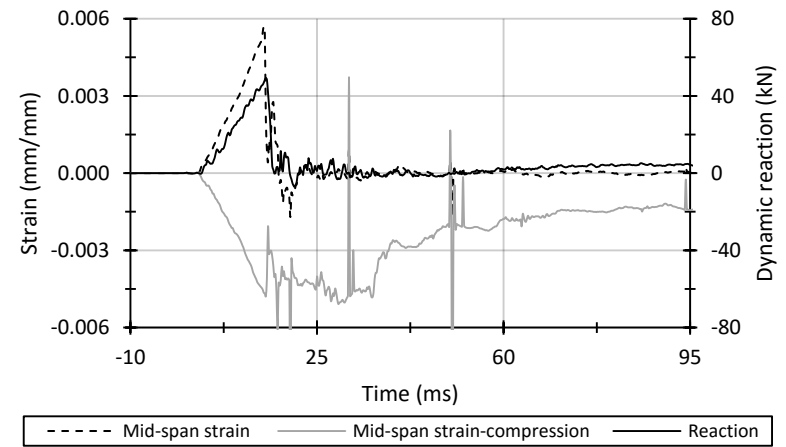
(a) Pressure and impulse



(b) Displacement and reaction

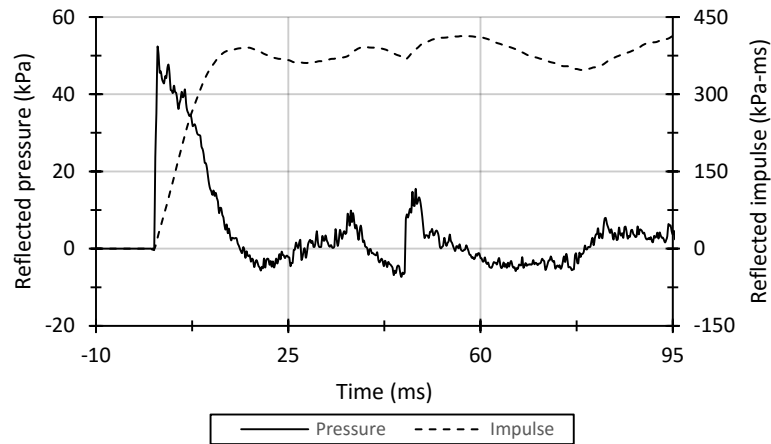


(c) Strain and displacement

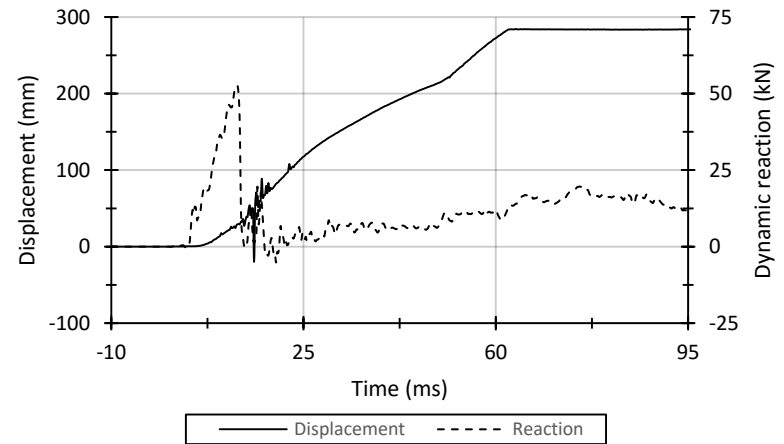


(d) Strain and reaction

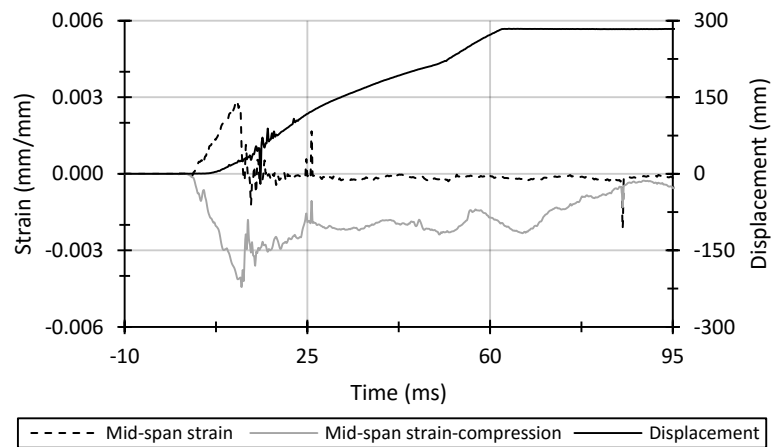
Figure C.16: Dynamic test results for B15.2-[80]



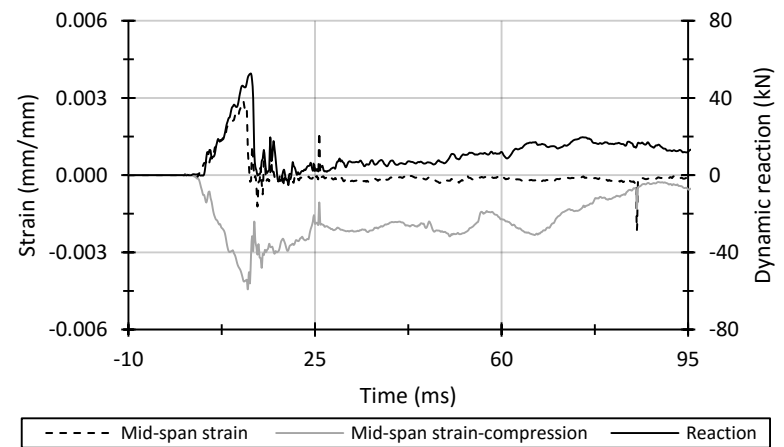
(a) Pressure and impulse



(b) Displacement and reaction

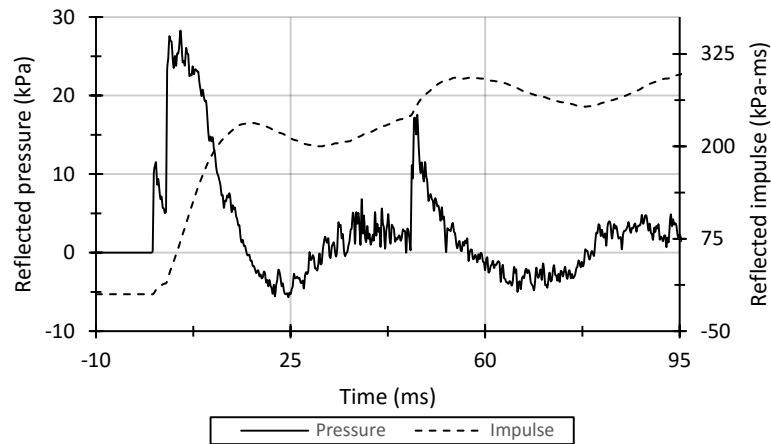


(c) Strain and displacement

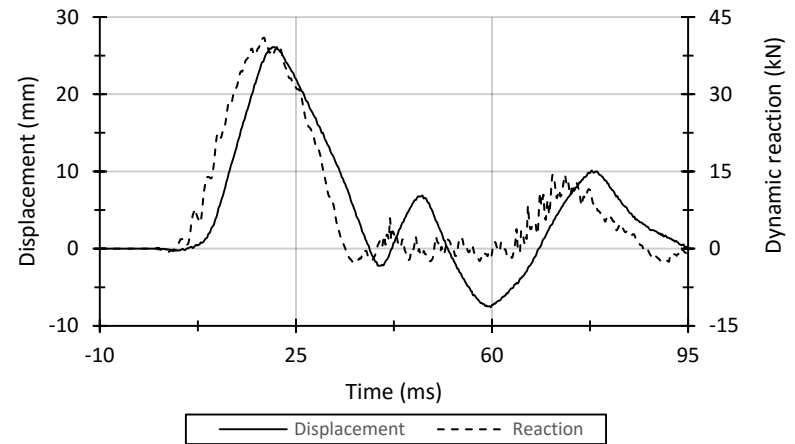


(d) Strain and reaction

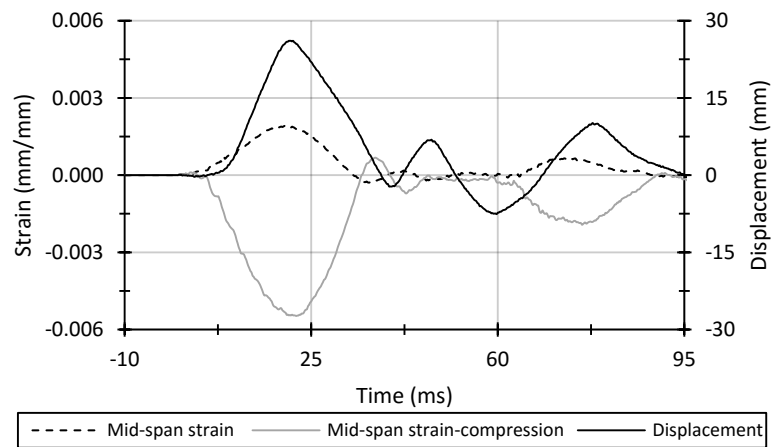
Figure C.17: Dynamic test results for B16.1-[80]



(a) Pressure and impulse

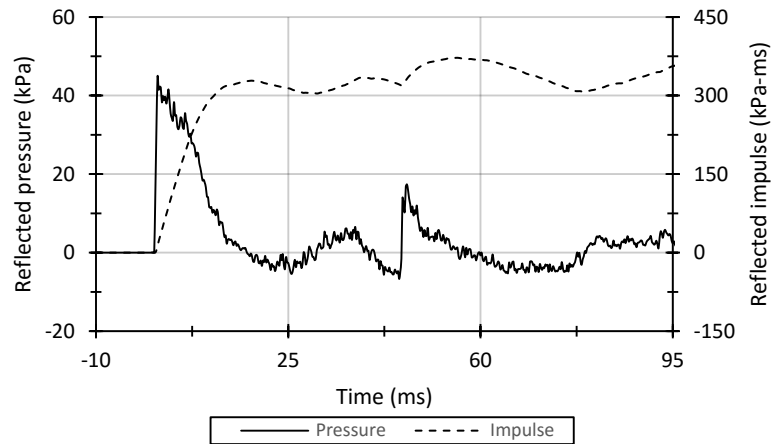


(b) Displacement and reaction

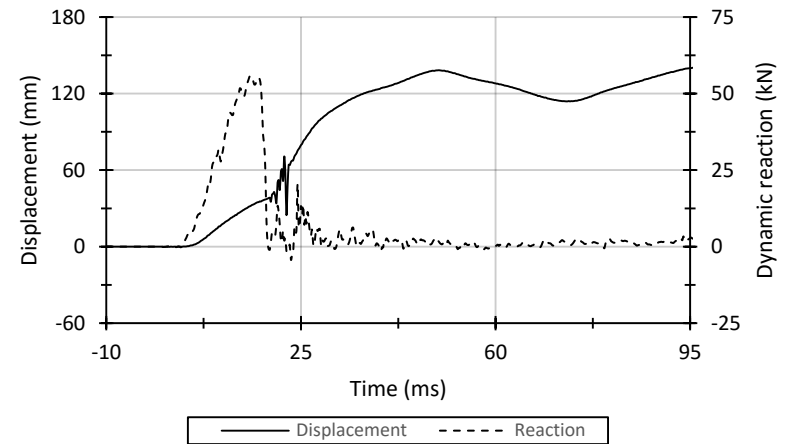


(c) Strain and displacement

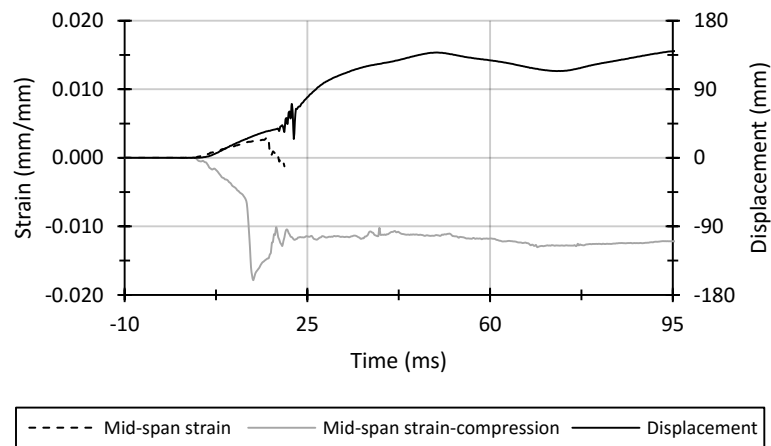
Figure C.18: Dynamic test results for B17.1-[80]



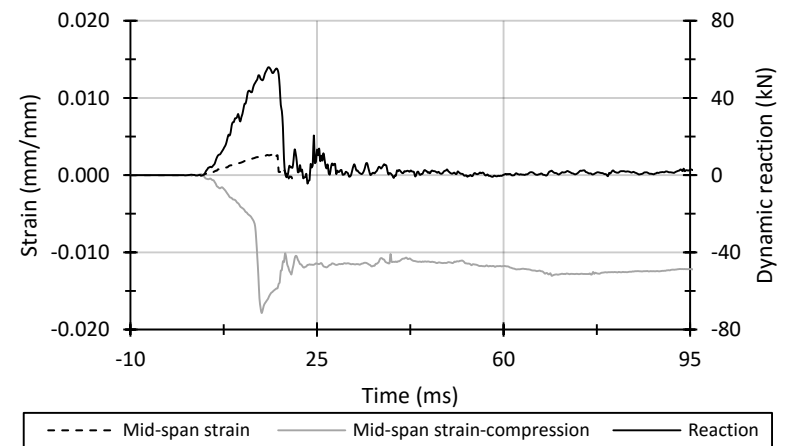
(a) Pressure and impulse



(b) Displacement and reaction

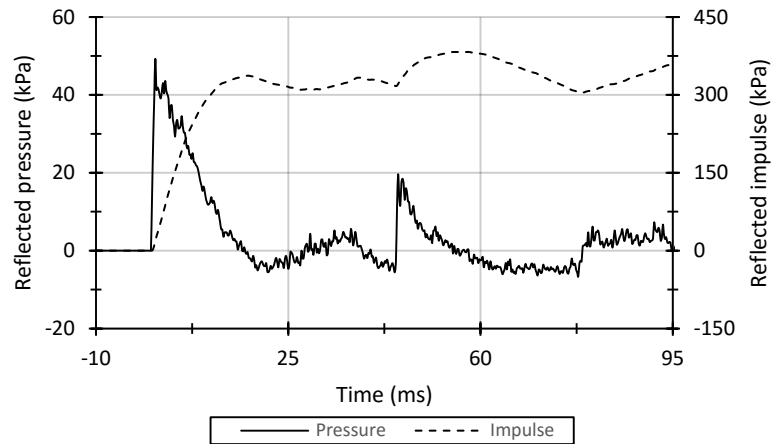


(c) Strain and displacement

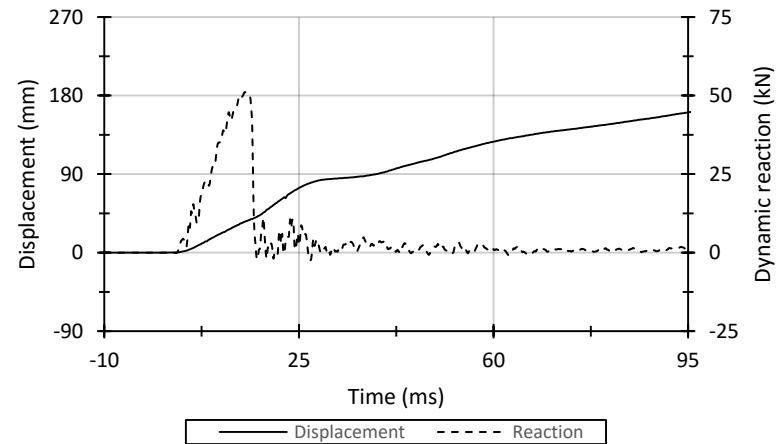


(d) Strain and reaction

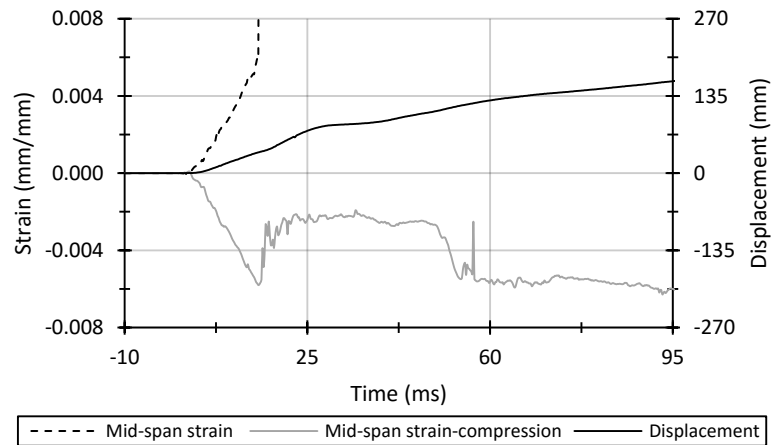
Figure C.19: Dynamic test results for B17.2-[80]



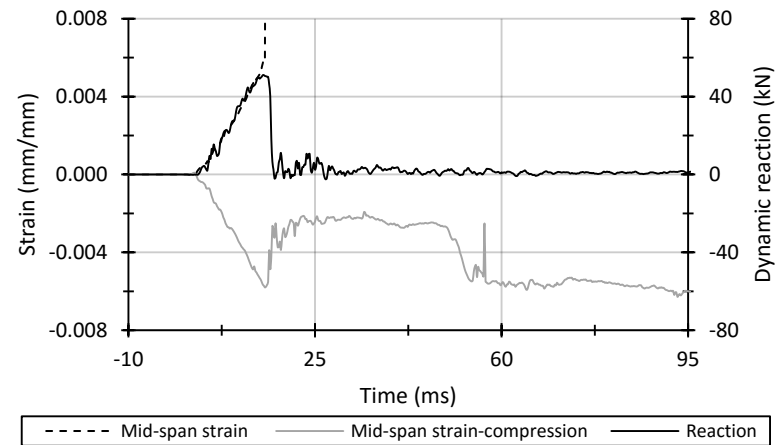
(a) Pressure and impulse



(b) Displacement and reaction

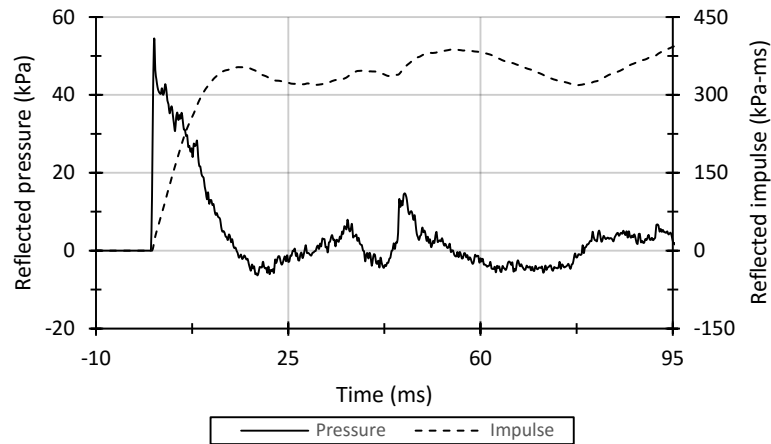


(c) Strain and displacement

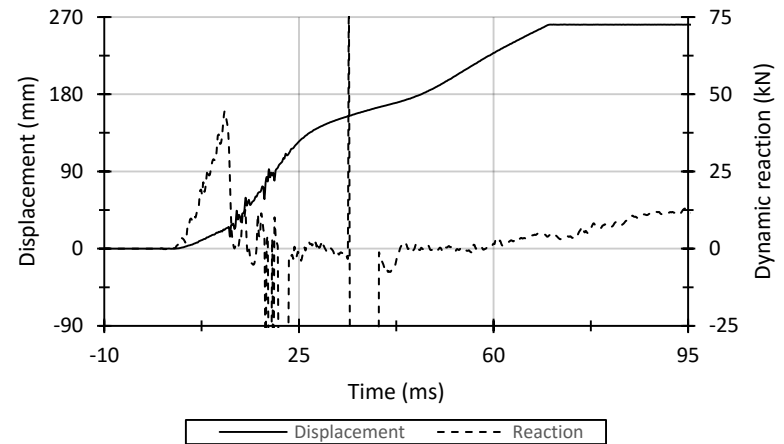


(d) Strain and reaction

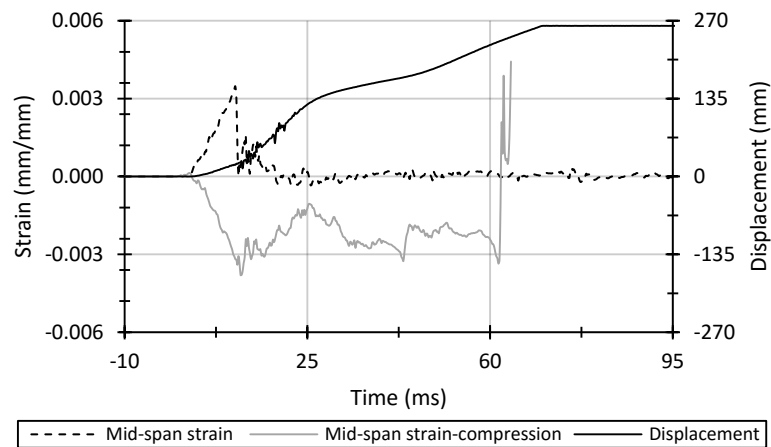
Figure C.20: Dynamic test results for B18.1-[80]



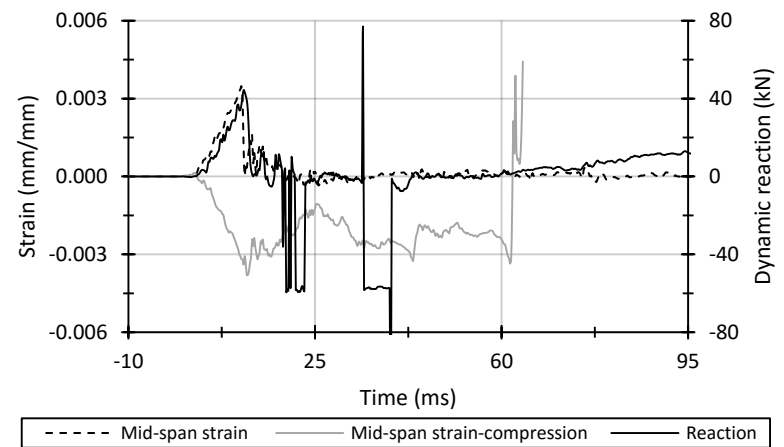
(a) Pressure and impulse



(b) Displacement and reaction

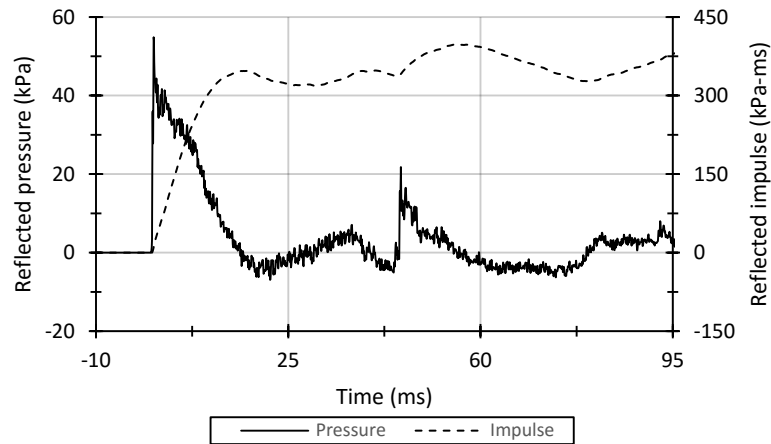


(c) Strain and displacement

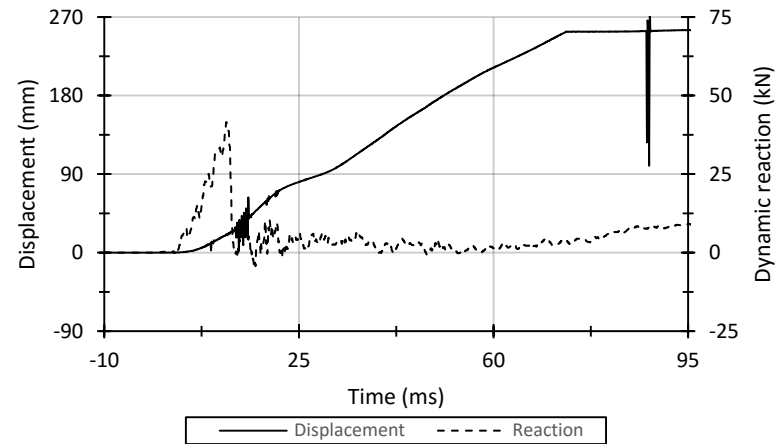


(d) Strain and reaction

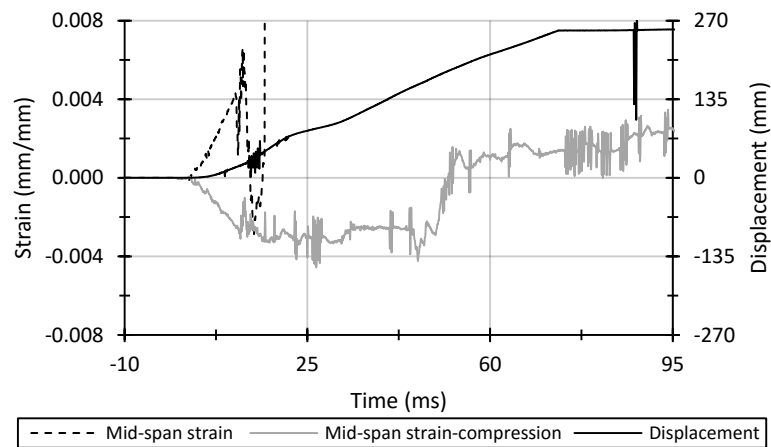
Figure C.21: Dynamic test results for B19.1-[80]



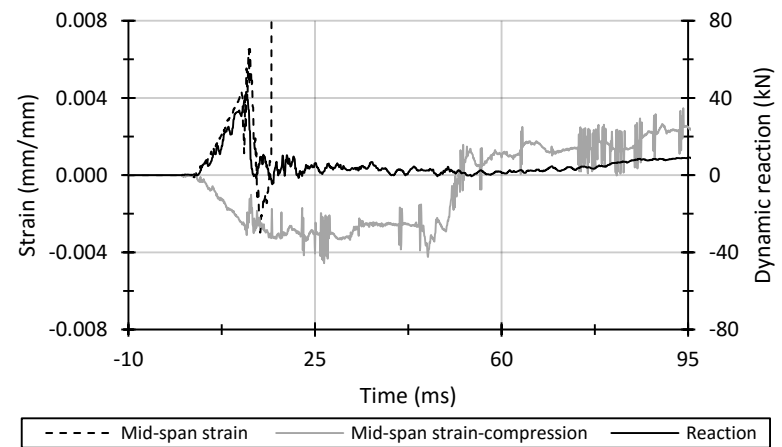
(a) Pressure and impulse



(b) Displacement and reaction

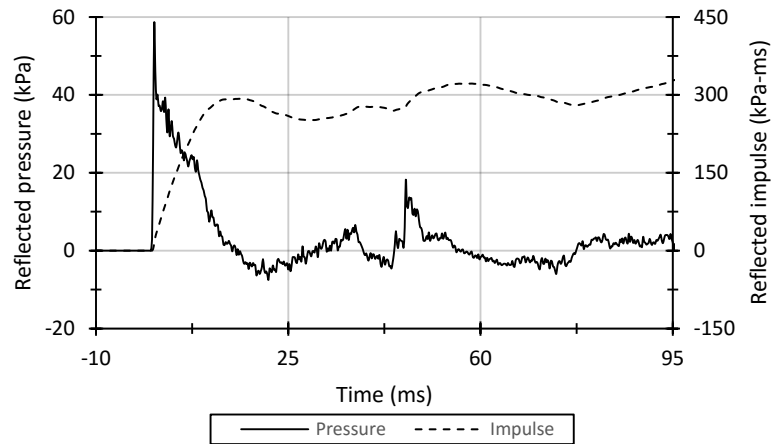


(c) Strain and displacement

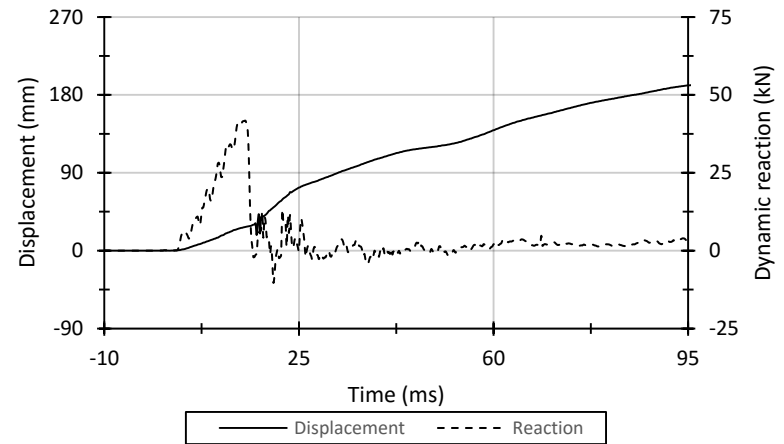


(d) Strain and reaction

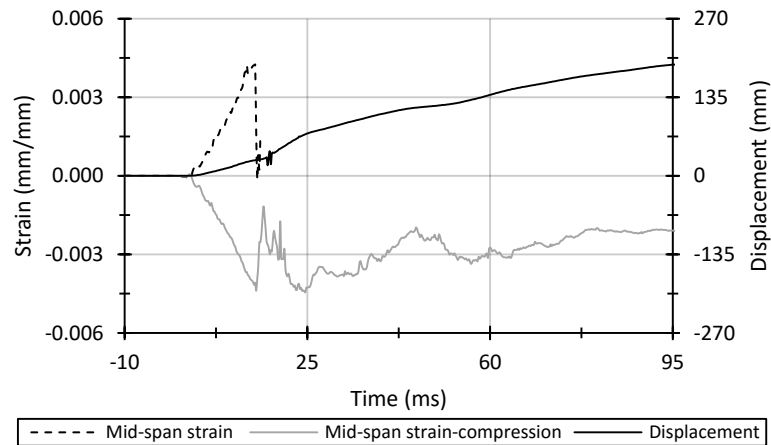
Figure C.22: Dynamic test results for B20.1-[80]



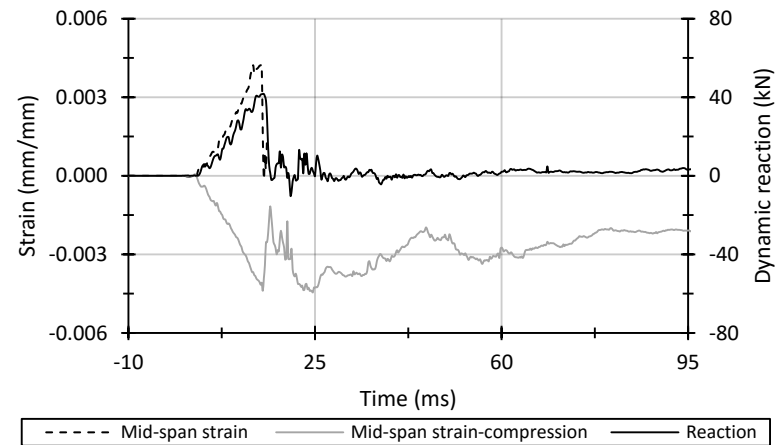
(a) Pressure and impulse



(b) Displacement and reaction



(c) Strain and displacement



(d) Strain and reaction

Figure C.23: Dynamic test results for B21.1-[80]

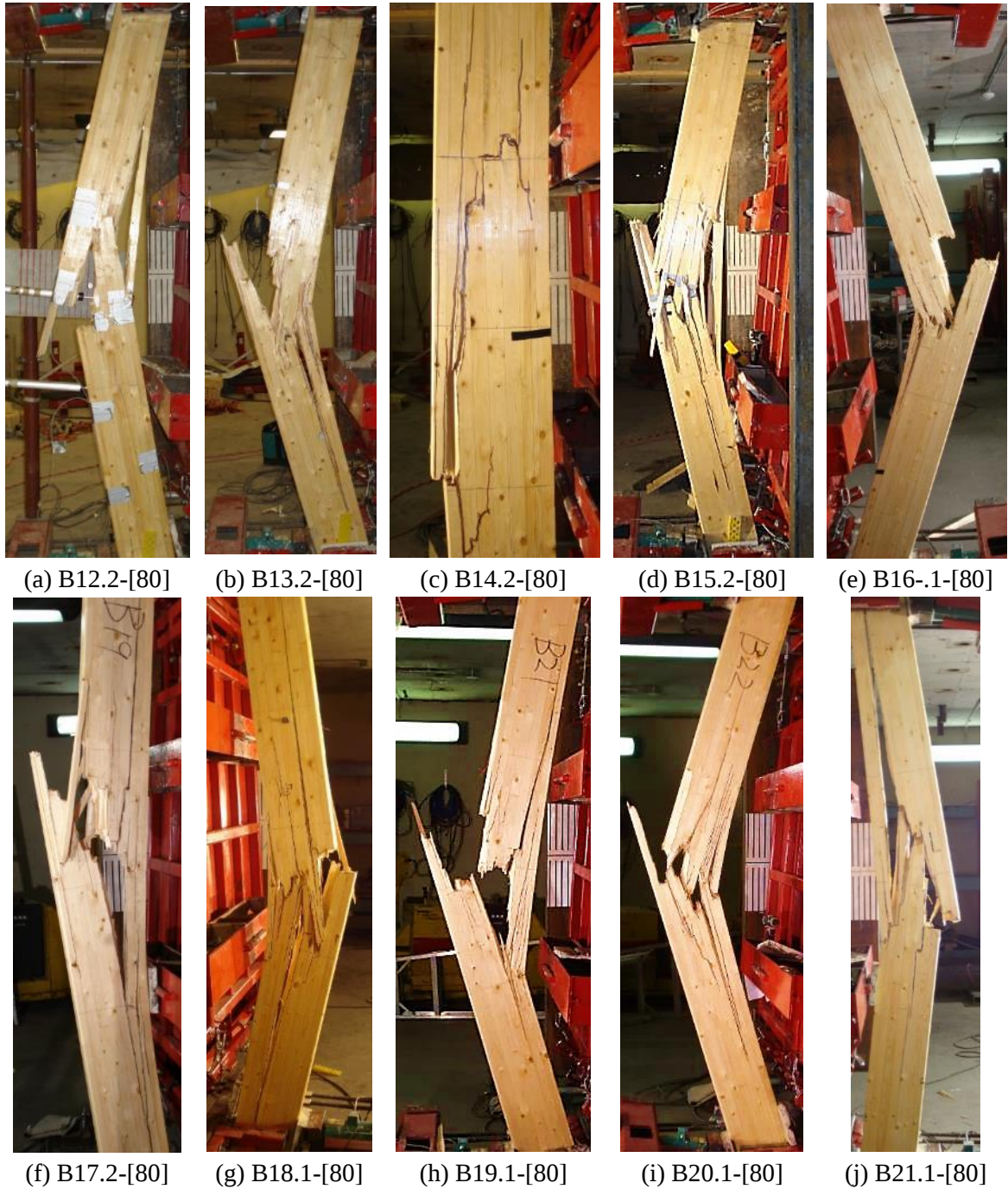
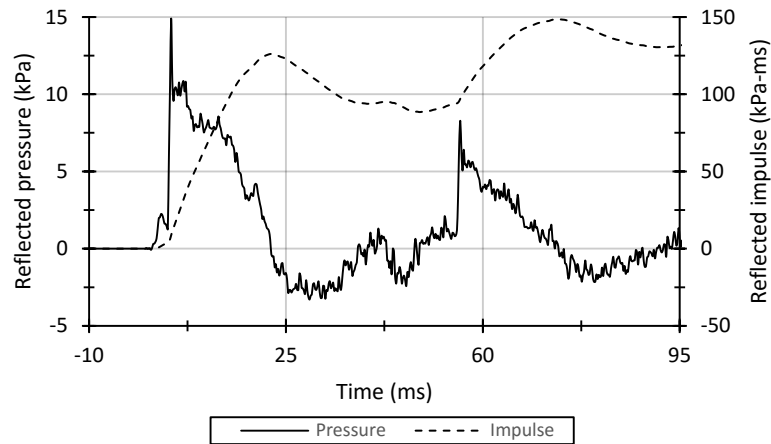
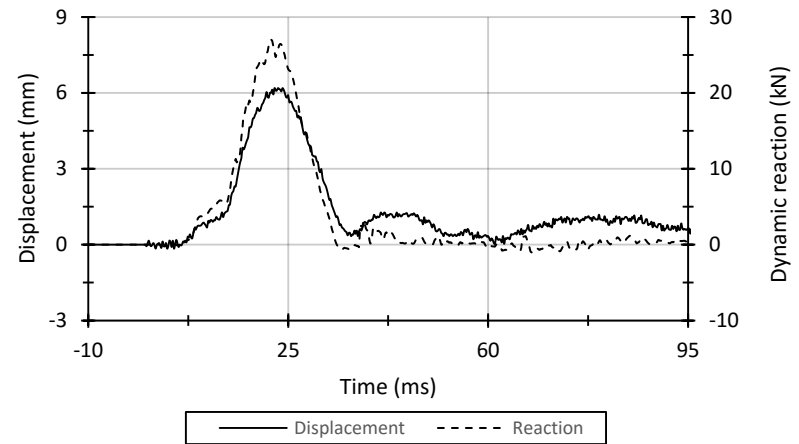


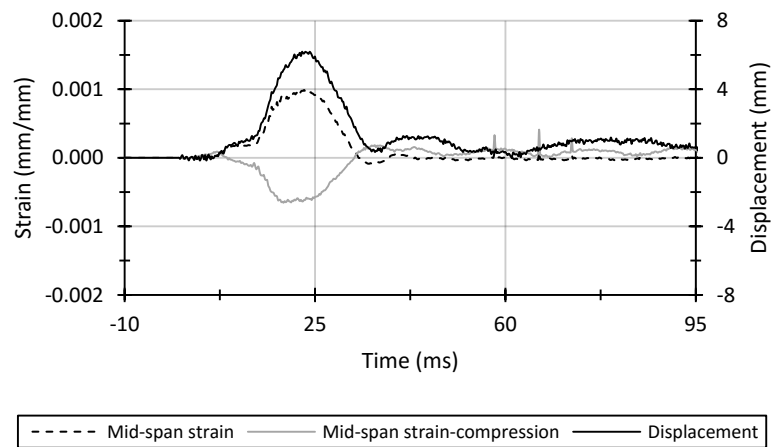
Figure C.24: Dynamic failure modes of B-[80] beams



(a) Pressure and impulse

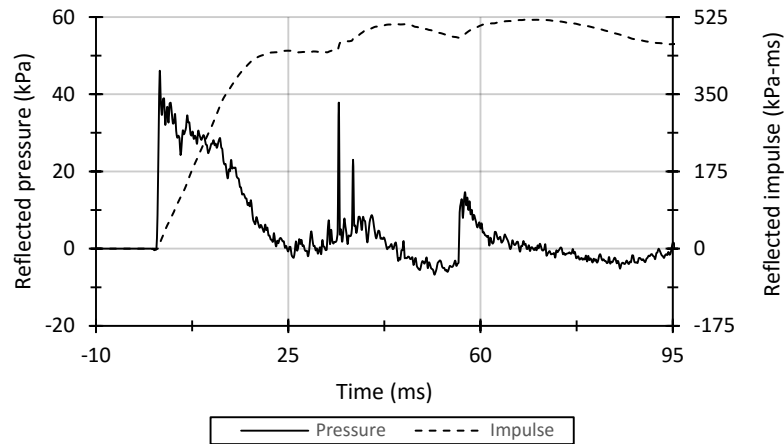


(b) Displacement and reaction

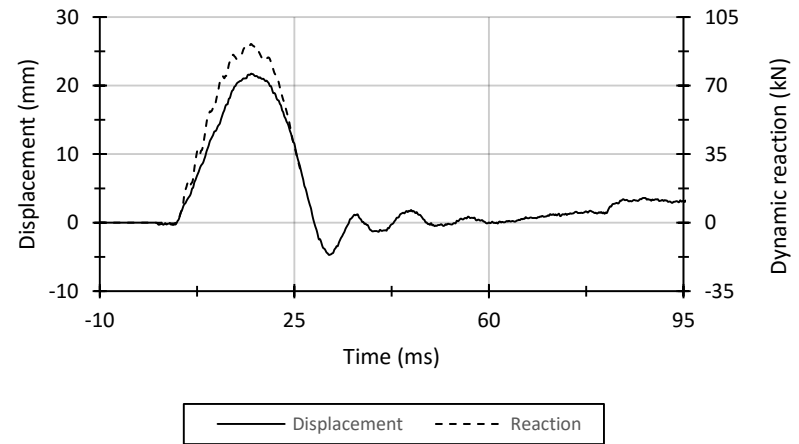


(c) Strain and displacement

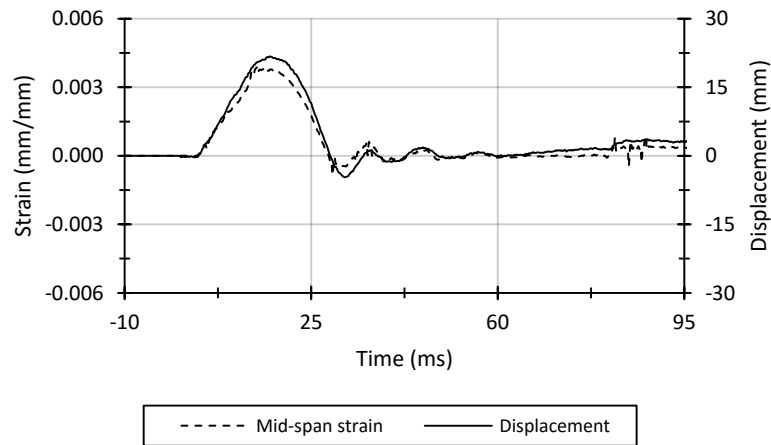
Figure C.25: Dynamic test results for B5.1-[86]



(a) Pressure and impulse

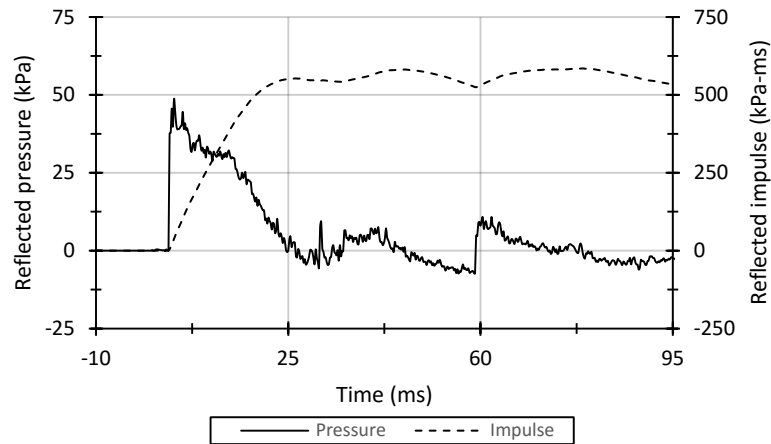


(b) Displacement and reaction

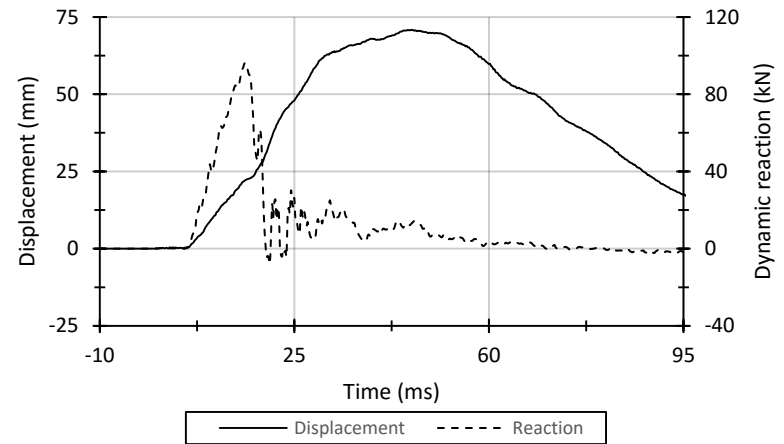


(c) Strain and displacement

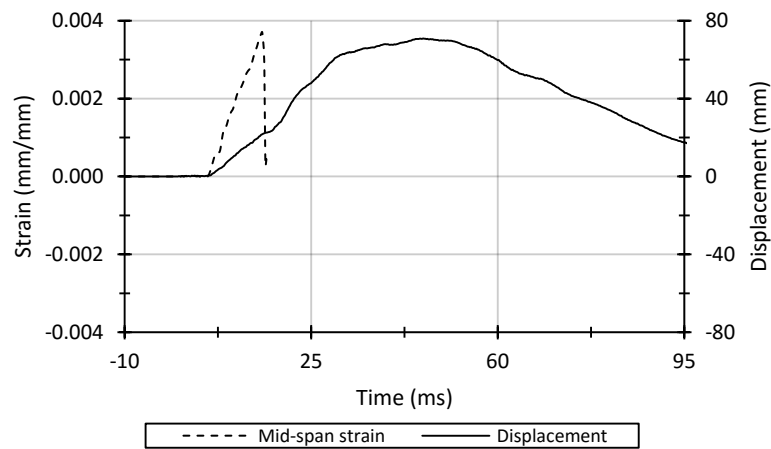
Figure C.26: Dynamic test results for B5.2-[86]



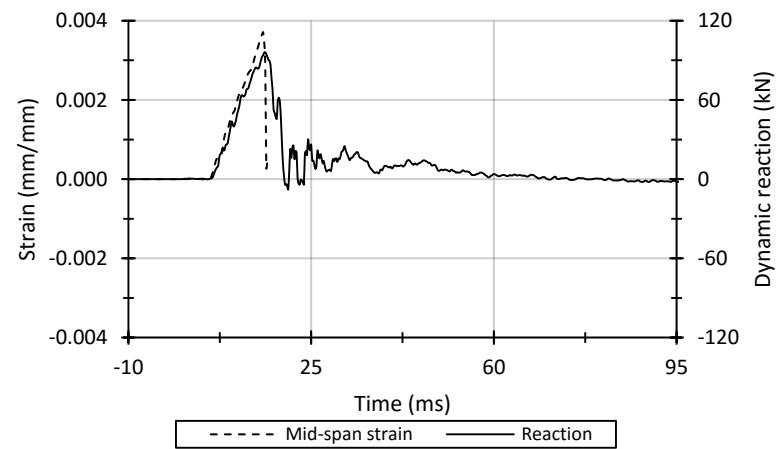
(a) Pressure and impulse



(b) Displacement and reaction

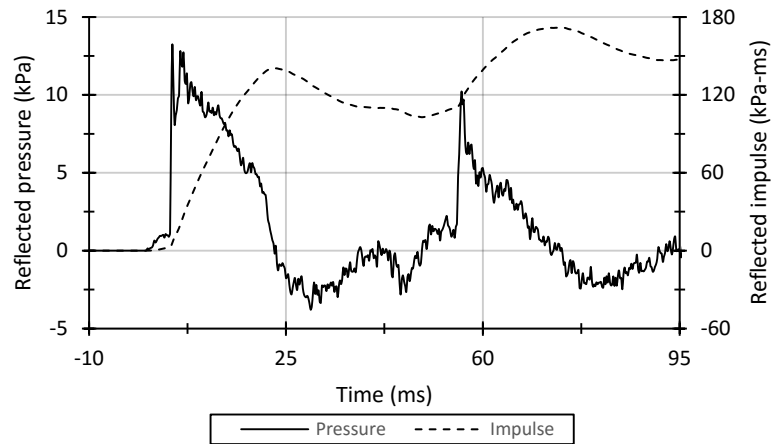


(c) Strain and displacement

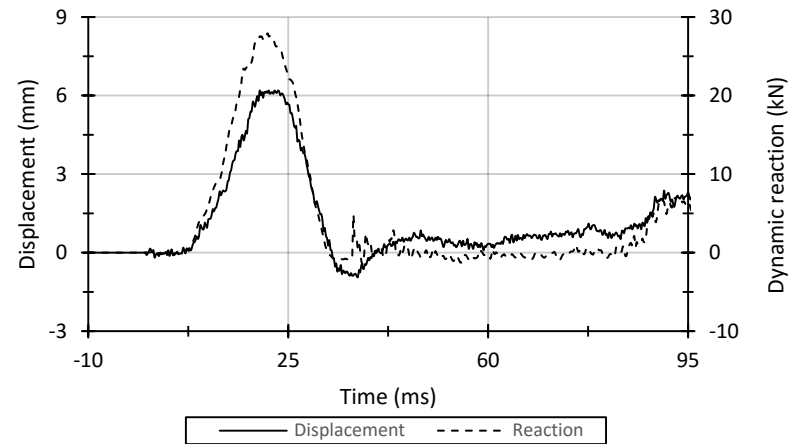


(d) Strain and reaction

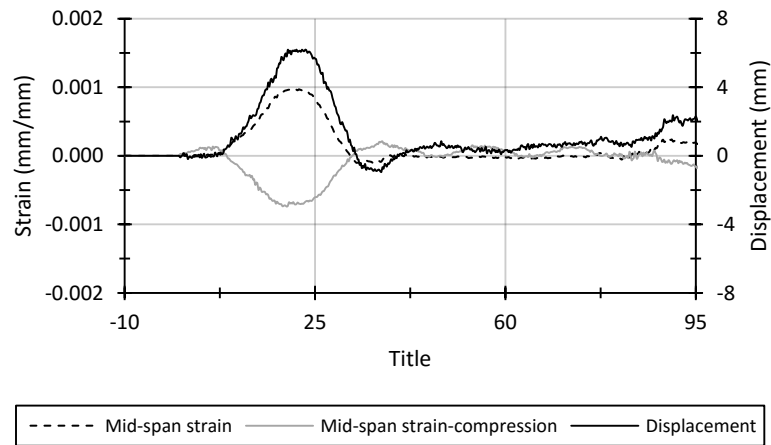
Figure C.27: Dynamic test results for B5.3-[86]



(a) Pressure and impulse

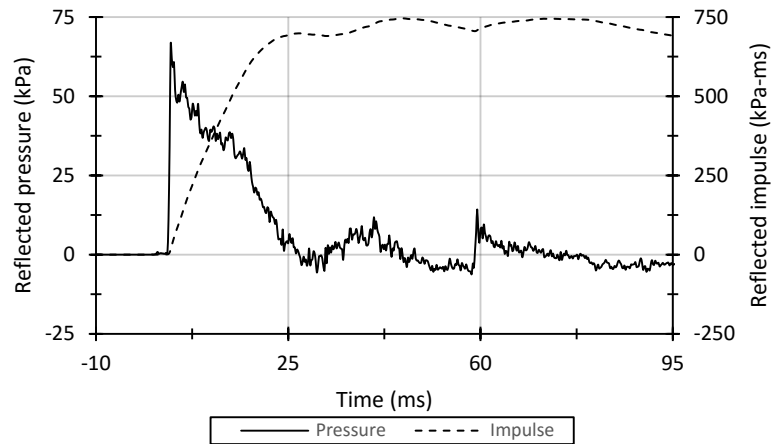


(b) Displacement and reaction

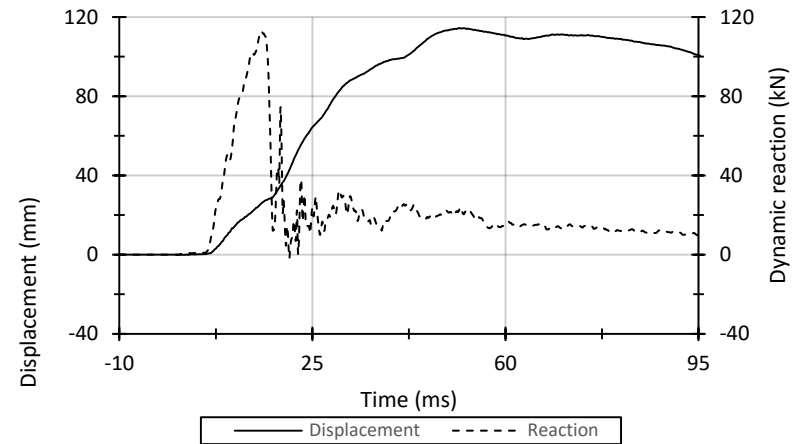


(c) Strain and displacement

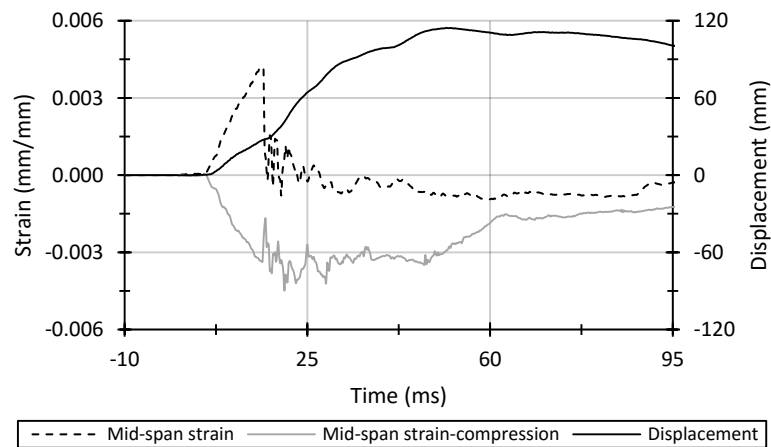
Figure C.28: Dynamic test results for B6.1-[86]



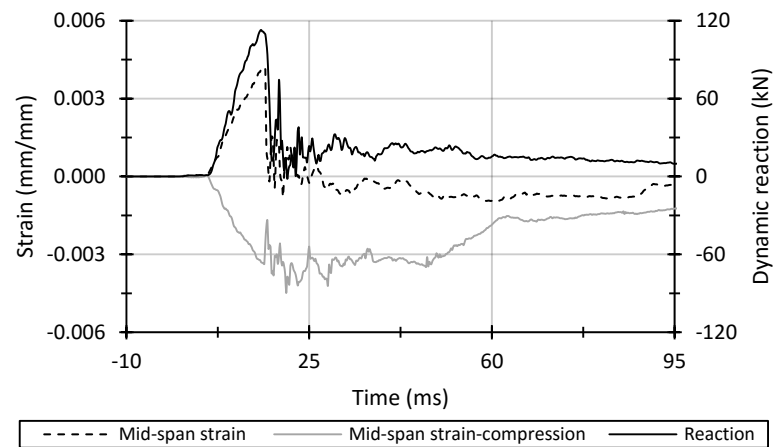
(a) Pressure and impulse



(b) Displacement and reaction

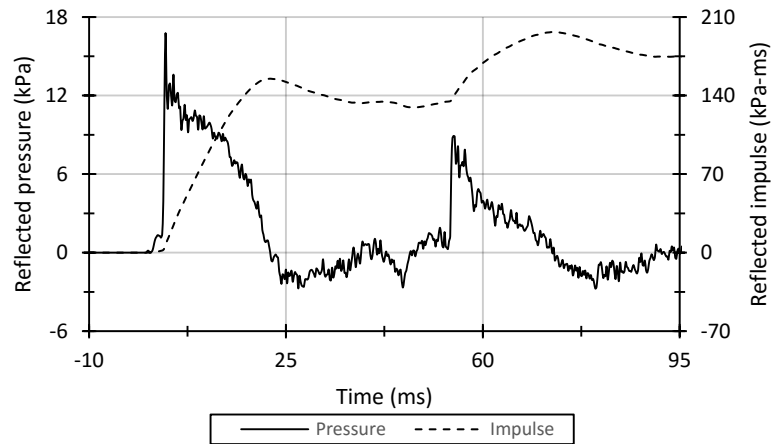


(c) Strain and displacement

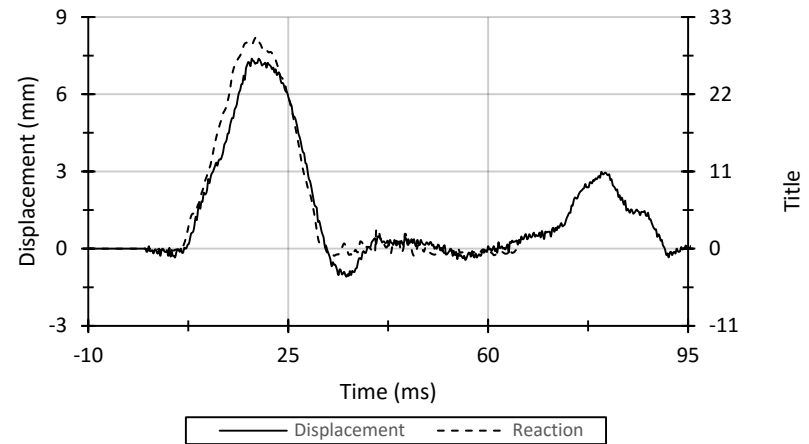


(d) Strain and reaction

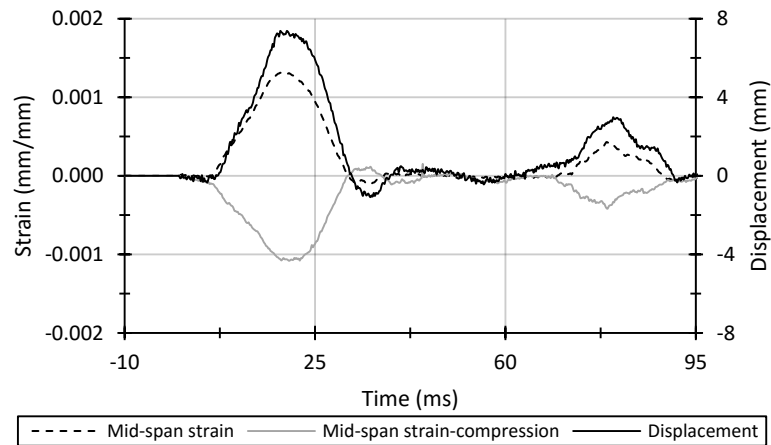
Figure C.29: Dynamic test results for B6.2-[86]



(a) Pressure and impulse

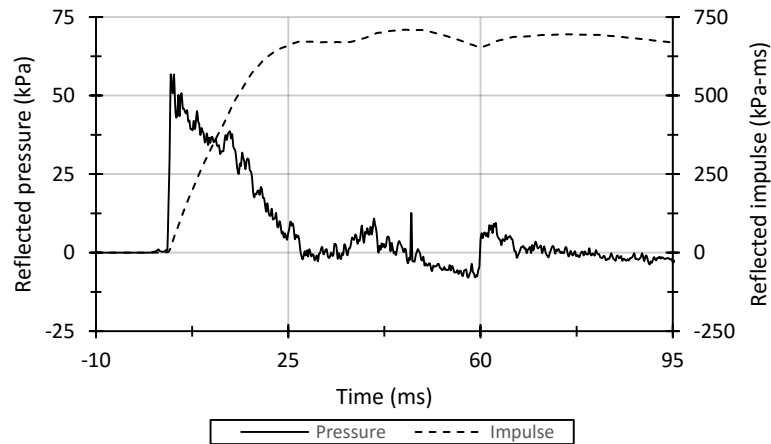


(b) Displacement and reaction

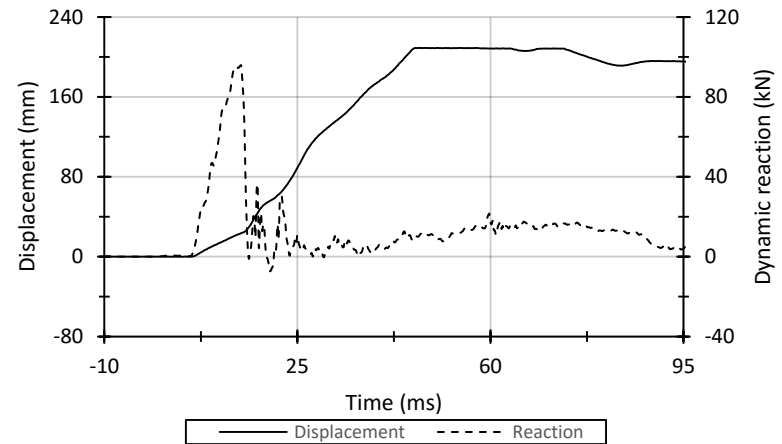


(c) Strain and displacement

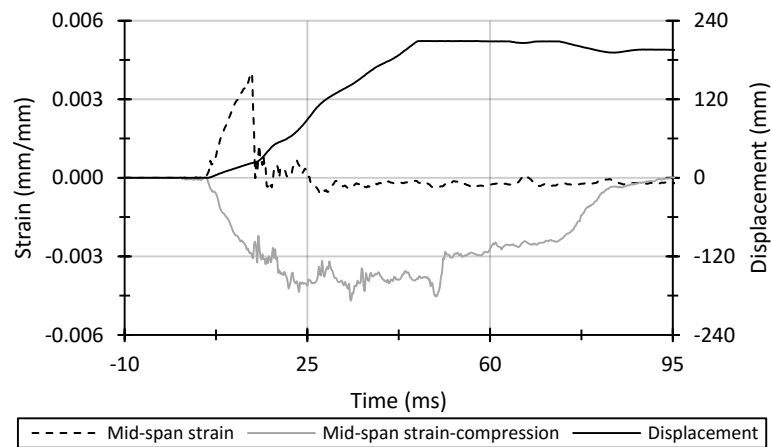
Figure C.30: Dynamic test results for B7.1-[86]



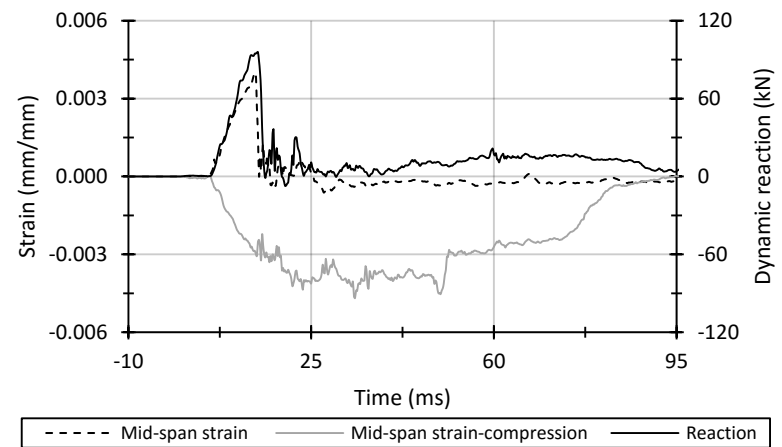
(a) Pressure and impulse



(b) Displacement and reaction

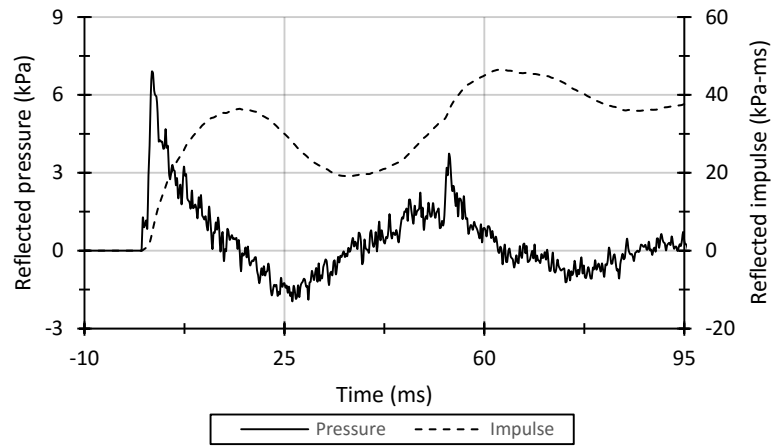


(c) Strain and displacement

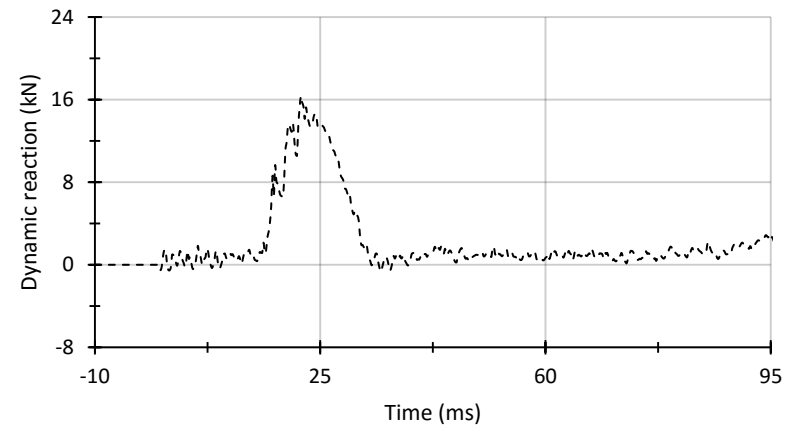


(d) Strain and reaction

Figure C.31: Dynamic test results for B7.2-[86]

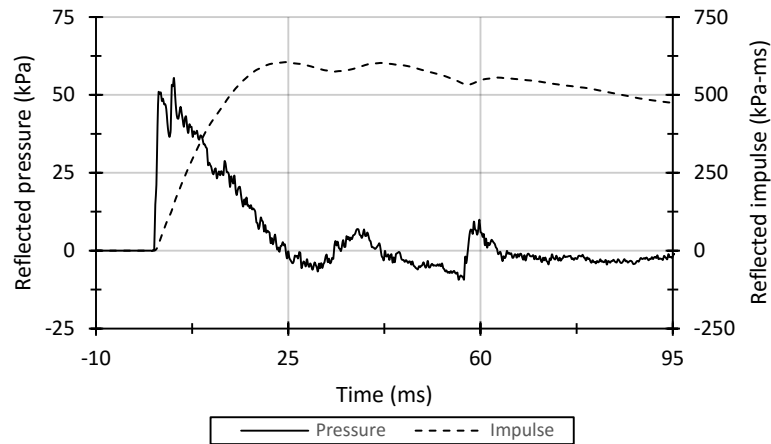


(a) Pressure and impulse

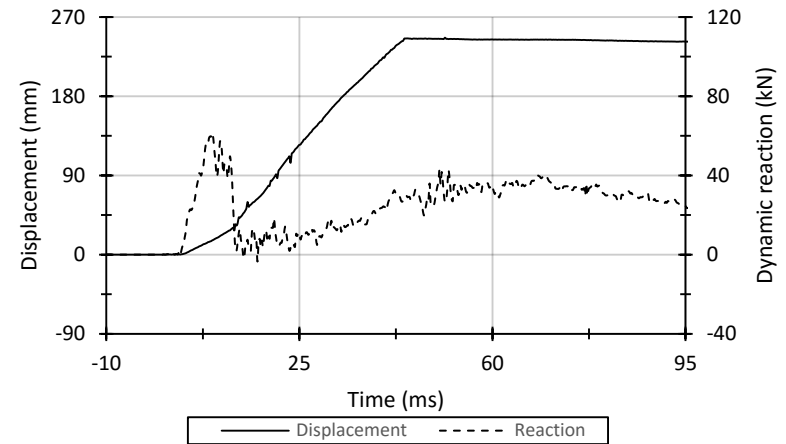


(b) Displacement and reaction

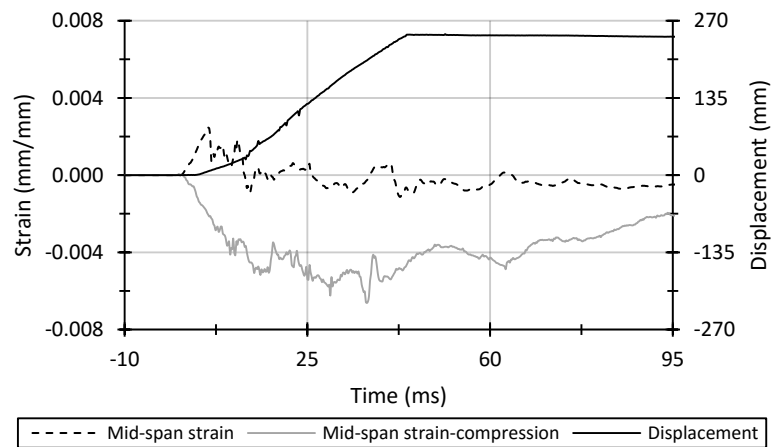
Figure C.32: Dynamic test results for B8.1-[86]



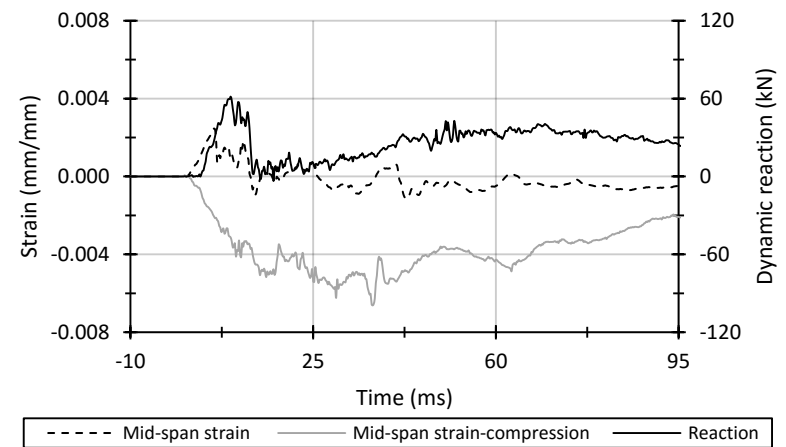
(a) Pressure and impulse



(b) Displacement and reaction



(c) Strain and displacement



(d) Strain and reaction

Figure C.33: Dynamic test results for B8.2-[86]



(a) B5.3-[86]



(b) B6.2-[86]

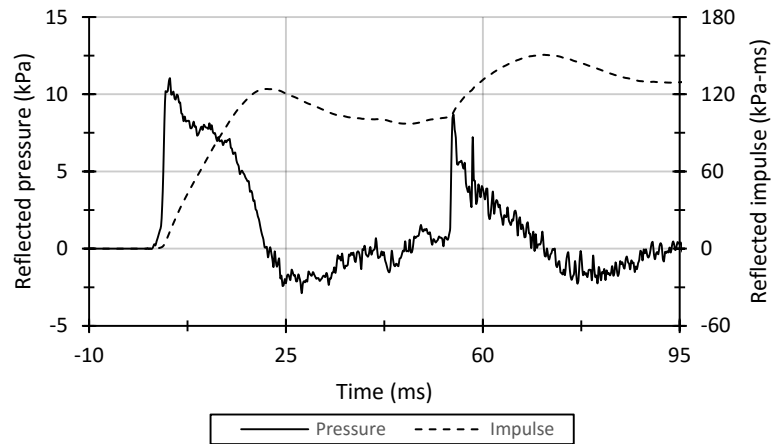


(c) B7.2-[86]

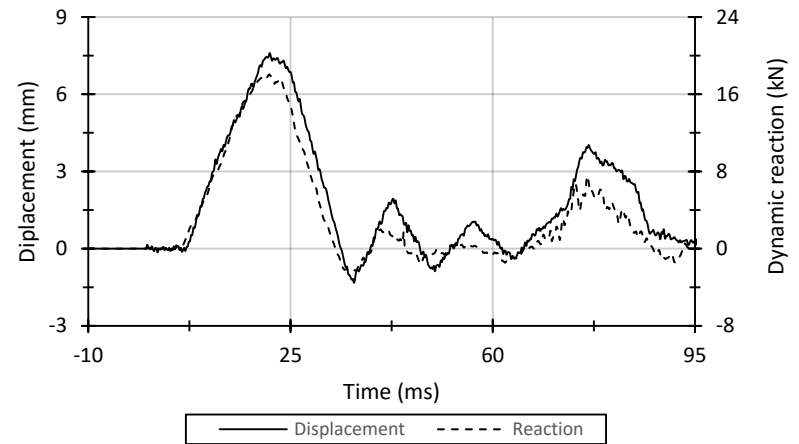


(d) B8.2-[86]

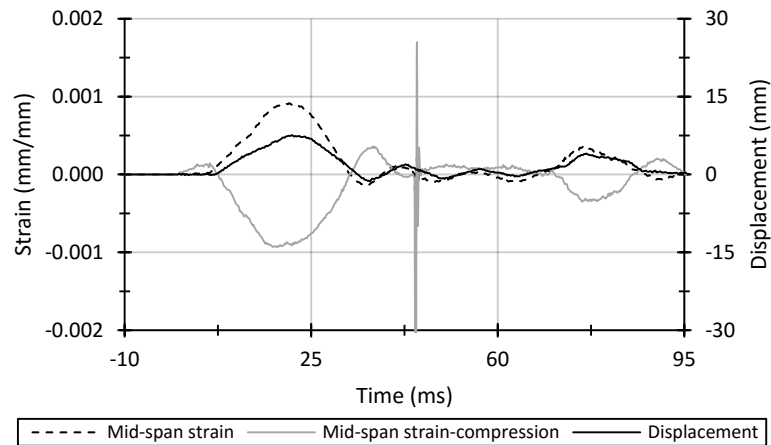
Figure C.34: Dynamic failure modes of B-[86] beams



(a) Pressure and impulse

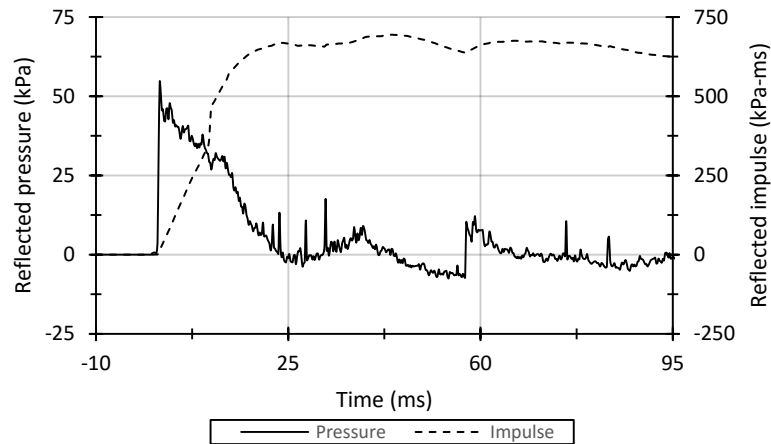


(b) Displacement and reaction

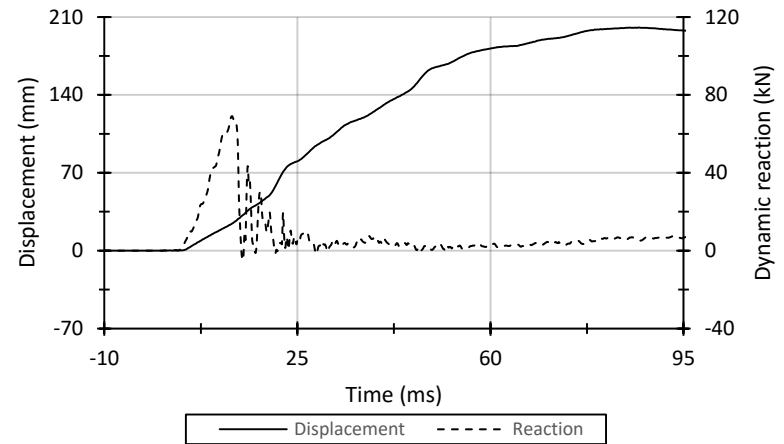


(c) Strain and displacement

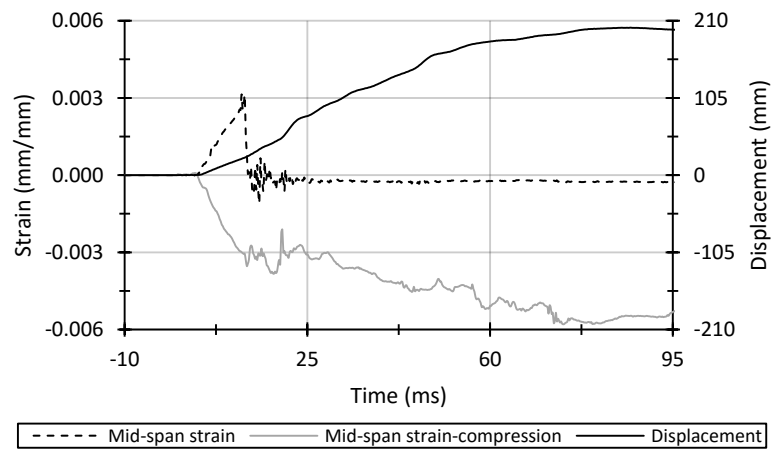
Figure C.35: Dynamic test results for B5.1-[137]



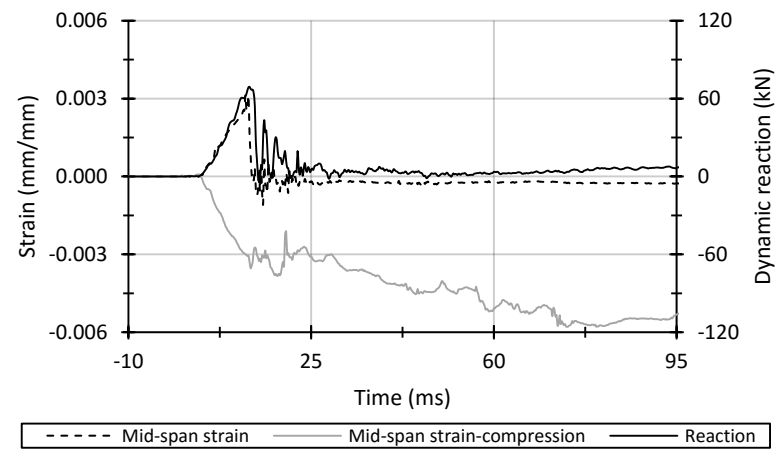
(a) Pressure and impulse



(b) Displacement and reaction

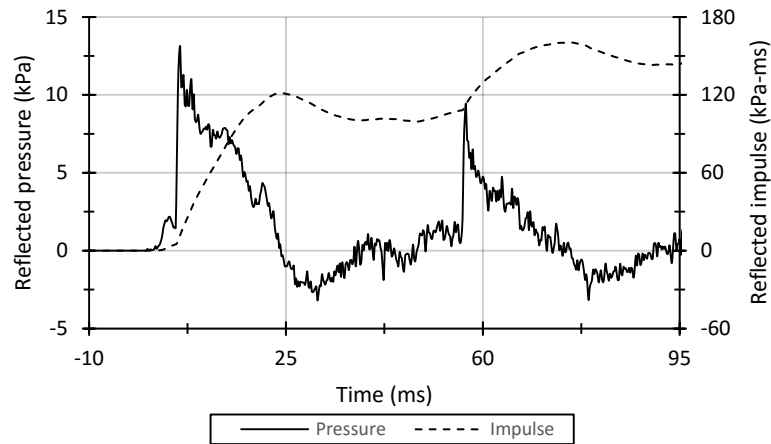


(c) Strain and displacement

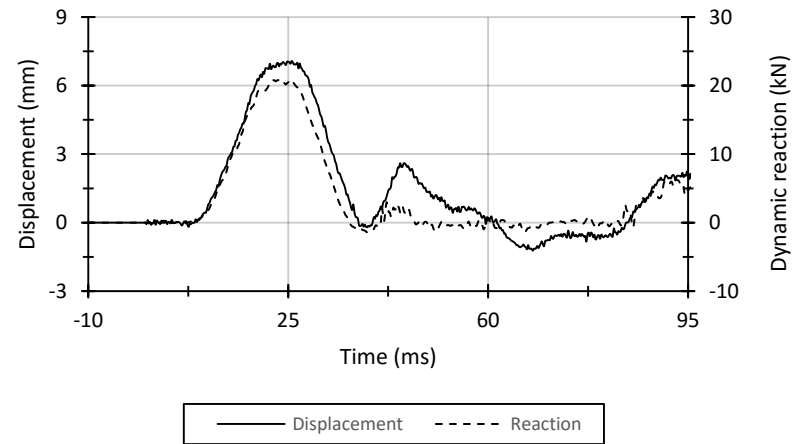


(d) Strain and reaction

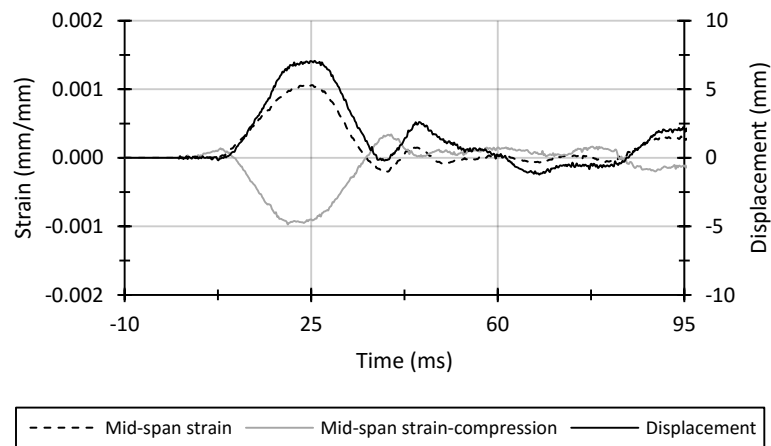
Figure C.36: Dynamic test results for B5.2-[137]



(a) Pressure and impulse

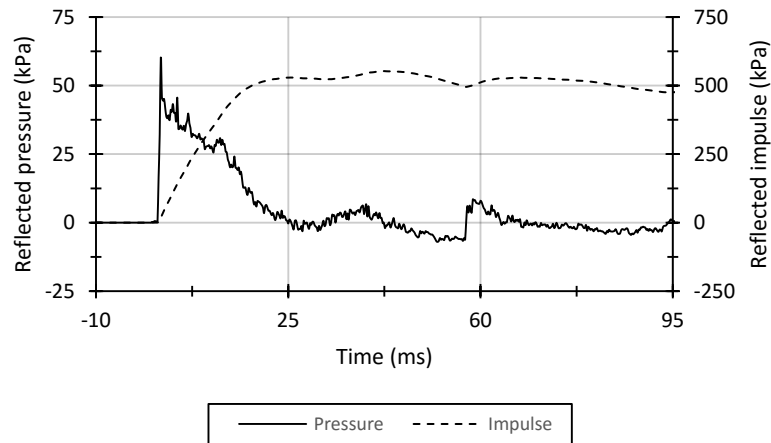


(b) Displacement and reaction

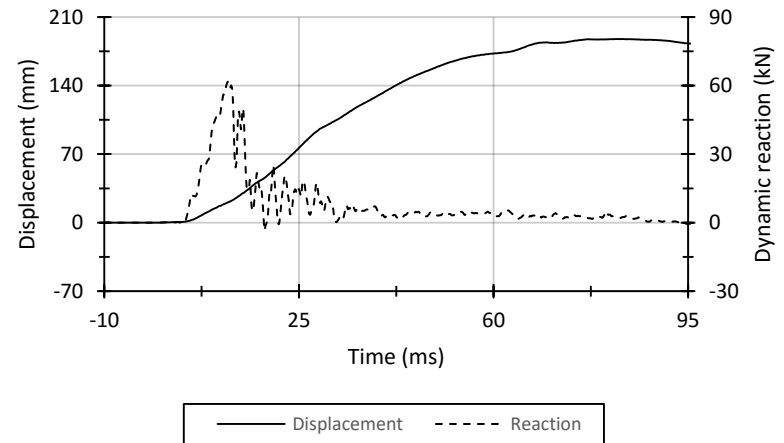


(c) Strain and displacement

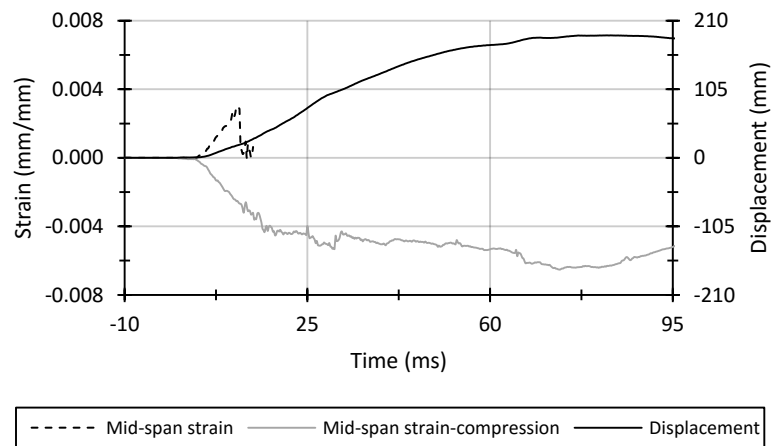
Figure C.37: Dynamic test results for B6.1-[137]



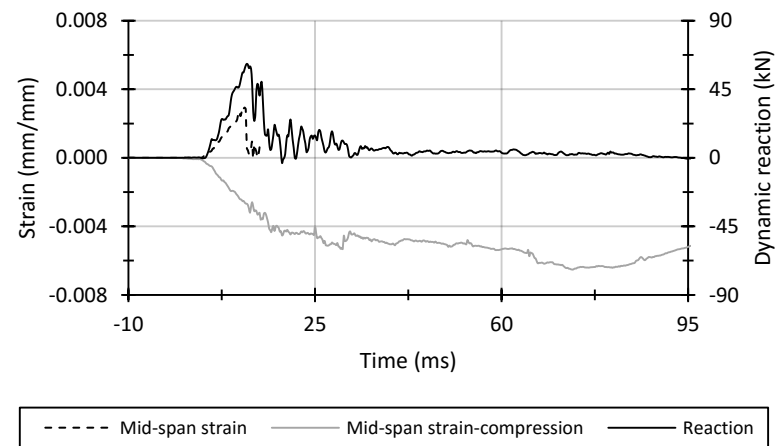
(a) Pressure and impulse



(b) Displacement and reaction

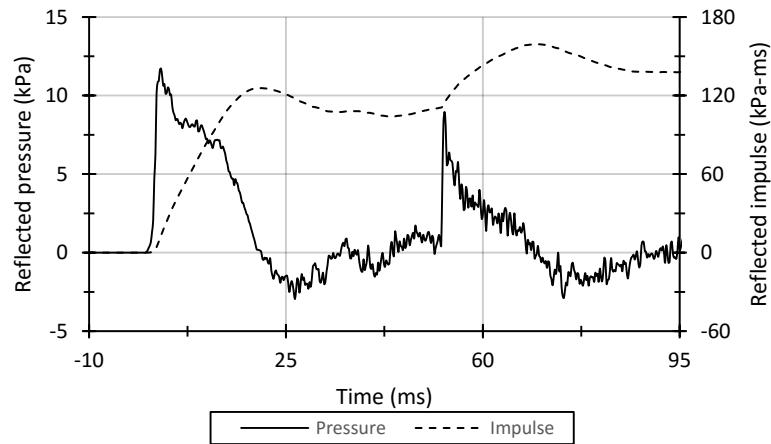


(c) Strain and displacement

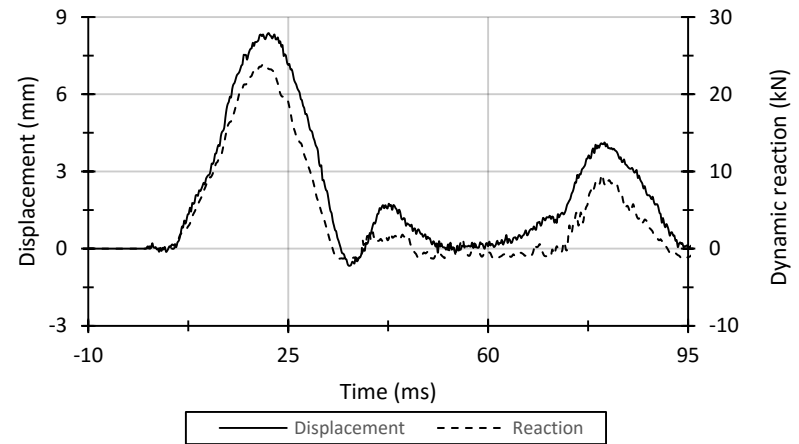


(d) Strain and reaction

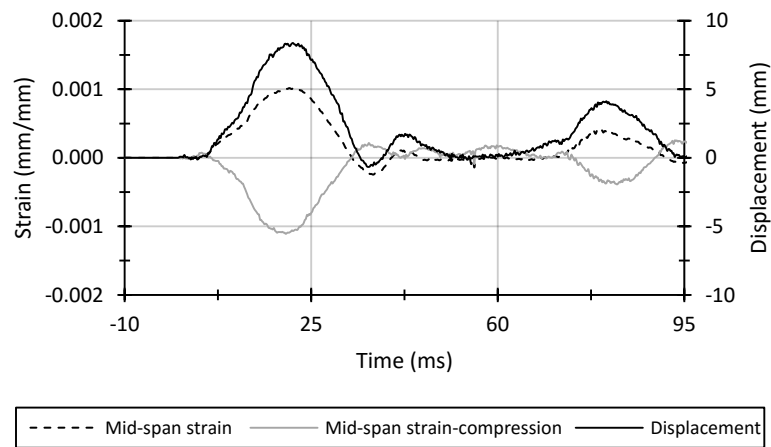
Figure C.38: Dynamic test results for B6.2-[137]



(a) Pressure and impulse

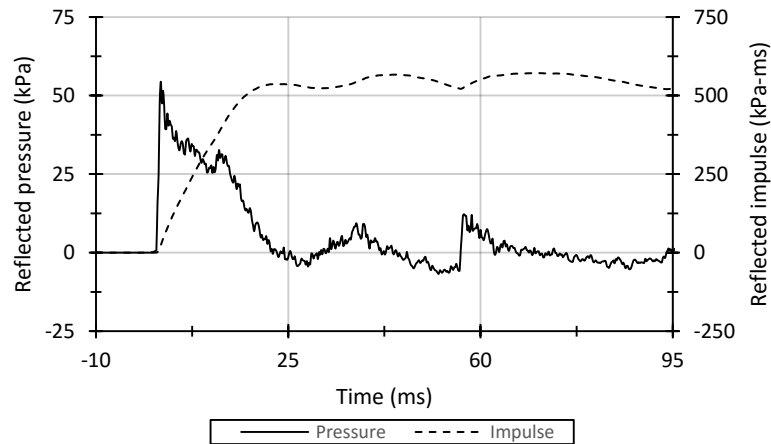


(b) Displacement and reaction

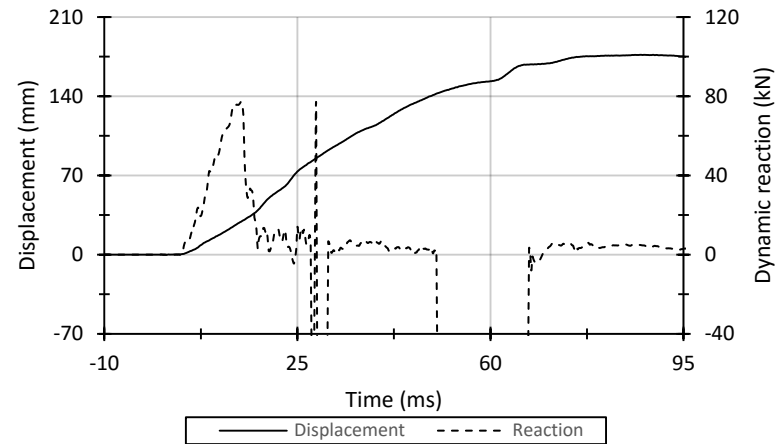


(c) Strain and displacement

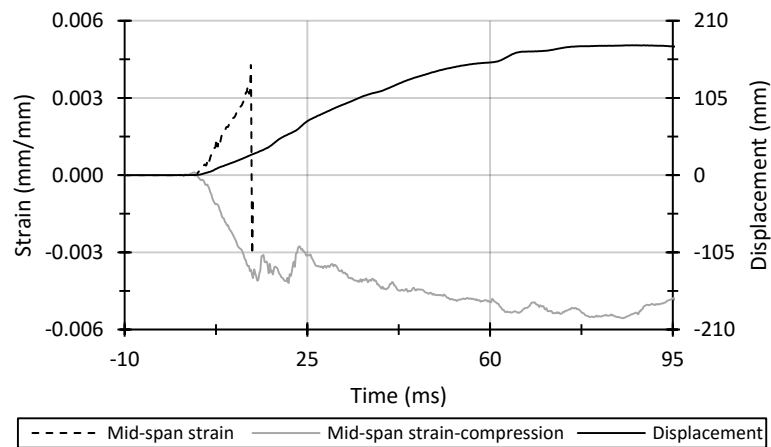
Figure C.39: Dynamic test results for B7.1-[137]



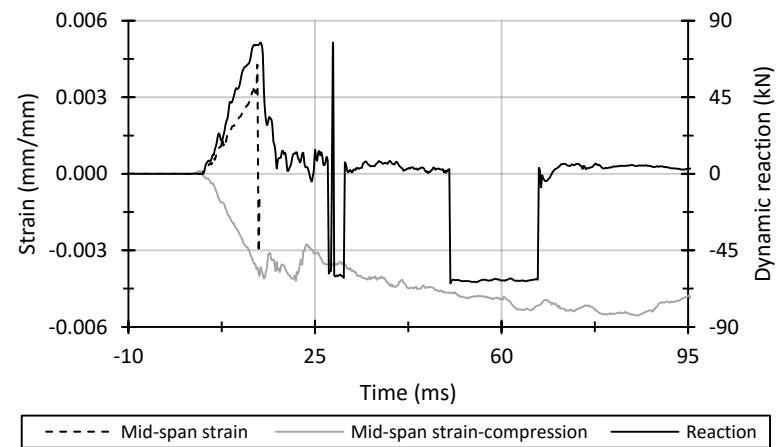
(a) Pressure and impulse



(b) Displacement and reaction

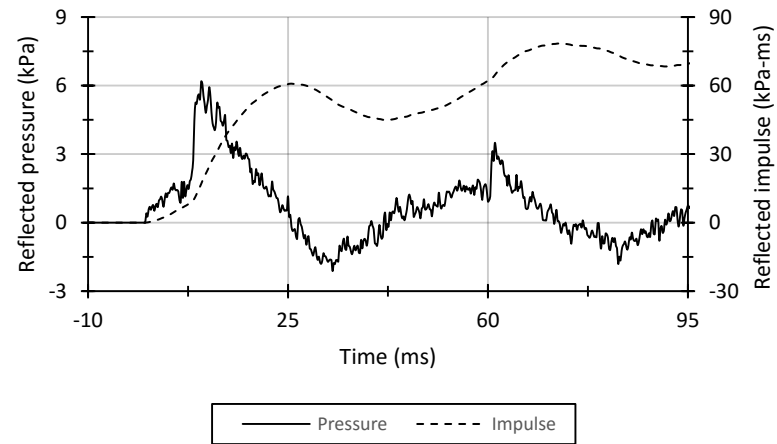


(c) Strain and displacement



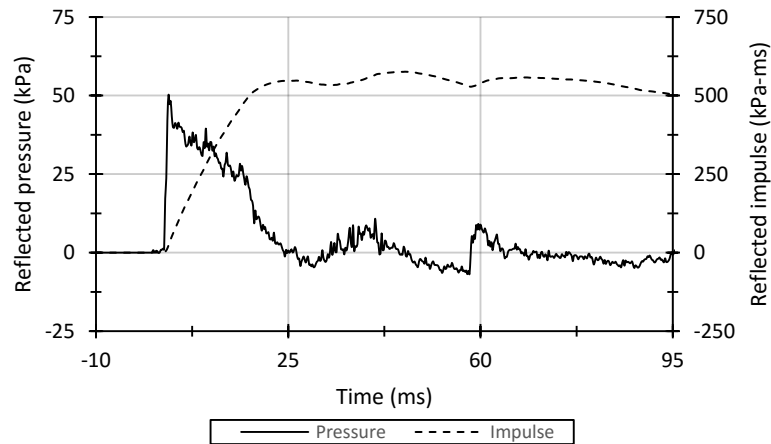
(d) Strain and reaction

Figure C.40: Dynamic test results for B7.2-[137]

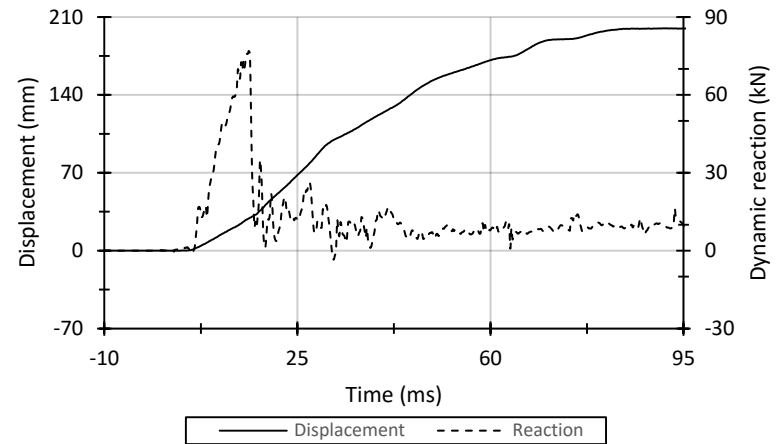


(a) Pressure and impulse

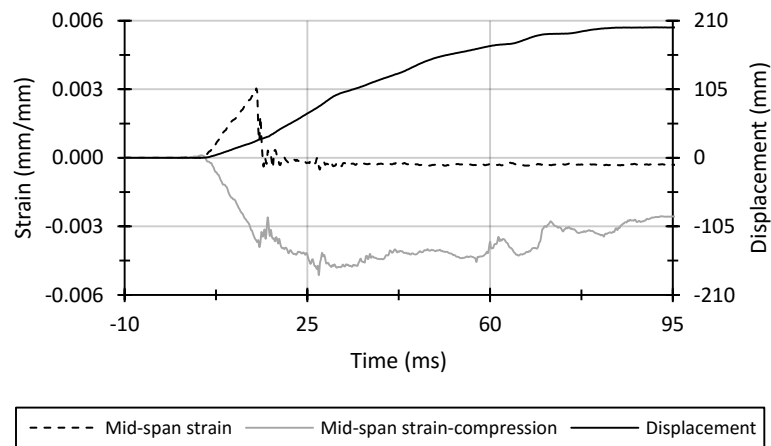
Figure C.41: Dynamic test results for B8.1-[137]



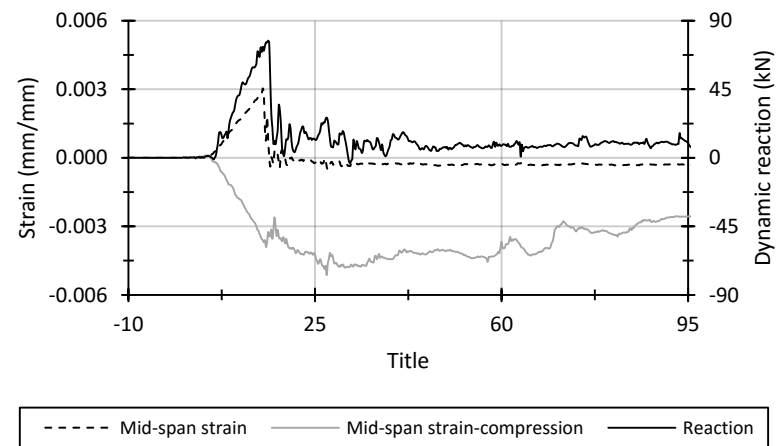
(a) Pressure and impulse



(b) Displacement and reaction

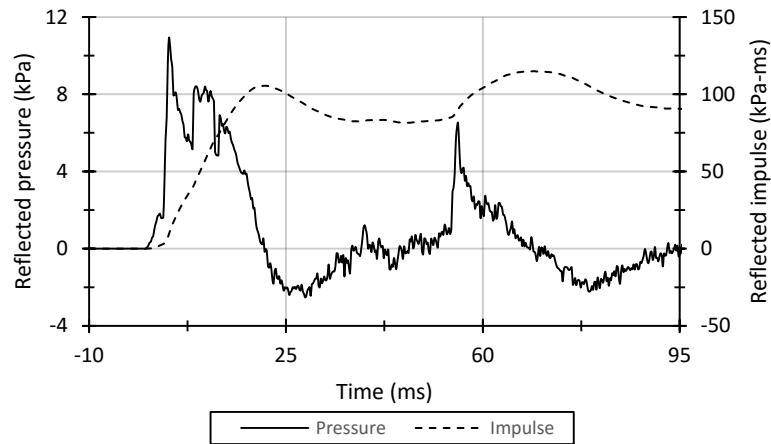


(c) Strain and displacement

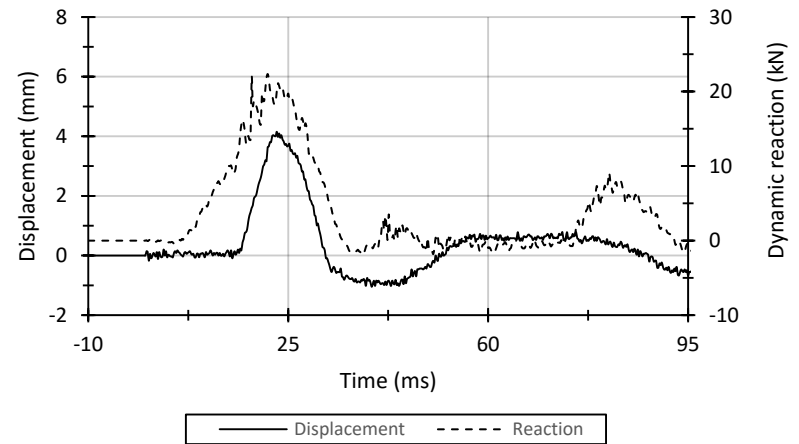


(d) Strain and reaction

Figure C.42: Dynamic test results for B8.2-[137]

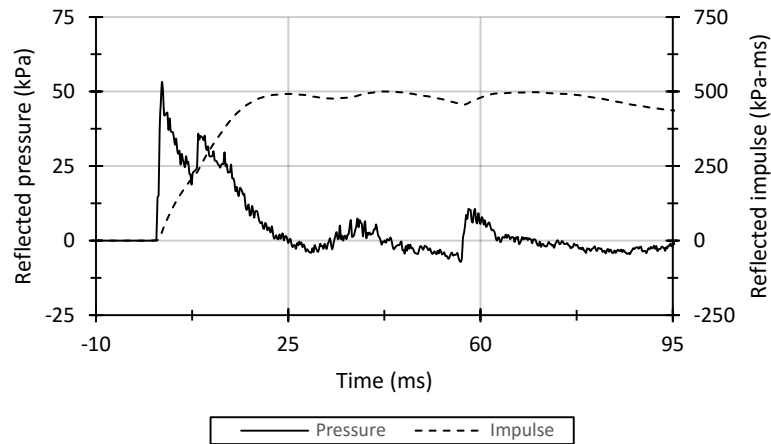


(a) Pressure and impulse

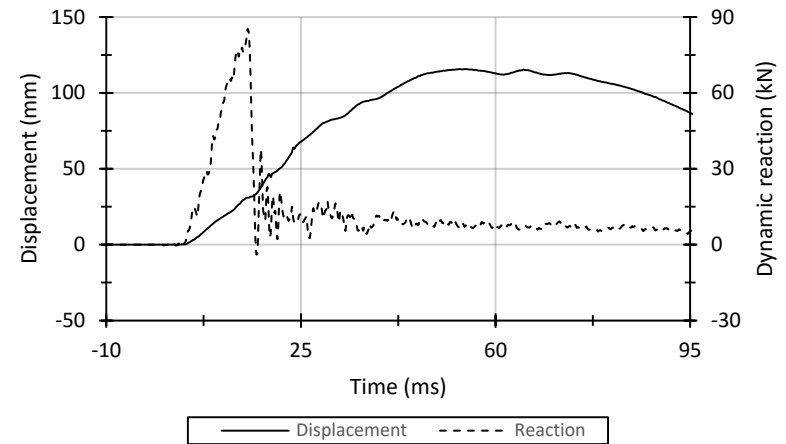


(b) Displacement and reaction

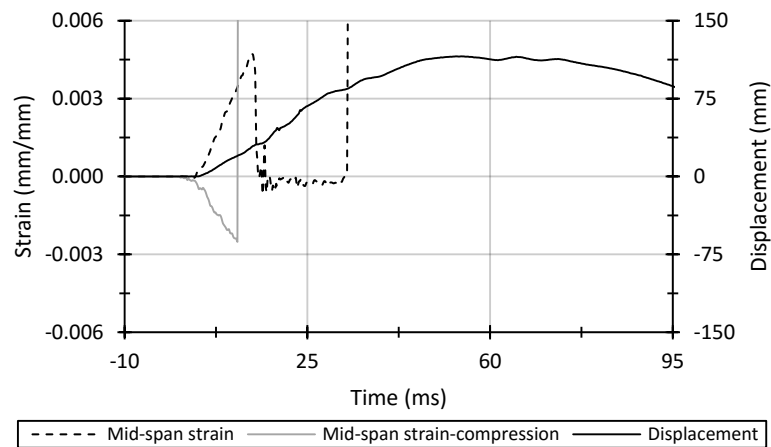
Figure C.43: Dynamic test results for B9.1-[137]



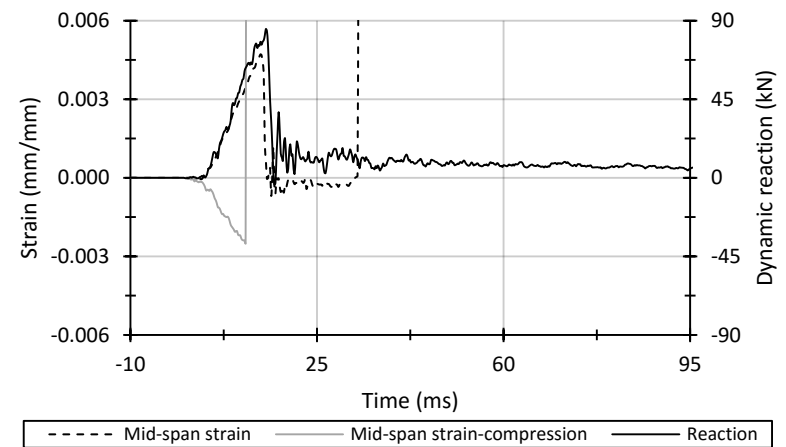
(a) Pressure and impulse



(b) Displacement and reaction



(c) Strain and displacement



(d) Strain and reaction

Figure C.44: Dynamic test results for B9.2-[137]



(a) B5.2-[137]



(b) B6.2-[137]



(c) B7.2-[137]



(d) B8.2-[137]



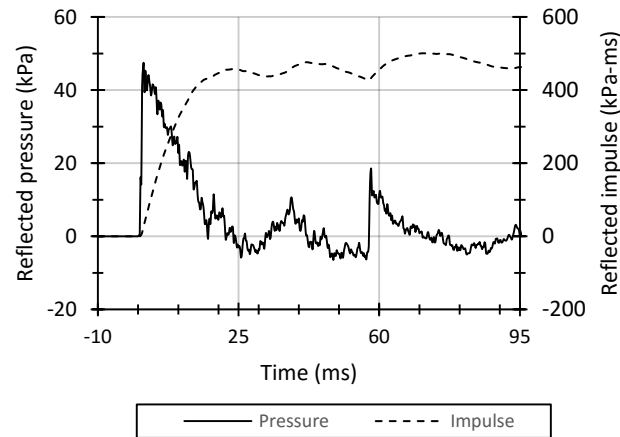
(e) B9.2-[137]

Figure C.45: Dynamic failure modes of B-[137] beams

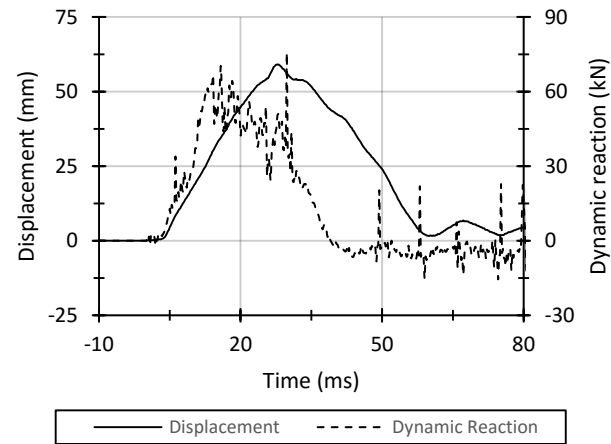
Appendix D – Test Results for Unretrofitted Glulam Columns

Table D.1: Columns dynamic test results

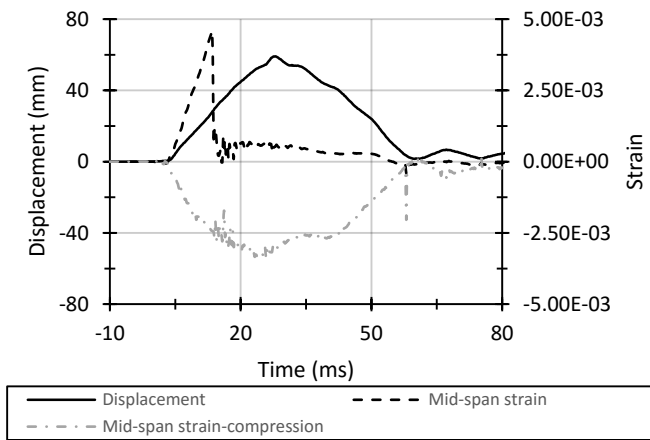
Specimen	P _R ^a (kPa)	I _R ^b (kPa-ms)	Δ _{max} ^c (mm)	t _{Δmax} ^d (ms)	R _{max,Δ} ^e (kN)	Δ _{Rmax,Δ} ^f (mm)	t _{Rmax,Δ} ^g (ms)	Δ _{ef-T} ^h (mm)	ε _{f-T} ⁱ × 10 ⁻⁴ (mm/mm)	t _{ef-T} ^j (ms)	ε̇ ^k (s ⁻¹)	ε _{f-C} ^l × 10 ⁻⁴ (mm/mm)	t _{ef-C} ^m (ms)	ε _{a,i} ⁿ × 10 ⁻⁴ (mm/mm)	P _{initial} ^o (kN)	P _{Rmax,Δ} ^p (kN)
C1.1-[137]	47.3	458.0	59.1	28.0	119.6	27.6	13.4	27.6	45.7	13.4	0.34	-33.6	25.8	-2.5	100.0	50.0
C2.1-[137]	54.1	604.8	102.9	29.6	145.1	30.2	13.4	28.1	35.4	12.8	0.28	-86.2	26.0	-2.2	160.0	108.9
C3.1-[137]	44.0	466.9	48.4	26.4	112.6	27.6	13.4	31.8	48.4	14.8	0.33	-43.4	20.4	-8.8	338.0	281.5
C4.1-[137]	60.4	664.2	62.6	17.6	157.7	30.0	13.4	30.7	49.8	12.4	0.40	-55.1	14.2	-9.6	330.0	271.7
C5.1-[137]	56.9	513.5	47.0	24.6	145.7	23.7	12.6	31.4	38.9	15.2	0.26	N/A ¹	N/A ¹	N/A ¹	540.2 ^{1,2}	531.7 ^{1,2}
C6.1-[137]	60.5	541.5	78.3	27.6	184.2	29.8	12.6	32.5	54.0	13.4	0.40	-202.1	20.0	-11.1	455.8 ^{1,2}	438.9 ^{1,2}
^a Reflected pressure				^b Reflected impulse				^c Maximum recorded displacement				^d Time to maximum recorded displacement				
^e Column dynamic resistance				^f Displacement at column max. resistance				^g Time to column max. resistance				^h Displacement at tensile rupture				
ⁱ Strain at tensile rupture				^j Time to tensile rupture				^k Strain rate				^l Maximum compressive strain				
^m Time to maximum compressive strain				ⁿ Iniatial axial strain due to concentric axial load				^o Initial applied axial load				^p Axial load at maximum resistance				
¹ Noise interference causing experimental time history to be unusable																
² Axial load interpolated linearly from hydraulic jack reading prior to and after the test																



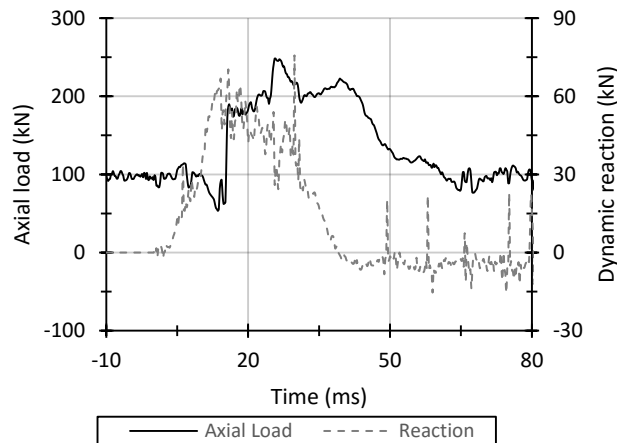
(a) Pressure and impulse



(b) Displacement and reaction



(c) Displacement and strain

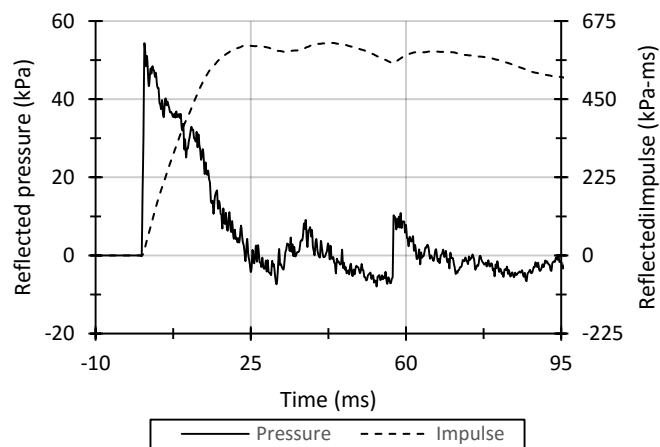


(d) Reaction and axial load

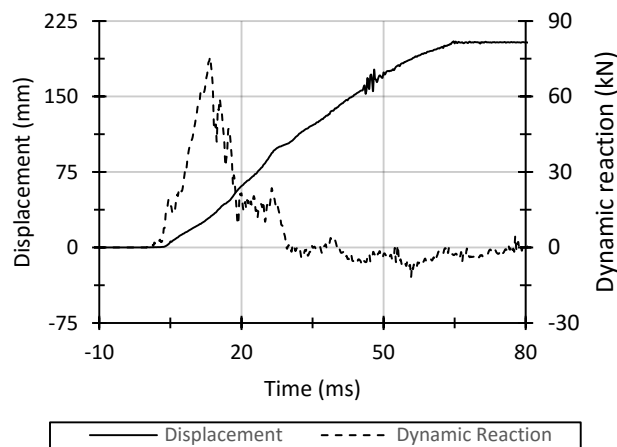


(e) Damage

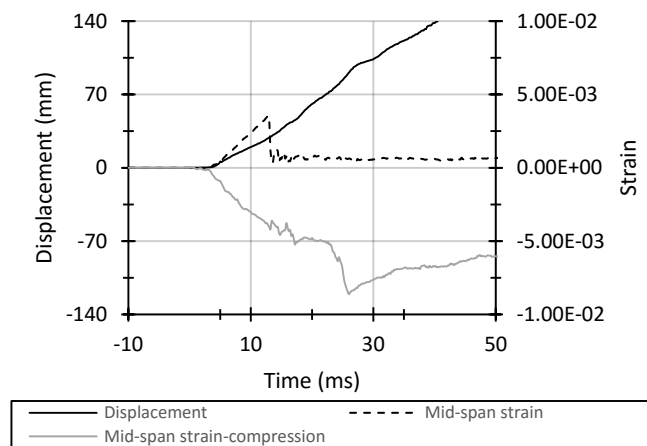
Figure D.1: Test results for C1.1-[137]



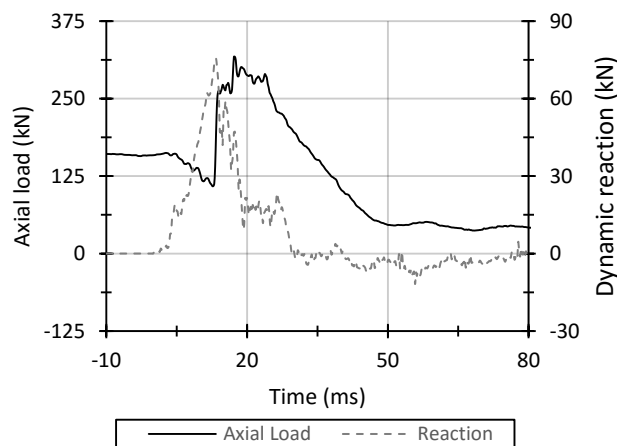
(a) Pressure and impulse



(b) Displacement and reaction



(c) Displacement and strain

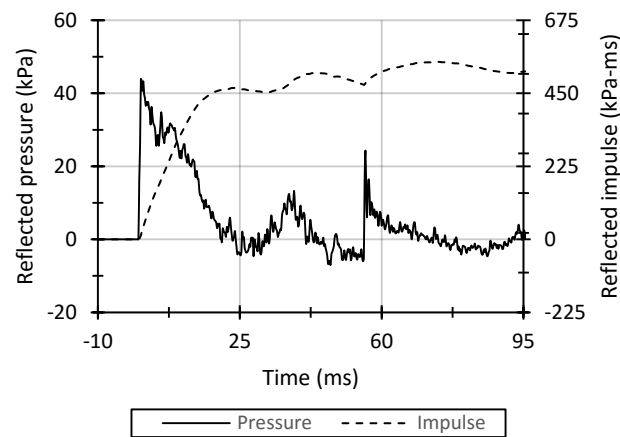


(d) Reaction and axial load

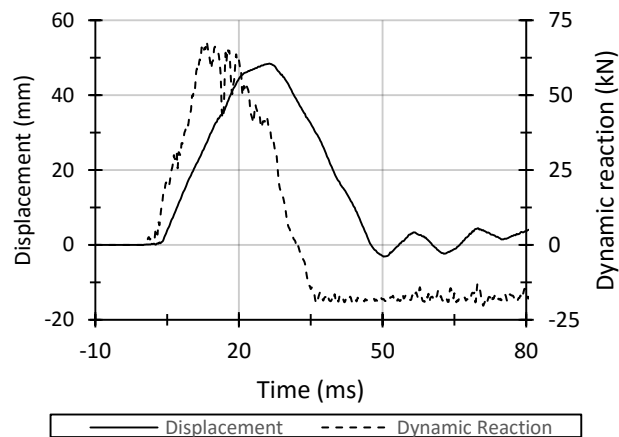


(e) Damage

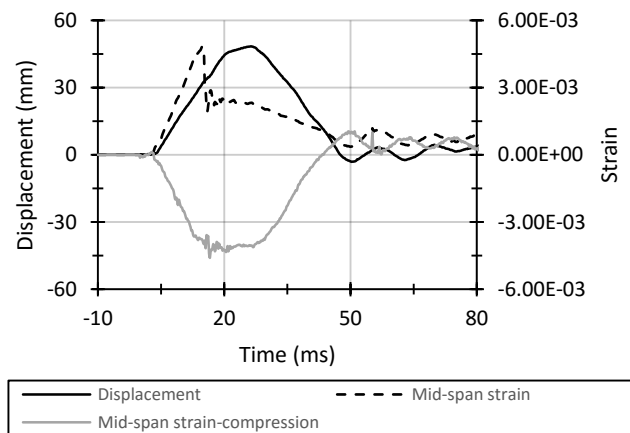
Figure D.2: Test results for C2.1-[137]



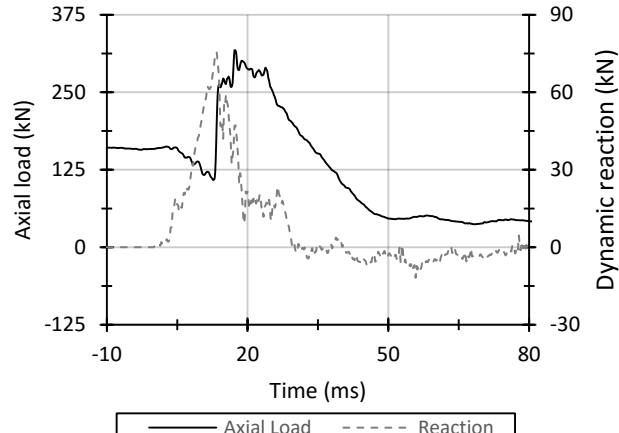
(a) Pressure and impulse



(b) Displacement and reaction



(c) Displacement and strain

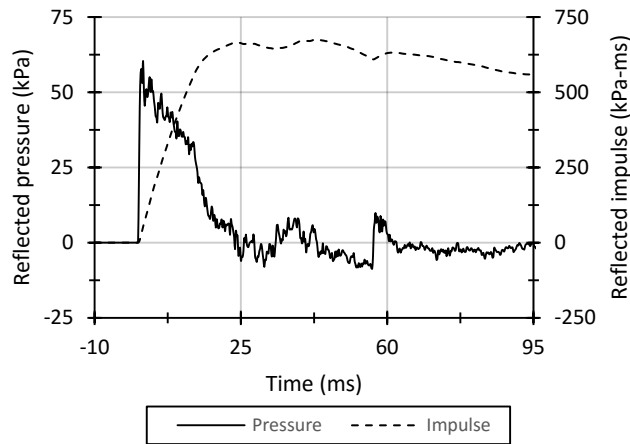


(d) Reaction and axial load

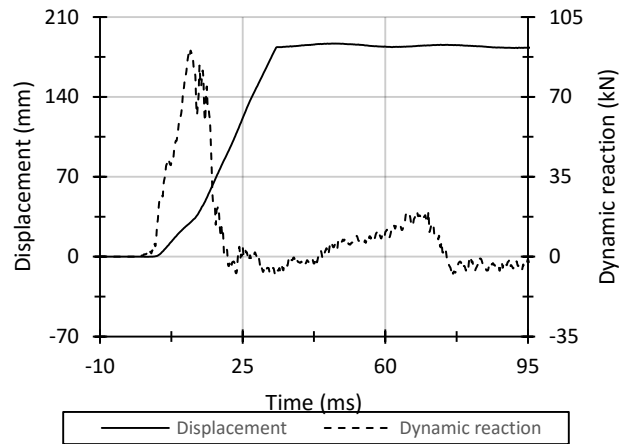


(e) Damage

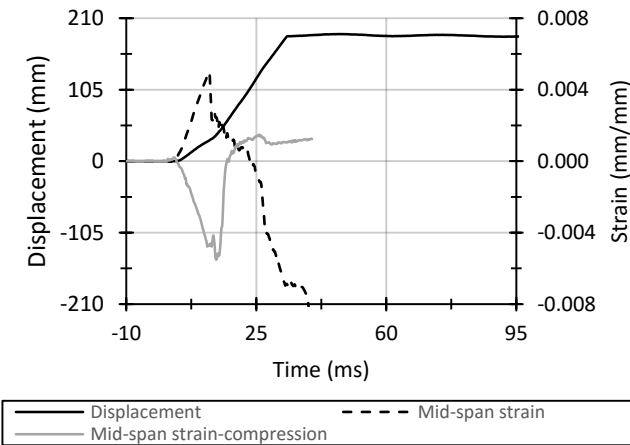
Figure D.3: Test results for C3.1-[137]



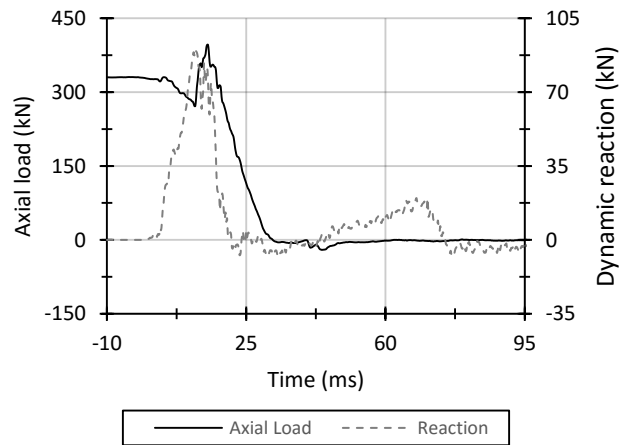
(a) Pressure and impulse



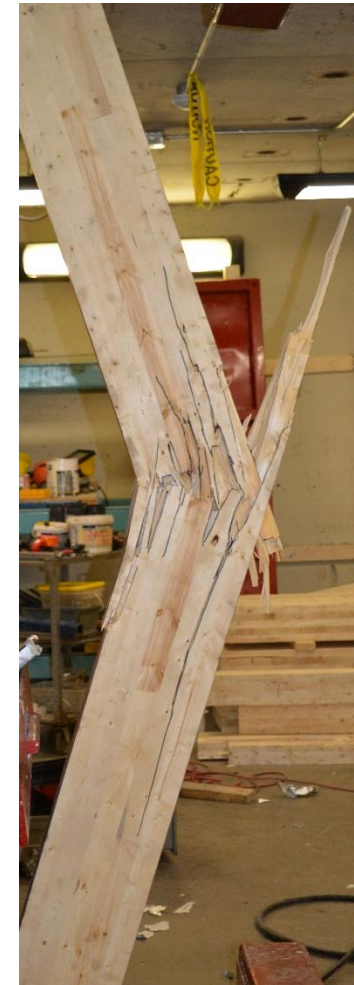
(b) Displacement and reaction



(c) Displacement and strain

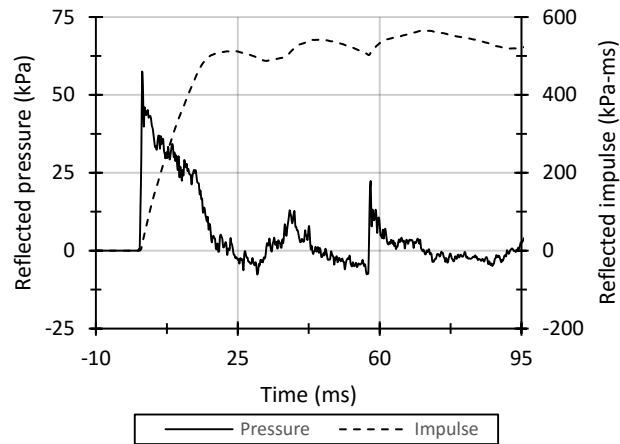


(d) Reaction and axial load

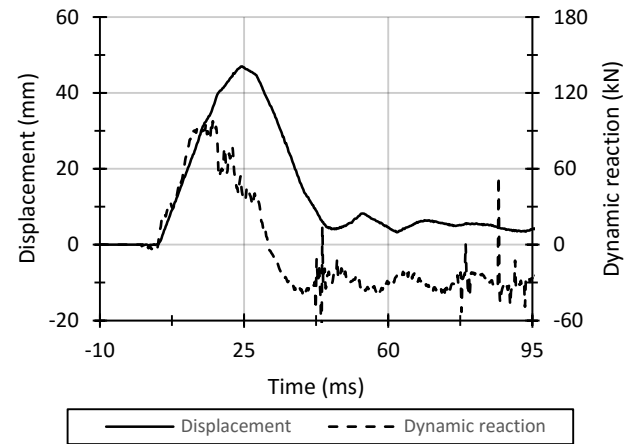


(e) Damage

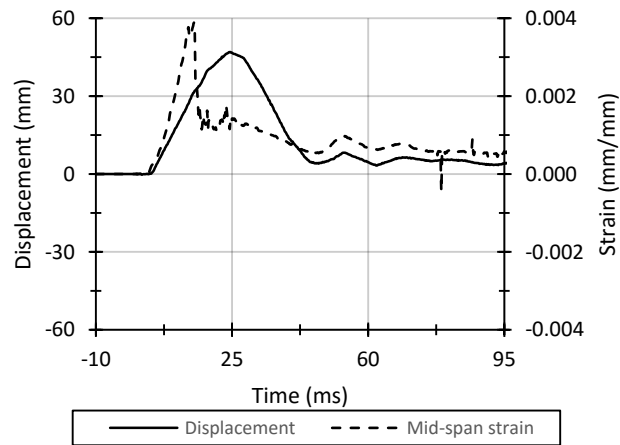
Figure D.4: Test results for C4.1-[137]



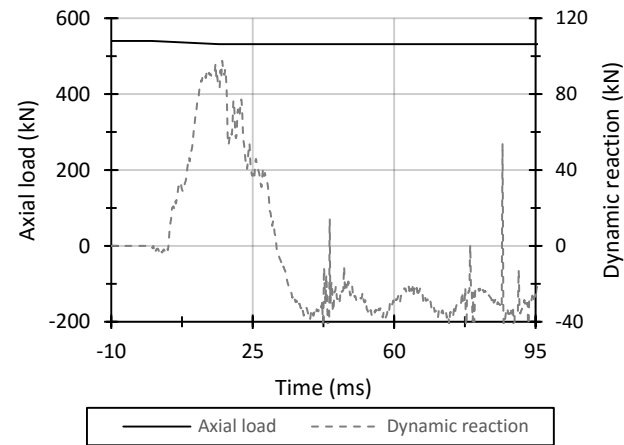
(a) Pressure and impulse



(b) Displacement and reaction



(c) Displacement and strain

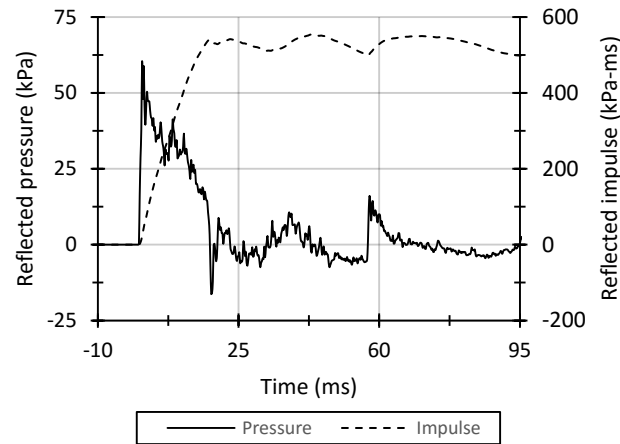


(d) Reaction and axial load

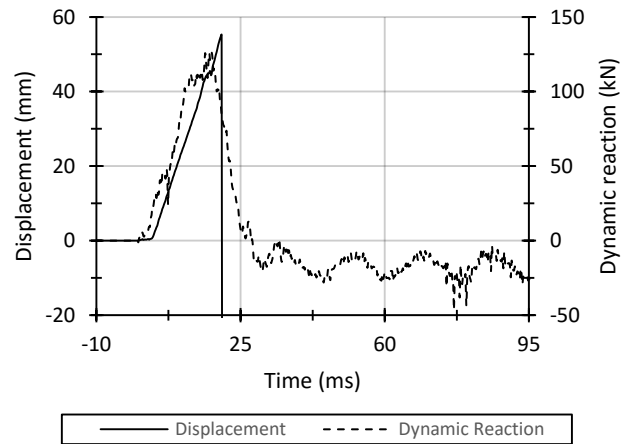


(e) Damage

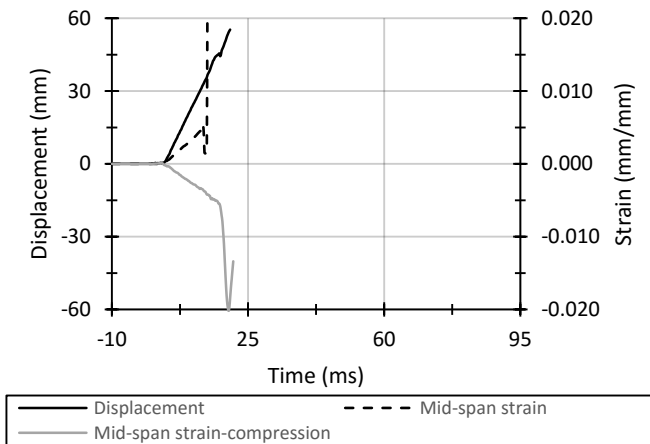
Figure D.5: Test results for C5.1-[137]



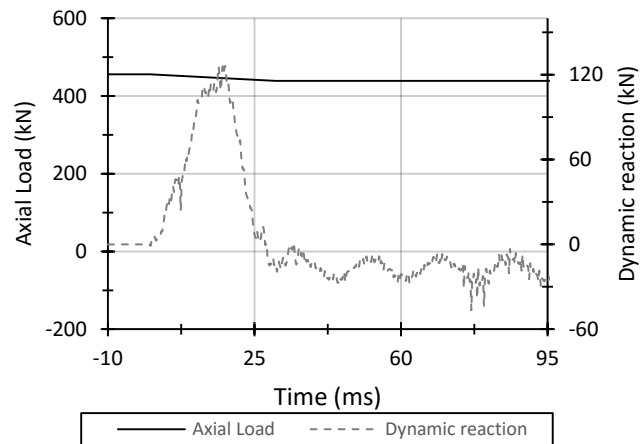
(a) Pressure and impulse



(b) Displacement and reaction



(c) Displacement and strain



(d) Reaction and axial load



(e) Damage

Figure D.6: Test results for C6.1-[137]

Appendix E – Test Results for Retrofitted Glulam Beams

Table E.1: Static test results of Retrofits 1 through 4

Specimen	R_{max}^a (kN)	ΔR_{max}^b (mm)	t_{Rmax}^c (s)	R_f^d (kN)	ΔR_f^e (mm)	t_{ARf}^f (min)	Δ_{max}^g (mm)	$t_{\Delta max}^h$ (min)	K^i (kN/mm)	$\epsilon_{t,w-f}^j \times 10^{-4}$ (mm/mm)	$\epsilon_{t,w-sf}^k \times 10^{-4}$ (mm/mm)	$t_{\epsilon t,w-f}^l$ (min)	$\epsilon_{t,FRP-f}^m \times 10^{-4}$ (mm/mm)	$t_{\epsilon t,FRP-f}^n$ (min)	$\epsilon_{c,w-f}^o \times 10^{-4}$ (mm/mm)	$t_{\epsilon c,w-f}^p$ (min)	$\dot{\epsilon}^q$ (s ⁻¹)
R1-1	196.7	32.9	4.02	196.7	32.9	4.02	155.5	6.42	6.4	39.2	37.5	4.16	40.2	4.19	-42.0	4.19	1.57E-05
R1-2	186.2	28.9	2.60	186.2	28.9	2.60	28.9	2.60	6.7	40.8	28.0	2.61	41.8	2.64	-37.8	3.73	2.61E-05
Average	191.5	30.9		191.5	30.9				6.6	40.0	32.7		41.0		-39.9		
R2-1	206.3	32.6	3.65	173.4	42.5	4.25	120.0	7.85	6.7	44.1	37.6	3.65	45.1	3.65	-47.3	5.08	2.01E-05
R2-2	189.8	33.2	2.80	189.8	33.2	2.80	110.7	5.11	6.8	49.0	35.0	2.83	51.4	2.77	-94.5	2.92	2.88E-05
Average	198.0	32.9		181.6	37.9				6.8	46.5	36.3		48.2		-70.9		
R3-1	214.3	38.5	3.53	207.0	53.5	4.41	85.6	6.34	6.6	43.6	-	3.28	94.2	4.40	-60.7	4.41	2.21E-05
R3-2	230.3	38.6	3.40	172.6	52.1	4.12	52.1	4.12	6.8	48.3	33.8	3.44	50.2	4.08	-	-	2.34E-05
Average	222.3	38.5		189.8	52.8				6.7	46.0	33.8		72.2		-60.7		
R4-1	242.2	40.3	3.80	242.2	40.3	3.80	63.5	4.69	7.5	43.2	20.3	3.80	44.2	3.80	-59.8	3.79	1.90E-05
R4-2	226.5	33.8	2.76	151.0	64.3	4.50	64.3	4.50	7.4	40.0	36.3	2.76	48.9	4.32	-62.0	2.76	2.42E-05
Average	234.4	37.0		196.6	52.3				7.4	41.6	28.3		46.5		-60.9		

^aMaximum resistance

^bDisplacement at maximum resistance

^cTime to maximum resistance

^dUltimate resistance

^eUltimate displacement

^fTime to ultimate resistance

^gMaximum recorded displacement

^hTime to maximum recorded displacement

ⁱStiffness

^jStrain at tensile rupture

^kWood side tensile failure strain

^lTime to tensile rupture

^mFRP failure strain

ⁿTime to FRP failure strain

^oMaximum compressive strain

^pTime to maximum compressive strain

^qStrain rate

Table E.2: Dynamic beams destructive test results for Retrofits 1 through 9 (Part I)

Specimen	P_R^a (kPa)	I_R^b (kPa-ms)	Δ_{max}^c (mm)	$t_{\Delta-max}^d$ (ms)	R_{max-d}^e (kN)	ΔR_{max}^f (mm)	t_{Rmax-d}^g (ms)	R_{f-d}^h (kN)	ΔR_f^i (mm)	K^j (kN/mm)
R1-3.1	52.2	577.1	96.3	40.0	156.1	26.7	12.0	156.1	26.7	6.9
R1-4.1	68.7	727.4	291.8	54.2	216.1	36.9	12.2	216.1	36.9	6.0
R2-3.1	62.3	717.6	84.7	28.4	198.0	32.7	12.4	119.4	72.6	6.0
R2-4.1	68.8	757.0	285.6	28.4	208.0	34.6	11.8	208.0	34.6	6.0
R3-3.1	63.5	698.4	65.0	22.8	213.0	40.3	14.0	168.7	65.0	6.4
R3-3.2	73.8	829.2	240.8	52.6	183.7	61.0	14.2	154.2	70.6	3.0
R3-4.1	67.6	818.2	59.6	23.4	242.3	41.7	14.2	161.5	59.6	6.5
R3-4.2	74.4	780.2	233.4	56.4	182.9	58.9	14.6	182.9	58.9	3.5
R4-3.1	54.4	785.0	36.2	19.6	237.4	36.0	19.0	237.4	36.0	7.2
R4-3.2	59.7	1037.0	168.6	74.4	295.7	46.6	15.6	295.7	46.6	7.0
R4-4.1	76.2	1110.0	64.6	26.0	274.6	43.5	15.2	252.1	56.7	6.6
R5-A	76.3	1,091	218.0	53.6	241.2	38.2	12.6	120.6	60.0	6.3
R5-B	71.5	1,092	156.5	40.6	242.9	37.7	12.4	121.5	128.0	6.5
R6-A	74.6	1,153	115.53	35.4	246.0	36.0	16.6	123.0	115.5	7.4
R6-B	65.7	1,170	138.8	32.2	234.3	38.9	13.0	117.2	122.0	6.4
R7-A	68.3	1,029	283.5	66.40	234.2	30.4	12.4	117.1	101.2	9.5
R7-B	70.6	1,166	64.6	25.0	274.2	26.3	14.2	137.1	64.5	10.8
R8-A	67.8	1,157	176.2	66.4	245.5	30.2	12.4	122.8	83.9	8.8
R8-B	82.0	1,169	100.4	22.8	239.5	30.1	12.0	119.7	89.0	8.5
R9-A	67.6	1,074	113.4	35.4	234.3	28.3	12.6	117.2	102.2	9.6
R9-B	78.5	1,139	61.3	18.2	223.4	26.7	10.4	111.7	61.3	8.8

^aReflected pressure

^bReflected impulse

^cMaximum recorded displacement

^dTime to maximum recorded displacement

^eMaximum dynamic resistance

^fDeflection at maximum resistance

^gTime to maximum resistance

^hUltimate resistance

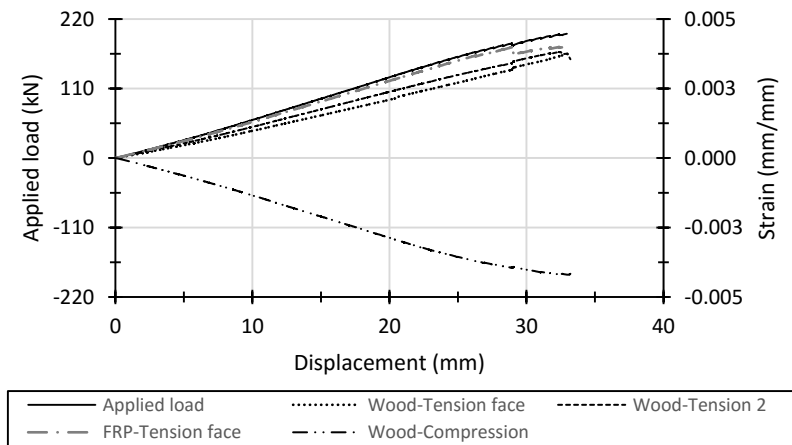
ⁱUltimate failure deflection

^jStiffness

Table E.3: Dynamic beams destructive test results for Retrofits 1 through 9 (Part II)

Specimen	Δ_{ef-t}^a (mm)	$\epsilon_{f-T}^b \times 10^{-4}$ (mm/mm)	$\epsilon_{f-Ts}^c \times 10^{-4}$	t_{ef-T}^d (ms)	$\epsilon_{f-FRP-UD}^e \times 10^{-4}$ (mm/mm)	$t_{ef-FRP-UD}^f$ (ms)	$\epsilon_{f-FRP-BD}^g \times 10^{-4}$ (mm/mm)	$t_{ef-FRP-BD}^h$ (ms)	$\epsilon_{f-C}^i \times 10^{-4}$ (mm/mm)	t_{ef-C}^j (ms)	$\dot{\epsilon}^k$ (s ⁻¹)
R1-3.1	26.7	39.5	32.5	12.0	88.9	40.0			-37.2	27.0	0.33
R1-4.1	33.59	49.1	40.5	36.9	51.9	12.4			-41.4	12.4	0.13
R2-3.1	31.90	42.8	34.7	12.2	61.1	12.0			-133.1	19.2	0.35
R2-4.1	-	-		-	-	-			-	-	-
R3-3.1	-	51.8	-	-	175.3	22.4			-113.9	18.2	-
R3-3.2	-	-	-	-	165.9	14.4	N/A		-73.6	13.8	
R3-4.1	30.13	49.8	33.8	11.2	118.6	20.4			-	-	0.44
R3-4.2	-	-		-	113.4	14.8			-	-	-
R4-3.1	-	-		-	51.6	19.0			-222.3	15.0	-
R4-3.2	-	53.5		13.0	80.9	16.0			-118.9	12.0	0.41
R4-4.1	43.03	47.4		15.0	69.3	18.4			-	-	0.32
R5-A	34.8	42.8	36.4	12.0	145.0	31.0	143.9	31.2	-164.3	26	0.36
R5-B	34.5	45.5	34.3	11.6	N/A		207.8	37.6	-69.9	23.8	0.39
R6-A	33.3	43.2	37.2	16.0	153.7	23.2	152.7	23.4	-54.7	22.6	0.27
R6-B	34.6	47.7	35.9	12.2	N/A		251.0	17.6	-59.7	18.0	0.39
R7-A	24.6	37.8	37.6	11.6	250.7	37.0	50.9	16.4	-98.7	18.6	0.33
R7-B	29.5	53.3	28.4	14.8	124.1	27.8	130.1	27.8	-47.3	21.2	0.33
R8-A	25.6	37.2	31.1	10.4	205.6	20.6	150.2	20.2	-	-	0.36
R8-B	17.5	42.3	33.5	8.2	N/A		251.6	37.0	-62.1	14.8	0.52
R9-A	22.8	46.4	39.6	10.6	65.2	10.2	47.5	12.4	-146.6	25.4	0.44
R9-B	24.9	44.9	36.2	9.6	206.5	17.4	165.8	17.2	-182.8	29.2	0.47

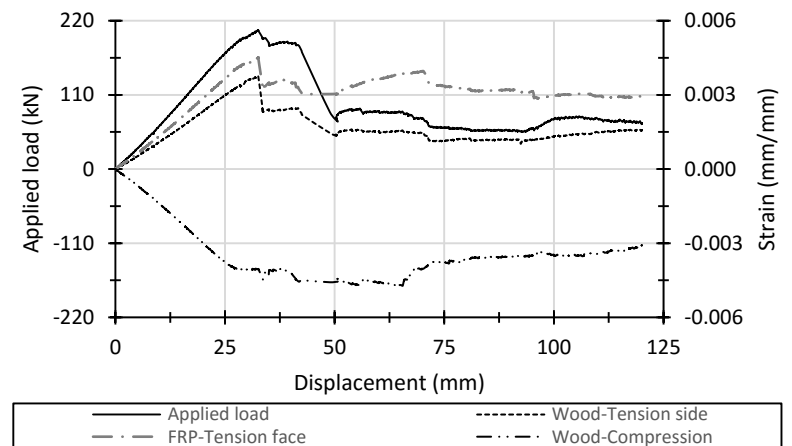
^aDisplacement at wood tensile rupture^bStrain at wood tensile rupture (tension face)^cStrain at wood tensile rupture (21.5 mm, side)^dTime to wood tensile failure^eUnidirectional FRP tensile failure strain^fTime to unidirectional FRP tensile failure strain^gBi-directional FRP tensile failure strain^hTime to bi-directional FRP tensile failure strainⁱMaximum wood compressive strain^jTime to maximum wood compressive strain^kStrain rate



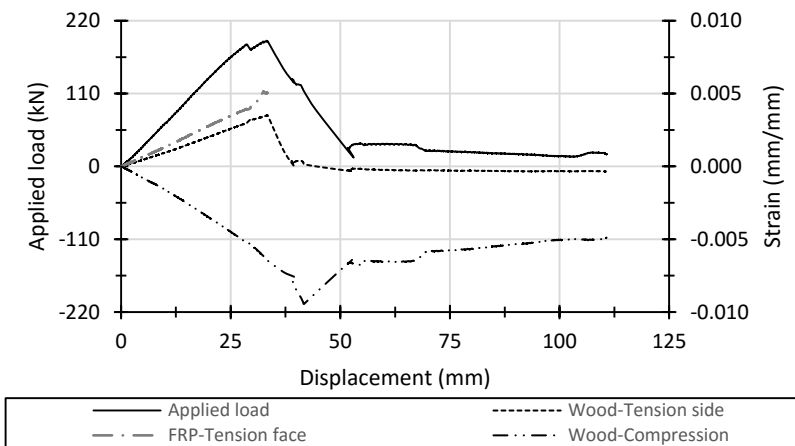
(a) R1-1-[137]



(b) R1-2-[137]



(c) R2-1-[137]



(d) R2-2-[137]

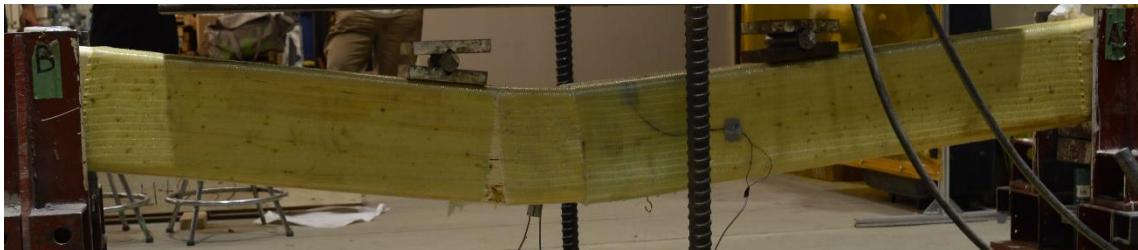
Figure E.1: Static test results for Retrofits 1 and 2



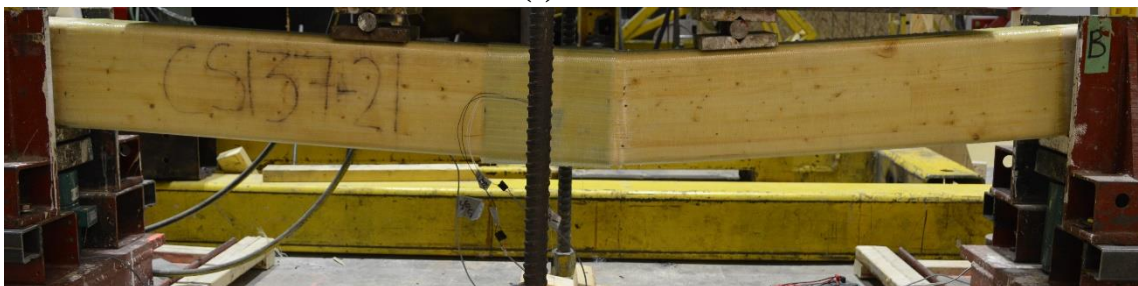
(a) R1-1



(b) R1-2

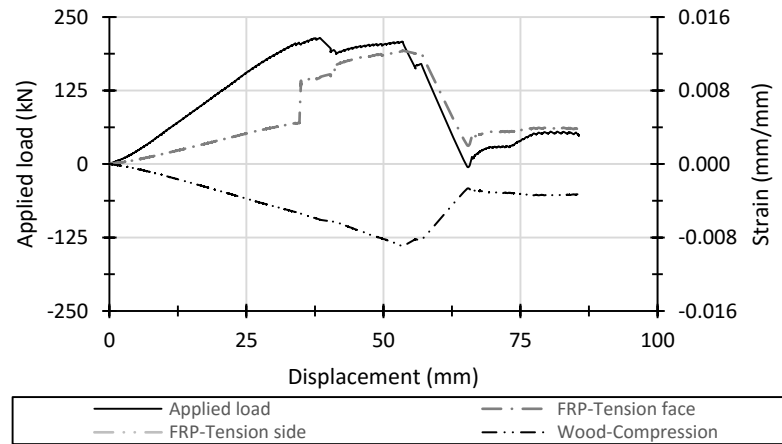


(c) R2-1

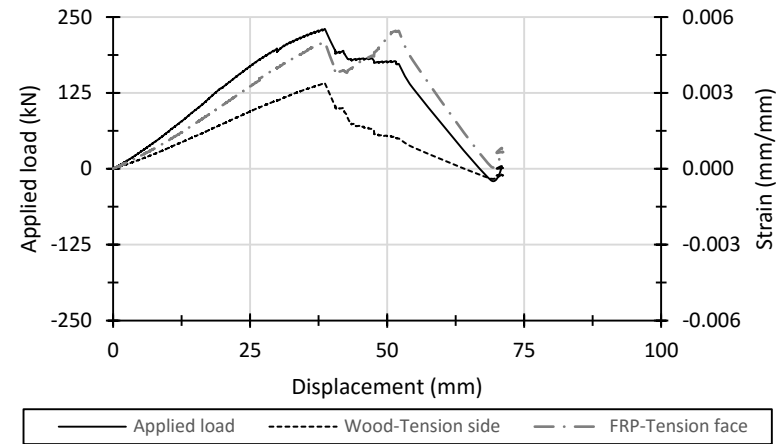


(d) R2-2

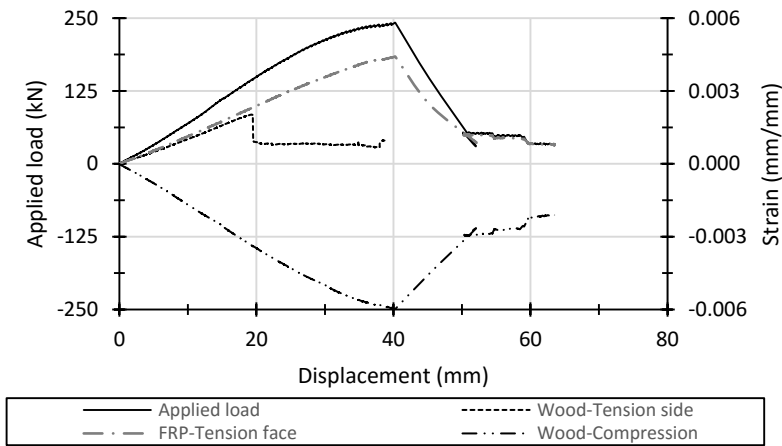
Figure E.2: Static failure modes of Retrofits 1 and 2



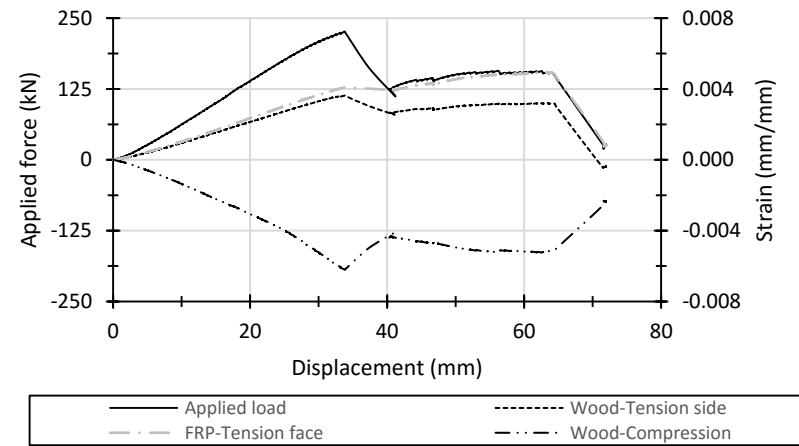
(a) R3-1-[137]



(b) R3-2-[137]



(c) R4-1-[137]

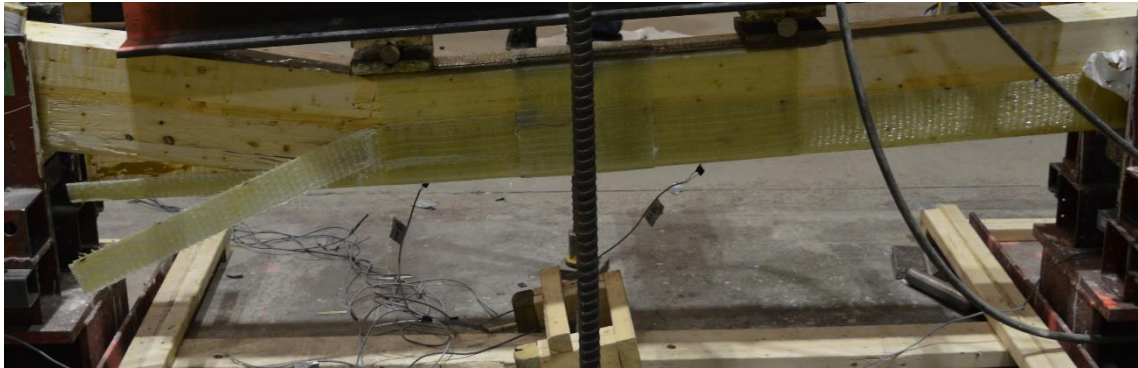


(d) R4-2-[137]

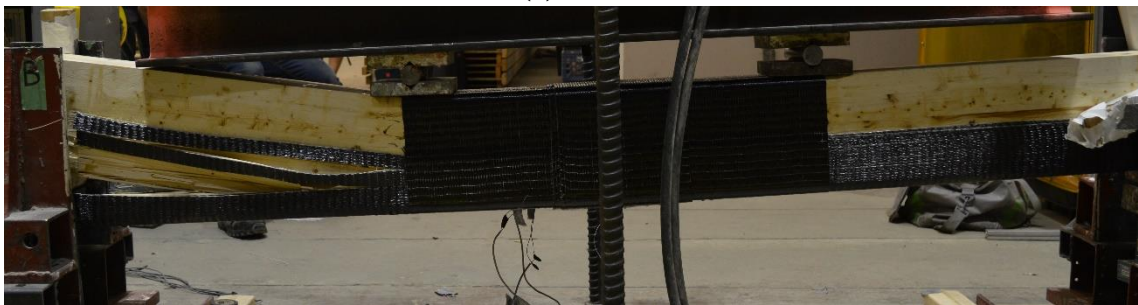
Figure E.3: Static test results for Retrofits 3 and 4



(a) R3-1



(b) R3-2

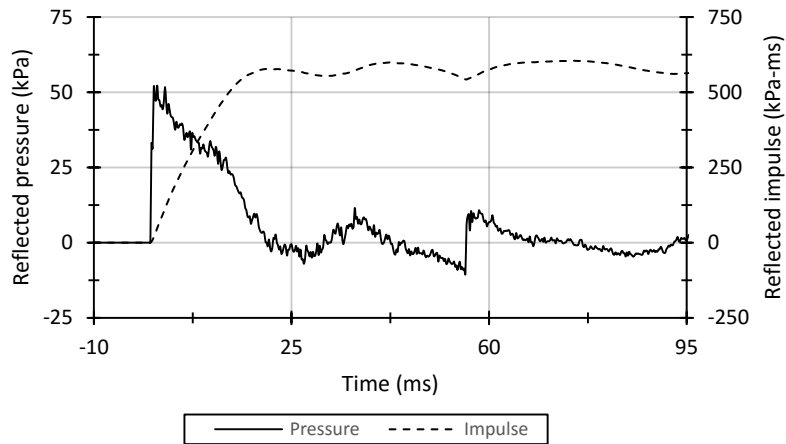


(c) R4-1

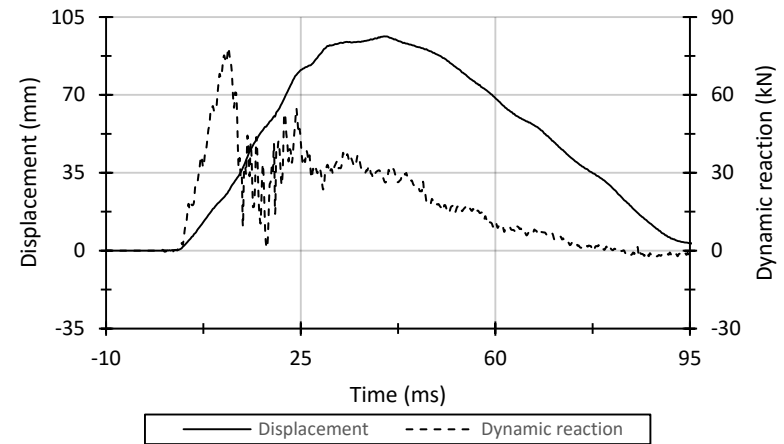


(d) R4-2

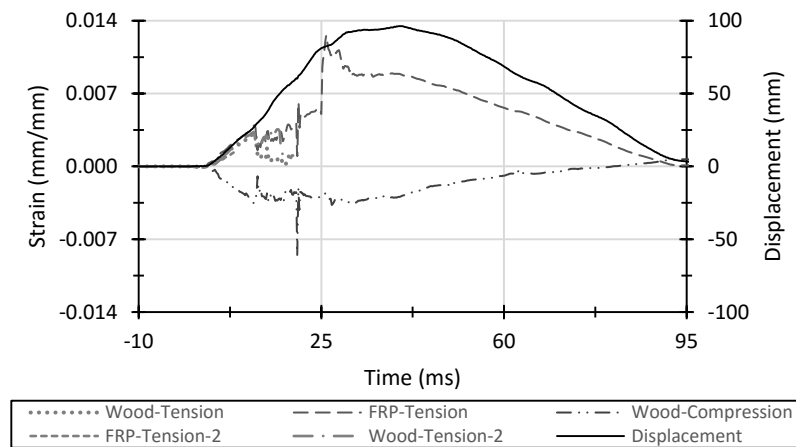
Figure E.4: Static failure modes of Retrofits 3 and 4



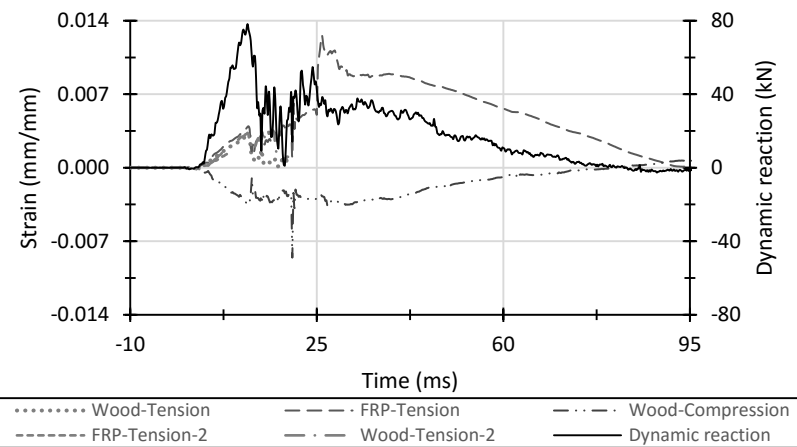
(a) Pressure and impulse



(b) Displacement and reaction

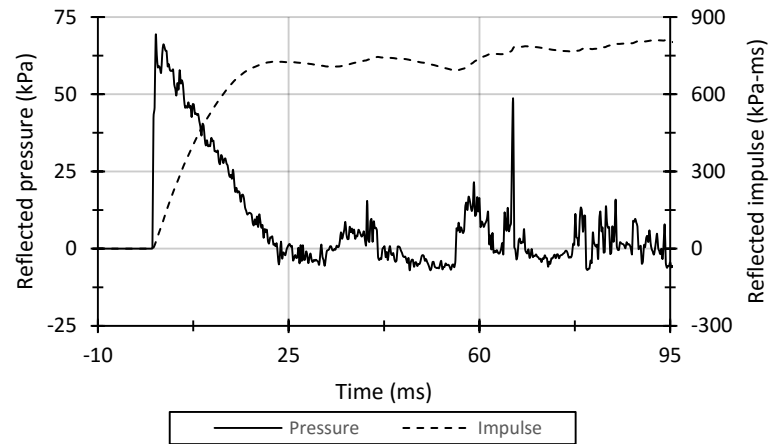


(c) Strain and displacement

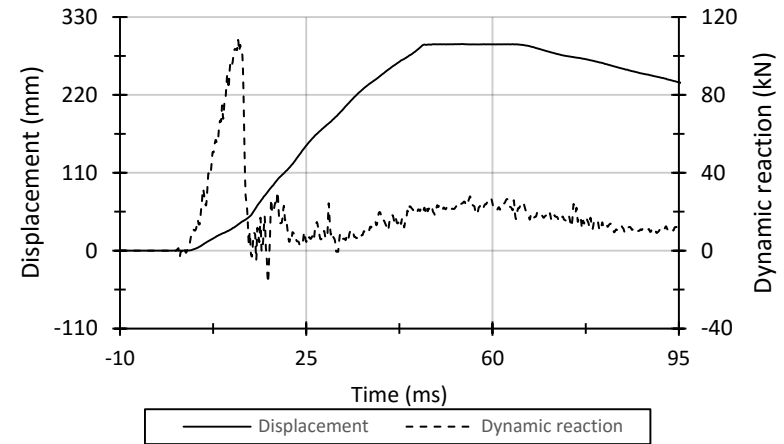


(d) Strain and reaction

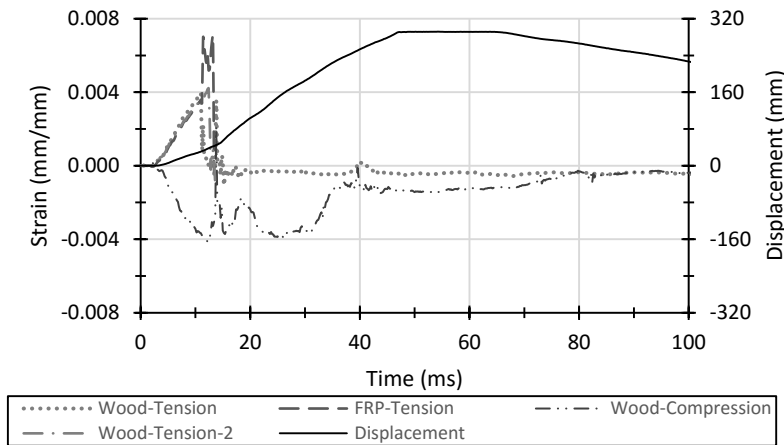
Figure E.5: Test results for R1-3.1-[137]



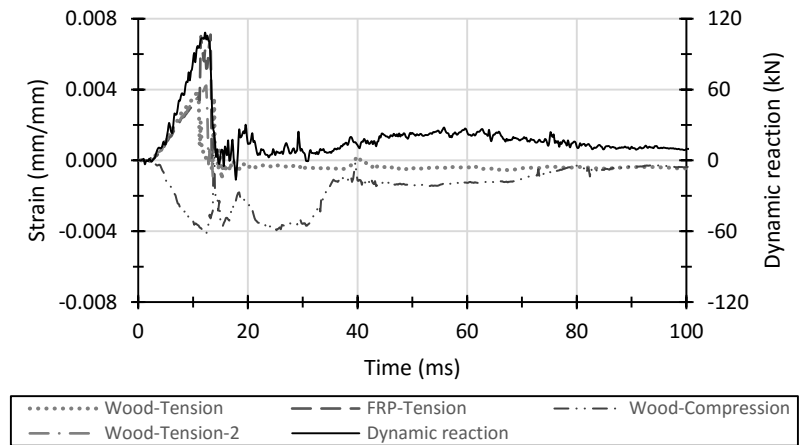
(a) Pressure and impulse



(b) Displacement and reaction



(c) Strain and displacement



(d) Strain and reaction

Figure E.6: Test results for R1-4.1-[137]



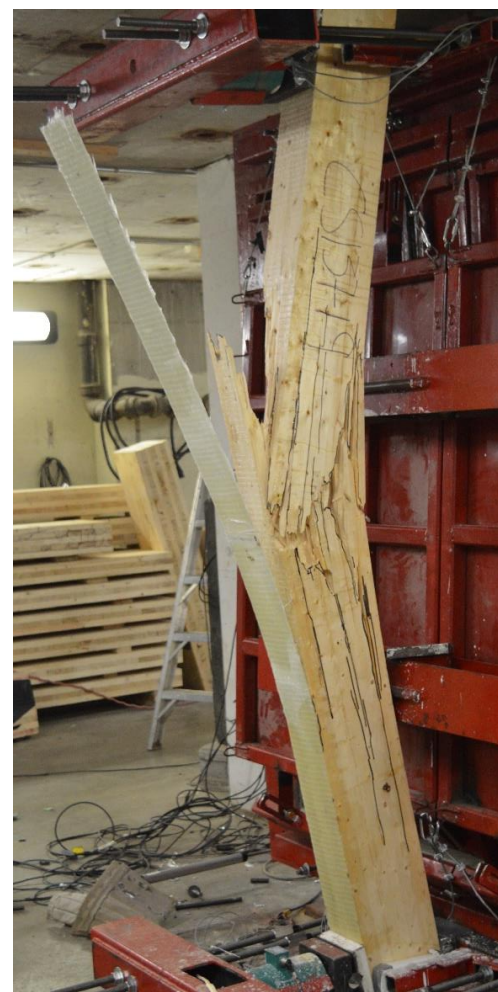
(a) R1-3.1 ($R_{\max-d}$)



(b) R1-3.1

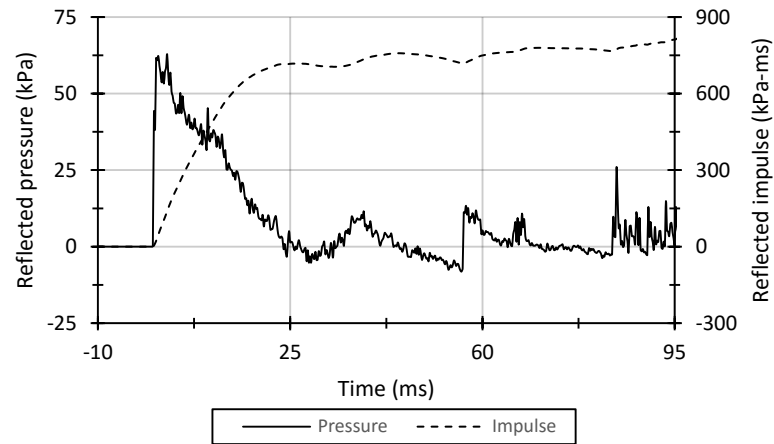


(c) R1-4.1 ($R_{\max-d}$)

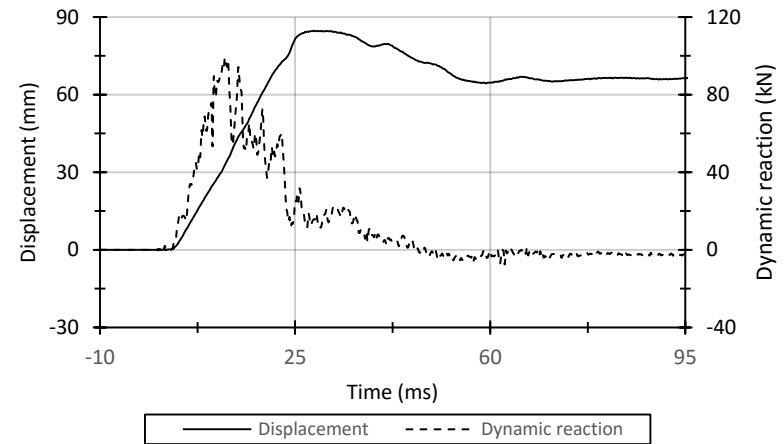


(d) R1-4.1

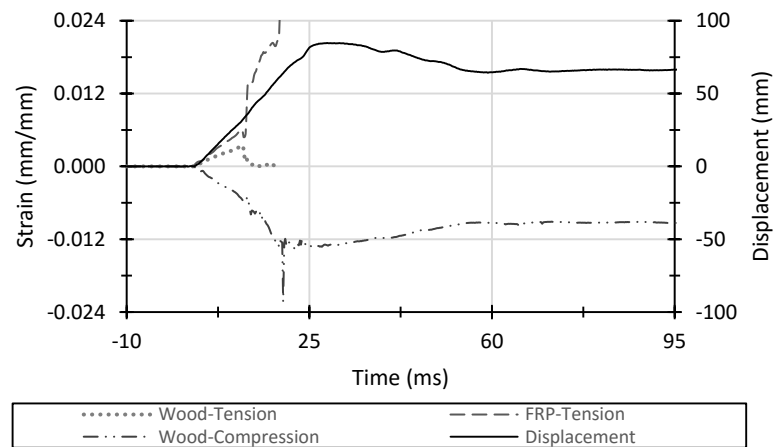
Figure E.7: Dynamic failure modes of Retrofit 1



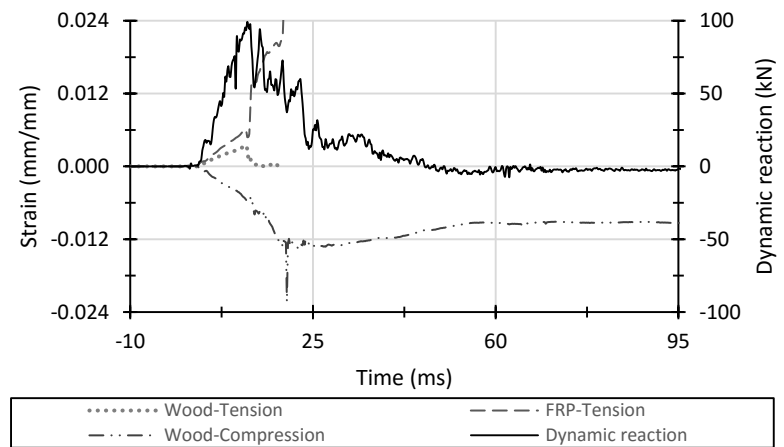
(a) Pressure and impulse



(b) Displacement and reaction

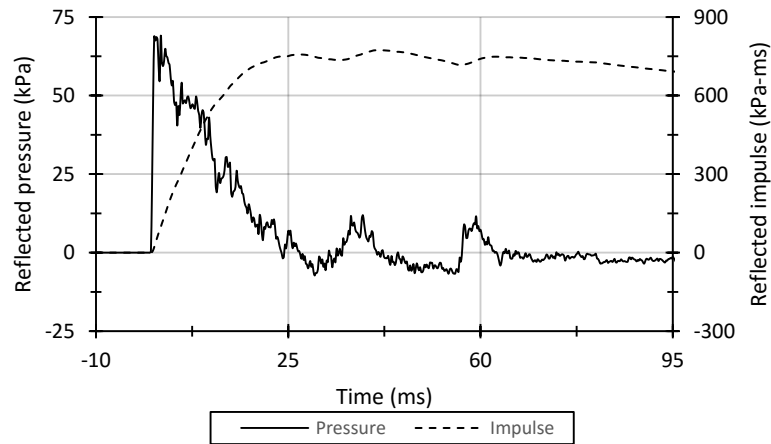


(c) Strain and displacement

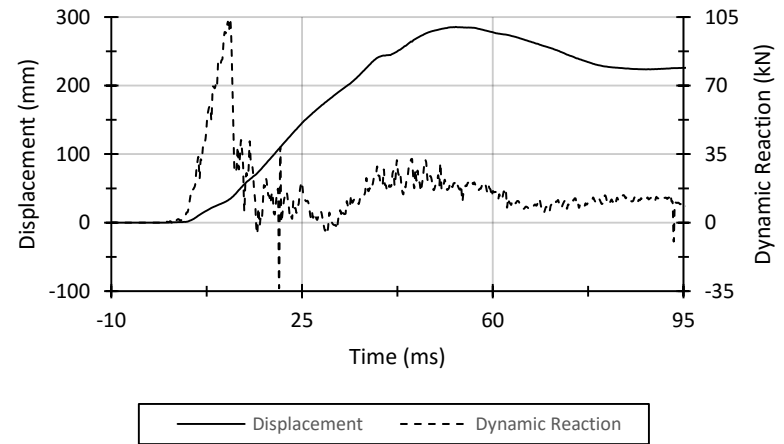


(d) Strain and reaction

Figure E.8: Test results for R2-3.1-[137]



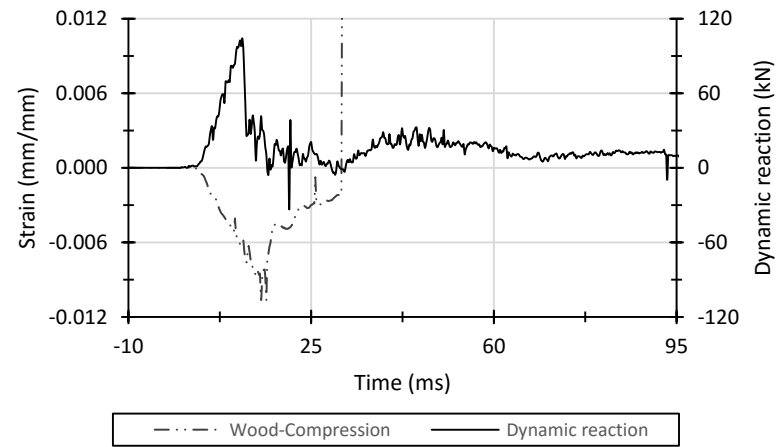
(a) Pressure and impulse



(b) Displacement and reaction



(c) Strain and displacement



(d) Strain and reaction

Figure E.9: Test results for R2-4.1-[137]



(a) R2-3.1 ($\Delta_{\max}$)



(b) R2-3.1

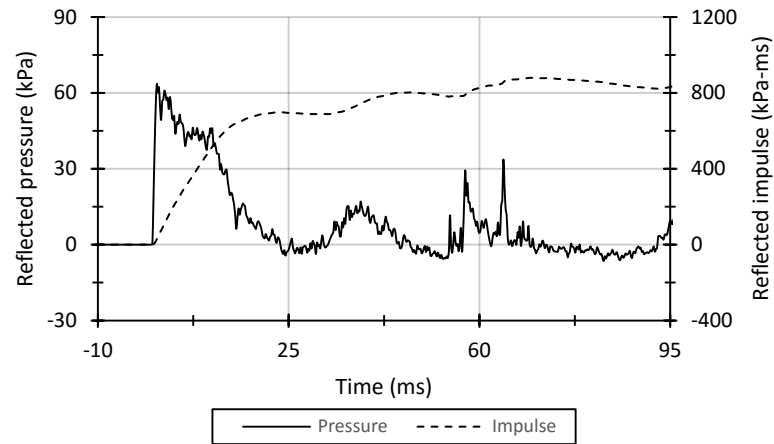


(c) R2-4.1 (Δ_{Rf})

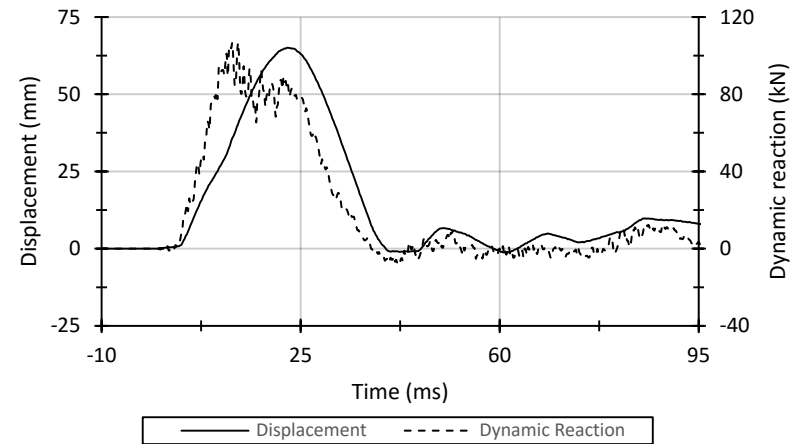


(d) R2-4.1

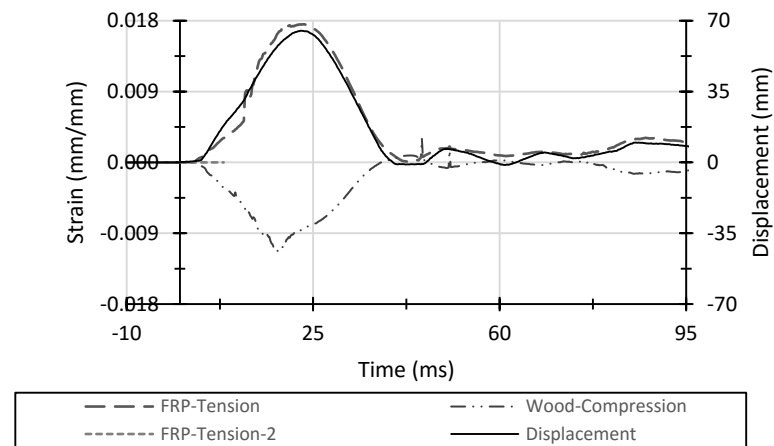
Figure E.10: Dynamic failure modes of Retrofit 2



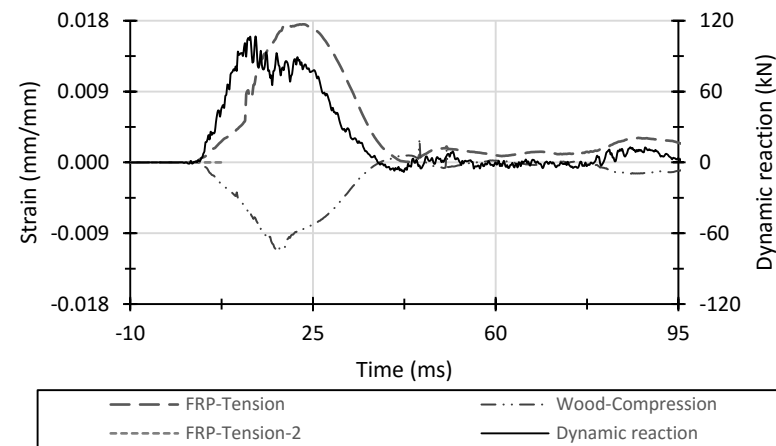
(a) Pressure and impulse



(b) Displacement and reaction

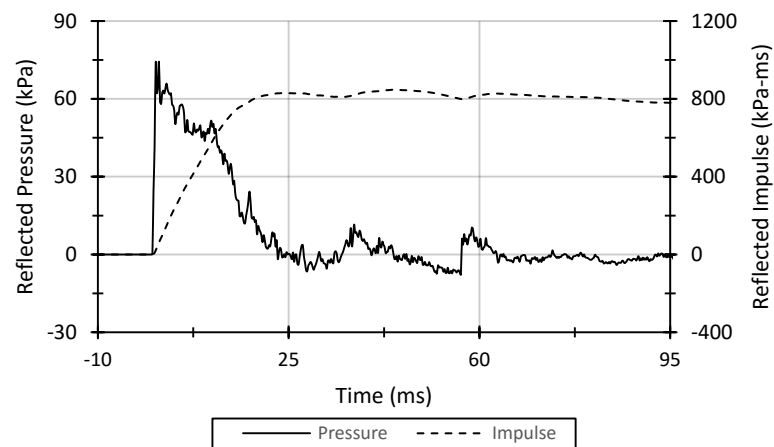


(c) Strain and displacement

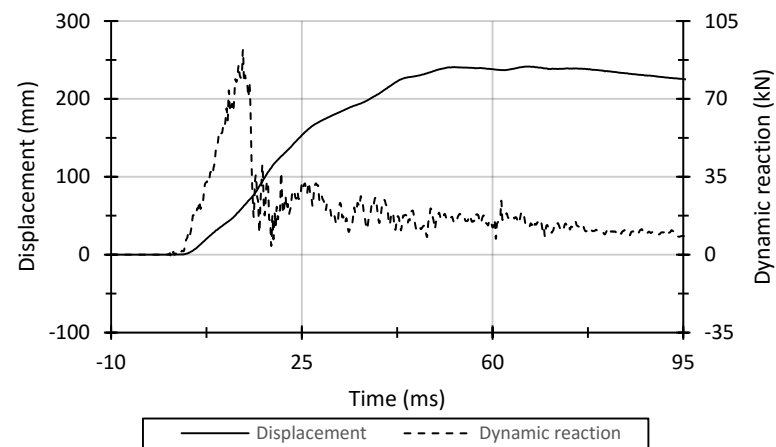


(d) Strain and reaction

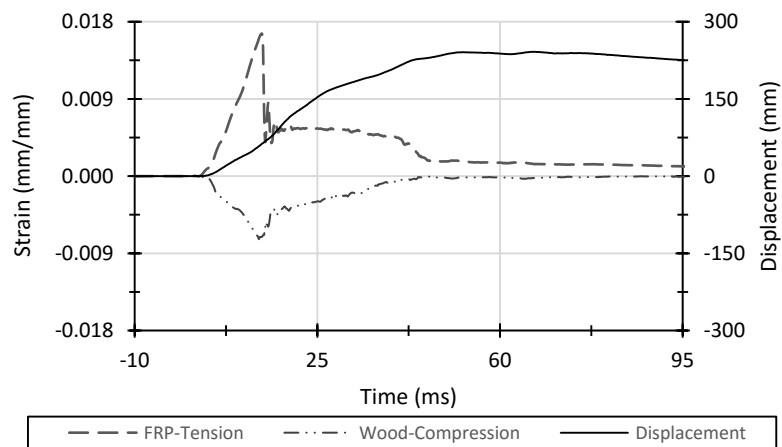
Figure E.11: Test results for R3-3.1-[137]



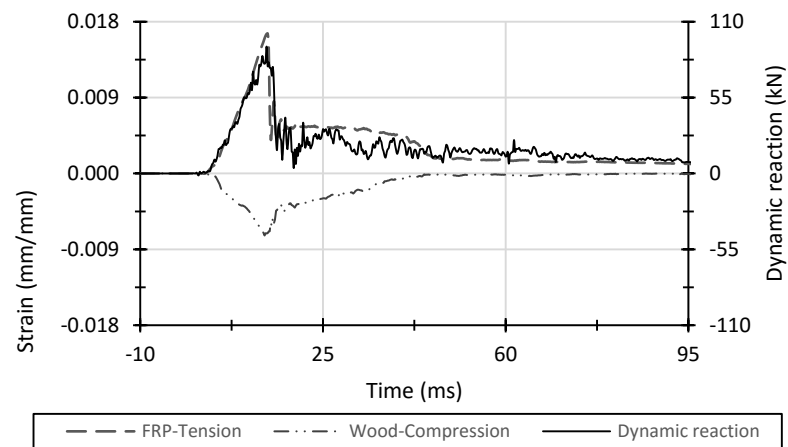
(a) Pressure and impulse



(b) Displacement and reaction

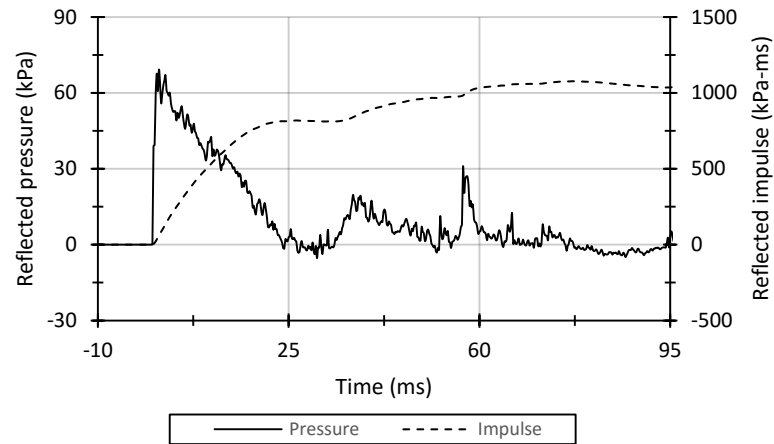


(c) Strain and displacement

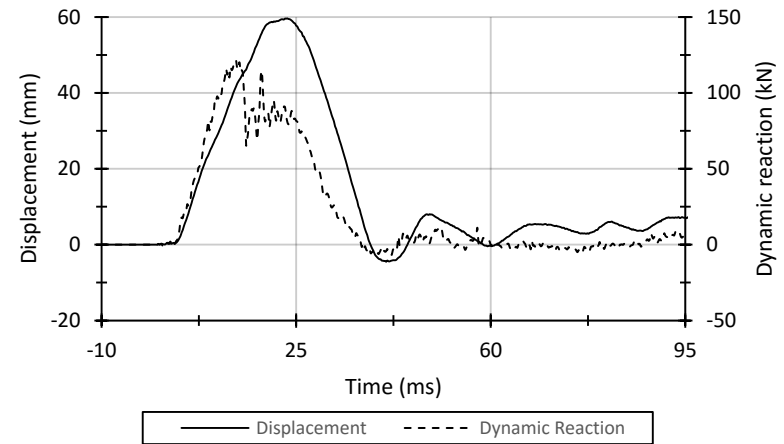


(d) Strain and reaction

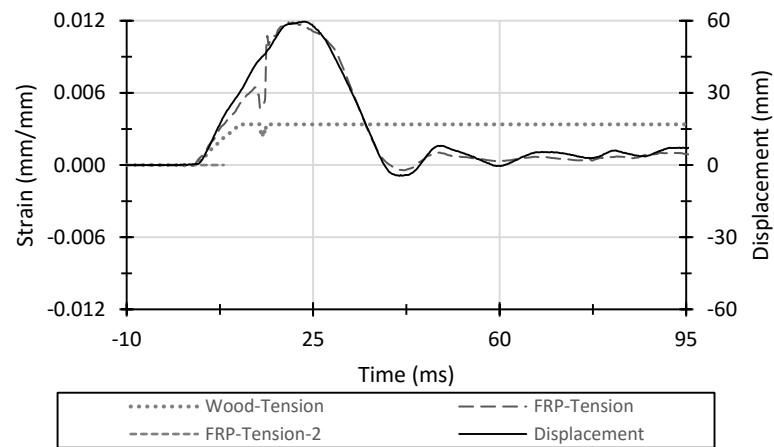
Figure E.12: Test results for R3-3.2-[137]



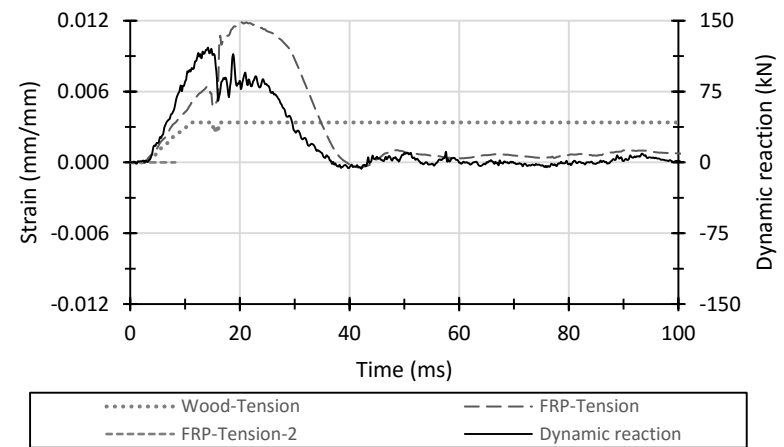
(a) Pressure and impulse



(b) Displacement and reaction

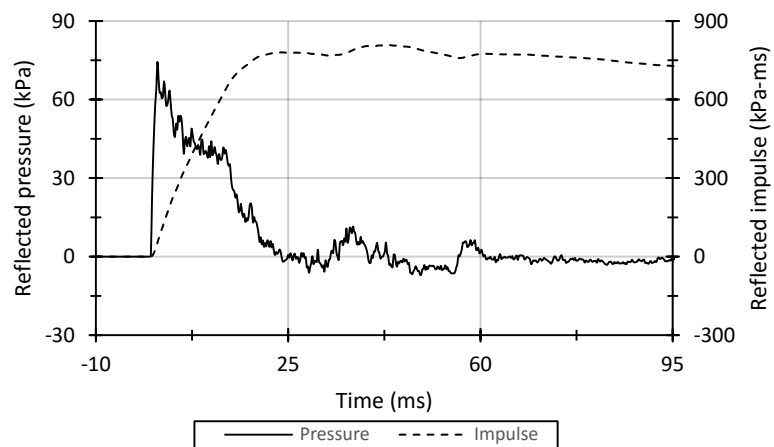


(c) Strain and displacement

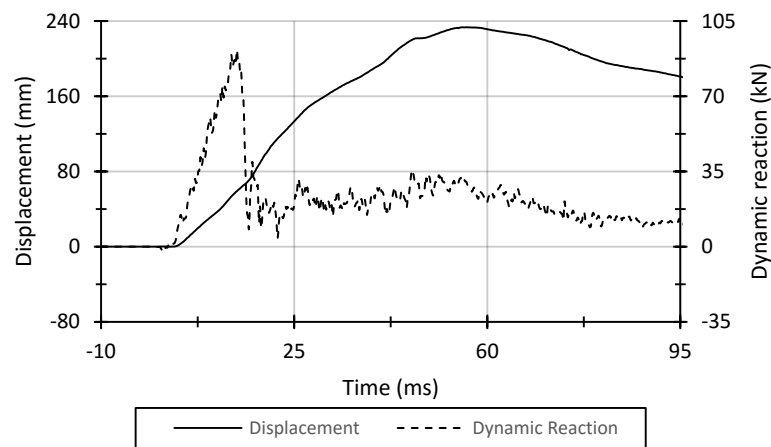


(d) Strain and reaction

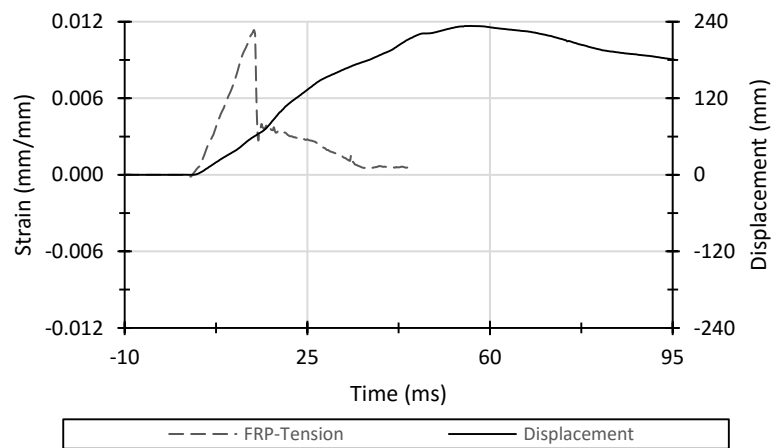
Figure E.13: Test results for R3-4.1-[137]



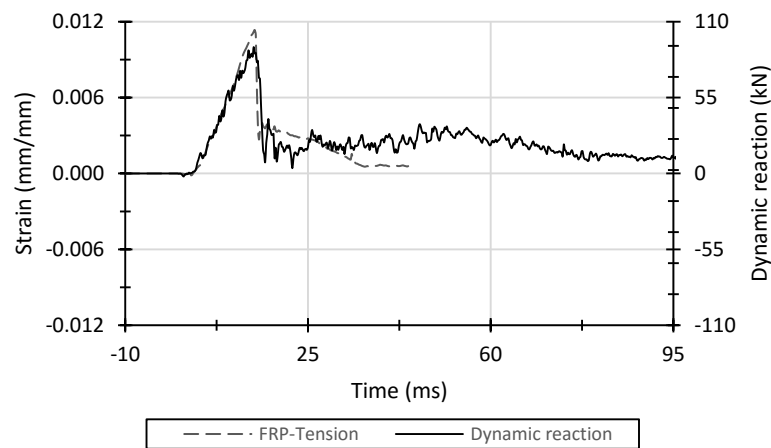
(a) Pressure and impulse



(b) Displacement and reaction



(c) Strain and displacement



(d) Strain and reaction

Figure E.14: Test results for R3-4.2-[137]



(a) R3-3.1 ($\Delta_{\max}$)



(b) R3-3.2

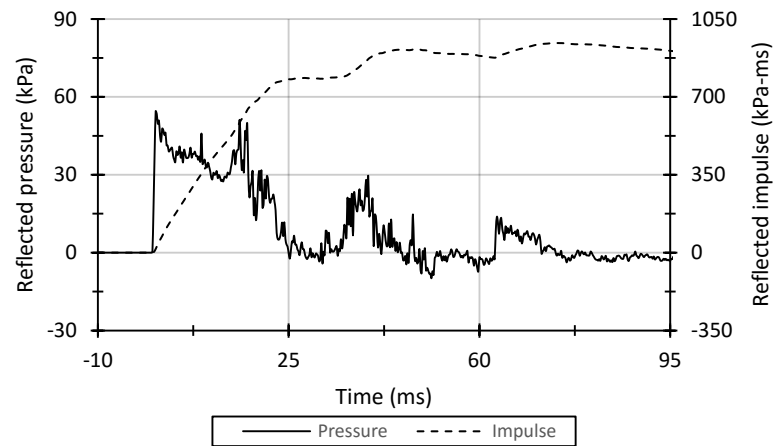


(c) R3-4.1 ($\Delta_{\max}$)

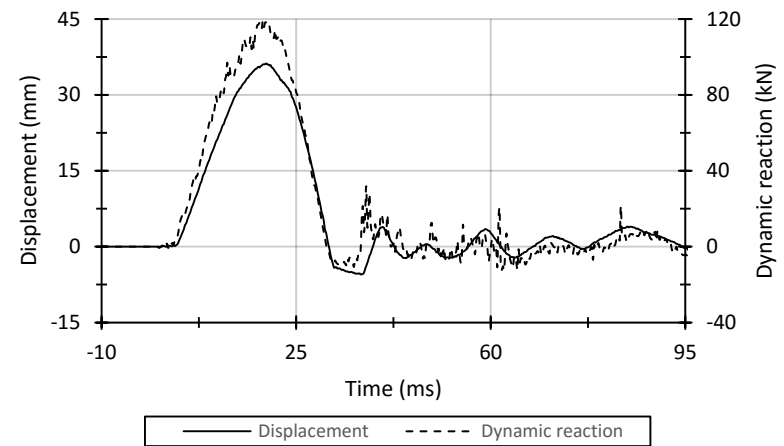


(d) R3-4.2

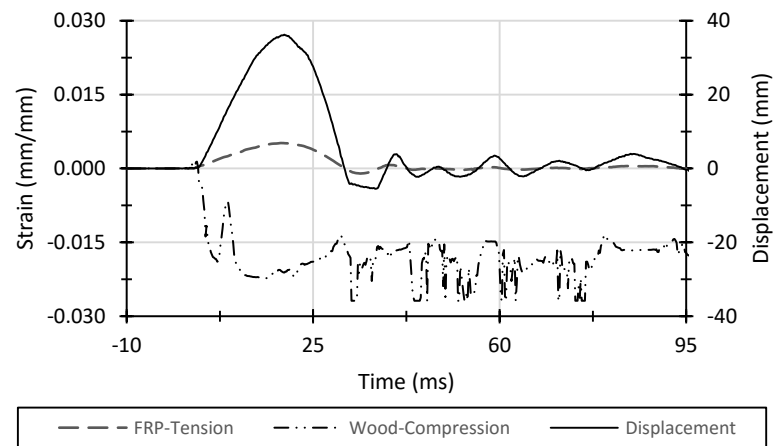
Figure E.15: Dynamic failure modes of Retrofit 3



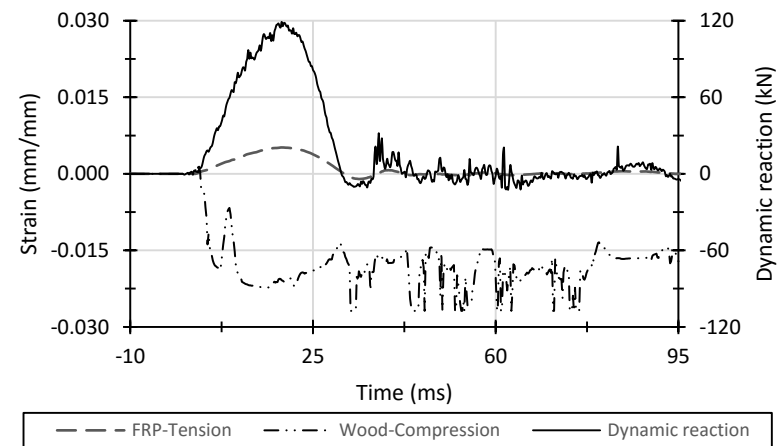
(a) Pressure and impulse



(b) Displacement and reaction

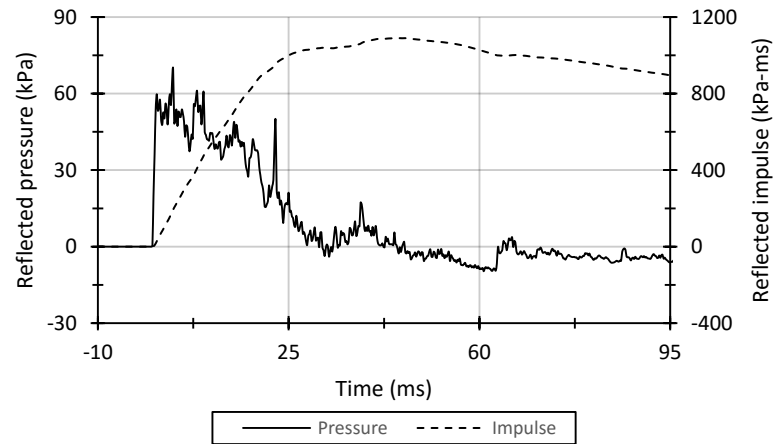


(c) Strain and displacement

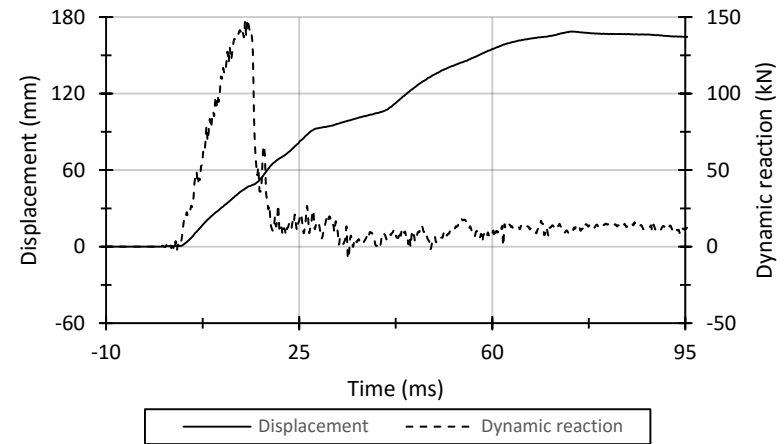


(d) Strain and reaction

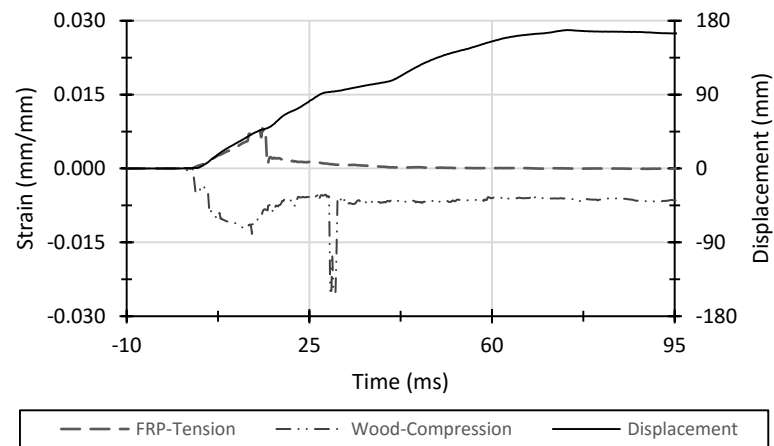
Figure E.16: Test results for R4-3.1-[137]



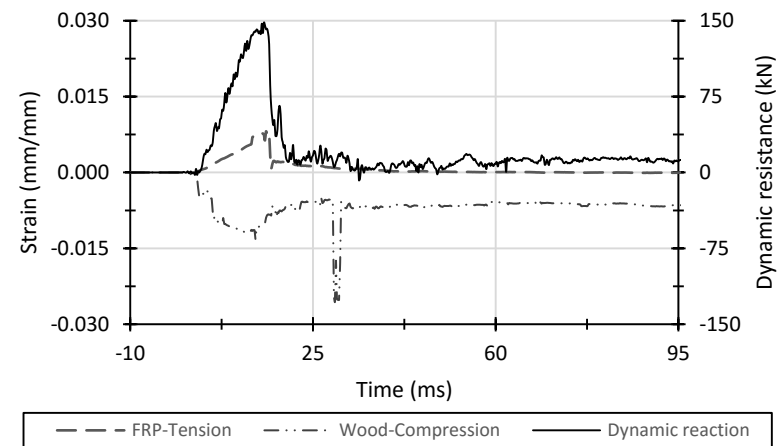
(a) Pressure and impulse



(b) Displacement and reaction

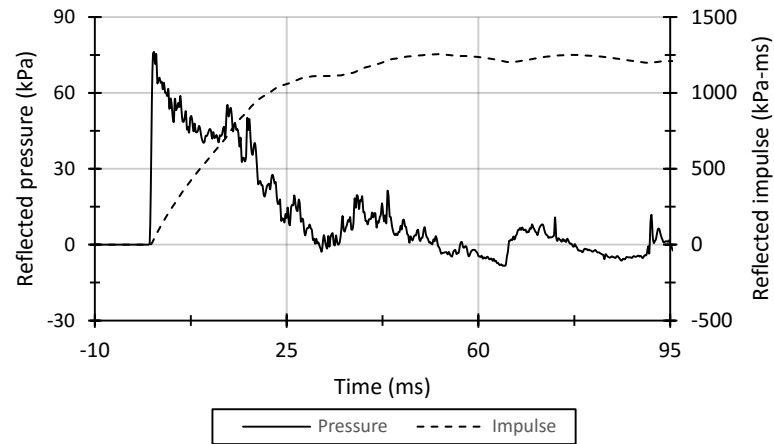


(c) Strain and displacement

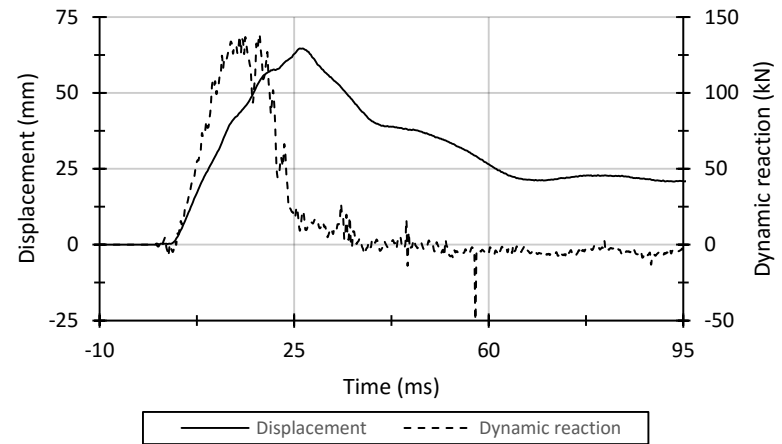


(d) Strain and reaction

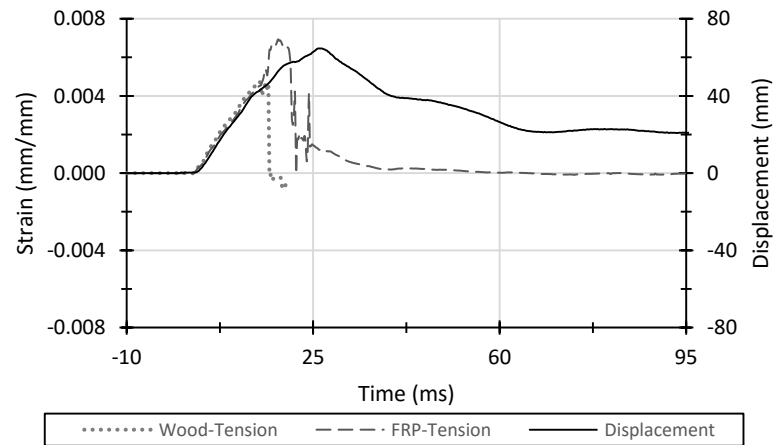
Figure E.17: Test results for R4-3.2-[137]



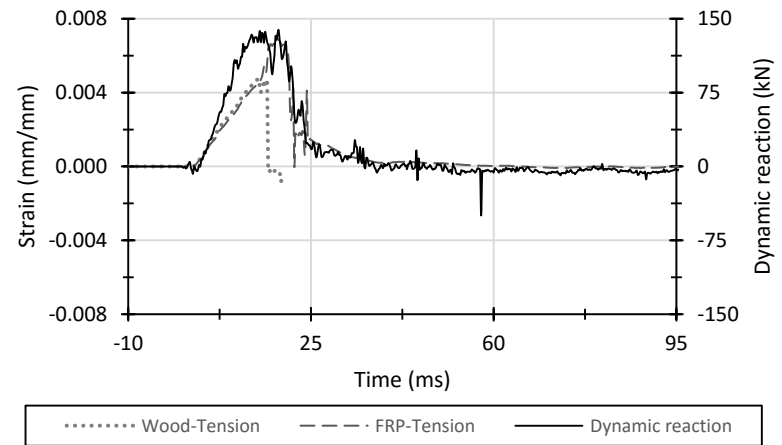
(a) Pressure and impulse



(b) Displacement and reaction



(c) Strain and displacement



(d) Strain and reaction

Figure E.18: Test results for R4-4.1-[137]



(a) R4-3.2 (Δ_{Rf})



(b) R4-3.2

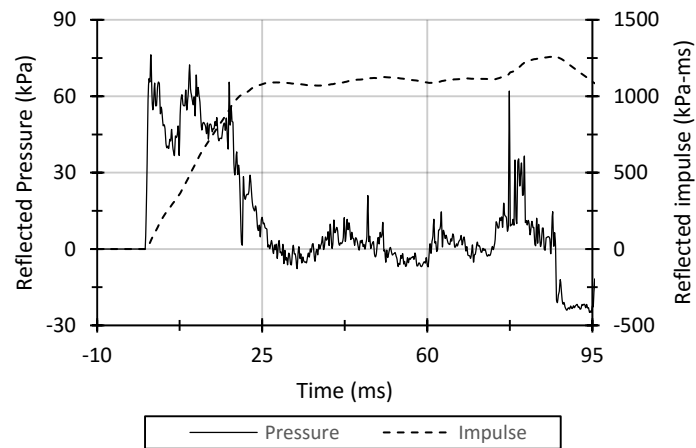


(c) R4-4.1 (Δ_{Rf})

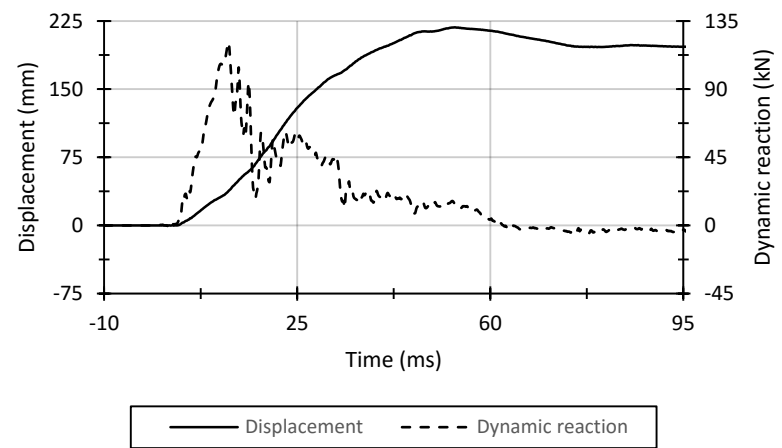


(d) R4-4.2

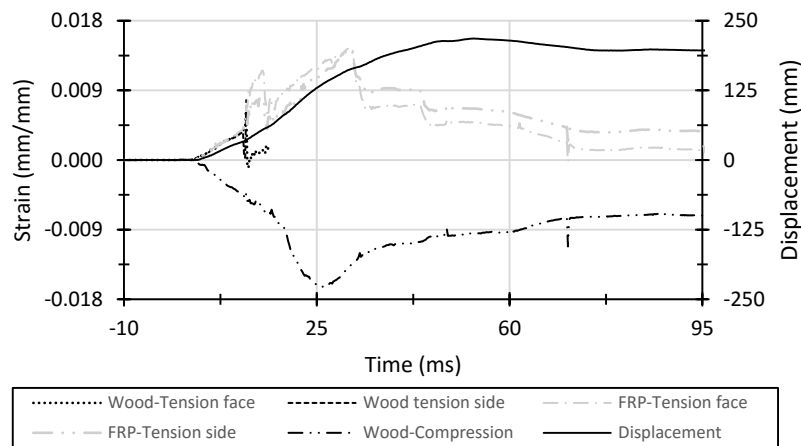
Figure E.19: Dynamic failure modes of Retrofit 4



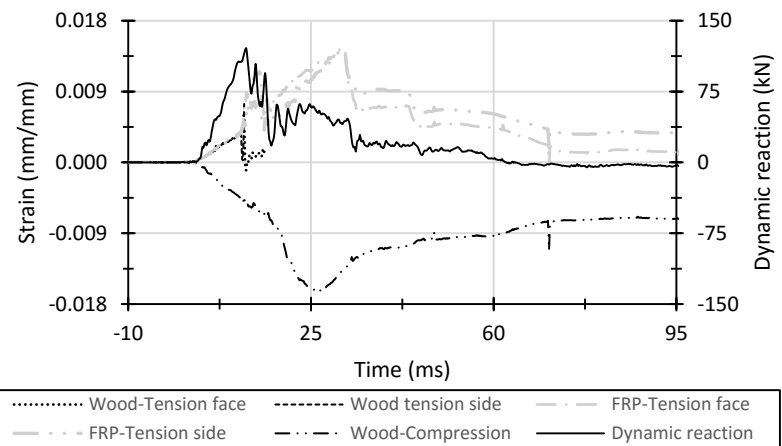
(a) Pressure and impulse



(b) Displacement and reaction

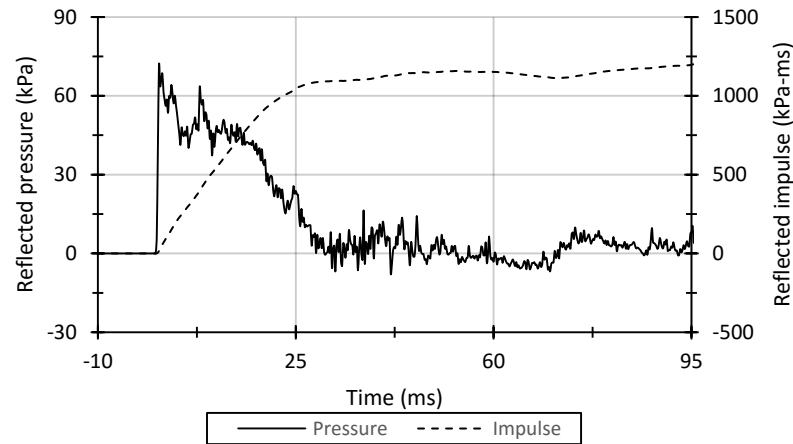


(c) Strain and displacement

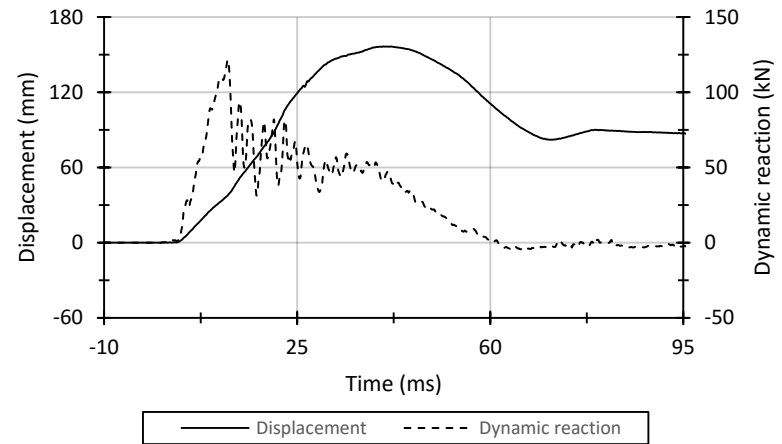


(d) Strain and reaction

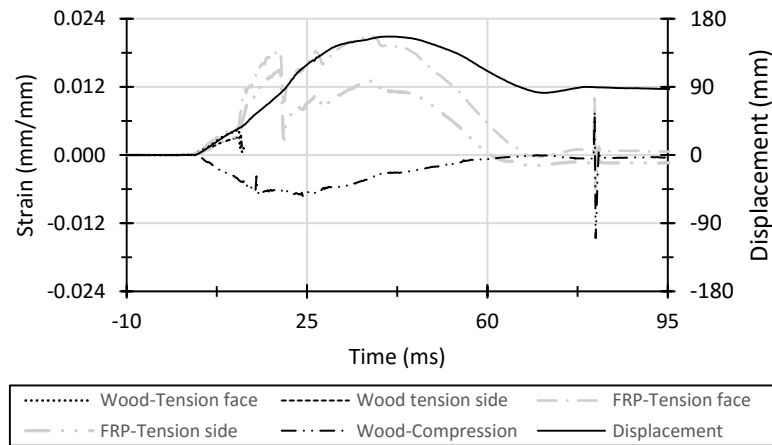
Figure E.20: Test results for R5-A-[137]



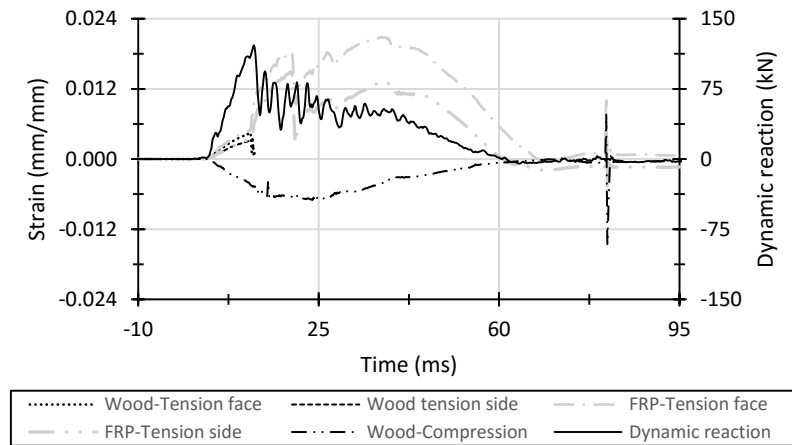
(a) Pressure and impulse



(b) Displacement and reaction



(c) Strain and displacement



(d) Strain and reaction

Figure E.21: Test results for R5-B-[137]



(a) R5-A (Δ_{Rf})



(b) R5-A

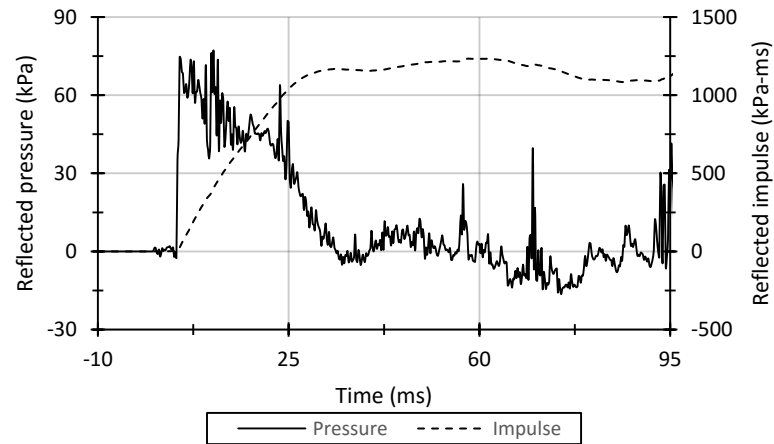


(c) R5-B (Δ_{max})

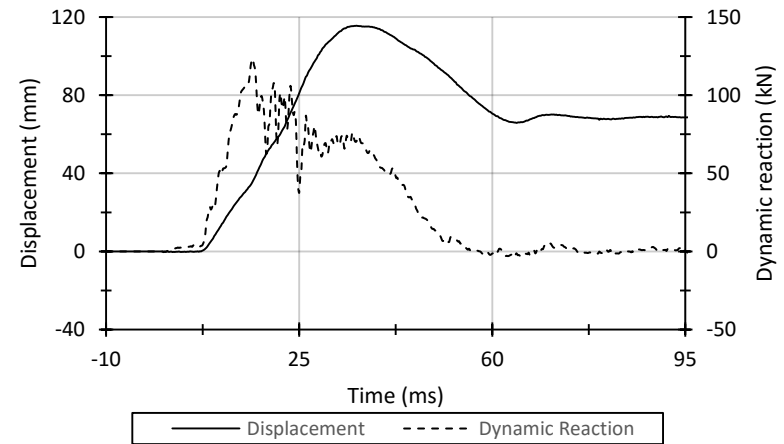


(d) R5-B

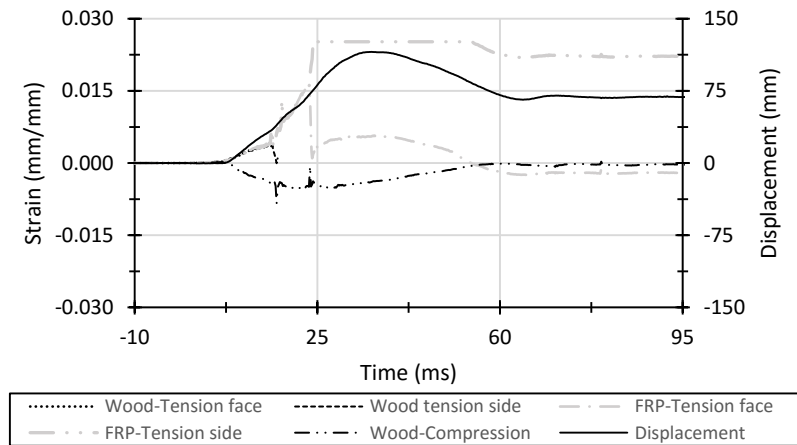
Figure E.22: Dynamic failure modes of Retrofits 5



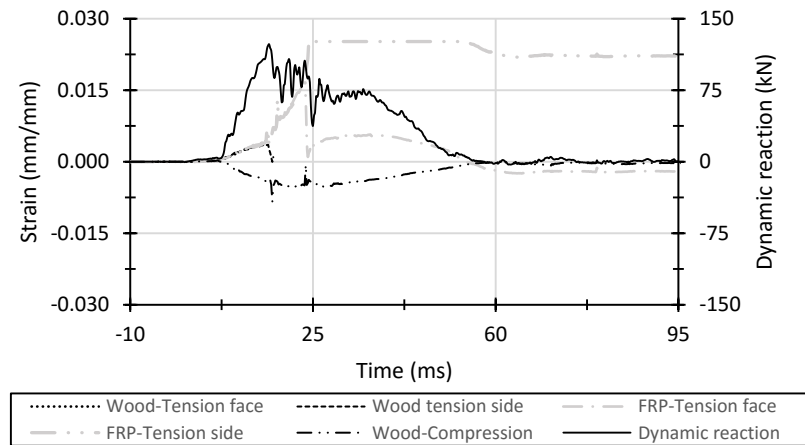
(a) Pressure and impulse



(b) Displacement and reaction

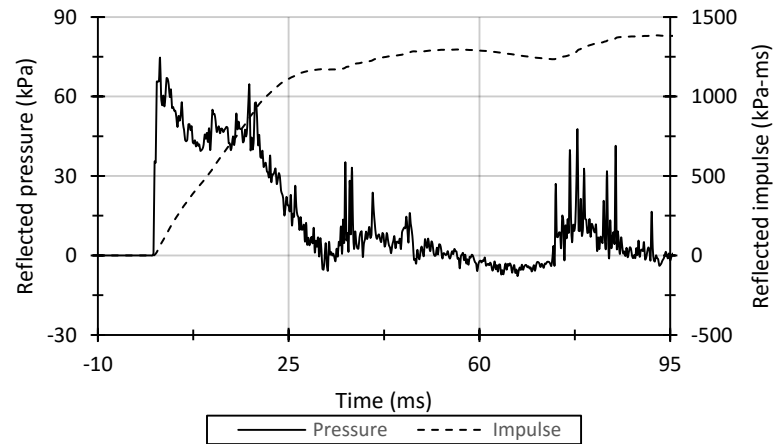


(c) Strain and displacement

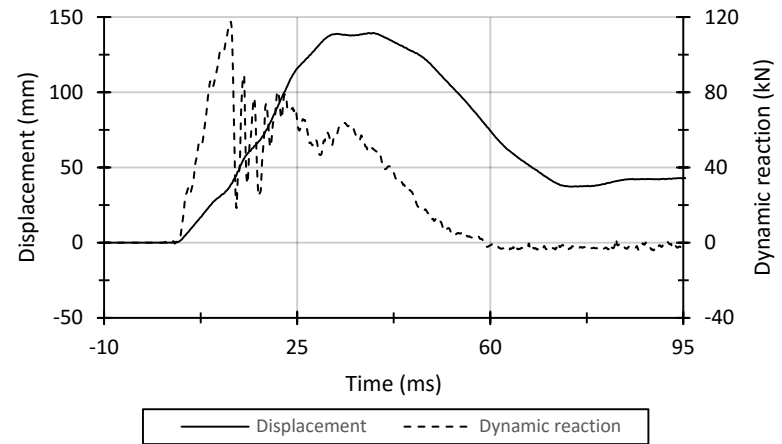


(d) Strain and reaction

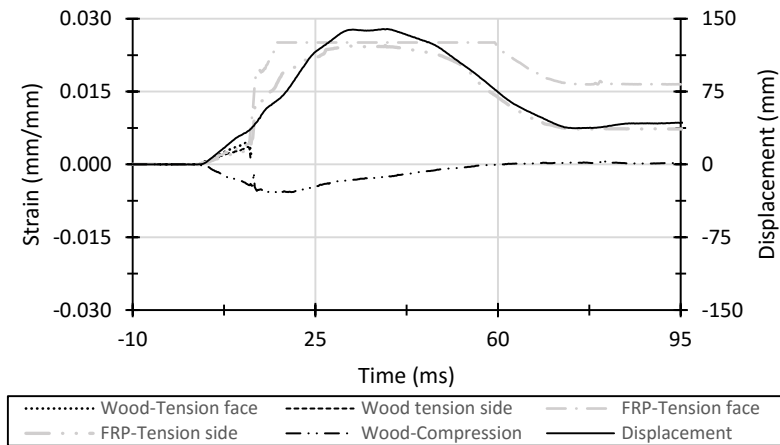
Figure E.23: Test results for R6-A-[137]



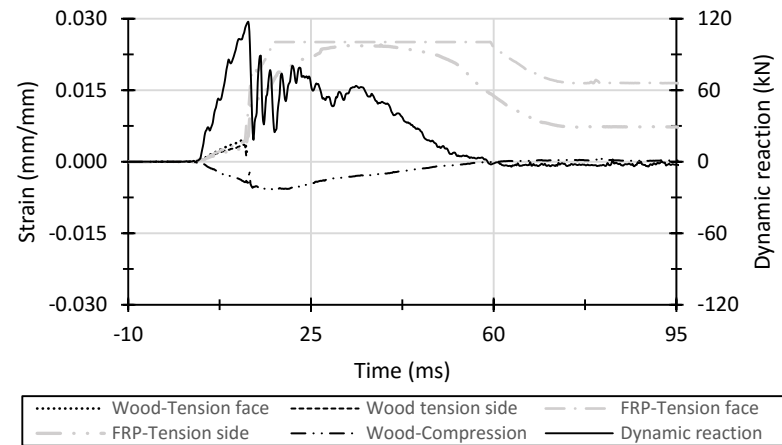
(a) Pressure and impulse



(b) Displacement and reaction



(c) Strain and displacement



(d) Strain and reaction

Figure E.24: Test results for R6-B-[137]



(a) R6-A ($\Delta_{\max}$)



(b) R6-A

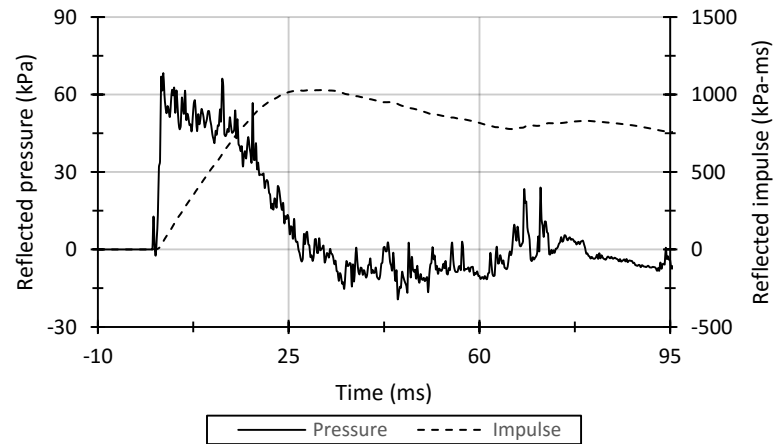


(c) R6-B ($\Delta_{\max}$)

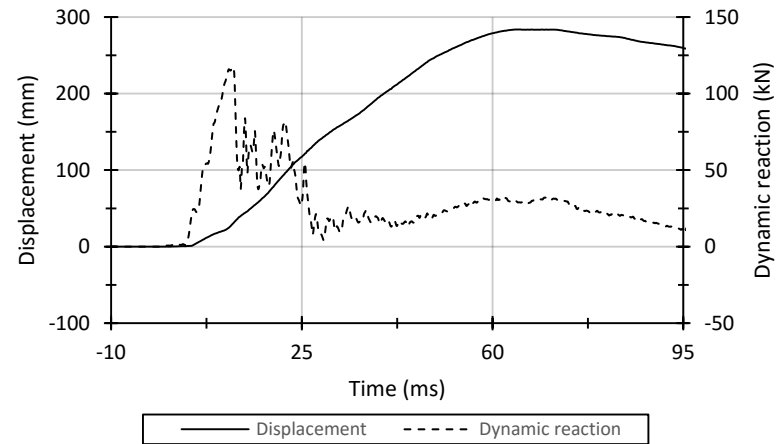


(d) R6-B

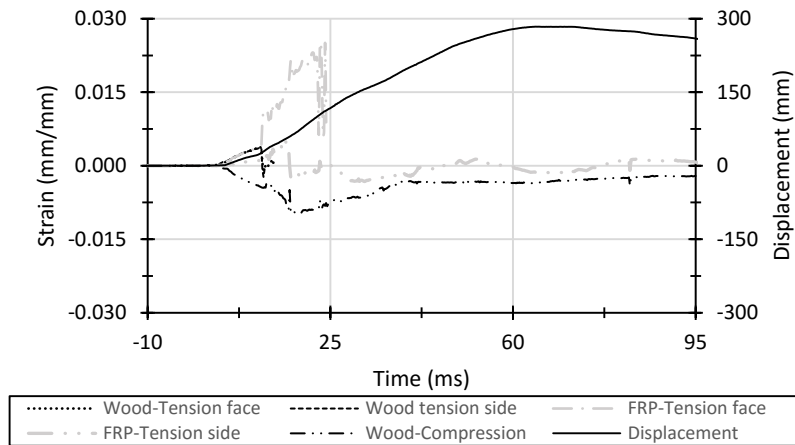
Figure E.25: Dynamic failure modes of Retrofits 6



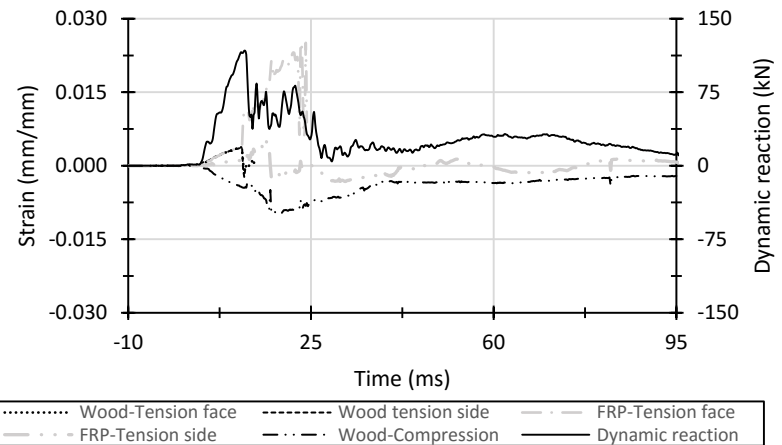
(a) Pressure and impulse



(b) Displacement and reaction

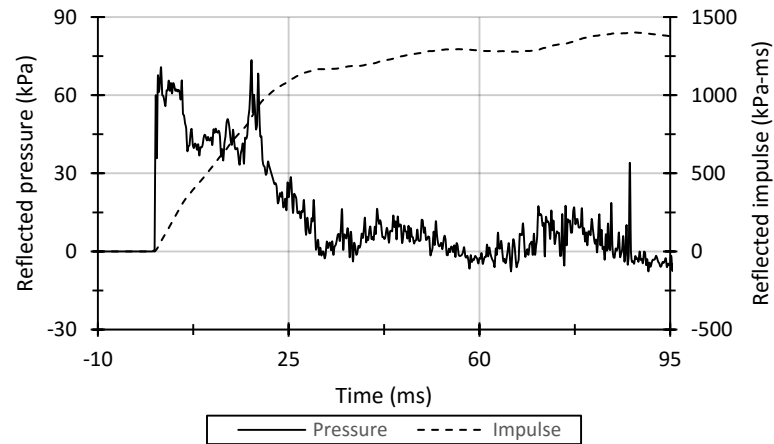


(c) Strain and displacement

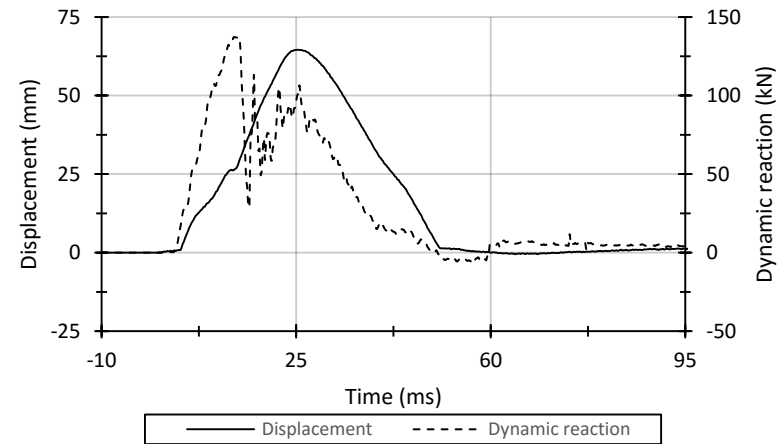


(d) Strain and reaction

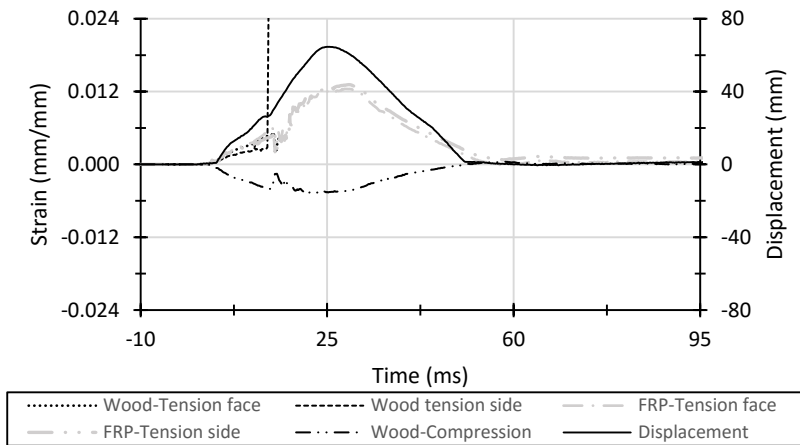
Figure E.26: Test results for R7-A-[137]



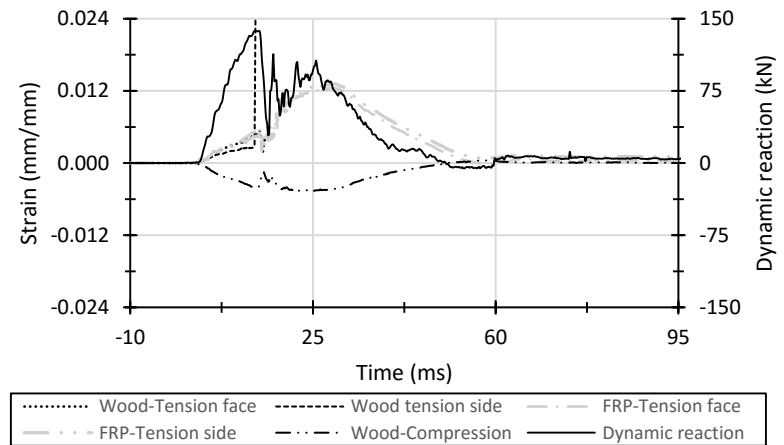
(a) Pressure and impulse



(b) Displacement and reaction



(c) Strain and displacement



(d) Strain and reaction

Figure E.27: Test results for R7-B-[137]



(a) R7-A (Δ_{Rf})



(b) R7-A

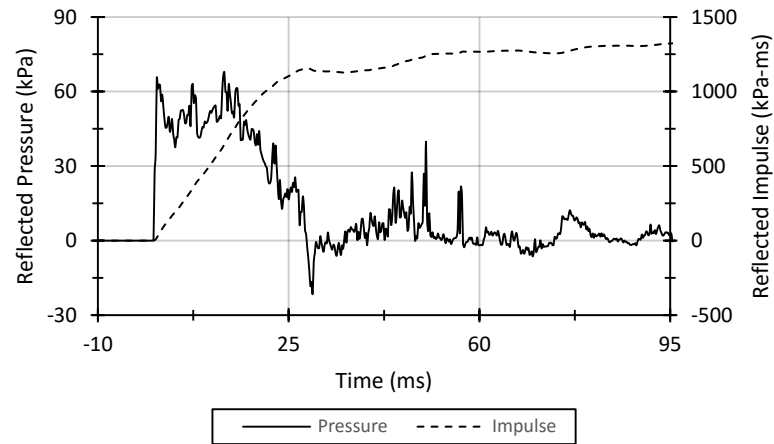


(c) R7-B (Δ_{max})

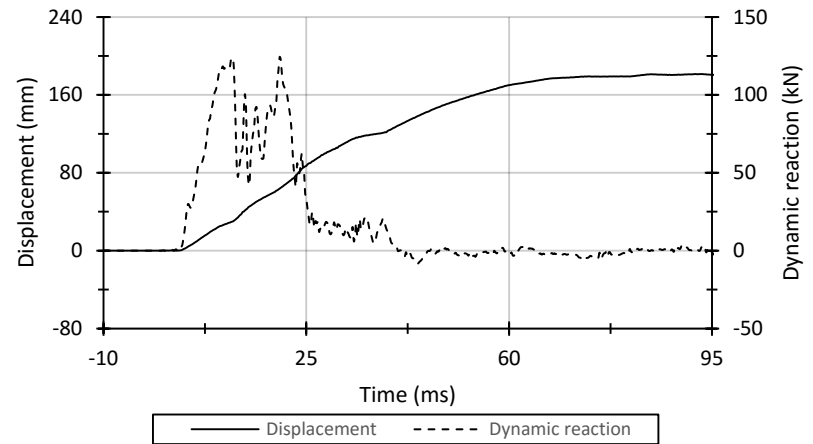


(d) R7-B

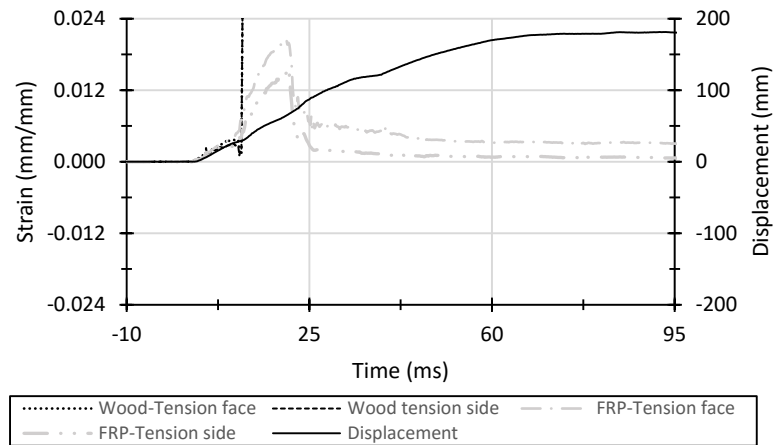
Figure E.28: Dynamic failure modes of Retrofits 7



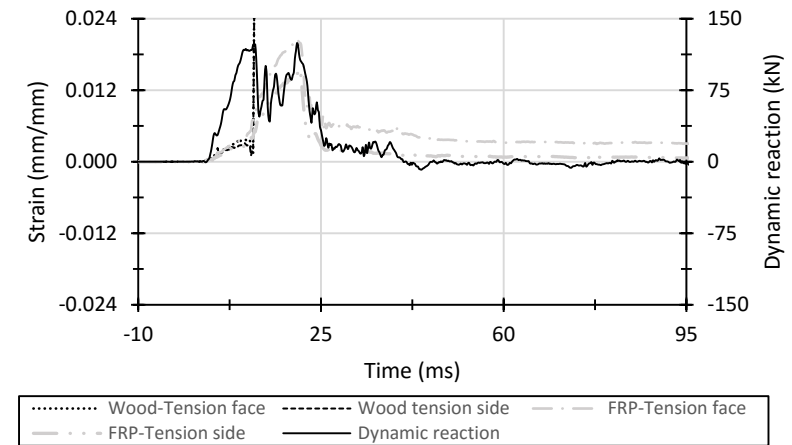
(a) Pressure and impulse



(b) Displacement and reaction

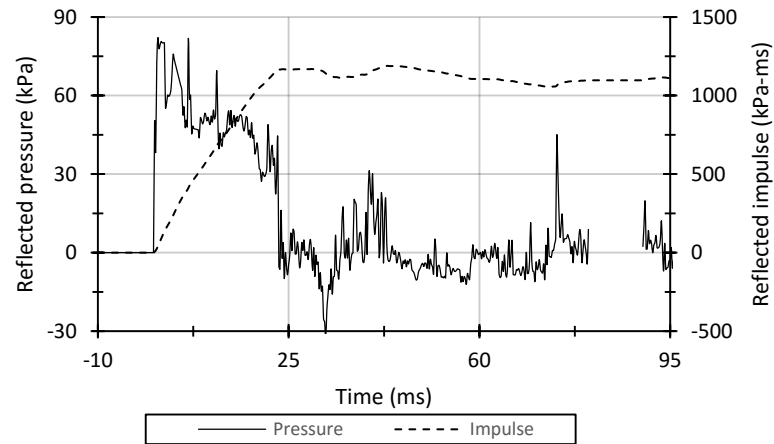


(c) Strain and displacement

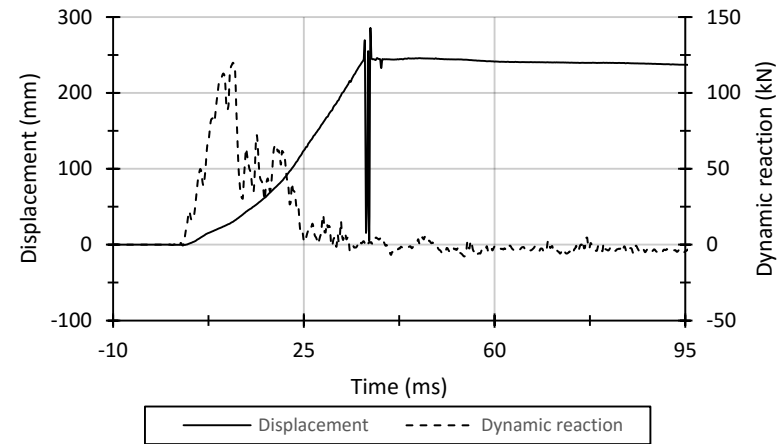


(d) Strain and reaction

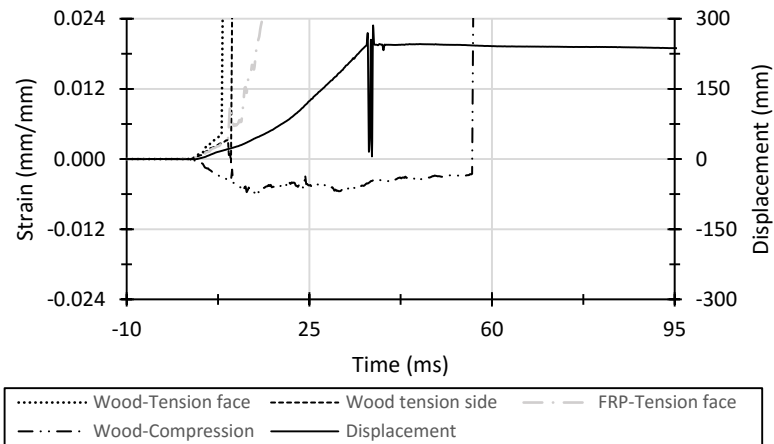
Figure E.29: Test results for R8-A-[137]



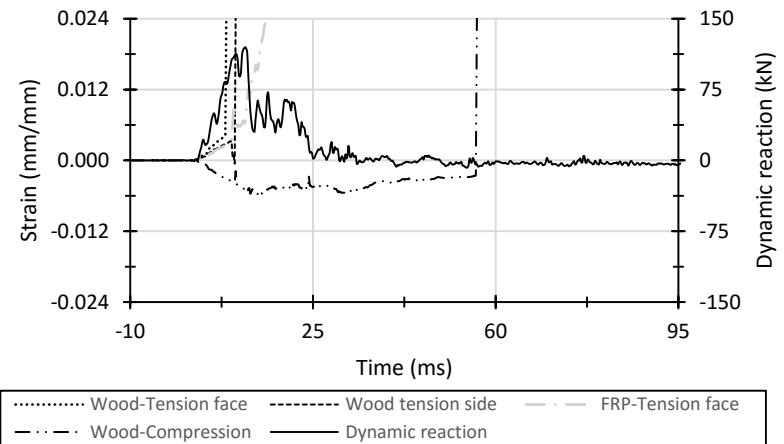
(a) Pressure and impulse



(b) Displacement and reaction



(c) Strain and displacement



(d) Strain and reaction

Figure E.30: Test results for R8-B-[137]



(a) R8-A (Δ_{Rf})



(b) R8-A

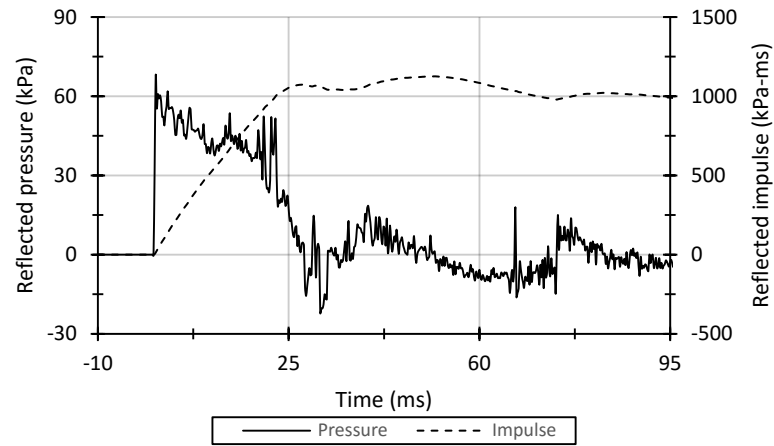


(c) R8-B (Δ_{Rf})

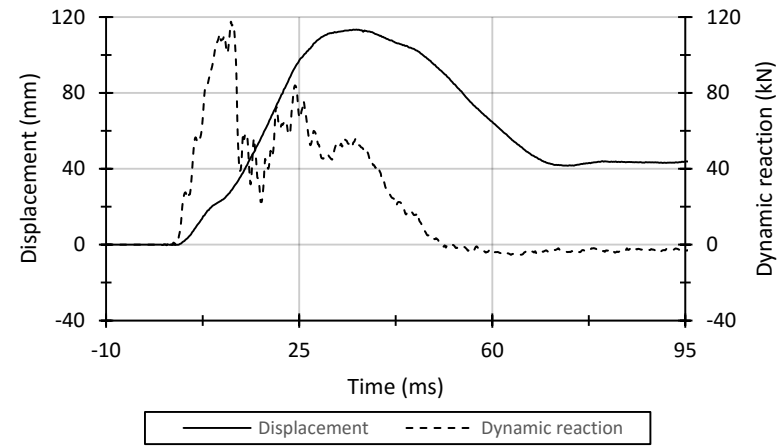


(d) R8-B

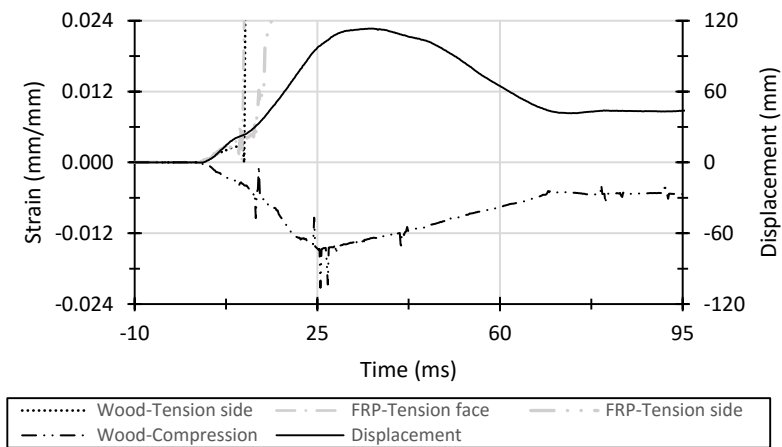
Figure E.31: Dynamic failure modes of Retrofits 8



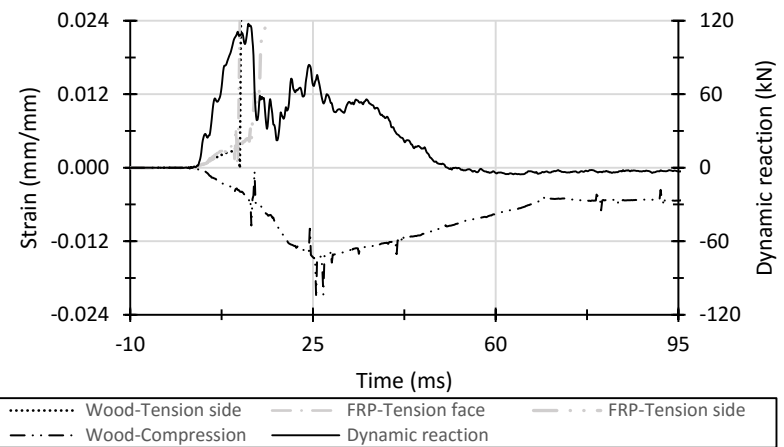
(a) Pressure and impulse



(b) Displacement and reaction

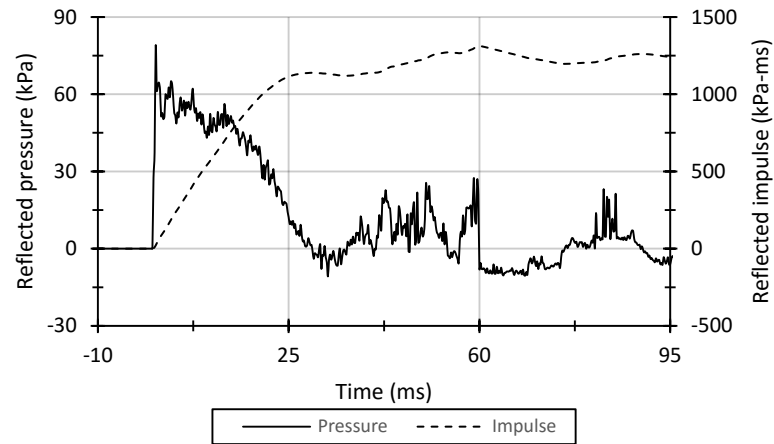


(c) Strain and displacement

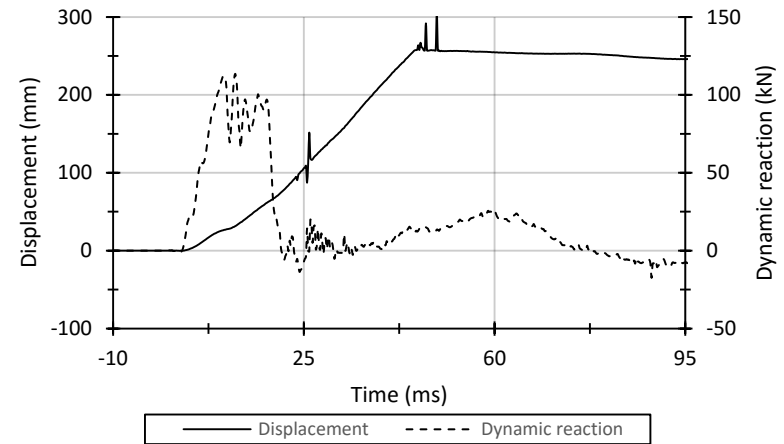


(d) Strain and reaction

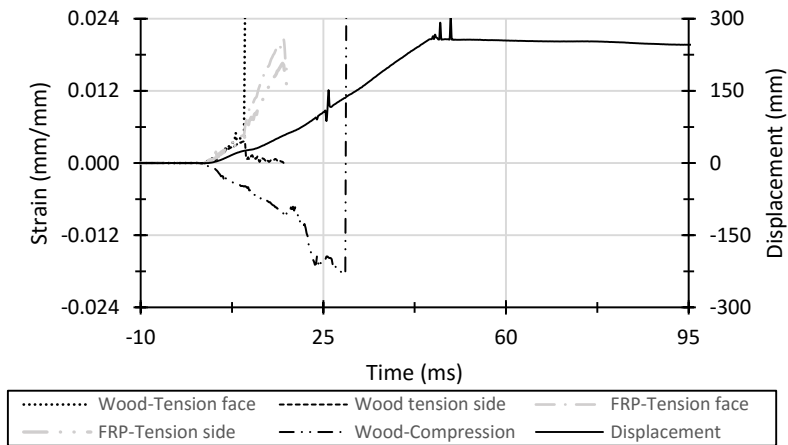
Figure E.32: Test results for R9-A-[137]



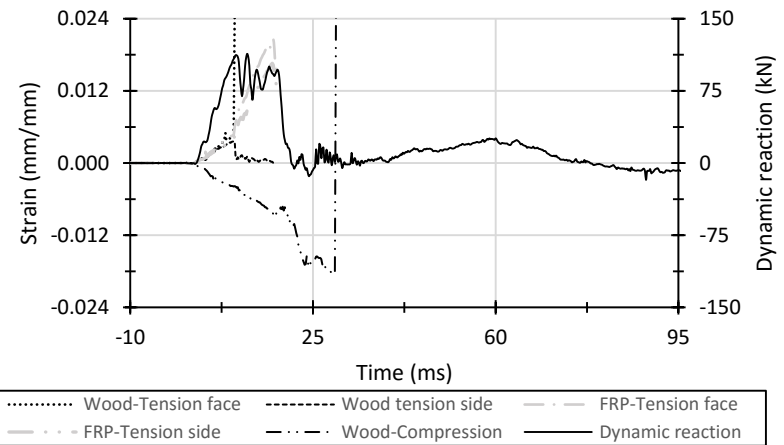
(a) Pressure and impulse



(b) Displacement and reaction



(c) Strain and displacement



(d) Strain and reaction

Figure E.33: Test results for R9-B-[137]



(a) R9-A ($\Delta_{\max}$)



(b) R9-A

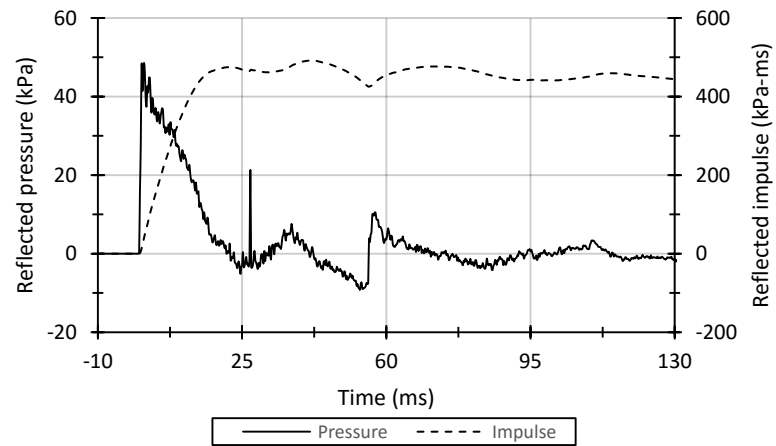


(c) R9-B (Δ_{Rf})

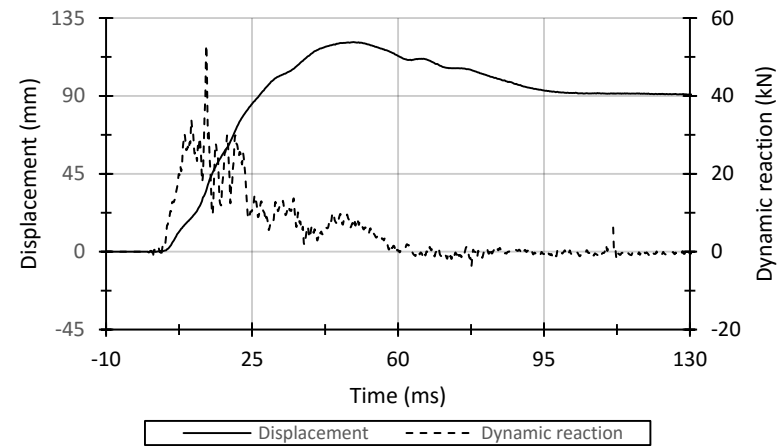


(d) R9-B

Figure E.34: Dynamic failure modes of Retrofits 9

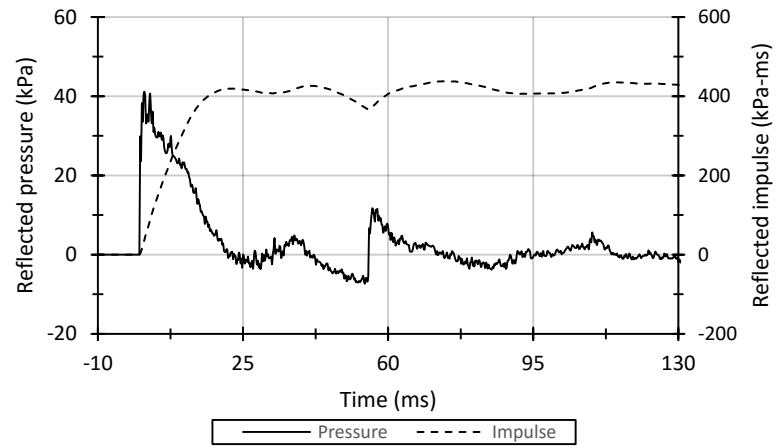


(a) Pressure and impulse

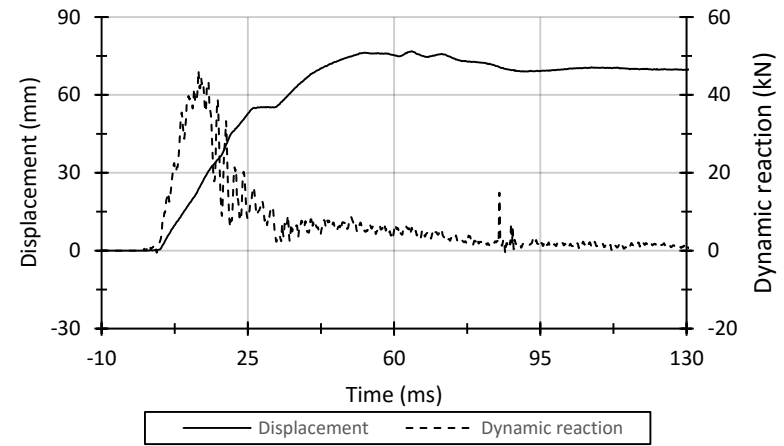


(b) Displacement and reaction

Figure E.35: Test results for D1.1-[137]

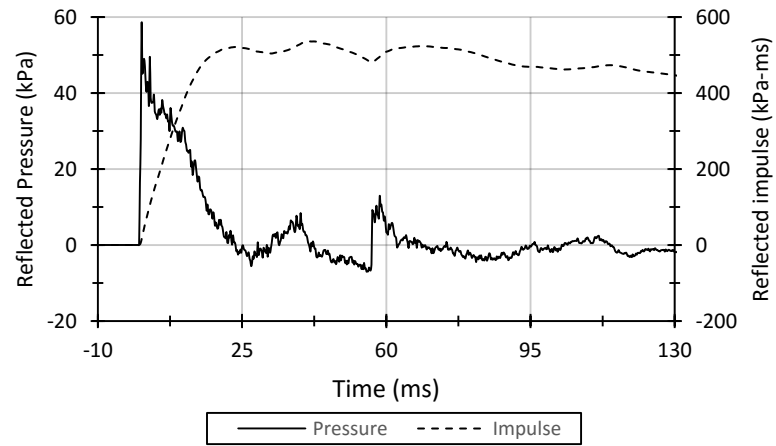


(a) Pressure and impulse

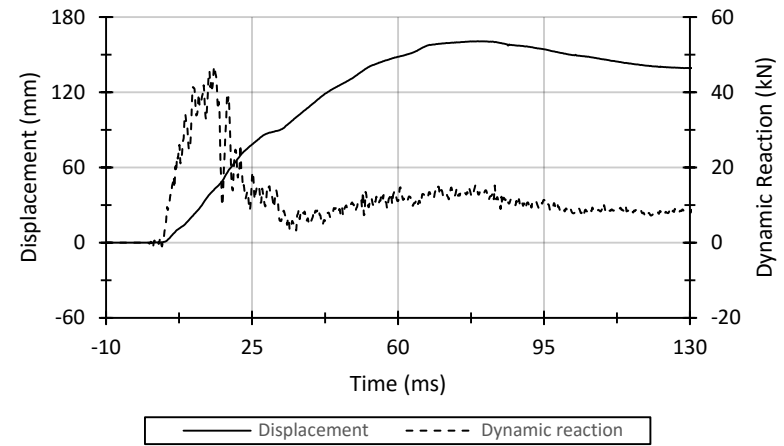


(b) Displacement and reaction

Figure E.36: Test results for D2.1-[137]



(a) Pressure and impulse



(b) Displacement and reaction

Figure E.37: Test results for D3.1-[137]



(a) D1.1



(b) D2.1



(c) D3.1 (Δ_{Rf})



(d) D3.1

Figure E.38: Dynamic failure modes of damaged beams

Appendix F – Derivation of Dynamic Resistance Based on SDOF Analysis

This appendix provides the derivation of the dynamic resistance of a beam when subjected to four-point bending in the elastic range. The system consists of both concentrated (LTD) and distributed (beam specimen) masses. The idealization of the actual dynamic test setup consisting of the LTD and specimen is shown in Figure F.1a.

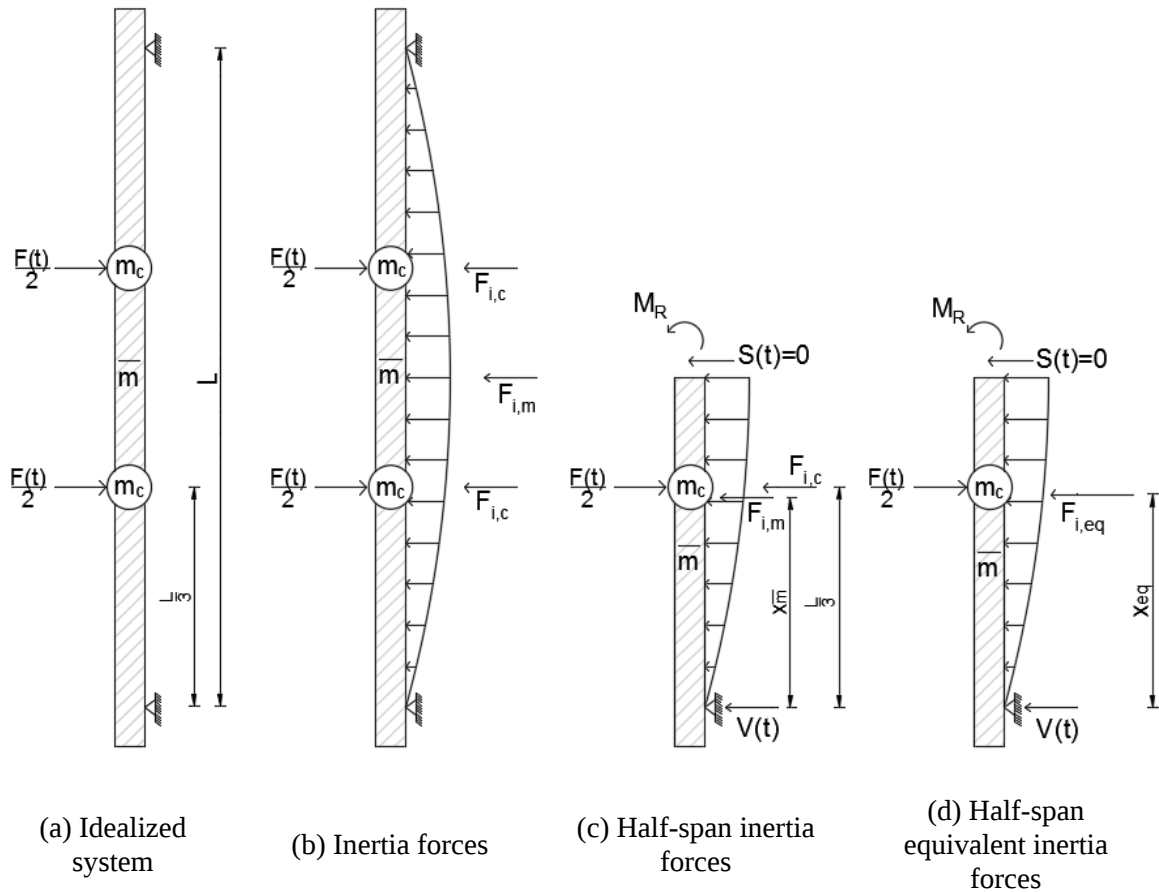


Figure F.1: Idealized system with external and internal forces for the elastic range

When subjected to an acceleration, $\ddot{y}$, inertia forces are generated as shown in Figure F.1b. As a result, the inertia force distribution of the beam follows that of the assumed static deflected shape (Biggs 1964), whereas the inertia forces resulting from the LTD are assumed to act at the third points in a direction opposite to that of the acceleration. The dynamic

resistance of the system as a function of time is developed by taking the sum of moments about where the equivalent inertia force acts, as shown in Figure F.1d.

Equation [F.1] shows the dynamic resistance of the beam as a function of time, $R(t)$, which is obtained by taking the sum of moments about the equivalent inertia force (Fig. F.1d) for the idealized system.

$$\begin{aligned}\sum M_{F_{i-eq}} &= 0 \\ -V(t) * x_{eq} - \frac{F(t)}{2} \left(\frac{L}{3} - x_{eq} \right) + M_R &= 0 \\ \frac{R(t) * L}{6} &= V(t) * x_{eq} + \frac{F(t)}{2} \left(\frac{L}{3} - x_{eq} \right) \\ R(t) &= \left\{ \frac{6}{L} \right\} \left\{ V(t)x_{eq} + 0.5 \left[\frac{L}{3} - x_{eq} \right] F(t) \right\} \quad [F.1]\end{aligned}$$

where $V(t)$ is the measured experimental dynamic reactions as a function of time, $F(t)$ is the applied force at the load point as a function of time, M_R is the dynamic bending capacity of the beam, L is the span, and x_i is the distance of the equivalent inertia force from the support.

In the derivation of Equation [F.1], M_R is expanded to include the resistance of the beam at mid-span. The location of the equivalent inertia force is directly linked to the ratio of the distributed mass, $\bar{m}$, to the concentrated mass, m_c . The location of the resultant inertia force of the beam for the lower half of the system (Fig. F.1c), $x_{\bar{m}}$, can be determined from the assumed deflected shape resulting in the simplified system shown in Figure F.1d. The deflected shape, $\varphi(x)$, for concentrated loads at third points can be obtained from Biggs (1964) and is shown in Equations [F.2] and [F.3] for the elastic range.

$$\text{For } x \leq \frac{L}{3}: \quad \varphi(x) = \frac{36x}{23L^3} (2L^2 - 3x^2) \quad [F.2]$$

$$\text{For } \frac{L}{3} < x < \frac{2L}{3}: \quad \varphi(x) = \frac{36}{23L^2} \left(3Lx - 3x^2 - \frac{L^2}{9} \right) \quad [F.3]$$

where x is the location along the beam length and L is the span.

The acceleration at any point along the span, $\ddot{y}(x)$, can be determined using Equation [F.4] assuming once again that the inertia forces' distribution is the same as the deflected shape (Biggs 1964).

$$\ddot{y}(x) = A\varphi(x) \quad [\text{F.4}]$$

where A is a constant and $\varphi(x)$ is the deflected shape along the beam span.

For the elastic range, $x_{\bar{m}}$ is shown in Equation [F.5] and is obtained through the following steps:

$$\begin{aligned} x_{\bar{m}} &= \frac{\int_0^{L/2} x F_{i,\bar{m}}(x) dx}{\int_0^{L/2} F_{i,\bar{m}}(x) dx} \\ x_{\bar{m}} &= \frac{\int_0^{L/2} x [\bar{m} \ddot{y}(x)] dx}{\int_0^{L/2} [\bar{m} \ddot{y}(x)] dx} \\ x_{\bar{m}} &= \frac{\int_0^{L/2} x [\bar{m} A \varphi(x)] dx}{\int_0^{L/2} [\bar{m} A \varphi(x)] dx} \\ x_{\bar{m}} &= \frac{\bar{m} A \int_0^{L/2} x \varphi(x) dx}{\bar{m} A \int_0^{L/2} \varphi(x) dx} \\ x_{\bar{m}} &= \frac{\int_0^{L/2} x \varphi(x) dx}{\int_0^{L/2} \varphi(x) dx} \\ x_{\bar{m}} &= \frac{\int_0^{L/3} x \left[\frac{36x}{23L^3} (2Lx - 3x^2) \right] dx + \int_{L/3}^{L/2} x \left[\frac{36}{23L^2} \left(3Lx - 3x^3 - \frac{L^2}{9} \right) \right] dx}{\int_0^{L/3} \left[\frac{36x}{23L^3} (2Lx - 3x^2) \right] dx + \int_{L/3}^{L/2} \left[\frac{36}{23L^2} \left(3Lx - 3x^3 - \frac{L^2}{9} \right) \right] dx} \\ x_{\bar{m}} &= \frac{\frac{4L^2}{115} + \frac{221L^2}{3312}}{\frac{11L}{69} + \frac{11L}{69}} \\ x_{\bar{m}} &= \frac{305}{958} L \quad [\text{F.5}] \end{aligned}$$

The distance from the support of the equivalent inertia force, x_{eq} , is bounded by the two extremes defined by the absence of distributed mass and concentrated mass as shown in Equation [F.6].

$$x_{\bar{m}} \leq x_{eq} \leq \frac{L}{3} \quad [\text{F.6}]$$

The inertia forces for the beam and the concentrated mass are obtained using Newton's second law as shown in Equations [F.7] and [F.8], respectively.

$$\begin{aligned} F_{i,\bar{m}} &= \int_0^{L/2} \bar{m}\ddot{y}(x)dx \\ F_{i,\bar{m}} &= \int_0^{L/2} \bar{m}A\varphi(x)dx \\ F_{i,\bar{m}} &= \bar{m}A \left[\int_0^{L/3} \varphi(x)dx + \int_{L/3}^{L/2} \varphi(x)dx \right] \\ F_{i,\bar{m}} &= \left(\frac{22\bar{m}L}{69} \right) A \end{aligned} \quad [\text{F.7}]$$

$$\begin{aligned} F_{i,c} &= m_c \ddot{y} \left(\frac{L}{3} \right) \\ F_{i,c} &= m_c A \varphi \left(\frac{L}{3} \right) \\ F_{i,c} &= \left(\frac{87m_c}{100} \right) A \end{aligned} \quad [\text{F.8}]$$

By taking the sum of moments about the support of the system with the two resulting inertia forces ($F_{i,\bar{m}}$ and $F_{i,c}$) and equating it to the sum of moments about the support of the system with an equivalent inertia force (F_{i-eq}), it is possible to solve for x_{eq} . The equivalent inertia force for half the beam is shown in Equation [F.9] followed by the derivation for x_{eq} as presented in Equation [F.10].

$$F_{i,eq} = F_{i,\bar{m}} + F_{i,c} = \left(\frac{22\bar{m}L}{69} + \frac{87m_c}{100} \right) A \quad [\text{F.9}]$$

$$\begin{aligned}
\sum M_A &= \sum M_{A-eq} \\
F_{i,\bar{m}} \left(\frac{305L}{958} \right) + F_{i,c} \left(\frac{L}{3} \right) - \left(\frac{F}{2} \right) \left(\frac{L}{3} \right) + M_R &= F_{i,eq}(x_{eq}) - \left(\frac{F}{2} \right) \left(\frac{L}{3} \right) + M_R \\
\frac{22\bar{m}L}{69} \left(\frac{305L}{958} \right) A + \frac{87m_c}{100} \left(\frac{L}{3} \right) A &= \left(\frac{22\bar{m}L}{69} + \frac{87m_c}{100} \right) A(x_{eq}) \\
x_{eq} &= \frac{0.102\bar{m}L^2 + 0.290m_cL}{0.319\bar{m}L + 0.870m_c} \quad [F.10]
\end{aligned}$$

where $\bar{m}$ is a distributed mass (i.e. the glulam specimen's distributed mass), L is the span length, m_c is the concentrated load (i.e. half of the LTD mass), A is a constant, F is the applied force (i.e. reflected pressure multiplied by effective loaded area of LTD), M_R is the dynamic bending moment at mid-span.

Appendix G – Process Details of Moment-Curvature and Resistance Curves Analysis

This appendix presents the flow charts for the approach employed in developed program to derive the moment-curvature relationships and resistance curves.

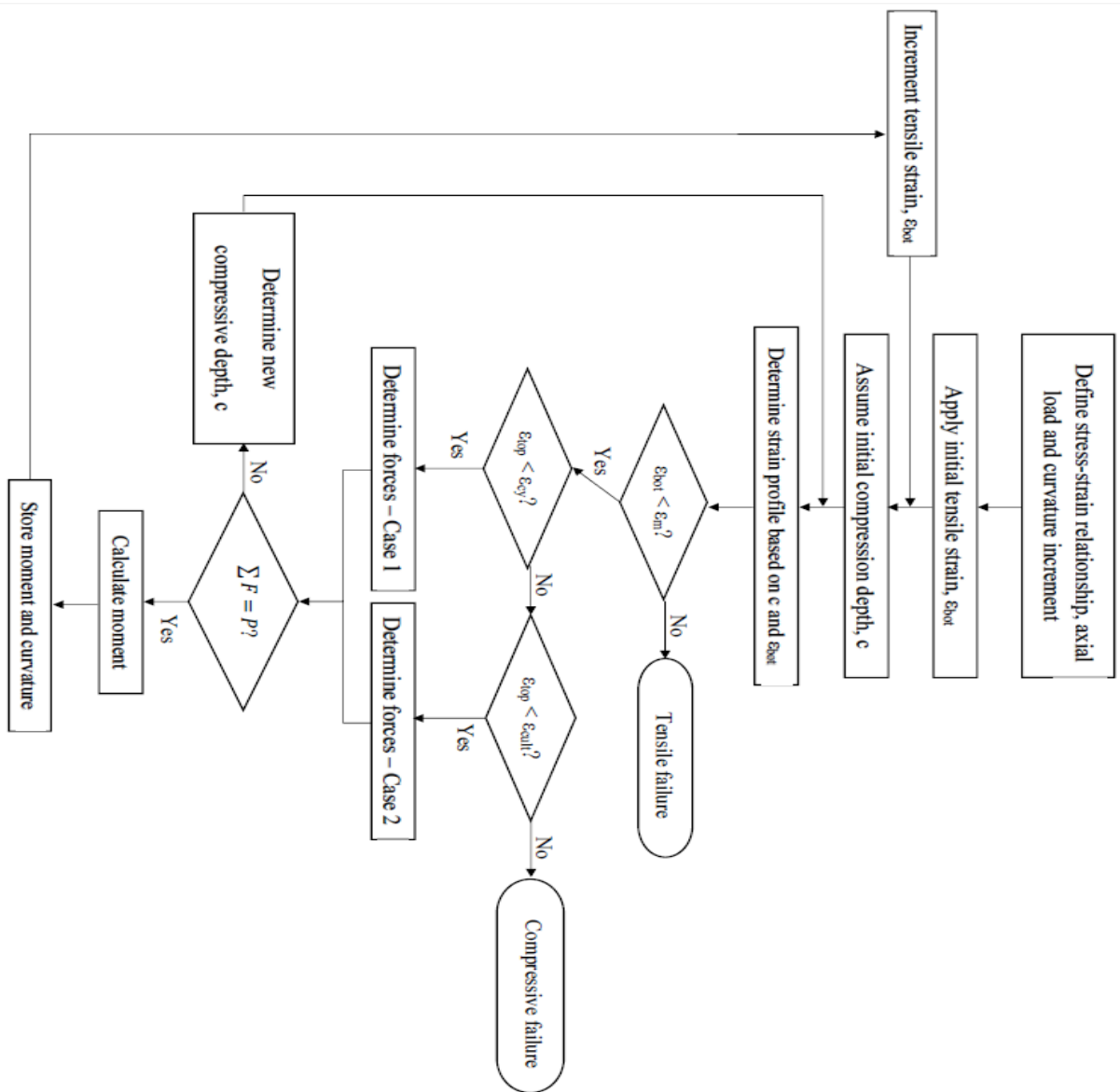


Figure G.1: Flowchart for moment-curvature process

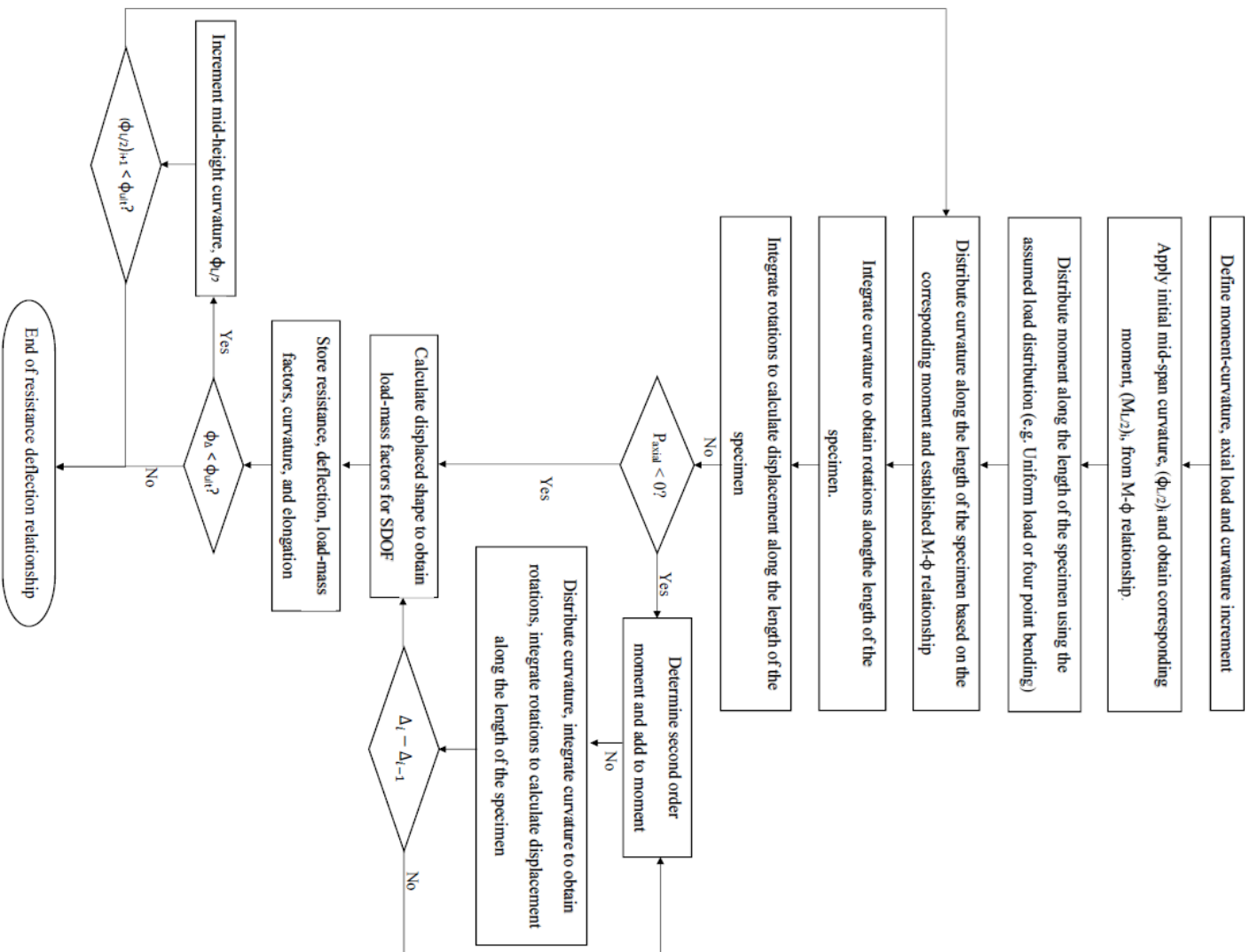


Figure H.2: Flowchart for resistance curve process

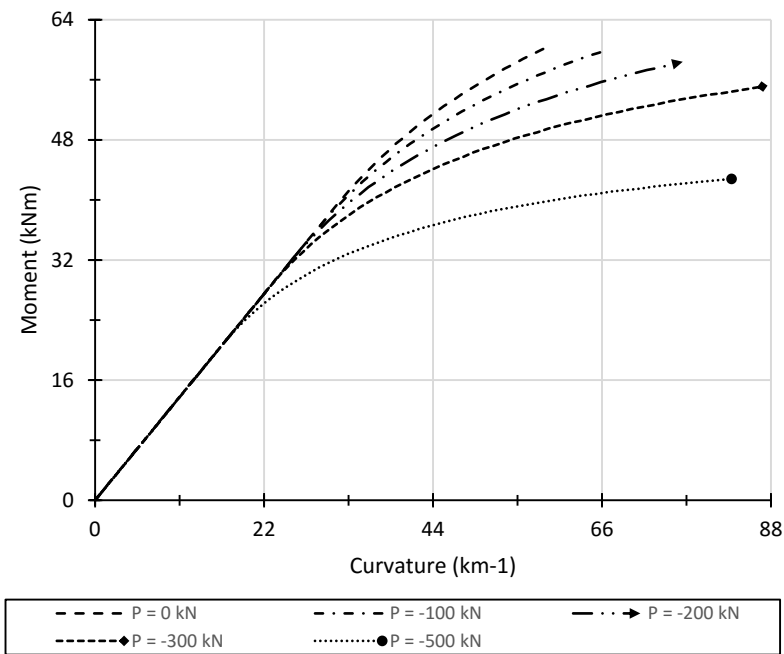
Appendix H – Effects of Material Properties on the Column Behaviour as Established by the Moment-Curvature Analysis

In order to investigate the effect of different ratios of tension and compression moduli on the behaviour of columns a hypothetical case was investigated and the results are presented in this appendix. The selected reference column has the following material properties: $\varepsilon_{tf} = 0.0045$ mm/mm, $E_{wc} = 10,000$ MPa, $f_{cy} = 35$ MPa, and $\varepsilon_{cult} = 0.014$ mm/mm.

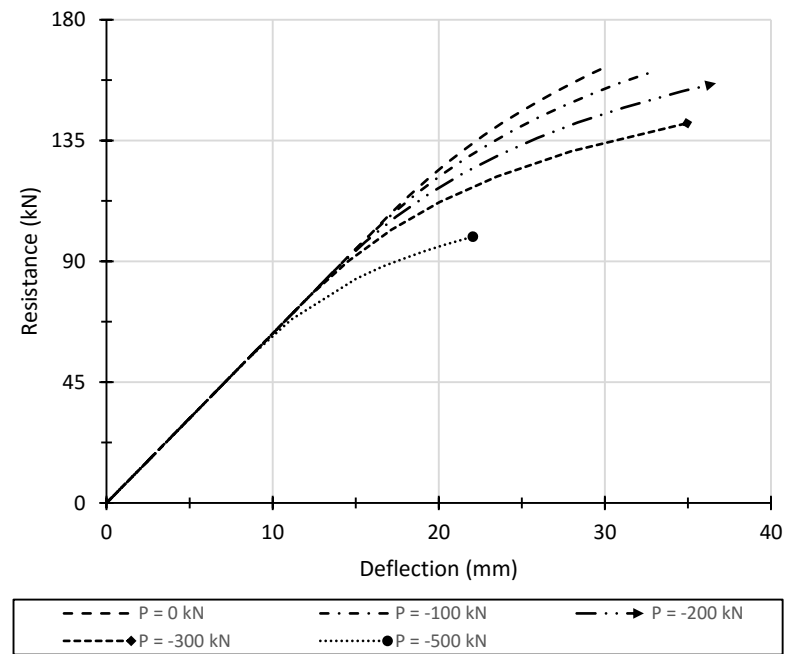
Figures H.1 to H.6 present the results from the sensitivity analysis for the ratios of tension to compression MOEs, respectively. The analyses were conducted for the B-[137] cross-section with a span length of 2,235 mm:

- a) Case 1 - $E_{wt} = E_{wc}$, $m = 0.01$;
- b) Case 2 - $E_{wt} = E_{wc}$, $m = 0.2$;
- c) Case 3 - $E_{wt} = 2E_{wc}$, $m = 0.01$;
- d) Case 4 - $E_{wt} = 2E_{wc}$, $m = 0.2$;
- e) Case 5 - $E_{wt} = 0.5E_{wc}$, $m = 0.01$;
- f) $E_{wt} = 0.5E_{wc}$, $m = 0.2$.

In general, it can be observed for all six cases that increasing the level of axial load results in a decrease in maximum moment and resistance which consistent with observations by Buchanan (1986) for the case where $E_{wt} = E_{wc}$. It can be observed that when $E_{wt} = E_{wc}$ there is no increase in initial stiffness with increasing levels of axial load. For cases where the tension MOE is either larger ($E_{wt} = 2E_{wc}$) or smaller ($E_{wt} = 0.5E_{wc}$) than the compression MOE, an initial stiffness decrease or increase, respectively, is observed with increasing levels of axial load. The results also indicate that at high axial load the flexural response is dominated by the P- Δ effects. Although documented here and corroborated by the test results, the sensitivity of the material properties, especially the effect of the tension and compression stiffness require further analysis.

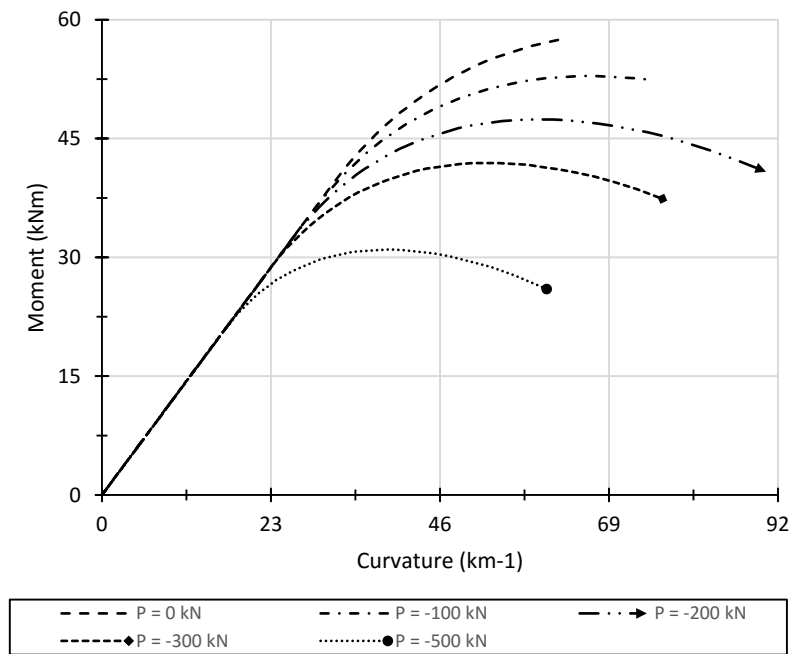


(a) Moment-curvature

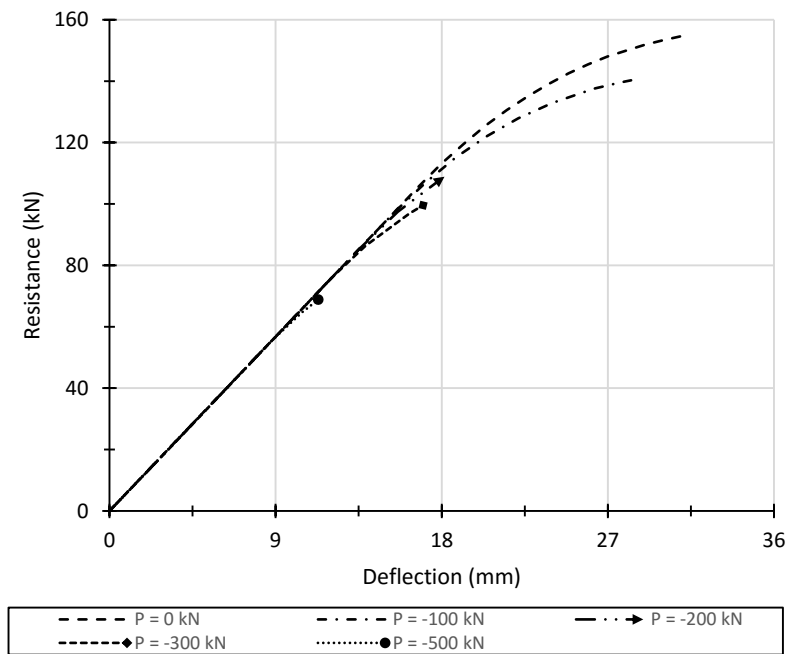


(b) Resistance curve

Figure H.1: Case 1 – $E_{wt} = E_{wc}$; $m = 0.01$

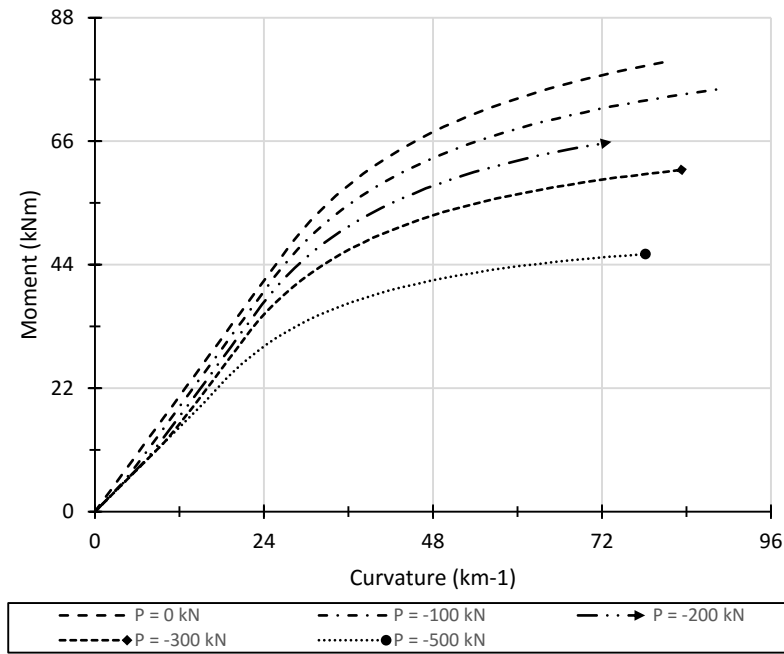


(a) Moment-curvature

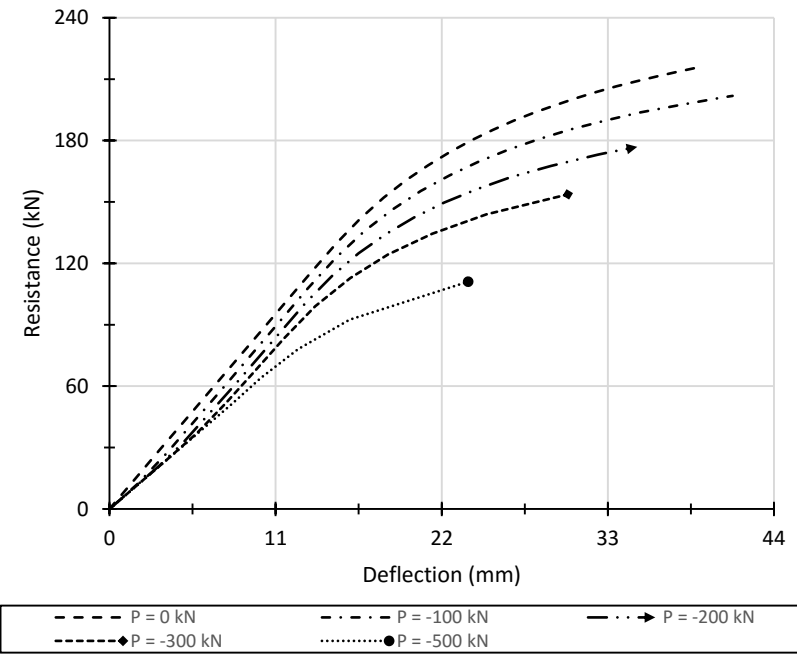


(b) Resistance curve

Figure H.2: Case 2 – $E_{wt} = E_{wc}$; $m = 0.2$

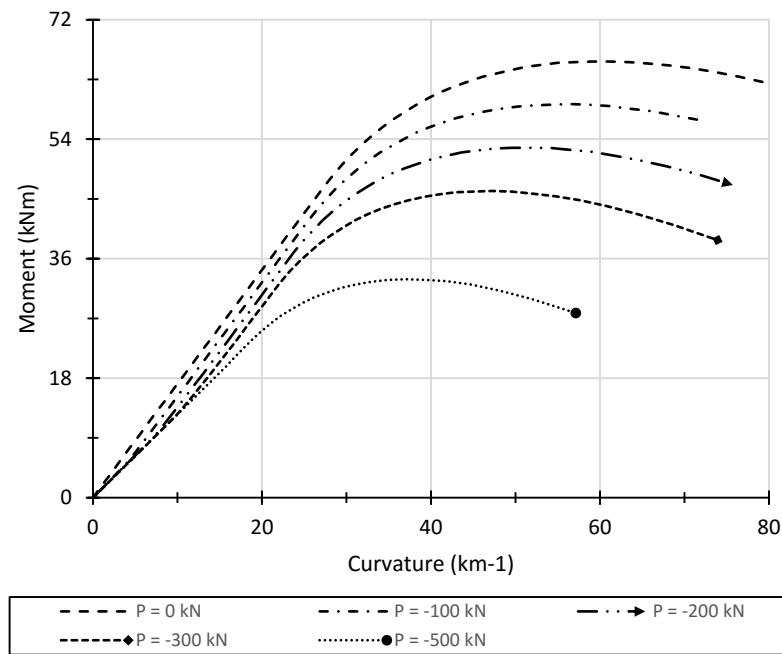


(a) Moment-curvature

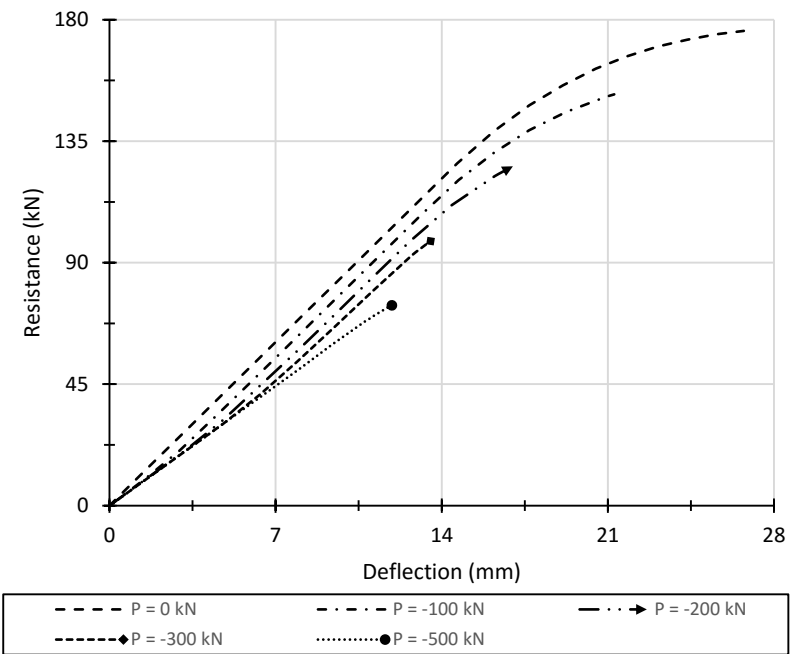


(b) Resistance curve

Figure H.3: Case 3 – $E_{wt} = 2E_{wc}$; $m = 0.01$

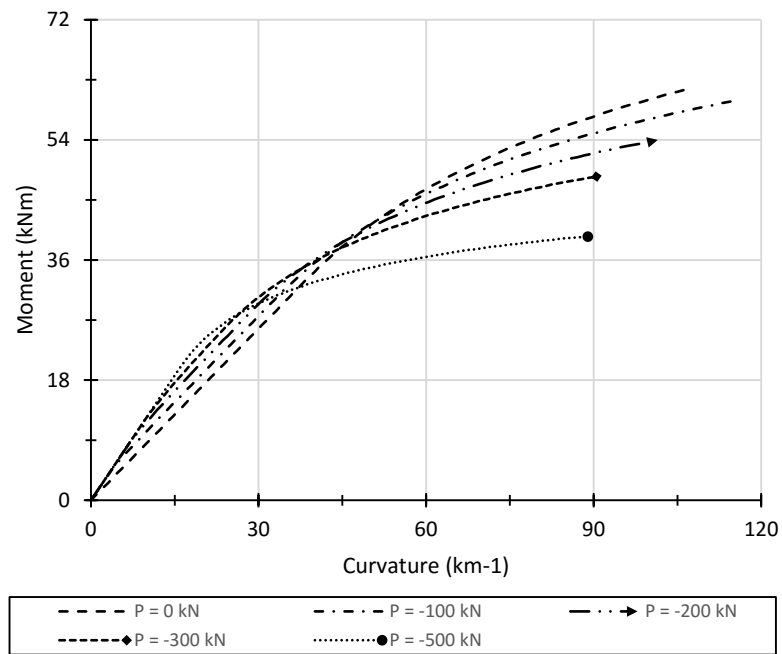


(a) Moment-curvature

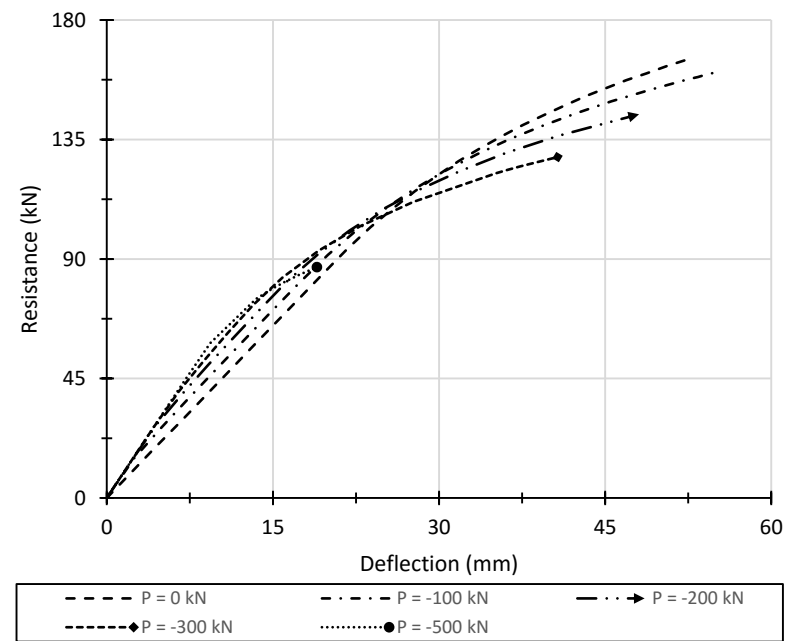


(b) Resistance curve

Figure H.4: Case 4 – $E_{wt} = 2E_{wc}$; $m = 0.2$

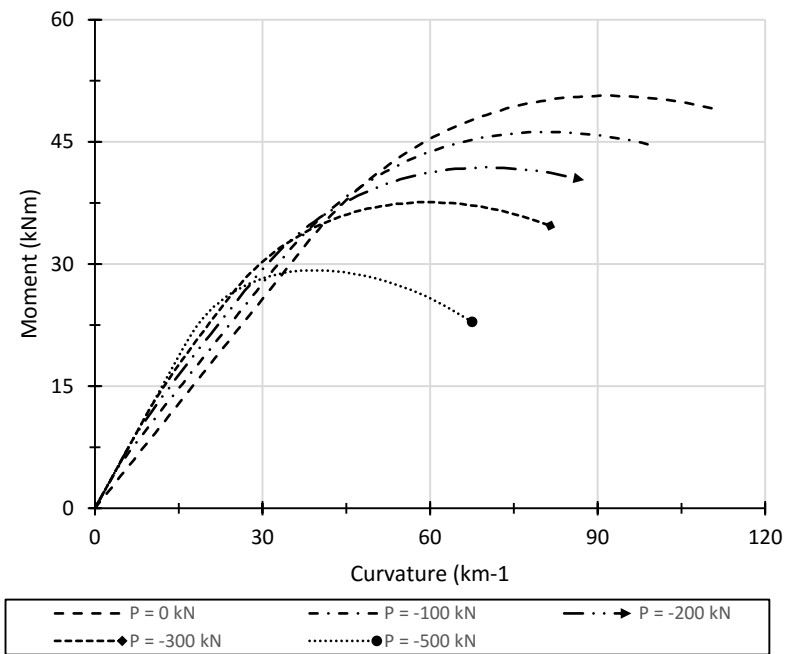


(a) Moment-curvature

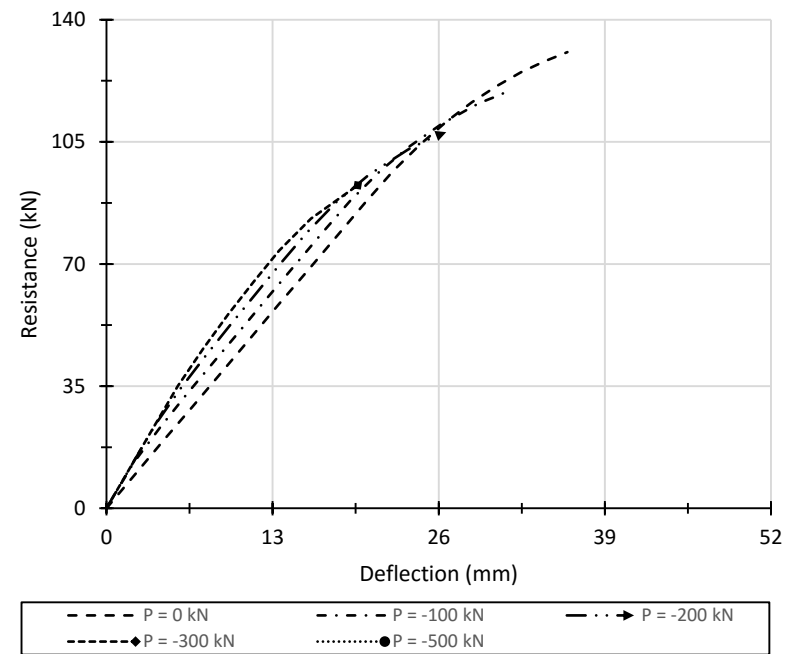


(b) Resistance curve

Figure H.5: Case 5 – $E_{wt} = 0.5E_{wc}$; $m = 0.01$



(a) Moment-curvature



(b) Resistance curve

Figure H.6: Case 6 – $E_{wt} = 0.5E_{wc}$; $m = 0.2$