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Monotonic Response of RC Exterior Beam-Column Joints Reinforced with Filler-Modules and FRP Composite Wraps/Gussets

Praveen Kumar Reddy Majjigapu
pmajjiga@mix.wvu.edu

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Monotonic Response of RC Exterior Beam-Column Joints Reinforced with Filler-Modules and FRP Composite Wraps/Gussets

Praveen Kumar Reddy Majjigapu

**Dissertation submitted
to the Benjamin M. Statler College of Engineering and Mineral Resources
at West Virginia University**

in partial fulfillment of the requirements for the degree of

**Doctor of Philosophy in
Civil and Environmental Engineering**

**Hota V.S. GangaRao, Ph.D., P.E., F.ASCE, F.SEI, Chair
Udaya B. Halabe, Ph.D., P.E., F.ASCE, F.SEI, F.ASNT
P.V. Vijay, Ph.D., P.E.
Ruifeng R. Liang, Ph.D.
Karthik N. Ramanathan, Ph.D.**

Wadsworth Department of Civil and Environmental Engineering

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2020**

Keywords: RC Exterior Beam-Column Joints; Filler-Modules; FRP Wraps; FRP Gussets;
Joint Shear; Joint Principal Strain; Strength; Stiffness; Ductility; Energy Dissipation

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ABSTRACT

Monotonic Response of RC Exterior Beam-Column Joints Reinforced with Filler-Modules and FRP Composite Wraps/Gussets

Praveen Kumar Reddy Majjigapu

A review of post-earthquake reconnaissance studies revealed that reinforced concrete (RC) structures, designed and built before the development of Uniform Building Code (UBC) seismic design guidelines in 1976, have suffered complete collapse or severe damages due to the brittle failure of exterior beam-column joints. Over the past 50 years, several studies were conducted to strengthen exterior joints of in-service structures, with limited emphasis on developing simple, economic and durable repair strategies to improve energy absorption through large inelastic joint deformations. Even less emphasis was devoted to developing repair procedures that minimize stress-concentrations at joint corners and enhancing the strength, ductility, and energy dissipation capabilities of concrete structures with an emphasis on joint resistance improvements.

To address the above limitations, a novel approach of reinforcing vulnerable joints with filler-modules and fiber-reinforced polymer (FRP) composite wraps/gussets have been proposed and evaluated, herein. The proposed approach involved bonding filler-modules at the reentrant corners of a joint and securing them with reinforcing dowels to minimize corner stress-concentrations through smoother stress transfer in and around a joint. Additionally, bonding of FRP composite wraps or gussets on to the exposed beam-column faces was done to reinforce the joint core, thus enhancing the strength and energy absorption through joint confinement and reducing joint shear demand so that the plastic hinge could form away from the joint core.

To investigate the efficacy of the proposed approach in enhancing the joint structural capacities, twenty 2D RC exterior (T) joints were designed as per pre-1976 construction deficiencies and experimentally evaluated in control (as-built) and reinforced conditions through the variations in (i) filler-module geometric shape; (ii) filler-module material properties, (iii) FRP material, (iv) FRP wrap/gusset configurations, (v) confinement due to partial (U-anchors) versus complete (360°-anchors) diagonal wraps, and (vi) shear transfer through reinforcing dowels. The performance of the test specimens was recorded through numerical values of loads versus deformation and strains at the rupture of concrete, de-bond of FRP wrap from the concrete surface, yielding of steel rebar, shear failure of column or joint panel through diagonal tension and beam flexural failure phenomena. Test data evaluations measured up to the peak loads revealed that the proposed approach of reinforcing joints with filler-modules and FRP wraps/gussets is immensely useful in enhancing the strength and ductility by ~300%, and energy dissipation by about 1200%. Depending upon the reinforcing scheme(s), the magnitude of failure- loads and patterns varied in a controlled manner. Joints tested in “control” condition exhibited shear failure through diagonal tension and diagonal compression while the strengthened specimens failed in beam flexure or column shear, but in a ductile manner through yielding of the column- or beam- rebars. The strains measured on rebar surfaces at different locations of joint- and beam- sections revealed a significant

reduction in strain progression towards the joint panel (beam-column overlap). It was also noted that the use of low-stiffness filler-modules such as syntactic foam and engineered wood coupled with FRP wraps has tremendously enhanced the structural response of reinforced joints. It was also observed that joints reinforced with filler-modules and FRP wraps or gussets exhibiting beam flexural failure had more energy dissipation capacity when compared to specimens that failed in column shear. Based on the experimental results of reinforced joint specimens, joint behavior is characterized into three zones, i.e., A (onset of filler-module cracking), B (idealized yield - defined as the point beyond which a truss mechanism primarily resists the forces), and C (Peak load - referred to as a highest numerical value of load recorded during the testing). Furthermore, limit states (principal tension and shear) for joint at the onset of filler-module cracking (i.e., point A) and idealized yield (i.e., point B) have been established as a function of the concrete tensile strength ($k\sqrt{f'_c}$).

The outcomes of this research have proven the ability of the proposed approach in strengthening concrete joints cost-effectively; thus, the overall structural integrity. Although the scope of this dissertation is limited to the evaluation of exterior beam-column joints designed before 1976, the concepts can be extended to other joint configurations, including timber and steel construction. Future research on the proposed approach must be directed towards evaluating the performance of joints with additional stiffness contributions from the slab, and transverse beams to establish joint curvature limit states. Furthermore, machine learning tools must be employed to train, evaluate, and develop strength prediction models after generating additional test data in a strategic sense with an understanding developed based on the current research. Besides, finite element analysis studies on joint inelastic behavior must be performed by incorporating material nonlinearity (post-cracking behavior of concrete joint or element) to arrive at the optimized shape of filler-modules and optimized fiber orientation of FRP wraps/gussets as a function of the substrate strength versus stiffness and bond strengths of glue lines of the substrate, filler-modules and FRP wrap or gusset.

Dedication

This dissertation is dedicated

To my

Father: Dr. Koti Reddy Majjigapu

Mother: Smt. Padma Majjigapu

Brother: Mr. Naveen Majjigapu

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Motivation

“When things go wrong, as they sometimes will;

When the road you’re trudging seems all uphill;

When the funds are low and the debts are high;

And you want to smile but you have to sigh.

When all is pressing you down a bit-

Rest if you must, but don’t you quit

Success is failure turned inside out;

The silver tint on the clouds of doubt;

And you can never tell how close you are;

It may be near when it seems far.

So stick to the fight when you’re hardest hit-

It’s when things go wrong that you must not quit.”

-John Greenleaf Whittier

Table of Contents

Table of Contents	viii
List of Tables	xiii
List of Figures.....	xvi
1. Introduction.....	1
1.1 Research Background	1
1.2 Joint Reinforcement with Filler-Modules and FRP Wraps or Gussets	4
1.3 Research Objectives	7
1.4 Research Scope	8
1.5 Research Significance	10
1.6 Organization of Chapters	10
2. RC Exterior Joints Built Before 1976: A Brief Review	12
2.1 Introduction.....	12
2.2 Vulnerability of Buildings with Deficient Joints under Extreme Events	12
2.3 Joint Definition and Classification.....	14
2.4 Force Transfer Mechanism in 2D RC Exterior Joints	16
2.5 Assessment of Joint Shear Strength.....	17
2.5.1 Computation of principal stresses in a joint panel	19
2.5.2 Strut mechanism.....	22
2.6 Capacity Design of Joints	23
2.7 Parameters Influencing the Behavior of RC Exterior Joints.....	24
2.7.1 Column axial compressive load	24
2.7.2 Concrete compressive strength	25
2.7.3 Joint aspect ratio	27
2.7.4 Presence of slab and transverse beam.....	28

2.7.5	End hooks of beam reinforcement	28
2.7.6	Joint transverse (shear) reinforcement	29
2.7.7	Failure modes	29
2.8	Summary	30
3.	RC Exterior Joint Repair with FRP Composites: A Brief Review	32
3.1	Introduction	32
3.2	FRP Composites in Repair and Rehabilitation of Infrastructure	32
3.3	FRP Composites in Joint Strengthening	33
3.4	Previous Experimental Studies on FRP Composites in Joint Strengthening	34
3.4.1	Effectiveness of FRP wraps on joint shear strengthening	34
3.4.2	Effectiveness of FRP anchors in preventing debond	39
3.4.3	Influence of FRP confinement on bond strength of hooked bars	44
3.4.4	Influence of surface preparation on rehab effectiveness	49
3.4.5	Effectiveness of FRP wrap layup on joint strengthening	53
3.4.6	Effectiveness of joint core confinement through FRP wrapping	58
3.4.7	Influence of FRP strengthening on joints with transverse beams	61
3.5	Summary	66
4.	Experimental Program: Materials, Casting, Test Protocols, and Data Procurement..	68
4.1	Introduction	68
4.2	Specimen Details	68
4.2.1	Control specimens	69
4.2.2	Reinforced specimens	72
4.3	Specimen Casting	92
4.3.1	Filler-module fabrication	96
4.3.2	Installation of filler-modules at joint corners	99
4.3.3	Surface preparation	100
4.3.4	FRP wrap/gusset installation	101
4.4	Materials	103

4.4.1	Concrete	103
4.4.2	Steel rebar	106
4.4.3	Engineered wood (PSL).....	109
4.4.4	Engineered syntactic foam.....	110
4.4.5	FRP composite reinforcement.....	111
4.4.6	FRP composite confined concrete cylinders.....	111
4.5	Test Setup of 2D RC Exterior Joints.....	112
4.6	Loading Protocol.....	114
4.7	Instrumentation	114
4.7.1	Measurement of the applied load.....	114
4.7.2	Measurement of displacement	115
4.7.3	Measurement of strain.....	116
4.8	Data Collection	121
4.8.1	Data acquisition system	121
4.9	Computation of Principal Strains.....	121
4.10	Computation of Shear Strain.....	122
4.11	Summary	122
5.	Behavior of RC Joints Reinforced with Concrete/Syntactic Foam Filler-Modules and FRP Wraps/Gussets	123
5.1	Introduction.....	123
5.2	Experimental Results	126
5.2.1	Behavior of control specimens (JI-1, JI-5, and JIII-1).....	126
5.2.2	Behavior of joint reinforced with concrete filler-modules (JI-2).....	134
5.2.3	Behavior of joints reinforced with filler-modules and FRP wraps/gussets	138
5.3	Discussion of Test Results	162
5.3.1	Strength.....	162
5.3.2	Failure mode	165
5.3.3	Stiffness.....	167

5.3.4	Ductility index	169
5.3.5	Energy dissipation.....	172
5.3.6	Strains in joint-, beam-, and column- rebars.....	175
5.4	Conclusions.....	177
6.	Behavior of RC Joints Reinforced with Engineered Wood (PSL) Filler-Modules and FRP Wrap.....	180
6.1	Introduction.....	180
6.2	Experimental Results	181
6.2.1	Behavior of control specimen	181
6.2.2	Behavior of joint reinforced with CFRP wrap.....	183
6.2.3	Behavior of joints reinforced with PSL curve- and wedge- filler-modules.....	185
6.2.4	Behavior of joint reinforced with PSL curve filler-modules and CFRP wraps without and with reinforcing dowels.....	188
6.2.5	Behavior of joint reinforced with PSL wedge filler-modules and CFRP/GFRP wraps	191
6.3	Discussion of Test Results	196
6.3.1	Strength.....	196
6.3.2	Failure mode	199
6.3.3	Stiffness.....	203
6.3.4	Ductility index	204
6.3.5	Energy dissipation.....	206
6.3.6	Strains in joint-, beam-, and column- rebars.....	208
6.4	Conclusions.....	210
7.	Summary, Conclusions, and Recommendations.....	212
7.1	Introduction.....	212
7.2	Summary	212
7.3	Conclusions.....	214
7.4	Recommendations for Future Research	216

References	218
Appendix-A: Beam-Column Joints Reinforced with Concrete/Syntactic Foam Filler-Modules and CFRP Wraps.....	230
Appendix-B: Beam-Column Joints Reinforced with Concrete Filler-Modules and CFRP Gussets.....	242
Appendix-C: Beam-Column Joints Reinforced with Concrete Filler-Modules and GFRP Wraps	270
Appendix-D: Beam-Column Joints Reinforced with Engineered Wood Filler-Modules and FRP Wraps	299
Appendix-E: FRP Wrapped Concrete Cylinders.....	320

List of Tables

Table 1. Details of the experimental program	9
Table 2. Principal tensile stress limit states based on the available literature	21
Table 3. Summary of exterior RC joint failure modes [39]	30
Table 4. CFRP repair steps adopted by [48]	35
Table 5. Specimen wise strengthening methods [48]	36
Table 6. Description of retrofit schemes [49]	41
Table 7. Summary of experimental results by [49]	43
Table 8. Summary of experimental results [50]	47
Table 9. CFRP wrapping schemes and test results [51]	52
Table 10. Description of CFRP configurations [52]	55
Table 11. Summary of test results [52]	57
Table 12. Summary of experimental results [45]	64
Table 13. Beam-column joint reinforcing scheme for JI-2 through JI-4	75
Table 14. Beam-column joint reinforcing scheme for JI-6 through JI-8	78
Table 15. Beam-column joint reinforcing scheme for JIII-2 through JIII-4	82
Table 16. Beam-column joint reinforcing scheme for JII-2	86
Table 17. Beam-column joint reinforcing scheme for JII-3 and JII-4	88
Table 18. Beam-column joint reinforcing scheme for JII-5 through JII-8	89
Table 19. Properties of plain concrete (joint) specimens	104
Table 20. Properties of plain concrete (filler-modules) specimens	105

Table 21. Properties of steel rebar specimens.....	106
Table 22. Design properties of Parallam® PSL [54]	109
Table 23. Mechanical properties of SF grade syntactic foam [55]	110
Table 24. Properties of FRP composite materials [56] [57]	111
Table 25. Properties of FRP confined plain concrete (compression test)	111
Table 26. Properties of FRP confined plain concrete (split-tension test)	111
Table 27. Parameters tested to evaluate the performance of joints reinforced with concrete/syntactic foam filler-modules with and without FRP wraps/gussets	125
Table 28. Modulus of rupture for control joints (JI-1, JI-5, JIII-1)	127
Table 29. Principal strain limit state for control joints without axial load influence.....	127
Table 30. Principal strain limit state for control joints with axial load influence.....	127
Table 31. Summary of experimental results of joints reinforced with concrete filler-modules .	135
Table 32. Modulus of rupture of joints with filler-modules and CFRP wraps	140
Table 33. Summary of experimental results of joints with concrete/syntactic foam filler-modules with/out CFRP wraps	141
Table 34. Modulus of rupture of joints with filler-modules and CFRP gussets	146
Table 35. Summary of experimental results of joints with filler-modules and CFRP gussets ...	148
Table 36. Modulus of rupture of joints with filler-modules and GFRP wraps	155
Table 37. Summary of experimental results of joints with filler-modules and GFRP wraps	156
Table 38. Load vs. deflection response of joints with filler-modules and FRP wraps/gussets...	164
Table 39. Stiffness of joints reinforced with filler-modules and FRP wraps/gussets.....	167
Table 40. Ductility index of joints with filler-modules and FRP wraps/gussets	170

Table 41. Energy dissipation capacity of joints with filler-modules and FRP wraps/gussets	174
Table 42. Rebar strain data of joints with filler-modules and FRP wraps/gussets	176
Table 43. Summary of test results: joints with filler-modules with/out FRP wraps or gussets..	177
Table 44. Parameters tested to evaluate the performance of joints reinforced with PSL filler-modules and FRP wraps.....	180
Table 45. Modulus of rupture of joints reinforced with PSL filler-modules and FRP wraps.....	194
Table 46. Summary of experimental results of joints reinforced with PSL filler-modules and FRP wraps	195
Table 47. Load vs. deflection of joints reinforced with PSL filler-modules and FRP wraps	196
Table 48. Failure modes of joints reinforced with PSL filler-modules and FRP wraps	200
Table 49. Stiffness of joints reinforced with PSL filler-modules and FRP wraps.....	203
Table 50. Ductility index of joints reinforced with PSL filler-modules and FRP wraps.....	204
Table 51. Energy dissipation of joints reinforced with PSL filler-modules and FRP wraps.....	206
Table 52. Rebar strain data of joints reinforced with PSL filler-modules and FRP wraps.....	208
Table 53. Summary of test results: joints reinforced with PSL filler-modules and FRP wraps .	210

List of Figures

Figure 1. Low-rise building collapse during 2016 Ecuador earthquake [9]	2
Figure 2. School building damage in Eastern Turkey due to exterior joint failure [10].....	3
Figure 3. 2D exterior (T) joint reinforcement using filler-modules and FRP wraps [33]-[34]	5
Figure 4. Beam-column-slab joint reinforcement using filler-modules and FRP gussets [35]	6
Figure 5. 2016 Ecuador earthquake - building damage (Juan Cevallos/Getty Images) [17]	13
Figure 6. Joint classification based on the arrangement in a framed system [19]	15
Figure 7. Joint sub-classification based on the arrangement of beams and columns [20].....	15
Figure 8. The force-transfer mechanism in a most practical case of RC exterior joint [19]	16
Figure 9. Resultant forces on a 2D exterior joint under beam and column loads [21]	17
Figure 10. Diagonal stress fields in an exterior joint [22]	19
Figure 11. Strut mechanism of shear force transfer in an exterior joint [26].....	22
Figure 12. Joint shear/rebar bond deterioration failure in exterior joints [27].....	23
Figure 13. Effect of column axial load on joint shear strength [32]	25
Figure 14. Story shear vs. joint shear strain [33]	26
Figure 15. Effect of joint aspect ratio [32].....	27
Figure 16. Steel reinforcement detailing of control and retrofitted test specimens [48]	34
Figure 17. Test setup of [48].....	36
Figure 18. FRP fan anchor [49]	39
Figure 19. Test specimen details [49]	40
Figure 20. Test setup of [49].....	40

Figure 21. Rehabilitation scheme of RS-1V-1X [49]	41
Figure 22. Rehabilitation scheme of RS-1V-2X [49]	42
Figure 23. Rehabilitation scheme of RS-1V-2X-2H.I [49].....	42
Figure 24. Rehabilitation scheme of RS-1V-2X-2H.II [49]	43
Figure 25. Experimental test setup [50]	45
Figure 26. Details of test specimen C16N [50].....	45
Figure 27. Details of test specimen C25N [50].....	46
Figure 28. Details of test specimen C32N [50].....	46
Figure 29. CFRP wrap details for C16N, C25N, and C32N [50]	47
Figure 30. Test specimen reinforcement details and dimensions [51].....	49
Figure 31. Test setup of joint specimen [51]	50
Figure 32. CFRP wrap layout [51].....	51
Figure 33. Details of the non-seismic joint [52]	53
Figure 34. CFRP configurations for strengthening non-seismic joints [52]	54
Figure 35. Test setup [52]	54
Figure 36. Joint steel reinforcement details [53]	58
Figure 37. CFRP rehab detailing of joint specimens [53]	59
Figure 38. Geometric details of test specimen [45]	61
Figure 39. Steel reinforcement detailing of test specimen [45]	62
Figure 40. FRP retrofitted specimen configurations [45]	62
Figure 41. Schematic of control joint specimen	69

Figure 42. Geometric layout of control joint specimens.....	70
Figure 43. Steel rebar layout of control joint specimens	71
Figure 44. Geometric layout of joint specimens reinforced with wedge filler-module.....	72
Figure 45. Steel rebar layout of joint specimens reinforced with wedge filler-module	73
Figure 46. Steel rebar layout of joint specimens reinforced with curve filler-module	74
Figure 47. Plywood formwork for casting concrete joint specimens	92
Figure 48. Preparation of reinforcement cage for beam and column sections.....	93
Figure 49. Grinding at select locations of rebar cage for installation of strain gages.....	93
Figure 50. Closer view of the reinforcement cage after assembling in a mold	94
Figure 51. Installing strain gage on steel rebar	95
Figure 52. Placement of concrete during test specimen preparation	96
Figure 53. Casting of concrete filler-modules	97
Figure 54. Making of Parallel Strand Lumber (PSL) filler-modules.....	98
Figure 55. Filler-module installation process at joint corners	99
Figure 56. Surface preparation through water-jet washing.....	100
Figure 57. Concrete surface before- and after- water-jet washing.....	100
Figure 58. FRP wrapping of joint specimen	101
Figure 59. FRP gusset used in the current study for reinforcing joint specimens	102
Figure 60. Load vs. position curves for plain concrete (joint) specimens	104
Figure 61. Load vs. position curves for plain concrete (filler-modules) specimens.....	105
Figure 62. Failure modes of grade 60 #6 rebar specimens	107

Figure 63. Failure modes of grade 60 #3 rebar specimens	107
Figure 64. Stress-strain plots for grade 60 #6 steel rebar specimens.....	108
Figure 65. Stress-strain plots for grade 60 #3 steel rebar specimens.....	108
Figure 66. Trus Joist Parallam® PSL [54].....	109
Figure 67. SF 3000 grade microsphere syntactic foam.....	110
Figure 68. Test setup of control joint specimens	113
Figure 69. Test setup of reinforced joint specimens	114
Figure 70. Linear variable displacement transducer (LVDT) (OMEGA Engineering) [58]	115
Figure 71. Schematic view of strain gage arrangement on steel rebar cage	116
Figure 72. Strain gage arrangement on the concrete surface	117
Figure 73. Strain gage arrangement on FRP wrap surface	118
Figure 74. Strain gage arrangement on FRP wrap surface	119
Figure 75. Strain gage arrangement on FRP wrap surface	120
Figure 76. Vishay 7000 data acquisition system [59].....	121
Figure 77. Shear rosette arrangement	122
Figure 78. Load vs. column rebar strain response of a typical joint specimen in this study	124
Figure 79. Initial diagonal tension cracking in the joint panel	128
Figure 80. Failure of JI-1	129
Figure 81. Failure of JI-5	129
Figure 82. Failure of JIII-1.....	130
Figure 83. Joint principal strains for JI-1	131

Figure 84. Joint shear strain for JI-1	131
Figure 85. Joint principal strains for JI-5.....	132
Figure 86. Joint shear strain for JI-5	132
Figure 87. Joint principal strains for JIII-1	133
Figure 88. Joint shear strain for JIII-1	133
Figure 89. Failure of JI-2 (front view)	136
Figure 90. Failure of JI-2 (rear view).....	136
Figure 91. Column principal strains for JI-2.....	137
Figure 92. Column shear strain for JI-2	137
Figure 93. Failure of JI-3	142
Figure 94. Failure of JI-4	142
Figure 95. Joint principal strains for JI-3.....	143
Figure 96. Joint shear strain for JI-3	143
Figure 97. Joint principal strains for JI-4.....	144
Figure 98. Joint shear strain for JI-4	144
Figure 99. Strain response along the longitudinal axis of filler-module (JI-7).....	147
Figure 100. Strain response close to the filler-module tip on beam section (JI-8)	147
Figure 101. Failure of JI-6	149
Figure 102. Failure of JI-7	149
Figure 103. Failure of JI-8	150
Figure 104. Joint principal strains for JI-6.....	151

Figure 105. Joint shear strain for JI-6	151
Figure 106. Joint principal strains for JI-7	152
Figure 107. Joint shear strain for JI-7	152
Figure 108. Joint principal strains for JI-8.....	153
Figure 109. Joint shear strain for JI-8	153
Figure 110. Failure of JIII-2.....	157
Figure 111. Failure of JIII-3.....	157
Figure 112. Failure of JIII-4.....	158
Figure 113. Joint principal strains for JIII-2	159
Figure 114. Joint shear strain for JIII-2	159
Figure 115. Joint principal strains for JIII-3	160
Figure 116. Joint shear strain for JIII-3	160
Figure 117. Joint principal strains for JIII-4	161
Figure 118. Joint shear strain for JIII-4	161
Figure 119. Load vs. deflection response of joints with filler-modules with/out CFRP wraps..	163
Figure 120. Load vs. deflection response of joints with filler-modules and CFRP gussets	163
Figure 121. Load vs. deflection response of joints with filler-modules and GFRP wraps	164
Figure 122. Buckling of the column longitudinal rebars (JI-2)	166
Figure 123. Cantilever beam with beam tip deflection.....	167
Figure 124. Force-displacement relationship of a ductile structure [63]	169
Figure 125. Energy dissipation capacity of joints with filler-modules and CFRP wraps.....	172

Figure 126. Energy dissipation capacity of joints with filler-modules and CFRP wraps.....	173
Figure 127. Energy dissipation capacity of joints with filler-modules and GFRP wraps.....	173
Figure 128. Failure of specimen JII-1	181
Figure 129. Joint principal strains in JII-1	182
Figure 130. Joint shear strain in JII-1	182
Figure 131. Failure of specimen JII-2.....	183
Figure 132. Joint principal strains in JII-2	184
Figure 133. Joint shear strains in JII-2.....	184
Figure 134. Failure of specimen JII-3 (left) and JII-4 (right)	185
Figure 135. Column principal strain in JII-3	186
Figure 136. Column shear strain JII-3	186
Figure 137. Column principal strain in JII-4	187
Figure 138. Column shear strain in JII-4	187
Figure 139. Failure of specimen JII-5 (left) and JII-6 (right)	188
Figure 140. Joint principal strain in JII-5.....	189
Figure 141. Joint shear strain in JII-5	189
Figure 142. Joint principal strain in JII-6.....	190
Figure 143. Joint shear strain in JII-6	190
Figure 144. Failure of specimen JII-7 (left) and JII-8 (right)	191
Figure 145. Joint principal strain in JII-7.....	192
Figure 146. Joint shear strain in JII-7	192

Figure 147. Joint principal strain in JII-8.....	193
Figure 148. Joint shear strain in JII-8	193
Figure 149. Load vs. deflection of joints reinforced with PSL filler-modules and FRP wraps..	197
Figure 150. Energy dissipation of joints reinforced with PSL filler-modules and FRP wraps ..	206
Figure A.1. Beam load vs. column longitudinal rebar strain	230
Figure A.2. Beam load vs. beam longitudinal rebar strain	231
Figure A.3. Beam load vs. joint tail rebar strain	231
Figure A.4. Beam load vs. column longitudinal rebar strain	232
Figure A.5. Beam load vs. beam longitudinal rebar strain	232
Figure A.6. Beam load vs. joint tail rebar strain	233
Figure A.7. Beam load vs. concrete strain in joint panel.....	233
Figure A.8. Beam load vs. filler-module longitudinal strain	234
Figure A.9. Beam load vs. column longitudinal rebar strain	235
Figure A.10. Beam load vs. beam longitudinal rebar strain	235
Figure A.11. Beam load vs. beam stirrup strain	236
Figure A.12. Beam load vs. column principal strain	236
Figure A.13. Beam load vs. column shear strain	237
Figure A.14. Beam load vs. filler-module longitudinal strain	237
Figure A.15. Beam load vs. filler-module longitudinal strain	238
Figure A.16. Beam load vs. column longitudinal rebar strain	239
Figure A.17. Beam load vs. beam longitudinal rebar strain	239

Figure A.18. Beam load vs. filler-module longitudinal strain	240
Figure A.19. Beam load vs. filler-module longitudinal strain	240
Figure A.20. Beam load vs. column principal strain	241
Figure A.21. Beam load vs. column shear strain	241
Figure B.1. Beam load vs. column longitudinal rebar strain	242
Figure B.2. Beam load vs. beam longitudinal rebar strain.....	243
Figure B.3. Beam load vs. joint tail rebar strain	243
Figure B.4. Beam load vs. beam stirrup strain.....	244
Figure B.5. Beam load vs. concrete strain in beam	244
Figure B.6. Beam load vs. concrete strain in beam	245
Figure B.7. Beam load vs. concrete strain in column	245
Figure B.8. Beam load vs. column longitudinal rebar strain	246
Figure B.9. Beam load vs. beam longitudinal rebar strain.....	246
Figure B.10. Beam load vs. beam stirrup strain.....	247
Figure B.11. Beam load vs. FRP strain in beam.....	248
Figure B.12. Beam load vs. beam principal strain.....	248
Figure B.13. Beam load vs. beam shear strain.....	249
Figure B.14. Beam load vs. filler-module longitudinal strain	249
Figure B.15. Beam load vs. filler-module longitudinal strain	250
Figure B.16. Beam load vs. column FRP strain.....	250
Figure B.17. Beam load vs. column rebar FRP strain	251

Figure B.18. Beam load vs. FRP strain in joint panel.....	251
Figure B.19. Beam load vs. filler-module FRP strain.....	252
Figure B.20. Beam load vs. FRP strain in column and filler-module.....	252
Figure B.21. Beam load vs. FRP strain in column and filler-module.....	253
Figure B.22. Beam load vs. column longitudinal rebar strain	254
Figure B.23. Beam load vs. beam stirrup strain.....	254
Figure B.24. Beam load vs. beam longitudinal rebar strain.....	255
Figure B.25. Beam load vs. filler-module longitudinal strain	256
Figure B.26. Beam load vs. filler-module longitudinal strain	256
Figure B.27. Beam load vs. FRP strain in beam.....	257
Figure B.28. Beam load vs. FRP strain in joint panel.....	257
Figure B.29. Beam load vs. beam principal strain	258
Figure B.30. Beam load vs. beam shear strain.....	258
Figure B.31. Beam load vs. filler-module FRP strain.....	259
Figure B.32. Beam load vs. FRP strain in column and filler-module.....	259
Figure B.33. Beam load vs. FRP strain in column and filler-module.....	260
Figure B.34. Beam load vs. column FRP strain.....	260
Figure B.35. Beam load vs. column FRP strain.....	261
Figure B.36. Beam load vs. column longitudinal rebar strain	262
Figure B.37. Beam load vs. beam longitudinal rebar strain.....	262
Figure B.38. Beam load vs. beam stirrup strain.....	263

Figure B.39. Beam load vs. filler-module longitudinal strain	264
Figure B.40. Beam load vs. filler-module longitudinal strain	264
Figure B.41. Beam load vs. FRP strain in beam.....	265
Figure B.42. Beam load vs. FRP strain in joint panel.....	265
Figure B.43. Beam load vs. beam principal strain	266
Figure B.44. Beam load vs. beam shear strain.....	266
Figure B.45. Beam load vs. filler-module FRP strain.....	267
Figure B.46. Beam load vs. FRP strain in column and filler-module.....	267
Figure B.47. Beam load vs. FRP strain in column and filler-module.....	267
Figure B.48. Beam load vs. column FRP strain.....	268
Figure B.49. Beam load vs. column FRP strain.....	269
Figure C.1. Beam load vs. column longitudinal rebar strain	270
Figure C.2. Beam load vs. beam longitudinal rebar strain.....	271
Figure C.3. Beam load vs. beam stirrup strain.....	271
Figure C.4. Beam load vs. joint tail rebar strain	272
Figure C.5. Beam load vs. concrete strain in the joint panel	273
Figure C.6. Beam load vs. concrete strain in the joint panel	273
Figure C.7. Beam load vs. concrete strain in the joint panel	274
Figure C.8. Beam load vs. column strain in the joint panel.....	274
Figure C.9. Beam load vs. column longitudinal rebar strain	275
Figure C.10. Beam load vs. beam longitudinal rebar strain.....	275

Figure C.11. Beam load vs. beam stirrup strain.....	276
Figure C.12. Beam load vs. filler-module longitudinal strain	277
Figure C.13. Beam load vs. filler-module longitudinal strain	277
Figure C.14. Beam load vs. FRP strain in the beam	278
Figure C.15. Beam load vs. FRP strain in the joint panel.....	278
Figure C.16. Beam load vs. beam principal strain	279
Figure C.17. Beam load vs. beam shear strain.....	279
Figure C.18. Beam load vs. filler-module FRP strain.....	280
Figure C.19. Beam load vs. FRP strain in column and filler-module.....	280
Figure C.20. Beam load vs. FRP strain in column and filler-module.....	281
Figure C.21. Beam load vs. column FRP strain.....	281
Figure C.22. Beam load vs. column FRP strain.....	282
Figure C.23. Beam load vs. column longitudinal rebar strain	283
Figure C.24. Beam load vs. beam longitudinal rebar strain.....	283
Figure C.25. Beam load vs. beam stirrup strain.....	284
Figure C.26. Beam load vs. filler-module longitudinal strain	285
Figure C.27. Beam load vs. filler-module longitudinal strain	285
Figure C.28. Beam load vs. FRP strain in the beam	286
Figure C.29. Beam load vs. FRP strain in the joint panel.....	286
Figure C.30. Beam load vs. beam principal strain	287
Figure C.31. Beam load vs. beam shear strain.....	287

Figure C.32. Beam load vs. filler-module FRP strain.....	288
Figure C.33. Beam load vs. FRP strain in column and filler-module.....	288
Figure C.34. Beam load vs. FRP strain in column and filler-module.....	289
Figure C.35. Beam load vs. column FRP strain.....	289
Figure C.36. Beam load vs. column FRP strain.....	290
Figure C.37. Beam load vs. column longitudinal rebar strain	291
Figure C.38. Beam load vs. beam longitudinal rebar strain.....	291
Figure C.39. Beam load vs. beam stirrup strain.....	292
Figure C.40. Beam load vs. filler-module longitudinal strain	293
Figure C.41. Beam load vs. filler-module longitudinal strain	293
Figure C.42. Beam load vs. FRP strain in the beam	294
Figure C.43. Beam load vs. FRP strain in the joint panel.....	294
Figure C.44. Beam load vs. beam principal strain	295
Figure C.45. Beam load vs. beam shear strain.....	295
Figure C.46. Beam load vs. filler-module FRP strain.....	296
Figure C.47. Beam load vs. FRP strain in column and filler-module.....	296
Figure C.48. Beam load vs. FRP strain in column and filler-module.....	297
Figure C.49. Beam load vs. column FRP strain.....	297
Figure C.50. Beam load vs. column FRP strain.....	298
Figure D.1. Beam load vs. column longitudinal rebar strain	299
Figure D.2. Beam load vs. beam longitudinal rebar strain	300

Figure D.3. Beam load vs. joint tail rebar strain	300
Figure D.4. Beam load vs. beam stirrup strain	301
Figure D.5. Beam load vs. joint principal strain	302
Figure D.6. Beam load vs. joint shear strain	302
Figure D.7. Beam load vs. column longitudinal rebar strain	303
Figure D.8. Beam load vs. beam longitudinal rebar strain	303
Figure D.9. Beam load vs. joint tail rebar strain	304
Figure D.10. Beam load vs. beam stirrup strain	304
Figure D.11. Beam load vs. column principal strain	305
Figure D.12. Beam load vs. column shear strain	305
Figure D.13. Beam load vs. column longitudinal rebar strain	306
Figure D.14. Beam load vs. beam longitudinal rebar strain	306
Figure D.15. Beam load vs. joint tail rebar strain	307
Figure D.16. Beam load vs. beam stirrup strain	307
Figure D.17. Beam load vs. column principal strain	308
Figure D.18. Beam load vs. column shear strain	308
Figure D.19. Beam load vs. column longitudinal rebar strain	309
Figure D.20. Beam load vs. beam longitudinal rebar strain	309
Figure D.21. Beam load vs. beam stirrup strain	310
Figure D.22. Beam load vs. joint tail rebar strain	310
Figure D.23. Beam load vs. beam principal strain	311

Figure D.24. Beam load vs. beam shear strain	311
Figure D.25. Beam load vs. column longitudinal rebar strain	312
Figure D.26. Beam load vs. beam longitudinal rebar strain	312
Figure D.27. Beam load vs. joint tail rebar strain	313
Figure D.28. Beam load vs. beam stirrup strain	313
Figure D.29. Beam load vs. beam principal strain.....	314
Figure D.30. Beam load vs. beam shear strain	314
Figure D.31. Beam load vs. column longitudinal rebar strain	315
Figure D.32. Beam load vs. beam longitudinal rebar strain	315
Figure D.33. Beam load vs. beam stirrup strain	316
Figure D.34. Beam load vs. column longitudinal rebar strain	317
Figure D.35. Beam load vs. beam longitudinal rebar strain	317
Figure D.36. Beam load vs. joint tail rebar strain	318
Figure D.37. Beam load vs. beam stirrup strain	318
Figure D.38. Beam load vs. beam principal strain.....	319
Figure D.39. Beam load vs. beam shear strain	319
Figure E.1. Stress vs. strain behavior of JI-CS	320
Figure E.1. Stress vs. strain behavior of JI-S1	320
Figure E.2. Stress vs. strain behavior of JI-S2	321
Figure E.3. Stress vs. strain behavior of JII-CS	321
Figure E.5. Stress vs. strain behavior of JII-S1	322

Figure E.6. Stress vs. strain behavior of JII-S2.....	322
Figure E.7. Stress vs. strain behavior of JI-CS	323
Figure E.8. Stress vs. strain behavior of JI-S1	323
Figure E.9. Stress vs. strain behavior of JI-S2.....	324
Figure E.10. Stress vs. strain behavior of JII-CS.....	324
Figure E.11. Stress vs. strain behavior of JII-S1.....	325
Figure E.12. Stress vs. strain behavior of JII-S2.....	325

1. Introduction

1.1 Research Background

Natural disasters across the globe are growing at an alarming rate, causing severe damages to infrastructure and humankind while leading to severe economic imbalances for developing countries [1]. According to a study published by the World Bank Group in 2016, each year, natural disasters cause approximately \$520 billion in losses while pushing 26 million people into poverty [2]. A significant portion of losses incurred during natural disasters is attributed to infrastructure damages. For example, the 1906 San Francisco earthquake incurred \$42 billion (estimated in 2006 dollar value) in direct losses due to damage of 28,000 buildings while causing 3,000 fatalities [3]. Predications by seismologists reveal that, by 2036, there exists a 63% chance of occurrence of at least one major earthquake in the Bay area with a magnitude of 6.7 or higher [4]. If such a seismic event were to occur in the Bay area, nearly 127,000 buildings will either collapse or experience extensive damages in the Bay area that are beyond repair conditions, leading to 3,400 deaths and \$120 billion in losses [5].

One of the critical aspects leading to severe building damages is the inability of beam-column joints (hereafter referred to as "joints") to swiftly transfer forces (axial, shear, bending, torsional) between joints and adjoining members under extreme forces. The structural damages are even worse in pre-1976 buildings that are designed for gravity loads due to the lack of UBC seismic design codes [6]. The primary reasons for such failures are attributed to soft or weak story constructions, inadequate seismic design capacity, and poor reinforcement detailing, especially in the joint panel. The failure behavior of joints is often characterized by its non-optimal designing and detailing such as loss of rebar anchorage, insufficient development length of beam reinforcement extending into the column, insufficient lap splice length, bars with hook-end, smooth and rounded rebar, and little or no transverse reinforcement resulting in weak column-strong beam combination [7]. This condition is even worse in exterior joints that are partially confined by the absence of structural elements (e.g., transverse beam, column, or slab) in one or more directions. Such joints in a framed system have been identified as more critical than the

interior joints because of potential brittle failure at/near the joint, leading to the sudden loss of load-carrying capacity and resulting in catastrophic structural collapse [8], as shown in Figures 1 and 2. Even to date, many such structures still exist in seismically active zones around the world and particularly in the western United States. Based on several post-earthquake reconnaissance studies [9]-[12] it is well recognized that joints are the Achilles heel in maintaining the integrity of structural systems and hence must be strengthened to minimize the risk of catastrophic collapse.

Figure 1. Low-rise building collapse during 2016 Ecuador earthquake [9]

Figure 2. School building damage in Eastern Turkey due to exterior joint failure [10]

Over the past 50 years, several strengthening techniques have been proposed and evaluated to enhance the performance of joints. These techniques include: (i) externally bonded fiber-reinforced polymer (FRP) wraps [15]-[19], (ii) addition of steel plates [20]-[21], (iii) steel haunch bolting at the interface of beam-column junction [22]-[24], (iv) concrete jacketing [11], (v) L-shaped polymer blocks bonded at the beam-column interface [12], (vi) planar joint enlargement [27]-[30], and (vii) concrete cover bonded together with CFRP jacket [31] and [32]. Despite the technical advancements and number of experimental studies conducted on the behavior of joints retrofitted under various schemes, relatively less effort has been placed on developing simple, yet economical and durable repair strategies to improve force transfer through joints. Even less emphasis was devoted to developing repair procedures that minimize stress-concentrations at joint corners, and enhancing the strength, ductility, and energy dissipation capacities. Hence, the current research using filler-modules and FRP composite wrap or gusset systems proposed by [33]-[35] has been undertaken to address some of the above concerns by experimentally evaluating twenty 2D exterior (T) reinforced concrete (RC) exterior joint specimens under monotonic loads. Although the research performed herein was focused on enhancing the behavior and integrity of exterior joints that are designed before 1976, the concepts can be extended to other joint configurations, including new construction.

1.2 Joint Reinforcement with Filler-Modules and FRP Wraps or Gussets

Based on the understanding of joint behavior and shortcomings of existing repair procedures, a novel approach of reinforcing existing and newly built joints with filler-modules and FRP composite wraps or gussets proposed by [33]-[35] has been evaluated herein. The method's geometric intricacies are identified in Figures 3 and 4. The proposed approach involved bonding filler-modules at the reentrant corners of a joint and securing them with reinforcing dowels to minimize corner stress-concentrations through the smoother stress transfer in and around a joint. Additionally, it required bonding of FRP composite wraps or gussets on to the exposed faces to reinforce and confine the joint core, thus further enhancing the joint resistance. This approach can significantly reduce the shear induced in the joint panel with the relocation plastic hinge formation away from the joint core. The proposed schemes can enable notable improvements as a retrofit option for the in-service structures and new construction of structural systems under extreme events. Several advantages of retrofitting structures with the proposed approach are: (i) performance enhancement such as strength, energy absorption, and ductility, (ii) structural integrity improvements, avoiding catastrophic failure through better joint confinement and moment transfer, (iii) rebar development length increase extending into a joint, (iv) corner stress-concentration abatement, (v) joint durability and fire resistance enhancement, (vi) avoidance of joint rebar congestion for new construction, (vii) economic corrections in detailing errors in the designs, and (viii) others.

Figure 3. 2D exterior (T) joint reinforcement using filler-modules and FRP wraps [33]-[34]

Figure 4. Beam-column-slab joint reinforcement using filler-modules and FRP gussets [35]

1.3 Research Objectives

The primary objective of this research was to experimentally evaluate the effectiveness of filler-modules and FRP wraps or gussets for strengthening the reinforced concrete exterior joint specimens for improved structural performance. Emphasis was laid on evaluating 2D exterior (T) joints of non-ductile concrete frames constructed before the development of UBC seismic codes in 1976. To accomplish the research objective, the following tasks have been performed:

1. Understand the state-of-the-art behavior of reinforced concrete 2D exterior joints based on the published literature.
2. Review existing rehabilitation techniques from a phenomenological viewpoint.
3. Propose and develop a simple, reliable, and economical retrofit approach to strengthen reinforced concrete 2D joints based on the understanding developed from tasks 1 and 2.
4. Design and construct reinforced concrete joints in control and reinforced conditions.
5. Test FRP wrapped concrete cylinders under axial compression and split-tension to establish the stress-strain relationship at the first crack and peak loads.
6. Investigate the efficacy of the proposed joint reinforcing schemes in improving structural responses such as strain distribution in rebars, strength, stiffness, ductility, energy dissipation, and failure modes under monotonic loads.
7. Provide recommendations for future research to advance the state-of-the-art of proposed research to 3D space frames with out-of-plane member attachments such as slab and transverse beams in two or more directions.

1.4 Research Scope

The scope of this study is limited to experimental evaluation of 2D RC exterior (T) joint specimens exhibiting pre-1976 construction deficiencies: (i) low strength concrete (~2,639, 3,309, 3,866 psi), (ii) inadequate anchorage length of beam bottom bars extending into the joint, (iii) no transverse reinforcement (i.e., stirrups) in the joint panel, and (iv) weak column-strong beam design. A total of twenty specimens were tested in four batches under static axial (column) and bending (beam) loads to determine the efficiency of the proposed reinforcing system in improving the structural capacities such as strength, stiffness, ductility, and energy dissipation. The influence of the following parameters on the joint performance is evaluated as a part of the proposed scope:

- (i) filler-module geometric shape: curve versus wedge.
- (ii) filler-module material properties: low- (3,376 psi), medium- (5,472 psi), and high- (9,545 psi) strength concrete; engineered syntactic foam microsphere (SF 3000); engineered wood, i.e., parallel strand lumber (PSL 2.0E).
- (iii) shear transfer through reinforcing dowels: with and without dowels for filler-modules.
- (iv) FRP material: glass (GFRP) versus carbon (CFRP).
- (v) FRP application: wrap (hand layup) versus gussets (prefabricated laminates).
- (vi) FRP anchors: no anchors, U-anchors, and 360°-anchors.

A summary of the experimental program is outlined in Table 1.

Table 1. Details of the experimental program

Joint	f'_c	CL	Reinforcing scheme
JI-1	2639 psi	$0.28f'_cA_g$	Control
JI-2			C-W(3.3 ksi)+D
JI-3			C-W(3.3 ksi)+D+CFRP Wrap+360°-anchor
JI-4			SF-W(4.3 ksi)+D+CFRP Wrap+360°-anchor
JI-5	2639 psi	$0.18f'_cA_g$	Control
JI-6			C-W(5.5 ksi)+D+CFRP Gusset+U-anchor
JI-7			C-W(5.5 ksi)+D+CFRP Gusset+U-anchor*
JI-8			C-W(9.5 ksi)+D+CFRP Gusset+U-anchor*
III-1	3866 psi	$0.18f'_cA_g$	Control
III-2			C-C(5.5 ksi)+D+GFRP Wrap+no anchor*
III-3			C-W(5.5 ksi)+D+GFRP Wrap+no anchor*
III-4			C-W(5.5 ksi)+D+GFRP Wrap+U-anchor*
II-1	3309 psi	$0.21f'_cA_g$	Control
II-2			CFRP Wrap
II-3			PSL-C+D
II-4			PSL-W+D
II-5			PSL-C+CFRP Wrap+360°-anchor
II-6			PSL-C+D+CFRP Wrap+360°-anchor
II-7			PSL-W+D+CFRP Wrap+360°-anchor
II-8			PSL-W+D+GFRP Wrap+360°-anchor

D: dowels; C-W: concrete wedge; C-C: concrete curve; SF-W: syntactic foam wedge; PSL-C: Parallel strand lumber curve; PSL-W: Parallel strand lumber wedge; CL: column axial load; f'_c : substrate concrete compressive strength; A_g : gross cross-sectional area of concrete (column); *CFRP layer bonded between filler-module and substrate beam and column that receive filler-modules at the reentrant corners.

1.5 Research Significance

The high vulnerability of existing structures built before 1976 is highlighted in a disaster resilience report [13]. For example, according to [14], if a 7.2 magnitude earthquake were to happen on the San Andreas Fault, nearly: (i) 25,000 buildings will be unsafe to occupy, (ii) 75,000 buildings will experience light to moderate damage, (iii) 3,500 buildings will have to be demolished and rebuilt, (iv) 200-300 people could be killed and (v) 7000 people could have injuries needing medical care. Also, 90% of San Francisco's 120,000 buildings erected in mid-1970 are not compliant with current seismic requirements [15]. Replacement of such vulnerable structures with new designs following current seismic guidelines is not an economically sustainable solution. Hence, to avoid catastrophe under future earthquakes and ensure safe living conditions, there is an immediate need to develop alternative reinforcing schemes that are practically feasible, economically viable, and able to withstand extreme events structurally.

Therefore, the proposed study to reinforce RC joints with filler-modules and FRP composite wraps or gussets will not only enhance safety, service life, and cost-effectiveness through retrofitting, leading to near disaster-proof structures but also immensely useful in minimizing the high stress-concentration at corners. Experimental evaluations accomplished through this study will help create new knowledge of reinforcing joints, including a fundamental understanding of failure mechanisms and strain limits. The outcomes of this dissertation will underpin the future development of design procedures and guidelines to reinforce new and pre-existing joints and avoid economic hardship for those living in areas vulnerable to seismic or hurricane forces.

1.6 Organization of Chapters

Research carried out in this study is organized into the following chapters:

Chapter 1: This chapter provided the research- background, objectives, scope, and significance.

Chapter 2: This chapter provided a brief review on studies related to RC joints built before 1976 with emphasis on critical parameters that influence the joint behavior, such as column axial compressive load, concrete compressive strength, joint aspect ratio, presence of slab and transverse

beam, end hooks (180° or 90°) of beam reinforcement and their embedment length into joint, joint shear reinforcement (hereafter referred to as “transverse reinforcement”), and joint failure modes. Finally, this chapter provided conclusions based on the critical outcomes of these studies.

Chapter 3: This chapter provided a brief review of the repair and retrofit of RC exterior joints using FRP composites. This review was performed based on the influence of various parameters that govern the FRP composite repair of joints. Finally, this chapter was concluded based on the key findings from the previous experimental investigations.

Chapter 4: This chapter provided in-depth information about the experimental program conducted on 2D RC exterior joints under monotonic loads, including the types of materials employed, their properties, and the load strain/deformation response. More specifically, the experimental program described the specimen details, specimen casting process, reinforcing schemes, material properties, test setup, loading protocol, and instrumentation employed to monitor, record, and collect test data.

Chapter 5: This chapter dealt with the experimental investigations of 2D RC exterior joints reinforced with concrete/syntactic foam filler-modules and FRP wraps or gussets, performed as part of this research. The various sections in this chapter presented experimental findings, discussions of test results, and conclusions.

Chapter 6: This chapter dealt with the experimental investigations of 2D RC exterior joints reinforced with PSL filler-modules and FRP wraps, performed as part of this research. The various sections in this chapter presented experimental findings, discussions of test results, and conclusions.

Chapter 7: A summary of conclusions based on the experimental investigations performed in chapters 5 and 6 was presented comprehensively. Besides, future recommendations for expanding the scope of this study are highlighted.

2. RC Exterior Joints Built Before 1976: A Brief Review

2.1 Introduction

Structures built before the launch of seismic design codes by UBC in 1976 exhibited severe damages due to the lack of understanding of seismic capacity design principles. Several laboratory studies performed on joint specimens replicating pre-1976 construction deficiencies (identified in Section 2.2) also confirmed the vulnerability of such structures under extreme events. To enhance safe and economical design practices, fundamentals of joint behavior under the influence of various parameters governing the joint behavior must be fully understood. This chapter, therefore, provides a comprehensive review of studies performed on 2D exterior (T) joints, as shown in Figure 7d, under the influence of critical parameters governing the joint behavior. The parameters reviewed in the following sections are:

- i) column axial compressive stresses,
- ii) concrete compressive strength,
- iii) joint aspect ratio,
- iv) presence of slab and transverse beam,
- v) end hooks (180° or 90°) of beam reinforcement and their embedment length into joint,
- vi) joint shear reinforcement (hereafter referred to as “transverse reinforcement”), and
- vii) joint failure modes.

2.2 Vulnerability of Buildings with Deficient Joints under Extreme Events

Joints of a moment-resisting frame play a crucial role in distributing the induced forces between beam, column, and slab and maintaining structural integrity, especially in multistory structures. The vulnerability of such structures designed and built with an inadequate understanding of structural responses under extreme load events is often dependent on the joint performance. This behavior is evident from many of the published post-earthquake structural damage data (Mexico 1985, Loma Prieta 1989, Kobe 1995, Kocaeli 1999, Tehuacan 1999, Izmit 1999, Athens 1999, Chi-Chi, 1999, Boumerdes 2003, L’Aquila 2009) and analyses [39]-[42]. The primary role of

joints is to maintain the structural integrity under combined forces (axial, shear, bending, and torsional) induced from seismic, wind, or blast loads. Under the above actions, higher joint shear forces and/or force gradient in the beam longitudinal rebars (due to moment reversals) are generated typically in a joint panel, resulting in either joint shear failure or rebar bond failure followed by the sudden loss of load resistance. This behavior ultimately may lead to the brittle collapse of the entire building frame, as shown in Figure 5, with low energy dissipation [16]. Therefore, to enhance safe and economical design practices, fundamentals of joint behavior under various loading scenarios must be fully understood.

Figure 5. 2016 Ecuador earthquake - building damage (Juan Cevallos/Getty Images) [17]

Understanding of structural damages based on the post-earthquake reconnaissance studies by [6], [13], [20], [44], [45] have confirmed the innate weaknesses of pre-1976 structures due to the following deficiencies identified at or near the joint panels of buildings: inadequate shear (transverse) reinforcement in a joint panel, ineffective confinement of beam or column plastic hinge regions, poor anchorage detailing for beam reinforcement extending into the joint panel,

inadequate column longitudinal and shear reinforcement near a joint, insufficient lap splice length of column rebars near the joint panels, and inferior quality construction materials when compared to existing standards. The highlighted deficiencies, especially in the joint region, have either led to partial damages (i.e., shear or rebar bond failure) or complete collapse (i.e., soft-story failures) of buildings.

2.3 Joint Definition and Classification

ACI 352R-02 defines a beam-column joint as “that portion of the column within the depth of the deepest beam that frames into the column.” Based on the geometry and arrangement in a framed system, ACI 352R classifies joints in three categories, namely interior, exterior, or corner joints, as shown in Figure 6. This classification is essential because of the fundamental differences in reinforcement anchorage mechanisms of the interior, exterior, and corner joints. Depending on the number of transverse beams meeting at a joint and the presence of columns above and below the joint panel, joints are further classified, as shown in Figure 7. Joints can be considered elastic or inelastic depending on their structural behavior [18]. In a framed system, if plastic deformations occur either in a beam or column or both without cracking in the joint panel, then its behavior is considered as elastic. On the other hand, if a joint panel undergoes inelastic deformation (e.g., seismic event) without significant cracking while forming a plastic hinge at a distance from the beam or column face, then the joint is considered as inelastic. Based on the high vulnerability of exterior joint damages under seismic activities, the author chose to investigate 2D exterior joints without transverse (shear) reinforcement in the joint panel for carrying out this research.

Figure 6. Joint classification based on the arrangement in a framed system [19]

Figure 7. Joint sub-classification based on the arrangement of beams and columns [20]

Note: A typical building frame has a slab contributing to the joint performance. For simplicity, the slab was excluded in Figure 7.

2.4 Force Transfer Mechanism in 2D RC Exterior Joints

Figure 8. The force-transfer mechanism in a most practical case of RC exterior joint [19]

As shown in Figure 8, when an exterior joint is subjected to seismic forces, the column bending moments of high magnitudes having opposite signs are generated on either side of a joint panel, causing enormous joint shear forces at the interface. To satisfy the equilibrium of forces in a joint panel, the beam reinforcement has to be in compression on one face and tension on the other, causing force gradient, as shown in Figure 8.

2.5 Assessment of Joint Shear Strength

The resultant forces acting on a joint panel as a consequence of applied beam and column loads are shown in Figure 9.

Figure 9. Resultant forces on a 2D exterior joint under beam and column loads [21]

Based on the equilibrium of forces, the horizontal shear force at the mid-height of a joint panel can be computed [21]:

$$V_{jh} = T - V_{col} \quad (2.1)$$

Where

V_{col} : horizontal column shear force

T : tensile force in the beam steel reinforcement can be computed as:

$$T = \frac{Pl_b}{jd} \quad (2.2)$$

Where

jd : the distance between tensile and compression force resultants at the beam-joint interface

The horizontal column shear force can be computed using:

$$V_{col} = \frac{P(l_b + 0.5h_c)}{l_c} \quad (2.3)$$

The nominal horizontal shear stress at the mid-height of a joint panel can be computed using:

$$v_{jh} = V_{jh}/b_j h_c \quad (2.4)$$

Where

b_j : effective width of the joint panel,

h_c : depth of the column.

2.5.1 Computation of principal stresses in a joint panel

Figure 10. Diagonal stress fields in an exterior joint [22]

When an exterior joint is subjected to seismic excitation, two types of stress fields, namely diagonal tension and compression, are induced in a joint panel, as shown in Figure 10.

Using Mohr's circle approach as proposed, principal compressive and tensile stresses at the mid-height of a joint panel can be computed [23]:

Principal compressive stress:

$$p_c = \frac{-f_a}{2} - \sqrt{\left(\frac{f_a}{2}\right)^2 + v_{jh}^2} \quad (2.5)$$

Principal tensile stress:

$$p_t = \frac{-f_a}{2} + \sqrt{\left(\frac{f_a}{2}\right)^2 + v_{jh}^2} \quad (2.6)$$

Joint shear stress:

$$v_{jh} = p_t \sqrt{1 + \frac{f_a}{p_t}} \quad (2.7)$$

The nominal (average) axial compressive stress on the column at the mid-height of a joint panel can be computed using:

$$f_a = N/b_j h_c \quad (2.8)$$

Where

N : Axial compressive load on the column

The principal tension stress limit states proposed by various authors [51] and [52] based on experimental evaluations of 2D exterior joints without transverse reinforcement, are summarized in Table 2. However, [23] in their publication highlighted that the principal tensile stress approach could yield conservative results as additional shear resistance beyond diagonal tensile cracking is provided by the diagonal compression strut developed in the joint panel.

Table 2. Principal tensile stress limit states based on the available literature

Joint anchorage	Principal tensile stress limit states		
	First crack	Peak load	Reference
$0.29\sqrt{f'_c}$	$0.42\sqrt{f'_c}$	[24]
$0.29\sqrt{f'_c}$	$0.29\sqrt{f'_c}$	[24]
$0.20\sqrt{f'_c}$	$0.20\sqrt{f'_c}$	[25]

2.5.2 Strut mechanism

Figure 11. Strut mechanism of shear force transfer in an exterior joint [26]

While considering force transfer mechanisms, exterior joints without transverse (shear) reinforcement in the joint panel need particular focus because of the limited compressive force developed on the free end of the joint (exposed) face due to the absence of transverse beams in one or more directions. Therefore, a node formation is essential to develop a strut mechanism for resisting joint shear. The node formation can be achieved by providing a 90° hook to the beam longitudinal reinforcement bent either into or away from the joint panel. In the case of beam reinforcement bent away from the joint panel, the strut will not be stable due to a lack of confinement from the bend radius, leading to premature failure.

On the other hand, joints with beam reinforcement bent towards the panel, exhibits diagonal strut formation between the node along the bent radius and other node formed due to compressive forces on the diagonally opposite end. The joint panel continues to resist shear until significant tensile stresses are developed, at which stage the strut loses its compressive strength, resulting in concrete crushing. Under specific span-to-depth ratios, the connection (beam + column + joint) behavior can also lead to shear compression failure.

2.6 Capacity Design of Joints

Under extreme seismic forces, joints of a moment-resisting frame exhibit higher shear forces than the members framing into the joint, causing inelastic joint deformations (perhaps with higher ductility). Besides, due to load reversals, exterior joints with longitudinal beam bars having 180° or no end hooks are required to develop higher rebar bond stresses to resist the force gradient in the joint panel. Inadequately designed joints structural capacities and anchorage can become weak links and cause catastrophic shear or rebar bond failure. To avoid catastrophe by maximizing the ductility and energy absorption of the overall structure, it is necessary to follow the capacity based design philosophy and make the joints stronger than the members framing into the joint. An in-depth discussion on the deterministic capacity design sequence of structures is provided by [18] in their textbook for further review if needed.

Figure 12. Joint shear/rebar bond deterioration failure in exterior joints [27]

2.7 Parameters Influencing the Behavior of RC Exterior Joints

As identified in section 2.2, a broad spectrum of deficiencies can be found in non-ductile reinforced concrete structures built before 1976. However, the ones pertinent to exterior joints have been identified and discussed in this section. Based on several reconnaissance studies performed by researchers, following deficiencies were found in exterior joints leading to a sudden loss of structural integrity [28]: (i) minimum joint transverse reinforcement (stirrups), (ii) low strength concrete (~ 3500 psi or lower), (iii) beam longitudinal bars bent into the joint with inadequate development length for complete force transfer, and (iv) weak-column and strong-beam design. Hence, the review below highlights the behavior of such joints under the influence of critical parameters governing the failure behavior:

2.7.1 Column axial compressive load

[29] investigated the effect of low axial load by testing joints with hooked bars and found that the influence of axial column load is almost negligible. [30] made similar conclusions, but also identified that the axial column load influences the joint failure modes. On the contrary, [40], [58]-[60] showed that specimens with higher axial loads exhibited increased joint shear capacity due to improved diagonal tension resistance in a joint failure scenario. Further, [31] also reported that specimens with low axial load ($\leq 0.1f'_cA_g$) exhibited 1.5 times more ductility in comparison to specimens with high axial load ($0.25f'_cA_g$). As shown Figure 13, no substantial influence on joint shear strength can be noted when specimens are subjected to an axial load less than $(0.20 - 0.25)f'_cA_g$. Moreover, the exterior joints under seismic loads are prone to load reversals, where the columns do not remain under constant axial loads. This behavior can alter the joint failure mechanism and can jeopardize structural integrity [27], especially in terms of energy absorption.

Figure 13. Effect of column axial load on joint shear strength [32]

2.7.2 Concrete compressive strength

[33] studied joint responses of 48 exterior joints under cyclic loading. Twenty-seven of their test specimens failed through yielding of beam rebar followed by joint shear failure (BJ) while the remaining 21 failed in joint shear failure without rebar yielding (J). The database was grouped using the following criteria: (i) no out-of-plane members (i.e., transverse beams and/or slabs), (ii) no eccentricity between beams and columns (i.e., the beam centroid coincides with the column centroid), (iii) conventional types of reinforcement anchorage with hooked bars (headed bars were excluded) and (iv) minimum amount of joint stirrups necessary for joint confinement. In general, the damage of a joint was caused by concrete cracking in or very close to a joint under diagonal tension, reinforcement yielding, and/or concrete crushing, which are identified as points A, B, and C, respectively, (see Figure 14). In case of exterior joints with “BJ” failure, the first stiffness change (Point A) occurred at the initiation of diagonal cracking [63] and [64] within the joint panel, while the second stiffness change (Point B) occurred at rebar yielding before concrete crushing

(Point C). It was also reported that for specimen with “J” failure, no significant stiffness change was observed around Point B as there was no rebar yielding. The concrete compressive strength for the database collected by [33] varied from ~ 3 to 13 ksi, and the joint shear stress was found to be proportional to the square root of the concrete compressive strength ($\sqrt{f'_c}$), at points B and C. The New Zealand code [34], however, limits the compression (strut) stress in a joint panel to $0.2f'_c$ which is induced from beam-column shear force transfer to avoid concrete crushing failure (see Figure 14). [66] and [67] suggested that principal compression stress be limited to $0.3f'_c$.

Figure 14. Story shear vs. joint shear strain [33]

2.7.3 Joint aspect ratio

Three different variations of joint aspect ratios (1.0, 1.5, and 2.0) were evaluated by [35] on exterior joints without transverse reinforcement within a joint. As shown in Figure 15, joint shear strength was found to be inversely proportional to the joint aspect ratio [35]. [33] examined the influence of joint aspect ratio based on joint shear reinforcement with beam rebar yielding (BJ) and without beam rebar yielding (J). By increasing the ratios of beam height to column depth (h_b/h_c) from 1 to 1.6, and beam width to column width (b_b/b_c) from 0.56 to 1, [33] reported: (i) slight reduction of joint shear stresses for J failure, and (ii) little influence on joint stresses and strains for BJ failure.

Figure 15. Effect of joint aspect ratio [32]

2.7.4 Presence of slab and transverse beam

[36] tested 2D exterior joints to investigate the influence of a slab and transverse beam on joint shear strength under uni- and bi-directional quasi-static loads and varying axial load. Test specimens were created to replicate pre-1976 construction including (i) plain round steel bars, (ii) 180° end hook anchorage for beam longitudinal reinforcement, and (iii) no joint transverse reinforcement. Their 2D study noted that the presence of slab at a joint significantly increased the beam yielding moment on the tension side while the confinement effect from the transverse beam (and slab) resulted in higher displacements before failure (i.e., better ductility). The additional ductility offered by the slab can be computed by arriving at an equivalent slab stiffness through finite element modeling. The equivalent stiffness thus obtained can be added to existing beam properties to arrive at a joint design procedure that is in-line with designs without a slab. Furthermore, it was also reported that for 3D specimens under bi-directional loading with axial load variations exhibited severe damage and rapid strength deterioration even with partial confinement induced by transverse beams [36].

2.7.5 End hooks of beam reinforcement

[29] performed tests on exterior joints with 90° and 180° hooks to evaluate their behavior under monotonic loading and concluded that the anchorage failure of hooked bars is primarily caused by concrete compression around the hook from stress concentration effect. [36] tested joints with slab and transverse beams and revealed that specimens with 180° hook (small diameter) resulted in the loss of bond for beam reinforcement, exhibiting pinching of hysteresis loop with limited energy dissipation. It was observed that for specimens with bars bent into the joint, the forces were resisted primarily by the concrete compression strut (after initial diagonal cracking) until it is ultimately weakened by diagonal tension strains. Among the above-described beam end hook configurations, [52], [70]-[74] concluded that joints with 180° hooks exhibited the worst performance with low energy dissipation due to the lack of concrete confinement.

2.7.6 Joint transverse (shear) reinforcement

[37] found that the confinement effect from joint transverse reinforcement positively influenced the concrete shear resistance. [54] and [70] carried out experimental tests on 3D joints with plain round bars and 180° hooks and no transverse reinforcement in the joint. Based on this study, it was concluded that the exterior joints with no transverse reinforcement exhibited lower shear strength with brittle behavior. Similarly, [38] experimentally evaluated four exterior joint specimens with transverse beams and floor slabs under quasi-static loading to evaluate non-ductile behavior in existing RC buildings, and found joint shear failure without beam-hinging.

2.7.7 Failure modes

Joint strength is often characterized based on the concrete compressive strength, and steel rebar (yield) strength. As reported in the literature, exterior joints typically exhibit two failure modes: (i) joint shear failure without beam reinforcement yielding, and (ii) joint shear failure after beam reinforcement yielding. [39] collected an extensive database of exterior reinforced (i.e., with joint stirrups) (189 tests – without joint transverse (tie) reinforcement, 14 tests – with one transverse beam, and 18 tests – with two transverse beams) and exterior unreinforced (i.e., no joint stirrups) (83 tests – without transverse beam, 29 tests – with one transverse beam, and nine tests – with two transverse beams) joint responses to develop a reliable joint shear strength determination model statistically. The database included: (i) concrete compressive strength, (ii) joint transverse reinforcement ratio, (iii) design joint shear stress, (iv) in-plane and out-plane geometry (i.e., presence of transverse beams in one or more directions), (v) joint panel geometry factors such as beam depth to column depth ratio and beam width to column width ratio, (vi) joint eccentricity (i.e., the beam centroid does not coincide with the column centroid), (vii) axial column load, (viii) ratio of “provided” to “required” column depth (or anchorage length) to beam rebar diameter, (ix) column-to-beam nominal moment ratio, and (x) ratio of intermediate column reinforcement strength to design joint shear demand. Based on [39] study, it was concluded that joint shear failure without rebar yielding was the predominant failure for both 110 of reinforced (with stirrups) and 57 of unreinforced (without stirrups) exterior joints. Joint shear failures without rebar yielding exhibit brittle failure due to diagonal tension, compromising the overall structural integrity [28].

Table 3. Summary of exterior RC joint failure modes [39]

Joint Type	Failure mode	Number of transverse beams		
		None	One	Two
Reinforced (with joint stirrups)	JS	110	6	-
Reinforced (with joint stirrups)	BJS	79	8	18
Unreinforced (without joint stirrups)	JS	57	20	-
Unreinforced (without joint stirrups)	BJS	26	9	9

2.8 Summary

This chapter reviewed the performance of pre-1976 non-ductile concrete frame joints and outlined the potentially catastrophic failure mechanisms. Besides, crucial parameters influencing the joint behavior were highlighted. The key learning points from this review are as follows:

1. Under seismic actions, due to moment reversals, higher joint shear forces are developed with significant force gradient across the joint panel, resulting in either shear or rebar bond failure.
2. Depending upon the detailing of beam longitudinal rebar extending into the joint panel (i.e., either bent away or towards the joint core), exterior joints typically exhibit two failure modes: (i) joint shear failure without beam reinforcement yielding, and (ii) joint shear failure after beam reinforcement yielding. Typically joint shear failures without rebar yielding are brittle and lead to catastrophic failure (which is collapse).
3. Joints with beam longitudinal rebar having 180° hooks did not form a compression strut in the joint panel due to inadequate confinement effect from beam longitudinal reinforcement, resulting in a premature rebar bond failure with low energy dissipation.
4. Under cyclic loading, the stress concentration effect around the 90° bend radii significantly deteriorated the bond between concrete and rebar surface, thus causing anchorage failure.
5. For specimens with longitudinal beam bars bent into the joint, the forces were resisted primarily by the concrete compression strut (after initial diagonal cracking) until it is ultimately weekend by diagonal tension strains.

6. Column axial loads up to $(0.20 - 0.25)f'_c A_g$ exhibited no substantial influence on joint shear strength.
7. The presence of transverse beams and slab at a joint significantly increased the beam yielding moment while resulting in higher displacements before failure.
8. The joint aspect ratio (h_b/h_c) varied between 1.0 and 1.6 had no significant effect on joint strength.
9. Higher f'_c coupled with specific span-to-depth ratios, can lead to shear compression failure.

3. RC Exterior Joint Repair with FRP Composites: A Brief Review

3.1 Introduction

Over the past 25+ years, experimental research has been focused on strengthening deficient joints (identified in section 2.2 of the previous chapter) through the use of fiber-reinforced polymer (FRP) composites. This chapter provides a review of such experimental studies based on the influence of parameters as identified and elaborated herein. The following sections of this chapter provide a brief introduction on FRP composite applications in repair and rehabilitation of infrastructure systems, leading to discussions on FRP research in joint strengthening. The scope of this chapter is limited to 2D exterior joints.

3.2 FRP Composites in Repair and Rehabilitation of Infrastructure

FRPs were first introduced in the 1940s for applications in defense and aerospace industries [40] and now emerging with numerous opportunities in the infrastructure industry because of their high strength to weight ratio, non-corrosiveness, and excellent durability. FRPs are regarded as disruptive materials to reinforce existing structures that have either deteriorated due to environmental influences or under-designed for external forces. In particular, FRPs are well known for strengthening structural components of seismically deficient buildings and bridges [79]-[81] due to their cost-effectiveness and ease of installation. In general, FRP composites have been employed to repair or rehabilitate:

1. Beams: [82]-[84]
2. Columns: [85]-[87]
3. Joints: [88]-[90]
4. Slabs: [91]-[93]
5. Corroded members: [94]-[96]
6. Fire/heat damaged members: [97]-[99]
7. Vehicular impact damaged members: [100]-[102]

3.3 FRP Composites in Joint Strengthening

One of the critical aspects of structural strengthening to resist extreme forces is to improve joint performance. Joint performance can be improved by bonding multiple sheets/strips of FRPs using epoxy or other adhesives across and around joints of in-service concrete structures. Many researchers adopted this technique to repair deficient joints and assess the performance under cyclic loads [17], [54], and [55]. A research study by [41] on repair of damaged RC joints using FRP laminates concluded that the repair scheme increases the strength of joints beyond its original design capacity. Another study by [42] on retrofit of beam-column joints using FRP overlays revealed significant improvements in strength and moderate increase in ductility. [43] conducted their work by experimenting with FRP retrofitted joints and concluded the increase in moment capacity of the joints. [44] evaluated RC joints rehabilitation with different FRP composite configurations, including U- and X- wraps. This study by [44] proved that FRP repairs can change the non-ductile joint shear failure to a ductile flexural failure mode. [45] experimented by strengthening several scaled joints using various FRP configurations and confirmed the increases in the strength, stiffness, and energy dissipation by up to 100%. In addition, [45] concluded that the failure of joints is governed by the partial or complete debonding of FRPs from the substrate. Rehabilitation of beam-column joints by [46] proved the potential of FRPs in restoring the strength of a damaged joint and the ability to substantially increase the structural capacities by preventing the failure in the joint panel. Web-bonded FRP application to repair RC beam-column joints evaluated by [47] was found to be successful in restoring the strength, stiffness, and ductility of joints by relocating the plastic hinge away from the joint panel into the beam.

3.4 Previous Experimental Studies on FRP Composites in Joint Strengthening

The following section provides a review of experimental work on FRP strengthening of RC exterior joints conducted by various researchers based on the parameters investigated through their research.

3.4.1 Effectiveness of FRP wraps on joint shear strengthening

[48] conducted an experimental study to evaluate the effectiveness of carbon fiber-reinforced polymer (CFRP) composite wrapping to strengthen joints that exhibit deficiencies typically encountered in Turkish construction. These deficiencies include low strength concrete (3.6 ksi or lower), structural type I steel (yield strength of 31.9 ksi), plain reinforcements (no lugs over steel bar surfaces), insufficient transverse reinforcement, lap-splicing lengths and beam sections higher than the column sections meeting at a joint (i.e., moment capacity of beams is higher than that of columns). Identified deficiencies were based on a comprehensive survey conducted on more than 50 buildings built between the 1950s and 1990s. Four 2D exterior joints were experimentally investigated by subsequently developing CFRP wrapping schemes based on the failure mode exhibited by the previous specimen. The control test specimen, including reinforcement details, are shown in Figure 16, and the CFRP repair methods are identified in Tables 4 and 5.

Figure 16. Steel reinforcement detailing of control and retrofitted test specimens [48]

Table 4. CFRP repair steps adopted by [48]

Step No.	CFRP application	Description of strengthening procedure
Step 1		Four horizontal holes of 12 mm in diameter were drilled across the beam to insert the anchorages.
Step 2		Longitudinal CFRP sheet was applied outside the column surface for eliminating lap splice defect and increasing moment capacities of column members.
Step 3		Top and bottom L-shaped CFRP sheets were applied to inside of the column face extending toward the beam surface for eliminating lap splice defect and increasing the resistance of joint region.
Step 4		One layer of diagonal CFRP fibers were placed on joint region to prevent diagonal crack propagation along the joint surface. Same application was done for opposite direction, resulting X-shaped CFRP orientation.
Step 5		Three layers of 150 mm wide CFRP patches were used for eliminating the rupture of CFRP materials in the joint region.
Step 6		Top and bottom portion of the column was wrapped for eliminating early debonding of already-applied L-shaped and longitudinal CFRP on column surface.
Step 7		Beam member is wrapped to prevent buckling of beam longitudinal reinforcement.
Step 8		To prevent the debonding, CFRP sheets were anchored to the already-applied beam wrapping by fan-shaped CFRP anchors.

Table 5. Specimen wise strengthening methods [48]

Strengthening method	Step 1	Step 2	Step 3	Step 4	Step 5	Step 6	Step 7	Step 8
	Holes	Longitudinal	L-shape	Diagonal	Patches	Column wrap	Beam wrap	Anchorage
Method 1 (TR-5-FRP-1)	—	—	—	1 layer	—	—	1 layer 500 mm	—
Method 2 (TR-5-FRP-2)	—	1 layer	1 layer	1 layer	—	2 layers	3 layers 300 mm	—
Method 3 (TR-5-FRP-3)	4 holes	1 layer	1 layer	1 layer	3 layers	2 layers	3 layers 300 mm	150 mm

The experimental specimens were tested as per the schematic shown in Figure 17 by applying a constant ($0.4f'_cA_g$) axial load and reversed-cyclic lateral load on the column section.

Figure 17. Test setup of [48]

The experimental results and failure behavior of test specimens evaluated by [48] are summarized as follows:

Specimen TR-5-Control:

- The first crack was identified at 0.35% (of specimen column height) drift, initiating at the top corner of the beam-column joint region.

- The maximum load levels of 7.2 kip in the push and 10.6 kip in pull directions were reached at 0.75% drift.
- Test specimen failed by diagonal shear cracking in the joint ($a/d \sim 3.7$).

Specimen TR-5-FRP-1:

- At 1.4% drift, buckling of CFRP sheets was observed near the top and bottom faces of the beam towards the back face of the column.
- At 2.75% drift, CFRP sheets ruptured at the corner of beam-column joints.
- Upon removal of CFRP sheets, buckling of column longitudinal rebars was noticed.

Specimen TR-5-FRP-2:

- At 1.4% drift, initiation of rupture was observed in diagonal CFRP sheets at the interface of the beam-column joints. Cracks, however, did not penetrate to the center of the joint core as their growth was arrested by the additional three layers of CFRP sheets, as illustrated in step 5.
- The maximum load levels of 10.3 kip in the push and 13.6 kip in pull directions were reached at 2.75% drift. At this stage, in addition to de-bonding and buckling of CFRP sheets on the back face of the column, cracks on the beam widened, leading to concrete crushing.
- At 3.5% drift, beam hinging occurred.

Specimen TR-5-FRP-3:

- At 2.2% drift, initiation of rupture was observed in diagonal CFRP sheets at the interface of the beam-column joints. As in the case of specimen TR-5-FRP-2, the crack growth was arrested by an additional three layers of CFRP sheets. In this case, the spalling of concrete cover occurred at the beam bottom location.
- The initiation of de-bond at the tip of CFRP diagonal fibers was observed at 2.75% drift. However, due to the anchorage of CFRP diagonal fibers, the de-bond did not progress towards the joint.

- At 3.5% drift, beam hinging occurred, followed by a buckling of beam bottom reinforcement.

Following are the key learning points based on the experimental evaluations performed by [48]:

- The absence of joint transverse (shear stirrups) reinforcement in the control specimen (TR-5-Control) has led to joint shear failure at a low lateral load level.
- Lack of joint confinement in the control specimen has led to faster stiffness degradation.
- Diagonal CFRP sheets bonded to the joint panel orienting fibers parallel to the principal planes and perpendicular to the anticipated crack direction prevented the formation of joint shear cracks. This arrangement of fibers resulted in a ductile behavior with decreased stiffness degradation at the end of each loading cycle.
- In the case of CFRP strengthened specimens, the lateral load resisting capacity was enhanced by 50-100%.
- In all the strengthened specimens, CFRP wrapping could not prevent the initiation of rupture at the beam-column interface, indicating the concentration of stresses around the corner.
- Beam anchor has prevented the de-bonding of sheets from the substrate, thus forcing plastic hinge formation away from the joint core.

3.4.2 Effectiveness of FRP anchors in preventing debond

[49] tested RC strong beam-weak column joints to assess the viability of externally bonded reinforcement on grooves (EBROG) method coupled with the CFRP fan anchorage system, as shown in Figure 18, to resist seismic loads without surface de-bonding of FRP. To accomplish this task, five half-scale 2D exterior beam-column joints with inadequate joint transverse reinforcement were tested under constant axial and reversal-cyclic lateral loads. The test specimen configuration and setup are shown in Figures 19 and 20.

Figure 18. FRP fan anchor [49]

Figure 19. Test specimen details [49]

Figure 20. Test setup of [49]

Descriptions of four specimens retrofitted using the EBROG method coupled with CFRP fan anchors are provided in Table 6 and shown in Figures 21 through 24.

Table 6. Description of retrofit schemes [49]

No	Specimen	f'_c (MPa)	Column strengthening		Joint strengthening		Beam strengthening		
			ply No.	width (mm)	ply No.	width (mm)	ply No.	width (mm)	Length (mm)
1	CS	35	–	–	–	–	–	–	–
2	RS-1V-1X	35	1	180	1	200	–	–	–
3	RS-1V-2X	34	1	180	2	360	–	–	–
4	RS-1V-2X-2H.I	34	1	180	2	360	2	180	150
5	RS-1V-2X-2H.II	37	1	180	2	360	2	180	300

Figure 21. Rehabilitation scheme of RS-1V-1X [49]

Figure 22. Rehabilitation scheme of RS-1V-2X [49]

Figure 23. Rehabilitation scheme of RS-1V-2X-2H.I [49]

Figure 24. Rehabilitation scheme of RS-1V-2X-2H.II [49]

Test results on the experimental evaluations of [49] are summarized in Table 7.

Table 7. Summary of experimental results by [49]

Specimen	Peak load (kN)		Average (kN)	Increase (%)	Drift ratio at peak load (%)		Ultimate load (kN)		Failure mode
	Push (+)	Pull (-)			Push (+)	Pull (-)	Push (+)	Pull (-)	
CS	28.90	27.96	28.43	-	1.75	1.40	24.53	22.18	column flexural hinging alongside joint shear failure
RS-1V-1X	42.27	43.52	42.89	51	2.75	2.75	28.06	31.84	FRP rupture followed by joint shear failure
RS-1V-2X	47.08	46.25	46.67	64	2.20	2.20	32.85	29.27	FRP rupture followed by joint shear failure
RS-1V-2X-2H.I	45.32	44.00	44.66	57	5.20	4.50	45.92	42.89	beam flexural hinging
RS-1V-2X-2H.II	48.91	49.63	49.27	73	5.20	5.20	48.31	50.23	beam flexural hinging

Following are the key learning points based on the experimental evaluations performed by [49]:

- EBROG method coupled with CFRP fan anchors was found to be efficient in preventing the de-bonding of CFRP sheets from its substrate. Based on the average values of peak loads in the push and pull directions, it was found that specimens RS-1V-1X, RS-1V-2X, RS-1V-2X-2H.I and RS-1V-2X-2H.II exhibited 51, 64, 57, and 73% increase in peak load,

respectively. This increase in peak loads is attributed to the amount of FRP bonded to the joint face and the effectiveness of EBROG in preventing debond failure.

- The maximum increase in ductility and energy absorption capacities of specimens strengthened using EBROG and anchor method was found to be 139% and 144%, respectively. This improvement in ductility and energy absorption was attributed to the flexural hinge formation in the beams away from the column face.
- This method of reinforcing joints is heavily dependent on the amount and orientation of CFRP sheets bonded directly to the joint faces. Therefore, it is challenging to employ this technique and rely on similar capacity enhancements in actual building joints with transverse beams and slab.

3.4.3 Influence of FRP confinement on bond strength of hooked bars

Twelve normal-strength concrete (28 and 31.4 MPa) beam-column joints wrapped with carbon fiber reinforced polymer (CFRP) sheets were tested by [50] to evaluate the effectiveness of wrap confinement on bond strength of hooked bars. Of these twelve specimens, only six specimens (28 MPa) with beam bars enclosed between column longitudinal bars are considered for review as they closely represent the arrangement in RC Joints. As shown in Figures 26 through 28, joints of three different reinforcement bar sizes with corresponding anchorage lengths (shorter than l_{dh} specified by ACI 318-05) are considered for evaluations under control and CFRP bonded (see Figure 29) conditions. The specimens were monotonically loaded in increments of 10 kN perpendicular to the longitudinal axis of the beam.

Figure 25. Experimental test setup [50]

Figure 26. Details of test specimen C16N [50]

Figure 27. Details of test specimen C25N [50]

Figure 28. Details of test specimen C32N [50]

Figure 29. CFRP wrap details for C16N, C25N, and C32N [50]

Summary of experimental results by [50] is provided in Table 8.

Table 8. Summary of experimental results [50]

Specimen	Tensile bar	CFRP sheets	Maximum load (kN)	Displacement measured under beam loading point (mm)
C16N	16	No	60.5	12.6
C16N-F	16	Yes	67.2	15.8
C25N	25	No	112.8	19.8
C25N-F	25	Yes	144.5	30.2
C32N	32	No	146.5	20.0
C32N-F	32	Yes	166.1	27.7

The experimental results and failure behavior of test specimens evaluated by [50] are summarized as follows:

- In the case of control specimens, the first crack originated at the joint corner on the tension face having an inclination of 20° - 30° with the horizontal. As the loading progressed, crack extended to the opposite corner of the joint interface. Following this, diagonal cracks appeared on the column at 10 cm below the tension side joint corner and propagated along the tail anchorage. The final failure of specimens occurred due to the high compressive stress concentrations at the inner radius of the bend resulting in the crushing of joint core concrete.
- Specimens strengthened with CFRP exhibited two different failure modes. (i) de-bonding or peeling off of CFRP sheets from the vertical face of the beam and (ii) tearing or shearing failure of CFRP sheets at the joint interface.

Following are the key learning points based on the experimental evaluations performed by:

- Improvement in load-carrying capacity of joints strengthened with CFRP sheets was found to be approximately 11%, 28%, and 11% for C16N-F, C25N-F, and C32N-F specimens, respectively when compared to their corresponding un-strengthened specimens.
- Both control and CFRP strengthened cases exhibited tension crack initiation at the interface of beam-column joints indicating the influence of stress concentrations.
- In the case of CFRP strengthened specimens, failure was either by CFRP de-bonding or tearing off of the CFRP sheets parallel to the fiber direction. The first case signifies the necessity of providing anchors for preventing such de-bonds, and the latter case reveals the need for using bi-directional fabric to avoid tearing off failure.
- FRP confinement effect on the bond strength of hooked bars attained by bonding CFRP sheets in a discontinuous manner (i.e., the beam or column sheets when bonded only to the vertical faces without the continuity of fabric between those two faces as a U-wrap or 360° wrap) is insignificant.
- Resting the column base directly on the strong floor, as shown in Figure 25, will generate reactions throughout the length of the column base, causing additional concrete

confinement that may delay the crack formations or avoid side-splitting failure of cover concrete along the tail portion of anchorage reinforcement. Furthermore, this behavior is not reflective of actual beam-column joints in a building.

3.4.4 Influence of surface preparation on rehab effectiveness

To assess the influence of CFRP composites in improving the joint strength, [51] tested 1/3-scale concrete joints by considering two types of surface preparation methods, namely wire brush and water jet (276 MPa up to 3 mm deep) coupled with structural adhesive. Dimensions of test specimen along with reinforcement details considered for evaluation and test setup are shown in Figures 30 and 31, respectively. Specimens were tested in an inverted position as opposed to their orientation in a typical building frame.

Figure 30. Test specimen reinforcement details and dimensions [51]

Figure 31. Test setup of joint specimen [51]

CFRP sheets were used to reinforce the deficient joints with the configurations shown in Figure 32 and listed in Table 9. The specimens were tested by applying a quasi-static cyclic lateral load on the column. At the end of testing, control specimens failed by shear damage (diagonal cracking initiated when the diagonal tensile stresses attained the concrete's tensile strength) in the joint region at low lateral loads. In the case of strengthened specimens, progressive delamination of CFRP sheets occurred starting at the face of the beam and progressing towards the joint. The strains measured on CFRP at the time of delamination varied from 0.1% to 0.35%, indicating the premature separation of CFRP from the concrete surface way before reaching its ultimate strain of 1.0%. The lower strain percent observed at delamination can be attributed to the use of inferior quality construction materials or inadequate surface preparation. The CFRP wrapped specimens exhibited failures similar to control specimens but at significantly higher load levels, as listed in Table 9, proving the effectiveness of CFRP wrapping in improving the joint shear strength.

Figure 32. CFRP wrap layout [51]

Table 9. CFRP wrapping schemes and test results [51]

f'_c	Specimen	Surface preparation	Cure temperature	CFRP Sheets ^a	L ₁	L ₂	L ₃	Push (kN)	Pull (kN)
20 MPa	1b	N.A.	N.A.	No	N.A.	N.A.	N.A.	~129	~129
	2b	N.A.	N.A.	No	N.A.	N.A.	N.A.	129	142
	3	Wire brush	350°F	45°	0	178	0	182	142
	4	Wire brush	350°F	±45°	0	178	0	187	187
	5	Wire brush	Room	45°	0	229	0	161	154
	6	Wire brush	Room	45°	432	229	406	158	140
	7	Wire brush	Room	45°	432	229	406	156	143
	8	Wire brush	Room	±45°	0	203	0	187	187
	9	Wire brush	Room	±45°	406	229	203	216	200
34 MPa	10b	N.A.	N.A.	No	N.A.	N.A.	N.A.	126	116
	11b	N.A.	N.A.	No	N.A.	N.A.	N.A.	148	98
	12	Water jet	Room	±45°	406	203	406	184	217
	13	Water jet	Room	±45°	406	203	406c	204	196
	14	Water jet	Room	±45°	406	203	406c	229	224

^aCFRP layout on each vertical face of beam, ^bbaseline specimens, ^conly on three sides.

Following are the key learning points based on the experimental evaluations performed by [51]:

- Surface preparation of test specimens using water jet coupled with structural adhesive was found to be more efficient in enhancing the shear strength of joints with no shear reinforcement and longitudinal bars bent away from the core. However, in practice, it is unlikely to have a beam without any shear reinforcement.
- Although CFRP strengthened specimens exhibited joint shear failure patterns similar to control specimens, the failure was contained.
- Capacity enhancements in specimens strengthened with a single layer of CFRP having 45° orientation were somewhat inconsistent compared to specimens strengthened with balanced ±45° orientation. Specimen with balanced ±45° orientation exhibited 31% higher load carrying capacity in comparison to a single layer of CFRP having 45° orientation.
- Delamination of CFRP initiated from the corners of the joint panel, indicating the high importance of minimizing stress concentrations and the need for additional anchorage through confinement or other mechanisms.

- The increase in nominal principal tensile stress developed in the joint panel was found to be in the range of (0.17 – 0.33) MPa for 45° unsymmetrical orientation and (0.62 – 1.21) MPa for symmetrical $\pm 45^\circ$ orientation parallel to the principal planes.
- Elevated temperature cure did not play a role in improving the shear strength of joints implying that the resin was adequately cured at room temperature before testing.

3.4.5 Effectiveness of FRP wrap layup on joint strengthening

[52] focused their research on enhancing the shear capacity of non-seismically designed joints and identifying the efficient CFRP wrapping configuration. Towards this goal, the authors cast seven specimens with no stirrup in the joint region, of which one specimen was evaluated in control (control) condition (see Figure 33) and the remaining six joints under varying CFRP configurations (see Figure 34). The description of CFRP configurations is provided in Table 10.

Figure 33. Details of the non-seismic joint [52]

Figure 34. CFRP configurations for strengthening non-seismic joints [52]

Figure 35. Test setup [52]

Table 10. Description of CFRP configurations [52]

No.	Specimen	T-shape	L-shape	X-shape	Strip
1	NS	None	None	None	None
2	SD	None	None	None	None
3	RNS-1	Both sides	On top and bottom of beam	None	None
4	RNS-2	Both sides	On top and bottom of beam	None	Strip on column
5	RNS-3	None	None	On three surfaces	None
6	RNS-4	None	On top and Bottom of beam	On three surfaces	None
7	RNS-5	Both sides	On top and bottom of beam	None	Strip on column and beam
8	RNS-6	Both sides (two layers)	On top and bottom of beam	None	Strip on column

Specimens were tested in setup, as shown in Figure 35, without applying column axial load.

The experimental results and failure behavior of test specimens evaluated by [52] are summarized as follows:

Specimen NS:

- The first crack (flexural) occurred at the beam-joint interface.
- At 12 mm displacement, joint shear cracks initiated and led to ultimate failure.

Specimen RNS-1:

- At 18 mm displacement, the L-shape CFRP attached to the column inner surface extending on to the beam de-bonded close to the joint region.
- At 24 mm displacement, T-shaped sheets bonded to the face of the joint with portions extending on to the beam, top and bottom column stubs, de-bonded towards the non-retrofitted face of the column causing slippage of the top longitudinal beam bars.

Specimen RNS-2:

- The damage was caused due to de-bonding of CFRP sheets similar to that of specimen RNS-1 but at a higher displacement. The addition of U-strip anchors delayed the de-bonding of CFRP sheets.

Specimen RNS-3:

- At 18 mm displacement, multiple flexural cracks appeared on the beam, especially close to the joint region. These cracks grew as the testing progressed.

- At 36 mm displacement, the spalling of concrete was noticed on the beam surfaces.
- Specimen experienced beam flexural failure with insignificant cracks in the joint region.

Specimen RNS-4:

- The failure behavior was similar to that of RNS-3.
- The addition of L-shaped CFRP sheets delayed the formation of flexural cracks to 24 mm displacement.
- As the test progressed, L-shaped sheets de-bonded close to the joint region but did not progress farther.
- At the time of failure, more cracks were seen in the joint region in comparison to specimen RNS-3, which was attributed to the beam strength and stiffness enhancement due to the addition of L-shaped sheets.

Specimen RNS-5:

- At 24 mm displacement, the L-shape CFRP sheets de-bonded close to the joint region.
- U-shape strips bonded to the beam prevented the de-bonding of CFRP sheets from its surfaces, causing complete detachment of L- and T- shape close to the joint region on the column surfaces.

Specimen RNS-6:

- At 24 mm displacement, the CFRP sheets de-bonded, causing cracks on the non-retrofitted side of the column.
- Although an increase in the number of CFRP layers prevented joint shear failure, it weakened the non-retrofitted face of the column resulting in the spalling of large portions of cover concrete.

Results based on the experimental evaluations performed are summarized in Table 11.

Table 11. Summary of test results [52]

Specimen	NS	RNS-1	RNS-2	RNS-3	RNS-4	RNS-5	RNS-6
Max. positive lateral load, kN	8.56	10.10	9.87	10.06	9.90	9.52	11.27
% increase comparing to NS	-	17.99	15.30	17.52	15.65	11.21	31.66

Following are the key learning points based on the experimental evaluations performed by [52]:

- Joint shear failure in the control specimen occurred due to the lack of transverse reinforcement.
- Bottom anchorage bars bent away from the joint panel did not provide adequate confinement to the joint core, causing brittle failure.
- Bonding discrete layers of L-and T-shape CFRP sheets without anchors has led to premature de-bonding from its substrate.
- The addition of U-strips enhanced the bond performance between underlying sheets and concrete substrate, thereby delaying the initiation of- or preventing- de-bond.
- 45° orientation of fibers (continuous U-fabric covering three surfaces) in the joint region with respect to the beam longitudinal axis, has provided the best performance. This is because the fibers are more or less oriented parallel to the principal planes reinforcing the concrete strut.
- Specimens strengthened with one layer of discrete T- + L-shape sheets and X- pattern exhibited similar strength improvement. However, close to the optimal orientation of fibers and improved confinement in the latter case exhibited more ductile behavior due to beam flexural failure.
- Reinforcing the joint with CFRP sheets in continuous X-pattern has improved the confinement of joint core resulting in 5.3 times ductility over the control specimen.
- Although strengthening with two layers of discrete T- and L- CFRP sheets on either side of a joint improved its strength by 31.7%, it did not provide adequate ductility as compared to specimen with continuous X- pattern.

3.4.6 Effectiveness of joint core confinement through FRP wrapping

To improve the FRP confinement effect on a joint core, [53] proposed a method of bonding concrete covers and wrapping with FRP composites. To evaluate the efficiency of proposed method, four joints without transverse reinforcement were tested under reversed cyclic loading. Reinforcement details of test specimens in control and rehab conditions are shown in Figures 36 and 37, respectively.

Figure 36. Joint steel reinforcement details [53]

Figure 37. CFRP rehab detailing of joint specimens [53]

The experimental results and failure behavior of test specimens evaluated by [53] are summarized as follows:

Specimen T0:

- Diagonal shear cracks developed at the joint center when the specimen reached a peak load of 35.8 kN.
- As the test progressed, bond cracks developed along the column reinforcement. Diagonal cracks grew in size and merged with bond split cracks leading to joint shear failure.
- At the end of the test, concrete was crushed entirely in the joint core.

Specimen TS:

- When the specimen reached a peak load of 77.2 kN, horizontal CFRP around the joint at the beam-column interface ruptured due to the formation of the flexural hinge at the column face.
- As the specimen was further loaded, CFRP at the joint de-bonded causing joint shear failure.
- At the end of the test, large flexural cracks were seen at the beam-column interface along with diagonal shear cracks in the joint area.

Specimen TS-1:

- Specimen TS-1 failed in joint shear due to the rupture of horizontal and vertical CFRP sheets in the joint area.
- When the specimen reached a peak load of 65.8 kN, initiation of CFRP rupture at the top and bottom face of the beam-column interface was noticed.
- As the specimen was loaded further, CFRP sheets started to de-bond in the joint region, following which CFRP sheets around the column ruptured.
- The specimen finally failed due to the complete de-bonding of CFRP sheet, resulting in joint shear failure.

Specimen TS-2:

- As the specimen reached a peak load of 140 kN, no de-bonding or rupture of CFRP was noticed in the joint region. Instead, this wrap configuration led to the formation of the flexural hinge in the beam outside the strengthened portion.
- Finally, specimen TS-2 failed in beam flexure with hinge formation at 250 mm away from the beam-column interface.

Following are the key learning points based on the experimental evaluations performed by:

- Specimen TO, TS, and TS-1 failed in joint shear with an increase in the load-carrying capacity of 84-115% depending on the thickness of CFRP wrap in and around the joint region. Specimen TS-2 failed through the beam hinging with an increase of 140% in load capacity.
- The maximum improvement in displacement ductility was found to be 87% for the TS-2 specimen that exhibited beam flexural failure.
- CFRP strengthened (TS-1) specimen with joint shear failure exhibited only a 33% improvement in displacement ductility.
- Improvement in energy dissipation capacity of TS, TS-1, and TS-2 was found to be 299, 99, and 407%, respectively. It should be noted that hinge formation in the beam irrespective

of whether the final failure was concentrated in the joint or beam has led to significant improvement in energy dissipation.

- Although bonding of concrete covers coupled with CFRP wraps has improved the performance of 2D joint specimens due to confinement of joint core, it is highly challenging to reinforce actual building joints with transverse beams and slab using this technique.

3.4.7 Influence of FRP strengthening on joints with transverse beams

To investigate the influence of FRP strengthening on joints with transverse beams, [45] evaluated 2/3- scale exterior joints under simulated seismic loads. Test specimens were created without stirrups in the joint to replicate poorly detailed RC frame. The geometric dimensions and reinforcement details of test specimens are shown in Figures 38 through 40. shows the FRP strengthening configurations adopted for the test specimens.

Figure 38. Geometric details of test specimen [45]

Figure 39. Steel reinforcement detailing of test specimen [45]

Figure 40. FRP retrofitted specimen configurations [45]

The experimental results and failure behavior of test specimens evaluated by [45] are summarized as follows:

Control specimens C1, C2, and T-C:

- All the control specimens exhibited diagonal cracking of joint panel at a displacement of 10-15 mm, followed by joint shear failure.

- Since the mode of failure is joint shear, bonding of FRP wraps perpendicular to the crack direction will increase the shear capacity of joints.

Specimen F11:

- At 20 mm displacement, the de-bonding of FRP sheets near the joint corners was noticed.
- At 35 mm, a horizontal crack was noticed propagating over the entire length of beam FRP. As a result of the horizontal crack in beam FRP, the tension was generated perpendicular to the fibers in column FRP causing a partial fracture.
- At 40 mm displacement, one side of the beam experienced complete debonding of FRP sheets while other face of the beam experienced de-bonding followed by tensile fracture perpendicular to the fiber orientation in FRP sheets.
- Ultimately, the specimen failed in joint shear.

Specimen F22:

- Similar to F11, the de-bonding of FRP sheets initiated near the joint corners leading to complete detachment of FRP. However, fracture of FRP was not observed in F22.
- Ultimately, the specimen failed in joint shear.

Specimen T-F33:

- At 25 mm displacement, the de-bonding of FRP sheets initiated near the joint corners.
- Between 30-35 mm displacements, a transverse beam experienced splitting crack due to lack of beam stirrups and the presence of a duct to accommodate prestressing rods. The splitting crack progressed towards the joint panel, causing it to fail in shear.

Specimen T-F22S2:

- At -20 mm displacement, partial debonding of FRP was noticed on the tension face and at the joint corners.
- Tensile fracture of beam FRP sheets was noticed near the corners resulting in a horizontal crack initiation at 20-25 mm displacement.

- At 35 mm displacement, the de-bonding of two FRP strips was noticed.
- The test stopped when the displacement reached 55 mm.

Table 12. Summary of experimental results [45]

Specimen	$f'_c (MPa)$	Maximum load (kN)		Energy dissipated (kN.mm)
		Push	Pull	
C1	19.5	31.32	27.13	-
C2	23.7	30.82	31.08	-
Average (C1,C2)	21.6	31.07	29.10	4,995
F11	22.8	42.76	42.44	6,395
F22	27.2	50.04	49.14	7,477
Average (F11,F22)	25.0	46.4	45.79	6,936
T-C	24.6	36.02	33.86	5,873
T-F33	26.0	44.26	44.45	7,695
T-F22S2	22.0	40.07	39.75	6,207

Following are the key learning points based on the experimental evaluations performed by:

- Control joint specimen with a transverse beam has shown a maximum of 16% and 18% improvements in strength and energy dissipation, respectively, over joint without transverse beam.
- Specimens without transverse beam (with a slight difference in concrete strength):
 - Maximum enhancements in strength and energy dissipation for the specimen with a single layer of FRP was found to be approximately 46% and 28%, respectively.
 - Maximum enhancements in strength and energy dissipation for the specimen with two layers of FRP was found to be approximately 69% and 50%, respectively.
- Specimens with a transverse beam (with a slight difference in concrete strength):
 - Maximum enhancements in strength and energy dissipation for the specimen with two layers of FRP and two FRP strips were found to be approximately 17% and 6%, respectively.
 - Maximum enhancements in strength and energy dissipation for the specimen with three layers of FRP was found to be approximately 31% and 31%, respectively.

- Comparing the FRP strength enhancements for joints with transverse beam (T-C Vs. T-F33 or T-F22S2) against the joints without transverse beam (Average(C1, C2) Vs. Average (F11, F22)), following conclusions are drawn:
 - The presence of transverse beam reduced the effectiveness of CFRP sheet strengthening by 54% and 20% in terms of strength and energy absorption capabilities, respectively. In the case of specimen strengthened with CFRP sheets and strips, this reduction was even higher, i.e., 77% reduction in strength and 85% reduction in energy absorption.
- In the case of FRP strengthened specimens, de-bonding always initiated at the high-stress concentration joint corners highlighting the need to redistribute stresses away from the joint corners or provide adequate anchorage.
- Due to the premature de-bonding of FRP sheets from its substrate, especially at joint corners, an increase in the number of FRP layers did not proportionately increase the joint capacities.
- Rupture of FRP sheets perpendicular to the fiber directions signifies the need to provide a more balanced layup.

3.5 Summary

FRPs, when used to strengthen joints, multiple sheets/strips are wrapped around a joint, using epoxy or other adhesives; these sheets are typically applied haphazardly, without maximizing the rupture stress resistance of the substrate materials through confinement. Therefore, there remains a serious concern of long-term integrity and the likelihood of cyclic loading on joints and components bonded in this manner. Furthermore, previous research was performed by randomly applying FRP composite sheets about a joint without focusing on minimizing stress concentration effects at joints. The stress concentration effects lead to de-bonding or delamination of FRP sheets from concrete substrate and even rupture prematurely at reentrant corners. Although small increases in mechanical properties, including energy absorption, were observed through random application of FRP sheets, the above-identified limitations lead to even more dramatic failures under seismic forces.

Current retrofit techniques are focused primarily on joint strengthening with composite wraps by wrapping columns and beams independently and then the beam-column joint area separately without providing uniform resistance to force transfer between structural elements (beam-column-slab). Furthermore, the majority of the research and evaluation work was focused on unidirectional carbon fiber composite layers that are placed orthogonal to each other on a layer-by-layer basis to build requisite composite thickness in and around a joint, as opposed to optimizing fabric architecture, which would provide higher strain to failure.

Key learning points based on the current review of concrete joints retrofitted with FRP composites are as follows:

- FRP repair can address several deficiencies identified in non-seismically designed RC joints and improve their performance.
- 45° orientation of fibers in the joint region with respect to the beam longitudinal axis has provided the best performance. This is because the fibers are more or less oriented parallel to the principal planes reinforcing the concrete strut.
- Increasing the number of FRP layers did not proportionally increase the joint capacities.

- Hinge formation in the beam section, irrespective of whether the final failure was concentrated in the joint or beam, has led to significant improvement in energy dissipation.
- Improvements in structural capacities of FRP strengthened 3D (with transverse beams) joints was lower than that of 2D (without transverse beams).
- In the case of FRP strengthened specimens, de-bonding always initiated at the high-stress concentration areas (joint corners), highlighting the need to redistribute stresses away from the corners or provide adequate anchorage to FRP wraps.
- The addition of U-strips enhanced the bond performance between underlying sheets and concrete substrate, thereby delaying the initiation of- or prevention of de-bond.
- Bonding discrete layers of L-and T-shape CFRP sheets without anchors has led to premature de-bonding from its substrate.
- Reinforcing the joint with CFRP sheets in continuous X-pattern has significantly improved the confinement of joint core resulting in higher ductility over the control specimen.
- Rupture of uni-directional FRP sheets perpendicular to the fiber directions signifies the need to provide balanced layup.
- Surface preparation of test specimens using a water jet coupled with structural adhesive was found to be more efficient in improving the shear capacity of joints when compared to the wire brush method.
- Although CFRP strengthened specimens exhibited joint shear failure patterns similar to that of control specimens, the failure was contained.
- Capacity enhancements in specimens strengthened with a single layer of CFRP having 45° orientation were rather inconsistent when compared to specimens strengthened with balanced $\pm 45^\circ$ orientation.
- EBROG (externally bonded reinforcement on groves) method coupled with CFRP fan anchors was found to be efficient in preventing the de-bonding of CFRP sheets from its substrate.

4. Experimental Program: Materials, Casting, Test Protocols, and Data Procurement

4.1 Introduction

This chapter provides in-depth information about the experimental program conducted on 2D reinforced concrete (RC) exterior beam-column joints under monotonic loads, including the types of materials employed, their mechanical properties, and the load strain/deformation response. More specifically, the experimental program described herein includes specimen details, specimen casting process, material properties, test setup, loading protocol, and instrumentation employed to collect test data. Additionally, test data collection procedures and the type of data gathered from the test samples are provided so that readers can appreciate the need for meticulous care in data collection to perform experimental analysis (Chapters 5 and 6) utilizing fundamental principles of mechanics of reinforced concrete structures with FRP wraps and filler-modules.

4.2 Specimen Details

Twenty full-scale test specimens were cast according to pre-1976 construction details to evaluate the behavior of joint specimens under monotonic loads. As identified in section 2.2, the pre-1976 joint deficiencies considered for this study focused on: (i) low strength concrete (~2600, 3300, 3800 psi), (ii) inadequate anchorage length of beam bottom bars extending into the joint, (iii) no transverse (stirrup) reinforcement in the joint panel, and (iv) weak column-strong beam design. All the test specimens were designed to ensure joint shear (diagonal tension) failure without yielding of rebars in beam/column by (i) providing span-to-depth ratios in the range of 2.5-5.5, (ii) limiting axial loads on the column to $0.28f'_c A_g$ or lower, and (iii) designing beams to exhibit higher moment and shear capacity than columns. Joint specimens were tested in both the control and reinforced conditions. The test specimen details are given in sections 4.2.1 and 4.2.2, respectively.

4.2.1 Control specimens

Four of the twenty joint specimens were tested in control condition to evaluate the joint shear capacity and identify the underlying failure mechanisms. The test results of control (control) specimens (Figure 42) were then compared with those of reinforced specimens to assess the structural capacity enhancements and note the changes in failure mode. The specimen casting details of control joints, including the steel rebar layout, cover, and geometric properties, are shown in Figures 41 through 43.

Figure 41. Schematic of control joint specimen

Figure 42. Geometric layout of control joint specimens

Figure 43. Steel rebar layout of control joint specimens

4.2.2 Reinforced specimens

A total of sixteen specimens were reinforced with filler-modules with/out FRP wrap/gusset to evaluate the strength, energy absorption and ductility enhancement of reinforced joints over control specimens. The construction details of reinforced joint specimens, including the steel rebar layout, cover, and geometric properties, are shown in Figures 44 through 46. A step-by-step procedure of reinforcing schemes including FRP wrap/gusset configurations for each specimen is provided in Tables 13 to 18.

Figure 44. Geometric layout of joint specimens reinforced with wedge filler-module

Figure 45. Steel rebar layout of joint specimens reinforced with wedge filler-module

Figure 46. Steel rebar layout of joint specimens reinforced with curve filler-module

Table 13. Beam-column joint reinforcing scheme for JI-2 through JI-4

Beam-column joint specimen reinforcing scheme	JI-2	JI-3	JI-4
<p>Shape: wedge</p> <p>C-FM-I</p> <p>Mat.: concrete (3.3 ksi)</p> <p>Dowels: yes</p> <p>Glue: Sikadur 31</p>	<p>Shape: wedge</p> <p>C-FM-I</p> <p>Mat.: concrete (3.3 ksi)</p> <p>Dowels: yes</p> <p>Glue: Sikadur 31</p>	<p>Shape: wedge</p> <p>SF 3000</p> <p>Mat.: foam (4.3 ksi)</p> <p>Dowels: yes</p> <p>Glue: Sikadur 31</p>
<p>Jet wash: N.A.</p> <p>Primer: N.A.</p>	<p>Jet wash: yes</p> <p>Primer: no</p>	<p>Jet wash: yes</p> <p>Primer: no</p>

<p>1-layer uniaxial CFRP wrap</p> <p>2-layers uniaxial CFRP wrap</p> <p>2-layers uniaxial CFRP wrap</p>	<p>N.A</p> <p>Sika 103C</p> <p>Sika 103C</p>
<p>1-layer 0/±45 CFRP U-wrap</p> <p>1'-0"</p>	<p>N.A</p> <p>Sika 103C</p> <p>Sika 103C</p>

<p>1-layer uniaxial CFRP 360°-anchor</p> <p>1-layer uniaxial CFRP 360°-anchor</p>	<p>N.A.</p> <p>Sika 103C</p> <p>Sika 103C</p>
<p>2-layers triaxial GFRP 360° wrap</p> <p>2-layers triaxial GFRP 360° wrap</p> <p>2-layers triaxial GFRP 360° wrap</p>	<p>N.A.</p> <p>Triaxial GFRP</p> <p>Triaxial GFRP</p>

Table 14. Beam-column joint reinforcing scheme for JI-6 through JI-8

Beam-column joint specimen reinforcing scheme	JI-6	JI-7	JI-8
N.A.	Primer: Sikadur 30 Sika 103C	Primer: Sikadur 30 Sika 103C
Shape: wedge C-FM-II Mat.: concrete (5.5 ksi) Dowels: yes Glue: Sikadur 31	Shape: wedge C-FM-II Mat.: concrete (5.5 ksi) Dowels: yes Glue: Sikadur 31	Shape: wedge C-FM-III Mat.: concrete (9.5 ksi) Dowels: yes Glue: Sikadur 31

<div>Jet wash: yes</div> <div>Primer: Sikadur 30</div> <div>Jet wash: yes</div> <div>Primer: Sikadur 30</div> <div>Jet wash: yes</div> <div>Primer: Sikadur 30</div>
<div>4-layers Sika 103C</div> <div>2-layers Sika 103C</div> <div>2-layers Sika 103C</div>

<p>1-layer 0/±45 CFRP gusset bonded on either side of specimen</p> <p>1'-0"</p>	<p>Sika 103C</p> <p>Sika 103C</p> <p>Sika 103C</p>
<p>2-layers uniaxial CFRP U-anchor</p> <p>2-layers uniaxial CFRP U-anchor</p> <p>12"</p>	<p>Sika 103C</p> <p>Sika 103C</p> <p>Sika 103C</p>

Triaxial GFRP

Triaxial GFRP

Triaxial GFRP

Table 15. Beam-column joint reinforcing scheme for JIII-2 through JIII-4

Beam-column joint specimen reinforcing scheme	JIII-2	JIII-3	JIII-4
<p>2-layers uniaxial CFRP wrap</p> <p>2-layers uniaxial CFRP wrap</p>	<p>Primer: Sikadur 30 Sika 103C</p>	<p>Primer: Sikadur 30 Sika 103C</p>	<p>Primer: Sikadur 30 Sika 103C</p>
<p>1'-3"</p> <p>1'-0"</p> <p>1'-0"</p> <p>3/8" DIA. X 15" LONG DOWEL</p> <p>3/8" DIA. X 20" LONG DOWEL</p> <p>6"</p> <p>Filler-module</p>	<p>Shape: curve C-FM-II Mat.: concrete (5.5 ksi) Dowels: yes Glue: Sikadur 31</p>	<p>Shape: wedge C-FM-II Mat.: concrete (5.5 ksi) Dowels: yes Glue: Sikadur 31</p>	<p>Shape: wedge C-FM-II Mat.: concrete (5.5 ksi) Dowels: yes Glue: Sikadur 31</p>

<p>3/8" DIA. X 15" LONG DOWEL</p> <p>3/8" DIA. X 20" LONG DOWEL</p> <p>6"</p> <p>1'-3"</p> <p>1'-0"</p> <p>1'-0"</p>	<p>Jet wash: yes</p> <p>Primer: Sikadur 30</p>	<p>Jet wash: yes</p> <p>Primer: Sikadur 30</p>	<p>Jet wash: yes</p> <p>Primer: Sikadur 30</p>
<p>2-layers uniaxial CFRP wrap</p> <p>2-layers uniaxial CFRP wrap</p>	<p>Sika 103C</p>	<p>Sika 103C</p>	<p>Sika 103C</p>

<p>1-layer 0/±45 GFRP U-wrap</p> <p>1'-0"</p>	Triaxial GFRP	Triaxial GFRP	Triaxial GFRP
<p>2-layers uniaxial CFRP U-anchor</p> <p>12"</p> <p>2-layers uniaxial CFRP U-anchor</p>	N.A	N.A.	Sika 103C

Triaxial GFRP

Triaxial GFRP

Triaxial GFRP

Table 16. Beam-column joint reinforcing scheme for JII-2

Beam-column joint specimen reinforcing scheme	JII-2
<p>Jet wash: yes</p> <p>Primer: no</p>
<p>Sika 103C</p>

<p>1-layer 0/±45 CFRP U-wrap</p> <p>12"</p>	<p>Sika 103C</p>
<p>2-layers triaxial GFRP 360° wrap</p>	<p>Triaxial GFRP</p>
<p>2-layers uniaxial CFRP 360° wrap</p> <p>2-layers uniaxial CFRP 360° wrap</p>	<p>Sika 103 C</p>

Table 17. Beam-column joint reinforcing scheme for JII-3 and JII-4

Beam-column joint specimen reinforcing scheme	JII-3	JII-4
<p>1'-3"</p> <p>1'-0"</p> <p>3/8" DIA. X 15" LONG DOWEL</p> <p>3/8" DIA. X 20" LONG DOWEL</p> <p>6"</p> <p>Filler-module</p>	<p>Shape: curve Mat.: PSL 2.0E Dowels: yes Glue: Sikadur 31</p>	<p>Shape: wedge Mat.: PSL 2.0E Dowels: yes Glue: Sikadur 31</p>

Table 18. Beam-column joint reinforcing scheme for JII-5 through JII-8

Beam-column joint specimen reinforcing scheme	JII-5	JII-6	JII-7	JII-8
Shape: curve Mat.: PSL 2.0E Dowels: no Glue: Sikadur 31	Shape: curve Mat.: PSL 2.0E Dowels: yes Glue: Sikadur 31	Shape: wedge Mat.: PSL 2.0E Dowels: yes Glue: Sikadur 31	Shape: wedge Mat.: PSL 2.0E Dowels: yes Glue: Sikadur 31
Jet wash: yes Primer: no	Jet wash: yes Primer: no	Jet wash: yes Primer: no	Jet wash: yes Primer: no

<p>1-layer uniaxial FRP wrap</p> <p>2-layers uniaxial FRP wrap</p> <p>2-layers uniaxial FRP wrap</p>	Sika 103C	Sika 103C	Sika 103C	Sika 100G
<p>1-layer 0/±45 FRP U-wrap</p> <p>1'-0"</p>	Sika 103C	Sika 103C	Sika 103C	Sika 100G

Sika 103C

Sika 103C

Sika 103C

Sika 100G

Column:
Sika 103C;
Beam:
Triaxial GFRP

Column:
Sika 103C;
Beam:
Triaxial GFRP

Column:
Sika 103C;
Beam:
Triaxial GFRP

Column:
Sika 103C;
Beam:
Triaxial GFRP

4.3 Specimen Casting

The joint test specimens were cast in T-shaped plywood molds resting horizontally on the floor over a plywood sheeting, as shown in Figure 47. To avoid bulging, the walls of the formwork were supported from outside using wooden blocks that are screwed to the plywood sheeting. The gaps along the edges between the plywood sheet and formwork were sealed with silicone material to prevent any leakage of concrete slurry. The inner walls of the formwork and floor of the plywood sheeting were made free of debris by vacuuming and then coated with an oil layer for easy release of the cured test specimen. The reinforcement cages for column and beam sections were constructed separately, as shown in Figure 48. Grade 60 deformed steel rebar of two different sizes (#6 and #3) were used for the construction of the reinforcement cage. The mechanical properties of steel bar specimens are provided in section 4.4.2. The #6 bars were used as longitudinal reinforcement for beam and column sections (Figure 48), while the #3 bars were used for stirrups. The reinforcement cages were ground at select locations as identified using a painter's tape (Figure 49) to allow for the installation of strain gages at a later stage.

Figure 47. Plywood formwork for casting concrete joint specimens

Figure 48. Preparation of reinforcement cage for beam and column sections

Figure 49. Grinding at select locations of rebar cage for installation of strain gages

As shown in Figure 50, the beam and column reinforcement cages were assembled in molds by separating the rebar cage from touching the plywood using appropriate size concrete chairs. This arrangement ensures adequate concrete cover thickness. Besides, 2 in. diameter cylindrical foam plugs were placed underneath the locations where the rebar was ground. To prevent sliding of foam plugs during concrete placement and vibration, they were secured to the floor of plywood using screws. Once the joint specimens are cured, these plugs were removed to access the rebar surface for the installation of strain gages, as shown in Figure 51. To avoid drilling through the thickness of beam and column sections for dowel insertion, PVC pipes were tied to the reinforcement cage that served as placeholders.

Figure 50. Closer view of the reinforcement cage after assembling in a mold

Figure 51. Installing strain gage on steel rebar

The joint test specimens were cast in three batches. Concrete for all three batches was supplied by a local manufacturer and transported to the lab facility using a transit mixer truck. As shown in Figure 52, concrete was placed in the forms while simultaneously vibrating with a pin-vibrator to minimize potential air voids and honeycombing. The test specimens were cured for seven days by sprinkling water after covering with wet burlap rolls. The formwork was stripped within 48 hours after the concrete pour.

Figure 52. Placement of concrete during test specimen preparation

4.3.1 Filler-module fabrication

Prefabricated filler-modules made of three different materials (concrete, parallel strand lumber (PSL), and syntactic foam) were investigated in the present study. The mechanical properties of each filler-module material are provided in section 4.4.

4.3.1.1 Concrete filler-modules

Concrete filler-modules were pre-cast by placing formwork against the joint specimens at their corners, as shown in Figure 53. Before creating the formwork for filler-modules, the aluminum tape is bonded to the joint specimen at its corners that receive filler-module. The aluminum tape acts as a barrier and alleviates the release of filler-modules from joint specimens. To ensure the alignment of dowel holes in filler-modules and joint specimen, PVC pipes were inserted as placeholders. The PVC pipes are removed within 24 hours after the concrete pour by gently tapping at their ends. The concrete filler-modules are cured in the same way as the joint specimens.

Figure 53. Casting of concrete filler-modules

4.3.1.2 *Parallel strand lumber (PSL) filler-modules*

Parallel Strand Lumber (PSL) filler-modules were created by gluing together two 5 in. thick triangular blocks using the Titebond III. Quick-release clamps were used to hold the blocks together until cured, as shown in Figure 54. The compressive force from the clamps squeezed out excess glue, thereby minimizing air voids. Dowel holes were drilled through the filler-modules by orienting them against the joint corners that receive the modules.

Figure 54. Making of Parallel Strand Lumber (PSL) filler-modules

4.3.1.3 *Syntactic foam (SF) filler-modules*

Syntactic foam filler-modules were cast in a similar process to PSL filler-modules.

4.3.2 Installation of filler-modules at joint corners

The steps illustrated in Figure 55 were followed for installation of prefabricated filler-modules at joint corners:

(a) Light sanding of FRP/concrete surface

(b) Coating Sikadur 31 on substrate

(c) Coating Sikadur 31 on filler-module(s)

(d) Orienting filler-module(s) at joint corner

(e) Securing Filler-modules with dowels bars

(f) Clamping of filler-modules until cured

Figure 55. Filler-module installation process at joint corners

4.3.3 Surface preparation

The efficiency of an FRP strengthened concrete structure in resisting applied loads is heavily dependent on the bond (force-transfer) between the underlying substrate and FRP. Therefore, for creating efficient force-transfer between the substrate and FRP, surface preparation is essential. Hence, all the joint specimens in the current study were jet washed with water, as shown in Figure 56. Figure 57 shows a closer view of the concrete substrate surface before and after jet washing. The jet washed specimens were dried at room temperature for at least seven days before the installation of filler-modules or externally bonded FRP wraps.

Figure 56. Surface preparation through water-jet washing

(a) Before jet wash (b) After jet wash
Figure 57. Concrete surface before- and after- water-jet washing

4.3.4 FRP wrap/gusset installation

Before bonding FRP wraps, Sikadur 30 was applied to the concrete substrate as a primer to fill any bug holes and create a uniform surface. The FRP wraps, cut to required shape and dimensions, were impregnated with Sika Hex 300 resin and bonded to the concrete substrate. Figure 58 illustrates the FRP wrap application process. After the application of each FRP layer, the wrap surface was smoothed using a hand-held paint roller by pressing against the specimen to squeeze out any excess resin, thus minimizing air voids.

(a) Primer coated joint specimen

(b) Saturation of glass fabric with epoxy resin

(c) Bonding FRP wrap to the concrete substrate

Figure 58. FRP wrapping of joint specimen

FRP gusset is a pre-cured composite laminate manufactured using hand lay-up process by gluing together a stack of carbon fabric layers after saturating with Sika Hex 300 resin. Figure 59 shows the FRP gusset configuration used in the current study. The gussets were bonded to the primed concrete substrate using Sikadur 30.

Figure 59. FRP gusset used in the current study for reinforcing joint specimens

Both the FRP wrapped and gusset reinforced joint specimens were cured for at least three days before testing the specimen to failure.

Note: Primer was applied only for specimens JI-5 through 8 and JIII-2 through 4.

4.4 Materials

The following section provides details of various materials employed for this study, including their mechanical properties.

4.4.1 Concrete

Twelve (6 in. x 12 in.) and eight (4 in. x 8 in.) plain concrete cylinder specimens were tested to establish the mechanical properties of concrete used for casting joint and filler-module specimens, respectively. The cylinder specimens were tested within seven days after joint specimen testing. Before stripping the plastic molds, all the cylinder specimens were cured for 28 days by immersing in a water bath. Procedures listed in ASTM C39 and ASTM C496 were followed for evaluating the concrete compressive and tensile strengths, respectively. A summary of concrete material properties used in casting joint specimens and filler-modules is provided in Tables 19 and 20, respectively.

Table 19. Properties of plain concrete (joint) specimens

Specimen ID		Unconfined concrete properties	
		f'_c (psi)	f_{ct} (psi)
J-I	C6-S1	2491	237
	C6-S2	2788	233
J-I Average:		2639	235
J-II	C6-S1	3363	305
	C6-S2	3254	286
J-II Average:		3309	296
J-III	C6-S1	3817	N.A.
	C6-S2	3916	N.A.
J-III Average:		3866	N.A

Figure 60. Load vs. position curves for plain concrete (joint) specimens

Table 20. Properties of plain concrete (filler-modules) specimens

Specimen ID	Unconfined concrete properties	
	f'_c (psi)	f_{ct} (psi)
C-FM-I	C4-S1	3374
	C4-S2	3377
C-FM-I Average:		3376
C-FM-II	C4-S1	5109
	C4-S2	5836
C-FM-II Average:		5472
C-FM-III	C4-S1	9691
	C4-S2	9400
C-FM-III Average:		9545

Figure 61. Load vs. position curves for plain concrete (filler-modules) specimens

4.4.2 Steel rebar

Two different sizes (#6 and #3) of grade 60 deformed steel rebar specimens were evaluated to establish their tensile properties under monotonic loading. The steel rebar specimens were tested in an Instron machine by following ASTM A 370 standard. Two of the three #6 rebar specimens failed near the grip (~1.5 in. above the grip) while all other specimens fractured within the gage length. The final failure of each specimen is shown in Figures 62 and 63. Figures 64 and 65 show the stress-strain curves of #6 and #3 rebar specimens, respectively. The stress and strain values corresponding to the plateau are defined as yield numbers for all rebar specimens. A summary of the mechanical properties of #6 and #3 rebar specimens is provided in Table 21.

Table 21. Properties of steel rebar specimens

Specimen ID		Rebar tensile properties				
		f_u (ksi)	ϵ_u	E_s (ksi)	f_y (ksi)	ϵ_y
Bar #6	S1	119.55	0.082	28260.50	73.34	0.0027
	S2	104.79	0.081	27948.16	72.72	0.0028
	S3	89.96	0.082	29778.16	73.76	0.0027
Bar #6 Average:		104.77	0.082	28662.27	73.27	0.0027
Bar #3	S1*	101.75	0.033 ⁺	25577.13	76.98	0.0031
	S2*	110.22	0.035 ⁺	23595.36	70.32	0.0030
	S3*	109.67	0.082	23739.46	78.21	0.0034
Bar #3 Average:		107.21	0.082	24303.98	75.17	0.0032

Note: *Deviation in mechanical properties is attributed to the low carbon content in steel rebar.

⁺Data ignored because of premature specimen failure.

(a) Bar #6 – S1

(b) Bar #6 – S2

(c) Bar #6 – S3

Figure 62. Failure modes of grade 60 #6 rebar specimens

(a) Bar #3 – S1

(b) Bar #3 – S2

(c) Bar #3 – S3

Figure 63. Failure modes of grade 60 #3 rebar specimens

Figure 64. Stress-strain plots for grade 60 #6 steel rebar specimens

Figure 65. Stress-strain plots for grade 60 #3 steel rebar specimens

4.4.3 Engineered wood (PSL)

Parallel strand lumber (PSL), popularly known as engineered wood, is manufactured by orienting wood strands in a parallel direction and gluing them together under specific temperature and pressure conditions. PSL is a high-quality composite wood product with consistent mechanical properties that makes it an excellent choice for structural applications. Compared to traditional wood, PSL is less susceptible to bowing, cupping, shrinkage, warping, and even easier to treat; thus, making it suitable for outdoor applications. The light-weight characteristic of PSL allows for easy installation at job sites, primarily when used as filler-modules. Besides, PSL is ideally suited for seismic and blast-resistant applications due to its higher energy-absorption capacity under shock loads than concrete. The design properties of Trus Joist® 2.0E Parallam® PSL (Figure 66) used in this study are listed in Table 22. For additional information on this material, refer to Trus Joist specifier's guide.

Figure 66. Trus Joist Parallam® PSL [54]

Table 22. Design properties of Parallam® PSL [54]

Grade	Modulus of Elasticity (<i>msi</i>)			Stress (<i>psi</i>)					
	G	E	E _{min}	F _b	F _t	F _{c⊥}	F _c	F _v (<i>psi</i>)	SG
2.0E	0.125	2.0	1.016	2900	2025	625	2900	290	0.50

4.4.4 Engineered syntactic foam

Syntactic Foam (SF) is a low-density composite foam that offers excellent mechanical properties for use in structural applications. Depending on the grade, the compressive and flexural strengths of commercially available SF range up to 6700 psi and 5800 psi, respectively. The light-weight characteristic of SF allows for easy installation at job sites, primarily when used as joint reinforcement (as proposed in this study). Besides, non-degradable SFs also offer excellent long-term performance under outdoor exposures, which makes it suitable for reinforcing joints of parking garages and bridge bents that are exposed to de-icing salts. Also, SFs are ideally suited for seismic and blast-resistant applications due to their high energy-absorption capacity under shock loads. The mechanical properties of SF 3000 Grade Microsphere Syntactic Foam (Figure 67) used in this study are listed in Table 23. For additional information on this material, refer to SF Grade Microsphere Syntactic Foam technical datasheet.

Figure 67. SF 3000 grade microsphere syntactic foam

Table 23. Mechanical properties of SF grade syntactic foam [55]

Grade	Density (lb/ft^3)	Compression properties		Flexural properties	
		Strength (psi)	Modulus (ksi)	Strength (psi)	Modulus (ksi)
SF 1500	35	2685	115	4390	412
SF 3000*	39	4320	261	4920	441
SF 5000	46	6700	498	5860	490

*Product used in the current study.

4.4.5 FRP composite reinforcement

Properties of FRP composite reinforcements used in this study are obtained from manufactures technical datasheets, and a summary is presented in Table 24.

Table 24. Properties of FRP composite materials [56] [57]

Specimen ID	FRP configuration	Ply thickness (in.)	FRP tensile properties	
			f_{fu} (ksi)	ϵ_{fu} ($\mu\epsilon$)
Sika 103C CFRP	Unidirectional fabric	0.04	181	0.0175
Sika 100G GFRP	Unidirectional fabric	0.04	88.6	-

4.4.6 FRP composite confined concrete cylinders

Two different strengths (~2.9 and 3.6 ksi) of plain concrete cylinder specimens were wrapped with FRP composite fabrics to evaluate the capacity enhancements through confinement under axial compression and split-tensile tests. Besides, the numerical values of load and strain at first crack and failure were recorded. A summary of the test results is presented in Tables 25 and 26.

Table 25. Properties of FRP confined plain concrete (compression test)

Specimen ID	J-I			J-II		
	CS	S-1	S-2	CS	S-1	S-2
Peak axial stress (psi)	2925	14701	3239	3555	15827	12640
Hoop strain at peak stress ($\mu\epsilon$)	0.012	0.009	0.0002	-0.003	0.003	0.0015
Axial strain at peak stress ($\mu\epsilon$)	-0.003	-0.03	-0.0008*	-0.002	-0.02	-0.015
Axial stress at the 1st crack (psi)	1440	2000	2200	1853	1703	2002
Axial strain at the 1st crack ($\mu\epsilon$)	0.0006	0.0007	0.0005	0.0004	0.0003	0.0005

*Experimental anomaly

Table 26. Properties of FRP confined plain concrete (split-tension test)

Property	J-I			J-II		
	CS	S-1	S-2	CS	S-1	S-2
Peak tensile strength (psi)	429	1234	1027	420	1691	1229
Tensile stress at 1st crack (psi)	200	200	200	N.A.	259	250

Note:

J-I S-1: 2 layers of Sika103C unidirectional carbon wrap with 3 in. overlap

J-I S-2: 2 layers of Sika103C unidirectional carbon wrap with 0.75 in. gap between the ends

J-II S-1: 2 layers of Sika103C unidirectional carbon wrap with no overlap and no gap

J-II S-2: 2 layers of Sika100G unidirectional glass wrap with no overlap and no gap

Only one specimen was tested per category

4.4.6.1 Summary of test results

1. The first tensile crack occurred at $\sim(4 - 5)\sqrt{f'_c}$.
2. Split tensile strength for fully confined (360°) FRP wrapped concrete cylinders ranged from $\sim(20 - 28)\sqrt{f'_c}$.
3. Axial strain of FRP confined concrete cylinders was $\sim(8-10)$ times of unconfined cylinder.
4. Hoop strain of FRP confined cylinders was ~ 3 times of unconfined cylinder.

4.5 Test Setup of 2D RC Exterior Joints

A schematic of the test setup of control and reinforced joint specimens is shown in Figures 68 and 69, respectively. The test specimens were designed such that the ends of the column and beam members represent the inflection points of a typical building frame. The specimen was set-up in a vertical position, i.e., the longitudinal axis of the column was oriented perpendicular to the strong-floor, and the longitudinal axis of the beam was oriented parallel to the strong-floor. The column was clamped to the loading frame at 6 in. from the column top and bottom ends using steel rollers and threaded rods. This arrangement provides lateral stability for the specimen against applied beam bending load. Besides, the column base was rested on roller support to allow for column rotation.

A pre-determined axial load was applied on the column through a hydraulic actuator using a manual pump. A load cell was placed between the column top surface and actuator to measure the applied axial load. The beam bending load was applied at 6 in. from the free end of the beam through a loading plate. A load cell situated between the hydraulic actuators and the loading plates

measured the load applied on to the beam. The actuator was pinned at the top end to allow for rotation during the test. The beam loading actuator was controlled using MTS having a maximum capacity of 110-kip. Sensors such as strain gages, and LVDTs were placed at critical locations of the test specimen to record the data.

Figure 68. Test setup of control joint specimens

Figure 69. Test setup of reinforced joint specimens

4.6 Loading Protocol

Before beginning each test, the predetermined axial load was slowly applied to the column until the desired load level was achieved. Monotonic bending load was applied at six inches from the free end of the beam using a hydraulic actuator. The maximum loading capacity of the MTS controlled hydraulic actuator used in the current experimental testing was 110 kips. The desired axial compression load was maintained for the entire duration of the test by manually adjusting the actuator through a hand pump. The test specimens were loaded until a significant drop in the load-carrying capacity was observed, indicating the failure of the test specimen.

4.7 Instrumentation

4.7.1 Measurement of the applied load

Two load cells that are connected to Vishay 7000 data acquisition were placed one each on the column and beam surfaces under the actuators, to record the numerical values of applied loads.

The load cells were calibrated before beginning the current experimental program. The axial load on the column was measured using a 200-kip load cell, while a 100-kip load cell was used to measure the beam bending load. The loads thus measured were plotted against the observed deflections (reported in Chapters 5 and 6) to compute the stiffness, ductility, and energy absorption capacities of joint test specimens.

4.7.2 Measurement of displacement

Linear variable displacement transducer (LVDT), shown in Figure 70, was placed at 6 in. from the free end of the beam to measure the vertical deflection under the load.

Figure 70. Linear variable displacement transducer (LVDT) (OMEGA Engineering) [58]

4.7.3 Measurement of strain

4.7.3.1 Strain measurement in steel rebar

Strain gages with a resistance of 120Ω or 350Ω are bonded to the rebar surface at several critical locations of the beam, column, and joint sections of the test specimen. The critical sections were determined based on the projected yield and hinge formation locations. The strains measured at various load levels were used to evaluate the strain penetration from the beam and column sections into the joint, reported in Chapters 5 and 6. A schematic view of the strain gage arrangement is shown in Figure 71.

Figure 71. Schematic view of strain gage arrangement on steel rebar cage

4.7.3.2 Strain measurement on concrete surface

As shown in Figure 72, a shear rosette was bonded to the concrete surface on the joint panel to determine the joint principal strains at first crack, rebar yielding, and peak load levels. The principal strains obtained were compared against the plain concrete cylinders tests to establish the limit states for potential shear or diagonal-tension failure modes, as reported in Chapters 5 and 6.

Figure 72. Strain gage arrangement on the concrete surface

4.7.3.3 Strain measurement on FRP composite wrap

Reinforced joint specimens were extensively instrumented with strain gages on FRP composite wrap surface as shown in Figure 73. A total of twenty-six strain gages were installed to monitor the strain distribution at various load levels on beam, column and joint sections, reported in Chapters 5 and 6. Besides, shear rosette was installed on the joint panel to compute the joint principal strains at the first crack, rebar yielding, and peak load levels. Furthermore, the strain levels on the FRP wrap surface will establish the failure modes such as local de-bond, buckling or rupture of FRP composite wrap.

Figure 73. Strain gage arrangement on FRP wrap surface

Figure 74. Strain gage arrangement on FRP wrap surface

Figure 75. Strain gage arrangement on FRP wrap surface

4.8 Data Collection

4.8.1 Data acquisition system

The test data from various measurement sensors, such as strain gages, load cells, and LVDTs, were collected through Vishay 7000 data acquisition system (shown in Figure 76) and processed using Strain Smart Software. The data thus processed is reduced and imported to a Microsoft Excel worksheet for further evaluations.

Figure 76. Vishay 7000 data acquisition system [59]

4.9 Computation of Principal Strains

Based on the strain data generated through the shear rosette (Figure 77), the numerical values of principal strain and the orientation of principal plane for each joint test specimen, as reported in Chapters 5 and 6, were computed using Equations 4.1 and 4.2.

Figure 77. Shear rosette arrangement

$$\varepsilon_{1,2} = \frac{\varepsilon_A + \varepsilon_C}{2} \pm \frac{1}{\sqrt{2}} \sqrt{(\varepsilon_A - \varepsilon_B)^2 + (\varepsilon_B - \varepsilon_C)^2} \quad (4.1)$$

$$\phi = \frac{1}{2} \tan^{-1} \left(\frac{\varepsilon_A - 2\varepsilon_B + \varepsilon_C}{\varepsilon_A - \varepsilon_C} \right) \quad (4.2)$$

4.10 Computation of Shear Strain

Using the strain data generated through shear rosette, the joint shear strain for each joint specimen was computed using Equation 4.3.

$$\gamma_{xy} = 2\varepsilon_B - \varepsilon_A - \varepsilon_C \quad (4.3)$$

4.11 Summary

This chapter provided the details of the experimental program on 2D exterior joint specimens tested under monotonic loads. As identified in previous sections, all the test specimens were instrumented appropriately to capture the global, as well as local behavior. The experimental data collected through LVDTs, load cells, and strain gages were evaluated meticulously to study the improvements in joint behavior with filler-modules and FRP wraps or gussets, as reported in Chapters 5 and 6.

5. Behavior of RC Joints Reinforced with Concrete/Syntactic Foam Filler-Modules and FRP Wraps/Gussets

5.1 Introduction

This chapter deals with the structural response evaluation of RC beam-column joints reinforced with concrete/syntactic foam filler-modules with/out fiber-reinforced polymer (FRP) composite wraps or gussets under monotonic loads. The joints without filler-modules are identified as control specimens, herein. The joint testing has been conducted in three phases to evaluate the influence of various parameters, as listed in Table 27:

- (i) Phase-I: joints reinforced with concrete/syntactic foam filler-modules with/out CFRP composite wraps,
- (ii) Phase II: joints reinforced with concrete filler-modules and CFRP gussets, and
- (iii) Phase III: joints reinforced with concrete filler-modules and GFRP composite wraps.

When conceptualizing the reinforcing schemes using filler-modules and FRP wraps for joint specimens, it was projected that the joint load versus strain response could be classified into three distinct zones, i.e., initiation of filler-module cracking in the tension zone - refers to point A, transition of tensile forces to steel rebar cage after significant concrete cracking - refers to point B, and peak load the specimen can resist before complete collapse - refers to point C. Based on the above hypothesis, the influence of the above parameters are studied by comparing the joint principal- and shear- strain responses at three salient points: A (onset of filler-module cracking), B (idealized yield - defined as the point beyond which a truss mechanism primarily resists the forces through internal rebar cage, FRP wrap, and rebar-concrete bond forces), and C (peak load - referred to as a highest numerical value of load recorded during the testing). Points A, B, and C were identified based on the load vs. strain response of the column rebar (CSL2, as identified in Figure 78). Based on the load vs. column rebar strain plot, “A” refers to the endpoint of the second slope change, and “B” refers to the endpoint of the third slope change. Point C is the peak load measured during laboratory testing. Figure 78 shows a generic load vs. column rebar strain of joint specimen reinforced with filler-modules and FRP wraps

or gussets. Besides, one specimen from each group is tested as a control specimen to establish the baseline value for computation of structural capacity enhancements in joints reinforced with filler-modules with/out FRP wraps or gussets. The joint specimens were constructed, tested, and evaluated per the procedures described in Chapter 4 of this manuscript. The experimental results monitored through load cells, LVDTs, and strain gages and recorded using MTS and StrainSmart® data acquisition systems are presented and discussed in the following sections. Addition test data on load versus strain response of joint specimens is presented in Appendices A through D. Furthermore, discussion of experimental results in terms of strength, stiffness, ductility, and energy dissipation capacity enhancements is provided in sections 5.3.

Figure 78. Load vs. column rebar strain response of a typical joint specimen in this study

Note:

1. The strains induced due to the axial load on the column were zeroed (before the application of beam bending load) and presented from the start of beam bending load, except for the specimens exhibiting compression failure. Based on the previous studies by [60], the influence of column axial load on the joint shear strength was found to be negligible and hence was not

reported. However, in the case of specimens exhibiting compression failure mode, both results (with/out the influence of axial load) are presented.

- Points A, B, and C were consistently identified with respect to the column rebar strain gage (CSL2) closer to the joint location. Based on the load vs. column rebar strain plot, “A” refers to the endpoint of the second slope change, and “B” refers to the endpoint of the third slope change. Point C is the peak load measured during laboratory testing.

Table 27. Parameters tested to evaluate the performance of joints reinforced with concrete/syntactic foam filler-modules with and without FRP wraps/gussets

Phase #	Joint	f'_c	CL	Reinforcing scheme
Phase I	JI-1	2639 psi	$0.28f'_cA_g$	Control
	JI-2			CW(3.3 ksi)+D
	JI-3			CW(3.3 ksi)+D+CFRP Wrap+360°-anchor
	JI-4			SF(4.3 ksi)+D+CFRP Wrap+360°-anchor
Phase II	JI-5	2639 psi	$0.18f'_cA_g$	Control
	JI-6			CW(5.5 ksi)+D+CFRP Gusset+U-anchor
	JI-7			CW(5.5 ksi)+D+CFRP Gusset+U-anchor*
	JI-8			CW(9.5 ksi)+D+CFRP Gusset+U-anchor*
Phase III	JI-1	3866 psi	$0.18f'_cA_g$	Control
	JI-2			CC(5.5 ksi)+D+GFRP Wrap+no anchor*
	JI-3			CW(5.5 ksi)+D+GFRP Wrap+no anchor*
	JI-4			CW(5.5 ksi)+D+GFRP Wrap+U-anchor*

D: dowels; CW: concrete wedge; CC: concrete curve; SW: syntactic foam wedge; CL: column axial load; f'_c : substrate concrete compressive strength; A_g : gross cross-sectional area of concrete (column); *CFRP layer bonded between filler-module and substrate beam and column that receive filler-modules at the reentrant corners.

5.2 Experimental Results

This section describes the structural response of joint specimens tested in control and reinforced (joints with filler-modules with/out FRP wraps/gussets) conditions.

5.2.1 Behavior of control specimens (JI-1, JI-5, and JIII-1)

Three joint specimens (JI-1, JI-5, and JIII-1) were tested under control conditions to assess their performance under monotonic loading. The parameters varied herein include concrete compressive strength (2639 vs. 3866 psi) and induced column axial load (0.18 vs. 0.28%). A span-to-depth (a/d) ratio of 4.8 was adopted to ensure the diagonal tension failure of joints.

As the beam loading was applied, all control specimens exhibited corner cracking at the interface of a beam-column joint. The observed corner cracks are referred to as typical flexural-tension (f_r) cracks associated with stress-concentrations effects. The strains measured on the surface of beam longitudinal rebar, close (~2.5 in) to the interface of the beam-column joint, are in the range of 91 - 143 $\mu\epsilon$. Assuming a linear strain variation along the depth of the beam, the strains on the concrete surface were computed in the range of 137 - 214 $\mu\epsilon$. Translation of these strains as a function of tensile stresses ($k\sqrt{f'_c}$) revealed k values in the range of 8 - 12, which are typical numbers reported in [61]. A summary of the observed strains and computations corresponding to the corner cracking (modulus of rupture) for each control specimen is presented in Table 28. These corner cracks observed during the initial stages of testing did not progress beyond the concrete clear cover depth as they were restrained by the dowel action of beam longitudinal rebar. As the loading increased beyond f_r , diagonal tension cracks originated within the joint panel. Figure 79 shows a typical corner- and initial diagonal tension- cracking observed in all control test specimens. The joint principal tensile- and shear- strains at the formation of diagonal cracking were measured as 53 - 70 $\mu\epsilon$ and 86 - 147 $\mu\epsilon$, respectively. Tables 29 and 30 present the summary of principal- and shear- strains for control specimens without and with the influence of column axial loads, respectively. Figures 83 through 88 show the load vs. principal- and shear- strain plots of control joint specimens.

Table 28. Modulus of rupture for control joints (JI-1, JI-5, JIII-1)

Joint	Beam load (lb)	Beam rebar strain ($\mu\epsilon$)	Concrete strain ($\mu\epsilon$)	$k-f_r$
JI-1	2,475	106	158	9
	2,889	143	214	12
JI-5	1,625	92	138	8
	2,035	141	212	12
JIII-1	2,014	91	137	8
	4,016	119	178	10

Table 29. Principal strain limit state for control joints without axial load influence

Joint	PoI	Beam load (lb)	Joint principal strain ($\mu\epsilon$)			Joint limit state	
			P(t)	P(c)	γ	k- P(t)	k- γ
JI-1	A	5,063	53	-94	-147	3.0	3.5
	B	11,076	43	-261	-266	2.4	6.4
	C	15,471	Off-scale	Off-scale	Off-scale	N.A.	N.A.
JI-5	A	2,183	57	-39	86	3.3	2.1
	B	10,886	Off-scale	Off-scale	Off-scale	N.A.	N.A.
	C	18,819	Off-scale	Off-scale	Off-scale	N.A.	N.A.
JIII-1	A	7,076	70	-16	86	4.0	2.1
	B	15,658	91	-142	72	5.2	1.8
	C	24,740	Off-scale	Off-scale	Off-scale	N.A.	N.A.

PoI: Point of interest; P(t): principal tensile strain; P(c): principal compressive strain; γ : shear strain

Table 30. Principal strain limit state for control joints with axial load influence

Joint	PoI	Beam load (lb)	Joint principal strain ($\mu\epsilon$)			Joint limit state	
			P(t)	P(c)	γ	k- P(t)	k- γ
JI-1	A	5,063	29.18	-160.48	-123.08	3.01	3.54
	B	11,076	36.87	-344.98	-242.33	2.44	6.42
JIII-1	A	7,076	79.70	-101.40	113.73	4.54	2.75
	B	15,658	-0.28	-126.95	100.27	-0.02	2.42

PoI: Point of interest; P(t): principal tensile strain; P(c): principal compressive strain; γ : shear strain

Figure 79. Initial diagonal tension cracking in the joint panel

Due to the sufficient anchorage length (90° hook) provided for the beam longitudinal rebar extending into the joint panel, the tensile forces induced in rebar by the initial diagonal tension cracking did not result in bond failure or debond of rebar from concrete (at this stage); thus forming a concrete strut to resist additional joint shear forces. Any additional loading applied on the beam beyond this point was primarily resisted by the concrete strut and rebar-concrete bond forces. Upon bond failure, the additional shear forces induced in the joint panel were primarily resisted by concrete strut mechanism, shown in Figure 11, until it is significantly weakened by the concrete tensile strain. Ultimately, all control specimens failed in joint shear mode (Figure 82) followed by concrete cover delamination, except for specimen II-1. Specimen II-1 exhibited diagonal compression failure by crushing of concrete in the joint panel due to the presence of high axial compressive force ($0.28f'_cA_g$) on the column. Figures 80 through 82 show the failure modes of control specimens tested under monotonic loading.

Figure 80. Failure of JI-1

Figure 81. Failure of JI-5

Figure 82. Failure of JIII-1

Figure 83. Joint principal strains for JI-1

Figure 84. Joint shear strain for JI-1

Figure 85. Joint principal strains for JI-5

Figure 86. Joint shear strain for JI-5

Figure 87. Joint principal strains for JIII-1

Figure 88. Joint shear strain for JIII-1

5.2.2 Behavior of joint reinforced with concrete filler-modules (JI-2)

Specimen JI-2, shown in Figure 45, was tested to evaluate the influence of concrete filler-modules with reinforcing dowels. As the beam loading began, at 2,899 lb, a jump in the strain gage reading on the reinforcing dowel embedded inside the filler-module has revealed a potential bond loss with the surrounding concrete due to the presence of stress-concentration around the dowels. At 9,067 lb, a crack in concrete close to the filler-module on the column face was observed. As the loading progressed to ~18,649 lb, a maximum shear strain of ~60 $\mu\epsilon$ (k-factor of 1.4) was recorded next to the crack location, as shown in Figure 90. Eventually, at 22,434 lb, the joint specimen failed in column shear, as shown in Figure 89. At the time of column failure, the strains in joint (JSH1)- and column (CSL2)- rebar were measured to be ~492 and ~241 $\mu\epsilon$, respectively. A summary of the experimental results is presented in Table 31. Compared to the control specimen JI-1, the improvements in load-carrying capacity and energy dissipation were noted to be ~45 and 35%, respectively. However, no change in the ductility index (section 5.3.4) was observed. Similar joint reinforcing scheme was introduced by [27] and [28] as planar joint enlargement and further evaluated by [62] under different test conditions with maximum strength increase of about 28% [62].

Although reinforcing joint specimen with concrete filler-modules prevented joint shear failure, the enhancement in load-carrying capacity was limited to the strength of the weakest member meeting at the joint, i.e., column. Premature failure of the column is attributed to the influence of stress concentration effects at the tip of filler-module, as shown in Figure 90. This behavior can be further improved by the addition of FRP wraps or gussets that arrest the crack growth and offer additional force resistance through confinement while lowering the stress concentration effects. Hence, further research (section 5.2.3) was carried out by reinforcing joint specimens with FRP wraps or gussets in addition to bonding filler-modules at reentrant corners.

Table 31. Summary of experimental results of joints reinforced with concrete filler-modules

Joint	PoI	Beam load (lb)	Column principal strain ($\mu\epsilon$)			Limit state		Steel rebar strain ($\mu\epsilon$)				Observed failure mode
			P(t)	P(c)	γ	k- P(t)	k- γ	JSH1	BSL1	BSL2	CSL2	
JI-2*	A	9,067	7	-14	20	0.42	0.49	28	204	-	1	Column shear
	B	11,542	9	-19	28	0.54	0.68	55	353	-	9	
	C	22,434	366	-23	-15	N.A.	N.A.	492	1,216	-	241	

PoI: Point of interest; P(t): principal tensile strain; P(c): principal compressive strain; γ : shear strain; *principal- and shear- strains measured on the column next to the filler-module, identified in Figure 90.

Note:

1. Principal strains are computed based on Eq. 4.1
2. Shear strains are computed based on Eq. 4.3
3. Upon concrete cracking (i.e., after point B), the stress distribution becomes increasingly complex, and hence, the Mohr's circle concept is not adequate to determine the principal stresses beyond this point.

Figure 89. Failure of JI-2 (front view)

Figure 90. Failure of JI-2 (rear view)

Figure 91. Column principal strains for JI-2

Figure 92. Column shear strain for JI-2

5.2.3 Behavior of joints reinforced with filler-modules and FRP wraps/gussets

A total of eight joint specimens (JI-3 through 8, JIII-2 through 4) were tested to evaluate the response of beam-column joints reinforced with filler-modules and FRP wraps/gussets under different parameters listed in Table 27. The following section provides a general overview of the behavior of test specimens, followed by a summary of experimental results related to each phase.

As the load on the beam increased, joint specimens reinforced with filler-modules and FRP wraps/gussets exhibited flexural-tension cracks at the tip of the filler-module, in the vicinity of beam section. This was visually realized through a cloud formation at the tip of the filler-module due to the local de-bonding of FRP. The computed strains on the concrete surface by extrapolating the beam longitudinal rebar strains (as explained in earlier sections) were found to be in the range of 184 - 441 $\mu\epsilon$, with “k” for flexural tension ranging from 10 - 25. This range is higher than the control specimens due to the additional tensile resistance offered by the FRP wrap/gusset. Furthermore, the flexural-tension cracks did not progress into the core concrete due to the presence of compressive stresses induced by the bottom filler-module, in addition to the (tensile) resistance against tension crack formation offered by longitudinal beam rebar and confinement effects from FRP wrap/gusset. At this stage of loading, the resisting force is redistributed on to the filler-modules through FRP wrap/gusset until the top filler-module is cracked in tension. These cracks initiated when the joint rebar strain reached 58 - 209 $\mu\epsilon$ and grew significantly when the strains reached 217 - 495 $\mu\epsilon$, revealing a controlled crack growth (because of wrap) due to the contribution of FRP wrap/gusset. Data from strain gages oriented along the longitudinal axis of the top filler-module revealed gradual cracking of filler-module at different stages of loading. The initial stage of filler-module cracking is referred to as point A and significant crack growth level (transition of force to column rebar) is referred to as point B. For design purposes, joints reinforced with filler-modules and FRP wrap/gusset are considered to behave in a linear elastic manner until point A. For operations under extreme event loading, point B is considered as idealized yield (defined as the point beyond which a truss mechanism primarily resists the forces through internal rebar cage, FRP wrap, and rebar-concrete bond forces) [63]. A truss mechanism primarily resists the induced force beyond point B. The test specimens finally failed either by beam flexure or column shear.

Based on the observed behavior of joint specimens under monotonic loading, the author believes that the response of test specimens (beam-column-joint, together) in terms of peak load, ductility, and energy dissipation capacity can be further enhanced by confining (with 360° FRP wraps) the beam and column sections (up to plastic hinge length), and then bonding the filler-modules, to utilize the full potential of refurbished joint sections. This approach may also help in containing the failure within the filler-module (outside of joint core), which can be easily replaced in case of distress during extreme events.

5.2.3.1 Behavior of joints reinforced with concrete/syntactic foam filler-modules and CFRP wraps (JI-3 and JI-4)

Two joint specimens (JI-3 and JI-4) were tested in the current phase to evaluate the response of beam-column joints reinforced with concrete/foam filler-modules and CFRP wraps. A summary of the experimental observations is presented in the following section.

1. The flexure-tension cracks appeared at the tip of the filler-module when the concrete strains reached 228 - 441 $\mu\epsilon$. The k-factor for flexural-tension cracks was computed to be in the range of 13 - 25. Based on the author's evaluation of FRP wrapped cylinders under split tension test, reported in section 4.4.6, the tensile strength of FRP wrapped cylinders was noted to be $\sim(20 - 28)\sqrt{f'_c}$. A summary of measured strains and computed k-factor is presented in Table 32.
2. As the loading increased resulting in internal stress redistribution, concrete cracking initiated in the top filler-module when the joint panel rebar strain reached 75 - 193 $\mu\epsilon$ and grew significantly when the rebar strains reached 217 - 329 $\mu\epsilon$ (i.e., at this strain level in the rebar, concrete reached its ultimate tensile strain).
3. The test specimens reached a peak load of ~60,773 - 64,107 lb and failed in beam-flexure. The measured rebar strains in the joint- and column- longitudinal reinforcement were found to be in the range of 1,175 - 2,263 $\mu\epsilon$ and 1,472 - 2,612 $\mu\epsilon$, respectively.
4. A summary of joint principal tensile- and shear- strains is presented in Table 33 and graphically shown in Figures 93 through 98.

5. Comparison of the principal- and shear- strains at points A and B for JI-3 and JI-4 reveals that the foam filler-module is more efficient in redistributing the forces between the joint, beam-, and column- rebars due to its comparatively higher strength properties and lower stiffness (higher strain to failure response) than concrete filler-module. Furthermore, it can be concluded that the high tensile strength of the top filler-module is influential in pushing the limits of Point A and B (i.e., prolonged elastic zone).
6. As seen from Figures 93 and 94, both the test specimens JI-3 and JI-4 failed in beam flexure, closer to the tip of the filler-module.

Table 32. Modulus of rupture of joints with filler-modules and CFRP wraps

Joint	Beam load (lb)	Beam rebar strain ($\mu\epsilon$)	Concrete strain ($\mu\epsilon$)	$k-f_r$
JI-1	2,475	106	158	9
	2,889	143	214	12
JI-3	7,094	152	228	13
	8,342	200	299	17
JI-4	8,229	169	254	14
	11,954	294	441	25

Table 33. Summary of experimental results of joints with concrete/syntactic foam filler-modules with/out CFRP wraps

Joint	PoI	Beam load (lb)	Joint principal strain ($\mu\epsilon$)			Limit state		Steel rebar strain ($\mu\epsilon$)				Observed failure mode
			P(t)	P(c)	γ	k- P(t)	k- γ	JSH1	BSL1	BSL2	CSL2	
JI-1	A	5,063	53	-94	-147	3.0	3.5	128	415	-	66	Joint Compression
	B	11,076	43	-261	-266	2.4	6.4	835	1,065	-	258	
	C	15,471	-	-	-	N.A.	N.A.	1,362	1,457	-	868	
JI-2*	A	9,067	7	-14	20	0.42	0.49	28	204	-	1	Column shear
	B	11,542	9	-19	28	0.54	0.68	55	353	-	9	
	C	22,434	366	-23	-15	N.A.	N.A.	492	1,216	-	241	
JI-3	A	21,976	75	-94	-166	4.27	4.00	75	-	855	28	Beam flexure
	B	27,308	107	-118	-221	6.08	5.33	217	-	1,116	84	
	C	60,773	1,990	-558	-2,447	N.A.	N.A.	1,175	-	4,809	1,472	
JI-4	A	21,574	22	-91	-110	1.23	2.65	193	-	664	114	Beam flexure
	B	23,970	40	-103	-137	2.25	3.32	329	-	773	135	
	C	64,107	1,005	-223	-1,182	N.A.	N.A.	2,263	-	2,608	2,612	

PoI: Point of interest; P(t): principal tensile strain; P(c): principal compressive strain; γ : shear strain; *principal- and shear- strains measured on the column next to the filler-module.

Note:

1. Principal strains are computed based on Eq. 4.1
2. Shear strains are computed based on Eq. 4.3
3. Upon concrete cracking (i.e., after point B), the stress distribution becomes increasingly complex, and hence, the Mohr's circle concept is not adequate to determine the principal stresses beyond this point.

Figure 93. Failure of JI-3

Figure 94. Failure of JI-4

Figure 95. Joint principal strains for JI-3

Figure 96. Joint shear strain for JI-3

Figure 97. Joint principal strains for JI-4

Figure 98. Joint shear strain for JI-4

5.2.3.2 Behavior of joints reinforced with concrete filler-modules and CFRP gussets (JI-6, JI-7, and JI-8)

Three joint specimens (JI-6, JI-7, and JI-8) were tested in the current phase to evaluate the response of beam-column joints reinforced with concrete filler-modules and CFRP gussets. Use of composite FRP gussets instead of wraps was investigated primarily for four reasons: (i) to enhance the quality of composite end product by adopting consistent manufacturing process, (ii) to avoid construction errors and provide ease of installation, (iii) to minimize downtime incurred during the field implementation process, and (iv) to reduce the hassle involved with fabric cutting and disposal of waste material on-site, i.e., gussets can be prefabricated. A summary of the experimental observations is presented in the following section.

1. The flexure-tension cracks appeared at the tip of the filler-module when the concrete strains reached 261 - 425 $\mu\epsilon$. The k-factor for flexural-tension cracks was computed to be in the range of 15 - 24. Based on the author's evaluation of FRP wrapped cylinders under split tension test, reported in section 4.4.6, the tensile strength of FRP wrapped cylinders was noted to be $\sim(20 - 28)\sqrt{f'_c}$. A summary of measured strains and computed k-factor is presented in Table 34.
2. As the loading increased resulting in internal stress redistribution, concrete cracking initiated in the top filler-module when the joint rebar strain reached 58 - 92 $\mu\epsilon$ and grew significantly when the strains reached 300 - 320 $\mu\epsilon$ (i.e., at this strain level in the rebar, concrete reached its ultimate tensile strain).
3. Based on the load vs. strain plots shown in Figures 99 and 100, some sort of bond-slip between the filler-module and underlying FRP was observed for specimens JI-7 and JI-8.
4. The test specimens reached a peak load of 51,018 - 72,040 lb and failed in column shear. The measured strains in the longitudinal joint- and column- reinforcement were found to be in the range of 1,126 - 1,427 $\mu\epsilon$ and 1,125 - 2,325 $\mu\epsilon$, respectively.
5. A summary of joint principal tensile- and shear- strains is presented in Table 35 and graphically shown in Figures 104 through 109.

6. Comparison of the principal- and shear- strain data of JI-6 and JI-7 revealed that the FRP fabric layer bonded to the beam and column sections before gluing filler-modules had a detrimental effect on the joint structural response (strength, ductility, and energy dissipation) due to the bond-slip between FRP layer and glue line at early stages of loading (~17,000 lb).
7. Comparison of the principal- and shear- strain data of JI-7 and JI-8 revealed that the use of high strength concrete filler-module (5472 vs. 9545 psi) on the tension side has very little influence on the joint response after point A due to its brittle nature and/or bond-slip between filler-modules and underlying FRP layers. As the crack in the top filler-module progresses between points A and B, a gradual increase in terms of the influence of joint response is seen due to the formation of compression strut, resulting from the high strength filler-module in the compression zone. This influence has been more pronounced after point B, due to the full engagement of the compression strut mechanism, resulting in better ductility and energy dissipation capacity of the joint specimen.
8. As shown in Figures 101 through 103, all the test specimens failed in column shear through the debonding of CFRP gussets. Besides, buckling of the column longitudinal rebars was noticed.

Table 34. Modulus of rupture of joints with filler-modules and CFRP gussets

Joint	Beam load (lb)	Beam rebar strain ($\mu\epsilon$)	Concrete strain ($\mu\epsilon$)	$k \cdot f_r$
JI-5	1,625	92	138	8
	2,035	141	212	12
JI-6	8,093	179	268	15
	11,030	283	425	24
JI-7	7,255	201	301	17
	9,338	281	421	24
JI-8	6,439	174	261	15
	9,008	277	416	24

Figure 99. Strain response along the longitudinal axis of filler-module (JI-7)

Figure 100. Strain response close to the filler-module tip on beam section (JI-8)

Table 35. Summary of experimental results of joints with filler-modules and CFRP gussets

Joint	PoI	Beam load (lb)	Joint principal strain ($\mu\epsilon$)			Limit state		Steel rebar strain ($\mu\epsilon$)				Observed failure mode
			P(t)	P(c)	γ	k- P(t)	k- γ	JSH1	BSL1	BSL2	CSL2	
JI-5	A	2,183	57	-39	86	3.3	2.1	-	156	93	37	Joint diagonal tension
	B	10,886	-	-	-	N.A.	N.A.	-	1,214	770	382	
	C	18,819	-	-	-	N.A.	N.A.	-	2,044	1,486	1,351	
JI-6	A	21,001	105	-166	265	6.01	6.41	85	354	811	-14	Column shear
	B	32,836	266	-350	605	15.14	14.62	312	861	1,384	113	
	C	58,961	1,958	-1,463	3,394	N.A.	N.A.	1,126	1,917	3,307	2,325	
JI-7	A	15,021	81	-80	154	4.63	3.72	92	431	540	2	Column shear
	B	25,638	221	-164	344	12.62	8.31	300	928	1,049	69	
	C	51,018	1,580	-943	2,473	N.A.	N.A.	1,242	1,997	2,833	1,392	
JI-8	A	20,413	91	-127	213	5.17	5.15	58	385	800	-34	Column shear
	B	33,316	259	-283	534	14.76	12.89	320	918	1,421	76	
	C	72,040	1,797	-1,829	3,597	N.A.	N.A.	1,427	2,396	11,426	1,125	

PoI: Point of interest; P(t): principal tensile strain; P(c): principal compressive strain; γ : shear strain

Note:

1. Principal strains are computed based on Eq. 4.1
2. Shear strains are computed based on Eq. 4.3
3. Upon concrete cracking (i.e., after point B), the stress distribution becomes increasingly complex, and hence, the Mohr's circle concept is not adequate to determine the principal stresses beyond this point.

Figure 101. Failure of JI-6

Figure 102. Failure of JI-7

Figure 103. Failure of JI-8

Figure 104. Joint principal strains for JI-6

Figure 105. Joint shear strain for JI-6

Figure 106. Joint principal strains for JI-7

Figure 107. Joint shear strain for JI-7

Figure 108. Joint principal strains for JI-8

Figure 109. Joint shear strain for JI-8

5.2.3.3 Behavior of joints reinforced with concrete filler-modules and GFRP wraps (JIII-2, JIII-3, and JIII-4)

Three joint specimens (JIII-2, JIII-3, and JIII-4) were tested in the current phase to evaluate the response of beam-column joints reinforced with concrete filler-modules and GFRP wraps. A summary of the experimental observations is presented in the following section.

1. The flexure-tension cracks appeared at the tip of filler-module when the concrete strains reached 184 - 374 $\mu\epsilon$. The k-factor for flexural-tension cracks was computed to be in the range of 10 - 21. Based on the author's evaluation of FRP wrapped cylinders under split tension test, reported in section 4.4.6, the tensile strength of FRP wrapped cylinders was noted to be $\sim(20 - 28)\sqrt{f'_c}$. A summary of measured strains and computed k-factor is presented in Table 36.
2. As the loading increased resulting in internal stress redistribution, concrete cracking initiated in the top filler-module when the joint rebar strain reached 71 - 209 $\mu\epsilon$ and grew significantly when the strains ranged from 434 - 495 $\mu\epsilon$ (i.e., at this strain level in the rebar, concrete reached its ultimate tensile strain).
3. The test specimens reached peak loads of ~53,072 - 77,391 lb and failed in column shear. The measured strains in the longitudinal joint- and column- reinforcement were found to be in the range of 1,919 - 2,445 $\mu\epsilon$ and 1,623 - 2,573 $\mu\epsilon$, respectively.
4. A summary of joint principal tensile- and shear- strains is presented in Table 37 and graphically shown in Figures 113 through 118.
5. Comparison of JIII-2 and JIII-3 revealed that the curvilinear filler-module exhibited better performance in redistributing the stresses around the joint panel, up to point B. This is because the joint response up to point A, and perhaps up to B, is governed by the top filler-module. Once the contribution of the top filler-module gets nearly ineffective beyond point B, the compression force is resisted by the bottom filler-module until it helps form a strut mechanism. Since the curvilinear filler-module has reduced thickness at its legs, it does not form an efficient strut (reduced width at its throat). Such strut of ineffective force resistance results in a slightly lower strength and energy dissipation capacity compared to a specimen

with wedge filler-module. The author believes that the performance of a joint reinforced with filler-modules and FRP wrap may be improved and/or optimized by providing a curvilinear filler-module on the tension side and wedge filler-module on the compression side.

6. Comparing JIII-3 with JIII-4 revealed that the specimen JIII-4 with U-anchors around the filler-modules certainly pushed point B to form at a higher load level (from 33,103 - 37,392 lb) by delaying the local de-bond of FRP wrap along the longitudinal axis of top filler-module, upon concrete cracking (internally) at point A. The increase in peak load from ~57,882 – 77,391 lb (JIII-3 vs. JIII-4) is attributed to the enhanced load resistance through strut formation from the bottom filler-module due to FRP confinement through a U-anchor. The delay in peak load, i.e., a longer plateau between points B and C, ensures superior ductility and energy dissipation capacity.
7. Figures 110 through 112 show the final failure of all the test specimens. Specimen JIII-2 and JIII-3 failed in a column shear after delamination of FRP wrap along the longitudinal axis of filler-module followed by crushing of concrete in the top filler-module. Specimen JIII-4 failed in column shear by the rupture of FRP wrapped around the column. Delamination of FRP from the surfaces of filler-module was prevented because of the U-anchors.

Table 36. Modulus of rupture of joints with filler-modules and GFRP wraps

Joint	Beam load (lb)	Beam rebar strain ($\mu\epsilon$)	Concrete strain ($\mu\epsilon$)	$k\cdot f_r$
JIII-1	2,014	91	137	8
	4,016	119	178	10
JIII-2	7,512	153	230	13
	10,454	235	352	20
JIII-3	9,318	190	285	16
	10,710	232	349	20
JIII-4	5,363	123	184	10 ⁺
	9,800	249	374	21

⁺Poor result may be attributed to local de-bond of FRP at the tip of filler-module at low load levels.

Table 37. Summary of experimental results of joints with filler-modules and GFRP wraps

Joint	PoI	Beam load (lb)	Joint principal strain ($\mu\epsilon$)			Limit state		Steel rebar strain ($\mu\epsilon$)				Observed failure mode
			P(t)	P(c)	γ	k- P(t)	k- γ	JSH1	BSL1	BSL2	CSL2	
JIII-1	A	7,076	70	-16	86	4.0	2.1	99	696	261	73	Joint/shear
	B	15,658	91	-142	72	5.2	1.8	1,023	1,646	829	212	
	C	24,740	-	-	-	N.A.	N.A.	1,573	2,535	1,535	1,171	
JIII-2	A	21,479	49	-116	151	2.78	3.64	209	938	654	31	Column shear
	B	27,364	100	-174	260	5.68	6.27	487	1,264	874	153	
	C	53,072	3,763	516	2,066	N.A.	N.A.	2,445	3,103	1,831	2,573	
JIII-3	A	26,502	146	-207	353	8.35	8.54	153	697	789	24	Column shear
	B	33,103	213	-301	514	12.13	12.40	434	1103	1,016	92	
	C	57,882	4,923	-1,576	6,377	N.A.	N.A.	1,919	2,223	1,946	2,009	
JIII-4	A	23,761	104	-109	209	5.91	5.04	71	480	770	-9	Column shear
	B	37,392	219	-201	408	12.50	9.85	495	1,139	1,288	195	
	C	77,391	6,003	-315	6,046	N.A.	N.A.	2,087	9,091	6,930	1,623	

PoI: Point of interest; P(t): principal tensile strain; P(c): principal compressive strain; γ : shear strain

Note:

1. Principal strains are computed based on Eq. 4.1
2. Shear strains are computed based on Eq. 4.3
3. Upon concrete cracking (i.e., after point B), the stress distribution becomes increasingly complex, and hence, the Mohr's circle concept is not adequate to determine the principal stresses beyond this point.

Figure 110. Failure of JIII-2

Figure 111. Failure of JIII-3

Figure 112. Failure of JIII-4

Figure 113. Joint principal strains for JIII-2

Figure 114. Joint shear strain for JIII-2

Figure 115. Joint principal strains for JIII-3

Figure 116. Joint shear strain for JIII-3

Figure 117. Joint principal strains for JIII-4

Figure 118. Joint shear strain for JIII-4

5.3 Discussion of Test Results

The primary objective of the experimental program was to evaluate the structural response (strength, stiffness, strain distribution, ductility index, energy dissipation, and failure mode) of joints reinforced with filler-modules with/out FRP wraps or gussets. Among the above responses, ductility index and energy dissipation capabilities are especially important while designing structures to resist extreme events such as earthquakes, wind, blast, and other dynamic forces. Therefore, the following sections 5.3.1 through 5.3.6 discussed the structural responses of reinforced joint specimens by comparing their behavior with the corresponding control specimens.

5.3.1 Strength

The load versus deflection responses of twelve joint specimens evaluated in the current study are shown in Figures 119 through 121. The loading was terminated when the test specimens either exhibited a sudden failure through the crushing of concrete or complete debonding of FRP wraps/gussets from the substrate, leading a sudden drop in the load-carrying capacity. A summary of the experimental results, including percentage increases with respect to corresponding control specimen and the change of failure modes of joints with concrete/foam filler-modules coupled with FRP wraps/gussets, is noted in Table 38.

All the joint reinforcing schemes were effective in enhancing the load-carrying capacity to as high as 314% over the control specimen. Specimens reinforced with filler-modules and FRP wraps/gussets exhibited higher increases in load-carrying capacity (~115-314%), while specimens reinforced with filler-modules alone have resulted in a nominal increase (~45%). The increase in load-carrying capacity of reinforced joints was governed by the rupture strength of strengthened beam or column member and the ability of rebar to develop plastic strains under confined conditions, at the location of hinge formation. For example, beam rebar of specimens JI-8 and JIII-4, presented in Table 42, exceeded the plastic strain limit and exhibited the highest strength increases in the corresponding batches. As shown in Figure 119, deflections measured at the ultimate/rupture of reinforced joints was as high as 5.6 in. compared to 1.8 in. for a control specimen, indicating an extended inelastic zone, resulting in superior strength, ductility and energy dissipation capacities.

Figure 119. Load vs. deflection response of joints with filler-modules with/without CFRP wraps

Figure 120. Load vs. deflection response of joints with filler-modules and CFRP gussets

Figure 121. Load vs. deflection response of joints with filler-modules and GFRP wraps

Table 38. Load vs. deflection response of joints with filler-modules and FRP wraps/gussets

Joint	Measured at Peak load			Observed failure mode
	Beam load (lb)	% increase	Δ (in)	
JI-1	15,471	Control	1.2	Joint compression
JI-2	22,434	45	1.2	Column shear
JI-3	60,773	293	2.4	Beam flexure
JI-4	64,107	314	3.6	Beam flexure
JI-5	18,819	Control	1.5	Joint diagonal tension
JI-6	58,961	213	2.3	Column shear
JI-7	51,018	171	2.4	Column shear
JI-8	72,040	283	2.8	Column shear
JIII-1	24,740	Control	1.3	Joint diagonal tension
JIII-2	53,072	115	1.9	Column shear
JIII-3	57,882	134	1.9	Column shear
JIII-4	77,391	213	3.0	Column shear

5.3.2 Failure mode

As identified in Table 38, all the control specimens (JI-1, JI-5, and JIII-1) exhibited brittle joint shear failure in concrete without the yielding of the beam or column rebars. Specimens JI-5 and JIII-1 exhibited diagonal tension failure while specimen JI-1 failed through the crushing of concrete (compression) in the joint core. The high axial load applied to the column of JI-1 did not allow the concrete to reach its full tensile capacity due to additional compressive (confinement) stresses induced on the joint panel, thus leading to the crushing of concrete. Although the failure of specimen JI-5 Figure 90 initiated through diagonal tension in the joint panel, it appears that the column portion above the joint panel (next to the top filler-module) has reached its shear capacity at the ultimate axial and bending load combinations. The newly developed joint reinforcement schemes with filler-modules with/out FRP wraps/gussets were effective in forcing the formation of plastic hinge away from the joint panel while ensuring the occurrence of the final failure in the column or beam sections. Furthermore, specimens (JI-3 and JI-4) reinforced with 360°-diagonal anchors around the joint exhibited beam flexural failure (Figure 93) as opposed to column shear failure (Figure 112) in specimens that are reinforced with no- or partial U-anchors. As shown in Figure 122, the buckling of longitudinal rebars in columns or beams was observed in all the reinforced joints depending upon the location of the final failure. Addition of filler-modules and FRP wraps/gussets enhanced the joint capacity well above the beam or column capacity; hence the failure was shifted to weakest member, i.e., column or beam meeting at a joint. To avoid catastrophic collapse, the optimal joint capacity with filler-modules and FRP wraps/gussets must be established to balance the capacities of column, beam, and joint.

Figure 122. Buckling of the column longitudinal rebar (JI-2)

5.3.3 Stiffness

Stiffness is defined as the ability of a structural member to resist deformation (translational, rotational, and shear) under applied loads. In the present study, the stiffness (tangent modulus) of beam-column joint specimens was determined by considering the slope of the initial load vs. deflection plot passing through the origin and connecting to the 30% of peak load value. It is generally accepted that concrete exhibits linear stress-strain response until 30% of the peak compressive stress. Following the definition adopted in the current study, a summary of the test results is presented in Table 39.

Table 39. Stiffness of joints reinforced with filler-modules and FRP wraps/gussets

Joint	Tangent modulus	% increase
JI-1	21,223	Control
JI-2	24,101	14
JI-3	29,027	37
JI-4	28,821	36
JI-5	23,025	Control
JI-6	31,737	38
JI-7	28,835	25
JI-8	37,803	64
JI-1	27,268	Control
JI-2	39,782	46
JI-3	37,975	39
JI-4	40,811	50

Figure 123. Cantilever beam with beam tip deflection

$$k \text{ (translational stiffness)} = \frac{3EI}{L^3} \dots\dots\dots(5.1)$$

$$k \text{ (rotational stiffness)} = \frac{EI}{L} \dots\dots\dots(5.2)$$

As seen from Figure 123 and Eq. 5.1 and 5.2, the stiffness of a structural member depends on the modulus of elasticity (E) of a material, the moment of inertia (I) of a section under consideration, and the span length (L). Based on the above understanding, the stiffness of a reinforced joint specimen is directly proportional to the material- and geometric- properties of the filler-module (depends on the E and I) and inversely proportional to the cube of the span length (depends on the location of hinge formation from the point of load application). Furthermore, the increase in stiffness also depends on the externally bonded FRP material properties and partially on the confinement effects from FRP and internal rebar cage.

As seen from Table 39, all the joint specimens reinforced with filler-modules with/out FRP wraps or gussets exhibited higher stiffness (up to 64%) compared to the control specimen. The increase in stiffness is smaller (~14%) for joints reinforced with filler modules alone, while it is higher in case of joints reinforced with high-strength concrete (9,545 psi) filler-modules and CFRP gussets (~64%). Comparing specimen JIII-3 (no anchor) and JIII-4 (with U-anchor) revealed that providing an anchor around the filler-module was influential in enhancing the stiffness of the joint specimen. Comparing specimen JI-6 with JI-7 revealed that bonding of FRP layers to the beam and column before gluing the filler-modules has a detrimental effect on the stiffness of joint specimen due to the potential slippage at the glue-line interface. Although an increase in the stiffness was noticed due to the addition of filler-modules, no consistent trend was noted in terms of material and geometric properties of filler-module because of the premature bond-slip at FRP/filler-module glue line.

5.3.4 Ductility index

The ductility of a structural member is defined as its ability to undergo large inelastic deformation without significant loss in the load-carrying capacity. The ductility index is defined as the ratio of the ultimate displacement (Δ_u) to the yield displacement (Δ_y). Since concrete structures do not exhibit yield behavior, the ductility index based on the idealized yield concept, as illustrated by [63] has been adopted in this study. According to this concept, the ductility index is defined as the ratio of the anticipated level of displacement (Δ_u) to the displacement at idealized yield (Δ_{yi}).

$$\mu = \Delta_u / \Delta_{yi} \dots\dots\dots(5.1)$$

Figure 124. Force-displacement relationship of a ductile structure [63]

Based on the above concept, the ductility index in this study was computed as the ratio of the ultimate displacement (i.e., displacement at point C) to idealized displacement (i.e., displacement at point B). In order to evaluate the ductility enhancement of joints reinforced with filler-modules and FRP wraps or gussets, the idealized displacement of corresponding control specimen was

considered as a reference point for all the joint specimens in a given batch. As per this definition, a summary of the ductility index for the twelve joint specimens is presented in Table 40.

Table 40. Ductility index of joints with filler-modules and FRP wraps/gussets

Joint	Peak values		Idealized yield	Ductility index ($\mu = \Delta_u / \Delta_{yi}$)
	Beam load (lb)	Δ_u (in)	Δ_{yi} (in)	
JI-1	15,471	1.2	0.6	2
JI-2	22,434	1.2	0.6	2
JI-3	60,773	2.4	0.6	4
JI-4	64,107	3.6	0.6	6
JI-5	18,819	1.5	0.5	3
JI-6	58,961	2.3	0.5	4
JI-7	51,018	2.4	0.5	4
JI-8	72,040	2.8	0.5	5
JII-1	24,740	1.3	0.6	2
JII-2	53,072	1.9	0.6	3
JII-3	57,882	1.9	0.6	3
JII-4	77,391	3.0	0.6	5

Note:

1. Typically, the ultimate displacement is taken at a 15% drop in the peak-load value. However, in this study, due to the lack of test data points beyond peak-load, the displacement at peak-load is considered as the ultimate displacement for computation of the ductility index.

The ductility index of reinforced joints was found to be as high as 6. It appears that reinforcing joint with filler-modules without FRP wraps (JI-2) did not offer any additional ductility beyond the control specimen (JI-1). The use of low-stiffness syntactic foam filler-module (JI-4) has profoundly influenced the enhancement of ductility index (6) compared to the concrete filler-module of high stiffness (JI-3). The improvement in ductility index of JI-4 over JI-3 is due to the near-uniform strain distribution in and around the joint panel of JI-4, as identified in Table 42. Strengthening JI-8 with high-strength filler-modules (9,545 psi) exhibited a slightly higher ductility index (5) than JI-7 with medium-strength filler-modules (5,472 psi). The improved behavior of JI-8 over JI-7 is primarily attributed to the additional strut resistance offered by the

higher strength filler-module (in the compression zone) in the inelastic region, i.e., between points B and C. The shape of the filler-modules, i.e., curve vs. wedge did not influence the ductility response of joint specimens. The confinement effect from the U-anchor on the compression filler-module of JIII-4 has provided additional strut resistance between points B and C, thus resulting in a higher ductility index.

5.3.5 Energy dissipation

Energy dissipation capability is an essential characteristic that a structure must exhibit when subjected to extreme events such as hurricanes and seismic forces. The energy dissipation capacity is computed as the area enclosed inside the load (P) vs. deflection (Δ) plot. The summation of consecutive areas computed at regular intervals of 0.1 in. deflection, measured up to the 15% drop in the peak-load, is defined as cumulative energy dissipation. However, in this study, the cumulative energy dissipation capacity of joint specimens is computed only up to the peak load. Therefore, it must be highlighted that there is a significant amount of energy dissipation (essential to prevent catastrophic failures) even beyond the peak load, as seen from load vs. deflection plots. The cumulative energy dissipation curves of all the twelve specimens are shown in Figures 125 through 127, and a summary is presented in Table 41.

Figure 125. Energy dissipation capacity of joints with filler-modules and CFRP wraps

Figure 126. Energy dissipation capacity of joints with filler-modules and CFRP wraps

Figure 127. Energy dissipation capacity of joints with filler-modules and GFRP wraps

Table 41. Energy dissipation capacity of joints with filler-modules and FRP wraps/gussets

Joint	Cumulative energy dissipated (lb.in)	% increase
JI-1	11,948	Control
JI-2	16,109	35
JI-3	80,930	577
JI-4	153,865	1,188
JI-5	20,513	Control
JI-6	77,062	276
JI-7	74,695	264
JI-8	122,018	495
III-1	19,184	Control
III-2	62,012	223
III-3	63,287	229
III-4	143,072	646

Specimen JI-2 reinforced with concrete filler-modules without FRP wraps/gussets dissipated 35% more energy compared to the control specimen (JI-1). The cumulative energy dissipated by specimens JI-3 and JI-4 was ~6 and 12 times higher than the control specimen (JI-1), respectively. The low stiffness of the syntactic-foam filler-modules in JI-4 has allowed the beam/column to flex more and contribute to the near-uniform distribution of strains in and around the joint panel, resulting in the highest energy dissipation capacity. The use of high-strength concrete filler-modules has allowed the specimen JI-8 to dissipate more energy (~5 times) compared to the control specimen JI-5. This behavior is attributed to the enhanced force resistance (strut formation) offered by the high-strength filler-modules in resisting compression forces between points B and C. The shape of the filler-modules did not seem to influence the energy dissipation capacity of joint specimens. The increase in strut strength through the confinement of the bottom filler-module using U-anchors has resulted in dissipating more energy (~6 times) compared to the control specimen III-1.

5.3.6 Strains in joint-, beam-, and column- rebars

In order to assess the strain progression from the beam- and column- rebars into the joint panel, several gages were mounted on steel rebars at critical locations, as shown in Figure 71. Results deduced from these strain gages at locations JSH1, BSL1, BSL2, and CSL2 are studied carefully and presented in Table 42 to assess the influence of various reinforcing parameters.

As expected, all the control specimens (JI-1, JI-5, and JIII-1) failed in the joint panel without yielding of the beam or column rebars. In all the three specimens, the strain gage location (BSL1) that is 3 in. away from the reentrant corner on the beam section exhibited higher strains indicating the formation of hinge next to the column face. This behavior can lead to the sudden progression of crack into the joint panel, causing brittle joint failure.

For specimen JI-2, the strains at BSL1 were lower than the control specimen (JI-1), indicating the distribution of strain over a certain length of rebar, i.e., the formation of hinge further away from the face of the column. However, the reinforcing technique adopted for JI-2 was unable to develop yield strains in the rebar, even at failure. Although specimen JI-3 failed through yielding of beam longitudinal rebar, it attracted more strains at BSL2 due to the use of stiffer concrete filler-module. The use of low stiffness syntactic foam filler-module in JI-4 has resulted in a near-uniform distribution of strains at all the locations, i.e., JSH1, BSL2, and CSL2, leading to a higher ductility index and energy dissipation capacity.

Specimens JI-6, JI-7, and JI-8 failed through the yielding of beam longitudinal rebar with higher strain values at BSL2 location. However, the use of a high-strength concrete filler-module in JI-8 has allowed the rebar to reach the plastic strain limit, resulting in higher ductility and energy dissipation capacities.

The use of wedge shape filler-modules in JIII-3 induced lower strains in the rebar at all the four locations (JSH1, BSL1, BSL2, and CSL2) compared to the curved filler-module in JIII-2. Confining the filler-modules of JIII-4 with U-anchors has resulted in superior performance by allowing the rebars to resist more forces by attaining the plastic strain limit.

Table 42. Rebar strain data of joints with filler-modules and FRP wraps/gussets

Joint	PoI	Beam load (lbs)	Rebar strain ($\mu\epsilon$)			
			JSH1	BSL1	BSL2	CSL2
JI-1	A	5,063	128	415	-	66
	B	11,076	835	1,065	-	258
	C	15,471	1,362	1,457	-	868
JI-2	A	9,067	28	204	-	1
	B	11,542	55	353	-	9
	C	22,434	492	1,216	-	241
JI-3	A	21,976	75	-	855	28
	B	27,308	217	-	1,116	84
	C	60,773	1,175	-	4,809	1,472
JI-4	A	21,574	193	-	664	114
	B	23,970	329	-	773	135
	C	64,107	2,263	-	2,608	2,612
JI-5	A	2,183	-	156	93	37
	B	10,886	-	1,214	770	382
	C	18,819	-	2,044	1,486	1,351
JI-6	A	21,001	85	354	811	-14
	B	32,836	312	861	1,384	113
	C	58,961	1,126	1,917	3,307	2,325
JI-7	A	15,021	92	431	540	2
	B	25,638	300	928	1,049	69
	C	51,018	1,242	1,997	2,833	1,392
JI-8	A	20,413	58	385	800	-34
	B	33,316	320	918	1,421	76
	C	72,040	1,427	2,396	11,426	1,125
JIII-1	A	7,076	99	696	261	73
	B	15,658	1,023	1,646	829	212
	C	24,740	1,573	2,535	1,535	1,171
JIII-2	A	21,479	209	938	654	31
	B	27,364	487	1,264	874	153
	C	53,072	2,445	3,103	1,831	2,573
JIII-3	A	26,502	153	697	789	24
	B	33,103	434	1,103	1,016	92
	C	57,882	1,919	2,223	1,946	2,009
JIII-4	A	23,761	71	480	770	-9
	B	37,392	495	1,139	1,288	195
	C	77,391	2,087	9,091	6,930	1,623

Red color indicates rebar yielding and attainment of plastic strains in some cases. Green color indicates the near-uniform distribution of strains in rebars.

5.4 Conclusions

A summary of the test results based on the experimental findings of joints reinforced with filler-modules and FRP wraps/gussets is presented in Table 43. Besides, the following sections (5.4.1.1 through 5.4.1.7) provide concluding remarks on various parameters evaluated in this study:

Table 43. Summary of test results: joints with filler-modules with/out FRP wraps or gussets

Joint	Peak load (lbs)	Ductility index	Energy dissipated (lb.in)	Failure mode
JI-1	15,471	2	11,948	Joint/compression
JI-2	22,434 (45)	2	16,109 (35)	Column shear
JI-3	60,773 (293)	4	80,930 (577)	Beam flexure
JI-4	64,107 (314)	6	153,865 (1,188)	Beam flexure
JI-5	18,819	3	20,513	Joint/diagonal tension
JI-6	58,961 (213)	4	77,062 (276)	Column shear
JI-7	51,018 (171)	4	74,695 (264)	Column shear
JI-8	72,040 (283)	5	122,018 (495)	Column shear
JII-1	24,740	2	19,184	Joint/diagonal tension
JII-2	53,072 (115)	3	62,013 (223)	Column shear
JII-3	57,882 (134)	3	63,287 (230)	Column shear
JII-4	77,391 (213)	5	143,073 (646)	Column shear

Numbers inside () indicate % increases with respect to the control specimen of a given batch.

5.4.1.1 Influence of concrete filler-modules without FRP wrap/gusset (JI-2)

Although reinforcing joints with concrete filler-modules has prevented the joint shear failure, the increases in strength and energy dissipation were limited to ~45% and 35%, respectively, when compared to the control specimens. This method of joint reinforcement did not allow the rebars to attain yield strains due to the premature column failure resulting from stress concentrations at the tip of filler-module and glue lines. This approach, if used to reinforce pre-1976 joints, especially weak column strong-beam designs, may lead to catastrophic collapse under extreme events.

5.4.1.2 Influence of CFRP wrap coupled with filler-modules (JI-2 vs. JI-3)

The addition of CFRP wrap to a joint specimen reinforced with concrete filler-modules has improved the strength, ductility, and energy dissipation by ~170%, 100%, and 402%, respectively, over specimen JI-2. This improvement is primarily due to the additional tensile resistance offered by the CFRP wrap in addition to the concrete confinement effect. This approach of reinforcing joints with CFRP wrap and 360°-anchor around the filler-modules has shifted the non-ductile column failure mode to a more ductile beam flexural failure.

5.4.1.3 Influence of concrete vs. foam filler-modules with CFRP wraps (JI-3 vs. JI-4)

Comparing the structural responses of joints reinforced with concrete filler-modules (JI-3) and syntactic foam filler-modules (JI-4) coupled with CFRP wraps revealed that the specimen JI-4 has exhibited superior strength, ductility, and energy dissipation capabilities. The low-stiffness property of the syntactic foam filler-module has allowed the adjoining beam and column members to flex more in unison with the rotational values of the beam and column meeting at the joint (instead of creating stress risers) and contribute to the near-uniform distribution of strains in and around the joint panel, resulting in a superior behavior. The use of syntactic foam filler-modules coupled with CFRP wraps to reinforce deficient joints may be essential to withstand extreme events and prevent catastrophic collapses.

5.4.1.4 Influence of medium- versus high- strength concrete filler-modules with CFRP gussets (JI-7 vs. JI-8)

Both the medium- and high- strength concrete filler-modules significantly improved the structural response of joints. The higher the strength of filler-module, the longer the plateau between points B and C, i.e., more prolonged inelastic zone. This behavior implies that the use of high-strength filler-modules will provide better ductility and energy dissipation capacity due to enhanced compressive force resistance (strut formation) offered by the bottom filler-module, while the steel cage tends to reach its maximum tensile force resistance. Besides, this will also delay the initial cracking of filler-module in the tension zone and push the occurrence of points A and B at higher load levels.

5.4.1.5 Influence of curve versus wedge filler-modules with GFRP wraps (JIII-2 vs. JIII-3)

The shape of the filler-modules did not significantly alter the behavior of joints in terms of strength, ductility, and energy dissipation capacities (< 9% variation). The low variation in test results is because the final failure was governed by the debonding of FRP wrap along the filler-module profile, followed by the crushing of concrete in the top filler-module.

5.4.1.6 Influence of CFRP U-anchor for filler-modules (JIII-3 vs. JIII-4)

The use of CFRP U-anchors around the filler-modules of JIII-4 significantly improved the strength, ductility, and energy dissipation capacities by 213%, 150%, and 646%, respectively, over the control specimen (JIII-1). The confinement effect from the U-anchor has prevented the filler-modules from crushing and allowed the rebar to attain plastic strains. This behavior has created a more prolonged inelastic zone, i.e., a longer plateau between points B and C thus, resulting in superior structural response over unconfined specimens.

5.4.1.7 Influence of hinge formation within- versus outside- the filler module length

Comparing the rebar strains in joints (JI-6 through JI-8) with (JIII-2 through JIII-4) reveals that the formation of hinge within the filler-module length is more beneficial in improving the strength, ductility, energy dissipation capacities. The formation of hinge within the filler-module length will offer additional resistance to induced forces because of the higher moment of inertia at that section.

6. Behavior of RC Joints Reinforced with Engineered Wood (PSL) Filler-Modules and FRP Wrap

6.1 Introduction

This chapter dealt with the structural response evaluation of RC beam-column joints reinforced with PSL wood filler-modules and fiber-reinforced polymer (FRP) composite wraps under monotonic loads. The joint without filler-modules is identified as control specimens, herein. The joint testing has been conducted to evaluate the influence of various parameters, as listed in Table 44. The behavior of joints reinforced with PSL filler-modules and FRP wraps was studied in a similar manner as elaborated in section 5.1. Furthermore, this chapter presented the experimental results (section 6.2) followed by discussion on test results (section 6.3) with emphasis on strength, stiffness, ductility index, strain distribution in rebars, energy dissipation, and failure modes. Finally, the conclusions based on the experimental observations are presented in section 6.4.

Table 44. Parameters tested to evaluate the performance of joints reinforced with PSL filler-modules and FRP wraps

Phase #	Joint	f'_c	CL	Reinforcing scheme
Phase IV	JII-1	3309 psi	$0.21f'_cA_g$	Control
	JII-2			CFRP Wrap
	JII-3			PSL-C+D
	JII-4			PSL-W+D
	JII-5			PSL-C+CFRP Wrap+360°-anchor
	JII-6			PSL-C+D+CFRP Wrap+360°-anchor
	JII-7			PSL-W+D+CFRP Wrap+360°-anchor
	JII-8			PSL-W+D+GFRP Wrap+360°-anchor

D: dowels; PSL: Parallel strand lumber; C: curve; W: wedge; CL: column axial load; f'_c : substrate concrete compressive strength; A_g : gross cross-sectional area of concrete (column)

6.2 Experimental Results

This section describes the structural response of joint specimens tested in control and reinforced (joints with PSL filler-modules and FRP wraps) conditions.

6.2.1 Behavior of control specimen

Figure 128. Failure of specimen JII-1

Specimen JII-1 was tested as a control specimen to establish the behavior of pre-1976 joints under monotonic loading and compare with reinforced joints, i.e., joints with filler-modules and FRP wraps. As the beam loading was applied, the specimen exhibited corner cracking at the interface of the beam and column. At the time of corner cracking, the strains on the rebar and concrete surface were noted to be 103-139 $\mu\epsilon$ and 154-208 $\mu\epsilon$, respectively. The load levels corresponding to these strains are provided in Table 45. As the loading on the beam continued, diagonal tension crack on the joint panel appeared and eventually led to joint shear failure with concrete cover delamination, as shown in Figure 128. A summary of the joint principal- and shear- strains measured during the testing are tabulated in Table 46. The beam load versus joint principal- and shear- strains are graphically shown in Figures 129 and 130, respectively.

Figure 129. Joint principal strains in JII-1

Figure 130. Joint shear strain in JII-1

6.2.2 Behavior of joint reinforced with CFRP wrap

Figure 131. Failure of specimen JII-2

Specimen JII-2 was strengthened using CFRP wraps in the joint panel. The main objective of this reinforcing scheme was to evaluate the effectiveness of CFRP wraps in preventing joint failure and enhancing the strength, ductility, and energy dissipation over the control specimen.

As the beam loading was applied, the flexure-tension cracks appeared close to the tip of the filler-module on the beam section when the concrete tensile strains reached $154\text{--}429\ \mu\epsilon$. The k-factor for flexural-tension cracks was computed to be in the range of 9-24, as reported in Table 45. Based on the author's evaluation of FRP wrapped cylinders under split tension test, the tensile strength of FRP wrapped cylinders was found to be $\sim(20\text{--}28)\sqrt{f'_c}$, as reported in section 4.4.6. As the loading continued, the crushing of concrete was delayed due to the confinement effect from CFRP wrap, thus allowing the rebar cage to resist additional tensile forces by yielding of rebars. Eventually, the specimen exhibited joint failure through the rupture of FRP at the joint corner, as shown in Figure 131. Although specimen JII-2 exhibited a similar failure to JII-1, the failure is contained due to the confinement effect from CFRP wraps, which resulted in plastic yielding ($\sim 14,330\ \mu\epsilon$) of beam longitudinal rebar closer to the joint panel. However, this approach of FRP wrapping around a joint panel in a real building scenario is not practically feasible due to the

presence of transverse beams and slab in two or more directions. A summary of the joint principal- and shear- strains measured during the testing are tabulated in Table 46. The beam load versus joint principal- and shear- strains are graphically shown in Figures 132 and 133, respectively.

Figure 132. Joint principal strains in JII-2

Figure 133. Joint shear strains in JII-2

6.2.3 Behavior of joints reinforced with PSL curve- and wedge- filler-modules

Figure 134. Failure of specimen JII-3 (left) and JII-4 (right)

Specimens JII-3 and JII-4 were tested to evaluate the influence of filler-module shape on the joint structural behavior. For this reason, specimen JII-3 was reinforced with PSL curve filler-modules and dowels, while specimen JII-4 was reinforced with wedge filler-modules and dowels.

Both the specimens JII-3 and JII-4 exhibited some distress at the tip of the filler-module during the initial stages of loading. The load values and corresponding strains in concrete and steel rebar at the time of distress are reported in Table 45. As the loading progressed further, the specimen JII-3 exhibited diagonal cracking in the joint panel and eventually merged with column shear crack, as shown in Figure 134. Finally, the specimen JII-3 failed through the spalling of concrete cover and buckling of longitudinal column rebars. At failure, the splitting of wood fibers on the top filler-module was noticed.

Specimen JII-4 failed in column shear next to the top filler-module, as shown in Figure 134. This crack originated at the tip of the filler-module due to the influence of stress-concentration and progressed into the column core. At failure, buckling of longitudinal rebars was observed along with side-splitting of column cover concrete. Comparing the failure modes of both specimens revealed that the wedge filler-module had attracted more stresses at its tip, unlike the curve filler-modules, which exhibited a better stress distribution between the joint and members meeting at a

joint. A summary of the joint principal- and shear- strains measured during the testing are tabulated in Table 46. The beam load versus column principal- and shear- strains next to the crack location for both specimens JII-3 and JII-4 are graphically shown in Figures 135 through 138.

Figure 135. Column principal strain in JII-3

Figure 136. Column shear strain JII-3

Figure 137. Column principal strain in JII-4

Figure 138. Column shear strain in JII-4

6.2.4 Behavior of joint reinforced with PSL curve filler-modules and CFRP wraps without and with reinforcing dowels

Figure 139. Failure of specimen JII-5 (left) and JII-6 (right)

Specimens JII-5 and JII-6 were tested to evaluate the influence of dowels on the structural behavior of joint reinforced with PSL filler-modules and CFRP wraps. For this reason, specimen JII-5 was reinforced with PSL curve filler-modules and CFRP wrap without dowels, while specimen JII-6 was reinforced with PSL curve filler-modules, CFRP wrap, and dowels.

Lack of dowels in specimen JII-5 has resulted in a joint shear failure due to the premature debonding of top filler-module, as opposed to a column shear failure in specimen JII-6 with dowels. The presence of dowels in JII-6 provided better composite action between the filler-modules and the members meeting at a joint through the efficient transfer of shear forces. The composite action developed due to dowel bars has distributed the stresses over a large area and reduced the shear demand in the joint panel., thus avoiding joint failure. Although specimen JII-5 exhibited joint shear failure, the confinement from 360° CFRP wrap around the joint has resulted in better strength and energy dissipation compared to the control specimen. A summary of the joint principal- and shear- strains measured during the testing are tabulated in Table 46. The beam load

versus joint principal- and shear- strains for both specimens JII-5 and JII-6 are graphically shown in Figures 140 through 143.

Figure 140. Joint principal strain in JII-5

Figure 141. Joint shear strain in JII-5

Figure 142. Joint principal strain in JII-6

Figure 143. Joint shear strain in JII-6

6.2.5 Behavior of joint reinforced with PSL wedge filler-modules and CFRP/GFRP wraps

Figure 144. Failure of specimen JII-7 (left) and JII-8 (right)

Specimens JII-7 and JII-8 were tested to evaluate the influence of CFRP versus GFRP on the structural behavior of joints reinforced with PSL wedge filler-modules. For this reason, specimen JII-7 was reinforced with PSL wedge filler-modules, dowels, and CFRP wrap, while specimen JII-8 was reinforced with PSL wedge filler-modules, dowels, and GFRP wrap.

As expected, the wedge filler-modules in both specimens attracted slightly higher strains close to the tip at every stage of beam loading. This response is evident from the rebar strain data reported in Table 52. It must be noted that both, CFRP and GFRP specimens exhibited similar strain response until the load on JII-7 was abruptly terminated due to the slippage of test specimen from the setup. As seen from Figure 144, both specimens JII-7 and JII-8 exhibited beam hinging. However, the use of GFRP wrap in JII-8 provided better stiffness compatibility with the underlying substrate resulting in additional force resistance until the beam longitudinal rebars at the 90° hook in the joint panel were fractured. A summary of the joint principal- and shear- strains measured during the testing are tabulated in Table 46. The beam load versus joint principal- and shear- strains for both specimens JII-7 and JII-8 are graphically shown in Figures 145 through 148.

Figure 145. Joint principal strain in JII-7

Figure 146. Joint shear strain in JII-7

Figure 147. Joint principal strain in JII-8

Figure 148. Joint shear strain in JII-8

Table 45. Modulus of rupture of joints reinforced with PSL filler-modules and FRP wraps

Joint	Beam load (lb)	Beam rebar strain ($\mu\epsilon$)	Concrete strain ($\mu\epsilon$)	$k-f_r$
JII-1	1,558	103	154	9
	2,005	139	208	12
JII-2	3,667	100	150	9
	6,111	286	429	24
JII-3	2,815	114	171	10
	4,759	225	338	19
JII-4	3,161	124	185	11
	4,788	219	329	19
JII-5	3,895	125	188	11
	6,131	251	376	21
JII-6	5,561	126	189	11
	8,151	216	325	19
JII-7	5,745	144	216	12
	7,812	232	348	20
JII-8	5,358	120	180	10
	8,193	228	343	20

Table 46. Summary of experimental results of joints reinforced with PSL filler-modules and FRP wraps

Joint	PoI	Beam load (lb)	Joint principal strain ($\mu\epsilon$)			Limit state		Steel rebar strain ($\mu\epsilon$)				Observed failure mode
			P(t)	P(c)	γ	k- P(t)	k- γ	JSH1	BSL1	BSL2	CSL2	
JII-1	A	12,724	56	-72	-81	3.2	2.0	931	1,040	N.A.	410	Joint shear/ diagonal tension
	B	13,295	64	-65	-51	3.6	1.2	1,021	1,110	N.A.	478	
	C	20,900	-	-	-	-	-	1,947	1,750	N.A.	1,380	
JII-2	A	10,808	202	-587	342	11.5	8.3	416	704	551	176	Joint shear failure
	B	14,995	583	-1414	875	33.2	21.1	820	1,060	812	350	
	C	35,273	4,125	-217	-3,788	N.A.	N.A.	3,235	14,330	2,311	1,909	
JII-3	A	11,289	64	-2	-31	3.7	0.7	132	462	712	34	Joint shear failure
	B	16,028	75	2	-27	4.3	0.7	434	818	1,135	66	
	C	31,695	-61	-454	187	-	-	2,142	1,858	2,533	1,877	
JII-4	A	15,404	45	12	33	2.6	0.8	202	630	1,053	86	Column shear failure
	B	17,668	54	13	41	3.1	1.0	290	809	1,217	110	
	C	34,989	-	-	-	-	-	1,595	2,014	2,776	2,651	
JII-5	A	12,014	216	-332	522	12.3	12.6	137	474	648	93	Joint failure
	B	23,219	753	-1,344	2,003	42.9	48.4	1,061	1,289	1,459	355	
	C	41,868	1,494	-4,071	4,559	85.2	110.1	2,061	3,298	7,412	2,680	
JII-6	A	29,512	286	-38	60	16.3	1.5	370	825	1,223	81	Column shear failure
	B	34,063	407	-30	48	23.2	1.2	522	1,048	1,534	106	
	C	61,592	1,984	802	-616	113.1	14.9	1,909	2,797	14,627	1,364	
JII-7	A	28,390	205	-478	89	11.7	2.2	445	1,044	1,516	90	Beam flexural failure
	B	38,946	550	-940	184	31.3	4.4	816	1,612	2,188	232	
	C	56,894	1,027	302	723	58.5	17.5	1,784	11,519	16,638	1,870	
JII-8	A	30,612	491	-58	-284	28.0	6.8	550	1,218	1,605	222	Beam flexural failure
	B	36,706	785	-62	-440	44.7	10.6	797	1,549	2,007	359	
	C	59,320	3,223	285	-1,190	183.7	28.7	2,340	16,375	22,325	2,157	

PoI: Point of interest; P(t): principal tensile strain; P(c): principal compressive strain; γ : shear strain.

6.3 Discussion of Test Results

To assess the effectiveness of joints reinforced with PSL filler-modules and FRP wraps in various configurations, the most critical parameters relevant for the structural response of joint specimens under monotonic loading are discussed in sections 6.3.1 through 6.3.6. The structural responses of joint specimens discussed herein include strength, stiffness, strain distribution, ductility index, energy dissipation, and failure modes. The assessment of the structural response of reinforced joint specimens was performed by comparing their response with the control specimen.

6.3.1 Strength

The load versus deflection of all the eight joint specimens is summarized in Table 47 and graphically shown in Figure 149. As seen from Table 47, all the joint reinforcement configurations evaluated in this study were proved to be effective in enhancing the load-carrying capacity to a maximum of 195% over the control specimen.

Table 47. Load vs. deflection of joints reinforced with PSL filler-modules and FRP wraps

Joint	Measured at peak load			Observed failure mode
	Beam load (lb)	% increase	Δ (in)	
JII-1	20,900	Control	1.3	Joint/diagonal tension
JII-2	35,273	69	2.4	Joint shear failure
JII-3	31,695	52	1.8	Joint/column shear failure
JII-4	34,989	67	1.7	Column shear failure
JII-5	41,868	100	2.9	Joint failure
JII-6	61,592	195	3.2	Column shear failure
JII-7*	56,894	172	3.4	Beam flexural failure
JII-8	59,320	184	4.6	Beam flexural failure

* Loading was terminated abruptly before reaching the ultimate load due to the slippage of the specimen from the test setup.

Specimen JII-2, reinforced with CFRP wrap without filler-modules, exhibited a 69% increase in load-carrying capacity over the control specimen. Although an increase in the load-carrying

capacity was noted for JII-2, failure was still concentrated in the joint panel. This increase in the load-carrying capacity of JII-2 is because of the confinement from the CFRP wrap, which delayed the crushing of concrete and allowed the steel rebars to resist additional tensile forces.

Figure 149. Load vs. deflection of joints reinforced with PSL filler-modules and FRP wraps

Among the specimens reinforced with filler-modules, the specimen JII-4 having wedge filler-modules exhibited a 67% increase in the load-carrying capacity as opposed to a 52% increase for specimen JII-3 with curved filler-modules. The additional increase in load-carrying capacity for JI-4 is attributed to the better compression (strut) resistance offered by the wedge filler-module when compared to a curved filler-module.

The use of dowels in specimen JII-6 significantly contributed to the increase in the load-carrying capacity (~47% higher) when compared to the specimen JII-5 without dowels. This increase reveals the significance of dowels in providing composite action between the filler-modules and underlying substrate, offering additional resistance through the efficient transfer of shear forces over a larger area of the beam, column, and joint sections.

Specimens reinforced with CFRP wraps (JII-7) and GFRP wraps (JII-8) exhibited ~184% and 172% increases in load-carrying capacity, respectively. No specific discussions were made comparing the contributions of CFRP vs. GFRP wraps because of the early termination of load for specimen JII-7.

6.3.2 Failure mode

The failure modes of joint specimens observed during the lab testing are presented in Table 48. As expected, the control specimen without transverse (stirrup) reinforcement exhibited joint shear failure before rebar yielding, which is commonly seen in buildings collapsed under earthquakes.

Specimen JII-2, reinforced with CFRP wrap, exhibited joint shear failure through the rupture of FRP at the reentrant corner in the tension zone. This behavior reveals the need for installing filler-modules at joint corners to redistribute stresses and avoid such failures.

Failure in specimen JII-3 initiated as a diagonal cracking in the joint panel but ultimately merged with column shear cracks, resulting in early loss of load-carrying capacity. The initiation of cracking in the joint panel reveals the need for using a filler-module of high-strength with increased throat depth to redistribute forces over a larger area of the specimen. In the case of joint reinforced with wedge filler-modules (JII-4), the failure initiated at the tip of the filler-module next to the column and progressed as a shear crack into the column panel. The initiation of cracking at the tip of the filler-module can be delayed or prevented by bonding FRP wraps in a direction perpendicular to the orientation of the cracks and further confining with 360°- or U- anchors. At failure, buckling of column longitudinal rebars was seen in both specimens JII-3 and JII-4.

Lack of dowels in specimen JII-5 has resulted in a joint shear failure due to the premature debonding of filler-module, as opposed to a column shear failure in specimen JII-6 with dowels. The joint shear failure in specimen JII-5 has resulted in a small increase of structural capacities compared to specimen JII-6.

Both specimens JII-7 and JII-8 reinforced with CFRP and GFRP wraps, respectively, exhibited beam flexural cracking. Besides, for specimen JII-8, splitting of wood fibers in the top filler-module along with the fracture of beam longitudinal rebars at the 90° hook was noticed. The superior performance of JII-8 is attributed to the better bond ability of GFRP to the underlying substrate (i.e., because of better stiffness compatibility between GFRP and substrate).

Note: Specimen JII-7 would have exhibited better response, provided the specimen was loaded until final failure.

Table 48. Failure modes of joints reinforced with PSL filler-modules and FRP wraps

Joint	Observed failure mode	Description of failure mode
JII-1		Joint shear/ diagonal tension
JII-2		Joint shear failure
JII-3		Column/joint shear failure

Joint	Observed failure mode	Description of failure mode
JII-4		Column shear failure
JII-5		Joint/beam failure
JII-6		Column shear failure

Joint	Observed failure mode	Description of failure mode
JII-7*		Beam flexural failure
JII-8		Beam flexural failure

* Loading was terminated abruptly before reaching the ultimate load due to the slippage of the specimen from the test setup.

6.3.3 Stiffness

The stiffness of each of the eight joint specimens was computed as defined in section 5.3.3. Furthermore, a summary of the computations is presented in Table 49.

Table 49. Stiffness of joints reinforced with PSL filler-modules and FRP wraps

Joint	Tangent modulus	% increase
JII-1	28,176	Control
JII-2	25,517	-9
JII-3	30,010	7
JII-4	32,036	14
JII-5	29,665	5
JII-6	29,250	4
JII-7*	31,106	10
JII-8	30,401	8

* Loading was terminated abruptly before reaching the ultimate load due to the slippage of the specimen from the test setup.

As seen in Table 49, the maximum increase in stiffness of joint specimens due to the addition of filler-modules with or without FRP wraps was found to be ~14%. The relatively small increase in stiffness is attributed to the low modulus of elasticity of PSL wood filler-modules parallel to the wood fiber orientation, which did not stiffen the beams, thus resulting in higher beam tip deflections.

6.3.4 Ductility index

The ductility index of all the joint specimens was computed as defined in section 5.3.4. Furthermore, a summary of the computations is presented in Table 50.

Table 50. Ductility index of joints reinforced with PSL filler-modules and FRP wraps

Joint	Measured at peak load		Idealized yield	Ductility index ($\mu = \Delta_u / \Delta_{yi}$)
	Beam load (lb)	Δ_u (in)	Δ_{yi} (in)	
JII-1	20,900	1.3	0.6	2
JII-2	35,273	2.4	0.6	4
JII-3	31,695	1.8	0.6	3
JII-4	34,989	1.7	0.6	3
JII-5	41,868	2.9	0.6	5
JII-6	61,592	3.2	0.6	5
JII-7*	56,894	3.4	0.6	6
JII-8	59,320	4.6	0.6	8

* Loading was terminated abruptly before reaching the ultimate load due to the slippage of the specimen from the test setup.

All the reinforcing schemes adopted in this study for joint strengthening were proved to be efficient in enhancing the ductility index over the control specimen. For specimen JII-2, the confinement effect from CFRP wrap contained the concrete from crushing and allowed the rebar cage to offer more resistance against induced tensile forces, thus resulting in additional deformation before the collapse. This increase of deformation in the inelastic zone due to CFRP confinement has resulted in a better ductility index over the control specimen.

The use of low-stiffness PSL wood filler-modules for reinforcing joint specimens JII-3 and JII-4 resulted in a better ductility index over the control specimen. This improvement in the ductility index of specimens JII-3 and JII-4 is attributed to the additional compressive force resistance (strut formation) offered by the bottom filler-module between the points B and C before any sudden failure taking place due to weakness of oriented material like engineered wood. Although, slightly better strain distribution in the rebars was observed for joint specimen reinforced with curve filler-modules over wedge filler-modules, no appreciable difference in the ductility index was noticed.

Comparing specimens JII-5 and JII-6 reveal that the use of dowels in JII-6 does not influence the ductility index. Although specimen JII-5 without dowels exhibited premature debonding of the top filler-module, the confinement effect from 360°-anchor around the filler-module and joint helped resist the forces in the inelastic zone without resulting in sudden collapse.

Among all the reinforcing schemes except for specimen JII-7, specimen JII-8 exhibited the highest ductility index of 8. The enhanced compositeness between GFRP wraps and the substrate due to the better strain compatibility of constituent materials resulted in shifting the joint/column failure to a ductile beam flexural failure, thus leading to a superior response of JII-8 over other reinforcing schemes.

Note: * Loading was terminated abruptly before reaching the ultimate load due to the slippage of the specimen from the test setup.

6.3.5 Energy dissipation

The cumulative energy dissipation capacity of each of the eight joint specimens was computed as defined in section 5.3.5. Furthermore, a summary of the computations is presented in Table 51 and graphically shown in Figure 150.

Table 51. Energy dissipation of joints reinforced with PSL filler-modules and FRP wraps

Joint	Cumulative energy dissipated (lb.in)	% increase
JII-1	17,458	Control
JII-2	56,373	223
JII-3	37,373	114
JII-4	37,488	115
JII-5	85,476	390
JII-6	126,654	625
JII-7*	133,506	665
JII-8	204,972	1,074

* Loading was terminated abruptly before the complete failure of the specimen.

Figure 150. Energy dissipation of joints reinforced with PSL filler-modules and FRP wraps

Specimen JII-2, reinforced with CFRP wraps without filler-modules, dissipated 223% more energy compared to the control specimen JII-1. The increase in energy dissipation capacity of JII-2 over JII-1 is attributed to the additional tensile resistance offered by the rebar cage due to the confined concrete effect from CFRP wrap between points B and C. No significant difference was observed in the energy dissipation capacity of specimens JII-3, and JII-4 reinforced with curve and wedge filler-modules, respectively. It must be highlighted that the presence of dowels alleviated the specimen JII-6 to dissipate 48% more energy compared to the specimen JII-5 without dowels. This response reveals that the presence of dowels ensured efficient composite action between the beam, column, and joint through shear transfer, thus resulting in a superior structural capacity. Among the joint specimens reinforced with filler-modules and FRP wraps, specimen JII-8 with GFRP wrap exhibited the highest (~11 times) energy dissipation capacity. The superior performance of JII-8 reveals that the strain/stiffness compatibility between FRP wraps and the underlying substrate is essential for the structures to dissipate more energy under earthquake forces, especially in the inelastic zone.

6.3.6 Strains in joint-, beam-, and column- rebars

In order to assess the strain progression from the beam- and column- rebars into the joint panel, several gages were mounted on steel rebars at critical locations, as shown in Figure 71. Results deduced from these strain gages at locations JSH1, BSL1, BSL2, and CSL2 are studied carefully and presented in Table 52 to assess the influence of various reinforcing schemes.

Table 52. Rebar strain data of joints reinforced with PSL filler-modules and FRP wraps

Joint	PoI	Beam load (lbs)	Rebar strain ($\mu\epsilon$)			
			JSH1	BSL1	BSL2	CSL2
JII-1	A	12,724	931	1,040	N.A.	410
	B	13,295	1,021	1,110	N.A.	478
	C	20,900	1,947	1,750	N.A.	1,380
JII-2	A	10,808	416	704	551	176
	B	14,995	820	1,060	812	350
	C	35,273	3,235	14,330	2,311	1,909
JII-3	A	11,289	132	462	712	34
	B	16,028	434	818	1,135	66
	C	31,695	2,142	1,858	2,533	1,877
JII-4	A	15,404	202	630	1,053	86
	B	17,668	290	809	1,217	110
	C	34,989	1,595	2,014	2,776	2,651
JII-5	A	12,014	137	474	648	93
	B	23,219	1,061	1,289	1,459	355
	C	41,868	2,061	3,298	7,412	2,680
JII-6	A	29,512	370	825	1,223	81
	B	34,063	522	1,048	1,534	106
	C	61,592	1,909	2,797	14,627	1,364
JII-7	A	28,390	445	1,044	1,516	90
	B	38,946	816	1,612	2,188	232
	C	56,894	1,784	11,519	16,638	1,870
JII-8	A	30,612	550	1,218	1,605	222
	B	36,706	797	1,549	2,007	359
	C	59,320	2,340	16,375	22,325	2,157

Red color indicates rebar yielding and attainment of plastic strains in some cases.

As expected, the control specimens (JII-1) failed in the joint panel without yielding of the beam or column rebar. The strain gage location (BSL1) that is 3 in. away from the reentrant corner on the beam section exhibited higher strains indicating the formation of hinge next to the column face. This behavior can lead to the sudden progression of crack into the joint panel, causing brittle joint failure.

Specimen JII-2, confined with CFRP wrap, failed in joint shear after yielding (reached plastic strain) of beam longitudinal rebar at BSL1. Yielding in rebar occurred before the attainment of peak-load.

Specimens JII-3 reinforced with curved PSL filler-modules exhibited near-uniform strain distribution in the rebars of the beam, column, and joint compared to specimen JII-4 reinforced with wedge filler-modules. The strain in rebar location BSL2 for specimen JII-4 yielded at the peak load.

In all the joint specimens reinforced with filler-modules and FRP wrap (JII-5 through 8), the rebar yielded at both locations BSL1 and BSL2. For all the specimens JII-5 through 8, yielding occurred first at BSL2 then followed by BSL1, indicating the gradual progression of yield strain towards the joint panel in a controlled manner. At ultimate load (i.e., collapse), all the four beam longitudinal rebars in JII-8 fractured at the 90⁰ hook in the joint panel, as shown in Table 48.

6.4 Conclusions

A summary of the test results based on the experimental findings of joints reinforced with PSL filler-modules and FRP wraps is presented in Table 53. Besides, the following sections (6.4.1.1 through 6.4.1.4) provide concluding remarks on various parameters evaluated in this study:

Table 53. Summary of test results: joints reinforced with PSL filler-modules and FRP wraps

Joint	Peak load (lbs)	Ductility index	Energy dissipated (lb.in)	Failure mode
JII-1	20,900	2	17,458	Joint/diagonal tension
JII-2	35,273 (69)	4 (100)	56,373 (223)	Joint shear failure
JII-3	31,695 (52)	3 (50)	37,373 (114)	Joint/column shear failure
JII-4	34,989 (67)	3 (50)	37,488 (115)	Column shear failure
JII-5	41,868 (100)	5 (150)	85,476 (390)	Joint failure
JII-6	61,592 (195)	5 (150)	126,654 (625)	Column shear failure
JII-7*	56,894 (172)	6 (200)	133,506 (665)	Beam flexural failure
JII-8	59,320 (184)	8 (300)	204,972 (1,074)	Beam flexural failure

Numbers inside () indicate % increases with respect to the control specimen of a given batch.

* Loading was terminated abruptly before the complete failure of the specimen.

6.4.1.1 Influence of CFRP wrap without filler-modules (JII-2)

Although reinforcing joints with CFRP wrap without filler-modules has increased the strength, ductility, and energy dissipation capacities by 69%, 100%, and 223%, respectively, over the control specimen, the failure was still concentrated in the joint panel. The increase in structural capacities is attributed to the confinement from the CFRP wrap, which delayed the crushing of concrete and allowed the steel rebars to resist additional tensile forces. This approach of reinforcing joints with CFRP wraps around a joint is challenging to implement on a real building frame due to the presence of transverse beam(s) and slab(s) in one or more directions.

6.4.1.2 Influence of curve- versus wedge- filler-module (JII-3 vs. JII-4)

Both the curve- and wedge- filler-module reinforced joint specimens exhibited an increase in the strength, ductility, and energy dissipation by up to 67%, 50%, and 115%, respectively. Although the increase in ductility and energy absorption was similar between curve- and wedge- filler-module reinforced joints, the additional ~10% increase in the strength of wedge filler-module specimen is attributed to the higher compression (strut) resistance offered by the wedge filler-module when compared to a curved filler-module.

6.4.1.3 Influence of dowels (JII-5 vs. JII-6)

Specimen JII-6 reinforced with dowels exhibited a ~47% increase in the strength and energy dissipation capacities over the specimen JII-5 reinforced without dowels. This behavior reveals that the presence of dowels ensured efficient composite action between the beam, column, and joint through shear transfer, thus resulting in a superior structural capacity.

6.4.1.4 Influence of CFRP versus GFRP wrap (JII-7 vs. JII-8)

Both the specimens reinforced with CFRP- and GFRP- wraps significantly improved the structural response of joints. No specific discussions were made comparing the contributions of CFRP vs. GFRP wraps because of the early termination of load for specimen JII-7. However, it should be highlighted that the trend in rebar strain distribution (shown in Table 52) of both specimens revealed a similar performance until the point of load termination.

Specimen JII-8 reinforced with GFRP wrap exhibited the highest ductility (~3 times) and energy dissipation capacity (~11 times) among all the reinforcing schemes evaluated in the current phase. The superior performance of JII-8 reveals that the strain/stiffness compatibility between FRP wraps and the underlying substrate is essential for the structures to dissipate more energy under earthquake forces, especially in the inelastic zone.

7. Summary, Conclusions, and Recommendations

7.1 Introduction

This chapter summarizes experimental findings and provides the key conclusions derived from the current study on “Monotonic response of RC exterior joints reinforced with filler-modules and FRP composite wraps/gussets.” The conclusions for joints evaluated in control- and reinforced- (with filler-modules and with/out FRP wraps/gussets) conditions are drawn primarily based on the strength, ductility, and energy dissipation corresponding to peak loads, because of their importance under extreme load events. The joint behavior is a function of many parameters, and their influence on the structural behavior has been exhaustively studied, with key conclusions reported herein. This chapter also provides recommendations for future research to expand the scope of the study to real structural scenarios for devising appropriate design methodologies.

7.2 Summary

Over the past 50 years, several studies were conducted to strengthen exterior joints of in-service structures, with limited emphasis on developing simple, economic and durable repair strategies to improve energy absorption through large inelastic joint deformations. Even less emphasis was devoted to developing repair procedures that minimize stress-concentrations at joint corners and enhancing the strength, ductility, and energy dissipation capabilities of concrete structures with an emphasis on joint resistance improvements.

To address the above limitations, a novel approach of reinforcing vulnerable joints with filler-modules and fiber-reinforced polymer (FRP) composite wraps/gussets have been proposed and evaluated, herein. A total of twenty 2D RC exterior (T) joints were designed as per pre-1976 construction deficiencies and experimentally evaluated in control (as-built) and reinforced conditions through the variations in (i) filler-module geometric shape; (ii) filler-module material properties, (iii) FRP material, (iv) FRP wrap/gusset configurations, (v) confinement due to partial (U-anchors) versus complete (360°-anchors) diagonal wraps, and (vi) shear transfer through reinforcing dowels. The performance of the test specimens was recorded through numerical values

of loads versus deformation and strains at the rupture of concrete, de-bond of FRP wrap from the concrete surface, yielding of steel rebar, shear failure of column or joint panel through diagonal tension and beam flexural failure phenomena. Test data evaluations measured up to the peak loads revealed that the proposed approach of reinforcing joints with filler-modules and FRP wraps/gussets is immensely useful in enhancing the strength and ductility by ~300%, and energy dissipation by about ~1200%. Depending upon the reinforcing scheme(s), the magnitude of failure- loads and patterns varied in a controlled manner. Joints tested in “control” condition exhibited shear failure through diagonal tension and diagonal compression while the strengthened specimens failed in beam flexure or column shear, but in a ductile manner through yielding of the column- or beam- rebars. The strains measured on rebar surfaces at different locations of joint- and beam- sections revealed a significant reduction in strain progression towards the joint panel (beam-column overlap). It was also noted that the use of low-stiffness filler-modules such as syntactic foam and engineered wood coupled with FRP wraps has tremendously enhanced the structural response of reinforced joints. It was also observed that joints reinforced with filler-modules and FRP wraps or gussets exhibiting beam flexural failure had more energy dissipation capacity when compared to specimens that failed in column shear. Based on the experimental results of reinforced joint specimens, joint behavior is characterized into three zones, i.e., A (onset of filler-module cracking), B (idealized yield - defined as the point beyond which a truss mechanism primarily resists the forces), and C (Peak load - referred to as a highest numerical value of load recorded during the testing). Furthermore, limit states (principal tension and shear) for joint at the onset of filler-module cracking (i.e., point A) and idealized yield (i.e., point B) have been established as a function of the concrete tensile strength ($k\sqrt{f'_c}$) and presented in Tables 33, 35, 37, and 46.

The behavior of joints reinforced with filler-modules and FRP wraps/gussets up to the point A and perhaps point B is governed by the location of the plastic hinge away from the joint panel, which primarily depends on the strength and stiffness of bottom filler-module (i.e., in the compression zone) and to some extent on the tensile resistance offered by the top filler-module (i.e., in the tension zone). For example, if the hinge forms within the filler-module length, the specimen offers additional force resistance because of the higher moment of inertia (I) at that section. Besides, the

shear deformations (γ) induced in the panel are lower due to the shorter lever-arm (i.e., distance from C.G. of the joint panel to the hinge point). Furthermore, the resistance to induced stresses in the joint panel can be enhanced by simply bonding FRP reinforcement to the exposed joint section as a “gusset” or by confining through 360° or U-wrap - depending on the accessibility of joint panel). On the other hand, if the hinge forms outside of the filler-module width (i.e., either in the beam or column section), the resistance offered is limited to the capacity of beam or column due to reduced “I”. Although this approach may yield better results (i.e., increase in the peak load, ductility, and energy absorption) over the control specimen, it may not necessarily maximize the overall structural response in relation to the enhanced joint capacity. Latter approach has been proposed and followed by several researchers, i.e., merely pushing the hinge formation farther away from the column face, into the beam section by providing steel haunches [22]-[24] or FRP wrapping of beam and column stubs up to their plastic hinge locations [15]-[19] or planar joint enlargement [27]-[30], and [125] and other approaches. Unlike other approaches, the current study is unique in terms of maximizing the structural response using low-stiffness filler-modules with moderate- to high- compressive strength coupled with FRP wraps/gussets.

7.3 Conclusions

The use of filler-modules coupled with FRP wraps/gussets for reinforcing 2D RC exterior joints has been developed and investigated in this study. The experimental findings and observations from this study, presented in sections 5.3 and 6.3 of this manuscript, revealed the following key conclusions:

1. The tensile strength of FRP wrapped cylinders tested under split tension was found to be $\sim(20-28)\sqrt{f'_c}$ as opposed to $\sim(6-8)\sqrt{f'_c}$ for normal weight plain concrete cylinders (section 4.4.6).
2. The test results of joint (control) specimens indicated poor performance of pre-1976 designs, resulting in brittle joint shear failure. Besides, joint (control) specimens subjected to axial column loads in the range of 0.18 to $0.21f'_cA_g$ resulted in joint failure through

diagonal tension cracking, while the specimen subjected to axial column load of $0.28f'_c A_g$ exhibited joint failure through the crushing of concrete (section 5.2.1, and 6.2.1).

3. Although FRP strengthening of joint panels (section 6.4.1.1) exhibited ~69%, 100%, 223% in strength, ductility, and energy dissipation capacities, respectively, the failure was still concentrated in the joint panel.
4. Reinforcing joints with filler-modules without FRP wraps/gussets exhibited a limited increase in strength, ductility, and energy dissipation capacities due to the premature column failure initiated at the tip of the filler-module (as shown in Table 43 and Table 53 and sections 5.4.1.1 and 6.4.1.2).
5. The use of filler-modules coupled with FRP wraps/gussets increased the strength, ductility, and energy dissipation by up to ~314%, 300%, ~1,118%, respectively, over the control specimen (Table 43). These increases are attributed to the relocation of plastic hinge away from the joint panel, resulting in either column shear or beam flexural failure. The joint specimens that failed in beam flexure exhibited superior structural response than joint specimens that failed in column shear.
6. Joint specimens reinforced with filler-modules, dowels, and FRP wraps/gussets were found to be effective in preventing the joint shear failure while forcing the plastic hinge to form in the beam or column sections (Tables 43 and 54).
7. The joint specimen reinforced with concrete filler-modules, CFRP wrap, and 360°-anchor, exhibited an increase of ~170%, 100%, and 402% in strength, ductility, and energy dissipation capacities over the joint specimen reinforced with concrete filler-modules without CFRP wrap and 360°-anchor (Table 43). This improvement is primarily due to the tensile resistance offered by the CFRP wrap in addition to the concrete confinement effect from 360°-anchor. This approach of reinforcing joints with CFRP wrap and 360°-anchor around the filler-modules has shifted the non-ductile column shear failure (which is not a preferred failure mode) to a more ductile beam flexural failure.
8. The low-stiffness syntactic foam filler-modules coupled with CFRP wraps facilitated a near-uniform strain distribution in the rebars- of column ($2612 \mu\epsilon$), beam ($2608 \mu\epsilon$) and joint panel ($2263 \mu\epsilon$) (Table 42) thus, resulting in superior performance over other joint reinforcing schemes.

9. Comparing joint specimens reinforced with medium- and high- strength filler-modules coupled with CFRP gussets, the latter exhibited ~41%, 25%, and 64% in strength, ductility, and energy dissipation capacities. The superior performance of the high-strength filler-module is attributed to the enhanced compression force (strut) resistance offered by the filler-module beyond the elastic zone. The higher the strength of filler-module, the longer the plateau between points B and C, i.e., more prolonged inelastic zone.
10. The shape of the filler-modules did not significantly alter the behavior of joints in terms of strength, ductility, and energy dissipation capacities (< 9% variation).
11. Dowel reinforcement in filler-modules increased the strength and energy dissipation of joint specimens by ~47% when compared to the specimen without the dowel reinforcement. This behavior reveals that the presence of dowels ensured efficient composite action between the beam, column, and joint through shear transfer, thus resulting in a superior structural capacity.
12. Among the joint reinforcing schemes using PSL filler-modules, the specimen reinforced with PSL wedge filler-modules, dowels, and GFRP wrap exhibited superior performance (section 6.4.1.4). The superior performance reveals the stiffness compatibility between FRP wraps and the underlying substrate is essential.

7.4 Recommendations for Future Research

The following recommendations are made for future research to expand the scope of this study:

1. Joints of a building frame are formed by the intersection of beams, columns, and slab in two or more directions. Therefore, to understand the behavior of a real structural scenario, future research on the current approach with filler-modules and FRP wraps/gussets must be directed towards evaluating the performance of joints with additional stiffness contributions from the slab, and transverse beams to establish joint limit states and to devise appropriate design methods.
2. Under seismic actions, exterior joints are subjected to varying axial load and bidirectional lateral load reversals. Therefore, the response of exterior joints reinforced with filler-

modules and FRP wraps/gussets under varying axial load and bidirectional lateral load reversals must be evaluated.

3. Varying a/d (distance from CG of joint to effective depth) ratios for both column and beam stubs meeting at a joint must be evaluated to investigate the influence of the current approach under different failure modes.
4. Due to the limited availability of experimental data on joints under many independent parameters, machine learning tools must be employed to train, evaluate and develop strength prediction models after generating additional test data in a strategic sense with an understanding developed from current research studies.
5. To optimize the joint performance, finite element analysis studies on joint inelastic behavior must be performed by incorporating material nonlinearity (post-cracking behavior of concrete joint or element) to arrive at the optimized shape of filler-modules and optimized fiber orientation of composite wraps/gussets as a function of the substrate strength and bond strengths of glue lines of the substrate, filler-modules and wrap or gusset.

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Appendix-A: Beam-Column Joints Reinforced with Concrete/Syntactic Foam Filler-Modules and CFRP Wraps

This section provides additional test data generated through experimental evaluation of beam-column joints reinforced with concrete/syntactic foam filler-modules with/out FRP wraps under monotonic loads.

A.1 Specimen JI-1

A.1.1 Strain in steel rebar

Figure A.1. Beam load vs. column longitudinal rebar strain

Figure A.2. Beam load vs. beam longitudinal rebar strain

Figure A.3. Beam load vs. joint tail rebar strain

A.2 Specimen JI-2

A.2.1 Strain in steel rebar

Figure A.4. Beam load vs. column longitudinal rebar strain

Figure A.5. Beam load vs. beam longitudinal rebar strain

Figure A.6. Beam load vs. joint tail rebar strain

A.2.2 Strain in concrete

Figure A.7. Beam load vs. concrete strain in joint panel

Figure A.8. Beam load vs. filler-module longitudinal strain

A.3 Specimen JI-3

A.3.1 Strain in steel rebar

Figure A.9. Beam load vs. column longitudinal rebar strain

Figure A.10. Beam load vs. beam longitudinal rebar strain

Figure A.11. Beam load vs. beam stirrup strain

A.3.2 Strain in FRP

Figure A.12. Beam load vs. column principal strain

Figure A.13. Beam load vs. column shear strain

Figure A.14. Beam load vs. filler-module longitudinal strain

Figure A.15. Beam load vs. filler-module longitudinal strain

A.4 Specimen JI-4

A.4.1 Strain in steel rebar

Figure A.16. Beam load vs. column longitudinal rebar strain

Figure A.17. Beam load vs. beam longitudinal rebar strain

A.4.2 Strain in FRP

Figure A.18. Beam load vs. filler-module longitudinal strain

Figure A.19. Beam load vs. filler-module longitudinal strain

Figure A.20. Beam load vs. column principal strain

Figure A.21. Beam load vs. column shear strain

Appendix-B: Beam-Column Joints Reinforced with Concrete Filler-Modules and CFRP Gussets

This section provides additional test data generated through experimental evaluation of beam-column joints reinforced with concrete filler-modules with CFRP gussets under monotonic loads.

B.1 Specimen JI-5

B.1.1 Strain in steel rebar

Figure B.1. Beam load vs. column longitudinal rebar strain

Figure B.2. Beam load vs. beam longitudinal rebar strain

Figure B.3. Beam load vs. joint tail rebar strain

Figure B.4. Beam load vs. beam stirrup strain

B.1.2 Strain in concrete

Figure B.5. Beam load vs. concrete strain in beam

Figure B.6. Beam load vs. concrete strain in beam

Figure B.7. Beam load vs. concrete strain in column

B.2 Specimen JI-6

B.2.1 Strain in steel rebar

Figure B.8. Beam load vs. column longitudinal rebar strain

Figure B.9. Beam load vs. beam longitudinal rebar strain

Figure B.10. Beam load vs. beam stirrup strain

B.2.2 Strain in FRP

Figure B.11. Beam load vs. FRP strain in beam

Figure B.12. Beam load vs. beam principal strain

Figure B.13. Beam load vs. beam shear strain

Figure B.14. Beam load vs. filler-module longitudinal strain

Figure B.15. Beam load vs. filler-module longitudinal strain

Figure B.16. Beam load vs. column FRP strain

Figure B.17. Beam load vs. column rebar FRP strain

Figure B.18. Beam load vs. FRP strain in joint panel

Figure B.19. Beam load vs. filler-module FRP strain

Figure B.20. Beam load vs. FRP strain in column and filler-module

Figure B.21. Beam load vs. FRP strain in column and filler-module

B.3 Specimen JI-7

B.3.1 Strain in steel rebar

Figure B.22. Beam load vs. column longitudinal rebar strain

Figure B.23. Beam load vs. beam stirrup strain

Figure B.24. Beam load vs. beam longitudinal rebar strain

B.3.2 Strain in FRP

Figure B.25. Beam load vs. filler-module longitudinal strain

Figure B.26. Beam load vs. filler-module longitudinal strain

Figure B.27. Beam load vs. FRP strain in beam

Figure B.28. Beam load vs. FRP strain in joint panel

Figure B.29. Beam load vs. beam principal strain

Figure B.30. Beam load vs. beam shear strain

Figure B.31. Beam load vs. filler-module FRP strain

Figure B.32. Beam load vs. FRP strain in column and filler-module

Figure B.33. Beam load vs. FRP strain in column and filler-module

Figure B.34. Beam load vs. column FRP strain

Figure B.35. Beam load vs. column FRP strain

B.4 Specimen JI-8

B.4.1 Strain in steel rebar

Figure B.36. Beam load vs. column longitudinal rebar strain

Figure B.37. Beam load vs. beam longitudinal rebar strain

Figure B.38. Beam load vs. beam stirrup strain

B.4.2 Strain in FRP

Figure B.39. Beam load vs. filler-module longitudinal strain

Figure B.40. Beam load vs. filler-module longitudinal strain

Figure B.41. Beam load vs. FRP strain in beam

Figure B.42. Beam load vs. FRP strain in joint panel

Figure B.43. Beam load vs. beam principal strain

Figure B.44. Beam load vs. beam shear strain

Figure B.45. Beam load vs. filler-module FRP strain

Figure B.46. Beam load vs. FRP strain in column and filler-module

Figure B.47. Beam load vs. FRP strain in column and filler-module

Figure B.48. Beam load vs. column FRP strain

Figure B.49. Beam load vs. column FRP strain

Appendix-C: Beam-Column Joints Reinforced with Concrete Filler-Modules and GFRP Wraps

This section provides additional test data generated through experimental evaluation of beam-column joints reinforced with concrete filler-modules with GFRP wraps under monotonic loads.

C.1 Specimen JIII-1

C.1.1 Strain in steel rebar

Figure C.1. Beam load vs. column longitudinal rebar strain

Figure C.2. Beam load vs. beam longitudinal rebar strain

Figure C.3. Beam load vs. beam stirrup strain

Figure C.4. Beam load vs. joint tail rebar strain

C.1.2 Strain in concrete

Figure C.5. Beam load vs. concrete strain in the joint panel

Figure C.6. Beam load vs. concrete strain in the joint panel

Figure C.7. Beam load vs. concrete strain in the joint panel

Figure C.8. Beam load vs. column strain in the joint panel

C.2 Specimen JIII-2

C.2.1 Strain in steel rebar

Figure C.9. Beam load vs. column longitudinal rebar strain

Figure C.10. Beam load vs. beam longitudinal rebar strain

Figure C.11. Beam load vs. beam stirrup strain

C.2.2 Strain in FRP

Figure C.12. Beam load vs. filler-module longitudinal strain

Figure C.13. Beam load vs. filler-module longitudinal strain

Figure C.14. Beam load vs. FRP strain in the beam

Figure C.15. Beam load vs. FRP strain in the joint panel

Figure C.16. Beam load vs. beam principal strain

Figure C.17. Beam load vs. beam shear strain

Figure C.18. Beam load vs. filler-module FRP strain

Figure C.19. Beam load vs. FRP strain in column and filler-module

Figure C.20. Beam load vs. FRP strain in column and filler-module

Figure C.21. Beam load vs. column FRP strain

Figure C.22. Beam load vs. column FRP strain

C.3 Specimen JIII-3

C.3.1 Strain in steel rebar

Figure C.23. Beam load vs. column longitudinal rebar strain

Figure C.24. Beam load vs. beam longitudinal rebar strain

Figure C.25. Beam load vs. beam stirrup strain

C.3.2 Strain in FRP

Figure C.26. Beam load vs. filler-module longitudinal strain

Figure C.27. Beam load vs. filler-module longitudinal strain

Figure C.28. Beam load vs. FRP strain in the beam

Figure C.29. Beam load vs. FRP strain in the joint panel

Figure C.30. Beam load vs. beam principal strain

Figure C.31. Beam load vs. beam shear strain

Figure C.32. Beam load vs. filler-module FRP strain

Figure C.33. Beam load vs. FRP strain in column and filler-module

Figure C.34. Beam load vs. FRP strain in column and filler-module

Figure C.35. Beam load vs. column FRP strain

Figure C.36. Beam load vs. column FRP strain

C.4 Specimen JIII-4

C.4.1 Strain in steel rebar

Figure C.37. Beam load vs. column longitudinal rebar strain

Figure C.38. Beam load vs. beam longitudinal rebar strain

Figure C.39. Beam load vs. beam stirrup strain

C.4.2 Strain in FRP

Figure C.40. Beam load vs. filler-module longitudinal strain

Figure C.41. Beam load vs. filler-module longitudinal strain

Figure C.42. Beam load vs. FRP strain in the beam

Figure C.43. Beam load vs. FRP strain in the joint panel

Figure C.44. Beam load vs. beam principal strain

Figure C.45. Beam load vs. beam shear strain

Figure C.46. Beam load vs. filler-module FRP strain

Figure C.47. Beam load vs. FRP strain in column and filler-module

Figure C.48. Beam load vs. FRP strain in column and filler-module

Figure C.49. Beam load vs. column FRP strain

Figure C.50. Beam load vs. column FRP strain

Appendix-D: Beam-Column Joints Reinforced with Engineered Wood Filler-Modules and FRP Wraps

This section provides additional test data generated through experimental evaluation of beam-column joints reinforced with engineered wood (PSL) filler-modules with FRP wraps under monotonic loads.

D.1 Specimen JII-1

D.1.1 Strain in steel rebar

Figure D.1. Beam load vs. column longitudinal rebar strain

Figure D.2. Beam load vs. beam longitudinal rebar strain

Figure D.3. Beam load vs. joint tail rebar strain

Figure D.4. Beam load vs. beam stirrup strain

D.1.2 Strain in concrete

Figure D.5. Beam load vs. joint principal strain

Figure D.6. Beam load vs. joint shear strain

D.3 Specimen JII-3

D.3.1 Strain in steel rebar

Figure D.7. Beam load vs. column longitudinal rebar strain

Figure D.8. Beam load vs. beam longitudinal rebar strain

Figure D.9. Beam load vs. joint tail rebar strain

Figure D.10. Beam load vs. beam stirrup strain

D.3.2 Strain in concrete

Figure D.11. Beam load vs. column principal strain

Figure D.12. Beam load vs. column shear strain

D.4 Specimen JII-4

D.4.1 Strain in steel rebar

Figure D.13. Beam load vs. column longitudinal rebar strain

Figure D.14. Beam load vs. beam longitudinal rebar strain

Figure D.15. Beam load vs. joint tail rebar strain

Figure D.16. Beam load vs. beam stirrup strain

D.4.2 Strain in concrete

Figure D.17. Beam load vs. column principal strain

Figure D.18. Beam load vs. column shear strain

D.5 Specimen JII-5

D.5.1 Strain in steel rebar

Figure D.19. Beam load vs. column longitudinal rebar strain

Figure D.20. Beam load vs. beam longitudinal rebar strain

Figure D.21. Beam load vs. beam stirrup strain

Figure D.22. Beam load vs. joint tail rebar strain

D.5.2 Strain in FRP

Figure D.23. Beam load vs. beam principal strain

Figure D.24. Beam load vs. beam shear strain

D.6 Specimen JII-6

D.6.1 Strain in steel rebar

Figure D.25. Beam load vs. column longitudinal rebar strain

Figure D.26. Beam load vs. beam longitudinal rebar strain

Figure D.27. Beam load vs. joint tail rebar strain

Figure D.28. Beam load vs. beam stirrup strain

D.6.2 Strain in FRP

Figure D.29. Beam load vs. beam principal strain

Figure D.30. Beam load vs. beam shear strain

D.7 Specimen JII-7

D.7.1 Strain in steel rebar

Figure D.31. Beam load vs. column longitudinal rebar strain

Figure D.32. Beam load vs. beam longitudinal rebar strain

Figure D.33. Beam load vs. beam stirrup strain

D.8 Specimen JII-8

D.8.1 Strain in steel rebar

Figure D.34. Beam load vs. column longitudinal rebar strain

Figure D.35. Beam load vs. beam longitudinal rebar strain

Figure D.36. Beam load vs. joint tail rebar strain

Figure D.37. Beam load vs. beam stirrup strain

D.8.2 Strain in FRP

Figure D.38. Beam load vs. beam principal strain

Figure D.39. Beam load vs. beam shear strain

Appendix-E: FRP Wrapped Concrete Cylinders

This section provides additional stress vs. strain plots generated through experimental evaluation of FRP wrapped concrete (4in. x 8in.) cylinders under axial-compression and split-tension tests.

E.1 Axial-Compression Test

Figure E.1. Stress vs. strain behavior of JI-CS

Figure E.2. Stress vs. strain behavior of JI-S1

Figure E.3. Stress vs. strain behavior of JI-S2

Figure E.4. Stress vs. strain behavior of JII-CS

Figure E.5. Stress vs. strain behavior of JII-S1

Figure E.6. Stress vs. strain behavior of JII-S2

E.2 Split-Tension Test

Figure E.7. Stress vs. strain behavior of JI-CS

Figure E.8. Stress vs. strain behavior of JI-S1

Figure E.9. Stress vs. strain behavior of JI-S2

Figure E.10. Stress vs. strain behavior of JII-CS

Figure E.11. Stress vs. strain behavior of JII-S1

Figure E.12. Stress vs. strain behavior of JII-S2

Note:

1. J-I S-1: 2 layers of Sika103C unidirectional carbon wrap with 3 in. overlap.
2. J-I S-2: 2 layers of Sika103C unidirectional carbon wrap with 0.75 in. gap between the ends.
3. J-II S-1: 2 layers of Sika103C unidirectional carbon wrap with no overlap and no gap.
4. J-II S-2: 2 layers of Sika100G unidirectional glass wrap with no overlap and no gap.
5. Only one specimen was tested per category.