Assessment of an Earthquake Resilient Bridge with Pretensioned, Rocking Columns

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Abstract

The seismic performance of a new bridge system is studied, tested and improved. The new bridge system: 1) reduces onsite construction time by using precast components, 2) eliminates major earthquake damage by utilizing rocking column and confinement of the column ends with a steel tube, and 3) maintains the system functionality after a strong earthquake by minimizing residual drift through the use of prestressing strands in the columns. Furthermore, it uses only conventional materials. The shaking table performance of a quarter-scale, two-span bridge constructed using the new system was compared with that of a conventional cast-in-place bridge with similar geometry tested in 2005. The new bridge system was constructed in about 20% of the time needed for the conventional cast-in-place system. In the tests, the conventional bridge suffered major concrete cracking and spalling, whereas in the new system, damage to the concrete was only cosmetic. In the conventional bridge, the longitudinal bars buckled and both the longitudinal and spiral reinforcement fractured, whereas in the new system the damage to the reinforcement was limited to longitudinal bar fracture, and that occurred only under excitations larger than the design level motion. The residual drift of the new system was essentially zero for all motions, whereas one of the exterior bents of the conventional bridge was so badly damaged and out of plumb that some of the supplemental mass on the bridge had to be removed and testing was stopped shortly thereafter. The only substantial damage that the new bridge system experienced was longitudinal reinforcing fracture. Therefore, ways to delay fracture were developed analytically.
Reinforcement fractures were audible during the shaking table tests of the pretensioned rocking system. Reinforcement fractures were estimated in three ways using:

1) audio recorded during each test, 2) measured rotations at column ends and 3) analytical models, which included a fatigue material. This analytical model was then used to explore methods to improve the performance of the system by delaying reinforcement fracture. The analytical parametric studies on the scaled model showed that increasing the bar size and the locally debonded length of the reinforcement were both effective strategies to reduce and delay bar fractures. For the shaking table experimental model configuration, the analytical model showed that increasing the longitudinal bars by one size and increasing the debonded length by 44% would delay bar fracture until an excitation 67% larger than the excitation where reinforcing bars first fractured in the physical experiment. The parametric study also was conducted for a prototype bent; this recommended values for longitudinal bar size, debonded lengths for longitudinal bars and effective prestressing for prestressing strands to delay the fracture of the longitudinal bars and the yielding of the prestressing strands until after the 150% design level motion.
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# Table of Content

Abstract ........................................................................................................................................... i
Acknowledgements ......................................................................................................................... iii
Table of Content ................................................................................................................................. v
List of Tables ....................................................................................................................................... x
List of Figures .................................................................................................................................... xiii
Notation ............................................................................................................................................ xxxiv

## Chapter 1. Introduction .................................................................................................................. 1
1.1 Background ................................................................................................................................ 1
1.2 Literature Review ......................................................................................................................... 2
  1.2.1 Socket Connection at the Bottom of the Column ................................................................. 2
  1.2.2 Hybrid-Bar-Socket Connection at the Top of the Column ...................................................... 4
  1.2.3 Pretensioned Columns ........................................................................................................... 5
  1.2.4 Subassembly Column Tests for the Proposed System ........................................................... 6
1.3 Research Scope and Objectives ..................................................................................................... 7
1.4 Organization of this Work ............................................................................................................ 9

## Chapter 2. Specimen Development ............................................................................................... 11
2.1 Introduction .................................................................................................................................. 11
2.2 Conventional Bridge Specimen ...................................................................................................... 12
2.3 Resilient Bridge Specimen ............................................................................................................ 13
  2.3.1 Column Design ....................................................................................................................... 14
  2.3.2 Bent Cap Design ..................................................................................................................... 15
  2.3.3 Footing Design ....................................................................................................................... 17
  2.3.4 Superstructure Beam Design ................................................................................................... 18
  2.3.5 Superstructure Masses ............................................................................................................ 19
  2.3.6 Construction Sequence Design .............................................................................................. 20
    2.3.6.1 Construction Sequence ...................................................................................................... 21
    2.3.6.2 Construction Sequence Analytical Results ........................................................................ 22
2.4 Instrumentation Plan ..................................................................................................................... 23
  2.4.1 Global Coordinate System ..................................................................................................... 24
  2.4.2 Superstructure Accelerometers .............................................................................................. 24
  2.4.3 Superstructure Displacements Potentiometers ....................................................................... 24
2.4.4 Strand Load Cell ......................................................... 25
2.4.5 Column Displacement Transducers ..................................... 25
2.4.6 Strain Gauges ............................................................... 25
   2.4.6.1 Longitudinal Reinforcement ......................................... 26
   2.4.6.2 Longitudinal Reinforcing Strands ..................................... 27
   2.4.6.3 Transverse Reinforcing ............................................... 27
2.5 Ground Motion Selection .................................................... 27

Chapter 3. Specimen Construction ............................................. 30

3.1 Introduction ......................................................................... 30
3.2 Construction and Schedule ................................................... 30
3.3 Construction of the Bridge .................................................... 32
   3.3.1 Precast Column Construction ........................................... 32
   3.3.2 Precast Bent Cap Construction ......................................... 33
   3.3.3 Footing Construction ...................................................... 33
   3.3.4 Bent Assembly ............................................................. 34
      3.3.4.1 Bent 1 Assembly ......................................................... 35
      3.3.4.2 Bent 2 Assembly ......................................................... 36
      3.3.4.3 Bent 3 Assembly ......................................................... 36
   3.3.5 Bridge Assembly .......................................................... 37
      3.3.5.1 Superstructure Beam Assembly ................................. 37
      3.3.5.2 Placement of Superstructure Beams .......................... 37
      3.3.5.3 Footing Attachment to the Shake Table ................... 38
      3.3.5.4 Longitudinal Post-tensioning .................................. 38
      3.3.5.5 Grouting Bent Cap to the Columns ....................... 39
   3.3.6 Instrumentation .......................................................... 40
3.4 Material Testing ............................................................... 41

Chapter 4. Observed and Measured Response ................................ 42

4.1 Introduction ......................................................................... 42
4.2 Observed Results ............................................................. 42
   4.2.1 Low Amplitude Motions ............................................... 43
   4.2.2 High Amplitude Motions .............................................. 43
      4.2.2.1 Bent 1 Observations ................................................ 44
      4.2.2.2 Bent 2 Observations ................................................ 45
      4.2.2.3 Bent 3 Observations ................................................ 45
4.3 Measured Results ............................................................. 46
   4.3.1 Reinforcement Strains .................................................. 46
Chapter 4. Structural Response Analysis

4.3.1 Longitudinal Reinforcement

4.3.1.1 Longitudinal Reinforcement .......................................................... 47
4.3.1.2 Lateral Reinforcement ................................................................. 48
4.3.1.3 Strands ....................................................................................... 48

4.3.2 Column Rotation.............................................................................. 49

4.3.3 Bridge Displacement......................................................................... 50
4.3.3.1 Maximum Transverse Displacement .............................................. 50
4.3.3.2 Residual Transverse Displacement ............................................... 52
4.3.3.3 Bent Vertical Displacement ......................................................... 53

4.3.4 Transverse Acceleration..................................................................... 53

4.3.5 Bridge Base Shear.............................................................................. 53

4.3.6 Modal Analysis.................................................................................. 56
4.3.6.1 White Noise Analysis ................................................................. 56
4.3.6.2 Pulse Motion Analysis ................................................................ 58
4.3.6.3 Square Wave Analysis ............................................................... 59

4.3.7 Instrumentation Tuning...................................................................... 59

Chapter 5. Longitudinal Reinforcing Fracture Estimation ....................... 61

5.1 Introduction......................................................................................... 61

5.2 Methodology....................................................................................... 61

5.3 Method Verification.............................................................................. 63

5.4 Bar Elongations at First Bar Fracture .................................................. 64

5.5 Bar Fracture Estimation using Strain Threshold Method ...................... 65

5.6 Low Cycle Fatigue Life using the Displacement Method ...................... 66
5.6.1 Fatigue Life Relationships for Reinforcing bar ................................. 67
5.6.2 Bar Fracture Estimation Using the Low Cycle Fatigue Method ....... 70

5.7 Bar Fracture Estimation using Acoustic Emissions ............................. 71

5.8 Fracture Evaluation by End of Testing ............................................... 73

Chapter 6. Specimen Comparison with the Conventional Bridge ............... 74

6.1 Introduction......................................................................................... 74

6.2 Systems Differences............................................................................ 74

6.3 Motion Differences.............................................................................. 75

6.4 Observed Damage Comparison.......................................................... 77

6.5 Measured Response Comparison....................................................... 79
6.5.1 Global Response Comparison.......................................................... 79
6.5.1.1 Measured Displacement Comparison .......................................... 79
6.5.1.2 Calculated Base Shear Comparison .................................................. 80
6.5.1.3 Shift in Fundamental Period .......................................................... 81
6.5.2 Local Response Comparison .............................................................. 82
6.5.2.1 Rotation Comparison ........................................................................ 82
6.5.2.2 Strain Comparison .......................................................................... 83
6.6 Construction Speed Comparison .............................................................. 85
6.7 Comparison Conclusions ........................................................................ 86

Chapter 7. Numerical Model Assessment .................................................. 88

7.1 Introduction ............................................................................................ 88
7.2 Model Assumptions ................................................................................ 88
7.2.1 Element Assumptions ........................................................................ 89
7.2.2 Section and Material Assumptions ..................................................... 89
7.2.3 Additional Assumptions ...................................................................... 92
7.2.3.1 Input Motions .................................................................................. 92
7.2.3.2 Anchorage Slip .............................................................................. 92
7.2.3.3 Damping .......................................................................................... 93
7.2.3.4 Additional Masses .......................................................................... 93
7.2.3.5 Numerical Procedure ...................................................................... 94
7.3 Model Assessment Prior to Reinforcement Fracture .............................. 94
7.3.1 Response History Assessment ............................................................. 95
7.3.1.1 Motion 14A (33% Design Level Motion) Assessment .................... 95
7.3.1.2 Motion 14B1 Assessment ................................................................. 96
7.3.1.3 Motion 14B2 Assessment ................................................................. 97
7.3.1.4 Motion 14C Assessment ................................................................. 97
7.3.1.5 Motion 15 (67% Design Level Motion) Assessment .................... 98
7.3.1.6 Motion 16 (Design Level Motion) Assessment .............................. 99
7.3.2 Accumulative Assessment .................................................................. 100
7.4 Post Fracture Model Assessment ........................................................... 101
7.4.1 Response History Assessment ............................................................. 102
7.4.1.1 Motion 17 (133% Design Level Motion) Assessment .................... 102
7.4.1.2 Motion 18 (177% Design Level Motion) Assessment .................... 103
7.4.1.3 Motion 19 (221% Design Level Motion) Assessment .................... 103
7.4.1.4 Motion 20B Assessment ................................................................. 104
7.4.2 Accumulative Assessment .................................................................. 105
7.4.2.1 Base Shear Assessment ................................................................. 105
7.4.2.2 End Rotation Assessment .............................................................. 106
7.4.2.3 Strand Response Assessment ......................................................... 106
List of Tables

Table 2-1: Prototype, the Conventional Bridge and the Resilient Bridge properties ..... 134
Table 2-2: Construction sequence result for each case for Bents 1 and 3 during the longitudinal post-tensioning .................................................................................................................. 135
Table 2-3: Instrumentation Summary .......................................................................................................................... 136
Table 2-4: Final experimental protocol up to Motion S5 ................................................................. 137
Table 2-5: Final experimental protocol from Motion 14B1 to 21C................................................... 138
Table 3-1: Laboratory activities during the first through third week........................................... 139
Table 3-2: Laboratory activities during the fourth through sixth week ......................... 140
Table 3-3: Concrete compressive strength of bridge components (psi) .................... 141
Table 3-4: Elastic Modulus of bridge components (ksi).............................................................. 141
Table 3-5: Non-Fiber grout compressive strength for Bent 2 (psi) ........................................ 141
Table 3-6: Non-Fiber grout compressive strength for Bent 1 and 3 (psi) ....................... 141
Table 3-7: Fiber grout compressive strength for Bent 1 (psi)........................................... 142
Table 3-8: Fiber grout compressive strength for Bent 2 and 3 (psi) ...................................... 142
Table 3-9: Steel properties for each reinforcing type for the column ................................ 142
Table 4-1: Damage progression by visual inspection for Bent 1 ........................................... 143
Table 4-2: Damage progression by visual inspection for Bent 2 ........................................... 144
Table 4-3: Damage progression by visual inspection for Bent 3 ........................................... 145
Table 4-4: Maximum load cell slippage for Bent 1 strands (kips) ................................... 146
Table 4-5: Maximum load cell slippage for Bent 2 strands (kips) ................................... 147
Table 4-6: Maximum load cell slippage for Bent 3 strands (kips) ................................... 148
Table 4-7: Maximum bent drifts for all motions ................................................................. 149
Table 4-8: Maximum measured table and bent acceleration ............................................... 150
Table 4-9: Period and damping ratio using transfer function from white noise wave. .... 151
Table 4-10: Period and damping ratio calculated at each bent during near field motion using logarithmic decrement method ......................................................................................... 151
Table 4-11: Periods calculated from square waves (Sec.) ...................................................... 152
Table 4-12: Damping ratio calculated from square waves (%) .............................................. 152
Table 5-1: Maximum bar elongation during Motion 17 at column interfaces

153

Table 5-2: Bar fracture summary for Motion 17 through Motion 21C using the Strain Threshold Method

154

Table 5-3: Bar fracture summary for Motion 17 through Motion 21C using the cumulative fatigue damage index from strain estimation using the Displacement Method

155

Table 5-4: The reinforcing steel fracture summary using maximum strain threshold, low cycle fatigue model and acoustic emission from Motion 17 through the end of testing

156

Table 6-1: Damage progression for both bridges after Motions 14A through 19

157

Table 6-2: First occurrence of damage states for both bridges

158

Table 6-3: Test schedule, maximum bent drifts and peak table acceleration

159

Table 6-4: Maximum base shear comparison for both the Resilient Bridge and the Conventional Bridge

160

Table 6-5: Period comparison using transfer function for both the Resilient Bridge and the Conventional Bridge

160

Table 7-1: Element, section and material level for each structural component

161

Table 7-2: Concrete04 material properties

162

Table 7-3: Reinforcing steel material properties

162

Table 7-4: Fatigue material properties

162

Table 7-5: Elastic-perfectly plastic gap material properties

162

Table 7-6: Initial strain material properties

162

Table 8-1: Parametric study cases for tuning the reinforcing steel parameters to delay the fracture in the scaled specimen

163

Table 8-2: Number of fractured bars, number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 24#6, 70% fpy effective prestressing and bent height=24ft

164

Table 8-3: Number of fractured bars, number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 18#7, 70% fpy effective prestressing and bent height=24ft

165
Table 8-4: Number of fractured bars, number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 14#8, 70% fpy effective prestressing and bent height=24ft .............................. 166

Table 8-5: Cases of no fracture for different bar sizes and design level motions .......... 166

Table 8-6: Number of fractured bars, number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 24#6 and bent height=24ft during 150% DE ................................................................. 167

Table 8-7: Number of fractured bars, number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 18#7 and bent height=24ft during 150% DE ................................................................. 167

Table 8-8: Number of fractured bars, number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 14#8 and bent height=24ft during 150% DE ................................................................. 167

Table 8-9: Number of fractured bars, Number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 24#6 and 50% fpy effective prestressing during 150% DE with different bent heights .................................................. 168

Table 8-10: Number of fractured bars, number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 18#7 and 50% fpy effective prestressing during 150% DE with different bent heights .................................................. 168

Table 8-11: Number of fractured bars, number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 14#8 and 50% fpy effective prestressing during 150% DE with different bent heights .................................................. 169
List of Figures

Figure 1-1: Socket connection ........................................................................................................... 171
Figure 1-2: Socket connection, second specimen by end of testing (Haraldsson et al. 2011) .............................................................................................................................. 171
Figure 1-3: Larger bar grouted duct connection (Pang et al. 2008) .............................................. 172
Figure 1-4: Hybrid-bar-socket connection between the prestressed, precast column and precast bent cap (Thonstad et al. 2014) ......................................................................................... 172
Figure 1-5: Hybrid-bar-socket connection, by end of testing (Davis et al. 2012) ......................... 173
Figure 1-6: Effective force verses displacement for the pretensioned column with socket connection between the precast column and the cast-in-place footing (Davis et al. 2012) ......................................................................................................................................................... 173
Figure 1-7: Effective force verses displacement for the pretensioned column with hybrid-bar-socket connection between the precast column and the precast bent cap (Davis et al. 2012) ................................................................................................................................................................................. 174
Figure 1-8: Effective force verses displacement for the pretensioned, rocking column with socket connection between the precast column and the cast-in-place footing (Thonstad et al. 2014) ......................................................................................................................................................................................... 174
Figure 1-9: Effective force verses displacement for the pretensioned, rocking column with hybrid-bar-socket connection between the precast column and the precast bent cap (Thonstad et al. 2014) ................................................................................................................................................................................................................................................................ 175
Figure 1-10: Pretensioned, rocking column with hybrid-bar-socket connection between the precast column and the precast bent cap by end of testing (Thonstad et al. 2014) ........................................................................................................................................................................................................................................................................ 175
Figure 2-1: Elevation of the prototype bridge (Ranf 2008) ............................................................. 176
Figure 2-2: Superstructure cross section of the prototype bridge (Ranf 2008) ......................... 176
Figure 2-3: Typical reinforcement for the Conventional Bridge columns ........................................ 176
Figure 2-4: Moment-rotation and base shear-drift relationship for each column ................. 177
Figure 2-5: Typical reinforcement for the Resilient Bridge columns .............................................. 178
Figure 2-6: Distribution of masses over the Resilient Bridge superstructure ......................... 179
Figure 2-7: Axial stress and the decompression stress at the rocking interface .......... 180
Figure 2-8: Linear-elastic SAP model for construction sequence design...................... 180
Figure 2-9: Case 1, Placing the concrete blocks over with timber shoring for the outer
bents ............................................................................................................................ 181
Figure 2-10: Case 2, post-tensioning the superstructure to 66.67% of the jacking force 182
Figure 2-11: Case 3, placing the first concrete block in each span near the bent caps... 182
Figure 2-12: Case 4, placing the concrete second block in each span near the bent caps
................................................................................................................................. 182
Figure 2-13: Case 5, placing the concrete third block in each span near the bent caps.. 183
Figure 2-14: Case 6, post-tensioning the superstructure to 100% of the jacking force.. 183
Figure 2-15: Case 7, after placing the steel baskets, removing the shoring and grouting
the hybrid-bar-socket connection ............................................................................ 183
Figure 2-16: Rendering for the Resilient Bridge after post-tensioning ....................... 184
Figure 2-17: Time-scaled acceleration histories for Motions 14A, 14B2 and 14C........ 185
Figure 2-18: Scaled spectral accelerations for Motions for Motions 14A, 14B2 and 14C
using 5% damping ratio ........................................................................................... 186
Figure 2-19: Scaled spectral displacements for Motions for Motions 14A, 14B2 and 14C
using 5% damping ratio ........................................................................................... 186
Figure 3-1: Steel cage for Bent 2 column at socket end ............................................ 187
Figure 3-2: Prestressing the columns ........................................................................ 187
Figure 3-3: Concrete casting of the column end ....................................................... 188
Figure 3-4: Bent 3, columns after casting ................................................................. 188
Figure 3-5: Reduced section duct and reinforcing bar ducts of Bent Cap 2............. 189
Figure 3-6: Concrete casting of Bent Cap 2 ............................................................. 189
Figure 3-7: Octagonal shapes at column location to protect the pretension hardware ... 190
Figure 3-8: Footing 3 steel cage ............................................................................... 190
Figure 3-9: Lifting the south column of Bent 2 ....................................................... 191
Figure 3-10: Installing the south column of Bent 3 .................................................. 191
Figure 3-11: Installing the south column of Bent 1 ................................................. 192
Figure 3-12: Socket connection of the south column of Bent 1 inside the footing over octagonal shaped pocket ................................................................. 192
Figure 3-13: Bents 2 and 3 after installing the columns with shoring .................. 193
Figure 3-14: Bent 1 after installing the columns with shoring ........................... 193
Figure 3-15: Bent Cap 2 placement .................................................................. 194
Figure 3-16: Bent Cap 3 placement ................................................................. 194
Figure 3-17: Column reduced section and bars inside Bent Cap 2 ..................... 195
Figure 3-18: Casting concrete around the south column for Footing 3 ............ 195
Figure 3-19: Bent 3 after concrete casting ....................................................... 196
Figure 3-20: Placing the spacer block on spacers and foam ............................ 196
Figure 3-21: Setting Bent 3 on shims and foams over spacer block ................. 197
Figure 3-22: Setting Bent 2 on shims and foams over Shake Table 2 .............. 197
Figure 3-23: Bent 1 on shake table with bent cap shoring .............................. 198
Figure 3-24: Alignment of Bent Cap 1 over the columns ................................. 198
Figure 3-25: Adding 4x6 lumber pieces to support the bent cap during the grouting .... 199
Figure 3-26: Placement of fiber grout over the top of north column with duct tape confining the grout ................................................................................. 199
Figure 3-27: North column after placing the bent cap over the grout ............... 200
Figure 3-28: Shoring for Bent 2 before the grout process ................................. 200
Figure 3-29: Lowering and aligning the bent cap over the columns ................. 201
Figure 3-30: Bent Cap 2 placing over the non-fiber grout ............................... 201
Figure 3-31: Checking the alignment of the Bent Cap 2 after grouting ............. 202
Figure 3-32: Grouting the first column bar into the bent cap using non fiber grout for Bent 2 ......................................................................................... 202
Figure 3-33: Grouting the duct between the reduced column section and Bent Cap 2 .. 203
Figure 3-34: Fully grouting Bent Cap 2 to the columns (30 minutes after the grouting) 203
Figure 3-35: Aligning the Bent Cap 3 over the columns ................................. 204
Figure 3-36: Fiber grouting of column tops of Bent 3 ..................................... 204
Figure 3-37: 5x5x1 in. foam pieces used to protect the transverse ducts during pouring the hydrostone ........................................................................... 205
Figure 3-38: Filling the longitudinal joints between the beams by hydrostone......... 205
Figure 3-39: Transverse post-tensioning the three beams .................................... 206
Figure 3-40: Beams after completion of transverse post-tensioning of each span ....... 206
Figure 3-41: Lifting the west span ........................................................................ 207
Figure 3-42: West span supported by jacks .......................................................... 207
Figure 3-43: Alignment of the east span ............................................................... 208
Figure 3-44: U shapes used to protect the duct during the filling process .............. 208
Figure 3-45: Filling the joints between bent cap and superstructure beams for Bent 1....................................................... 209
Figure 3-46: Grouting the Bent 2 footing to Shake Table 2 .................................. 209
Figure 3-47: Placement the concrete blocks over the outer bents ....................... 210
Figure 3-48: Removing the jacks at east side of Bent 2 ...................................... 210
Figure 3-49: Post-tensioning the middle tendon .................................................. 211
Figure 3-50: Placement of all concrete blocks after the first stage of post-tensioning... 211
Figure 3-51: Post-tensioning the north tendon after placing all concrete blocks ........ 212
Figure 3-52: Final bridge condition after post-tensioning .................................... 212
Figure 3-53: Grouting the ducts of the north column of Bent 3 .......................... 213
Figure 3-54: Grouting the ducts of the south column of Bent 1 .......................... 213
Figure 3-55: Placement of steel plates at the middle of each span ..................... 214
Figure 4-1: Maximum gap opening for Bent 1, north column, bottom connection, Motion 15 through Motion 21C ................................................................. 215
Figure 4-2: Maximum gap opening for Bent 1, north column, top connection, Motion 16 through Motion 21C ................................................................. 216
Figure 4-3: Maximum gap opening for Bent 1, south column, bottom connection, Motion 16 through Motion 20C ................................................................. 217
Figure 4-4: Maximum gap opening for Bent 1, south column, top connection, Motion 18 through Motion 20C ................................................................. 218
Figure 4-5: Damage progression for Bent 1, north column, bottom connection, high amplitude motions ................................................................. 219
Figure 4-6: Damage progression for Bent 1, north column, top connection, high amplitude motions ................................................................. 220
Figure 4-7: Damage progression for Bent 1, south column, bottom connection, high amplitude motions.......................... 221

Figure 4-8: Damage progression for Bent 1, south column, top connection, high amplitude motions.................................................. 222

Figure 4-9: Damage progression for Bent 2, north column, bottom and top connections, high amplitude motions.................................................. 223

Figure 4-10: Damage progression for Bent 2, south column, bottom connection, high amplitude motions.................................................. 224

Figure 4-11: Damage progression for Bent 2, south column, top connection, high amplitude motions.................................................. 225

Figure 4-12: Damage progression for Bent 3, north column, bottom connection, high amplitude motions.................................................. 226

Figure 4-13: Damage progression for Bent 3, north column, top connection, high amplitude motions.................................................. 227

Figure 4-14: Damage progression for Bent 3, south column, bottom connection, high amplitude motions.................................................. 228

Figure 4-15: Damage progression for Bent 3, south column, top connection, high amplitude motions.................................................. 229

Figure 4-16: Final damage for the north column of Bent 1 .................................. 230

Figure 4-17: Final damage for the south column of Bent 1 .................................. 231

Figure 4-18: Final damage for the north column of Bent 2 .................................. 232

Figure 4-19: Final damage for the south column of Bent 2 .................................. 233

Figure 4-20: Final damage for the north column of Bent 3 .................................. 234

Figure 4-21: Final damage for the south column of Bent 3 .................................. 235

Figure 4-22: Average strain ductility envelopes at the interface for each bent ........ 236

Figure 4-23: Longitudinal reinforcement strain ductility envelopes for the north bar of Bent 3 at bottom connection of the north column................................. 237

Figure 4-24: Tensile strain distribution at top connection for the south bar in Bent 1, north column for given drift ratios .................................................. 237

Figure 4-25: Average strand strain envelopes for each column without offset .......... 238
Figure 4-26: Average strand strain envelopes for each column .............................................. 239
Figure 4-27: Bonded strands inside the south column of Bent 2 at 8 in. from the column
interface after the bent demolishing .................................................................................... 240
Figure 4-28: Average end rotation for each bent during Motion 14A ................................. 241
Figure 4-29: Average end rotation for each bent during Motion 18 ................................. 242
Figure 4-30: Average connection rotation-drift contribution for Bents 1 and 3 by the end
of Motion 19 ......................................................................................................................... 243
Figure 4-31: Motion 14A, drift and acceleration response histories for each bent .......... 244
Figure 4-32: Motion 14A, transverse deflection plot of superstructure at instant of peak
and valley deflections for each bent .................................................................................... 245
Figure 4-33: Motion 16, drift and acceleration response histories for each bent ............ 246
Figure 4-34: Motion 16, transverse deflection plot of superstructure at instant of peak and
valley deflections for each bent .......................................................................................... 247
Figure 4-35: Motion 19, drift and acceleration response histories for each bent .......... 248
Figure 4-36: Motion 19, transverse deflection plot of superstructure at instant of peak and
valley deflections for each bent .......................................................................................... 249
Figure 4-37: Motion 20B, drift and acceleration response histories for each bent .......... 250
Figure 4-38: Motion 20B, transverse deflection plot of superstructure at instant of peak
and valley deflections for each bent .................................................................................... 251
Figure 4-39: Motion 21C, drift and acceleration response histories for each bent ........ 252
Figure 4-40: Motion 21C, transverse deflection plot of superstructure at instant of peak
and valley deflections for each bent .................................................................................... 253
Figure 4-41: Maximum drift and residual drift ratios for each bent from Motion 14A to
the end of testing .................................................................................................................. 254
Figure 4-42: Motion 14A, vertical deflection plot of each bent cap at instant of bent peak
and valley deflections ........................................................................................................ 255
Figure 4-43: Motion 16, vertical deflection plot of each bent cap at instant of bent peak
and valley deflections ........................................................................................................ 256
Figure 4-44: Motion 19, vertical deflection plot of each bent cap at instant of bent peak
and valley deflections ........................................................................................................ 257
Figure 4-45: Motion 20B, vertical deflection plot of each bent cap at instant of bent peak and valley deflections ........................................................................................................... 258

Figure 4-46: Motion 21C, vertical deflection plot of each bent cap at instant of bent peak and valley deflections ........................................................................................................... 259

Figure 4-47: Measured base shear for each bent and total measured base shear for the bridge during Motion 14A ........................................................................................................... 260

Figure 4-48: Measured base shear for each bent and total measured base shear for the bridge during Motion 14B1 ........................................................................................................... 261

Figure 4-49: Measured base shear for each bent and total measured base shear for the bridge during Motion 14C ........................................................................................................... 262

Figure 4-50: Measured base shear for each bent and total measured base shear for the bridge during Motion 15 ........................................................................................................... 263

Figure 4-51: Measured base shear for each bent and total measured base shear for the bridge during Motion 16 ........................................................................................................... 264

Figure 4-52: Measured base shear for each bent and total measured base shear for the bridge during Motion 17 ........................................................................................................... 265

Figure 4-53: Measured base shear for each bent and total measured base shear for the bridge during Motion 18 ........................................................................................................... 266

Figure 4-54: Measured base shear for each bent and total measured base shear for the bridge during Motion 19 ........................................................................................................... 267

Figure 4-55: Base shear envelopes measured from accelerometers and inertia masses. 268

Figure 4-56: Response spectra for transverse table acceleration before testing, and after Motions 14C, 16, and 17 ........................................................................................................... 269

Figure 4-57: Response spectra for transverse table acceleration after Motions 18, 19, and 20B, and at the end of testing ........................................................................................................... 270

Figure 4-58: FFT plots for each bent before testing, after Motions 14C, 16, and 17 ...... 271

Figure 4-59: FFT plots for each bent after Motions 18, 19, and 20B, and by the end of testing ................................................................................................................................. 272

Figure 4-60: Transfer function plots before testing, and after Motions 14C, 16, and 17 273
Figure 4-61: Transfer function plots after Motions 18, 19, and 20B, and by the end of testing ................................................................. 274

Figure 4-62: Bridge period and damping ratio from transfer function after white noise waves ................................................................. 275

Figure 4-63: Comparison between table and bent displacement during Motion 20B .... 276

Figure 4-64: Damping and period estimation using logarithmic decrement for free vibration part of Motion 20B ................................................................. 277

Figure 4-65: Comparison between table and bent displacement during Motion 21C .... 278

Figure 4-66: Damping and period estimation using logarithmic decrement for free vibration part of Motion 21C ................................................................. 279

Figure 4-67: Damping and period estimation using logarithmic decrement for free vibration during square wave after Motion 21A ................................................................. 280

Figure 4-68: Comparison between the measured displacement and the calculated displacement from accelerometers for each bent during Motion 14A ............. 281

Figure 4-69: Comparison between the measured displacement and the calculated displacement from accelerometers for each bent during Motion 16 ..................... 282

Figure 4-70: Comparison between the measured displacement and the calculated displacement from accelerometers for each bent during Motion 19 ..................... 283

Figure 4-71: Comparison between the measured displacement and the calculated displacement from accelerometers for each bent during Motion 20B ............. 284

Figure 4-72: Comparison between the measured displacement and the calculated displacement from accelerometers for each bent during Motion 21C ............. 285

Figure 5-1: Rotation of column at north-south direction (transverse direction) ......... 286

Figure 5-2: Rotation of column at east-west direction (longitudinal direction) .......... 286

Figure 5-3: Connection geometry during gap opening at north-east components used to perform the equations of the Displacement Method ......................................................... 287

Figure 5-4: Displacement Method and strain gauge reading comparison for bottom connection of north column of Bent 1 (1NB) during Motion 15 ......................... 288

Figure 5-5: Displacement Method and strain gauge reading comparison for top connection of north column of Bent 1 (1NT) during Motion 15 ......................... 288
Figure 5-6: Displacement Method and strain gauge reading comparison for bottom connection of south column of Bent 1 (1SB) during Motion 15 ............................ 289
Figure 5-7: Displacement Method and strain gauge reading comparison for top connection of south column of Bent 1 (1ST) during Motion 15............................ 289
Figure 5-8: Displacement Method and strain gauge reading comparison for bottom connection of north column of Bent 3 (3NB) during Motion 15 ....................... 290
Figure 5-9: Displacement Method and strain gauge reading comparison for top connection of north column of Bent 3 (3NT) during Motion 15 ....................... 290
Figure 5-10: Displacement Method and strain gauge reading comparison connection of south column of Bent 3 (3SB) during Motion 15 ............................... 291
Figure 5-11: Displacement Method and strain gauge reading comparison for top connection of south column of Bent 3 (3ST) during Motion 15 ....................... 291
Figure 5-12: Displacement Method and strain gauge maximum strain comparison for north and south bars of Bents 1 and 3 during Motion 15 ............................... 292
Figure 5-13: Strains using Displacement Method for 1NB during Motion 17 ........ 292
Figure 5-14: Flow chart for the Low Cycle Fatigue Method for each bar ............. 293
Figure 5-15: Cumulative fatigue damage index for all longitudinal bars after Motion 15 ................................................................................................................. 294
Figure 5-16: Cumulative fatigue damage index for all longitudinal bars after Motion 16 ................................................................................................................. 295
Figure 5-17: Cumulative fatigue damage index for all longitudinal bars after Motion 17 ................................................................................................................. 296
Figure 5-18: Cumulative fatigue damage index for all longitudinal bars after Motion 18 ................................................................................................................. 297
Figure 5-19: Cumulative fatigue damage index for all longitudinal bars after Motion 19 ................................................................................................................. 298
Figure 5-20: Cumulative fatigue damage index for all longitudinal bars of Bent 1 after Motion 20A ................................................................. 299
Figure 5-21: Cumulative fatigue damage index for all longitudinal bars of Bent 1 after Motion 20B ................................................................. 300
Figure 5-22: Cumulative fatigue damage index for all longitudinal bars of Bent 1 after Motion 21A .......................................................... 301
Figure 5-23: Cumulative fatigue damage index for all longitudinal bars of Bent 1 after Motion 21B .......................................................... 302
Figure 5-24: Cumulative fatigue damage index for all longitudinal bars of Bent 1 after Motion 21C .......................................................... 303
Figure 5-25: Cumulative fatigue damage index for the bottom connection bars of north column of Bent 1 .......................................................... 304
Figure 5-26: Cumulative fatigue damage index for the top connection bars of north column of Bent 1 .......................................................... 304
Figure 5-27: Cumulative fatigue damage index for the bottom connection bars of south column of Bent 1 .......................................................... 305
Figure 5-28: Cumulative fatigue damage index for the top connection bars of south column of Bent 1 .......................................................... 305
Figure 5-29: Cumulative fatigue damage index for the bottom connection bars of north column of Bent 2 .......................................................... 306
Figure 5-30: Cumulative fatigue damage index for the top connection bars of north column of Bent 2 .......................................................... 306
Figure 5-31: Cumulative fatigue damage index for the bottom connection bars of south column of Bent 2 .......................................................... 307
Figure 5-32: Cumulative fatigue damage index for the top connection bars of south column of Bent 2 .......................................................... 307
Figure 5-33: Cumulative fatigue damage index for the bottom connection bars of north column of Bent 3 .......................................................... 308
Figure 5-34: Cumulative fatigue damage index for the top connection bars of north column of Bent 3 .......................................................... 308
Figure 5-35: Cumulative fatigue damage index for the bottom connection bars of south column of Bent 3 .......................................................... 309
Figure 5-36: Cumulative fatigue damage index for the top connection bars of south column of Bent 3 .......................................................... 309
Figure 5-37: Filtered sound using 1NB GoPro slow motion during Motion 17 ........... 310
Figure 5-38: Filtered sound using 1NB GoPro slow motion during Motion 18 ........... 310
Figure 5-39: Filtered sound using 3SB GoPro slow motion during Motion 19 .......... 311
Figure 5-40: Connections after demolishing the bent ............................................ 312
Figure 6-1: Bent drift comparison using 2-DOF model ........................................ 313
Figure 6-2: Damage progression comparison during Motions 16 through 19 (Resilient Bridge on left and Conventional Bridge on right) ............................................. 314
Figure 6-3: Bent drifts and bridge COM displacement comparison during Motion 14A. .................................................................................................... 315
Figure 6-4: Bent base shear and total base shear comparison during Motion 14A. ....... 316
Figure 6-5: Bent drifts and bridge COM displacement comparison during Motion 15. 317
Figure 6-6: Bent base shear and total base shear comparison during Motion 15. ........ 318
Figure 6-7: Bent drift and Bridge COM displacement comparison during Motion 16. . 319
Figure 6-8: Bent base shear and total base shear comparison during Motion 16. ........ 320
Figure 6-9: Bent Drifts and COM displacement comparison during Motion 17 ........ 321
Figure 6-10: Bent Base shear and total base shear comparison during Motion 17 ....... 322
Figure 6-11: Bent Drifts and COM displacement comparison during Motion 18 ....... 323
Figure 6-12: Bent Base shear and total base shear comparison during Motion 18 ....... 324
Figure 6-13: Bent Drifts and COM displacement comparison during Motion 19 ....... 325
Figure 6-14: Bent Base shear and total base shear comparison during Motion 19 ....... 326
Figure 6-15: Residual drifts comparison during Motion 19 ....................................... 327
Figure 6-16: Base shear vs COM displacement for both the Resilient Bridge and the Conventional Bridge ................................................................. 328
Figure 6-17: Transfer function comparison for both bridges before testing and after Motions 14A and 17 ................................................................. 329
Figure 6-18: Transfer function comparison for both bridges after Motions 18 and 19. . 330
Figure 6-19: Change in fundamental period through the test for both the Resilient Bridge and the Conventional Bridge ................................................. 330
Figure 6-20: Average drift-rotation comparison for Bent 1 during Motions 14A and 18. ........................................................................................................... 331
Figure 6-21: Average drift-rotation comparison for Bent 2 during Motions 14A and 18. .................................................................................................................. 332
Figure 6-22: Average drift-rotation comparison for Bent 3 during Motions 14A and 18. .................................................................................................................. 333
Figure 6-23: Maximum drift ratio vs. corresponding average rotation for Bent 1 in both the Resilient Bridge and the Conventional Bridge. ......................................... 334
Figure 6-24: Maximum drift ratio vs. corresponding average rotation for Bent 2 in both the Resilient Bridge and the Conventional Bridge. ......................................... 334
Figure 6-25: Maximum drift ratio vs. corresponding average rotation for Bent 3 in both the Resilient Bridge and the Conventional Bridge. ......................................... 335
Figure 6-26: Average connection rotation-drift contribution comparison for Bent 1 in both the Resilient Bridge and the Conventional Bridge. ............................... 335
Figure 6-27: Average connection rotation-drift contribution comparison for Bent 3 in both the Resilient Bridge and the Conventional Bridge. ............................... 336
Figure 6-28: Tensile strain distribution comparison at the top connection for the south bar of Bent 3 north column of the Resilient Bridge (1NT) and the East bar of Bent 1 West column of the Conventional Bridge (1WT) for given drift ratios. ............... 336
Figure 6-29: Longitudinal reinforcement strain envelopes comparison at the both connection for the north bar of Bent 3 north column of the Resilient Bridge (3NB) and the West bar of Bent 3 West column of the Conventional Bridge (3WB)...... 337
Figure 6-30: Onsite bent construction time for both the Resilient Bridge and the Conventional Bridge. ............................................................................................................ 338
Figure 7-1: Three dimensional sketch for the numerical model of the Resilient Bridge 339
Figure 7-2: Three-dimensional sketch for the rocking connection at the top of the column .................................................................................................................. 340
Figure 7-3: Detailed sketch for the rocking column modeling..................................... 341
Figure 7-4: Section discretization .................................................................................. 342
Figure 7-5: Depth of neutral axis as a function of the strain in the longitudinal reinforcement (Ranf 2007) .................................................................................. 342
Figure 7-6: Drift ratios and base shear for each bent during Motion 14A............... 343
Figure 7-7: C.O.M. displacement and twisting displacement for the bridge during Motion 14A

Figure 7-8: Total base shear for the bridge during Motion 14A

Figure 7-9: Drift ratio vs. base shear for each bent during Motion 14A

Figure 7-10: Drift ratios and base shear for each bent during Motion 14B1

Figure 7-11: C.O.M. displacement and twisting displacement for the bridge during Motion 14B1

Figure 7-12: Total base shear for the bridge during Motion 14B1

Figure 7-13: Drift ratio vs. base shear for each bent during Motion 14B1

Figure 7-14: Drift ratios and base shear for each bent during Motion 14B2

Figure 7-15: C.O.M. displacement and twisting displacement for the bridge during Motion 14B2

Figure 7-16: Total base shear for the bridge during Motion 14B2

Figure 7-17: Drift ratio vs. base shear for each bent during Motion 14B2

Figure 7-18: Drift ratios and base shear for each bent during Motion 14C

Figure 7-19: C.O.M. displacement and twisting displacement for the bridge during Motion 14C

Figure 7-20: Total base shear for the bridge during Motion 14C

Figure 7-21: Drift ratio vs. base shear for each bent during Motion 14C

Figure 7-22: Drift ratios and base shear for each bent during Motion 15

Figure 7-23: C.O.M. displacement and twisting displacement for the bridge during Motion 15

Figure 7-24: Total base shear for the bridge during Motion 15

Figure 7-25: Drift ratio vs. base shear for each bent during Motion 15

Figure 7-26: Drift ratios and base shear for each bent during Motion 16

Figure 7-27: C.O.M. displacement and twisting displacement for the bridge during Motion 16

Figure 7-28: Total base shear for the bridge during Motion 16

Figure 7-29: Drift ratio vs. base shear for each bent during Motion 16
Figure 7-30: Base shear envelopes for each bent and total base shear for the bridge up to fracture

Figure 7-31: Strain ductility envelopes for each bent

Figure 7-32: Drift ratios for each bent during Motion 17

Figure 7-33: C.O.M. displacement for the bridge during Motion 17

Figure 7-34: Drift ratios for each bent during Motion 18

Figure 7-35: C.O.M. displacement for the bridge during Motion 18

Figure 7-36: Drift ratios for each bent during Motion 19

Figure 7-37: C.O.M. displacement for the bridge during Motion 19

Figure 7-38: Drift ratios for each bent during Motion 20B

Figure 7-39: C.O.M. displacement for the bridge during Motion 20B

Figure 7-40: Base shear envelopes for each bent and total base shear for the bridge up to Motion 19

Figure 7-41: Average connection rotation drift contribution for each bent

Figure 7-42: Average strand force envelopes for each column

Figure 7-43: Average strand force envelopes for each column with no offset

Figure 7-44: Average strand strain envelopes for each column

Figure 7-45: Average strand strain envelopes for each column with no offset

Figure 7-46: Cumulative fatigue damage index for Bent 1 from Motions 15 and 16

Figure 7-47: Cumulative fatigue damage index for Bent 1 from Motions 17 and 18

Figure 7-48: Cumulative fatigue damage index for Bent 2 from Motions 16 and 17

Figure 7-49: Cumulative fatigue damage index for Bent 2 from Motions 18 and 19

Figure 7-50: Cumulative fatigue damage index for Bent 2 from Motions 15 and 16

Figure 7-51: Cumulative fatigue damage index for Bent 3 from Motions 17 and 18

Figure 8-1: Percentage of fractured bars using Low Cycle Fatigue, Strain Threshold, Analytical Model, and Acoustic Emissions

Figure 8-2: Cumulative fatigue damage index for each bar after only Motion 17

Figure 8-3: Cumulative fatigue damage index for each bar after only Motion 18

Figure 8-4: Cumulative fatigue damage index for each bar after only Motion 20B
Figure 8-5: Drift ratio comparison for Motion 17: Experimental protocol and individual motion ......................................................................................................................... 373
Figure 8-6: Drift ratio comparison for Motion 18: Experimental protocol and individual motion ......................................................................................................................... 373
Figure 8-7: Drift ratio comparison for Motion 20B: Experimental protocol and individual motion ......................................................................................................................... 374
Figure 8-8: Number of fractured bars: experimental protocol and individual motions.. 374
Figure 8-9: Cumulative fatigue damage index for Bent 1 after the first three successive design level motion (DE) ......................................................................................................................... 375
Figure 8-10: Cumulative fatigue damage index for Bent 2 after the first three successive design level motion (DE) ......................................................................................................................... 376
Figure 8-11: Cumulative fatigue damage index for Bent 3 after the first three successive design level motion (DE) ......................................................................................................................... 377
Figure 8-12: Maximum numerical drift ratios and number of fractured bars for Bents 1 and 3 for different debonded lengths using bar sizes 3 and 4 .................. 378
Figure 8-13: Cumulative fatigue damage index in case of using bar # 4 and 24 d#4 as debonded length during Motion 19 ......................................................................................................................... 379
Figure 8-14: Drift ratio comparison using bar size # 4 during Motion 19: debonded lengths 16d#4 and 24d#4 .......................................................................................................................... 380
Figure 8-15: Total base shear envelopes for the bridge case of using bar size # 3 with 9 in. debonded length and using bar size # 4 with 13 in. debonded length during the 221% Design Level Motion ......................................................................................................................... 380
Figure 8-16: Column cross section for prototype bents .......................................................................................................................... 381
Figure 8-17: Debonded length to prevent fracture for different bar sizes .................. 382
Figure 8-18: Response spectrum comparison for the 150% Design Level Motion and FEMA ground motions. .......................................................................................................................... 382
Figure 8-19: Response spectrum comparison for the 150% Design Level Motion and design response spectra for multiple cities .......................................................................................................................... 383
Figure A-1: Overall specimen dimensions .......................................................................................................................... 385
Figure A-2: Bent dimensions in east-west direction.......................................................................................................................... 386
Figure A-3: Bent dimensions in north-south direction .................................................. 387
Figure A-4: Superstructure beam dimensions (Johnson 2006) .................................. 388
Figure A-5: Column dimensions ................................................................................ 389
Figure A-6: Bent Caps 1 and 3, beam reinforcement .................................................. 390
Figure A-7: Bent Caps 1 and 3, cantilever reinforcement .......................................... 391
Figure A-8: Bent Caps 1 and 3, ledge reinforcement ................................................... 392
Figure A-9: Bent Cap 2, beam reinforcement ............................................................. 393
Figure A-10: Bent Cap 2, ledge reinforcement ........................................................... 394
Figure A-11: Footing reinforcement of Bents 1 and 2 .................................................. 395
Figure A-12: Footing reinforcement of Bent 3 ............................................................ 396
Figure A-13: Spacer block reinforcement (Johnson 2006) ........................................... 397
Figure A-14: Superstructure beams reinforcement (Johnson 2006) ............................ 398
Figure A-15: Superstructure beams, ledge reinforcement (Johnson 2006) ............... 399
Figure A-16: Global coordinate system ..................................................................... 400
Figure A-17: Accelerometer locations ....................................................................... 401
Figure A-18: Displacement transducer locations ...................................................... 402
Figure A-19: Strand load cell locations ..................................................................... 403
Figure A-20: Curvature rod locations of Bent 1 ........................................................ 404
Figure A-21: Curvature rod locations of Bent 2 ........................................................ 405
Figure A-22: Curvature rod locations of Bent 3 ........................................................ 406
Figure A-23: Strain gage locations of Bent 1 ............................................................. 407
Figure A-24: Strain gage locations of Bent 2 ............................................................. 408
Figure A-25: Strain gage locations of Bent 3 ............................................................. 409
Figure B-1: Strains using the Displacement Method for 1NB during Motion 17 ...... 411
Figure B-2: Strains using the Displacement Method for 1NT during Motion 17 ........ 411
Figure B-3: Strains using the Displacement Method for 1SB during Motion 17 ....... 412
Figure B-4: Strains using the Displacement Method for 1ST during Motion 17 ....... 412
Figure B-5: Strains using the Displacement Method for 2NB during Motion 17 ...... 413
Figure B-6: Strains using the Displacement Method for 2NT during Motion 17 ....... 413
Figure B-7: Strains using the Displacement Method for 2SB during Motion 17 ...... 414
Figure B-8: Strains using the Displacement Method for 2ST during Motion 17 .......... 414
Figure B-9: Strains using the Displacement Method for 3NB during Motion 17 .......... 415
Figure B-10: Strains using the Displacement Method for 3NT during Motion 17 ........ 415
Figure B-11: Strains using the Displacement Method for 3SB during Motion 17 ........ 416
Figure B-12: Strains using the Displacement Method for 3ST during Motion 17 ....... 416
Figure B-13: Strains using the Displacement Method for 1NB during Motion 18 ....... 417
Figure B-14: Strains using the Displacement Method for 1NT during Motion 18 ...... 417
Figure B-15: Strains using the Displacement Method for 1SB during Motion 18 ...... 418
Figure B-16: Strains using the Displacement Method for 1ST during Motion 18 ...... 418
Figure B-17: Strains using the Displacement Method for 2NB during Motion 18 ...... 419
Figure B-18: Strains using the Displacement Method for 2NT during Motion 18 ........ 419
Figure B-19: Strains using the Displacement Method for 2SB during Motion 18 ........ 420
Figure B-20: Strains using the Displacement Method for 2ST during Motion 18 ........ 420
Figure B-21: Strains using the Displacement Method for 3NB during Motion 18 ...... 421
Figure B-22: Strains using the Displacement Method for 3NT during Motion 18 ....... 421
Figure B-23: Strains using the Displacement Method for 3SB during Motion 18 ...... 422
Figure B-24: Strains using the Displacement Method for 3ST during Motion 18 ...... 422
Figure B-25: Strains using the Displacement Method for 1NB during Motion 19 ....... 423
Figure B-26: Strains using the Displacement Method for 1NT during Motion 19 ....... 423
Figure B-27: Strains using the Displacement Method for 1SB during Motion 19 ....... 424
Figure B-28: Strains using the Displacement Method for 1ST during Motion 19 ....... 424
Figure B-29: Strains using the Displacement Method for 2NB during Motion 19 ...... 425
Figure B-30: Strains using the Displacement Method for 2NT during Motion 19 ....... 425
Figure B-31: Strains using the Displacement Method for 2SB during Motion 19 ....... 426
Figure B-32: Strains using the Displacement Method for 2ST during Motion 19 ....... 426
Figure B-33: Strains using the Displacement Method for 3NB during Motion 19 ...... 427
Figure B-34: Strains using the Displacement Method for 3NT during Motion 19 ....... 427
Figure B-35: Strains using the Displacement Method for 3SB during Motion 19 ....... 428
Figure B-36: Strains using the Displacement Method for 3ST during Motion 19 ....... 428
Figure B-37: Strains using the Displacement Method for 1NB during Motion 20A ...... 429
Figure B-38: Strains using the Displacement Method for 1NT during Motion 20A... 429
Figure B-39: Strains using the Displacement Method for 1SB during Motion 20A ... 430
Figure B-40: Strains using the Displacement Method for 1ST during Motion 20A ... 430
Figure B-41: Strains using the Displacement Method for 2NB during Motion 20A... 431
Figure B-42: Strains using the Displacement Method for 2NT during Motion 20A... 431
Figure B-43: Strains using the Displacement Method for 2SB during Motion 20A ... 432
Figure B-44: Strains using the Displacement Method for 2ST during Motion 20A ... 432
Figure B-45: Strains using the Displacement Method for 3NB during Motion 20A... 433
Figure B-46: Strains using the Displacement Method for 3NT during Motion 20A... 433
Figure B-47: Strains using the Displacement Method for 3SB during Motion 20A ... 434
Figure B-48: Strains using the Displacement Method for 3ST during Motion 20A ... 434
Figure B-49: Strains using the Displacement Method for 1NB during Motion 20B... 435
Figure B-50: Strains using the Displacement Method for 1NT during Motion 20B... 435
Figure B-51: Strains using the Displacement Method for 1SB during Motion 20B ... 436
Figure B-52: Strains using the Displacement Method for 1ST during Motion 20B... 436
Figure B-53: Strains using the Displacement Method for 2NB during Motion 20B... 437
Figure B-54: Strains using the Displacement Method for 2NT during Motion 20B... 437
Figure B-55: Strains using the Displacement Method for 2SB during Motion 20B ... 438
Figure B-56: Strains using the Displacement Method for 2ST during Motion 20B... 438
Figure B-57: Strains using the Displacement Method for 3NB during Motion 20B... 439
Figure B-58: Strains using the Displacement Method for 3NT during Motion 20B... 439
Figure B-59: Strains using the Displacement Method for 3SB during Motion 20B ... 440
Figure B-60: Strains using the Displacement Method for 3ST during Motion 20B... 440
Figure B-61: Strains using the Displacement Method for 1NB during Motion 21A... 441
Figure B-62: Strains using the Displacement Method for 1NT during Motion 21A... 441
Figure B-63: Strains using the Displacement Method for 1SB during Motion 21A ... 442
Figure B-64: Strains using the Displacement Method for 1ST during Motion 21A ... 442
Figure B-65: Strains using the Displacement Method for 2NB during Motion 21A... 443
Figure B-66: Strains using the Displacement Method for 2NT during Motion 21A... 443
Figure B-67: Strains using the Displacement Method for 2SB during Motion 21A ... 444
Figure C-1: Cumulative fatigue damage index for each bar during Motion 18: bar size .......................................................... 460
Figure C-2: Cumulative fatigue damage index for each bar during Motion 18: bar size 460
Figure C-3: Cumulative fatigue damage index for each bar during Motion 18: bar size 461
Figure C-4: Cumulative fatigue damage index for each bar during Motion 18: bar size 461
Figure C-5: Cumulative fatigue damage index for each bar during Motion 18: bar size 462
Figure C-6: Cumulative fatigue damage index for each bar during Motion 18: bar size 462
Figure C-7: Cumulative fatigue damage index for each bar during Motion 18: bar size 463
Figure C-8: Cumulative fatigue damage index for each bar during Motion 18: bar size 463
Figure C-9: Cumulative fatigue damage index for each bar during Motion 19: bar size 464
Figure C-10: Cumulative fatigue damage index for each bar during Motion 19: bar size .......................................................... 464
Figure C-11: Cumulative fatigue damage index for each bar during Motion 19: bar size .......................................................... 465
Figure C-12: Cumulative fatigue damage index for each bar during Motion 19: bar size .......................................................... 465
Figure C-13: Cumulative fatigue damage index for each bar during Motion 19: bar size .......................................................... 466
Figure C-14: Cumulative fatigue damage index for each bar during Motion 19: bar size .......................................................... 466
Figure C-15: Cumulative fatigue damage index for each bar during Motion 19: bar size .......................................................... 467
Figure C-16: Cumulative fatigue damage index for each bar during Motion 19: bar size #4 and 33d#3 debonded length .......................................................... 467
Figure C-17: Drift ratio comparison using bar size #3 during Motion 18: debonded lengths 24d#3 and 27d#3 .......................................................... 468
Figure C-18: Drift ratio comparison using bar size #3 during Motion 18: debonded lengths 24d#3 and 30d#3 .......................................................... 468
Figure C-19: Drift ratio comparison using bar size #3 during Motion 18: debonded lengths 24d#3 and 33d#3 .......................................................... 469
Figure C-20: Drift ratio comparison using bar size #4 during Motion 18: debonded lengths 24d#3 and 27d#3 ........................................................................................................................................ 469
Figure C-21: Drift ratio comparison using bar size #4 during Motion 18: debonded lengths 24d#3 and 30d#3 ........................................................................................................................................ 470
Figure C-22: Drift ratio comparison using bar size #4 during Motion 18: debonded lengths 24d#3 and 33d#3 ........................................................................................................................................ 470
Figure C-23: Drift ratio comparison using bar size #3 during Motion 19: debonded lengths 24d#3 and 27d#3 ........................................................................................................................................ 471
Figure C-24: Drift ratio comparison using bar size #3 during Motion 19: debonded lengths 24d#3 and 30d#3 ........................................................................................................................................ 471
Figure C-25: Drift ratio comparison using bar size #3 during Motion 19: debonded lengths 24d#3 and 33d#3 ........................................................................................................................................ 472
Figure C-26: Drift ratio comparison using bar size #4 during Motion 19: debonded lengths 24d#3 and 27d#3 ........................................................................................................................................ 472
Figure C-27: Drift ratio comparison using bar size #4 during Motion 19: debonded lengths 24d#3 and 30d#3 ........................................................................................................................................ 473
Figure C-28: Drift ratio comparison using bar size #4 during Motion 19: debonded lengths 24d#3 and 33d#3 ........................................................................................................................................ 473
Notation

The following symbols are used in this work:

For the Resilient Bridge:

**1NT**: Top connection (Connection Location) of North column (Column notation) of Bent 1 (Bent Number).

**1NT-N**: North Bar of the Top connection (Connection Location) of North column (Column notation) of Bent 1 (Bent Number).

Connection Location: Top or Bottom
Column Notation: North or South
Bent Number: Bent 1 or Bent 2 or Bent 3

For the Conventional Bridge:

**1ET**: Top connection (Connection Location) of East column (Column notation) of Bent 1 (Bent Number).

Connection Location: Top or Bottom
Column Notation: East or West
Bent Number: Bent 1 or Bent 2 or Bent 3
Chapter 1. Introduction

1.1 Background

Increasingly, bridge owners are adopting methods for accelerating onsite construction to reduce construction impacts on the traveling public, especially in urbanized areas. One method for accelerating construction is to prefabricate components or, in some cases, entire portions of the bridge. Prefabrication decreases onsite construction time and has the additional benefits of higher construction quality and greater work-zone safety (FHWA 2011). Although these techniques are common in low seismic zones, owners are reluctant to use them in areas of high seismicity because of the high demands on the precast connections. Many strategies have been developed to shorten construction time onsite, but their seismic performance provides no improvement over that of cast-in-place construction (Marsh et al. 2011).

In seismic regions, bridges should ideally have the ability to resist moderate earthquakes without damage, to withstand extreme earthquakes without collapse, and to return to service quickly. Conventional reinforced concrete bridge design relies on large inelastic deformations at the ends of the columns during an earthquake. This leads to flexural cracking and crushing, eventually the spalling of cover concrete and exposed longitudinal and transverse reinforcement. Longitudinal reinforcement fracture is typically due to bar buckling and subsequent re-straightening, which occurs along with yielding and fracturing of the transverse reinforcement.

This dissertation documents and provides the development of a new bridge system that attempts to delay or even eliminate damage to the column concrete after an earthquake
using prefabricated, pretensioned columns that utilize partially unbonded conventional reinforcement and unbonded pretensioning strands.

### 1.2 Literature Review

Other researchers have been working on ways to accelerate construction and mitigate damage during earthquakes. For example, some of these systems provide confinement to the plastic hinge regions of the column through either steel jackets (Schaefer 2013; Kennedy 2015) or fiber reinforced concrete (Kavianipour and Saiidi 2012; Panagiotou et al. 2015) in order to prevent cover spalling. By retaining the concrete cover, bar buckling is prevented, and fracture of longitudinal reinforcing steel is instead attributed to low-cycle fatigue, since the reinforcing bars undergo many large strain reversals during the ground motion. Other researcher have examined ways to use new materials to mitigate damage (Varela and Saiidi, 2014; Nakashoji and Saiidi, 2014; Trono et al. 2014; Motaref et al. 2010) and segmental construction (Motaref et al. 2014; Sideris et al. 2014). This dissertation focuses on utilizing a full prefabricated columns and conventional materials to reduce earthquake damage and increase construction speed. A survey of the previous work related to this topic is provided as background for the proposed system.

#### 1.2.1 Socket Connection at the Bottom of the Column

Haraldsson et al. (2011) tested three columns to develop a recommendation for socket connection between precast column and cast-in-place footing. The socket connection concept intentionally roughens the concrete in the embedded part of the column inside the cast-in-place footing. The roughness of the concrete in the embedded part of the column is
a way to improve the transfer of forces from the column to the footing (Thonstad et al. 2014).

Three specimens were tested under cyclic loading at the Structural Laboratory at the University of Washington. The first specimen represented a precast equivalent of a typical Washington Department of Transportation cast-in-place column-to-footing connection, with a socket embedment depth of 1.1 times the column diameter with flexural reinforcement in a slot at the embedment part of the column. The second specimen was similar to the first specimen but without flexural reinforcement and slot. The third specimen was similar to the second specimen but with depth of 0.5 times the column diameter to study the effect of the embedment length of the socket connection. None of the three specimens had crossing reinforcement between the precast column and the cast-in-place footings (Thonstad et al. 2014). Figure 1-1 shows the socket connection prior to the casting of the concrete of the footing.

The construction of the specimen showed advantages over many precast connections because of avoiding the use of the grout and the high tolerance that the connection provides. In the cyclic loading tests, the response of all three specimens was nearly identical to that of comparable cast-in-place specimens. No damage occurred to the footing in any specimen. Figure 1-2 shows the damage in the column in second specimen; the damage was similar to the damage in the cast-in-place columns. In the proposed system, the socket connection is used at the bottom connection between the precast column and cast-in-place footing.
1.2.2 Hybrid-Bar-Socket Connection at the Top of the Column

For non-prestressed columns, a new connection between the precast column and precast bent cap was developed at the University of Washington. The connection involved column bars being grouted into corrugated metal ducts in precast bent cap (Pang et al. 2008). Aligning and adjusting a large number of small size bars into precast ducts in a bent cap is difficult, so the connection utilizes fewer large size bars. The use of large size bars led to the need for longer bonded length inside the bent cap. Figure 1-3 shows a sketch for the large bar grouted duct connection.

For prestressed columns, the strands need to be anchored to the bent cap. A hybrid-bar-socket connection was developed between the column and bent cap, as shown schematically in Figure 1-4. This connection works similarly in concept to the socket connection between the precast column and cast-in-place footing. The difference between this connection and the socket connection is: 1) The column section is reduced at the top and is fitted into a metal duct in the precast bent cap with a roughened exterior surface, 2) Several smaller ducts surrounding the central duct accept the column longitudinal bars that are also protruding from the top of the clear height of the column to facilitate the transfer of forces into the bent cap, and 3) Grout is used to anchor the bars and reduced section into the bent cap, as well as to provide construction tolerances between the precast columns and bent caps (Thonstad et al. 2014).

Davis et al. (2012) tested one pretensioned column with hybrid-bar-socket connection between the precast column and bent cap. The response of the specimen was controlled by the column properties with no damage occurring in the connection region. Figure 1-5 shows the damage in the column by the end of testing.
1.2.3 Pretensioned Columns

One method of reducing damage that has been investigated more extensively is post-tensioning the column (Rouse and Billington 2003; Billington and Yoon 2004; Cohagen et al. 2008; Yamashita and Sanders 2009; Ou et al. 2010; Mashal and Palermo 2014; Guerrini et al. 2015). An alternative to this is pretensioning the column in a prefabricated unit. The advantages of using pretensioning over post-tensioning include:

1- Saving the construction time at the construction site by prefabricating the element.

2- Providing compression forces to the column that prevent the column from being cracked during lifting and handling.

3- Eliminating corrosion because of the absence of the anchorage devices.

By pretensioning the precast column using partially unbonded strands, post-earthquake residual displacements are reduced. The strands run through the column and are bonded in the column embedded in the footing and the reduced section of the column inside the bent cap, whereas the strands are unbonded through the clear height of the column to provide sufficient length to distribute the elongation over a longer length. Because the unbonded length of the tendon is large in comparison to the expected deformations, the strands remain essentially elastic during the event. These elastic elements provide additional restoring forces that promote the structure re-centering, returning the columns to their original position after the earthquake (Thonstad et al. 2014).

Davis et al. (2012) tested two columns using unbonded prestressing strands with socket connection between the column and footing (PreT-SF) and with hybrid-bar-socket connection between the column and bent cap (PreT-CB). Figure 1-5 shows the damage of
the pretensioned column with hybrid-bar-socket connection between the column and bent cap. Figure 1-6 and Figure 1-7 show the effective force versus displacement for both columns. The unloading curves showed that the pretensioned specimens would be likely to re-center better, and have lower residual drifts than the conventional RC specimens without pretensioning. Both pretensioned columns returned to within approximately 1% of vertical after being displaced laterally to a drift ratio of 10%. The damage of these column was similar to the damage of the conventional columns including spalling, reinforcement exposure, spiral exposure, reinforcement buckling and reinforcement fracture.

1.2.4 Subassembly Column Tests for the Proposed System

As part of this project, subassembly tests for column-to-footing and column-to-bent cap were tested under cyclic loading at the Structural Laboratory at the University of Washington. The columns used the column rocking concept to mitigate the damage to the concrete (Schaefer et al. 2013; Kennedy 2015). The difference between these specimens and specimens PreT-SF and PreT-CB were in the use of confining tube “steel shoe” and partially debonding for the longitudinal reinforcement at the interface to promote the rocking behavior. Figure 1-8 and Figure 1-9 show the effective force versus displacement for both columns.

The response of both specimens was dominated by the concentration of the rotations at the column interface, resulting in nearly identical hysteretic behaviors between the two columns, as shown in Figures 1-8 and 1-9. The connections maintained their ability to re-center and retained over 80% of their peak flexural capacity out to drift ratios of more than 10%. In both tests, the column end details successfully suppressed any concrete
spalling and bar buckling. The only significant damage was the eventual fracture of the longitudinal bars at approximately 6% and 7% drift for the footing and bent cap specimens respectively. This small drift difference was consistent with the length of local debonding of the longitudinal reinforcement, which was 33% longer in the bent cap connection. The prestressing strands in the columns remained essentially elastic (below a strain of 0.0086) to the design target drift ratio of 3% (Thonstad et al. 2016). Figure 1-10 shows the pretensioned, rocking column with hybrid-bar-socket connection between the precast column and the precast bent cap at the end of testing.

1.3 Research Scope and Objectives

Building on the past research, a pretensioned rocking bridge system has been developed that achieves three major objectives, using only conventional materials:

1- Accelerates onsite construction time through the use of precast columns and bent caps.

2- Minimizes post-earthquake residual displacements and keeps the bridge functional after strong shaking by vertically pretensioning the columns.

3- Reduces damage to the column concrete by using a confined rocking detail at the ends of the columns.

In order to develop and then verify the proposed system, an experimental and analysis program has been conducted. The main objectives for the experimental parts of the study are to:

1- Confirm construction speed of the proposed system in a laboratory environment.
2- Confirm the damage mitigation for the concrete using the confined tubes.

3- Confirm the improved performance of the new system over the conventional design and construction.

As a result of the experimental study, it became clear that the fracture of the conventional reinforcement was a critical element of the system performance. Therefore a damage (fracture) investigation was conducted to:

1- Provide a simple acoustic approach for detecting the fracture of the longitudinal bars using the audio records.

2- Develop a new technique of estimating the strain response for each longitudinal bar from the column rotation at the end of the columns.

3- Develop strain threshold and low cycle fatigue models to identify the fracture of the longitudinal bars.

The objectives of the analytical investigations are to:

1- Develop a detailed analytical model of the tested bridge, which includes rocking interfaces, debonded longitudinal bars and debonded prestressing strands.

2- Develop a detailed analytical model of the tested bridge that captures the damage of the longitudinal reinforcement using implicit fatigue models.

3- Study the effect of bar size, debonded length and prestressing level on delaying the fracture of the longitudinal bars and the yielding of the prestressing strands.

4- Identify the optimum values for bar size, debonded length and prestressing level using prototype models to provide design guidelines for practice.
1.4 Organization of this Work

This dissertation consists of nine chapters. The current chapter presents a background of the project, the literature review of the previous work done on subassembly tests, and the scope and objectives of the study.

Chapter 2 describes development of the scaled specimen including column design, bent cap design, footing design, and checking the capacities of the reused components such as the superstructure beams and spacer blocks, instrumentation plans and the applied earthquake records in the experiment.

The experimental setup of the scaled bridge is presented in Chapter 3. This includes a description of the construction of different bridge components, bridge instrumentation, sequence of erection in the laboratory, and material testing results.

Different results from the experiment are presented in Chapter 4, including the global behavior of the bridge and the local behavior of different elements.

Chapter 5 shows the estimation of reinforcing steel fracture during the test using data from column rotations using two approaches: 1) A certain strain threshold and 2) Low cycle fatigue model. This chapter continues the estimation of reinforcing steel fracture from acoustic emissions from the audio records during each test, and the verification of those methods by specimen demolition and inspection.

Chapter 6 presents a full comparison between the new system and similar conventional bridge tested in 2005 including peak displacements, residual displacements, base shear, rotations, strains and dynamic properties using the shared motions between the two experiments.
Chapter 7 describes the assumptions of the numerical model using OpenSEES for the new system. This chapter includes full comparison for most of the motions that used during the test in two stages: 1) Prior to reinforcement fracture and 2) After reinforcement fracture. The comparisons were divided into two sections: 1) Response history comparison for drift ratios, bridge center-of-mass (COM) displacement and base shear, and 2) Accumulative comparison for base shear envelopes, average reinforcing bar strain envelopes, average strand force and strain envelopes, rotation to drift contribution envelopes, and accumulative fatigue damage indices.

Chapter 8 shows the parametric studies that were conducted on different parameters including longitudinal bar sizes, debonded length for the scaled specimen in addition to prestressing level in prototype bent. This chapter provides recommendations for this parameters to delay the fracture of the longitudinal bars and the yielding of prestressing strands during the 150% Design Level Earthquake.

Summary and conclusions are presented in Chapter 9, along with proposed future research.
Chapter 2. Specimen Development

2.1 Introduction

This chapter describes the design of a new system with conventional cast-in-place construction. The experiments for both specimens were conducted at the Network for Earthquake Engineering Simulation (NEES) equipment site at the University of Nevada, Reno. To facilitate the comparison, the two bridges had similar geometries, superimposed masses and member strengths. The bridge constructed using traditional cast-in-place methods, hereafter referred to as the Conventional Bridge, was tested in 2005 (Johnson 2006; Ranf 2007; Johnson et al. 2008). The bridge constructed using the precast, pretensioned rocking system, hereafter referred to as the Resilient Bridge, was tested in 2014 (Thonstad et al. 2016; Mantawy et al. 2016). Both bridges were quarter scale specimens.

The design of the specimen components are discussed in Section 2.3, including column design, bent cap design, footing design, and checking the capacities of the reused components such as the superstructure beams and spacer blocks. The instrumentation plan is discussed in Section 2.4; the purpose of designing the instrumentation configuration is to track the local and global responses of the bridge and its components. Section 2.5 discusses the ground motion selection; the ground motion scheme includes three different ground motions to examine the new system response during both the far and near fault ground motions.
2.2 Conventional Bridge Specimen

The Conventional Bridge was a quarter scale specimen determined from a prototype bridge. The prototype bridge for the shaking table specimen was a two-span section of a continuous reinforced-concrete, box-girder bridge. The superstructure was assumed to be supported by three two-column bents on drilled shafts. The length of each span was approximately 120 ft. Illustrations of the elevation and cross-section of the prototype bridge are shown in Figure 2-1 and Figure 2-2.

The Conventional Bridge had three bents and two spans of 30 ft. each, with overhangs at the outer bents to support a portion of the superimposed dead load. Each bent contained two 12-in. diameter columns with a center-to-center spacing of 75 in. The clear column heights (from the tops of the footings to the bottoms of the bent caps) were 6 ft., 8 ft. and 5 ft. for Bents 1, 2 and 3, respectively. The exterior bents, which had the shorter columns, were placed on spacer blocks in order to maintain a level superstructure.

Both spans of the superstructure consisted of three precast slabs, which were placed between the bents, post-tensioned transversely with Dywidag bars, and then post-tensioned longitudinally to the bent caps with strand tendons to provide a continuous superstructure.

The target axial load ratio for the columns in both specimens was 8%. In addition to the self-weight of the structural components, superimposed masses were placed on the superstructure to match the scaled prototype bridge mass. Although the distribution of these masses varied slightly between the two bridges, the total weight of the superstructure above mid-height of each column was the same, 268 kips.

Figure 2-3 shows typical details for the cast-in-place bents of the Conventional Bridge. The column longitudinal reinforcement consisted of 16#3, leading to a
reinforcement ratio of 1.56%. Those bars were anchored in the cast-in-place bent cap by embedment of the projecting column bars. Because the scaled depth of the bent cap, the beam depth was inadequate for properly anchoring the longitudinal reinforcement, steel bars were welded across the ends of adjacent longitudinal reinforcement to provide the needed anchorage. Transverse confinement was provided by spirals made from W2.9 diameter wire spaced at 1.25 in., leading to a volumetric transverse reinforcement ratio of 0.9%.

2.3 Resilient Bridge Specimen

The Resilient Bridge components consisted of six precast columns, three precast bent caps, three cast-in-place footings, two reused spacer blocks and six reused superstructure beams. Appendix A shows the overall dimensions and reinforcement drawing for each component from Figure A-1 to Figure A-15.

The Resilient Bridge was designed to match as closely as possible the overall specimen dimension, column strength, supplementary mass and loading protocol of the Conventional Bridge, to facilitate a comparison between the conventional design and the proposed system. There were some differences in the Resilient Bridge because the new system required a deeper footing to provide sufficient length for the socket connection and a deeper bent cap to develop sufficient length of the hybrid-bar-socket connection. The differences between the final specimens between the Conventional Bridge and the Resilient Bridge are described in the following subsections.

Table 2-1 shows a comparison in the overall dimensions and reinforcement between the prototype bridge, the Conventional Bridge and the Resilient Bridge.
2.3.1 Column Design

In the Resilient Bridge, the reinforcement in the columns was selected so that 60% of the column’s flexural strength was provided by the prestressing strand, and 40% was provided by the mild reinforcement. Since the reinforcing ratio used in the Conventional Column was $\rho = 1.56\%$, target reinforcing ratios of $\rho_p = 0.6(f_{sy}/f_{py})*1.56\% = 0.23\%$ and $\rho_s = 0.62\%$ were chosen for the strand and the mild reinforcement respectively, where $f_{sy}$ is the yield stress of the mild steel reinforcing, and $f_{py}$ is the yield stress of the strand. The prestressing strands were tensioned to 180 ksi before concrete casting.

The column longitudinal reinforcement consisted of six #3 (Gr 60) and four 3/8-in. diameter epoxy coated prestressing strands (Gr 270), leading to an equivalent reinforcement ratio (accounting for the higher strength of the prestressing strands) of 1.7%. At each rocking interface, the longitudinal mild-steel reinforcement was debonded to delay bar fracture. The debonded length was 3 in. within the clear height of the column and 6 in. within the footing or the bent cap, for a total debonded length of 9 in. The depth of the bent caps was increased to 19 in. in the Resilient Bridge (instead of 15 in. in the Conventional Bridge), in order to provide adequate development length for the column’s reinforcement. The precast column bases were connected to the combined cast-in-place footings using “wet socket” connection (Haraldsson et al. 2013). The column embedment depth, 22 in., was the same for the three bents. The tops of the columns were connected to the precast bent caps using a “hybrid-bar-socket” connection (Davis et al 2012).

In order to delay the yielding of the strands, the strands were unbonded over the greatest length possible, and they were bonded only within the depth of footings and bent caps. Since strand sizes were not commercially available to meet the scaling requirements,
the bonded lengths did not meet the AASHTO guidelines for development within the footing and bent cap; therefore strand vices were placed at the ends of the strand to provide additional anchorage. Load cells were placed between the column and the strand vices at the top of each column to detect whether the strand vices were needed to prevent slip.

At the column-to-footing and the column-to-bent cap connections, the concrete was protected by a confined rocking detail (Schaefer et al. 2013; Kennedy 2015). The concrete was confined by a 12-in. diameter, 0.134-in. thick circular steel tube that was 6-in. height. A 0.25-in. thick annular end plate was welded to the bottom of the tube to promote rigid body rotation of the columns (Thonstad et al. 2016; Mantawy et al. 2016). In order to arrest any cracks that could form at the boundary of the steel confining tube and to distribute the high compressive forces due to the rocking behavior of the system, six #4 reinforcement were welded to the end plate, extending into the clear height of the column for a length of 24 in.

Figure 2-4 shows the design moment-rotation and force-drift for each rocking section for each column. Figures 2-5 and A-5 show the typical details for the columns of the Resilient Bridge.

2.3.2 Bent Cap Design

The bent caps were designed as standalone members. However because the bent caps were post-tensioned to the superstructure, which was a solid section, and because the bent caps had additional components such as cantilevers and ledges, the bent caps had both additional strength and stiffness beyond what they would have in the prototype. This was not considered to be an issue for modeling since the prototype was representative of
contemporary design, in which case, the bent caps are designed to remain essentially elastic. The primary goal of the shake table model was to capture the nonlinear bridge response that occurs in the columns. In continuous reinforced concrete bridges, nonlinearity occurs essentially in the substructure and the superstructure remains elastic (Johnson 2006).

Bent Caps 1 and 3 were 19-in. depth, 98-in. length and 72-in. width. Bent Cap 2 was inverted T beam with 19-in. depth, 98-in. length, 21-in. top width and 35-in. bottom width. The dimensions of the bent caps are shown in Figures A-2 and A-3.

Figure A-6 through Figure A-10 show the reinforcement for the identical bent caps (Bent Caps 1 and 3) and the symmetric bent cap (Bent Cap 2). The bent caps were designed as protected elements (i.e. to remain elastic). The reinforcement was calculated to satisfy the flexural and shear requirement for the beam and ledge part of Bent Cap 2 in addition to the cantilever portion for Bent Caps 1 and 3. Six-inch corrugated aluminum pipes with 19-in. depth were 75 in. apart to accept the column reduced section. Six aluminum pipes with 19-in. depth were centered around the 6-in. corrugated pipes to accept the #3 reinforcement at the column tops. W2.9 spiral with 11.25-in. diameter was used to confine the 1.25-in. diameter pipes.

Longitudinal post-tensioned ducts were installed with 30 in. center-to-center distance to match the distance between the ducts inside the reused superstructure beams.

It was impossible to place the mass over the outer bent caps over the columns due to the usage of the stand load cells and strand vices. Twenty-four kips of lead plates were replaced by one concrete block with a weight of 20 kips. The difference in weight was balanced by making the width of the outer bent caps longer by 1 foot and the depth of the
bent cap deeper by 4 in. from what used in the Conventional Bridge. As mentioned in Section 2.3.1, the depth of the bent caps was deeper in the Resilient Bridge than in the Conventional Bridge to provide an adequate length to anchor the column bars inside the hybrid-bar-socket connection.

2.3.3 Footing Design

The specimen footings were designed to be strong and stiff to avoid any footing damage. The footings needed to resist the column moment and shear, as well as the clamping forces between the footings and the shake tables. Reinforcement drawings of the footings are shown in Figure A-11 for Bents 1 and 2, and Figure A-12 for Bent 3. The footings for Bents 1 and 2 are identical because the 2-ft. spacer blocks provide Bent 1 with the additional height needed to match the height of Bent 2, which had columns that were 2 ft. taller. The footing for Bent 3, the shortest of the three bents, was approximately 1 ft. taller than the other footings. This is because Bent 3 was 1 ft. shorter than Bent 1. The embedded socket depth of each column was 22.75 in. with additional 7.25 in. of attached strand vices outside the column. The depth of the footing of Bents 1 and 2 were 30 in. (deeper by 1 ft. than the Conventional Bridge). The footing depth of Bent 3 was 42 in. (deeper by 1 ft. from what used in the Conventional Bridge). The additional 1 ft. in the footing was due to the 22.75-in. column socket depth in addition to the extended strand vices.

Space blocks were constructed for the Conventional Bridge and reused in the Resilient Bridge. They were used underneath the footings of Bents 1 and 3 to maintain a level superstructure. Figure A-13 shows the reinforcement of the spacer blocks.
2.3.4 Superstructure Beam Design

The 2-span superstructure consisted of six beams (three beams each span) and was designed by Johnson 2006. The design represents the prototype box girder superstructure scaled to a solid superstructure. Because of the high relative strength and stiffness of the prototype superstructure compared to the substructure, the superstructure remains essentially elastic when the bridge is subjected to seismic excitation. Also, because the bridge is post-tensioned, the superstructure remains generally uncracked. For these reasons, it was not necessary to model the superstructure in detail, which would have been very difficult considering since the cells would have a scaled web width of only 3 in. Instead, the model superstructure was a solid rectangular section with bending properties equivalent to those of the prototype. The quarter-scale moment of inertias for bending about both orthogonal axes of the superstructure were calculated to match the prototype section by multiplying the prototype moments of inertias by $1/256$, which is the fraction of the length scale to the fourth power. The resulting section for the two spans, essentially a slab, was a rectangular section having a width of 90 in., a depth of 14 in. and a length of 324 in. This enabled the two spans to be easily transported, stored, and assembled on the shake tables. Each span was constructed with three beams that each had a cross section of 30 in. that were transversely post-tensioned to form each span (Johnson 2006).

Figures A-14 and A-15 show the reinforcement of the superstructure beams including the ledge reinforcement. As the superstructure beams were used several times before this test, it was necessary to repair the crushed areas over the ledges and over the sides caused by previous transverse and longitudinal post-tensioning.
Due to the longitudinal post-tensioning, it was necessary to check the stress over the superstructure to keep it uncracked. Section 2.3.6 shows the construction sequence of the superstructure and the values of the maximum tensile and compressive stress over the superstructure cross section during construction.

2.3.5 Superstructure Masses

In the Conventional Bridge, the column axial load level was designed by the axial load ratio, which is defined as the ratio of the axial load and the product of the gross section and the specified concrete compressive strength. The target axial load ratio for the bridge model was 8.2%, which represents a typical bridge column axial load ratio. An axial load ratio of 8.2% in the model is equivalent to a column axial load of 46.5 kips. Due to the scaling effect, masses of the quarter-scale model provide a smaller axial stress than in the prototype. Some of the required axial load was provided by the self-weight of the bridge model. The rest was provided by superimposed dead load that was attached to the top of the superstructure (Johnson 2006). In the Resilient Bridge, the target axial load ratio was chosen to match the Conventional Bridge. The total mass value was the same as the Conventional Bridge. In the Resilient Bridge the added weight consisted of two concrete blocks of 20 kips each over each outer bent cap, increasing the bent caps width by 1 ft. and the depth by 4 in. in comparison to the Conventional Bridge, and three concrete blocks of 20 kips in addition to steel plates of 5.1 kips over each span.

Figure 2-6 shows the distribution of the masses over the superstructure of the Resilient Bridge.
2.3.6 Construction Sequence Design

The purpose of post-tensioning sequence design is to minimize the moment on the column, especially the rocking at the interface to be under the decompression moment. The decompression moment is defined as the moment at the rocking interface needed to initiate the first gap opening (i.e. the moment needed to overcome the effect of pretensioning force and axial forces due to dead load, as shown in Figure 2-7). The decompression moment due to the pretensioning force was 86.7 in-kips. The decompression moment during the construction was calculated using the following equation

\[ M_{\text{dec}} = M_{\text{dec, pre}} + M_{\text{dec, axial}} \]

\[ = 86.7 + \frac{I}{\text{depth} \cdot A} \times R \]  

(Eq. 2-1)

\[ = 86.7 + 1.5 \times R \]

Where

\( M_{\text{dec}} \): Decompression moment (in-kips);

\( M_{\text{dec, pre}} \): Decompression moment due to pretensioning (86.7 in-kips);

\( M_{\text{dec, axial}} \): Decompression moment due to axial load (in-kips);

\( R \): Axial load during each case of construction sequence (kips);

\( I \): Moment of inertia of the rocking section (1017.9 in\(^4\));

\( A \): Cross section area (113.09 in\(^2\));

depth: the rocking interface radius (6 in).
2.3.6.1 Construction Sequence

The construction sequence was designed by investigating different loading and post-tensioning stages. The final proposed sequence was to place one concrete block over each of the end bents, then post-tensioning all the tendons to 66.67% of the jacking force (480 kips out of 720 kips), then placing the remaining three concrete blocks from the outer bent caps toward the middle of the spans (six total for the bridge) and finally post-tensioning the tendons to 100% of the jacking force. The grouting of the reduced section and column bars inside the outer bent caps was postponed until after the post-tensioning. The purpose of postponing the grouting was to minimize the moment in the columns caused by the construction sequence.

Linear elastic models using SAP 2000 software were created to track the maximum bending moment at the column interfaces to keep the moment under the decompression moment at the interface and the stress in the superstructure to avoid cracking the superstructure during the construction sequence. Figure 2-8 shows the SAP model of the bridge. The construction sequence was as follows: 1) placement of the concrete blocks over the outer bent caps with timber shoring to support them. Superstructure beams were simply supported between the bent caps, as shown in Figure 2-9 for case 1, 2) post-tensioning the superstructure to 66.67% of the jacking force, as shown in Figure 2-10 for case 2, 3) placing the first concrete block on each span near the outer bent caps, as shown in Figure 2-11 for case 3, 4) placing the second concrete block on each span near the outer bent caps, as shown in Figure 2-12 for case 4, 5) placing the last concrete block on each span, as shown in Figure 2-13 for case 5, 6) post-tensioning the superstructure to 100% of the jacking force, as shown in Figure 2-14 for case 6, and 7) grouting the hybrid-bar-socket connection of the
outer bent caps, removing the timber shoring and placing the steel plates on the middle of each span, as shown in Figure 2-15 for case 7.

2.3.6.2 Construction Sequence Analytical Results

Table 2-2 shows that the maximum moment occurred at the rocking interface at the bottom and top connection of Bents 1 and 3. The factor of safety was calculated by dividing the decompression moment calculated from Eq. 2-1 by the maximum moment that was determined at column rocking connections.

For case 1, the moment was very small at the bottom connections because the outer concrete blocks were supported over the columns and the shoring as shown Figure 2-9.

For case 2, the factor of safety after post-tensioning the superstructure to 66.67% of the jacking force, was 1.1, which means that the decompression moment was 10% higher than the maximum moment that occurred at the column interfaces.

For case 3 and 4, the factor of safety increased to 1.43 and 1.6 respectively for Bent 1 and to 1.4 and 1.7 respectively for Bent 3. It is not surprising that adding the concrete blocks increase the axial load on the column, therefore increased the decompression moment.

For case 5, by adding the last concrete blocks, the roller supports, representing the shoring, showed tension reactions which meant that the bent caps lifted from the shoring. The model was run after removing the shoring, as shown in Figure 2-13. The factor of safety increased to 1.9 and 1.85 for Bents 1 and 3 respectively.

For case 6, by post-tensioning the superstructure to 100% of the jacking force, the roller supports, representing the shoring, showed compression reactions which meant that
the bent caps were supported over the shoring, as shown in Figure 2-15. The factor of safety dropped to 1.2 for Bents 1 and 3.

For case 7, after removing the shoring, placing the steel plates and grouting the hybrid-bar-socket connection of Bents 1 and 3, the model was modified by changing the pins over the column to be rigidly connected to the bent caps. No significant change in factor of safety occurred in comparison with case 6.

The straining actions were tracked for the superstructure, the maximum compressive stress for all cases was 1.18 ksi, whereas the allowable compressive stress was 2.25 ksi (factor of safety of 1.9). The maximum tensile stress for all cases for the bent caps and superstructure beams was 0.304 and 0.18 ksi respectively, whereas the allowable was 0.53 ksi (factor of safety of 1.74 and 2.96 for the bent caps and superstructure respectively). Figure 2-16 shows the rendering for the bridge after post-tensioning.

2.4 Instrumentation Plan

The bridge was instrumented with 395 channels to record accelerations, displacements, bar/strand strains, and the change in the strand forces. A summary of the instrumentation plan is shown in Table 2-3. Transverse, longitudinal, and vertical accelerations of the superstructure at each bent and mid spans were measured using accelerometers. Superstructure displacements were measured using potentiometers and column rotations were measured using displacement transducers. The strains in the longitudinal reinforcement, transverse reinforcement, longitudinal strands and confining tube within critical column sections were measured using strain gauges. Potential slippage of strands at top of the columns was measured using load cells.
2.4.1 Global Coordinate System

The global coordinate system is essential for large scale experimentation to define the polarity of each instrument. The south direction of the laboratory was taken as the positive direction of y-axis (transverse direction). The West direction of the laboratory was taken as the positive direction of x-axis (longitudinal direction). The upward direction was taken as the positive direction of z-axis (vertical direction). Figure A-16 shows the global coordinate system used in the Resilient Bridge experiment.

2.4.2 Superstructure Accelerometers

Five tri-axial accelerometers were installed in the superstructure to measure the transverse, longitudinal and vertical acceleration of the superstructure and bent caps. Each bent cap was instrumented with a tri-axial accelerometer at the middle span (between the columns). The superstructure was instrumented with tri-axial accelerometers at the middle of each span (between each bent). The acceleration records were used to calculate the base shear using the tributary mass assumption, as mention in Section 4.3.5 and Section 6.5.1.2 and to calculate the bridge fundamental period using a transfer function, as mentioned in Section 4.3.6 and Section 6.5.1.3. Figure A-17 shows the accelerometers distribution over the superstructure.

2.4.3 Superstructure Displacements Potentiometers

Five potentiometers were installed to measure the transverse displacement of the superstructure. Each bent cap was instrumented at the north side of the superstructure with a potentiometer. The in-span displacement was measure at the third of the span measured from the outer bent caps. Four potentiometers were installed at the outer bent caps (two
potentiometers for each bent cap) to measure the longitudinal displacement of the superstructure. Sixteen potentiometers were installed to measure the vertical displacement of the superstructure (four for each bent cap and two at each mid-span). Figure A-18 shows the displacement potentiometers distribution over the superstructure.

2.4.4 Strand Load Cell

Twenty-four load cells were placed between the column and the strand vices at the top of each column to detect whether the strand vices were needed to prevent slip. Figure A-19 shows the distribution of the load cells.

2.4.5 Column Displacement Transducers

Seventy-two displacement transducers were installed to measure the rotation of the column ends. Each column end had six displacement transducers, four of them installed in the transverse direction at approximately 3 in. and 9 in. from the column interface, and two of them installed in the longitudinal direction at approximately 3 in. from the column interface. Figure A-20 through Figure A-22 show the distribution of the displacement transducers. The displacement transducers were used in Chapter 5 to estimate the strain in the debonded reinforcing bars after the strain gauges were damaged.

2.4.6 Strain Gauges

Strain gauges were installed to monitor the strains in different reinforcing components: longitudinal reinforcement, transverse spirals, longitudinal prestressing strands and confining tube “steel tube”.
2.4.6.1 Longitudinal Reinforcement

For the longitudinal reinforcement in Bent 1, the north and south bars were instrumented for the north column at three levels: at 10 in. inside the anchorage component (bent cap and footing), at the interface and 7 in. inside the column, as shown in Figure A-23. The north and south bars were instrumented for the south column at three levels: at 9 in. inside the anchorage component (bent cap and footing), at the interface and at 6 in. inside the column, as shown in Figure A-23.

For the longitudinal reinforcement in Bent 2, the north bars were instrumented for the north column at three levels: at 8 in. inside the anchorage component (bent cap and footing), at the interface and 7 in. inside the column, as shown in Figure A-24. The north and south bars were instrumented for the south column at three levels: at 9 in. inside the anchorage component (bent cap and footing), at the interface and at 6 in. inside the column, as shown in Figure A-24.

For the longitudinal reinforcement in Bent 3, the north and south bars were instrumented at three levels: at 10 in. inside the anchorage component (bent cap and footing), at the interface and 7 in. inside the column for the north column, as shown in Figure A-25.

The north-west bars and the east-south bars of Bents 1 and 3 were instrumented at the top and bottom interfaces, as shown in Figures A-23 and A-25.
2.4.6.2 Longitudinal Reinforcing Strands

All the 24 prestressing strands were instrumented at the middle height of the columns using a pair of strain gauges to monitor the strains, as shown in Figure A-23 through Figure A-25.

The strain gauges were installed at the University of Washington and were attached to the data acquisition system to monitor the strains during prestressing. The strain gauges were unattached to the data acquisition to allow the columns to be shipped to the University of Nevada, Reno. The strain offsets were added to the strains recorded during the experiment to obtain the final strains.

2.4.6.3 Transverse Reinforcing

The spirals were instrumented at 12 in. from the interfaces to monitor the strain outside the confining tube, as shown in Figure A-23 through Figure A-25.

The north side of the bottom confining tubes of Bent 3 were instrumented by strain gauge rosette to measure the strains in the confining tube during the experiment.

2.5 Ground Motion Selection

A majority of the selected motions were the same as in the Conventional Bridge experiment (Johnson 2006) in order to facilitate a comparison between the new system and the conventional system. Preliminary OpenSEES models were used to evaluate the effects of far field ground motions. The intent was to add these motions to the loading protocol without altering the system performance during later motions that were comparable to the previous experiment. The final motion protocol eliminated some low-amplitude motions from the previous experiment and added Sylmar and Takatori motions at high-amplitude motions.
The excitations were based on the 90 deg. and 360 deg. components of the Century City Country Club North (CCN90/CCN360) record from the 1994 Northridge California Earthquake, the 360 deg. component of the Sylmar- Olive View Med. Center (SYL360) record from the 1994 Northridge California Earthquake and the 0 deg. component of the Takatori (TAK000) record from the 1995 Kobe, Japan Earthquake. Low-amplitude motions consisted of coherent, incoherent (different motion intensity for each shake table) and biaxial coherent motions, whereas high-amplitude motions consisted of only coherent motions in the transverse direction of the bridge due to the absence of abutments. White-noise and square wave excitations were distributed throughout testing to track the bridge dynamic properties including the bridge periods and damping. Sinusoidal waves were added to evaluate the dynamic response of the bridge subjected to harmonic motions. Because of the one-quarter geometric scale, the time coordinate of the input was multiplied by square root of the 0.25 bridge scale factor, 0.5.

Tables 2-4 and 2-5 list the complete test protocol. The bridge was monitored after each motion to decide the magnitude of the square waves, white noise in addition to deciding which motions would be used in high level motions to keep the bridge comparable to the Conventional Bridge tested at the University of Nevada, Reno in 2005.

To investigate the bridge behavior with different excitations including near fault motions, Sylmar and Takatori motions were added after Motion 14A. The acceleration histories were scaled to have similar structural demands to the Century City motion (Motions 14B1, 14B2 and 14C).
The Design Level Motion was defined as the first motion that caused a drift ratio of 3% on the Conventional Bridge (Motion 16, PGA of 0.75 g of CCN record, 100% Design Level Motion).

Figure 2-17 shows the acceleration histories for the scaled Century City motion (Motion 14A), Sylmar motion (Motion 14B2), and Takatori motion (Motion 14C). Figure 2-18 shows the spectral acceleration comparison for the three motions. Figure 2-19 shows the spectral displacement comparison for the three motions. Both spectral acceleration and displacement were derived using 5% damping ratio. The figures show that the Sylmar and Takatori motions produce higher displacement demands than the Century City motion at longer periods.
Chapter 3. Specimen Construction

3.1 Introduction

This chapter describes the construction and assembly of the bridge specimen (Section 3.2) including a discussion for the construction stages for each component, the schedule of time spent on the shake tables, and details of the assembly stages.

3.2 Construction and Schedule

This section briefly describes the construction and assembly of the bridge. The bridge construction details are provided in Section 3.3. The bridge (see Figure 2-16) was constructed from 20 structural components, which consisted of onsite construction, offsite construction, and reused components at the Earthquake Engineering Laboratory, University of Nevada, Reno. Six columns and three bent caps were cast at the University of Washington and shipped to the University of Nevada, Reno. The columns were aligned in the footing formwork and the spread footings were cast in-place in an outdoor fabrication yard next to the Earthquake Engineering Laboratory. The three footing and column bents were then moved onto the shake tables. Pre-constructed spacer blocks were used between the bottom of the footings and the shake tables to maintain a level superstructure due to the variations in column heights. Two different types of grout were used to connect the bent caps to the columns. A fiber reinforced grout pad was used at the column-to-bent cap interface to allow for the adjustment of the bent cap elevation and level. A standard, non-fiber reinforced, grout was used to connect the column’s reduced section and longitudinal reinforcement to the bent cap. The placement of the non-fiber grout was postponed until
after the longitudinal post-tensioning was completed to reduce the secondary moments in the columns due to slab shortening as discussed in Section 2.3.6.

After the bent and spacer blocks were aligned, grouted and vertically bolted to the shake tables, six precast slabs post-tensioned together to provide a continuous superstructure. The superstructure slabs had been utilized for the Conventional Bridge and were reused in this bridge (each span, consisting of three slabs). The three slabs were assembled on the lab floor and post-tensioned transversely. The preassembled spans were placed onto the bent cap ledge, and were supported on jacks over the safety frames between each bent cap. Hydrostone was placed between the bent caps and the superstructure. The superstructure longitudinal post-tensioning was conducted in stages to allow the placement of the superimposed masses. The procedure was designed to minimize the secondary moments on the columns due to post-tensioning (see Section 2.3.6). Eight concrete blocks, with a total weight 160 kips, and 10.2 kips of steel plates were placed on the superstructure to provide a representative structural mass, scaled from the superstructure of the prototype bridge.

The schedule of the events on the shake tables is listed in Tables 3-1 and 3-2. The duration of the shake table usage, beginning with table preparation and ending with the completion of the disassembling of the bridge and cleaning off the tables, was 39 days from May 26, 2014 to July 21, 2014. The schedule includes table preparation, assembly of the bridge on the tables, attaching instrumentation, both low and high amplitude testing, disassembly of the bridge, and cleaning the shake tables. Time on the tables is an important aspect of shake table testing, especially when testing on multiple shake tables.
3.3 Construction of the Bridge

The bridge model was composed of 17 major precast components and 3 major cast-in-place components, six superstructure beams and two spacer blocks, six precast columns and three precast bent caps that were cast at the University of Washington, and three cast-in-place footings. The columns and bent caps were cast at the Structural Laboratory at the University of Washington between December 2013 and February 2014 with all interior instrumentation. Strain gauges for longitudinal, spiral and strands were attached during the construction, and load cells were attached at the top of each column before releasing the pretension strand force. Footing formwork and reinforcement were fabricated by the middle of March, and the column and bent caps from the University of Washington were delivered to the University of Nevada, Reno on March 20, 2014. The columns were installed inside the footing reinforcement from March 21, 2014 to March 27, 2014. A crane was used on April 2, 2014 to place the bent cap on timber shoring on top of each bent before casting the footing to verify the constructability of the bent cap when moved onto the shake tables. Casting of three footings around the column was on April 4, 2014. By April 26, 2014, all bent caps, sets of column-footing, spacer blocks and superstructure beams were cast or repaired and ready for the laboratory stage.

3.3.1 Precast Column Construction

Total number of six columns were constructed at the Structural Laboratory at University of Washington. The columns were different heights. The two 5-foot columns (Bent 3 columns) were constructed first in November 2013, then the two 6-foot columns (Bent 1 columns) were constructed during December 2013 and January 2014, and finally the two
8-foot columns (Bent 2 columns) were constructed during February 2014. Details of the reinforcement are provided in Section 2.3.1. Figure 3-1 shows the steel cage for one of Bent 2 columns (the figure shows the column cage at the socket connection including the confining tube in blue cover and the debonded bars inside the footing in red duct tape).

The prestressing strands were tensioned to 180 ksi before casting, as shown in Figure 3-2. High slump concrete was poured, as shown in Figure 3-3. Figure 3-4 shows the final construction of Bent 3 columns.

3.3.2 Precast Bent Cap Construction

The precast bent caps were constructed at the University of Washington. The symmetric bent cap (Bent 2 bent cap) was constructed during November 2013, while the outer bents were constructed during December 2013 and February 2014.

As discussed in Chapter 2, the bent caps were designed to be protected components, which means that they were to stay elastic during the experiment. Figure 3-5 shows the alignment of the ducts of the bent cap of Bent 2. Figure 3-6 shows the concrete casting of Bent Cap 2.

3.3.3 Footing Construction

Reinforcement for three spread footings were assembled in January 2014. Two of the footings were 30 in. depth to accommodate the 22.75 in. embedded socket connection, while the third footing was 42 in. depth with 22.75 in. embedded socket connection. The third footing was deeper due to the shorter column. Timber octagon shaped pockets were fabricated below where the column would be placed in order to protect the pretension
hardware during concrete casting. The pockets were 7.25 in. depth for Bent 1 and Bent 2 and 19.25 in. depth for Bent 3. Figure 3-7 shows the octagon shaped pockets lined up at one footing before finishing the reinforcement, as shown in Figure 3-8.

The columns arrived on March 20, 2014. The columns were lifted using a forklift and placed at the appropriate position and orientation, as shown in Figure 3-9 to Figure 3-12. Wood shoring was fabricated to brace each column, as shown at Figures 3-13 and 3-14. It was critical that the columns were oriented correctly and spaced properly. Figure 3-15 to Figure 3-17 show the crane placing the bent caps to verify the constructability.

After placing the bent cap at the top of each bent, footing concrete was cast, as shown in Figures 3-18 and 3-19.

### 3.3.4 Bent Assembly

Due to bent stability and a requirement to not exceed the decompression moment of the column connections, bent cap grouting was postponed until after placing the footing and columns for Bents 1 and 3 on the shake table and on the laboratory ground for the symmetric bent (Bent 2). The shake tables were prepared, prior to the placement of the spacer blocks and Bent 2, by inserting threaded rods into the table top, that would be used to post-tension the footings and safety frames to the tables. Square 6x6x2 in. pieces of high density foam with a 1.5 in. hole in each center were used to seal the ducts beneath the footings to keep grout out of the ducts and the anchor holes in the shake tables. The foam pieces were 0.5 in. thicker than the target thickness of the grout beneath the footings and blocks so that the weight of the footings and blocks would compress the foam and seal the duct region. The 1.5 in spacing beneath the spacer blocks was provided using long pieces
of standard 2x4 in lumber. A 3.0-in. gap for the grout was provided beneath the center bent. The extra 1.5 in. of grout was provided for the center bent (Bent 2) so that the top of the three bents would be at the same elevation. Since an additional 1.5 in. layer of grout was required on the end bents between the spacer blocks and footings. Figure 3-20 to Figure 3-22 show the table preparation, and spacer block and bent placement on the shake tables.

3.3.4.1 Bent 1 Assembly

After placing the spacer block on Shake Table 1, new foam pieces and spacers were glued on top of the spacer block, and the pre-constructed footing and column were placed on the spacers. Timber shoring was fabricated for safety during the grouting process of the bent cap, as shown in Figure 3-23. The bent cap was lifted by the laboratory crane and aligned on the top of the column, as shown in Figure 3-24. Then it was lifted 6 in. from the top of the columns, and a 4x6 in. lumber placed between the bent cap and the shoring, as shown in Figure 3-25. A fiber grout was mixed and placed at the top of each column. Shims were used to create a gap of about ¼ in. Duct tape was used around the top of the steel confining tube to provide confinement for the grout during the lowering process of the bent cap, as shown in Figure 3-26. The crane lifted the bent cap again and the lumber pieces were removed to allow the bent cap to be placed on the fiber grout, as shown in Figure 3-27. The hybrid-bar-socket connection grout was postponed until after the post-tensioning was completed. Wet towels were used around the fiber grout and water sprayed at beginning and end of each working day for three days to provide moisture condition during the grout curing.
3.3.4.2 Bent 2 Assembly

As Bent 2 was the intermediate and symmetric bent, the grouting process was done off table because the bent was inherently stable during the lifting process. Both fiber and non-fiber grout were placed before the longitudinal post-tensioning of the superstructure.

Timber shoring was placed to increase safety during the grout placement, as shown in Figure 3-28. The bent cap was lifted by the laboratory crane and aligned at the top of the columns, as shown in Figure 3-29. Then it was lifted 6 in. from the top of the columns, and a 4x6 in. lumber placed between the bent cap and the shoring. A fiber grout was mixed and placed at the top of each column with shims of approximately ¼ in. Duct tape used around the top of the steel confining tube to provide confinement for the grout during the lowering process of the bent cap, as shown in Figure 3-30. The crane lifted the bent cap again and the lumber pieces removed to allow the bent cap to be place over the fiber grout, as shown in Figure 3-31. The hybrid-bar-socket connection grout was placed four hours after the fiber grout placement. A non-fiber grout was mixed for five minutes then placed using funnel and tube inside the reduced section and reinforcing bar ducts, as shown in Figure 3-32 to Figure 3-34. Wet towels were used around the fiber grout and over the non-fiber grout, and water was sprayed at beginning and end of each working day for three days to provide moisture condition during the grout curing.

3.3.4.3 Bent 3 Assembly

Placing and grouting Bent 3 was the same as Bent 1, as shown Section 3.3.4.1. Figures 3-35 and 3-36 show the different stages of placing and grouting of Bent 3.
3.3.5 Bridge Assembly

3.3.5.1 Superstructure Beam Assembly

The two spans of the superstructure consisted three beams that were transversely post-tensioned on the laboratory floor. The beams were supported by rigid steel beams resting on the laboratory floor. Each beam was oriented and aligned in longitudinal direction to match the transverse ducts, and in transverse and vertical directions to match the longitudinal post-tensioning ducts and the concrete block ducts (the supplemental masses). Square 5x5x1 in. pieces of low density foam, with a 1.5 in. hole in the center, were used to seal the ducts around the transverse post-tensioning rods to prevent the hydrostone from leaking inside the ducts, as shown in Figure 3-37. Hydrostone was poured between the beam longitudinal joints to fill them before the transverse post-tensioning, as shown in Figure 3-38. After the hydrostone cured, the transverse post-tensioning was applied to the beam assemblages. The transverse post-tensioning for each span consisted of five Dywidag rods tensioned to 100 kips each, as shown in Figure 3-39. The rods were post-tensioned in each beam starting with the mid-span and working outward. Figure 3-40 shows the end of transverse post-tensioning for both set of beams.

3.3.5.2 Placement of Superstructure Beams

To eliminate the torsion moment over Bent 2 during the placement of the first span, safety frames with jacks were used to hold the span. Four safety frames were used during experiment to support the bridge in case of unpredicted failure, and during construction as construction frames to hold the spans until the longitudinal post-tensioning process.
The west span (between Bent 2 and Bent 3) was lifted using two cranes, lowered down onto the jacks, then aligned in transverse and vertical directions to match the longitudinal ducts location at each bent cap. The east span (between Bent 1 and Bent 2) was put in place using the same procedure. Figure 3-41 to Figure 3-43 show the placement and the alignment process of each span. The joints between the bent caps and beams were filled with backer rods for the narrow joints and foam for the wide joints, then silicon was used to inject the gaps around the backer rods and foam. An inverted U-shape foam was used to protect the longitudinal duct during the joint filling process, as shown in Figure 3-44. Hydrostone was mixed and poured to fill the joints between the superstructure and the bent caps at four locations, as shown in Figure 3-45. The hydrostone was cured for three days.

3.3.5.3 Footing Attachment to the Shake Table

After the placement of each span, formwork was placed around the spacer blocks and footing to allow for the grouting of the spacer blocks to the shake tables. Non-shrinkage grout was mixed to fill the gaps around the spacer blocks and footing, as shown in Figure 3-46. The threaded rods were tensioned to attach the footing with spacer blocks to the shake tables.

3.3.5.4 Longitudinal Post-tensioning

The longitudinal post-tensioning process was done to convert the superstructure from a simply supported condition to a continuous condition. As mention in Section 2.3.6, the design of the post-tensioning was done to minimize the secondary moments at the column ends during the construction and to prevent tension at the base of the columns during
construction. The first step was placement of the concrete blocks over the outer bents, as shown in Figure 3-47, followed by releasing the jacks to allow the superstructure to be supported over the bent caps. Releasing the jacks at east and west sides of Bent 2 was done synchronously to gradually transfer the load to the bent cap, as shown in Figure 3-48. The next step, wedges and wedge plates were installed at the anchorage ends followed by the first stage of post tensioning. The first stage of post-tensioning started by fully jacking seven strands in the middle tendon at the same time. 270 kips of jacking force resulted in an effective post-tensioning force of 240 kips, as shown in Figure 3-49. The next step was jacking the south tendon to 50% of the target effective post-tensioning force, which equals 120 kips, then jacking the north tendon to the same amount. Three concrete blocks were then placed over each span in symmetric order from span end toward the central bent (Bent 2), as shown in Figure 3-50. The final stage of the post-tensioning started by jacking the south tendon to 100% of the target post-tensioning force, 240 kips, then the north tendon by the same amount, as shown in Figure 3-51. Finally, the wood shoring over Shake Tables 1 and 3 was removed after the post-tensioning process. Figure 3-52 shows the bridge after the post-tensioning process was completed.

3.3.5.5 Grouting Bent Cap to the Columns

The purpose of postponing the bent caps grouting for the outer bents until after the post-tensioning process was complete, was to create a pin joint at top of the outer bents during the post-tensioning to reduce the secondary moments on the columns during the post-tensioning process.

The non-fiber grout was used to grout both the reduced column section duct and reinforcing bar ducts at four positions at Bents 1 and 3. The grout was placed using a funnel,
as shown in Figures 3-53 and 3-54. Two sets of steel plates, with weight of 5.1 kips each, were placed at middle of each span after the grouting process, as shown in Figure 3-55.

### 3.3.6 Instrumentation

Instrumentation for the bridge was attached in three stages. The first stage was attaching the strain gauges to the column cages during the construction process at the University of Washington. The second stage was connecting these strain gauges and displacement transducers to the data acquisition system after the superstructure beams were placed and before the superstructure was post-tensioned. The final stage was the connecting of all other transducers and strain gauges after the longitudinal post-tensioning.

Ideally, the strain gauges in the columns would have been monitored throughout the construction process of the model, including transportation of the bent components, the placement of superstructure beams, post-tensioning and addition of masses. However, the gauges were not attached to the data acquisition until after the superstructure beams were placed. For both the longitudinal post-tensioning and the addition of masses, the strain gauges were monitored in real time to ensure that the strains in the columns were close to the predicted strains. After the placement of the supplemental mass, it was determined that the strains in the bar were consistent with strain that was predicted from analysis. The strain gauges were restored to zero prior to testing.
3.4 Material Testing

The material tests were important for two reasons. The first was to make sure that the material properties were in an acceptable range. The second was to determine the constitutive relationships so that the numerical modeling would be accurate.

Tables 3-3 and 3-4 show the concrete compressive strength and elastic modulus for each component.

Tables 3-5 and 3-6 show the compressive strength for the non-fiber grout used to fill each column ducts.

Tables 3-7 and 3-8 show the compressive strength for the fiber grout that placed over the top of the columns.

Table 3-9 shows the steel types, the elastic modulus, the yield strength, and the ultimate strength for the longitudinal reinforcement, the transverse spiral, and the longitudinal post-tensioning strands.
Chapter 4. Observed and Measured Response

4.1 Introduction

This chapter discusses the observed and measured response from the bridge shake table experiment. Observations and results from this experiment served two purposes. The first was to verify the new design approach. The second was to compare the new proposed bridge system to the conventional bridge previously tested at University of Nevada, Reno (Johnson 2006) as discussed in Chapter 6. The measured response of the Resilient Bridge and its components is discussed in this chapter. This includes the global response (e.g. relative displacements, base shear and modal periods), and the local response (e.g. strains, and rotations) as well as the visual damage progression and the observations made during the experiment.

4.2 Observed Results

Observations were taken during the experiment are discussed in two sections: 1) low amplitude testing and 2) high amplitude testing. The observations included visual inspection of the bridge between motions that were documented. During each motion, GoPro cameras were used at the top and bottom connections of each column (12 total), high definition cameras were set at each table mezzanine to capture the motion of each bent (3 total), and a GoPro camera was attached at the laboratory crane to capture the bridge movement in horizontal projection view.

Basic data analyses were checked after each motion to track the bridge response and understand the damage state of the bridge. The measured data is discussed later in this.
chapter in the measured response section. The motions used during testing are provided in Tables 2-4 and 2-5.

4.2.1 Low Amplitude Motions

During low amplitude testing (Motion 1A through Motion 9B as mentioned in Table 2-4), the columns were allowed to exceed their decompression moment, but not the moment that would initiate yielding of longitudinal reinforcement. The strain history data was reviewed after each motion to ensure that the longitudinal reinforcement in the column had not yielded during any low amplitude motions.

The superstructure and substructure of the bridge was inspected for cracking throughout the low amplitude motions, and none was observed. No slippage for the longitudinal strands inside the columns and slabs occurred during any low amplitude motions.

The superimposed masses attached at the top of the superstructure were evaluated after the low amplitude motions, to check for any slippage, none was detected. Both fiber and non-fiber grouts at column to bent cap connection were not cracked during the low amplitude motions.

4.2.2 High Amplitude Motions

Similar to the low amplitude motions, no damage was observed in the superstructure including the concrete in bent caps, superstructure beams, as well as the longitudinal post-tensioning for the superstructure. Nor was there any visible damage in the footings that were bolted to the shake tables or the masses that were attached to the superstructure.
Camera videos were important to investigate the rocking at top and bottom during the motion. Tables 4-1 to Table 4-3 show the damage progression at top and bottom connections of each column by visual inspection after each motion for all bents.

Figure 4-1 to Figure 4-4 show the maximum gap opening in the rocking interfaces at the top and bottom connections of each column in Bent 1 from the GoPro cameras.

Figure 4-5 to Figure 4-15 show the damage progression for bridge bents throughout the high amplitude motions by visual inspection. Figure 4-16 to Figure 4-21 show the final damage for bridge bents after removing the instruments.

### 4.2.2.1 Bent 1 Observations

Descriptions of the damage progression for Bent 1 are listed in Table 4-1. Photographs of the maximum gap opening in the rocking interface are shown in Figure 4-1 to Figure 4-4. Photographs of the damage progression are shown in Figure 4-5 to Figure 4-8. Photographs for final damage for the column ends at the top and bottom connections of the north and south columns are shown in Figures 4-16 and 4-17. During Motion 14A, first opening in the rocking interfaces occurred, and the gaps closed after the end of the motion at each rocking interface. During Motion 14C, the first significant cracks occurred at the octagonal corners with minor flaking at the bottom connections of both columns and the top connection of the south column. Increase in crack width and new cracks occurred at the octagonal corners at each column ends during Motion 15. During Motion 16, grout pad flaking occurred for both fiber grout pads at the top of the columns. During Motion 18, confining tube bulge occurred at the north side of the bottom confining tube of the north column and on both sides of the bottom confining tube of the south column. During Motion 19, the confining tube bulge increased, and grout pad crushed about 2 to 2.5 in. from the
top of both columns; in addition there were horizontal flexural cracks at 6 to 8 in. away from the end of the confining tubes at the bottom of both columns and at the top of the north column.

4.2.2.2 Bent 2 Observations

Descriptions of the damage progression for Bent 2 are listed in Table 4-2. Photographs of the damage progression are shown in Figure 4-9 to Figure 4-11. Photographs for final damage for the column ends at the top and bottom connections of the north and south columns are shown in Figures 4-18 and 4-19. During Motion 15, first opening in the rocking interfaces occurred, and the gaps closed after the end of the motion at each rocking interface. During Motion 16, the first significant cracks occurred at octagonal corners with minor flaking at the bottom connections and the top connection of the south column. Increase in crack width and new cracks occurred at octagonal corners at the column ends during Motion 17. During Motion 17, cracking in footing around the columns occurred. During Motion 18, cracks with spalling occurred at octagonal corners above the confining tube at the column ends and continued during Motion 19.

4.2.2.3 Bent 3 Observations

Descriptions of the damage progression for Bent 3 are listed in Table 4-3. Photographs of the damage progression are shown in Figure 4-12 to Figure 4-15. Photographs for final damage for the column ends at the top and bottom of the north and south columns are shown in Figures 4-20 and 4-21. During Motion 14A, first opening in the rocking interfaces occurred, and the gaps closed after the end of the motion at each rocking interface. During Motion 14C, first significant cracks at octagonal corners with minor flaking occurred at the
bottom connections and the top connection of the north column. Increase in crack width and new cracks occurred at octagonal corners at the column ends after Motion 15. During Motion 16, grout pad flaking occurred for both fiber grout pads at the top of the columns. During Motion 17 cracks at octagonal corners with spalling initiated at the bottom connections and the top of the south column, in addition to cracking in the footing around the north column. During Motion 18, confining tube bulge occurred at the north side of the bottom confining tube of both columns. During Motion 19, confining tube bulge increased, and grout pad crushed about 2 to 2.5 in. from the top of both columns, in addition to horizontal flexural cracks at 6 to 8 in. away from the end of the confining tubes at the bottom of both columns and at the top of the north column.

4.3 Measured Results

This section describes the local results from instruments including reinforcement strains, strand slippage, strand strains and column end rotation, and the global results including superstructure displacements, superstructure accelerations, base shear and dynamic properties throughout the test. The instruments were described in Section 2.4 and Figure A-17 to Figure A-25. Data was recorded at 256 Hz.

4.3.1 Reinforcement Strains

Reinforcement strains include longitudinal reinforcement, lateral spirals and strands in the columns. While strain gauges are accurate instruments, cracks and interaction of aggregates and ribs on the bars with the concrete can cause highly localized strains. Therefore, erratic measurements may be recorded during some motions.
4.3.1.1 Longitudinal Reinforcement

Average longitudinal strain ductility envelopes for each bent measured at the interfaces are shown in Figure 4-22. In the bridge columns, strains were largely concentrated to the connection interface; they were distributed along the debonded region, and dropped rapidly outside it. The longitudinal reinforcement yielding was first recorded during Motion 13. The slope of the strain ductility envelopes were nearly the same for each bent and equal to 333.33*ε_y (3.33*ε_y per 1% drift ratio).

Figure 4-23 shows the strain envelopes for the extreme tensile reinforcement at the bottom connection of the north column of Bent 3 as a sample for all reinforcement bars. The reinforcement was instrumented in three locations to roughly describe the distribution of strain in the bar: 1) at the interface, 2) above the interface and 3) below the interface as mentioned in Section 2.4.6.1. The strains at the column and in the anchorage region (in either the bent cap or footing) were below the yield strain due to the high confinement that both the confining tube and footing provides for the reinforcing bar outside the debonded region. However, the strains at the interface were high due to the concentration of the column rotation at the interface. The measured strain at the interface at 1.5% drift ratio was six times the measured strain at the bonded region inside the column and 12 times the measured strain at the bonded region inside the footing.

Figure 4-24 shows the distribution of the measured strain in the extreme tensile reinforcement at the top connection of the north column of Bent 1 as a sample for all reinforcement bars. The distribution of strain in the reinforcement was highly localized at the interface and demonstrated that the bar was able to achieve excellent bonding within the confined rocking detail.
The first longitudinal bar fractured during Motion 17 in Bent 1, multiple longitudinal bars fractured starting from Motion 18 up to the end of testing. Reinforcing steel fracture estimation is presented in Chapter 5.

4.3.1.2 Lateral Reinforcement

Strains were monitored in the spiral 6 in. above the confined rocking detail. No spiral yielding was recorded during any of the experimental motions, with 85% of the yield strain as maximum strain recorded.

Strains in the confining tube were monitored using strain rosettes located 1.5 in. above the column-footing interface on the north side of both columns of Bent 3. The confining tubes remained elastic until approximately 3.0% drift then yielded and bulged in Bents 1 and 3 during Motion 18 at drifts of approximately 9.2% and 6.0%, respectively.

4.3.1.3 Strands

Table 4-4 to Table 4-6 list the maximum strand slippage. Figure 4-25 shows the average strand strain envelopes for each column up to Motion 17. Figure 4-26 shows the average strand strain envelopes for each column with the prestressed strain offset to Motion 17. The first strand yield occurred at 3% drift ratio. The behavior of prestressing strands was the same for all columns except of the south column of Bent 2. The strands were bonded for a large distance inside the column as was noticed during the demolition. As described previously, the prestressing strands were to be unbonded with the clear height of the column. The extra bonded length inside the south column of Bent 2 constrained the rocking of that column and led to low strain measurement at the interface. Figure 4-27 shows the bonded strands of the south column of Bent 2 after the bent demolition.
4.3.2 Column Rotation

Rocking interface rotations at end of each column were obtained from the deformations measured by the displacement transducers (Novotecknik) attached to ends of the columns. The rotation at each level was calculated based on equation (Eq. 4-1).

\[ \theta_i = \frac{\Delta_1 - \Delta_2}{x_1 + D + x_2} \]  
(Eq. 4-1)

Where:

\( \theta \) = angle of rotation at each level;

\( \Delta \) = displacement of each Novotecknik;

\( x \) = distance between the Novotecknik and the column;

\( D \) = column diameter (12 in.).

Figures 4-28 and 4-29 show the average bent rotation in each bent during Motion 14A and Motion 18. The average rotations were between 65-70% of the measured drift ratio during Motion 14A. The low value of the rotation contribution to the drift was due to the relatively low amplitude motion (Motion 14A compared to Motion 18) which prevented significant rocking and caused flexural rotations in the columns. During Motion 18, the average rotation were between 90-95% of the measured drift ratio due to the significant rocking that occurred during Motion 18.

The average of the four individual connection rotations of each over the first 9 in. of the column was divided by the bent drift ratio, and this ratio was used as a measure of the distribution of the column rotation (Rotation-Drift Contribution). The values are shown in Figure 4-30 for Bents 1 and 3. The interface rotation contribution to the drift ratio (within
0 in. to 3 in.) increased from 60% (at 0.5% drift ratio) to 87.5% (at 4% drift ratio), whereas
the second level rotation contribution to the drift ratio (within 3 in. to 9 in.) dropped from
15% (at 0.5% drift ratio) to 5% (at 4% drift ratio). At larger drifts, 90-95% of the
deformation was concentrated at the column interfaces.

4.3.3 Bridge Displacement

Bridge displacements were recorded in five locations for the transverse direction, four
locations for longitudinal direction and 16 locations for vertical direction as discussed in
Section 2.4.3. Motions 14A, 16, 19, 20B, and 21C are presented in this section as
representatives for all the motions. The “peak displacement” terminology refers to the
maximum displacement in the south direction and the “valley displacement” terminology
refers to the maximum displacement in the north direction.

4.3.3.1 Maximum Transverse Displacement

Table 4-7 shows the absolute maximum drift ratio for each bent for all motions. Figure 4-31 shows the drift and acceleration response histories for each bent during Motion 14A, where the maximum drift ratios were 1.08%, 0.61%, and 0.75% for Bents 1, 2, and 3 respectively. Figure 4-32 shows the bridge deformation shape at the instant of the peak and valley displacements for each bent during Motion 14A. The superstructure deformations at the instant of the peak and valley displacements of Bents 1 and 2, and the valley displacement for Bent 3 show a combination of superstructure translation and in-plane rotation, whereas the superstructure deformation at the instant of the peak displacement of Bent 3 shows a superstructure translation only.
Figure 4-33 shows the drift and acceleration response histories for each bent during Motion 16, where the maximum drift ratios were 3.25%, 2.01%, and 3.13% for Bents 1, 2, and 3 respectively. Figure 4-34 shows the bridge deformation shape at the instant of the peak and valley displacements for each bent during Motion 16. The superstructure deformations at the instant of the peak and valley displacements of Bents 1 and 2, and the valley displacement for Bent 3 show a combination of superstructure translation and slight in-plane rotation, whereas the superstructure deformation at the instant of the peak displacement of Bent 3 shows a superstructure translation only.

Figure 4-35 shows the drift and acceleration response histories for each bent during Motion 19, where the maximum drift ratios were 11.81%, 7.62%, and 13.18% for Bents 1, 2, and 3 respectively. Figure 4-36 shows the bridge deformation shape at the instant of the peak and valley displacements for each bent during Motion 19. The superstructure deformations at the instant of the peak displacements of Bents 1 and 2, and the valley displacement for Bent 3 show a combination of superstructure translation and significant in-plane rotation, whereas the superstructure deformations at the instant of the peak displacement of Bent 3 and the valley displacement of Bents 1 and 2, show a superstructure translation only.

Figure 4-37 shows the drift and acceleration response histories for each bent during Motion 20B, where the maximum drift ratios were 10.83%, 7.89%, and 12.41% for Bents 1, 2, and 3 respectively. Figure 4-38 shows the bridge deformation shape at the instant of the peak and valley displacements for each bent during Motion 20B. The superstructure deformations at the instant of the peak and valley displacements of Bents 1 and 2, and the peak displacement for Bent 3 show superstructure translation only, whereas the
superstructure deformation at the instant of the valley displacement of Bent 3 shows a combination of superstructure translation and in-plane rotation.

Figure 4-39 shows the drift and acceleration response histories for each bent during Motion 21C, where the maximum drift ratios were 11.99%, 7.72%, and 12.87% for Bents 1, 2, and 3 respectively. Figure 4-40 shows the bridge deformation shape at the instant of the peak and valley displacements for each bent during Motion 21C. The superstructure deformations at the instant of the peak and valley displacements of Bent 1 show a combination of superstructure translation and significant in-plane rotation, whereas the superstructure deformations at the instant of the peak and valley displacement of Bents 2 and 3 show a combination of superstructure translation and slight in plane rotation.

The maximum drift ratio for Bent 1 was 11.99% during Motion 21C. The maximum drift ratio for Bent 2 was 7.89% during Motion 20B. The maximum drift ratio for Bent 3 was 13.18% during Motion 19.

4.3.3.2 Residual Transverse Displacement

Figure 4-41 shows the residual drift ratio after each motion starting from Motion 14A (Only Motion 14A was chosen from Motion 14 series). The maximum residual drift ratio of Bent 1 was less than 0.1% for all motions as shown in Figure 4-41-a. The maximum residual drift ratio of Bent 2 was less than 0.05% for all motions, as shown in Figure 4-41-b. The maximum residual drift ratio of Bent 3 was less than 0.4% for all motions, as shown in Figure 4-41-c. The maximum residual drift ratio was 0.4% during Motion 21C (the last motion).
4.3.3 Bent Vertical Displacement

Figure 4-42 to Figure 4-46 show the vertical displacement of each bent at the instant of the peak and valley transverse displacements for the same bent for the selected motions. The maximum vertical displacement occurred through Motion 16 was less than 0.1 in. Maximum vertical displacement occurred during Motion 19; it was 1.15 in. at Bent 3 at the south side. The vertical displacement of the bent consisted of linear translation and slight bent cap rotation and was observed at high level motion.

4.3.4 Transverse Acceleration

Table 4-8 shows the maximum shake table accelerations and the maximum bent acceleration for each motion. The maximum shake table accelerations were 1.78 g, 1.83 g, and 1.96 g for Bents 1, 2, and 3 respectively during Motion 19. The maximum acceleration of Bent 1 was 0.82 g during Motion 18. The maximum acceleration of Bent 2 was 0.72 g during Motion 16. The maximum acceleration of Bent 3 was 1.03 g during Motion 19.

Table 4-8 shows the maximum accelerations during all the motions for both shake tables and bents in bold font. Motions 16 and 20A were intended to be identical; the maximum accelerations for Bents 1, 2, and 3 during Motion 20A were 50% of the measured accelerations during Motion 16, as shown by underline font; this is attributed to the longitudinal bar fractures that caused a drop in bridge stiffness (starting from Motion 17).

4.3.5 Bridge Base Shear

The base shear could not be measured directly. There are two approaches to calculate the base shear from the instruments. The first approach uses the inertia force of the deck, which
was taken as the bent acceleration multiplied by the estimated inertial mass tributary to that bent. The accuracy of this approach depends on the estimates of the inertial masses which change during each of motion due to the change in the tributary masses. The second method uses the table actuator force minus friction force and the inertial resistance of the table, platen and column base for that bent. The second approach was expected to be more accurate because the table and base of each bent were isolated entities, and their masses could therefore be estimated with greater accuracy than that of a tributary region of the deck.

To calculate the total base shear during each motion, the summation of each bent base shear is the total base shear. To develop the total hysteretic base shear curve, the bridge center-of-mass displacement (COM) was used. The COM displacement was approximated as the average of the three relative displacements between the tables and superstructure.

Figure 4-47 to Figure 4-54 show the hysteretic force-drift relationships for each bent and the hysteretic force-COM relationships for the bridge from Motion 14A through Motion 19. The plots are fixed in the horizontal and vertical limits to permit a visual investigation for the base shear evolution during the motions.

Figure 4-47 to Figure 4-49 show that the hysteretic force-displacement relationships were almost linear with excellent re-centering capabilities during Motions 14A, 14B2, and 14C respectively. The calculated base shear from shake table actuator was similar to what was calculated from inertial forces for Bent 1, Bent 3 and the bridge, whereas the calculated base shear from the shake table actuator was higher by 30% from what calculated from inertial forces for Bent 2.
Figures 4-50 and 4-51 show that the hysteretic force-displacement relationships started to imitate the flag hysteretic shape with an excellent re-centering capabilities during Motions 15 and 16. The total base shear from the shake table actuator was similar to what was calculated from inertial forces, whereas the calculated base shear from shake table actuator was higher by 30% from what was calculated from inertial forces for Bent 2 and lower by 20% from what calculated from inertial forces for Bents 1 and 3.

Figure 4-52 shows that the hysteretic force-displacement relationships were a flag shape for Bents 1 and 3 with excellent re-centering capabilities during Motion 17. The total base shear from the shake table actuator was similar to what was calculated from inertial forces, whereas the calculated base shear from the shake table actuator was higher by 30% from what calculated from inertial forces for Bent 2 and lower by 15% from what calculated from inertial forces for Bents 1 and 3.

Figures 4-53 and 4-54 show that the hysteretic force-displacement relationships were in irregular cycles in flag shape for Bents 1 and 3 during Motions 18 and 19. The irregularity in the hysteretic plots were due to the fracture of the longitudinal reinforcement which caused noisy acceleration records. The total base shear from the shake table actuator was similar to what was calculated from inertial forces, whereas the calculated base shear from shake table actuator was higher by 30% from what calculated from inertial forces for Bent 2 and lower by 20% from what calculated from inertial forces for Bents 1 and 3.

As a trend, the base shear in Bents 1 and 3 was higher than the base shear in Bent 2. This is not surprising because the twisting of the deck added to the transverse displacements of the outer bents and Bent 2 had the longest column height.
As a trend, the total base shear hysteretic plots cannot be used to assess the re-centering capabilities of the system because the bridge total base shear does not occur at the same time as the COM displacement.

Figure 4-55 shows the base shear-drift ratio envelopes for each bent in addition to the total base shear-COM displacement envelopes for the bridge using both the inertial force and shake table actuator approaches.

The envelopes from inertial forces for Bents 1 and 3 were higher by 15-20% than what was calculated using shake table actuator after 3% drift ratio; this was due to the lack of the accurate estimation for the tributary masses during each motion, in addition to the noise in the acceleration records due to the fracture of the longitudinal bars. Figure 4-55-d shows excellent agreement between the two approaches for the total base shear. The agreement between the inertial force approach and shake table actuator approach in estimating the total base shear is not surprising because all the bridge masses are used without any coupling.

4.3.6 Modal Analysis

4.3.6.1 White Noise Analysis

White noise motions were used between the earthquake motions, as shown in Table 2-4 and Table 2-5, to estimate the dynamic properties including the periods of vibration and damping ratios of the bridge in each direction. The transfer function between the accelerations measured at the bents and the accelerations of the shake tables was obtained. To obtain the transfer function for each white noise, three steps were performed. The first step was to estimate response spectrum for each table and comparing them with the average
response spectrum, then using the average of table accelerations as single input in the transfer function. The second step was to perform the Fast Fourier Transformation (FFT) for the measured acceleration at each bent to ensure that the peak of each FFT was in same frequency range, then using the average acceleration of the bents as single output in the transfer function. The third step was to estimate the transfer function using Single Input/Single Output Algorithm (SISO).

Figures 4-56 and 4-57 show the response spectrum for each bent in addition to the average response spectrum before testing, for white noise waves after Motions 14C, 16, 17, 18, 19, 20B and by the completion of all motions. All the comparison between the shake table response spectra and the average response spectrum showed excellent agreement between them within the period range that estimated later using transfer function. The agreement allowed using the average table acceleration as single input in the transfer function.

Figures 4-58 and 4-59 show the FFT plots for the acceleration at each bent before testing, for white noise waves after Motions 14C, 16, 17, 18, 19, 20B, and by the completion of all motions. The comparison between the FFT from each bent acceleration showed that the FFTs reached their maxima at the same range of frequency. The agreement allowed using the average bent acceleration as single output in the transfer function.

Figures 4-60 and 4-61 show the transfer function plots for the average table acceleration as single input and the average bent acceleration as single output before testing, for white noise waves after Motions 14C, 16, 17, 18, 19, 20B and by the completion of all motions. The period of the first mode of vibration was estimated by finding the period for which the transfer function reached its maximum value.
Table 4-9 and Figure 4-62 show the change in the fundamental period and damping ratio that calculated using the transfer function.

The fundamental period changed from 0.24 seconds to 0.282 seconds from pretest state to bridge state after Motion 14C, from 0.303 seconds after Motion 15 to 0.97 after Motion 18, to 1.03 seconds after Motion 20A, to 1.19 second after Motion 21B and to 1.33 second by the completion of all motions.

The damping ratios from the transfer function ranged from 4% to 14.7% in transverse direction of the bridge. Maximum damping ratio was 14.65% during white noise wave following Motion 21A, Minimum damping ratio was 4.05% during white noise wave following Motion 9B.

### 4.3.6.2 Pulse Motion Analysis

The free vibration of the bridge during the near field motions (Motions 20B and 21C), allows the use of the displacement response history to estimate the fundamental period and the damping ratio for each bent individually.

Figures 4-63 and 4-65 show the response history of shake table displacement and bent displacement for Motions 20B and 20C respectively. The comparison shows that the bridge start to freely vibrate after 11 seconds during Motion 20B and after 17 seconds during Motion 21C.

Figures 4-64 and 4-66 show the peak and valley of the displacement response during the free vibration for Motions 20B and 21C respectively.

The periods and damping ratios were calculated using the logarithmic decrement, as shown in Table 4-10. The average periods of vibration were 1.32 and 1.46 seconds.
during Motions 20B and 21C respectively. The average damping ratio were 6.87% and 8.88% during Motions 20B and 21C respectively.

4.3.6.3 Square Wave Analysis

Square waves motions were used between the earthquake motions, as shown in Table 2-4 and Table 2-5. Most of square waves contained noise due to the low range of the shake table stroke that was used in the square waves. No data could be obtained before Motion 18. The square waves contained two strokes (one in the north and one in the south direction) separated by free vibration time.

Figure 4-67 shows the free vibration response of each bent after the first and second pulse for the square wave following Motion 21A.

Table 4-11 lists the calculated periods from the first and second pulse for each bent. Table 4-12 lists the calculated damping ratio using logarithmic decrement from the first and second pulse for each bent.

4.3.7 Instrumentation Tuning

Each bent were instrumented by transverse string pot to measure the transverse displacement and with three dimensional accelerometers to measure the bent accelerations. By double integrating the acceleration, the displacement at same location can be calculated. Figure 4-68 to Figure 4-72 show the comparison between the measured displacement and the calculated displacement from acceleration records for each bent during Motions 14A, 16, 19, 20B and 20C.
The plots show excellent agreement between the measured and calculated displacement. The agreement proved that both accelerometers and string pots are reliable instruments.
Chapter 5. Longitudinal Reinforcing Fracture Estimation

5.1 Introduction

This chapter shows the estimation of reinforcing steel fracture during the test using data from displacement transducers (Novotechniks) attached at each end of the columns. The displacement transducers were placed to calculate the rotation at each end of the columns throughout the test. However, they were used to estimate the strain in longitudinal reinforcing steel and then a strain threshold method and a low cycle fatigue model were developed. This chapter continues the estimation of reinforcing steel fracture from sound analysis using the audio records during each test, and the verification of those methods by specimen demolition.

5.2 Methodology

Using the assumption of rigid body rotation for the columns during the excitation, elongation of each bar can be calculated from both North-South displacement transducers (transverse direction) and East-West displacement transducers (longitudinal direction) (e.g. the elongation of the north column bar at the bottom connection of Bent 1 can be calculated by geometric rotation of the column end assuming rigid body rotation from both 1NBR3/1NBR4 and 1NBR1/1NBR2 in transverse and longitudinal directions, respectively), as shown in Figures 5-1 and 5-2. Figures A-20 to Figure A-22 show the location of displacement transducers at the column ends for Bents 1, 2 and 3. Strains can be calculated by assuming that the elongations occurred over the debonded length of the bar.
The strains calculated from this method are the total strains at the rocking interfaces due to the rotation at the interfaces. Figure 5-3 and the following equations show the calculation of bar elongation/gap elongation at predicted bar fracture.

\[
L_{(n-s)1} = \frac{Z_{n3} - Z_{n4}}{L_{(n-s)}} \times Z_{n3} \quad \text{(Eq. 5-1)}
\]

\[
L_{(n-s)2} = L_{(n-s)} - L_{(n-s)1} \quad \text{(Eq. 5-2)}
\]

\[
Z_1 = \frac{(L_{(n-s)1} - L_{n3} - Y)}{L_{(n-s)1}} \times Z_{n3} \quad \text{(Eq. 5-3)}
\]

\[
Z_2 = \frac{(L_{(n-s)1} - L_{n3} - Y - X)}{L_{(n-s)1}} \times Z_{n3} + \frac{H \times (Z_{n1} - Z_{n2})}{2 \times (0.5 \times D_{shoe} + L_{n1})} \quad \text{(Eq. 5-4)}
\]

\[
Z_3 = \frac{(L_{(n-s)1} - L_{n3} - Y - X)}{L_{(n-s)1}} \times Z_{n3} + \frac{-H \times (Z_{n1} - Z_{n2})}{2 \times (0.5 \times D_{shoe} + L_{n2})} \quad \text{(Eq. 5-5)}
\]

\[
Z_4 = \frac{(L_{(n-s)1} - L_{n3} - Y - 3X)}{L_{(n-s)1}} \times Z_{n3} + \frac{H \times (Z_{n1} - Z_{n2})}{2 \times (0.5 \times D_{shoe} + L_{n1})} \quad \text{(Eq. 5-6)}
\]

\[
Z_5 = \frac{(L_{(n-s)1} - L_{n3} - Y - 3X)}{L_{(n-s)1}} \times Z_{n3} + \frac{-H \times (Z_{n1} - Z_{n2})}{2 \times (0.5 \times D_{shoe} + L_{n2})} \quad \text{(Eq. 5-7)}
\]

\[
Z_6 = \frac{(L_{(n-s)1} - L_{n3} - Y - 4X)}{L_{(n-s)1}} \times Z_{n3} \quad \text{(Eq. 5-8)}
\]

Where

\[Z_1\] to \[Z_6\]: the gap sizes or the bar elongations;

\[Z_{n1}\] and \[Z_{n2}\]: displacement transducer reading in east and west direction respectively;

\[Z_{n3}\] and \[Z_{n4}\]: displacement transducer reading in north and south direction respectively;

\[L_{(n-s)}\] = distance between the north and south displacement transducers;
\( L_{(n-s)1} \) = distance from the south displacement transducer and section neutral axis;

\( L_{(n-s)2} \) = the distance from the north displacement transducer and section neutral axis;

\( L_{n1} \) and \( L_{n2} \): displacement from the confining tube to the east and west displacement transducer respectively;

\( L_{n3} \) and \( L_{n4} \): displacement from the confining tube to the north and south displacement transducer respectively;

\( D_{shoe} \) = The confining tube diameter (12 in.)

\( X \) = the distance between bar 1 (north bar) and bar 2 (east-north bar) in north-south direction (2.473 in);

\( H \) = the distance between bar 1 (north bar) and bar 2 (east-north bar) in east-west direction (4.283 in.)

\( Y \) = distance from the north edge of the confining tube and the center of bar 1 (north bar) (1.05 in.)

This method of estimating the strains from the displacement transducers hereafter referred to “Displacement Method”.

5.3 Method Verification

This method is only applied for high level motions (Motion 15 to Motion 21C), when the rocking gap was significant. Unfortunately all strain gauges were damaged starting from Motion 16. No direct strain data were recorded before the first bar fractured. This method of calculating the strain from bar elongation was compared to the strain from strain gauges during Motion 15 (where strain gauges were not damaged). Figure 5-4 to Figure 5-11 show
the strain comparison using both strain gauge reading and Displacement Method for Bents 1 and 3 during Motion 15 in the response history plots. The plots show agreement between the Displacement Method and the strain gauge readings. Figure 5-12 shows the maximum strain comparison obtained from both the strain gauge reading and the Displacement Method. The Displacement Method underestimates the strains due to the residual strains that were monitored by strain gauges, whereas the Displacement Method cannot detect them. The agreement between strain gauge reading and the Displacement Method was within 90-95%. This agreement allows the Displacement method to be an alternative to estimating bar strains at rocking interfaces during high amplitude motions.

5.4 Bar Elongations at First Bar Fracture

The Displacement Method was used to calculate the bar elongation at 72 locations (6 bars x 12 locations). Table 5-1 shows the maximum elongation for each bar. The maximum elongation was 0.536 in. (approximately 6% strain) for the north bar of the bottom connection of the north column of Bent 1.

The first bar fractured during Motion 17 (where the first fracture sound was heard). According to this estimation, the first fractured bar was the north bar of the bottom connection of the north column of Bent 1 because this bar had the maximum elongation values among all bars.

Figure 5-13 shows the bars’ strains for the bottom connection of the north column of Bent 1 during Motion 17. The plot with gray background indicates the fractured bar (1NB-N).
Figure B-1 to Figure B-12 show the bar strains at Motion 17 in hierarchical order from Bent 1 to Bent 3, then north column to south column and then from bottom to top connection. The plots in each figure are in a hexagonal pattern in order for the bar to be easily visualized according to the bar distribution in the column cross section. The plots with gray background indicates the fractured bar (1NB-N).

5.5 **Bar Fracture Estimation using Strain Threshold Method**

The 6% strain value was used as the threshold value to estimate bar fracture for motions beyond Motion 17. Figure B-13 to Figure B-96 show the bar strain (gap elongation \(*10^6/\text{debonded length at fractured bar location}\)) during Motion 18 through Motion 21C in hierarchical order from Bent 1 to Bent 3, then north column to south column and then from bottom to top connection. The plot with a gray background indicates the occurrence of a fractured bar. The y-axis for the plot in a gray background should be multiplied by the debonded length then divided by \(10^6\) to be converted from the strain to gap elongation at the fractured bar location. Table 5-2 summarizes the bar fracture estimation during Motion 17 through Motion 21C using the Strain Threshold Method.

During Motion 18, as shown in Figure B-13 to Figure B-24, all the bars fractured at the top and bottom connections for both north and south columns for Bent 1 (9.25% drift ratio) except for the north-east bars of the north column at the top and bottom connections and the south-east bars of the north and south columns at the top connection. For Bent 3 only the south bar of the north column at the top connection (maximum drift ratio of 6%). No bar fractured in Bent 2 (maximum drift ratio of 4.30%).
During Motion 19, as shown in Figure B-25 to Figure B-36, the remaining bars fractured in Bent 1 except for the north-east bar at the bottom connection of the north column. All the bars fractured in Bent 3 except for the south-east of the south column at the top connection (maximum drift ratio of 13.2%). Since most of Bent 1 bars fractured, the bridge stiffness was shifted to Bent 3; this caused a higher drift ratio, which caused the fracture of Bent 3 bars. Multiple bars fractured in Bent 2 as following: north, north-west, south-west and south bars at the bottom and the top connections of the north column, the north, south-east, south-west and south bars at the bottom connection of the south column, and the north, north-west, south-west and south bars at the top connection of the south column (maximum drift ratio of 7.6%). The south-east bar at the top connection of the south column of Bent 3 (maximum drift ratio of 12.7%) was the only bar that fractured after Motion 19 (during Motion 20B), as shown in Figure B-37 to Figure B-96.

By the end of testing, 63 bars fractured including all the bars at the interfaces for both Bents 1 and 3 except for the north-east bar at the bottom connection of the north column of Bent 1 in addition to 16 bars fractured at Bent 2. Only nine bars survived by the end of testing. Table 5-2 shows the fractured bar locations at the end of each test starting from Motion 17 through Motion 21C, and the surviving bars in Bent 1 and Bent 2 by the end of testing.

5.6 Low Cycle Fatigue Life using the Displacement Method

Using 6% strain as a threshold strain for the bars to fracture seems to be a very small value as the experimental direct tensile test shows that the strain at failure is more typically around 19%. 
The obtained strain histories for each bar using the Displacement Method are highly irregular, containing large number of random cycles of varying amplitudes. Therefore, the rain-flow counting method was used to extract the strain cycles of high amplitudes. The rain-flow method is very common in fatigue analysis, which defines the corresponding number of cycles at each stress or strain amplitude. It was developed in 1968 (Matsuishi and Endo 1968), and since then it has been widely used in structural engineering applications.

5.6.1 Fatigue Life Relationships for Reinforcing bar

The fatigue life of a material is defined as the number of cycles at specific deformation amplitudes resulting in failure. Fatigue strength is defined as the maximum transient stress or strain range (S) that may be repeated without causing failure for a specified number of loading cycles (N), which are the main parameters for S-N curves for engineering materials. This range is defined as the algebraic difference between the maximum and the minimum stress or strain in a stress cycle. Typical strain amplitudes in reinforced concrete (RC) members during moderate to severe seismic events can easily exceed the yield strain of almost 0.2% for (Grade 60) reinforcing steel. The number of cycles to failure at such large amplitudes is usually less than 1000 cycles, which could result from long-duration earthquakes or during an experimental test with a sequence of motions. Low-cycle fatigue is defined as failure in a material due to a relatively small number of load or deformation cycles (< 1000), and typically involves large deformations that exceed the elastic limit. Based on the available literature, there are several fatigue life relationships were developed through cyclic testing of reinforcing bars under constant amplitude loading. The existing
relationships predict the number of cycles to failure at a given strain amplitude. These relationships are in an exponential form, though when it is plotted in a log-scale it becomes linear. The constants of the fatigue formulae are obtained from test results performed by several researchers including various types of steel. The equation developed by (Koh and Stephens 1991) has two major fatigue parameters (constants). The equation is presented

\[ \varepsilon_a = \frac{\Delta \varepsilon}{2} = \varepsilon_f \cdot (2N_f)^m \]  

(Eq. 5-9)

Where

\( \varepsilon_a \): the total strain amplitude;

\( N_f \): the number of cycles to failure;

\( \varepsilon_f \) and \( m \): parameters are defining a linear relationship in log-scale.

The research done by (Mander et al. 1994) was one of the first investigations of the low-cycle fatigue behavior of reinforcing bars where both reinforcing steel and prestressing steel were tested under constant strain amplitudes. Different fatigue life relationships were developed for both types of steel to define a single line, which could be used in estimating the approximate number of cycles to failure under constant amplitude strain as follows:

\[ \varepsilon_a = \frac{\Delta \varepsilon}{2} = 0.0795 \cdot (2N_f)^{-0.448} \]  

(Eq. 5-10)

Where

\( \varepsilon_a \): the total strain amplitude;

\( N_f \): the number of cycles to failure.

After determining the number of cycles for each strain level, the results were used to estimate the accumulated damage due to reduction in the fatigue life. This reduction is
calculated using the Palmgren-Miner Rule (Miner 1945) to assess the condition of the reinforcing in the members. It was assumed that the damage due to fatigue type loading (stress or strain cycles) is accumulating linearly according to the number of loading cycles. It simply states that the fatigue life of a structure or an element is exhausted when the sum of all the fatigue fractions at different loading amplitudes from a random fatigue loading is equal to unity.

\[
\frac{n_1}{N_1} + \frac{n_2}{N_2} + \frac{n_3}{N_3} + \cdots = \sum \frac{n}{N} = 1 \quad \text{(Eq. 5-11)}
\]

Where

\(n\): the number of cycles obtained from strain response history using the rain flow algorithm for each strain increment;

\(N\): the maximum number of cycle until fracture using low cycle fatigue equations for each strain increment.

The Palmgren-Miner damage rule was used to calculate the cumulative fatigue damage in the longitudinal reinforcing bars. For the strain histories obtained from the analysis of the Displacement Method, the number of strain cycles at each strain increment “\(n\)” was determined using the rain-flow counting method. The available fatigue life relationships were used to calculate the number of cycles to failure at each strain level “\(N\)”. The increase in the cumulative fatigue damage is corresponding to the reduction in the fatigue life of the structural member subjected to fatigue type loading.
5.6.2 Bar Fracture Estimation Using the Low Cycle Fatigue Method

Most of the current research on the fatigue life of the reinforcing bars were solely conducted on bar testing. No direct study provides fatigue life analysis for reinforcing bars inside RC structure during shake table experiment with irregular cycles. This is the first time that strain estimation, using the Displacement Method, has been used for estimating reinforcing steel fatigue life as a part of large scale experiment with irregular cycles.

Using strains from the Displacement Method during Motions 15 through 21C, the number of cycles for each strain increment 0.0002 (0.1*ε_y) were calculated using the rain flow algorithm for 72 locations at the top and bottom connections of each column, and using the Mander equation (Eq. 5-10) to estimate the number of cycles needed until failure at each strain increment. A Cumulative fatigue damage index was calculated for each bar at column interfaces using the Palmgren-Miner damage rule. Figure 5-14 shows the flow chart of the Low Cycle Fatigue Method.

After Motion 15, the cumulative fatigue damage indices were lower than 8%, as shown in Figure 5-15. During Motion 16, the cumulative fatigue damage indices were lower than 30%, as shown in Figure 5-16.

During Motion 17, the north bar of the bottom connection of the north column of Bent 1 (1NB-N) had cumulative fatigue damage index of more than 100%, which predicts that (1NB-N) is the first fractured bar. Whereas the other 71 bars had cumulative fatigue damage index less than 100%, as shown in Figure 5-17.

During Motion 18, 19 longitudinal bars from Bent 1 and eight longitudinal bars from Bent 3 had cumulative fatigue damage indices of more than 100%, which predicts that those bars already fractured (total of 27 bars), as shown in Figure 5-18.
During Motion 19, four longitudinal bars from Bent 1, ten longitudinal bars from Bent 2 and 16 longitudinal bars from Bent 3 had cumulative fatigue damage indices more than 100%, which predicts that those bars already fractured during the motion (total of 30 bars). All Bent 1 and Bent 3 bars fractured by the end of Motion 19, as shown in Figure 5-19. As all the bars at the interfaces of Bents 1 and 3 fractured, only the cumulative fatigue damage indices were calculated for Bent 2 during Motion 20A through Motion 21C.

During Motion 20A, five longitudinal bars fractured, see Figure 5-20. During Motion 20B, three longitudinal bars fractured, see Figure 5-21. During Motion 21A, no longitudinal bars fractured, see Figure 5-22. During Motion 21B, three longitudinal bars fractured, see Figure 5-23. During Motion 21C, three longitudinal bars fractured, see Figure 5-24. Figure 5-15 to Figure 5-24 show the cumulative fatigue damage indices for all the bars at the interfaces of Bent 1 through Bent 3 with “N” symbol for the new fractured bars during Motions 17 and 18.

Figure 5-25 to Figure 5-36 show the cumulative fatigue damage index for each bar in bar charts. Table 5-3 shows the summary of fracture bars during all motions.

5.7 Bar Fracture Estimation using Acoustic Emissions

The bridge was monitored using different camera types. Each column connection (12 locations) had a GoPro camera to monitor the rocking behavior of each connection during each motion. Due to technical issues some of the GoPro cameras did not monitor some motions. The first bar fracture occurred during Motion 17. The only indication of the bar fracture was the sound during the motion itself. The visual investigation could not determine the fractured bar location due to the self-centering behavior and the confinement
detail at the connections, which prevent any visual inspection after the motion. The cameras were used to document the damage progress and were used to define the number of fractured bar during each motion starting from Motion 17. The first method was to count the number of fractured bars using the fracture sound in camera audio; this method underestimates the number of fractured bar because of the compressed sound of multiple fractured bars in one sound. The second method was to scale the audio time up (using slow motion audio), which isolated the bar fractured sounds (resample the time step in time domain). To obtain the slow motion audio, Windows Live Video Maker was used to resample the time up to 0.5, 0.25 and 0.125 times the original audio, then a Matlab script was used to reduce the audio noise due to machine vibrations. The audio analysis shows that the 0.25 resample ratio (lengthening the time by factor of 4) was reasonable to isolate the compressed sound of fractured bar then remove the noise by filtering the audio noise. 1NB GoPro was used to estimate the amplitude of the first fractured bar and to define noise threshold for 1NB camera for later motions, as shown in Figure 5-37. The same camera was used for Motion 18 to get the clear bar fracture sounds as most of fractured bars were in Bent 1. Figure 5-38 shows the estimation of sound of bar fractures in black circle shape, 24 sounds of fractured bar (four sounds more than the threshold strain method and three sounds less than the low cycle fatigue estimation). Most of fractured bars during Motion 19 were in Bent 3, so 3SB GoPro was used to estimate the bar fracture, 29 sound of fractured bar (11 sounds fewer than the threshold method and one sound less than the low cycle fatigue estimation), shown in Figure 5-39. Table 5-4 shows the reinforcing steel fracture summary using maximum strain threshold, low cycle fatigue and acoustic emission models from Motion 17 through the end of testing.
5.8  Fracture Evaluation by End of Testing

The bents were demolished by saw cutting the column 2 in. away from the end of the confining tube. The demolished structure showed that all the bars fractured by the end of testing at the top and the bottom connection of each bent. Figure 5-40 shows the rocking interfaces at the selected connections. The demolished structure shows agreement with the Low Cycle Fatigue Method by the end of testing.
Chapter 6. Specimen Comparison with the Conventional Bridge.

6.1 Introduction

This chapter compares the new system with the conventional system. From previous chapters, it can be noticed that the new design shows superior performance over the current design in terms of resisting major earthquake with cosmetic damage to concrete, resilience and functionality after the largest common earthquake (CCN 1.66 PGA). The new bridge configuration tested as part of this study is called the “Resilient Bridge” whereas the previous bridge tested by Johnson et al. in 2005 is called the “Conventional Bridge” (Johnson 2006).

This chapter presents a full comparison between the two bridge systems including; peak displacements, residual displacements, base shear, rotations, strains and dynamic properties using the shared motions between the two experiments.

6.2 Systems Differences

Chapter 2 presented the Resilient Bridge design, whereas Chapter 3 presented the construction of the bridge specimen, and Chapter 4 presented the measured response.

A brief summary for the Conventional Bridge system is presented to provide a background for the shake table test conducted at University of Nevada, Reno in 2005 (Johnson 2006; Ranf 2007). The Conventional Bridge was a two-span bridge; its length was 67.3 ft. The spans were 30 ft. The clear column heights from the top of the footings to the bottom of bent caps were 6, 8 and 5 ft. for Bents 1, 2 and 3 respectively. The system was designed to investigate the response of a system with irregular bent stiffness due to
different bent heights. The column bases were rigidly attached to the spread footing and bent cap. The column longitudinal reinforcement was 16#3 with 1.56% reinforcing steel ratio.

The superstructure of the bridge consisted of six precast slabs post-tensioned transversely and longitudinally to provide a continuous superstructure. Each span, consisting of three precast beam, was assembled and post-tensioned transversely using ten 1.25 in. diameter rods. Each rod was stressed to 100 kips, to connect the precast slabs, therefore preventing interface slippage and providing flexural capacity in the lateral direction. Each set of spans was placed between the bents and aligned with longitudinal post-tensioning ducts embedded into the three precast caps. The superstructure was longitudinally post-tensioned to provide continuous superstructure between the bent caps and beams. Six concrete blocks with total weight of 120 kips and lead with total weight of 58.2 kips were placed on the superstructure to represent the prototype bridge mass.

6.3 Motion Differences

The differences in input motions between the Resilient Bridge and Conventional Bridge were compared using a two degree-of-freedom (2DOF) model of the Conventional Bridge. The 2DOF model was intended to capture both the transverse motion of the structure as well as the twisting behavior caused by the varying column heights between the three bents. In addition, accidental and intentional incoherency existed in the table input motions, which were expected to further excite twisting of the superstructure.

The bridge specimen was idealized as a rigid beam excited by the three table motions through nonlinear springs representing the individual bents. The spring strengths
and stiffnesses were taken as the pre-test values based on pushover analyses presented in Johnson (2006). The individual bent stiffnesses of 67.3, 31.2, and 110.2 kips/in. for Bents 1, 2 and 3 were consistent with those computed using the recommendations presented in (Elwood and Eberhard 2009) of 71.7, 34.8, and 112 kips/in. The shear strength of the bents were taken as 40.4, 30.9, and 57.8 kips for Bents 1, 2 and 3 respectively which were close to the maximum bent base shear estimates given in (Johnson 2006) for the Conventional Bridge during the 177% Design Level Motion (Motion 18) of 49.2, 41.9, and 51.5 kips for Bents 1, 2, and 3 respectively. The bents were modeled using elastic-perfectly-plastic (EPP) hysteretic elements. The motions of each test were run individually; residual displacements due to the loading history were ignored.

The maximum drift ratios for each of the three springs in each of the shared motions are given in Figure 6-1. Prior to the Design Level Motion (Motion 16) the two responses were nearly identical. During the Design Level Motion (Motion 16), the responses of Bents 1 and 2 were nearly the same for the two experimental motions, however the response of Bent 3 was higher for the Conventional Bridge motion (2005 Motion) with a drift ratio of 4.2% versus only 3.0% for the Resilient Bridge motion. For the rest of the motions (133%, 177% and 221% Design Level Motions), the response of the model was either nearly the same or somewhat larger for the Resilient Bridge motions, except for during the 221% Design Level Motion (Motion 19) in Bent 3, in which the Conventional Bridge motion produced a slightly higher response.
6.4 Observed Damage Comparison

For similar drift ratios, the Resilient Bridge had much less visible damage than the Conventional Bridge. Table 6-1 shows a comparison between the two systems during the 33% through 221% Design Level Motion (Motions 14A through 19). Table 6-2 shows the first occurrence of the key damage stages for each bent in both bridges. Figure 6-2 shows side by side comparison for both bridges.

No damage to either the Resilient Bridge or the Conventional Bridge was observed in the low amplitude motions (less than 10.6% Design Level Motion). No damage was observed in the superstructure during the high amplitude motions; all masses and footings remained rigidly attached to the bridge superstructure and shake table through testing.

Damage to the column concrete was nearly eliminated in the Resilient Bridge. During the 33% Design Level Motion (Motion 14A), flexural cracks with 0.003 in. maximum width occurred in all of the bents in the Conventional Bridge. During the 67% Design Level Motion (Motion 15), first cracks occurred at the octagonal corners at the column ends for both Bents 1 and 3 of the Resilient Bridge, whereas the flexural crack widths increased to 0.01 in. for Bent 1 and 0.01 in. with 4 in. spalling for Bent 3 in the Conventional Bridge.

During the Design Level Motion (Motion 16), grout pad flaking occurred for both fiber grout pads at top of the columns of Bents 1 and 3 of the Resilient Bridge, whereas the flexural crack widths in the column increased to 0.02 in. with 5-in. spalling, therefore, the transverse reinforcement was exposed. The flexural crack widths in the column increased to 0.013 in. with 6 in. spalling for Bent 3 in the Conventional Bridge.
During the 133% Design Level Motion (Motion 17), cracks at octagonal corners with spalling occurred at the ends of all columns of the Resilient Bridge, in addition, first longitudinal bar fractured in Bent 1 (at 5.7% drift ratio) as mentioned in Chapter 5. In the Conventional Bridge the flexural cracks widened to 0.02 in. with 6-in. spalling, in addition to transverse reinforcement exposed for Bents 1 and 3.

During the 177% Design Level Motion (Motion 18), in the Resilient Bridge, local buckling (bulge) of the confining tube occurred at the bottom connections of Bent 1 and Bent 3, and spalling in Footing 1 around the north and south column of Bent 1. In addition, multiple longitudinal bars fractured in Bents 1 and 3 as mentioned in Chapter 5. In the Conventional Bridge, flexural cracks width of Bent 3 widened to 0.08 in. with 5.6-in. spalling height, transverse and longitudinal reinforcing exposed and longitudinal reinforcing buckled.

During the 221% Design Level Motion (Motion 19), additional local buckling (bulge) of the confining tube, grout pad crushed about 2-2.5 in. from the 1/4 in. gap at the top of Bents 1 and 3, in additional to horizontal flexural cracks at 6-8 in. away from the end of the confining tube of Bents 1 and 3 in the Resilient Bridge. In the Conventional Bridge, more spalling, transverse and longitudinal reinforcing exposed and longitudinal reinforcing buckling occurred for Bents 1 and 3. The first longitudinal bar fractured at Bent 3 at a 7.85% drift ratio.

The damage in the Resilient Bridge can be either repaired or controlled before the design. The grout pad crashing can be repaired by injecting a new grout without closing the bridge. The bulging of the steel tube can be eliminated by using thicker tube wall. The
yielding of the prestressing strands and fracture of the longitudinal reinforcement can be controlled by designer as mentioned in Chapter 8.

6.5  Measured Response Comparison

The instrumentation plan for the Resilient Bridge was prepared to match global instrumentation plan for the Conventional Bridge to provide strong points of comparison during data analysis phase. More instruments were added to capture the unique characteristics of the Resilient Bridge such as strand load cells, strand strain gauges, and strain gauge rosettes for confining tubes.

This section is divided into two subsections. The first subsection compares the global response of each bridge such as peak displacement, residual displacements, calculated base shear and change in the fundamental period. The second subsection compares the local responses such as column rotations and strains in both longitudinal and transverse reinforcement.

6.5.1  Global Response Comparison

This section compares the global responses of each bridge such as peak displacement, residual displacements, calculated base shear and change in fundamental period.

6.5.1.1  Measured Displacement Comparison

Table 6-3 shows the maximum drift ratio for each bent and the peak table acceleration for both bridges. Figures with odd numbering from Figure 6-3 to Figure 6-13 show the Bent drifts and center-of-mass (COM) displacements during the 33% to 221% Design Level Motions (Motion 14A throughout Motion 19). The maximum drift ratios were similar for
both bridges up to the Design Level Motion (Motion 16). Beyond the 177% Design Level Motion, the Resilient Bridge produced higher drift ratios over the Conventional Bridge due to reduction of bridge stiffness caused by longitudinal reinforcing fracture that occurred during the 177% and 221% Design Level Motions (Motions 18 and 19). Figure 6-15 shows the residual drift ratio for both bridges from Motion 14A through the final motion; showing that the residual drifts for the Resilient Bridge were much smaller than the Conventional Bridge. The “post-peak” motions were different between the two bridges; three different motions, including near field and pulse motions (Sylmar and Takatori), were added for the Resilient Bridge to determine the bridge functionality after the biggest motion. The residual drift ratios of the Resilient Bridge was lower than 0.1% for Bents 1 and 2 during all motions and lower than 0.2% for Bent 3 up to the 221% Design Level Motion (Motion 19) and not higher than 0.4% by the end of testing, approximately 0.25 in. The “post-peak” motions were added to the Conventional Bridge after removing the mass over Bent 3 (Motions 21 and 22). The residual drift ratio of the Conventional Bridge was 0.4% for Bent 1 and 1.7% for Bent 3 with significant damage after Motion 22 including 1-in. permanent deformation. The residual drift ratios for the Conventional Bridge seem to be small values; this is in part due to the choice of ground motion. The Century City record is a far-field motion. Typically, these motions produce smaller residuals than near-field, pulse type excitations.

The residual drift ratios were consistently lower for the Resilient Bridge than for the Conventional Bridge.

6.5.1.2 Calculated Base Shear Comparison

During shake table testing for the bridge systems, it was difficult to attach load cells to measure the base shear directly during the test; therefore two methods were used to
calculate the base shear of each bent. The first method was to multiply the measured acceleration at the top of each bent by its tributary mass. The second method was to subtract the product of shake table feedback acceleration times the mass that was rigidly attached to the table (including the table platen, spacer block and footing) and friction force, from the shake table force feedback as discussed in Chapter 4. The rigidly attached weights attributed to shake table actuators were 80.5 kips, 66.7 kips and 84.6 kips for shake tables 1, 2 and 3 respectively for the Resilient Bridge and 76.6 kips, 62.7 kips and 80.7 kips for shake tables 1, 2 and 3 respectively for the Conventional Bridge. Due to the lack of actuator force feedbacks, except for the 67% and 177% Design Level Motions (Motions 15 and 18) for the Conventional Bridge, only the first method is presented.

Table 6-4 summarizes the maximum base shear for each bent and maximum total base shear for both bridges. Figures with even numbering from Figure 6-4 to Figure 6-14 show calculated bent base shear and bridge base shear comparison for both bridges.

The total base-shear envelopes were slightly higher for the Resilient Bridge, as shown in Figure 6-16. Figure 6-16-c shows that displacement demand for both systems were similar up to the 133% Design Level Motion (Motion 17). The displacement demand was higher for the Resilient Bridge during the 177% and 221% Design Level Motions (Motions 18 and 19) due to longitudinal reinforcement fracture.

6.5.1.3 Shift in Fundamental Period

Transfer function for average input motion as a single input and average bent acceleration as a single output from the white noise run after the shared motions were used to calculate the first mode period for both the Resilient Bridge and the Conventional Bridge Figures 6-17 and 6-18 show the transfer function comparison after motions. Table 6-5 and Figure 6-
19 shows a comparison of changes in period for both bridges. The period for the Resilient Bridge was lower than the Conventional Bridge up to the 133% Design Level Motion (Motion 17). The rate of change in the period was higher for the Resilient Bridge due to fracture of longitudinal reinforcing bars during the motions beyond the 133% Design Level Motion (Motion 17).

### 6.5.2 Local Response Comparison

This section compares the two systems in local response such as column rotations and strains in both longitudinal and transverse reinforcement.

#### 6.5.2.1 Rotation Comparison

Differences in measured behavior were consistent with the observed distribution of deformations in the column. The deformations concentrated at the ends for the Resilient Bridge are shown in Figure 6-20 to Figure 6-22.

During the 33% Design Level Motion (Motion 14A), the average base rotation was between 60-65% of the total drift, while the average rotation was nearly 50% for the Conventional Bridge (plots a and b).

During the 177% Design Level Motion (Motion 18), the average base rotation constituted between 90-95% of the total drift, while the average rotation was nearly 75% for the Conventional Bridge (plots c and d).

Figure 6-23 to Figure 6-25 compare the average rotations for Bents 1, 2 and 3 for both bridges corresponding to the maximum drift ratio for the 33% through 221% Design Level Motions (Motions 14A through 19).
At each bent, the average of the four individual connection rotations over the first 9 in. of the column was divided by the bent drift ratio, and this ratio was used as a measure of the distribution of the column rotation. The values are shown in Figures 6-26 and 6-27 for Bents 1 and 3 respectively. At all drifts, but particularly at larger drifts, the concentration of the deformations at the ends of the columns was greater in the Resilient Bridge than in the Conventional Bridge. The Conventional Bridge shows flexural rotation due to moment concentration at plastic hinge zones which caused spalling, transverse reinforcing exposure and fracture at the column ends. This concentration led to increased strain along longitudinal reinforcing bars in the plastic hinge region.

6.5.2.2 Strain Comparison

The distribution of strains in the reinforcement was consistent with the manner in which each specimen deformed. In the Resilient Bridge columns, strains were distributed along the debonded region at the connection interface, and dropped rapidly outside it. In the Conventional Bridge columns, in which the bars were not debonded, the strains in the reinforcement were distributed throughout the plastic hinge regions. Despite this difference, longitudinal reinforcement yielding was first recorded during the 20% and 33% Design Level Motions (Motions 13 and 14A) both bridges. This similarity is attributable to the fact that the debonded length in the Resilient Bridge was chosen to be nearly equal to the calculated plastic hinge length of the Conventional Bridge.

Figure 6-28 shows the distribution of the measured strain in the extreme tensile reinforcement at the top connection of the north column of Bent 1 in the Resilient Bridge and the corresponding location in the Conventional Bridge. Strains are shown for drift ratios of 1.1% and 2.1%. In both bridges, the reinforcement was instrumented in three
locations: below, at, and above the interface. Due to the use of debonding sleeves in the Resilient Bridge, the locations of the gauges are not identical between the two specimens although the bonded length between gauges was similar. During the low level motions, strains at the rocking interface for Bent 1 of the Resilient Bridge were lower than the interface strains of Bent 1 for the Conventional Bridge (Motions 12 and 13). The distribution of strain in the reinforcement of the Resilient Bridge was highly localized at the interface. The bar had excellent bond within the confining tubes.

Figure 6-29 shows the strain envelopes for the extreme tensile reinforcement at the bottom connection of the north column of Bent 3 in the Resilient Bridge and the corresponding location in the Conventional Bridge. The strains at the interface and in the anchorage region (in either the bent cap or footing) were similar for both bridges. However, the strains within the columns near the plastic hinge location were much higher for the Conventional Bridge (up to four times at 1.6% drift ratio). This difference is consistent with the formation of a plastic hinge in the conventional column, and the associated progressive debonding near the location of the column strain gauge, located 6 in. above the interface. The fracture of longitudinal reinforcement is discussed in Section 6.4.

For the transverse reinforcement, in the Resilient Bridge, strains were monitored in the spiral at 3 in. above the end of the confining tube. The spiral remained elastic at all times, and the maximum strain recorded was 85% of the yield strain. In the Conventional Bridge, strains were monitored in the spiral at 6 in. from the interfaces, near the end of the calculated plastic hinge. For the Conventional Bridge, the spiral first yielded in Bent 3 during the 20% Design Level Motion (Motion 13), and fractured in Bent 3 during the 221% Design Level Motion (Motion 19) at 7.9% drift ratio.
Bulging of the confining tube first occurred during the 177% Design Level Motion (Motion 18) at the bottom connections of Bents 1 and 3. The bulging increased during the subsequent motions. The confining tube detail was successful in preventing spalling damage at the ends of the columns, although its wall thickness and length (D/90 and D/2 respectively) were chosen empirically. This damage state could be delayed or eliminated by using a tube with a thicker wall. In the earlier quasi-static tests (Schaefer et al. 2013; Kennedy 2015) the thickness of the tube wall was D/80 and no bulging occurred.

Strains in the confining tube were monitored using strain rosettes located 1.5 in. above the column-footing interface on the north side of both columns of Bent 3. The confining tubes remained elastic until approximately 3.0% drift, then yielded and bulged in Bents 1 and 3 at drift ratios of approximately 9.2% and 6.0%, respectively.

6.6 Construction Speed Comparison

The advantages of using precast bent system have already been demonstrated in the field for a bridge that did not have prestressed columns (Khaleghi et al. 2012). Figure 6-30 compares the onsite construction time for both bridge systems in the laboratory environment. Although the laboratory is not a typical construction site and many parameters, including lab technician experience, affected the scheduling of each bridge, the bents of the Resilient Bridge were built in one week, compared with five weeks for the bents of the Conventional Bridge (80% saving in the construction time). In the Resilient Bridge, the column-to-bent-cap connection needed only three days of curing before installing the superstructure beams. In the Conventional Bridge, 14 days were needed between casting the column and the bent cap, and another 14 days were required after
casting the bent cap before placing the superstructure beams. The time savings was therefore largely attributed to precasting, which eliminated the column and cap-beam curing cycles.

6.7 Comparison Conclusions

1- The Resilient Bridge produced less damage compared to the Conventional Bridge up to the Design Level Motion. Final damage in the Resilient Bridge included longitudinal reinforcing fracture, confining tube bulging and grout pad flaking. The first longitudinal bar fracture occurred at 5.7% drift. In the Conventional Bridge, damage consisted of major spalling, transverse reinforcing buckling, and longitudinal reinforcing buckling and fracture with significant damage of Bent 3 at 7.8% drift.

2- Total base shear for the Resilient Bridge was slightly higher than the Conventional Bridge up to the Design Level Motion and higher for the Resilient Bridge during the larger motions.

3- Since pretensioning strands did not fracture and the residual drifts for the Resilient Bridge were essentially zero after exceeding 12% drift, the primary issue with the Resilient Bridge was significant longitudinal reinforcing fracture.

4- Displacement responses for the Resilient Bridge were same as the Conventional Bridge up to the 133% Design Level Motion (Motion 17). Beyond the 133% Design Level Motion (Motion 17), the Resilient Bridge produced higher displacement than the Conventional Bridge, while residual drifts were lower than 0.4% for Resilient Bridge.
5- The rate of change of the Resilient Bridge periods were higher than the Conventional Bridge. The change in period for the fundamental mode in the Resilient Bridge was 0.28 to 1.03 seconds, whereas the change in period for fundamental mode in the Conventional Bridge was 0.32 to 0.75 seconds.

6- The Resilient Bridge was functional after a maximum drift ratio of 13% with very small residual drift. The bridge was able to resist three different types of motion. However, the Conventional Bridge had significant damage for Bent 3 at 7.8% drift which made necessary to remove the mass over the bent to continue the experiment.

7- Rotations for the Resilient Bridge were concentrated at the top and bottom rocking interfaces; this eliminated the damage within bridge columns compared to the Conventional Bridge where flexural rotations caused severe damage to bridge columns.

8- For low amplitude motions (less than 10.6% Design Level Motion), strains in the longitudinal reinforcement for the Resilient Bridge were lower on average than the Conventional Bridge.

9- The Resilient Bridge showed shorter onsite construction time (80% saving in construction time) by using the prefabricated columns and bent caps. The saving was more than a month over the Conventional Bridge.

7.1 Introduction

Three dimensional numerical model was developed in OpenSEES to simulate the dynamic response of the conventional 2-span bridge tested at the University of Nevada, Reno, 2005 (Ranf 2008). The new model for the Resilient Bridge had to include debonded reinforcement, debonded pretensioning strands and the rocking boundary to simulate the experimental specimen and to use the model later for parametric studies, as presented in Chapter 8.

This chapter describes the assumptions of the numerical model using OpenSEES for the pretensioned, rocking system. The chapter includes full comparison for most of the motions used during the test in two stages: 1) Prior to reinforcement fracture and 2) After reinforcement fracture. The comparisons were divided into two sections: 1) Response history comparison for drift ratios, bridge center-of-mass (COM) displacement and base shear, and 2) Accumulative comparison for base shear envelopes, average reinforcement strain envelopes, average strand force and strain envelopes, rotation to drift contribution envelopes, and accumulative fatigue damage indices.

7.2 Model Assumptions

Modeling assumptions, including element assumptions, section assumptions and material assumptions, are discussed in this section. Figure 7-1 shows sketch for the three dimensional stick model of the Resilient Bridge using OpenSEES. Figures 7-2 and 7-3
show detailed sketches for the rocking column modeling. Table 7-1 shows the element, section, material levels for each structural component of the bridge columns.

### 7.2.1 Element Assumptions

As shown in Figures 7-2 and 7-3, different elements were used to model the bridge including: 1) Zero length section for the rocking interfaces, 2) Co-rotational truss element for the debonded portion of the prestressing strands from footing interface to bent cap interface, 3) Truss elements with length equal to the debonded length in addition to four times the bar diameter on either side of debonded length for the debonded reinforcement (Kennedy 2015), 4) Rigid elements between the column section and strand nodes at the interface, 5) Rigid elements between the column section and bar nodes at the end of the debonded regions, 6) Force beam element with concrete confinement of the confining tube without bars for the confining tube sections inside the debonded region, 7) Force beam element with concrete confinement of the confining tube with bars for the tube sections inside the bonded region of the tube, 8) Rigid elements between the end of rocking details and bent caps, and 9) Rigid elements between the bridge superstructure and additional masses.

Figure 7-2 shows a three-dimensional sketch for the model details at the top connection between the columns and bent caps. Figure 7-3 shows a two-dimensional sketch for the column details at the top and bottom connections.

### 7.2.2 Section and Material Assumptions

Table 7-1 lists the section and material assumptions of each structural component used in the numerical model. For different column sections, as shown in Figure 7-3. Nonlinear
beam column sections using five integration points (Gauss-Lobatto integration points) were defined.

The concrete sections were divided in both the radial and tangential directions. The tangential direction was divided into 40 sectors, and the radial direction was divided into 20 circles inside the core and 2 circles inside the cover, as shown in Figure 7-4. The difference between the concrete in core and cover was because of the difference in the concrete properties caused by the spiral and confining tube.

For the portion of the column between the rocking interface and the end of the debonded bars inside the confining tube, the core and cover fiber sections were 880 fibers of confined concrete calculated from the effect of the confining tube using Concrete04 Material. Concrete04 Material properties are shown in Table 7-2-b.

For the portion of the column between the end of the debonded bars inside the confining tube and the end of the confining tube, the core and cover fiber sections were 880 fibers of confined concrete calculated from the effect of the confining tube using Concrete04 Material. Concrete04 Material properties are shown in Table 7-2-b. Six steel fibers with area of 0.11 in$^2$ were assigned for #3 bars, and six steel fibers with area of 0.20 in$^2$ were assigned for #4 bars. The reinforcing bars were assigned with Reinforcing Steel Material. The Reinforcing Steel Material properties are shown in Table 7-3.

For the portion of the column between the ends of the confining tubes, the core fiber sections were 800 fibers of confined concrete calculated from the effect of the spirals using Concrete04 Material. Concrete04 properties are shown in Table 7-2-c. The cover fiber sections were 80 fibers of unconfined concrete using Concrete04 Material. Concrete04 Material properties are shown on Table 7-2-a. Six steel fibers with area of 0.11 in$^2$ were assigned for #3 bars, and six steel fibers with area of 0.20 in$^2$ were assigned for #4 bars. The reinforcing bars were assigned with Reinforcing Steel Material. The Reinforcing Steel Material properties are shown in Table 7-3.
in² were assigned for #3 bars and six steel fibers with area of 0.20 in² were assigned for #4 bars. The reinforcing bars were assigned with Reinforcing Steel Material. The Reinforcing Steel Material properties are shown in Table 7-3.

For the debonded reinforcing bars, as shown in Figures 7-2 and 7-3, truss element were used with one fiber section (area of 0.11 in²) for #3 bar. The truss elements were connected to the column sections using rigid elements at Level 1, as shown in Figure 7-3. The reinforcing bars were assigned with Reinforcing Steel Material incorporated with Fatigue Material. The Reinforcing Steel and Fatigue Material properties are shown Tables 7-3 and 7-4 respectively.

For the rocking interface, the section were divided into 880 fibers and assigned with Elastic-Perfectly Plastic Gap Material (EEP Gap). The EEP Gap Material properties are shown in Table 7-5-a.

For the prestressing strands, as shown in Figures 7-2 and 7-3, a co-rotational truss element was used with one fiber section (area of 0.085 in²) for 3/8 strand. The co-rotational truss elements were connected to the column sections using rigid elements at Level 2, as shown in Figure 7-3. The prestressing strands were assigned with Elastic-Perfectly Plastic Gap Material (EEP Gap) incorporated with Initial Strain Material to take in account the effect of the prestressing forces. The EEP Gap and the Initial Strain Material properties are shown Tables 7-5-b and 7-6 respectively.
7.2.3 Additional Assumptions

7.2.3.1 Input Motions

The input motions involved multiple support excitation including displacement, velocity and acceleration shake-table feedback in both biaxial directions. By using displacement, velocity and acceleration feedback, the program imposed the displacements at the defined nodes and use the input accelerations and velocity without differentiating the displacement to obtain them. By using the feedback from the tables, these are the actual inputs into the specimens.

The motions were run consecutively to take in account the effect of low cycle fatigue from each individual motion.

7.2.3.2 Anchorage Slip

The anchorage slippage was calibrated for concrete using the effective concrete depth that was defined as 0.25 times the column diameter (Berry and Eberhard 2004), as shown in Figure 7-5.

For steel, the length of truss elements representing the debonded steel bars were iterated using the debonded sleeve length and the debonded sleeve length plus 2, 3 and 4 times the bar diameter at each end, to obtain the right bridge stiffness. The final length of truss elements used in the calibrated model was the debonded sleeve length in addition to four times the bar diameter at each end, this assumption agreed with Kennedy (2015).
7.2.3.3 Damping

Rayleigh proportional damping was used in the nonlinear response history analyses cases in OpenSEES. Rayleigh damping in the analytical models of the bridge was only calculated based on mass proportional damping according to equation (Eq. 7-1) (Chopra, 2007);

\[
\zeta_n = \frac{a_0}{2} \frac{1}{\omega_n}
\]  
(Eq. 7-1)

where:

\( \zeta_n \) = damping ratio for \( n \)th mode;
\( a_0 \) = mass-proportional damping coefficient;
\( \omega_n \) = \( n \)th mode frequency (rad/sec).

The targeted modal damping ratio in the analyses was set to be 5%; the value was calculated as an average damping ratio from transfer function for single input/single output through the test, as mentioned in Chapter 4.

7.2.3.4 Additional Masses

There are two sources of bridge mass: 1) Mass from the bridge components’ self-weight which were modeled as lumped masses at the nodes of the bridge superstructure, and 2) Mass from the external weights added to the top of the superstructure to condense the masses lost due to scaling from prototype to scaled specimen. The bridge superstructure was divided into multiple massless elements then the mass due to self-weight was then defined at the superstructure nodes. Half of the column mass was lumped at the top of each column. The external masses were lumped at their center-of-gravity then connected to the
bridge superstructure using rigid elements. The distribution of the masses is shown in Figure 2-6.

Exact superstructure masses and weights were modeled with a total of 274 kips vertical reaction which is off by 1 kips from the real specimen. The difference in superstructure masses between the specimen and the model was 0.36% due to the thicker bent caps inside each span.

7.2.3.5 Numerical Procedure

The Newton-Raphson algorithm was the default method for advancing to the next time step in the OpenSEES analysis. If this method failed to converge within a tolerance of $1 \times 10^{-8}$ after 1000 iterations, the algorithm was temporarily switched to the Broyden algorithm. To reduce computational demand, this algorithm was switched back to the Newton-Raphson algorithm once convergence was obtained (Ranf 2008).

The Newmark average acceleration method (Newmark 1959) was used as the integration algorithm in OpenSEES. The suggested values of $\alpha = 0.5$ and $\beta = 0.25$ were used to create an unconditionally stable integration algorithm and to minimize errors associated with numerical damping (Ranf 2008).

7.3 Model Assessment Prior to Reinforcement Fracture

The model assessment was divided into two sections: 1) Prior to reinforcement fracture, and 2) Post reinforcement fracture. In this section the model assessment prior to fracture is discussed in two stages: 1) Response history assessment for bent drift ratio, bent base shear, the center-of-mass displacement, twisting displacement and total base shear, and 2) Accumulative assessment for bridge base shear and average reinforcing strains.
7.3.1 Response History Assessment

In this section, response history assessment for each motion from Motion 14A through Motion 16 (prior to fracture motions) is discussed.

7.3.1.1 Motion 14A (33% Design Level Motion) Assessment

Figure 7-6 shows the comparison between the measured and analytical results for the drift ratio of each bent (plots a, b, and c) and a comparison between the measured and analytical results for the base shear of each bent (plots d, e, and f). The numerical model performed well in matching both the amplitude and period of the measured drift ratio and base shear with agreement between 85-90% (coefficient of determination $R^2$) with the measured response.

Figure 7-7 shows the comparisons between the measured and analytical results for the center-of-mass (COM) displacement and twisting displacement (twisting displacement is defined as the difference between the displacements of outer bents). The numerical model performed well in matching both the amplitude and period except between time of 11 to 11.5 seconds where the numerical model underestimated the twisting displacement.

Figure 7-8 shows the comparisons between the measured and analytical results for the total base shear, as shown in Section 4.3.5. The numerical model performed well in matching both the amplitude and period of the measured base shear.

Figure 7-9 shows the comparisons between the measured and analytical results for drift ratio and base shear relationship for each bent. The numerical model performed well in matching the stiffness of Bents 1 and 2, and slightly overestimated the stiffness of Bent 3. This trend is also visible for the remaining tests.
7.3.1.2 Motion 14B1 Assessment

Figure 7-10 shows the comparison between the measured and analytical results for the drift ratio of each bent (plots a, b, and c) and a comparison between the measured and analytical results for the base shear of each bent (plots d, e, and f). The numerical model performed well in matching both the amplitude and period of the measured drift ratio and base shear with agreement between 85-90% (coefficient of determination $R^2$) with the measured response of Bents 1 and 2, however the model underestimated the drift ratio of Bent 3.

Figure 7-11 shows the comparisons between the measured and analytical results for the center-of-mass (COM) displacement and twisting displacement. The numerical model performed well in matching both the amplitude and period except between time of 8.25 to 8.75 seconds where the numerical model underestimated the twisting displacement.

Figure 7-12 shows the comparisons between the measured and analytical results for the total base shear. The numerical model performed well in matching both the amplitude and period of the measured base shear with 95% agreement (coefficient of determination $R^2$).

Figure 7-13 shows the comparisons between the measured and analytical results for drift ratio and base shear relationship for each bent. The numerical model performed well in matching the stiffness of Bents 1 and 2 and slightly overestimated the stiffness of Bent 3. This trend is also visible for the remaining tests.

The fact that Motion 14B1 is very low amplitude motion leads to absence of significant rocking in the numerical model.
7.3.1.3 **Motion 14B2 Assessment**

Figure 7-14 shows the comparison between the measured and analytical results for the drift ratio of each bent (plots a, b, and c) and a comparison between the measured and analytical results for the base shear of each bent (plots d, e, and f). The numerical model performed well in matching both the amplitude and period of the measured drift ratio and base shear with agreement between 90-95% (coefficient of determination $R^2$) with the measured response of Bents 1 and 2, however the model underestimated the drift ratio of Bent 3.

Figure 7-15 shows the comparisons between the measured and analytical results for the center-of-mass (COM) displacement and twisting displacement. The numerical model performed well in matching both the amplitude and period.

Figure 7-16 shows the comparisons between the measured and analytical results for the total base shear. The numerical model performed well in matching both the amplitude and period of the measured base shear with 95% agreement.

Figure 7-17 shows the comparisons between the measured and analytical results for drift ratio and base shear relationship for each bent. The numerical model performed well in matching the stiffness of Bents 1 and 2 and slightly overestimated the stiffness of Bent 3. This trend is also visible for the remaining tests.

Since Motion 14B2 is a medium amplitude near field motion, it shows that the model is predicting both far field and near field ground motions very well.

7.3.1.4 **Motion 14C Assessment**

Figure 7-18 shows the comparison between the measured and analytical results for the drift ratio of each bent (plots a, b, and c) and a comparison between the measured and analytical
results for the base shear of each bent (plots d, e, and f). The numerical model performed well in matching both the amplitude and period of the measured drift ratio and base shear with agreement between 90% (coefficient of determination $R^2$) with the measured response of all bents.

Figure 7-19 shows the comparisons between the measured and analytical results for the center-of-mass (COM) displacement and twisting displacement. The numerical model performed well in matching both the amplitude and period except for the peak values of COM displacement and twisting displacement where the model underestimated them by 20%.

Figure 7-20 shows the comparisons between the measured and analytical results for the total base shear. The numerical model performed well in matching both the amplitude and period of the measured base shear with 95% agreement (coefficient of determination $R^2$).

Figure 7-21 shows the comparisons between the measured and analytical results for drift ratio and base shear relationship for each bent. The numerical model performed well in matching the stiffness of Bents 1 and 2 and slightly overestimated the stiffness of Bent 3. This trend is also visible for the remaining tests.

Since Motion 14C is a medium amplitude near field motion, it shows that the model is predicting both far field and near field ground motions very well.

7.3.1.5 Motion 15 (67% Design Level Motion) Assessment

Figure 7-22 shows the comparison between the measured and analytical results for the drift ratio of each bent (plots a, b, and c) and a comparison between the measured and analytical
results for the base shear of each bent (plots d, e, and f). The numerical model performed well in matching both the amplitude and period of the measured drift ratio and base shear with agreement between 75-80% (coefficient of determination $R^2$) with the measured response of all bents. The model underestimated the drift ration between 10.5 to 11.5 seconds for Bent 1 and between 9.5 to 10.5 seconds for Bent 3.

Figure 7-23 shows the comparisons between the measured and analytical results for the center-of-mass (COM) displacement and twisting displacement. The numerical model performed well in matching both the amplitude and period except for the peak values of COM displacement and twisting displacement where the model overestimated the twisting displacement between 9.5 to 10.5 seconds and underestimated it between 10.5 to 15 seconds.

Figure 7-24 shows the comparisons between the measured and analytical results for the total base shear. The numerical model performed well in matching both the amplitude and period of the measured base shear with 85-90% agreement (coefficient of determination $R^2$).

Figure 7-25 shows the comparisons between the measured and analytical results for drift ratio and base shear relationship for each bent. The numerical model performed well in matching the stiffness of Bents 1 and 2 and slightly overestimated the stiffness of Bent 3. This trend is also visible for the remaining tests.

7.3.1.6 Motion 16 (Design Level Motion) Assessment

Figure 7-26 shows the comparison between the measured and analytical results for the drift ratio of each bent on the left plots (plots a, b, and c) and a comparison between the measured
and analytical results for the base shear of each bent on the right plots (plots d, e, and f). The numerical model performed well in matching both the amplitude and period of the measured drift ratio and base shear with agreement between 90-95% (coefficient of determination $R^2$) with the measured response of Bents 1 and 2, however the model underestimate the drift ratio between 10.5 to 11.5 seconds for Bent 3.

Figure 7-27 shows the comparisons between the measured and analytical results for the center-of-mass (COM) displacement and twisting displacement. The numerical model performed well in matching both the amplitude and period for COM displacement and overestimating the twisting displacement between 12 to 12.5 seconds.

Figure 7-28 shows the comparisons between the measured and analytical results for the total base shear. The numerical model performed well in matching both the amplitude and period of the measured base shear with 95% agreement (coefficient of determination $R^2$).

Figure 7-29 shows the comparisons between the measured and analytical results for drift ratio and base shear relationship for each bent. The numerical model performed well in matching the stiffness of Bents 1 and 2 and slightly overestimated the stiffness of Bent 3. This trend is also visible for the remaining tests.

7.3.2 Accumulative Assessment
Cumulative comparisons were conducted to compare the behavior of different elements in the model with the measured response. Bent base shear and total bridge base shear from the calibrated model were compared to those values measured during the test using accelerations record and shake-table actuator force (Thonstad et al. 2016; Mantawy et al.
2016). The comparison (see Figure 7-30) shows excellent base shear estimation from the calibrated model except in the comparison between analytical base shear for Bent 2 and the base shear calculated from actuator force.

Average strain ductility envelopes using the calibrated model was compared with the measured average strain ductility, as shown in Figure 7-31. The figure shows excellent agreement with same strain ductility slope. The ductility slope equals $333.33^\epsilon_y$ ($3.33^\epsilon_y$ per 1\% drift ratio). The shift in strain in measured response between 0.75\% and 1.00\% drift ratio was due to the residual strains in the measured strains and the use of a pair strain gauges that led to a reduction in reinforcement bar area at the interface which the calibrated model cannot detect.

7.4 Post Fracture Model Assessment

The calibrated model prior to reinforcement fracture was used to calibrate the post reinforcement fracture motions by incorporating the Fatigue Material with the Reinforcing Steel Material. This material model accounts for the effects of low cycle fatigue. A modified rain flow cycle counter has been implemented to track strain amplitudes. This cycle counter is used in concert with a linear strain accumulation model (i.e. Miner’s Rule), based on Coffin-Manson log-log relationships describing low cycle fatigue failure (OpenSEES).

Once the fatigue material model reaches a damage level of 1.0, the force (or stress) of the parent material becomes zero ($1.0e^8$ times the call to the material) (OpenSEES). The calibrated model was run twice: 1) with Fatigue Material incorporated with Reinforcing Steel Material (fracture included) and 2) without Fatigue Material (Fracture excluded) in
order to contrast the effect of including the fracture effects in the model. The calibrated model using the Fatigue Material showed superior agreement between the analytical responses and measured responses, as presented in the following sections.

7.4.1 Response History Assessment

In this section, response history assessments for each motion from Motion 17 through Motion 19 and Motion 20B (post reinforcement fracture motions) are discussed.

7.4.1.1 Motion 17 (133% Design Level Motion) Assessment

Figure 7-32 shows the comparison between the measured and analytical results for the drift ratio of each bent in case of fracture included (black lines for analytical response), and the measured and analytical results for the drift ratio of each bent in case of fracture excluded (blue lines for analytical response). There is no significant change in results because only one bar fractured during Motion 17. The numerical model performed well in matching both the amplitude and period of the measured drift ratio and base shear with agreement between 85-90% (coefficient of determination $R^2$) with the measured response of all bents. However, the model underestimated the drift ratio between 14.5 to 17 seconds for Bent 1.

Figure 7-33 shows the comparisons between the measured and analytical results for the center-of-mass (COM) displacement for the case of fracture included (see Figure 7-33-a) and the comparisons between the measured and analytical results for the center-of-mass (COM) displacement for the case of fracture excluded (see Figure 7-33-b). The numerical model performed well in matching both the amplitude and period for COM displacement and underestimating the COM displacement between 14.5 to 17 seconds.
7.4.1.2 Motion 18 (177% Design Level Motion) Assessment

Figure 7-34 shows the comparison between the measured and analytical results for the drift ratio of each bent in case of fracture included (black lines for analytical response) and the measured and analytical results for the drift ratio of each bent in case of fracture excluded (blue lines for analytical response). The numerical model performed well in matching both the amplitude and period of the measured drift ratio and base shear with agreement between 85-90% (coefficient of determination $R^2$) with the measured response of all bents for the case of fracture included. However, the model showed less agreement if fracture is excluded.

Figure 7-35 shows the comparisons between the measured and analytical results for the center-of-mass (COM) displacement for the case of fracture included (see Figure 7-35-a) and the comparisons between the measured and analytical results for the center-of-mass (COM) displacement for the case of fracture excluded (see Figure 7-35-b). The numerical model performed well in matching both the amplitude and period for COM displacement in case of fracture included.

Motion 18 assessment shows that including fracture in the model can lead to predicting the measured response.

7.4.1.3 Motion 19 (221% Design Level Motion) Assessment

Figure 7-36 shows the comparison between the measured and analytical results for the drift ratio of each bent in case of fracture included (black lines for analytical response) and the measured and analytical results for the drift ratio of each bent in case of fracture excluded (blue lines for analytical response). The numerical model performed well in matching both
the amplitude and period of the measured drift ratio and base shear with agreement between 85-90% (coefficient of determination $R^2$) with the measured response of all bents for the case of the fracture included. However, the model showed poor agreement if fracture is excluded.

Figure 7-37 shows the comparisons between the measured and analytical results for the center-of-mass (COM) displacement for the case of fracture included (see Figure 7-37-a) and the comparisons between the measured and analytical results for the center-of-mass (COM) displacement for the case of fracture excluded (see Figure 7-37-b). The numerical model performed well in matching both the amplitude and period for COM displacement for the case of fracture included. The numerical model did not match the measured response if fracture is excluded.

Motion 19 assessment shows that including fracture in the model can lead to predicting the measured behavior specially the COM displacement.

7.4.1.4 Motion 20B Assessment

Figure 7-38 shows the comparison between the measured and analytical results for the drift ratio of each bent in case of fracture included (black lines for analytical response) and the measured and analytical results for the drift ratio of each bent in case of fracture excluded (blue lines for analytical response). The numerical model performed well in matching both the amplitude and period of the measured drift ratio and base shear with agreement between 90-95% (coefficient of determination $R^2$) with the measured response of all bents for the case of the fracture included. However the model showed no agreement if fracture is excluded.
Figure 7-39 shows the comparisons between the measured and analytical results for the center-of-mass (COM) displacement for the case of fracture included (see Figure 7-39-a) and the comparisons between the measured and analytical results for the center-of-mass (COM) displacement for the case of fracture excluded (see Figure 7-39-b). The numerical model performed well in matching both the amplitude and period for COM displacement for the case of fracture included. The numerical model did not match the measured response if fracture is excluded.

Motion 20B assessment shows that including fracture in the model can lead to predicting the measured behavior, and especially the COM displacement.

### 7.4.2 Accumulative Assessment

Cumulative comparisons were conducted to compare the behavior of different elements in the model with the measured response.

#### 7.4.2.1 Base Shear Assessment

The base shear envelopes for each bent and the bridge total base shear using the calibrated model are compared to the measured response up to Motion 19 as shown in Figure 7-40. There is excellent agreement between the analytical base shear and the measured base shear from shake table actuators for Bents 1 and 3. There is a good comparison between the analytical base shear and measured base shear from inertia forces for Bent 2. The analytical total base shear showed excellent correlation with the measured total base shear from either inertia masses or shake table actuator. The overall agreement between the calibrated bridge base shear and the measured base shear is within 90% (coefficient of determination $R^2$).
7.4.2.2 End Rotation Assessment

Figure 7-41 shows the comparisons of the average connection rotation-drift contribution for all bents. The calibrated model overestimated the column end rotation by 7.5% on average compared to the measured response.

7.4.2.3 Strand Response Assessment

The average strand forces using the calibrated model were compared to the measured response (forces estimated from strain using constitutive laws) for all the columns up to Motion 17, as shown in Figures 7-42 and 7-43. The comparison shows excellent agreement between the analytical and measured average strand force except of the south column of Bent 2. The reason for the lack of agreement between the analytical and measured response for the south column of Bent 2 is that about 12 in. of the strands were bonded through the column which was discovered when the specimen was demolished (see Section 4.3.1.3).

The average strand strain using the calibrated model was compared to the measured response for all the columns up to Motion 17, as shown in Figures 7-44 and 7-45. The comparisons show excellent agreement except for the south column of Bent 2.

7.4.2.4 Fatigue Damage Index Assessment

Figures 7-46 and 7-47 show the cumulative fatigue damage index comparisons for each bar of Bent 1 from Motion 15 through Motion 18. The comparison agreement between the analytical model and the measured data using the low cycle fatigue model was within 85-90%. The calibrated model underestimated the number of fractured bars by four bars (15 bars were fractured compared to 19 bars fractured using the low cycle fatigue model after Motion 18).
Figures 7-48 and 7-49 show the cumulative fatigue damage index comparisons for each bar of Bent 2 from Motion 16 through Motion 19. The comparison agreement between the analytical model and the measured data using the low cycle fatigue model was within 80%. The calibrated model underestimated the number of fractured bars by two bars (eight bars were fractured compared to ten bars fractured using the low cycle fatigue model after Motion 19).

Figures 7-50 and 7-51 show the cumulative fatigue damage index comparisons for each bar of Bent 3 from Motion 15 through Motion 18. The comparison agreement between the analytical model and the measured data using the low cycle fatigue model was within 80%. The calibrated model overestimated the number of fractured bars by four bars (12 bars were fractured compared to eight bars fractured using the low cycle fatigue model after Motion 18).

During Motion 19, both the calibrated model and the measured data from the low cycle fatigue models predicted that all the reinforcing bars at the Bents 1 and 3 interfaces fractured.
Chapter 8. Parametric Studies.

8.1 Introduction

A concern identified in the experiment was the fracture of the reinforcing steel as the magnitude of motions increased. Using the calibrated numerical model, as discussed in Chapter 7, methods for delaying the fracture beyond the Maximum Considered Earthquake (MCE) can be determined through a parametric study. The parametric study for the scaled bridge is divided into three phases: 1) Running individual motions instead of the entire motion protocol used in the experiment in order to exclude the fatigue effect of previous motions. The objective of this phase is to determine the effect of an individual motion on an undamaged bridge, 2) Multiple design level motions (Motion 16 (100% Design Level motion)), one after the other, in order to determine how many design level ground motions cause first fracture, and 3) Changing the bar size and the debonded length in order to delay the fracture to be beyond 1.5 times MCE (Motion 19, 221% Design Level motion).

In order to make the recommendations directly applicable to actual bridges, a prototype model was also used. The prototype studies were conducted using a single bent.

8.2 Parametric Study for Scaled Bridge

8.2.1 Fracture Summary through the Test

As discussed in Chapter 5, four methods of estimating the reinforcing bar fracture were presented: Strain Threshold Method, Low Cycle Fatigue Method, Acoustic Emission Method using audio analysis, and Specimen Demolition. Chapter 7 presents the calibration
of the model using reinforcing steel material incorporating the low cycle fatigue effect. All mentioned methods could predict the fracture after each motion except the Specimen Demolition. Figure 8-1 shows the comparison between methods in terms of the percentage of the total fractured bars by the end of each motion, for Motion 17 to Motion 21C.

8.2.2 Individual Motions

Figure 8-2 to Figure 8-4 show the cumulative fatigue damage index for each bar by running individual Motions 17, 18 and 20B. Figure 8-5 to Figure 8-7 show the numerical drift ratio comparisons between the case of running the experimental protocol and individual motions for Motions 17, 18 and 20B.

By running the individual motions, no bars fractured during Motion 17 with maximum fatigue damage index equal to 48%, as shown in Figure 8-2; this is in comparison to one bar being fractured when the motion was part of the experimental protocol. The numerical drift ratio for each bent when running the individual Motion 17 was identical to the numerical drift ratio by running Motion 17 in the experimental protocol scheme, as shown in Figure 8-5.

Only four bars fractured when running only Motion 18 instead of 27 bars when running Motion 18 in the experimental protocol, as shown in Figure 8-3. A 50% reduction in the maximum drift ratio for Bents 1 and 3 in case of running the individual Motion 18, as shown in Figure 8-6.

No bars fractured when running only Motion 20B instead of 16 bars when running Motion 20B in the experimental protocol, as shown in Figure 8-4. The maximum drift ratio when running the individual motion was 6% instead of 11% as part of the loading protocol.
for Bent 1. A similar or higher reduction is observed in Bents 2 and 3, as shown in Figure 8-7.

Figure 8-8 shows the number of the fractured bars during Motion 17 through Motion 19 when running the individual motions and when running the experimental protocol.

This analysis clearly shows that the multiple runs within the experimental protocol greatly affected the results. Bars still fractured but the number was greatly reduced. The motions added were designed to not increase the demand, but they impacted the strain history and therefore the number of bars that fractured.

### 8.2.3 Running Multiple Design Level Motions

This section examined the impact of multiple design level earthquakes on bar fracture. In the experimental protocol, Motion 16 was set as the design level earthquake. Multiple design level earthquakes (Motion 16) were run. It was determined that three multiple design level earthquakes could be applied without fracture. The maximum value of fatigue damage index was 75%. Figure 8-9 to Figure 8-11 show the cumulative fatigue damage index during the first three successive design level motions for Bents 1, 2, and 3 respectively.

This result, as with Section 8.2.2, shows that the loading protocol used in the actual experiment was quite severe. The design level earthquake was established by the first motion that caused 3% drift for the outer bents in the Conventional Bridge (Motion 16, PGA of 0.75 g).
8.2.4 Fracture Enhancement by Tuning Reinforcing Steel Parameters

As shown in Chapter 2, the bridge specimen was designed to have the same overall geometry and added mass, and approximately the same lateral strength as the Conventional Bridge; therefore the column cross section at the rocking interface included six #3 bars and four 3/8 in. prestressing strands. It was anticipated that increasing the debonded length of the reinforcing bars at the column interface would lead to delaying the fracture, however the increase in the debonded length could also cause increase in bridge flexibility that could lead to extra displacements and extra strains over the lengthened debonded length. To evaluate the impact of increasing the debonded length and the bar size in enhancing the longitudinal reinforcing behavior, case studies were run with different bar sizes and debonded lengths. Table 8-1 shows the parametric study cases.

Figure 8-12 shows the maximum drift ratios and number of fractured bars for all of cases in Table 8-1. The fracture can be delayed by using 30d₃ (30 times the bar diameter of bar size #3) as the debonded length with slight increase in drift ratio in Bent 1 during Motion 18. During Motion 19, using 33d₃ as the debonded length, reduces the number of fractured bars from 12 bars (in case of 24d₃ as the debonded length) to eight bars in Bent 1, and from eight bars (in case of 24d₃ as the debonded length) to five bars in Bent 3.

When using bar size #4, no bar fracture occurred in either Bent 1 or Bent 3 during Motion 18 for any different debonded lengths. During Motion 19, using 22.75d₄ as the debonded length (debonded length of 12.375 in. equivalent to 33d₃) reduces the number of fractured bars from 12 bars (in case of 24d₃ as the debonded length and bar size #3) to one bar in Bent 1, and from eight bars to one in Bent 3. Using a higher bar size by one and increasing the debonded length by 37.5% reduces the number of fractured bar by 90%.
Figure 8-13 shows that no bar fractured during Motion 19 with maximum cumulative fatigue damage index of 82% for the case 17 (Bar size #4 and debonded length of 24d_{#4} (equivalent to 34.6d_{#3})). Figure 8-14 shows numerical drift ratio comparison between case 17 and case 13 (Bar # 4 and debonded length of 16d_{#4} (equivalent to 24d_{#3})). Slight increase in the drift ratio occurred during Motion 19 for case 17.

Increasing the bar size by one size in addition to increasing the debonded length by 44% delayed the fracture beyond 1.5 times MCE (Motion 19); the fractures were delayed with no change in bridge stiffness and 9% increase in bridge base shear, as shown in Figure 8-15.

In Appendix C, Figure C-1 to Figure C-16 show the fatigue damage index for each bar during Motion 18 and 19. Figure C-17 to Figure C-28 show the numerical drift ratio comparison between each case and the reference case, as shown in Table 8-1.

8.3 Parametric Study for Prototype Bents

The purpose of conducting parametric study for prototype bents is to provide an estimation for the optimum debonded length to delay the fracture of the longitudinal reinforcement and optimum effective prestressing to delay the yielding of the prestressing strands during the 150% Design Level Motion. In order to achieve this goal, the parametric study was conducted in four phases.

Phase 1: Analytical studies for different bar sizes for bent with 24-ft. height with different debonded using different multiplications for the Design Level Motion (125%, 150% and 175% Design Level Motion of CCN motion) in order to delay the longitudinal bar fracture.
Phase 2: Using the optimum debonded length for each bar size during the 150% Design Level Motion with different effective prestressing in order to delay the strand yielding during the 150% Design Level Motion.

Phase 3: Using the optimum design from Phase 1 (optimum debonded length) and from Phase 2 (optimum effective prestressing) with different heights from the bents (24 ft., 26 ft., 28 ft., 30 ft. and 32 ft.) during the 150% Design Level Motion.

Phase 4: Examining the 150% Design Level Motion against Federal Highway Administration (FEMA P695/2009) ground motions and against the design response spectrum of multiple cities in the west coast.

The column cross section was scaled to the prototype level using the same steel-to-strand ratio for the tested columns. The strands in the prototype cross section is 36 strands of 0.5-in. diameter, as shown in Figure 8-16. The longitudinal reinforcement for the prototype cross section were 24#6 (as shown in Figure 8-16-a), 18#7 (as shown in Figure 8-6-b) and 14#8 (as shown in Figure 8-16-c). The bent height was 24 ft. that used to perform the analysis in Phases 1 and 2; this height was changed in Phase 3 to study the relationship between debonded length and column height (24 ft. to 32 ft. in intervals of 2 ft.). The prototype bent model used the same modeling assumption as mentioned in Chapter 7.

8.3.1 Phase 1: Delaying Fracture by Iterating on the Debonded Length

Seven different debonded lengths were analyzed for the column cross section with 24#6 longitudinal bars and 70% $f_{py}$ as effective prestressing using the 125%, 150% and 175% Design Level Motion of Century City Motion (CCN), as shown in Table 8-2 (the cases are
described using the percentage of the Design Level Motion). The debonded lengths were from 24d (18 in.) to 48d (36 in.) as intervals of 4d (3 in.). As shown in Table 8-2, during the 125% Design Level Motion, 24d (18 in.) was sufficient to prevent fracture of the 96 bars, as shown in Case 1-125%. During the 150% Design Level Motion, 32d (24 in.) was sufficient to prevent fracture of the 96 bars, as shown in Case 3-150%. During the 175% Design Level Motion, 48d (36 in.) was sufficient to prevent fracture of the 96 bars, as shown in Case 7-175%.

Another seven different debonded lengths were analyzed for the column cross section with 18#7 longitudinal bars and 70% \( f_{py} \) as effective prestressing using the 125%, 150% and 175% Design Level Motion of Century City Motion (CCN), as shown in Table 8-3. The debonded lengths were from 20d (17.5 in.) to 44d (38.5 in.) with 4d (3.5 in.) as the interval. During the 125% Design Level Motion, 20d (17.5 in.) was sufficient to prevent fracture of the 72 bars, as shown in Case 8-125%. During the 150% Design Level Motion, 28d (24.5 in.) was sufficient to prevent fracture of the 72 bars, as shown in Case 10-150%. During the 175% Design Level Motion, 40d (36 in.) was sufficient to prevent fracture of the 72 bars as shown in Case 13-175%.

Another five different debonded lengths were analyzed for the column cross section with 14#8 longitudinal bars and 70% \( f_{py} \) as effective prestressing using the 125%, 150% and 175% Design Level Motion of Century City Motion (CCN), as shown in Table 8-3. The debonded lengths were from 20d (20 in.) to 36d (36 in.) with 4d (4 in.) as iteration step. During the 125% Design Level Motion, 20d (20 in.) was sufficient to prevent fracture of the 54 bars, as shown in Case 15-125%. During the 150% Design Level Motion, 24d (24 in.) was sufficient to prevent fracture of the 54 bars, as shown in Case 16-150%. During
the 175% Design Level Motion, 32d (32 in.) was sufficient to prevent fracture of the 54 bars, as shown in Case 18-175%.

Table 8-2 through Table 8-4 show the number of bar fractured (percentage of fracture), number of yielded tendons (percentage of yielding) and maximum drift ratio for each case. Table 8-5 summarizes the cases where no fracture occurred. Figure 8-17 shows the debonded length as a factor of bar diameter for each bar size during the 125%, 150% and 175% Design Level Motion. It can be noticed that the debonded length required to delay fracture at a certain motion level changes slightly for a given bar size. For example, the debonded length is 20 in. (20d), 24 in. (24d) and 32 in. (32d) for bar size #8 during 125%, 150% and 175% Design Level Motion respectively, while it is 17.5 in. (20d), 24.5 in. (28d) and 36 in. (40d) for bar size #7, and it is 18 in. (24d), 24 in. (32) and 36 in. (48d) for bar size #6 for the same motions.

8.3.2 Phase 2: Delaying Strand Yielding by Iterating on Prestressing

Delaying the yield of the prestressing strands can be done by either increasing the area of strands (using higher number of strands) with same prestressing force or by decreasing the effective prestressing. The tested column utilized around 70% $f_{py}$ as average effective prestressing. Using the recommended debonded length from Phase 1, the effective prestressing was changed to 60% $f_{py}$ and 50% $f_{py}$ while keeping the area of the strand the same.

In case of the bent with 24#6 as the longitudinal reinforcement during the 150% Design Level Motion, it was sufficient to use 32d (24 in.) to prevent the fracture of the reinforcing steel but all the strands yielded. By reducing the effective prestressing to 50%
f_{py}, no strands yielded with maximum strain equal to 0.984 of the yield strain of the prestressing strands whereas one bar fractured due to the increase in bent flexibility, as shown in Table 8-6 (case 20). To prevent both the fracture of the longitudinal bars and the yielding of prestressing strands, additional 4d (3 in.) was added to the previous debonded length (debonded length increased from 32d (24 in.) to 36d (27 in.)) with 50% f_{py} as effective prestressing as shown in Table 8-6 (case 23).

In case of the bent with 18#7 as the longitudinal reinforcement during the 150% Design Level Motion, it was sufficient to use 28d (24.5 in.) to prevent the fracture of the reinforcing steel but all the strands yielded. By reducing the effective prestressing to 50% f_{py}, no strands yielded with maximum strain equal to 0.99 of the yield strain of the prestressing strands, and no bars fractured even after the increase in bent flexibility, as shown in Table 8-7 (case 26).

In case of the bent with 14#8 as longitudinal reinforcement during the 150% Design Level Motion, it was sufficient to use 24d (24 in.) to prevent the fracture of the reinforcing steel but all the strands yielded. By reducing the effective prestressing to 50% f_{py}, no strands yielded with maximum strain equal to 0.99 the yield strain of the prestressing strands, in addition to no bars fractured even after the increase in bent flexibility, as shown in Table 8-8 (case 29).

It is possible to delay of longitudinal reinforcement fracture and the yielding of the prestressing strand by decreasing the prestressing and sometimes increasing the debonded length with 7% decrease in maximum bent base shear.
8.3.3 Phase 3: Examining the Bent Heights

All the parametric studies conducted in Phases 1 and 2 were for a bent with a 24 ft. clear height. The optimum debonded length and effective prestressing from Phase 2 for each bar size were used for analyzing taller bents. The height was investigated from 24 ft. to 32 ft. with 2 ft. intervals. When changing the column height for the bents with 24#6 and 14#8, no change in the debonded length or effective prestressing that was obtained from Phases 1 and 2 was necessary to prevent fracture of the longitudinal bars or yielding of prestressing strands at 150% Design Level Motion, as shown in Table 8-9 and Table 8-11.

Whereas for bent with 18#7, many bars fractured using the debonded length from Phases 1 and 2 (28d (24.5 in)). The debonded length was then increased to 32d (28 in.) and no fractured occurred for all different bent heights, as shown in Table 8-10.

8.3.4 Phase 4: Examining the 150% Design Level Motion

The evaluation of the 150% Design Level Motion that was used in the parametric studies were conducted in two steps: first, by comparing the response spectrum of the 150% Design Level Motion to the response spectra for the ground motions recommended in (FEMA P695/2009) and second, by comparing the response spectrum of the 150% Design Level Motion to different cities in the west coast of the United States.

Figure 8-18 shows that the response spectrum of the 150% Design Level Motion is higher than all the motions mentioned in (FEMA P695/2009).

Figure 8-19 shows the comparison between the used 150% Design Level Motion and the design response spectrum for different cities (Seattle, Portland, San Francisco, Los Angeles and Santa Barbara). The design response spectra for all cites are less than the
response spectrum for the 150% Design Level Motion, whereas for San Francisco and Santa Barbara, the 150% Design Level Motion is less than the 150% design response spectra. The design response spectrum for each city is based on AASHTO 2014 for 2% probability of exceedance in 50 years return period. Therefore, the design motion utilized in this study are representative of a high seismic zone. Since there is only a slight increase in drift for increasing the unbonded length, the recommended unbonded lengths can be used for high, medium or low seismic regions.

8.4 Recommendations for Practice

The parametric studies show that both the fracture of the longitudinal reinforcement and the yielding of prestressing strands can be delayed during the 100%, 125% and 150% Design Level Motion. The parametric studies include three different bar sizes (24#6, 18#7, and 14#8) and 36-0.5” strands using the same steel and strand ratio as the tested columns ($\rho_p = 0.23\%$ and $\rho_s = 0.62\%$, with equivalent reinforcing ratio of 1.56%), as shown in Section 2.3.1. It can be noticed that for different bar sizes, the debonded length required to delay fracture at a certain motion level changes slightly for a given bar size. The recommendation is to use less number of bars therefore larger bar size to reduce the construction hassle during the placement of the bent caps over the column.

Bar size #8 or higher is recommended using 24 times the bar size to delay the fracture during the 150% Design Level Motion and 32 times the bar size to delay the fracture during the 175% Design Level Motion. The effective prestressing can be from 50% to 70% of the strand yield stress if yielding of the strands under 150% Design Level Motion is acceptable.
In order to delay both the fracture of the longitudinal bars and the yielding of prestressing strands during the 150% Design Level Motion, a bar size #8 or larger is recommended using 24 times the bar size to delay the fracture with an effective prestressing equal to 50% of the strand yield stress.
Chapter 9. Summary and Conclusions.

9.1 Summary

A new bridge system has been developed to: 1) reduce onsite construction time by using precast components, 2) reduce major earthquake damage by utilizing column rocking and confinement of the column ends with a steel tube, and 3) maintain the system functionality after a strong earthquake by minimizing residual drift through the use of pretensioning strands in the columns. Furthermore, the system only uses conventional materials. This chapter summarizes the experimental and numerical study findings on the new bridge system.

9.1.1 Experimental Response Summary

The shaking table experiment was conducted at the Network for Earthquake Engineering Simulation (NEES) equipment site at the University of Nevada, Reno (UNR). The bridge had geometry, superimposed masses and member strengths similar to a conventional bridge that was tested at the same site by a UNR-University of Washington team in 2005 (Johnson 2006). The bridge constructed using conventional cast-in-place methods is referred to as the Conventional Bridge (Johnson 2006; Ranf 2007; Johnson et al. 2008). The bridge constructed as part of this dissertation (Thonstad et al. 2016; Mantawy et al. 2016), is referred to as the Resilient Bridge, used a precast, pretensioned rocking system.

The two bridges were subjected to similar loading protocols. In the Conventional Bridge, the motions (up to 221% of Design level) caused so much damage that the masses were removed and the test was stopped shortly thereafter. In the Resilient Bridge, the
damage at similar demands was much lower, and further excitations, using near-fault motions of increasing amplitude, were applied.

The Resilient Bridge was designed to have the same strength as the Conventional Bridge. From estimates of the base shears of the two bridges, the lateral strength of the Resilient Bridge was found to be slightly higher than that of the Conventional Bridge. This difference was due to the limited number of reinforcement sizes available for the 25% scale specimens. The bridges were designed to have the similar strength.

The columns for the two bridges deformed differently. The Conventional Bridge columns experienced distributed curvature, concentrated in the plastic hinge regions, and failed when the concrete spalled, the spiral fractured and the longitudinal bars buckled and fractured. The Resilient Bridge had concentrated rotations at the column ends, the columns rocked as nearly rigid bodies, damage to the concrete was negligible, longitudinal bars fractured at high drift levels, and the bridge re-centered even under the most intense near-field motions (Mantawy et al. 2016).

Several observations can be drawn from the comparison between the Resilient Bridge and the Conventional Bridge:

1. The use of precast components and new connection details in the Resilient Bridge allowed the bents to be constructed in 20% of the field time needed to construct the bents of the Conventional Bridge.

2. The maximum drift ratios of the two bridges were essentially identical through the Design Level Motion. After this motion, the peak(1,4),(996,994)
3. The residual drifts, after the ground motion had stopped, were consistently smaller in the Resilient Bridge than in the Conventional Bridge. Under the excitations common to both bridges, residual drifts in the Resilient Bridge were, in all cases, less than about 0.2%. During the 221% Design Level Motion, the residual drift in the Conventional Bridge reached 0.5%, and extensive damage required removal of the added inertial mass blocks. By the end of testing, the maximum residual drift was 1.7% in Bent 3. By contrast, the residual drift in the Resilient Bridge never exceeded 0.4%, despite it being subjected to an additional series of high-intensity, near-field ground motions that caused peak drifts of more than 12%. The low residual drifts were achieved by the use of locally unbonded pretensioning strands in the columns.

4. Damage to the column concrete was effectively eliminated in the Resilient Bridge. Every column in the Conventional Bridge sustained both extensive cracking and major spalling, which started during the 67% Design Level Motion. By contrast, the only cracks that formed in the Resilient Bridge were during the 221% Design Level motion; they were few in number and never exceeded hairline width. In the Resilient Bridge, the grout at the interface suffered some crushing, but it could be repaired by grout injection without the need to close the bridge.

5. Bar buckling was eliminated in the Resilient Bridge, but bar fracture occurred earlier than in the Conventional Bridge. Bar buckling first occurred in the Conventional Bridge at a drift ratio of 5.5% in Bent 3, but it never occurred in the Resilient Bridge. The steel confining tube was effective at preventing concrete spalling and bar buckling, but this reduction in damage also prevented visual
inspection of the reinforcement and could hinder post-earthquake repair of the columns. The first bar fractures occurred at 7.8% and 5.6% drift ratio in the Conventional and Resilient Bridges respectively.

6. Prestressing strands existed only in the columns of the Resilient Bridge, where they were bonded at the ends and debonded in the clear height of the column. This arrangement allowed them to remain elastic up to a drift ratio of about 3%, first achieved in the Design Level Motion. Even though some of the strands yielded, the system still provided excellent re-centering characteristics throughout the entire test program.

9.1.2 Reinforcing Steel Fracture Summary

During the 133% Design Level motion, the first longitudinal bar fracture sound was heard. Multiple sounds were heard during the following motions. Due to the usage of a confining tube to minimize the damage, the pretensioning strands to provide self-centering capabilities for the column, and debonded sleeves to debond the reinforcing bar at the rocking interface, it was impossible to visually investigate which bar fractured by end of each motion. The strains in the reinforcing bars were monitored using strain gauges; the strain gauges were damaged before first bar fracture. An alternative method of estimating the strains in the reinforcing bars was developed using the displacement transducers at column ends, called the “Displacement Method”.

Using the Displacement Method, the elongation response histories at each bar were calculated and the strain response histories were estimated by assuming that the strains occurred over the debonded length. These strain histories were used to estimate the
fractured bars by two methods: 1) 6% strain threshold, “Strain Threshold Method”, and 2) low cycle fatigue analysis using strain response histories, “Low Cycle Fatigue Method”. To verify the fracture estimation, two approaches were performed, first by audio analysis for the fracture sounds, “Acoustic Emission Method”, and second, by specimen demolition.

Several observations can be drawn from the study of the fracture of the longitudinal bars:

1. The strains in the reinforcement, estimated based on the measured connection rotation, showed good agreement with the measured strain response histories prior to the eventual failure of the strain gauges.
2. The experimental program that was used during the testing caused the accumulation of inelastic strain cycles in the reinforcement which caused them to fatigue and fracture.
3. The bar fractures estimated using the experimental connection rotations and a low cycle fatigue model showed superior agreement with the acoustical estimation and specimen demolition compared with the fractures estimated using a strain threshold approach.

9.1.3 Numerical Studies Summary

A numerical model using OpenSEES software was calibrated to capture the bridge response under the experimental program. The numerical model showed excellent agreement with the measured response during the motions prior to bar fracture and the post-fracture motions. The numerical model was then used to extend the study beyond the experimental program to investigate the sensitivity of bar fracture to the system properties.
The parametric study was divided into three phases: 1) Individual motions only, instead of the experimental protocol in order to exclude the fatigue effect of previous motions. The objective of this phase was to determine which motion causes first fracture, 2) Multiple design level motions, (Motion 16 100% Design Level Motion) to determine how many design level ground motions cause first fracture, and 3) Altering reinforcing parameters by changing the bar size and the debonded region in order to delay the fracture to be beyond 1.5 times the Design Level Motion (Maximum Credible Earthquake (MCE)).

Parametric studies on prototype bents shows that using bar size #8 or larger with debonded length equal to 24 times the bar diameter and 50% of the strand yield stress as effective prestressing prevented the fracture of the longitudinal bars and the yield of prestressing strands during the 150% Design Level Motion.

Several observations can be drawn from the analytical and parametric studies:

1. The analytical model showed excellent agreement with the measured response during the motions prior to fracture. The model which included the “fatigue material” showed superior agreement with the local and global responses of the bridge in the post fracture motions, when compared to the model that neglected fatigue failures

2. The experimental program caused an increase in the accumulation of cycles due to multiple motions that caused bar fracture to occur earlier. When the analytical model was subjected to the motions individually, rather than the loading protocol, no fractures occurred during the 133% Design Level motion, and an 85% and 30% reduction of the number of bar fractures were observed during 177% and 221% Design Level Earthquakes.
3. The new bridge specimen can resist three design level earthquakes without fracture of the longitudinal reinforcement.

4. The number of fractured bars could have been reduced in the experimental tests by increasing the debonded length of the reinforcing without significant changes to the bridge’s elastic stiffness and maximum displacement.

5. The number of fractured bars could have been eliminated in the experimental tests by increasing the size of the bars and increasing the unbonded length.

6. From the prototype analytical study, using bar size #8 or larger with a debonded length equal to 24 times the bar diameter and effective pretensioning equal to 50% of the strand yield stress prevented longitudinal bar fracture and pretensioning strand yielding during the 150% Design Level Motion with the same steel-to-strand ratio as the scaled model.

7. The design motions utilized in parametric study are representative of a high seismic zone. Since there is only a slight increase in drift for increasing the unbonded length, the recommended unbonded lengths can be used for high, medium or low seismic regions.

9.2 Conclusions

1. The precast system enables the bridge bents to be built faster in the field than conventional construction with improved seismic response.

2. The confining steel tubes at the ends of the columns mitigate the damage of concrete and eliminate the buckling of longitudinal reinforcement in the precast
system. In the conventional systems, both concrete crushing and buckling of reinforcement bars occur in the plastic hinge zones.

3. Debonding the longitudinal steel at the rocking interfaces in the column, using 24 times bar diameter was effective to delay the fracture of longitudinal bars during the Design Level Earthquake.

4. The reduction in concrete damage using the confining steel tubes also prevented visual inspection of the reinforcement and could hinder post-earthquake repair of the columns. Therefore an acoustic approach was investigated to identify the number of fractured bars.

5. The experimental program that was used during the testing caused the accumulation of inelastic strain cycles in the reinforcement which caused them to fatigue and fracture. This type of damage occurs in high performance bridge systems where the buckling of the longitudinal bars is inhibited.

6. The partially unbonded, pretensioned strands in the columns are effective to eliminate the residual displacement during the Design Level Earthquake.

7. In hybrid system where both longitudinal reinforcement and prestressing strands are used, the fracture of the longitudinal bars should be included in the numerical models using low cycle fatigue materials (“fatigue material” in OpenSEES) in order to capture the local and global response of the system.

8. The damage in the resilient system can be repaired or controlled in practice. The crushing of the grout at the column-to-bent cap connections can be repaired by injecting a new grout. The bulging of confining tubes can be prevented using thicker tube wall. The fracture of the longitudinal reinforcement can be controlled
by increasing the size of the bars and increasing the debonded length. The yielding of pretensioned strands can be controlled by lowering the prestressing level.

9. In the prototype environment, the resilient system can be designed to perform without any damage to concrete, longitudinal steel and pretensioning strands during motions equivalent to 150% Design Level motion in high seismic zones.

9.3 Future Work

To further clarify the bridge system the following work is recommended.

1. Develop performance-based design fragility curves for the new system with different steel-to-strand ratio.

2. Study the impact of long-duration motion on the fracture of the longitudinal bars.

3. Study the impact of multi-axial ground motions on the behavior of the bridge system including the contribution of the bridge abutments.

4. Develop the Acoustical Estimation approach using the sounds from multiple cameras to define the location of the fractured bar.
References


Thonstad, T., Haraldsson, O., Stanton, J.F, and Eberhard, M.O. (2014) “ABC Bridge Bent that Self-centers after an Earthquake“ National Accelerated Bridge Construction Conference, Miami, United States


Table 2-1: Prototype, the Conventional Bridge and the Resilient Bridge properties

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<tr>
<th>Property</th>
<th>Prototype</th>
<th>Conventional Bridge</th>
<th>Resilient Bridge</th>
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<td>12</td>
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<td>Long. Reinforcement</td>
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<td>16#3</td>
<td>6#3 mild steel 4 strands 3/8</td>
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<td>30</td>
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<td>90 x 14</td>
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<td>Flat Slab</td>
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<td>--------------</td>
<td>-----------------</td>
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<tr>
<td></td>
<td>Top of The column</td>
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<td>Top of The column</td>
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<td>M3 - 3</td>
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<td>Case 1</td>
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<tr>
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<td>Case 3</td>
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## Table 2-3: Instrumentation Summary

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<td><strong>Accelerometers</strong></td>
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<td>Table velocities (T, L)</td>
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<td>Transverse reinforcement strain</td>
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<td>Strand strain</td>
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Table 2-4: Final experimental protocol up to Motion S5

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<td>12</td>
<td></td>
<td>High Level Coherent</td>
<td>10.6%</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
<tr>
<td>13</td>
<td></td>
<td>Motion</td>
<td>20%</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
<tr>
<td>14</td>
<td></td>
<td>0.15g CCN (d)</td>
<td>33%</td>
<td>SQ1414A SQ1414B</td>
<td>SQ1414A SQ1414B</td>
</tr>
<tr>
<td>S1</td>
<td>14-July-2014</td>
<td>Sinusoidal Motion</td>
<td>N.A.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S2</td>
<td></td>
<td>0.05g, 0.25sec</td>
<td>N.A.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S3</td>
<td></td>
<td>0.10g, 0.25sec</td>
<td>N.A.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9C</td>
<td></td>
<td>Biaxial Motion</td>
<td>N.A.</td>
<td>SQ1414C SQ1414D</td>
<td>WN1414C WN1414D</td>
</tr>
<tr>
<td>S4</td>
<td></td>
<td>Sinusoidal Motion</td>
<td>N.A.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.15g, 0.30sec</td>
<td>N.A.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*numbers in brackets are for Shake Table 1, 2 and 3 respectively.
Table 2-5: Final experimental protocol from Motion 14B1 to 21C.

<table>
<thead>
<tr>
<th>Motion</th>
<th>Date</th>
<th>Description</th>
<th>DE</th>
<th>Followed by (Square Wave)</th>
<th>Followed by (White Noise)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S5</td>
<td></td>
<td>Sinusoidal Motion 0.10g, 0.30sec</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
<tr>
<td>14B1</td>
<td></td>
<td>High Level Coherent Motion 0.20g SYL</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
<tr>
<td>14B2</td>
<td></td>
<td>High Level Coherent Motion 0.40g SYL</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
<tr>
<td>14C</td>
<td>14-July-2014</td>
<td>High Level Coherent Motion 0.20g TAK</td>
<td>N.A.</td>
<td>SQ1415</td>
<td>WN1415A</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>High Level Coherent Motion 0.50g CCN (d)</td>
<td>67%</td>
<td>N.A.</td>
<td>WN1516A</td>
</tr>
<tr>
<td>16</td>
<td></td>
<td>High Level Coherent Motion 0.75g CCN (d)</td>
<td>100%</td>
<td>N.A.</td>
<td>WN1617A</td>
</tr>
<tr>
<td>17</td>
<td></td>
<td>High Level Coherent Motion 1.00g CCN (d)</td>
<td>133%</td>
<td>SQ1718A</td>
<td>WN1718A</td>
</tr>
<tr>
<td>18</td>
<td></td>
<td>High Level Coherent Motion 1.33g CCN (d)</td>
<td>177%</td>
<td>SQ1819A</td>
<td>WN1819A</td>
</tr>
<tr>
<td>19</td>
<td></td>
<td>High Level Coherent Motion 1.66g CCN (d)</td>
<td>221%</td>
<td>SQ1920A</td>
<td>WN1920A</td>
</tr>
<tr>
<td>20A</td>
<td>15-July-2014</td>
<td>High Level Coherent Motion 0.75g CCN (d)</td>
<td>100%</td>
<td>SQ2021A</td>
<td>WN2021A</td>
</tr>
<tr>
<td>20B</td>
<td></td>
<td>High Level Coherent Motion 0.843g SYL</td>
<td>N.A.</td>
<td>SQ2121A</td>
<td>WN2121A</td>
</tr>
<tr>
<td>21A</td>
<td></td>
<td>High Level Coherent Motion 0.40g TAK</td>
<td>N.A.</td>
<td>SQ2100A</td>
<td>WN2100A</td>
</tr>
<tr>
<td>21B</td>
<td></td>
<td>High Level Coherent Motion 0.611g TAK</td>
<td>N.A.</td>
<td>SQ2200A</td>
<td>WN2200A</td>
</tr>
<tr>
<td>21C</td>
<td></td>
<td>High Level Coherent Motion 0.80g TAK</td>
<td>N.A.</td>
<td>SQ2300A</td>
<td>WN2300A</td>
</tr>
</tbody>
</table>

Note: 1- The letter A and C followed by square waves and white noise indicates the transverse direction, whereas B and D indicates the longitudinal direction.

2- d: depth of bedrock used in the Conventional Bridge specimen.
**Table 3-1: Laboratory activities during the first through third week**

<table>
<thead>
<tr>
<th>Day</th>
<th>Activities</th>
</tr>
</thead>
<tbody>
<tr>
<td>27-May-2014</td>
<td>Table preparation including shims, foam and threaded rods</td>
</tr>
<tr>
<td></td>
<td>Placement of spacer blocks and Bent 3 on shake table</td>
</tr>
<tr>
<td>28-May-2014</td>
<td>Shoring of Bent 3 for grouting process</td>
</tr>
<tr>
<td>29-May-2014</td>
<td>Shoring of Bent 2</td>
</tr>
<tr>
<td></td>
<td>Fiber grouting of Bent Cap 2 to the columns</td>
</tr>
<tr>
<td></td>
<td>Placement of spacer blocks and Bent 1 on shake table</td>
</tr>
<tr>
<td>30-May-2014</td>
<td>Non-Fiber grouting of Bent Cap 2 to the columns</td>
</tr>
<tr>
<td></td>
<td>Shoring of Bent 1 for grouting process</td>
</tr>
<tr>
<td>02-June-2014</td>
<td>Fiber grouting of Bent 3 bent cap</td>
</tr>
<tr>
<td>03-June-2014</td>
<td>Fiber grouting of Bent 1 bent cap</td>
</tr>
<tr>
<td></td>
<td>Installation of Shake Table 2 Safety Frames.</td>
</tr>
<tr>
<td>04-June-2014</td>
<td>Placement of Bent 2</td>
</tr>
<tr>
<td></td>
<td>Moving superstructure beams inside the lab</td>
</tr>
<tr>
<td></td>
<td>Assembling the west span (between Bent 2 and Bent 3) on the ground</td>
</tr>
<tr>
<td></td>
<td>including foam, backer rods insulations</td>
</tr>
<tr>
<td>05-June-2014</td>
<td>Sealing the gaps underneath the west beams using silicon</td>
</tr>
<tr>
<td></td>
<td>Assembling the east span (between Bent 1 and Bent 2)</td>
</tr>
<tr>
<td>06-June-2014</td>
<td>Filling the joints between both spans by hydrostone</td>
</tr>
<tr>
<td>09-June-2014</td>
<td>Transverse post-tensioning of both east and west spans on the ground</td>
</tr>
<tr>
<td>11-June-2014</td>
<td>Installing the support jacks over the safety frames</td>
</tr>
<tr>
<td></td>
<td>Placing and aligning of both east and west spans over the jacks</td>
</tr>
<tr>
<td>12-June-2014</td>
<td>Placing formwork around footing and spacer blocks for grouting process</td>
</tr>
<tr>
<td></td>
<td>Sealing the gaps between the bent caps and superstructure beams</td>
</tr>
<tr>
<td>13-June-2014</td>
<td>Grouting the spacer blocks to shake tables for Bents 1 and 3</td>
</tr>
<tr>
<td></td>
<td>Grouting the footing of Bents 1 and 3 to spacer blocks</td>
</tr>
<tr>
<td></td>
<td>Grouting the footing of Bent 2 to the shake table</td>
</tr>
</tbody>
</table>
### Table 3-2: Laboratory activities during the fourth through sixth week

<table>
<thead>
<tr>
<th>Day</th>
<th>Activities</th>
</tr>
</thead>
<tbody>
<tr>
<td>16-June-2014</td>
<td>Filling the joints between the bent caps and superstructure beams by hydrostone</td>
</tr>
<tr>
<td>17-June-2014</td>
<td>Attaching strain gauge wires of Bent 3 to strain gauge blocks. Running the cable of strain gauge blocks of Bent 3 to the data acquisition system Starting the attachment of the displacement transducers and accelerometers</td>
</tr>
<tr>
<td>18-June-2014</td>
<td>Attaching strain gauge wires of Bent 1 and 2 to strain gauge blocks. Running the cable of strain gauge blocks of Bent 1 and 2 to the data acquisition system.</td>
</tr>
<tr>
<td>19-June-2014</td>
<td>Tensioning the footing/spacer blocks to the shake table Finishing the attachment of the displacement transducers and accelerometers</td>
</tr>
<tr>
<td>20-June-2014</td>
<td>Diagnostic for instrumentation phase I</td>
</tr>
<tr>
<td>25-June-2014</td>
<td>Weighing and moving the concrete blocks inside the laboratory Installing 11 longitudinal prestressing strands inside the superstructure</td>
</tr>
<tr>
<td>26-June-2014</td>
<td>Placing the concrete blocks over Bent 1 and Bent 3. Post-tensioning phase I. Placing the concrete blocks on the spans Post-tensioning phase II.</td>
</tr>
<tr>
<td>27-June-2014</td>
<td>Non-fiber grouting bent cap of Bents 1 and 3 to the columns</td>
</tr>
<tr>
<td>30-June-2014</td>
<td>Torqueing the concrete blocks to the superstructure Attaching the strands load cell to converter boards Starting the attachment of string pots</td>
</tr>
<tr>
<td>01-July-2014</td>
<td>Placing the steel plates over mid spans Finishing the attachment of string pots.</td>
</tr>
<tr>
<td>02-July-2014</td>
<td>Running the cables to data acquisition boxes</td>
</tr>
<tr>
<td>03-July-2014</td>
<td>Diagnostic for all instruments</td>
</tr>
</tbody>
</table>
### Table 3-3: Concrete compressive strength of bridge components (psi)

<table>
<thead>
<tr>
<th>Component</th>
<th>After 7 days</th>
<th>After 14 days</th>
<th>After 28 days</th>
<th>Test day</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bent 1 columns and Bent Cap 3</td>
<td>7,830</td>
<td>8,888</td>
<td>10,267</td>
<td>10,602</td>
</tr>
<tr>
<td>Bent 2 columns and Bent Cap 1</td>
<td>7,875</td>
<td>8,631</td>
<td>9,418</td>
<td>10,204</td>
</tr>
<tr>
<td>Bent 3 columns and Bent Cap 2</td>
<td>7,066</td>
<td>8,672</td>
<td>9,503</td>
<td></td>
</tr>
<tr>
<td>All Footings</td>
<td>3,592</td>
<td>4,882</td>
<td>5,288</td>
<td>7,578</td>
</tr>
</tbody>
</table>

### Table 3-4: Elastic Modulus of bridge components (ksi)

<table>
<thead>
<tr>
<th>Component</th>
<th>Test day</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bent 1 columns and Bent Cap 3</td>
<td>5,805</td>
</tr>
<tr>
<td>Bent 2 columns and Bent Cap 1</td>
<td>5,962</td>
</tr>
<tr>
<td>Bent 3 columns and Bent Cap 2</td>
<td>5,605</td>
</tr>
</tbody>
</table>

### Table 3-5: Non-Fiber grout compressive strength for Bent 2 (psi)

<table>
<thead>
<tr>
<th>Column</th>
<th>After 4 days</th>
<th>After 10 days</th>
<th>After 27 days</th>
<th>Test day</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bent 2, North column</td>
<td>4,500</td>
<td>6,180</td>
<td>6,070</td>
<td>6,400</td>
</tr>
<tr>
<td>Bent 2, South column</td>
<td>5,910</td>
<td>5,810</td>
<td>6,360</td>
<td>7,830</td>
</tr>
</tbody>
</table>

### Table 3-6: Non-Fiber grout compressive strength for Bent 1 and 3 (psi)

<table>
<thead>
<tr>
<th>Column</th>
<th>After 4 days</th>
<th>Test day</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bent 1, North column</td>
<td>6,410</td>
<td>7,640</td>
</tr>
<tr>
<td>Bent 1, South column</td>
<td>4,900</td>
<td>8,560</td>
</tr>
<tr>
<td>Bent 3, North column</td>
<td>4,840</td>
<td>8,500</td>
</tr>
<tr>
<td>Bent 3, South column</td>
<td>4,840</td>
<td>8,500</td>
</tr>
</tbody>
</table>
**Table 3-7:** Fiber grout compressive strength for Bent 1 (psi)

<table>
<thead>
<tr>
<th>Bent</th>
<th>After 3 days</th>
<th>After 10 days</th>
<th>Test day</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bent 1</td>
<td>6,850</td>
<td>8,180</td>
<td>9,550</td>
</tr>
</tbody>
</table>

**Table 3-8:** Fiber grout compressive strength for Bent 2 and 3 (psi)

<table>
<thead>
<tr>
<th>Bent</th>
<th>After 5 days</th>
<th>After 10 days</th>
<th>Test day</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bent 2</td>
<td>6,880</td>
<td>8,890</td>
<td>9,770</td>
</tr>
<tr>
<td>Bent 3</td>
<td>7,540</td>
<td>7,660</td>
<td>7,800</td>
</tr>
</tbody>
</table>

**Table 3-9:** Steel properties for each reinforcing type for the column

<table>
<thead>
<tr>
<th>Steel</th>
<th>Type</th>
<th>E (ksi)</th>
<th>Fy (ksi)</th>
<th>Fu (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel #3</td>
<td>A706</td>
<td>28,875</td>
<td>70.5</td>
<td>102</td>
</tr>
<tr>
<td>Steel #4</td>
<td>A706</td>
<td>28,675</td>
<td>64.8</td>
<td>88.7</td>
</tr>
<tr>
<td>Strands</td>
<td>A882</td>
<td>30,800</td>
<td>240</td>
<td>268.2</td>
</tr>
<tr>
<td>Spiral</td>
<td>---</td>
<td>29,200</td>
<td>99.4</td>
<td>---</td>
</tr>
</tbody>
</table>
Table 4-1: Damage progression by visual inspection for Bent 1

<table>
<thead>
<tr>
<th>Motion</th>
<th>Bent 1 North column, Top connection</th>
<th>Bent 1 South column, Top connection</th>
</tr>
</thead>
<tbody>
<tr>
<td>14A</td>
<td>First opening in rocking interface, gaps closed after end of the motion</td>
<td>First opening in rocking interface, gaps closed after end of the motion</td>
</tr>
<tr>
<td>14C</td>
<td>-------</td>
<td>First significant cracks at octagonal corners, minor flaking at N-NW and N-NE sides at 4 in. from the confining tube</td>
</tr>
<tr>
<td>15</td>
<td>New cracks at octagonal corners at S-SE and S-SW sides extending 2 in. from the confining tube</td>
<td>New cracks at octagonal corners at N-NW side at 4 in. from the confining tube</td>
</tr>
<tr>
<td>16</td>
<td>Grout pad flaking, Cracks at octagonal corners with minor spalling at E-SE, S-SW sides (crack propagation)</td>
<td>Grout pad flaking, Cracks at octagonal corners with minor spalling at N-NE side (crack propagation)</td>
</tr>
<tr>
<td>18</td>
<td>Cracks at octagonal corners with spalling at S-SE and S-SW sides (crack propagation)</td>
<td>Cracks at octagonal corners with spalling at N-NE side (spalling) and NW side (flaking) at the confining tube</td>
</tr>
<tr>
<td>19</td>
<td>Grout pad crushed about 2-2.5 in. Horizontal cracks at S-SE side at (5-6 in.) from the confining tube all the way across both faces</td>
<td>Grout pad crushed about 2-2.5 in.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Motion</th>
<th>Bent 1 North column, Bottom connection</th>
<th>Bent 1 South column, Bottom connection</th>
</tr>
</thead>
<tbody>
<tr>
<td>14A</td>
<td>First opening in rocking interface, gaps closed after end of the motion</td>
<td>First opening in rocking interface, gaps closed after end of the motion</td>
</tr>
<tr>
<td>14C</td>
<td>First significant cracks at octagonal corners with minor flaking at N-NW and N-NE sides at 4 in. from the confining tube</td>
<td>Hairline circumferential cracks in footing at N side, 1 in. from the confining tube</td>
</tr>
<tr>
<td>15</td>
<td>Increase in crack width at octagonal corners with minor spalling at N-NW and N-NE sides</td>
<td>Increase in crack width at octagonal corners, minor spalling at S-SW side</td>
</tr>
<tr>
<td>16</td>
<td>Cracks at octagonal corners with minor spalling at S-SE side</td>
<td>Cracks at octagonal corners with minor spalling at N-NW side at the confining tube</td>
</tr>
<tr>
<td>17</td>
<td>Cracks at octagonal corners with spalling at N-NE and E-NE sides (crack propagation and spalling) and SW side (flaking) at the confining tube</td>
<td>Cracks at octagonal corners with spalling at N-NW, S-SE and W sides at the confining tube</td>
</tr>
<tr>
<td>18</td>
<td>Confining tube bulge at N side, Spalling in footing around column at N side at 1/2 in. From the confining tube (1/4 in. deep), W side at 2 in. from confining tube and N side at 1 in. from confining tube</td>
<td>Confining tube bulge at N and S sides, Cracks at octagonal corners with spalling at S-SW and SE, Spalling in footing around column at N and NW sides at 1 in. from the confining tube and S side at 3/4 in. from the confining tube</td>
</tr>
<tr>
<td>19</td>
<td>Increase in bulge in the confining tubes at N side, Cracks at octagonal corners with spalling at E-NE side (spalling), S-SW and S-SE sides (crack propagation).</td>
<td>Increase in bulge in confining tubes at N and S sides, Horizontal cracks at S-SE side at 8 in. from the confining tube</td>
</tr>
</tbody>
</table>
### Table 4-2: Damage progression by visual inspection for Bent 2

<table>
<thead>
<tr>
<th>Motion</th>
<th>Bent 2 North column, Top connection</th>
<th>Bent 2 South column, Top connection</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>Cracks at octagonal corners, minor spalling at S-SW side at 2 in. from the confining tube</td>
<td>Cracks at octagonal corners, minor spalling at N-NW side at 3.5 in. from the confining tube and N-NE side at 2 in. from the confining tube</td>
</tr>
<tr>
<td>17</td>
<td>Cracks at octagonal corners with spalling at S-SW side (spalling) and S-SE side, crack at 3 in. from the confining tube</td>
<td>Cracks at octagonal corners with spalling at N-NW and S-SW sides at 2.5 in. from the confining tube</td>
</tr>
<tr>
<td>18</td>
<td>Cracks at octagonal corners with spalling at S-SE side (crack propagation), N-NE side (flaking) and N-NW side (propagates to NW column face)</td>
<td>Cracks at octagonal corners with spalling at S-SE (spalling)</td>
</tr>
<tr>
<td>19</td>
<td>Cracks at octagonal corners with spalling at N-NW side (spalling) and W side at the confining tube (flaking)</td>
<td>------</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Motion</th>
<th>Bent 2 North column, Bottom connection</th>
<th>Bent 2 South column, Bottom connection</th>
</tr>
</thead>
<tbody>
<tr>
<td>9C</td>
<td>Circumferential hairline crack propagation around the confining tube at S-SW side</td>
<td>Circumferential hairline crack propagation around the confining tube at S-SW side</td>
</tr>
<tr>
<td>14B2</td>
<td>------</td>
<td>Circumferential cracks in footing at NE-N-NW sides</td>
</tr>
<tr>
<td>14C</td>
<td>First significant cracks at octagonal corners, minor flaking at S-SE side, 1/2 in. from face of column</td>
<td>------</td>
</tr>
<tr>
<td>16</td>
<td>Cracks at octagonal corners with minor spalling at N-NE and N-NW sides</td>
<td>Cracks at octagonal corners with minor spalling at S-SW side at 3 in. from the confining tube</td>
</tr>
<tr>
<td>17</td>
<td>Cracks at octagonal corners with spalling at N-NE side Cracking in footing around column at E side at 1/2 in. from column face (flaking)</td>
<td>Cracks at octagonal corners with spalling at S-SW, S-SE, N-NW and N sides Cracking in footing around column at SE side with minor flaking</td>
</tr>
<tr>
<td>18</td>
<td>Cracks at octagonal corners with spalling at E-NE side (spalling) and E and S sides (flaking) at the confining tube</td>
<td>Cracks at octagonal corners with spalling at N-NE side (spalling)</td>
</tr>
<tr>
<td>19</td>
<td>Cracks at octagonal corners with spalling at N-NW side (spalling), S-SE side (crack)</td>
<td>Cracks at octagonal corners with spalling at N-NW side (Crack propagation, spalling) and S-SW side (spalling for the corner)</td>
</tr>
</tbody>
</table>
### Table 4-3: Damage progression by visual inspection for Bent 3

<table>
<thead>
<tr>
<th>Motion</th>
<th>Bent 3 North column, Top connection</th>
<th>Bent 3 South column, Top connection</th>
</tr>
</thead>
<tbody>
<tr>
<td>14A</td>
<td>First opening in rocking interface, gap closed after end of the motion</td>
<td>First opening in rocking interface, gap closed after end of the motion</td>
</tr>
<tr>
<td>14C</td>
<td>First significant cracks at octagonal corners, minor flaking at S-SW side</td>
<td>----</td>
</tr>
<tr>
<td>15</td>
<td>New cracks at octagonal corners at S-SW and N sides at the confining tube</td>
<td>New cracks at octagonal corners at N-NE, N-NW, and S-SW sides at the confining tube</td>
</tr>
<tr>
<td>16</td>
<td>Grout pad flaking Cracks at octagonal corners, minor spalling at S-SE side at the confining tube</td>
<td>Grout pad flaking Cracks at octagonal corners, minor spalling at N-NE and N-NW sides (crack propagation) with minor spalling</td>
</tr>
<tr>
<td>17</td>
<td>----</td>
<td>Cracks at octagonal corners with spalling at S-SW side (crack propagation)</td>
</tr>
<tr>
<td>18</td>
<td>Cracks at octagonal corners with spalling at N side (minor crack) and S-SE side (crack propagation) at the confining tube</td>
<td>Cracks at octagonal corners with spalling at S-SW, N-NE and N-NW sides (spalling)</td>
</tr>
<tr>
<td>19</td>
<td>Grout pad crushed about 2-2.5 in.</td>
<td>Grout pad crushed about 2-2.5 in.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Motion</th>
<th>Bent 3 North column, Bottom connection</th>
<th>Bent 3 South column, Bottom connection</th>
</tr>
</thead>
<tbody>
<tr>
<td>14A</td>
<td>First opening in rocking interface, gap closed after end of the motion</td>
<td>First opening in rocking interface, gap closed after end of the motion</td>
</tr>
<tr>
<td>14C</td>
<td>First significant cracks at octagonal corners, minor flaking at N-NW side</td>
<td>----</td>
</tr>
<tr>
<td>15</td>
<td>New cracks at octagonal corners at N-NW side</td>
<td>New cracks at octagonal corners at S-SW, S-SE sides at the confining tube</td>
</tr>
<tr>
<td>17</td>
<td>Cracks at octagonal corners with spalling at S-SE and S-SW sides at the confining tube and N-NW and E sides at 4 in. above the confining tube. Cracking in footing around column at NE-N- NW sides at 3/4 in. from column face</td>
<td>Cracks at octagonal corners with spalling at S-SW side (crack propagation), N-NW side (spalling)</td>
</tr>
<tr>
<td>18</td>
<td>Confining tube bulge at N side. Cracks at octagonal corners with spalling at SW side (hairline cracks)</td>
<td>Confining tube bulge at N side. Cracks at octagonal corners with spalling at N-NW and E sides (spalling)</td>
</tr>
<tr>
<td>19</td>
<td>Increase in bulge in the confining tubes at N side Horizontal cracks at E, E-SE and SE sides at 2.5 in. above the confining tube Spalling in footing around column at S side at 1-1/4 in. from face of column (1/2 in. deep) and N side at 3/4 in. from face of column Spalling in footing around column at S side at N side at 1/2 in. from the confining tube (1/4 in. deep)</td>
<td>Increase in bulge in the confining tubes at N side Cracks at octagonal corners with spalling at NE side (hairline crack) and SW side (minor spalling) Horizontal cracks at N side at 7 in. from the confining tube all the way across face and SW-S-SE side at 6 in. above the confining tube</td>
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Table 4-4: Maximum load cell slippage for Bent 1 strands (kips)

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<th></th>
<th></th>
<th></th>
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<th></th>
<th></th>
<th></th>
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<td>1NP3</td>
<td>1NP4</td>
<td>1SP1</td>
<td>1SP2</td>
<td>1SP3</td>
<td>1SP4</td>
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<td>-0.32</td>
<td>-0.01</td>
<td>0.75</td>
<td>0.62</td>
<td>-0.41</td>
</tr>
<tr>
<td>1B</td>
<td>0.67</td>
<td>0.22</td>
<td>0.08</td>
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<td>-0.01</td>
<td>0.79</td>
<td>0.57</td>
<td>-0.41</td>
</tr>
<tr>
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<td>-0.01</td>
<td>0.79</td>
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<td>9A</td>
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<td>0.08</td>
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<td>0.04</td>
<td>0.79</td>
<td>0.62</td>
<td>-0.36</td>
</tr>
<tr>
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<td>0.03</td>
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<td>0.57</td>
<td>-0.36</td>
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<td>0.08</td>
<td>-0.32</td>
<td>-0.01</td>
<td>0.79</td>
<td>0.62</td>
<td>-0.41</td>
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<td>0.22</td>
<td>0.13</td>
<td>-0.32</td>
<td>-0.01</td>
<td>0.84</td>
<td>0.57</td>
<td>-0.36</td>
</tr>
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<td>-0.32</td>
<td>0.04</td>
<td>0.79</td>
<td>0.62</td>
<td>-0.36</td>
</tr>
<tr>
<td>9C</td>
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<td>0.04</td>
<td>0.66</td>
<td>0.62</td>
<td>-0.26</td>
</tr>
<tr>
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</tr>
<tr>
<td>14B2</td>
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<td>0.13</td>
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<td>-0.26</td>
</tr>
<tr>
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<td>0.13</td>
<td>-0.16</td>
<td>-0.05</td>
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<td>-0.51</td>
</tr>
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</tr>
<tr>
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<td>0.17</td>
<td>0.29</td>
<td>27.70</td>
<td>0.13</td>
<td>0.75</td>
<td>0.57</td>
<td>-0.41</td>
</tr>
<tr>
<td>20A</td>
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<td>0.03</td>
<td>3.35</td>
<td>0.04</td>
<td>0.84</td>
<td>0.73</td>
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<td>0.08</td>
<td>25.95</td>
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<td>0.84</td>
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</tr>
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</table>
Table 4-5: Maximum load cell slippage for Bent 2 strands (kips)

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<th>Load Cell</th>
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<th>South Column</th>
</tr>
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<td>Motion</td>
<td>2NP1</td>
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</tr>
<tr>
<td>1A</td>
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</tr>
<tr>
<td>1B</td>
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<td>0.01</td>
</tr>
<tr>
<td>4</td>
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<td>0.01</td>
</tr>
<tr>
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<tr>
<td>9A</td>
<td>0.31</td>
<td>0.01</td>
</tr>
<tr>
<td>9B</td>
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</tr>
<tr>
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<td>0.06</td>
</tr>
<tr>
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<td>0.06</td>
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<td>0.01</td>
</tr>
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<td>0.01</td>
</tr>
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</tr>
<tr>
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</tr>
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<td>-0.09</td>
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<tr>
<td>19</td>
<td>0.14</td>
<td>0.11</td>
</tr>
<tr>
<td>20A</td>
<td>0.25</td>
<td>-0.04</td>
</tr>
<tr>
<td>20B</td>
<td>0.25</td>
<td>-0.04</td>
</tr>
<tr>
<td>21A</td>
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<td>0.01</td>
</tr>
<tr>
<td>21B</td>
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</tr>
<tr>
<td>21C</td>
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<td>0.01</td>
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Table 4-6: Maximum load cell slippage for Bent 3 strands (kips)

<table>
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</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3NP1</td>
<td>3NP2</td>
</tr>
<tr>
<td>Motion</td>
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<td></td>
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<td>1A</td>
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<td>0.69</td>
</tr>
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<td>1B</td>
<td>0.62</td>
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</tr>
<tr>
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<td>0.62</td>
<td>0.72</td>
</tr>
<tr>
<td>5</td>
<td>0.59</td>
<td>0.72</td>
</tr>
<tr>
<td>6</td>
<td>0.59</td>
<td>0.72</td>
</tr>
<tr>
<td>9A</td>
<td>0.65</td>
<td>0.72</td>
</tr>
<tr>
<td>9B</td>
<td>0.62</td>
<td>0.72</td>
</tr>
<tr>
<td>12</td>
<td>0.62</td>
<td>0.72</td>
</tr>
<tr>
<td>13</td>
<td>0.65</td>
<td>0.72</td>
</tr>
<tr>
<td>14A</td>
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<td>0.72</td>
</tr>
<tr>
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<td>0.56</td>
</tr>
<tr>
<td>14B1</td>
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<td>0.59</td>
</tr>
<tr>
<td>14B2</td>
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<td>0.59</td>
</tr>
<tr>
<td>14C</td>
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</tr>
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<td>0.65</td>
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<tr>
<td>18</td>
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<td>0.62</td>
</tr>
<tr>
<td>19</td>
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<td>0.65</td>
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<tr>
<td>21B</td>
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<td>0.81</td>
</tr>
<tr>
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</table>
Table 4-7: Maximum bent drifts for all motions

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</thead>
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<tr>
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<tr>
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<td>0.07</td>
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<tr>
<td>6</td>
<td>0.08</td>
</tr>
<tr>
<td>9A</td>
<td>0.04</td>
</tr>
<tr>
<td>9B</td>
<td>0.08</td>
</tr>
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</tr>
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<td>9C</td>
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</tr>
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</tr>
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<td>14B2</td>
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Table 4-8: Maximum measured table and bent acceleration

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<th>Max. measured transverse table acceleration (g)</th>
<th>Max. measured transverse bent acceleration (g)</th>
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<td>0.07</td>
<td>0.07</td>
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<td>0.14</td>
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<td>0.09</td>
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<td>0.23</td>
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<td>1.83</td>
</tr>
<tr>
<td>20A</td>
<td>0.75</td>
<td>0.79</td>
<td>0.77</td>
</tr>
<tr>
<td>20B</td>
<td>0.843</td>
<td>0.94</td>
<td>0.92</td>
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<tr>
<td>21A</td>
<td>0.40</td>
<td>0.40</td>
<td>0.39</td>
</tr>
<tr>
<td>21B</td>
<td>0.611</td>
<td>0.61</td>
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</tr>
<tr>
<td>21C</td>
<td>0.80</td>
<td>0.80</td>
<td>0.89</td>
</tr>
</tbody>
</table>
Table 4-9: Period and damping ratio using transfer function from white noise wave

<table>
<thead>
<tr>
<th>White Noise</th>
<th>After Motion</th>
<th>Period (Sec.)</th>
<th>Damping Ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WN0001A75</td>
<td>Pre Test</td>
<td>0.240</td>
<td>5.53</td>
</tr>
<tr>
<td>WN01041A</td>
<td>1B</td>
<td>0.240</td>
<td>5.28</td>
</tr>
<tr>
<td>WN0912A</td>
<td>9B</td>
<td>0.242</td>
<td>4.05</td>
</tr>
<tr>
<td>WN1414A</td>
<td>14A</td>
<td>0.286</td>
<td>8.33</td>
</tr>
<tr>
<td>WN1414C</td>
<td>9C</td>
<td>0.286</td>
<td>9.13</td>
</tr>
<tr>
<td>WN1415</td>
<td>14C</td>
<td>0.288</td>
<td>6.55</td>
</tr>
<tr>
<td>WN1516</td>
<td>15</td>
<td>0.33</td>
<td>8.40</td>
</tr>
<tr>
<td>WN1617</td>
<td>16</td>
<td>0.405</td>
<td>5.68</td>
</tr>
<tr>
<td>WN1718</td>
<td>17</td>
<td>0.581</td>
<td>5.90</td>
</tr>
<tr>
<td>WN1819</td>
<td>18</td>
<td>0.940</td>
<td>6.74</td>
</tr>
<tr>
<td>WN1920</td>
<td>19</td>
<td>1.030</td>
<td>6.91</td>
</tr>
<tr>
<td>WN2021</td>
<td>20A</td>
<td>1.030</td>
<td>7.02</td>
</tr>
<tr>
<td>WN2121</td>
<td>20B</td>
<td>1.190</td>
<td>8.49</td>
</tr>
<tr>
<td>WN2100</td>
<td>21A</td>
<td>1.190</td>
<td>14.65</td>
</tr>
<tr>
<td>WN2200</td>
<td>21B</td>
<td>1.190</td>
<td>7.77</td>
</tr>
<tr>
<td>WN2300</td>
<td>21C</td>
<td>1.333</td>
<td>10.51</td>
</tr>
</tbody>
</table>

Table 4-10: Period and damping ratio calculated at each bent during near field motion using logarithmic decrement method

<table>
<thead>
<tr>
<th>Motion</th>
<th>Quantity</th>
<th>Bent 1</th>
<th>Bent 2</th>
<th>Bent 3</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>20B</td>
<td>Period (Sec.)</td>
<td>1.31</td>
<td>1.33</td>
<td>1.33</td>
<td>1.32</td>
</tr>
<tr>
<td></td>
<td>Damping (%)</td>
<td>8.49</td>
<td>6.14</td>
<td>5.97</td>
<td>6.87</td>
</tr>
<tr>
<td>21C</td>
<td>Period (Sec.)</td>
<td>1.52</td>
<td>1.54</td>
<td>1.31</td>
<td>1.46</td>
</tr>
<tr>
<td></td>
<td>Damping (%)</td>
<td>7.58</td>
<td>9.79</td>
<td>9.26</td>
<td>8.88</td>
</tr>
</tbody>
</table>
Table 4-11: Periods calculated from square waves (Sec.)

<table>
<thead>
<tr>
<th>White Noise</th>
<th>After Motion</th>
<th>Bent 1 1st/2nd Pules</th>
<th>Bent 2 1st/2nd Pules</th>
<th>Bent 3 1st/2nd Pules</th>
<th>Average 1st/2nd Pules</th>
</tr>
</thead>
<tbody>
<tr>
<td>SQ1819</td>
<td>18</td>
<td>0.72/0.70</td>
<td>0.73/N.A.</td>
<td>0.49/0.49</td>
<td>0.65/0.60</td>
</tr>
<tr>
<td>SQ 1920</td>
<td>19</td>
<td>0.82/0.82</td>
<td>0.80/0.82</td>
<td>0.91/N.A.</td>
<td>0.84/0.82</td>
</tr>
<tr>
<td>SQ 2021</td>
<td>20A</td>
<td>0.8/0.83</td>
<td>0.81/0.78</td>
<td>0.85/0.79</td>
<td>0.82/0.80</td>
</tr>
<tr>
<td>SQ 2121</td>
<td>20B</td>
<td>0.89/0.89</td>
<td>0.88/0.84</td>
<td>0.89/0.86</td>
<td>0.89/0.86</td>
</tr>
<tr>
<td>SQ 2100</td>
<td>21A</td>
<td>0.87/0.84</td>
<td>0.86/0.88</td>
<td>0.87/0.84</td>
<td>0.87/0.85</td>
</tr>
<tr>
<td>SQ 2200</td>
<td>21B</td>
<td>0.88/0.87</td>
<td>0.90/0.85</td>
<td>0.93/0.88</td>
<td>0.90/0.87</td>
</tr>
<tr>
<td>SQ 2300</td>
<td>21C</td>
<td>0.99/0.97</td>
<td>0.95/0.94</td>
<td>0.74/0.74</td>
<td>0.89/0.88</td>
</tr>
</tbody>
</table>

Table 4-12: Damping ratio calculated from square waves (%)

<table>
<thead>
<tr>
<th>White Noise</th>
<th>After Motion</th>
<th>Bent 1 1st/2nd Pules</th>
<th>Bent 2 1st/2nd Pules</th>
<th>Bent 3 1st/2nd Pules</th>
<th>Average 1st/2nd Pules</th>
</tr>
</thead>
<tbody>
<tr>
<td>SQ1819</td>
<td>18</td>
<td>9.44/10.37</td>
<td>7.59/N.A.</td>
<td>5.21/7.39</td>
<td>7.41/8.88</td>
</tr>
<tr>
<td>SQ 1920</td>
<td>19</td>
<td>16.60/19.00</td>
<td>10.00/15.62</td>
<td>15.78/N.A.</td>
<td>14.13/17.31</td>
</tr>
<tr>
<td>SQ 2100</td>
<td>21A</td>
<td>13.42/13.14</td>
<td>10.17/10.13</td>
<td>8.19/7.70</td>
<td>10.60/10.32</td>
</tr>
<tr>
<td>SQ 2200</td>
<td>21B</td>
<td>10.27/12.50</td>
<td>9.45/10.42</td>
<td>8.22/7.30</td>
<td>9.31/10.07</td>
</tr>
<tr>
<td>SQ 2300</td>
<td>21C</td>
<td>6.44/6.94</td>
<td>6.07/7.18</td>
<td>9.47/8.29</td>
<td>7.33/7.47</td>
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</table>
Table 5-1: Maximum bar elongation during Motion 17 at column interfaces

<table>
<thead>
<tr>
<th>Location</th>
<th>Location</th>
<th>Max. Bar elongation (in.)</th>
<th>Location</th>
<th>Location</th>
<th>Max. Bar elongation (in.)</th>
<th>Location</th>
<th>Location</th>
<th>Max. Bar elongation (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1NB</strong></td>
<td>N</td>
<td><strong>0.536</strong></td>
<td>NE</td>
<td>NE</td>
<td>0.363</td>
<td>NW</td>
<td>NW</td>
<td>0.501</td>
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<tr>
<td></td>
<td>SE</td>
<td>0.285</td>
<td>SW</td>
<td>SW</td>
<td>0.363</td>
<td>S</td>
<td>S</td>
<td>0.400</td>
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<tr>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>N</td>
<td>0.196</td>
<td>NE</td>
<td>NE</td>
<td>0.252</td>
<td>NW</td>
<td>NW</td>
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<td>S</td>
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<td></td>
</tr>
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<td></td>
<td>N</td>
<td>0.267</td>
<td>NE</td>
<td>NE</td>
<td>0.305</td>
<td>NW</td>
<td>NW</td>
<td>0.444</td>
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<td></td>
<td>SE</td>
<td>0.311</td>
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<td>SW</td>
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<td>S</td>
<td>S</td>
<td>0.447</td>
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<td></td>
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<td>N</td>
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<td>S</td>
<td>0.492</td>
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<td></td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>
Table 5-2: Bar fracture summary for Motion 17 through Motion 21C using the Strain Threshold Method

<table>
<thead>
<tr>
<th>Motion</th>
<th>Bent 1</th>
<th>Bent 2</th>
<th>Bent 3</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1NB</td>
<td>1NT</td>
<td>1SB</td>
<td>1ST</td>
</tr>
<tr>
<td>17</td>
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</tr>
<tr>
<td>18</td>
<td>NW</td>
<td>SE</td>
<td>SW</td>
<td>S</td>
</tr>
<tr>
<td>19</td>
<td>--</td>
<td>NE</td>
<td>SE</td>
<td>--</td>
</tr>
<tr>
<td>20A</td>
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<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>20B</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>21A</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>21B</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>21C</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Fractured By End Of Testing</td>
<td>N NW SE SW</td>
<td>N NW SE SW</td>
<td>N NW SE SW</td>
<td>N NW SE SW</td>
</tr>
<tr>
<td>Survived By End Of Testing</td>
<td>NE -- -- --</td>
<td>NE SE NE SE NE SE</td>
<td>NE -- -- --</td>
<td>NE SE NE SE NE SE</td>
</tr>
</tbody>
</table>
Table 5-3: Bar fracture summary for Motion 17 through Motion 21C using the cumulative fatigue damage index from strain estimation using the Displacement Method

<table>
<thead>
<tr>
<th>Motion</th>
<th>Bent 1</th>
<th>Bent 2</th>
<th>Bent 3</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1NB</td>
<td>1NT</td>
<td>1SB</td>
<td>1ST</td>
</tr>
<tr>
<td>17</td>
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<td>--</td>
<td>--</td>
</tr>
<tr>
<td>18</td>
<td>NE</td>
<td>NW</td>
<td>SE</td>
<td>SW</td>
</tr>
<tr>
<td>19</td>
<td>SE</td>
<td>NE</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>20A</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>20B</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
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<tr>
<td>21A</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
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<tr>
<td>21B</td>
<td>--</td>
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<td>--</td>
<td>--</td>
</tr>
<tr>
<td>21C</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Fractured By End Of Testing</td>
<td>N</td>
<td>N</td>
<td>N</td>
<td>N</td>
</tr>
<tr>
<td>Survived By End Of Testing</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
</tbody>
</table>
Table 5-4: The reinforcing steel fracture summary using maximum strain threshold, low cycle fatigue model and acoustic emission from Motion 17 through the end of testing

<table>
<thead>
<tr>
<th>Motion</th>
<th>Bent 1</th>
<th>Bent 2</th>
<th>Bent 3</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Max. Strain Threshold</td>
<td>Low Cycle Fatigue</td>
<td>Max. Strain Threshold</td>
<td>Low Cycle Fatigue</td>
</tr>
<tr>
<td>17</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>18</td>
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</tr>
<tr>
<td>19</td>
<td>3</td>
<td>4</td>
<td>16</td>
<td>10</td>
</tr>
<tr>
<td>20A</td>
<td>0</td>
<td>0</td>
<td>5</td>
<td>0</td>
</tr>
<tr>
<td>20B</td>
<td>0</td>
<td>0</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>21A</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>21B</td>
<td>0</td>
<td>0</td>
<td>3</td>
<td>0</td>
</tr>
<tr>
<td>21C</td>
<td>0</td>
<td>0</td>
<td>3</td>
<td>0</td>
</tr>
<tr>
<td>End of Test</td>
<td>23</td>
<td>24</td>
<td>16</td>
<td>24</td>
</tr>
</tbody>
</table>
### Table 6-1: Damage progression for both bridges after Motions 14A through 19

<table>
<thead>
<tr>
<th>Motion</th>
<th>Bridge</th>
<th>Damage Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>14A</td>
<td>Conventional</td>
<td>Cracks at all bents (0.003 in. width).</td>
</tr>
<tr>
<td></td>
<td>Resilient</td>
<td>Increase in crack width at octagonal corners with minor spalling at 1NB and SB. New cracks at octagonal corners at 1NB, 1NT, 1ST, 3NB and 3ST</td>
</tr>
<tr>
<td>15</td>
<td>Conventional</td>
<td>Cracks at Bent 1 (0.01 in. max. width) with flaking. Cracks at Bent 2 (0.003 in. max. width). Cracks at Bent 3 (0.01 in. max. width) with 4 in. spalling.</td>
</tr>
<tr>
<td></td>
<td>Resilient</td>
<td>Grout pad flaking at 1NT, 1ST, 3NT and 3ST. Cracks at octagonal corners with minor spalling above all shoes except 3NB and 3SB 6-2-A.</td>
</tr>
<tr>
<td>16</td>
<td>Conventional</td>
<td>Cracks at Bent 1 (0.02 in. max. width) with 5 in. spalling Cracks at Bent 2 (0.007 in. max. width) with flaking Cracks at Bent 3 (0.013 in. max. width) with 4 in. spalling and transverse reinforcement exposed 6-2-a.</td>
</tr>
<tr>
<td></td>
<td>Resilient</td>
<td>Cracks at octagonal corners with spalling above all shoes except 1ST and 1NT 6-2-b. Cracking in footing around column at 2NB, 2SB and 3NB. First longitudinal reinforcing fracture occurred at interface of bottom connection of north column of Bent 1</td>
</tr>
<tr>
<td>17</td>
<td>Conventional</td>
<td>Cracks at Bent 1 (0.02 in. width) with 6 in. spalling and transverse reinforcement exposed. Cracks at Bent 2 (0.01 in. width) with flaking. Cracks at Bent 3 (0.02 in. width) with 6 in. spalling and transverse reinforcement exposed 6-2-b.</td>
</tr>
<tr>
<td></td>
<td>Resilient</td>
<td>Cracks at octagonal corners with spalling above all shoes except 1NB 6-2-e. Spalling in footing around column at 1NB and 1ST. Local buckling of confined rocking detail at north side of 1NB, 3NB and 3SB. Local buckling of confined rocking detail at north and south sides of 1SB. Multiple longitudinal reinforcing fractures.</td>
</tr>
<tr>
<td>18</td>
<td>Conventional</td>
<td>Cracks at Bent 1 (0.02 in. width) with 6 in. spalling and transverse reinforcement exposing 6-2-c. Cracks at Bent 2 (0.02 in. width) with 4 in. spalling. Cracks at Bent 3 (0.08 in. width) with 5.6 in. spalling, transverse reinforcement exposed, longitudinal reinforcing exposing and buckling of longitudinal reinforcing.</td>
</tr>
<tr>
<td></td>
<td>Resilient</td>
<td>Cracks at octagonal corners with spalling at 1NB, 2NB, 2NT, 2SB and 3SB Increase in local buckling of confined rocking detail at north side of 1NB, 3NB and 3SB Increase in local buckling of confined rocking detail at north and south sides of 1ST Grout pad crushed about 2-2.5 in. at 1NT, 1ST, 3NT and 3ST 6-2-d Horizotal cracks at 1NB, 1NT, 1SB, 3NB, 3NB and 3SB 6-2-e. Spalling in footing around column at 1SB, 3NB, 3SB Multiple longitudinal reinforcing fractures</td>
</tr>
<tr>
<td>19</td>
<td>Conventional</td>
<td>Cracks at Bent 1 (0.03 in. width) with 6 in. spalling and transverse reinforcement exposed 6-2-d. Cracks at Bent 2 (0.08 in. width) with 6 in. spalling Cracks at Bent 3 (0.08 in. width) with 6 in. spalling, transverse reinforcement exposing, longitudinal reinforcing exposing, buckling of longitudinal reinforcing and transverse reinforcing fracture 6-2-e.</td>
</tr>
</tbody>
</table>

NB: North column - Bottom connection. 1: Bent 1  
NT: North column - Top connections. 2: Bent 2  
SB: South column - Bottom connections. 3: Bent 3  
ST: South column - Top connections 6-2-A: Damage is shown in Figure 6-2-A
<table>
<thead>
<tr>
<th>Damage State</th>
<th>Bent 1 Resilient Bridge</th>
<th>Bent 1 Conventional Bridge</th>
<th>Bent 2 Resilient Bridge</th>
<th>Bent 2 Conventional Bridge</th>
<th>Bent 3 Resilient Bridge</th>
<th>Bent 3 Conventional Bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexural Cracking</td>
<td>11.8% (19)</td>
<td>2.17% (15)</td>
<td>N.A.</td>
<td>1.30% (15)</td>
<td>13.2% (19)</td>
<td>2.44% (15)</td>
</tr>
<tr>
<td>Cosmetic Spalling</td>
<td>2.1% (15)</td>
<td>3.66% (16)</td>
<td>3.5% (17)</td>
<td>3.51% (18)</td>
<td>4.3% (17)</td>
<td>2.44% (15)</td>
</tr>
<tr>
<td>Grout Pad Crushing</td>
<td>9.2% (18)</td>
<td>N.A.</td>
<td>7.6% (19)</td>
<td>N.A.</td>
<td>6.0% (18)</td>
<td>N.A.</td>
</tr>
<tr>
<td>Long. Bar Exposure</td>
<td>N.A.</td>
<td>3.12% (20)</td>
<td>N.A.</td>
<td>5.46% (22)</td>
<td>N.A.</td>
<td>5.50% (18)</td>
</tr>
<tr>
<td>Long. Bar Yielding</td>
<td>0.47%(14A)</td>
<td>0.88% (13)</td>
<td>0.55%(14A)</td>
<td>0.58% (14A)</td>
<td>0.48%(14A)</td>
<td>0.52% (13)</td>
</tr>
<tr>
<td>Long. Bar Buckling</td>
<td>N.A.</td>
<td>3.12% (20)</td>
<td>N.A.</td>
<td>5.46% (22)</td>
<td>N.A.</td>
<td>7.85% (19)</td>
</tr>
<tr>
<td>Long. Bar Fracture</td>
<td>5.7%(17)</td>
<td>N.A.</td>
<td>7.60% (19)</td>
<td>N.A.</td>
<td>6.0% (18)</td>
<td>7.85% (19)</td>
</tr>
<tr>
<td>Spiral Yielding</td>
<td>N.A.</td>
<td>3.66% (16)</td>
<td>N.A.</td>
<td>3.51% (18)</td>
<td>N.A.</td>
<td>0.52% (13)</td>
</tr>
<tr>
<td>Spiral Fracture</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>7.85% (19)</td>
</tr>
<tr>
<td>Strand Yielding</td>
<td>3.25% (16)</td>
<td>N.A.</td>
<td>4.29% (18)</td>
<td>N.A.</td>
<td>3.13% (16)</td>
<td>N.A.</td>
</tr>
<tr>
<td>Tube Yielding</td>
<td>Not Measured</td>
<td>N.A.</td>
<td>Not Measured</td>
<td>N.A.</td>
<td>3.13% (16)</td>
<td>N.A.</td>
</tr>
<tr>
<td>Tube Bulging</td>
<td>9.2% (18)</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>6.0% (18)</td>
<td>N.A.</td>
</tr>
</tbody>
</table>
**Table 6-3**: Test protocol, maximum bent drifts and peak table acceleration

<table>
<thead>
<tr>
<th>Motion</th>
<th>Motion Type</th>
<th>Description</th>
<th>Peak measured trans. bent drift (%)</th>
<th>Peak measured table accelerations (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Resilient Bridge/Conventional Bridge</td>
<td>Resilient Bridge/Conventional Bridge</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>1A</td>
<td>Low Level Coherent</td>
<td>CCN90 (0.08g PGA)</td>
<td><strong>0.04/0.13</strong></td>
<td><strong>0.03/0.09</strong></td>
</tr>
<tr>
<td>1B</td>
<td></td>
<td>CCN90 (0.15g PGA)</td>
<td><strong>0.07/0.21</strong></td>
<td><strong>0.07/0.14</strong></td>
</tr>
<tr>
<td>4</td>
<td>Low Level incoherent</td>
<td>CCN90 (0.07g-0.18g)</td>
<td><strong>0.07/0.15</strong></td>
<td><strong>0.06/0.11</strong></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>CCN90 (0.18g-0.07g)</td>
<td><strong>0.07/0.17</strong></td>
<td><strong>0.06/0.09</strong></td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>CCN90 (0.18g-0.07g)</td>
<td><strong>0.08/0.17</strong></td>
<td><strong>0.06/0.11</strong></td>
</tr>
<tr>
<td>9A</td>
<td>Biaxial</td>
<td>CCN90/CCN360 (0.08g PGA)</td>
<td><strong>0.04/0.10</strong></td>
<td><strong>0.04/0.04</strong></td>
</tr>
<tr>
<td>9B</td>
<td></td>
<td>CCN90/CCN360 (0.15g PGA)</td>
<td><strong>0.08/0.21</strong></td>
<td><strong>0.07/0.16</strong></td>
</tr>
<tr>
<td>12</td>
<td></td>
<td>CCN90 (0.08g PGA)</td>
<td><strong>0.14/0.32</strong></td>
<td><strong>0.10/0.18</strong></td>
</tr>
<tr>
<td>13</td>
<td></td>
<td>CCN90 (0.15g PGA)</td>
<td><strong>0.41/0.88</strong></td>
<td><strong>0.29/0.45</strong></td>
</tr>
<tr>
<td>14A</td>
<td></td>
<td>CCN90 (0.25g PGA)</td>
<td><strong>1.09/1.05</strong></td>
<td><strong>0.63/0.58</strong></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>CCN90 (0.50g PGA)</td>
<td><strong>2.05/2.17</strong></td>
<td><strong>1.46/1.30</strong></td>
</tr>
<tr>
<td>16</td>
<td></td>
<td>CCN90 (0.75g PGA)</td>
<td><strong>3.28/3.66</strong></td>
<td><strong>2.02/2.39</strong></td>
</tr>
<tr>
<td>17</td>
<td></td>
<td>CCN90 (1.00g PGA)</td>
<td><strong>5.72/2.75</strong></td>
<td><strong>3.49/2.03</strong></td>
</tr>
<tr>
<td>18</td>
<td></td>
<td>CCN90 (1.33g PGA)</td>
<td><strong>9.25/3.86</strong></td>
<td><strong>4.30/3.51</strong></td>
</tr>
<tr>
<td>19</td>
<td></td>
<td>CCN90 (1.66 g PGA)</td>
<td><strong>11.87/4.89</strong></td>
<td><strong>7.62/4.45</strong></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td>CCN90 (1.00g PGA)</td>
<td>N.A./3.12</td>
<td>N.A./3.21</td>
</tr>
<tr>
<td>21*</td>
<td></td>
<td>CCN90 (1.00g PGA)</td>
<td>N.A./4.18</td>
<td>N.A./3.57</td>
</tr>
<tr>
<td>22*</td>
<td></td>
<td>CCN90 (1.50g PGA)</td>
<td>N.A./4.53</td>
<td>N.A./5.46</td>
</tr>
<tr>
<td>20A</td>
<td>Post Peak</td>
<td>CCN90 (0.75g PGA)</td>
<td><strong>6.46/N.A.</strong></td>
<td><strong>5.18/N.A.</strong></td>
</tr>
<tr>
<td>20B</td>
<td></td>
<td>SLY360 (0.843g PGA)</td>
<td><strong>10.83/N.A.</strong></td>
<td><strong>7.89/N.A.</strong></td>
</tr>
<tr>
<td>21A</td>
<td></td>
<td>TAK00 (0.4g PGA)</td>
<td><strong>5.92/N.A.</strong></td>
<td><strong>4.38/N.A.</strong></td>
</tr>
<tr>
<td>21B</td>
<td></td>
<td>TAK00 (0.811g PGA)</td>
<td><strong>8.61/N.A.</strong></td>
<td><strong>5.83/N.A.</strong></td>
</tr>
<tr>
<td>21C</td>
<td></td>
<td>TAK00 (0.8g PGA)</td>
<td><strong>11.99/N.A.</strong></td>
<td><strong>7.72/N.A.</strong></td>
</tr>
</tbody>
</table>
**Table 6-4: Maximum base shear comparison for both the Resilient Bridge and the Conventional Bridge**

<table>
<thead>
<tr>
<th>Motion</th>
<th>Bent 1 (kips)</th>
<th>Bent 2 (kips)</th>
<th>Bent 3 (kips)</th>
<th>Total (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Resilient Bridge</td>
<td>Conventional Bridge</td>
<td>Resilient Bridge</td>
<td>Conventional Bridge</td>
</tr>
<tr>
<td>1A</td>
<td>6</td>
<td>9</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>1B</td>
<td>12</td>
<td>15</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>4</td>
<td>10</td>
<td>17</td>
<td>5</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>12</td>
<td>15</td>
<td>7</td>
<td>4</td>
</tr>
<tr>
<td>6</td>
<td>13</td>
<td>8</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>9A</td>
<td>6</td>
<td>5</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>9B</td>
<td>12</td>
<td>11</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>12</td>
<td>19</td>
<td>18</td>
<td>7</td>
<td>6</td>
</tr>
<tr>
<td>13</td>
<td>35</td>
<td>34</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>14A</td>
<td>59</td>
<td>47</td>
<td>19</td>
<td>13</td>
</tr>
<tr>
<td>15</td>
<td>71</td>
<td>66</td>
<td>25</td>
<td>21</td>
</tr>
<tr>
<td>16</td>
<td>65</td>
<td>64</td>
<td>29</td>
<td>21</td>
</tr>
<tr>
<td>17</td>
<td>70</td>
<td>57</td>
<td>27</td>
<td>17</td>
</tr>
<tr>
<td>18</td>
<td>95</td>
<td>72</td>
<td>29</td>
<td>22</td>
</tr>
<tr>
<td>19</td>
<td>72</td>
<td>67</td>
<td>23</td>
<td>19</td>
</tr>
</tbody>
</table>

**Table 6-5: Period comparison using transfer function for both the Resilient Bridge and the Conventional Bridge**

<table>
<thead>
<tr>
<th>Motion</th>
<th>Resilient Bridge</th>
<th>Conventional Bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before Testing (WN0001)</td>
<td>0.23 Sec.</td>
<td>0.32 Sec.</td>
</tr>
<tr>
<td>After 14 (WN1415)</td>
<td>0.29 Sec.</td>
<td>0.41 Sec.</td>
</tr>
<tr>
<td>After 17 (WN1718)</td>
<td>0.58 Sec.</td>
<td>0.66 Sec.</td>
</tr>
<tr>
<td>After 18 (WN1819)</td>
<td>0.94 Sec.</td>
<td>0.73 Sec.</td>
</tr>
<tr>
<td>After 19 (WN1920)</td>
<td>1.03 Sec.</td>
<td>0.79 Sec.</td>
</tr>
</tbody>
</table>
Table 7-1: Element, section and material level for each structural component

<table>
<thead>
<tr>
<th>Structural Component</th>
<th>Element Level</th>
<th>Section Level</th>
<th>Material Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete inside the confining tube with debonded Steel</td>
<td>Force Beam Element</td>
<td>880 concrete fibers with confined concrete from confining tube</td>
<td>Concrete04, see Table 7-2-b</td>
</tr>
<tr>
<td>Concrete inside the confining tube with bonded Steel</td>
<td>Force Beam Element</td>
<td>880 concrete fibers with confined concrete from confining tube</td>
<td>Concrete04, see Table 7-2-b</td>
</tr>
<tr>
<td>Confined concrete of the column</td>
<td>Force Beam Element</td>
<td>800 concrete fibers with confined concrete from spirals 6 steel fibers</td>
<td>Concrete04, see Table 7-2-c</td>
</tr>
<tr>
<td>Unconfined concrete of the column</td>
<td>Force Beam Element</td>
<td>80 concrete fibers with unconfined concrete</td>
<td>Concrete04, see Table 7-2-a</td>
</tr>
<tr>
<td>Rocking interface</td>
<td>Zero Length Section</td>
<td>880 fibers</td>
<td>EPP Gap Material, see Table 7-5-a</td>
</tr>
<tr>
<td>Debonded steel bar</td>
<td>Truss Element</td>
<td>One Fiber for each bar with 0.20 in² cross section area</td>
<td>Reinforcing Steel Material, see Table 7-3 with Fatigue Material, see Table 7-4</td>
</tr>
<tr>
<td>Bonded steel bar</td>
<td>Truss Element</td>
<td>One Fiber for each bar with 0.20 in² cross section area</td>
<td>Reinforcing Steel Material, see Table 7-3</td>
</tr>
<tr>
<td>Prestressing strands</td>
<td>Co-rotational Truss</td>
<td>One Fiber for each strand with 0.085 in² cross section area</td>
<td>EPP Gap Material, see Table 7-5-b with Initial Strain Material, see Table 7-6</td>
</tr>
</tbody>
</table>
Table 7-2: Concrete04 material properties

<table>
<thead>
<tr>
<th>Concrete04</th>
<th>f_c (ksi)</th>
<th>e_c (strain)</th>
<th>e_cu (strain)</th>
<th>E_c (ksi)</th>
<th>F_t (ksi)</th>
<th>E_t (strain)</th>
</tr>
</thead>
<tbody>
<tr>
<td>a) Unconfined</td>
<td>-10</td>
<td>-0.002</td>
<td>-0.005</td>
<td>5700</td>
<td>0.1</td>
<td>0.0002</td>
</tr>
<tr>
<td>b) Confined with tube</td>
<td>-15.46</td>
<td>-0.0075</td>
<td>-0.044</td>
<td>5700</td>
<td>0.1</td>
<td>0.0002</td>
</tr>
<tr>
<td>c) Confined with spiral</td>
<td>-11.88</td>
<td>-0.0039</td>
<td>-0.0128</td>
<td>5700</td>
<td>0.1</td>
<td>0.0002</td>
</tr>
</tbody>
</table>

Table 7-3: Reinforcing steel material properties

<table>
<thead>
<tr>
<th>Reinforcing Steel</th>
<th>F_y (ksi)</th>
<th>F_u (ksi)</th>
<th>E_s (ksi)</th>
<th>E_sh (ksi)</th>
<th>e_sh (strain)</th>
<th>e_art (strain)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcing bars</td>
<td>70.5</td>
<td>102</td>
<td>28675</td>
<td>290</td>
<td>0.0125</td>
<td>0.1525</td>
</tr>
</tbody>
</table>

Table 7-4: Fatigue material properties

<table>
<thead>
<tr>
<th>Fatigue</th>
<th>E_0 (strain)</th>
<th>m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcing bars</td>
<td>0.0745</td>
<td>-0.448</td>
</tr>
</tbody>
</table>

Table 7-5: Elastic-perfectly plastic gap material properties

<table>
<thead>
<tr>
<th>EPP Gap</th>
<th>E (ksi)</th>
<th>F_y (ksi)</th>
<th>gap</th>
<th>eta</th>
<th>damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>a) Rocking interface</td>
<td>1425</td>
<td>-15.46</td>
<td>0</td>
<td>-0.01</td>
<td>No damage</td>
</tr>
<tr>
<td>b) Prestressing Strands</td>
<td>2850</td>
<td>245</td>
<td>0.041</td>
<td>0</td>
<td>Damage</td>
</tr>
</tbody>
</table>

Table 7-6: Initial strain material properties

<table>
<thead>
<tr>
<th>Initial Strain</th>
<th>Initial Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestressing Strands</td>
<td>0.00545</td>
</tr>
</tbody>
</table>
Table 8-1: Parametric study cases for tuning the reinforcing steel parameters to delay the fracture in the scaled specimen

<table>
<thead>
<tr>
<th>Case</th>
<th>Motion</th>
<th>Reference</th>
<th>Bar Size</th>
<th>Debonded Length (in.)</th>
<th>Debonded Length as function of bar diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>18</td>
<td>Yes</td>
<td>#3</td>
<td>9.00</td>
<td>24d₃ (Equivalent to 16d₄)</td>
</tr>
<tr>
<td>2</td>
<td>18</td>
<td>No</td>
<td>#3</td>
<td>10.125</td>
<td>27d₃ (Equivalent to 18.25d₄)</td>
</tr>
<tr>
<td>3</td>
<td>18</td>
<td>No</td>
<td>#3</td>
<td>11.25</td>
<td>30d₃ (Equivalent to 20.5d₄)</td>
</tr>
<tr>
<td>4</td>
<td>18</td>
<td>No</td>
<td>#3</td>
<td>12.375</td>
<td>33d₃ (Equivalent to 22.75d₄)</td>
</tr>
<tr>
<td>5</td>
<td>18</td>
<td>Yes</td>
<td>#4</td>
<td>9.00</td>
<td>16d₄ (Equivalent to 24d₃)</td>
</tr>
<tr>
<td>6</td>
<td>18</td>
<td>No</td>
<td>#4</td>
<td>10.125</td>
<td>18.25d₄ (Equivalent to 27d₃)</td>
</tr>
<tr>
<td>7</td>
<td>18</td>
<td>No</td>
<td>#4</td>
<td>11.25</td>
<td>20.5d₄ (Equivalent to 30d₃)</td>
</tr>
<tr>
<td>8</td>
<td>18</td>
<td>No</td>
<td>#4</td>
<td>12.375</td>
<td>22.75d₄ (Equivalent to 33d₃)</td>
</tr>
<tr>
<td>9</td>
<td>19</td>
<td>Yes</td>
<td>#3</td>
<td>9.00</td>
<td>24d₃ (Equivalent to 16d₄)</td>
</tr>
<tr>
<td>10</td>
<td>19</td>
<td>No</td>
<td>#3</td>
<td>10.125</td>
<td>27d₃ (Equivalent to 18.25d₄)</td>
</tr>
<tr>
<td>11</td>
<td>19</td>
<td>No</td>
<td>#3</td>
<td>11.25</td>
<td>30d₃ (Equivalent to 20.5d₄)</td>
</tr>
<tr>
<td>12</td>
<td>19</td>
<td>No</td>
<td>#3</td>
<td>12.375</td>
<td>33d₃ (Equivalent to 22.75d₄)</td>
</tr>
<tr>
<td>13</td>
<td>19</td>
<td>Yes</td>
<td>#4</td>
<td>9.00</td>
<td>16d₄ (Equivalent to 24d₃)</td>
</tr>
<tr>
<td>14</td>
<td>19</td>
<td>No</td>
<td>#4</td>
<td>10.125</td>
<td>18.25d₄ (Equivalent to 27d₃)</td>
</tr>
<tr>
<td>15</td>
<td>19</td>
<td>No</td>
<td>#4</td>
<td>11.25</td>
<td>20.5d₄ (Equivalent to 30d₃)</td>
</tr>
<tr>
<td>16</td>
<td>19</td>
<td>No</td>
<td>#4</td>
<td>12.375</td>
<td>22.75d₄ (Equivalent to 33d₃)</td>
</tr>
<tr>
<td>17</td>
<td>19</td>
<td>No</td>
<td>#4</td>
<td>13.00</td>
<td>24d₄ (Equivalent to 34.67d₃)</td>
</tr>
</tbody>
</table>
Table 8-2: Number of fractured bars, number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 24#6, 70% $f_{py}$ effective prestressing and bent height=24ft.

<table>
<thead>
<tr>
<th>Case</th>
<th>Debonded Length</th>
<th>DE</th>
<th># of Fractured bars (Fracture %)</th>
<th># of yielded tendons (Yield %)</th>
<th>Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-125%</td>
<td>24d (18 in.)</td>
<td>125%</td>
<td>0</td>
<td>4 (50%)</td>
<td>3.25</td>
</tr>
<tr>
<td>1-150%</td>
<td>150%</td>
<td>17 (17.7%)</td>
<td>8 (100%)</td>
<td></td>
<td>4.01</td>
</tr>
<tr>
<td>1-175%</td>
<td>175%</td>
<td>Analysis stopped after 14.5 sec</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2-125%</td>
<td>28d (21 in.)</td>
<td>125%</td>
<td>0</td>
<td>4 (50%)</td>
<td>3.33</td>
</tr>
<tr>
<td>2-150%</td>
<td>150%</td>
<td>3 (3.13%)</td>
<td>8 (100%)</td>
<td></td>
<td>3.88</td>
</tr>
<tr>
<td>2-175%</td>
<td>175%</td>
<td>Analysis stopped after 14.5 sec</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3-125%</td>
<td>32d (24 in.)</td>
<td>125%</td>
<td>0</td>
<td>4 (50%)</td>
<td>3.33</td>
</tr>
<tr>
<td>3-150%</td>
<td>150%</td>
<td>0</td>
<td>8 (100%)</td>
<td></td>
<td>3.85</td>
</tr>
<tr>
<td>3-175%</td>
<td>175%</td>
<td>38 (39.6%)</td>
<td>8 (100%)</td>
<td></td>
<td>4.66</td>
</tr>
<tr>
<td>4-125%</td>
<td>36d (27 in.)</td>
<td>125%</td>
<td>0</td>
<td>8 (100%)</td>
<td>3.57</td>
</tr>
<tr>
<td>4-150%</td>
<td>150%</td>
<td>0</td>
<td>8 (100%)</td>
<td></td>
<td>4.01</td>
</tr>
<tr>
<td>4-175%</td>
<td>175%</td>
<td>18 (18.75%)</td>
<td>8 (100%)</td>
<td></td>
<td>4.69</td>
</tr>
<tr>
<td>5-125%</td>
<td>40d (30 in.)</td>
<td>125%</td>
<td>0</td>
<td>8 (100%)</td>
<td>3.63</td>
</tr>
<tr>
<td>5-150%</td>
<td>150%</td>
<td>0</td>
<td>8 (100%)</td>
<td></td>
<td>4.36</td>
</tr>
<tr>
<td>5-175%</td>
<td>175%</td>
<td>7 (7.30%)</td>
<td>8 (100%)</td>
<td></td>
<td>4.66</td>
</tr>
<tr>
<td>6-125%</td>
<td>44d (33 in.)</td>
<td>125%</td>
<td>0</td>
<td>8 (100%)</td>
<td>3.60</td>
</tr>
<tr>
<td>6-150%</td>
<td>150%</td>
<td>0</td>
<td>8 (100%)</td>
<td></td>
<td>4.20</td>
</tr>
<tr>
<td>6-175%</td>
<td>175%</td>
<td>3 (3.13%)</td>
<td>8 (100%)</td>
<td></td>
<td>4.93</td>
</tr>
<tr>
<td>7-125%</td>
<td>48d (36 in.)</td>
<td>125%</td>
<td>0</td>
<td>8 (100%)</td>
<td>3.73</td>
</tr>
<tr>
<td>7-150%</td>
<td>150%</td>
<td>0</td>
<td>8 (100%)</td>
<td></td>
<td>4.05</td>
</tr>
<tr>
<td>7-175%</td>
<td>175%</td>
<td>0</td>
<td>8 (100%)</td>
<td></td>
<td>5.12</td>
</tr>
</tbody>
</table>
Table 8-3: Number of fractured bars, number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 18#7, 70% $f_{py}$ effective prestressing and bent height=24ft.

<table>
<thead>
<tr>
<th>Case</th>
<th>Debonded Length</th>
<th>DE</th>
<th># of Fractured bars (Fracture %)</th>
<th># of yielded tendons (Yield %)</th>
<th>Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8-125%</td>
<td>20d (17.5 in.)</td>
<td>125%</td>
<td>0</td>
<td>8 (100%)</td>
<td>3.6</td>
</tr>
<tr>
<td>8-150%</td>
<td></td>
<td>150%</td>
<td>1 (1.4%)</td>
<td>8 (100%)</td>
<td>3.87</td>
</tr>
<tr>
<td>8-175%</td>
<td></td>
<td>175%</td>
<td>42 (58.8%)</td>
<td>8 (100%)</td>
<td>4.48</td>
</tr>
<tr>
<td>9-125%</td>
<td>24d (21 in.)</td>
<td>125%</td>
<td>0</td>
<td>8 (100%)</td>
<td>3.62</td>
</tr>
<tr>
<td>9-150%</td>
<td></td>
<td>150%</td>
<td>1 (1.4%)</td>
<td>8 (100%)</td>
<td>3.92</td>
</tr>
<tr>
<td>9-175%</td>
<td></td>
<td>175%</td>
<td>42 (58.8%)</td>
<td>8 (100%)</td>
<td>5.24</td>
</tr>
<tr>
<td>10-125%</td>
<td>28d (24.5 in.)</td>
<td>125%</td>
<td>0</td>
<td>4 (50%)</td>
<td>3.3</td>
</tr>
<tr>
<td>10-150%</td>
<td></td>
<td>150%</td>
<td>0</td>
<td>8 (100%)</td>
<td>4.08</td>
</tr>
<tr>
<td>10-175%</td>
<td></td>
<td>175%</td>
<td>21 (29.2%)</td>
<td>8 (100%)</td>
<td>4.72</td>
</tr>
<tr>
<td>11-125%</td>
<td>32d (28 in.)</td>
<td>125%</td>
<td>0</td>
<td>8 (100%)</td>
<td>3.61</td>
</tr>
<tr>
<td>11-150%</td>
<td></td>
<td>150%</td>
<td>0</td>
<td>8 (100%)</td>
<td>4.21</td>
</tr>
<tr>
<td>11-175%</td>
<td></td>
<td>175%</td>
<td>5 (6.95%)</td>
<td>8 (100%)</td>
<td>4.53</td>
</tr>
<tr>
<td>12-125%</td>
<td>36d (31.5 in.)</td>
<td>125%</td>
<td>0</td>
<td>8 (100%)</td>
<td>3.56</td>
</tr>
<tr>
<td>12-150%</td>
<td></td>
<td>150%</td>
<td>0</td>
<td>8 (100%)</td>
<td>4.07</td>
</tr>
<tr>
<td>12-175%</td>
<td></td>
<td>175%</td>
<td>1 (1.40%)</td>
<td>8 (100%)</td>
<td>5.07</td>
</tr>
<tr>
<td>13-125%</td>
<td>40d (36 in.)</td>
<td>125%</td>
<td>0</td>
<td>8 (100%)</td>
<td>3.53</td>
</tr>
<tr>
<td>13-150%</td>
<td></td>
<td>150%</td>
<td>0</td>
<td>8 (100%)</td>
<td>4.13</td>
</tr>
<tr>
<td>13-175%</td>
<td></td>
<td>175%</td>
<td>0</td>
<td>8 (100%)</td>
<td>4.55</td>
</tr>
<tr>
<td>14-125%</td>
<td></td>
<td>125%</td>
<td>0</td>
<td>8 (100%)</td>
<td>3.56</td>
</tr>
<tr>
<td>14-150%</td>
<td></td>
<td>150%</td>
<td>0</td>
<td>8 (100%)</td>
<td>4.22</td>
</tr>
<tr>
<td>14-175%</td>
<td></td>
<td>175%</td>
<td>0</td>
<td>8 (100%)</td>
<td>4.60</td>
</tr>
</tbody>
</table>
Table 8-4: Number of fractured bars, number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 14#8, 70% $f_{py}$ effective prestressing and bent height=24ft.

<table>
<thead>
<tr>
<th>Case</th>
<th>Debonded Length</th>
<th>DE</th>
<th># of Fractured bars (Fracture %)</th>
<th># of yielded tendons (Yield %)</th>
<th>Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15-125%</td>
<td>20d (20 in.)</td>
<td>125%</td>
<td>0</td>
<td>4 (50%)</td>
<td>3.32</td>
</tr>
<tr>
<td>15-150%</td>
<td>20d (20 in.)</td>
<td>150%</td>
<td>1 (1.8%)</td>
<td>8 (100%)</td>
<td>3.80</td>
</tr>
<tr>
<td>15-175%</td>
<td>20d (20 in.)</td>
<td>175%</td>
<td>22 (40%)</td>
<td>8 (100%)</td>
<td>4.20</td>
</tr>
<tr>
<td>16-125%</td>
<td>24d (24 in.)</td>
<td>125%</td>
<td>0</td>
<td>8 (100%)</td>
<td>3.51</td>
</tr>
<tr>
<td>16-150%</td>
<td>24d (24 in.)</td>
<td>150%</td>
<td>0</td>
<td>8 (100%)</td>
<td>4.00</td>
</tr>
<tr>
<td>16-175%</td>
<td>24d (24 in.)</td>
<td>175%</td>
<td>11 (16.9%)</td>
<td>8 (100%)</td>
<td>4.30</td>
</tr>
<tr>
<td>17-125%</td>
<td>28d (28 in.)</td>
<td>125%</td>
<td>0</td>
<td>8 (100%)</td>
<td>3.60</td>
</tr>
<tr>
<td>17-150%</td>
<td>28d (28 in.)</td>
<td>150%</td>
<td>0</td>
<td>8 (100%)</td>
<td>4.40</td>
</tr>
<tr>
<td>17-175%</td>
<td>28d (28 in.)</td>
<td>175%</td>
<td>2 (3.60%)</td>
<td>8 (100%)</td>
<td>4.72</td>
</tr>
<tr>
<td>18-125%</td>
<td>32d (32 in.)</td>
<td>125%</td>
<td>0</td>
<td>8 (100%)</td>
<td>3.66</td>
</tr>
<tr>
<td>18-150%</td>
<td>32d (32 in.)</td>
<td>150%</td>
<td>0</td>
<td>8 (100%)</td>
<td>4.15</td>
</tr>
<tr>
<td>18-175%</td>
<td>32d (32 in.)</td>
<td>175%</td>
<td>0</td>
<td>8 (100%)</td>
<td>4.50</td>
</tr>
<tr>
<td>19-125%</td>
<td>36d (36 in.)</td>
<td>125%</td>
<td>0</td>
<td>8 (100%)</td>
<td>3.69</td>
</tr>
<tr>
<td>19-150%</td>
<td>36d (36 in.)</td>
<td>150%</td>
<td>0</td>
<td>8 (100%)</td>
<td>4.23</td>
</tr>
<tr>
<td>19-175%</td>
<td>36d (36 in.)</td>
<td>175%</td>
<td>0</td>
<td>8 (100%)</td>
<td>4.62</td>
</tr>
</tbody>
</table>

Table 8-5: Cases of no fracture for different bar sizes and design level motions

<table>
<thead>
<tr>
<th>Case</th>
<th>Reinforcement</th>
<th>Debonded Length</th>
<th># of Fractured bars (Fracture %)</th>
<th># of yielded tendons (Yield %)</th>
<th>Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-125%</td>
<td>24#6</td>
<td>24d (18 in.)</td>
<td>0</td>
<td>4 (50%)</td>
<td>3.25</td>
</tr>
<tr>
<td>8-125%</td>
<td>18#7</td>
<td>20d (17.5 in.)</td>
<td>0</td>
<td>8 (100%)</td>
<td>3.6</td>
</tr>
<tr>
<td>15-125%</td>
<td>14#8</td>
<td>20d (20 in.)</td>
<td>0</td>
<td>4 (50%)</td>
<td>3.32</td>
</tr>
<tr>
<td>3-150%</td>
<td>24#6</td>
<td>32d (24 in.)</td>
<td>0</td>
<td>8 (100%)</td>
<td>3.85</td>
</tr>
<tr>
<td>10-150%</td>
<td>18#7</td>
<td>28d (24.5 in.)</td>
<td>0</td>
<td>8 (100%)</td>
<td>4.08</td>
</tr>
<tr>
<td>16-150%</td>
<td>14#8</td>
<td>24d (24 in.)</td>
<td>0</td>
<td>8 (100%)</td>
<td>4.00</td>
</tr>
<tr>
<td>7-175%</td>
<td>24#6</td>
<td>48d (36 in.)</td>
<td>0</td>
<td>8 (100%)</td>
<td>5.12</td>
</tr>
<tr>
<td>13-175%</td>
<td>18#7</td>
<td>40d (36 in.)</td>
<td>0</td>
<td>8 (100%)</td>
<td>4.55</td>
</tr>
<tr>
<td>18-175%</td>
<td>14#8</td>
<td>32d (32 in.)</td>
<td>0</td>
<td>8 (100%)</td>
<td>4.50</td>
</tr>
</tbody>
</table>
Table 8-6: Number of fractured bars, number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 24#6 and bent height=24ft during 150% DE

<table>
<thead>
<tr>
<th>Case</th>
<th>Debonded Length</th>
<th>( f_{pe} )</th>
<th># of Fractured bars (Fracture %)</th>
<th># of yielded tendons (Yield %)</th>
<th>Strain Ductility</th>
<th>Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>32d (24 in.)</td>
<td>50% ( f_{py} )</td>
<td>1 (1.04%)</td>
<td>0</td>
<td>0.984</td>
<td>4.1</td>
</tr>
<tr>
<td>21</td>
<td>60% ( f_{py} )</td>
<td>0</td>
<td>4 (50%)</td>
<td>1.03</td>
<td>3.93</td>
<td></td>
</tr>
<tr>
<td>22</td>
<td>70% ( f_{py} )</td>
<td>0</td>
<td>8 (100%)</td>
<td>1.12</td>
<td>3.85</td>
<td></td>
</tr>
<tr>
<td>23</td>
<td>50% ( f_{py} )</td>
<td>0</td>
<td>0</td>
<td>0.99</td>
<td>4.23</td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>60% ( f_{py} )</td>
<td>0</td>
<td>4 (50%)</td>
<td>1.05</td>
<td>4.06</td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>70% ( f_{py} )</td>
<td>0</td>
<td>8 (100%)</td>
<td>1.13</td>
<td>4.01</td>
<td></td>
</tr>
</tbody>
</table>

Table 8-7: Number of fractured bars, number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 18#7 and bent height=24ft during 150% DE

<table>
<thead>
<tr>
<th>Case</th>
<th>Debonded Length</th>
<th>( f_{pe} )</th>
<th># of Fractured bars (Fracture %)</th>
<th># of yielded tendons (Yield %)</th>
<th>Strain Ductility</th>
<th>Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>26</td>
<td>28d (24.5 in.)</td>
<td>50% ( f_{py} )</td>
<td>0</td>
<td>0</td>
<td>0.99</td>
<td>4.17</td>
</tr>
<tr>
<td>27</td>
<td>60% ( f_{py} )</td>
<td>0</td>
<td>8 (100%)</td>
<td>1.08</td>
<td>4.12</td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>70% ( f_{py} )</td>
<td>0</td>
<td>8 (100%)</td>
<td>1.17</td>
<td>4.08</td>
<td></td>
</tr>
</tbody>
</table>

Table 8-8: Number of fractured bars, number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 14#8 and bent height=24ft during 150% DE

<table>
<thead>
<tr>
<th>Case</th>
<th>Debonded Length</th>
<th>( f_{pe} )</th>
<th># of Fractured bars (Fracture %)</th>
<th># of yielded tendons (Yield %)</th>
<th>Strain Ductility</th>
<th>Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>29</td>
<td>24d (24 in.)</td>
<td>50% ( f_{py} )</td>
<td>0</td>
<td>0</td>
<td>0.99</td>
<td>4.24</td>
</tr>
<tr>
<td>30</td>
<td>60% ( f_{py} )</td>
<td>0</td>
<td>4 (50%)</td>
<td>1.05</td>
<td>4.04</td>
<td></td>
</tr>
<tr>
<td>31</td>
<td>70% ( f_{py} )</td>
<td>0</td>
<td>8 (100%)</td>
<td>1.14</td>
<td>4.00</td>
<td></td>
</tr>
</tbody>
</table>
Table 8-9: Number of fractured bars, Number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 24#6 and 50% $f_{py}$ effective prestressing during 150% DE with different bent heights

<table>
<thead>
<tr>
<th>Case</th>
<th>Debonded Length</th>
<th>Bent Height</th>
<th># of Fractured bars (Fracture %)</th>
<th># of yielded tendons (Yield %)</th>
<th>Strain Ductility</th>
<th>Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>32</td>
<td></td>
<td>24</td>
<td>0</td>
<td>0</td>
<td>0.99</td>
<td>4.23</td>
</tr>
<tr>
<td>33</td>
<td>36d (27 in.)</td>
<td>26</td>
<td>0</td>
<td>0</td>
<td>0.92</td>
<td>4.18</td>
</tr>
<tr>
<td>34</td>
<td></td>
<td>28</td>
<td>0</td>
<td>0</td>
<td>0.97</td>
<td>5.60</td>
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<tr>
<td>35</td>
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<td>30</td>
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<td>0</td>
<td>0.87</td>
<td>5.20</td>
</tr>
<tr>
<td>36</td>
<td></td>
<td>32</td>
<td>0</td>
<td>0</td>
<td>0.86</td>
<td>4.80</td>
</tr>
</tbody>
</table>

Table 8-10: Number of fractured bars, number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 18#7 and 50% $f_{py}$ effective prestressing during 150% DE with different bent heights

<table>
<thead>
<tr>
<th>Case</th>
<th>Debonded Length</th>
<th>Bent Height</th>
<th># of Fractured bars (Fracture %)</th>
<th># of yielded tendons (Yield %)</th>
<th>Strain Ductility</th>
<th>Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>37</td>
<td></td>
<td>24</td>
<td>0</td>
<td>0</td>
<td>0.99</td>
<td>4.17</td>
</tr>
<tr>
<td>38</td>
<td>28d (24.5 in.)</td>
<td>26</td>
<td>6 (8.5%)</td>
<td>4 (50%)</td>
<td>1.05</td>
<td>5.72</td>
</tr>
<tr>
<td>39</td>
<td></td>
<td>28</td>
<td>1 (1.4%)</td>
<td>0</td>
<td>0.99</td>
<td>5.75</td>
</tr>
<tr>
<td>40</td>
<td></td>
<td>30</td>
<td>0</td>
<td>0</td>
<td>0.88</td>
<td>5.11</td>
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<td>41</td>
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<td>0.82</td>
<td>4.59</td>
</tr>
<tr>
<td>42</td>
<td></td>
<td>24</td>
<td>0</td>
<td>0</td>
<td>0.98</td>
<td>4.01</td>
</tr>
<tr>
<td>43</td>
<td></td>
<td>26</td>
<td>0</td>
<td>0</td>
<td>0.99</td>
<td>5.47</td>
</tr>
<tr>
<td>44</td>
<td>32d (28 in.)</td>
<td>28</td>
<td>0</td>
<td>0</td>
<td>0.99</td>
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Table 8-11: Number of fractured bars, number of yielded tendons and drift ratios for the case of longitudinal reinforcement of 14#8 and 50% $f_{py}$ effective prestressing during 150% DE with different bent heights

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<tr>
<th>Case</th>
<th>Debonded Length</th>
<th>Bent Height</th>
<th># of Fractured bars (Fracture %)</th>
<th># of yielded tendons (Yield %)</th>
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Figures
Figure 1-1: Socket connection

Figure 1-2: Socket connection, second specimen by end of testing (Haraldsson et al. 2011)
Figure 1-3: Larger bar grouted duct connection (Pang et al. 2008)

Figure 1-4: Hybrid-bar-socket connection between the prestressed, precast column and precast bent cap (Thonstad et al. 2014)
Figure 1-5: Hybrid-bar-socket connection, by end of testing (Davis et al. 2012)

Figure 1-6: Effective force verses displacement for the pretensioned column with socket connection between the precast column and the cast-in-place footing (Davis et al. 2012)
Figure 1-7: Effective force verses displacement for the pretensioned column with hybrid-bar-socket connection between the precast column and the precast bent cap (Davis et al. 2012)

Figure 1-8: Effective force verses displacement for the pretensioned, rocking column with socket connection between the precast column and the cast-in-place footing (Thonstad et al. 2014)
Figure 1-9: Effective force verses displacement for the pretensioned, rocking column with hybrid-bar-socket connection between the precast column and the precast bent cap (Thonstad et al. 2014)

Figure 1-10: Pretensioned, rocking column with hybrid-bar-socket connection between the precast column and the precast bent cap by end of testing (Thonstad et al. 2014)
**Figure 2-1:** Elevation of the prototype bridge (Ranf 2008)

**Figure 2-2:** Superstructure cross section of the prototype bridge (Ranf 2008)

**Figure 2-3:** Typical reinforcement for the Conventional Bridge columns
Figure 2-4: Moment-rotation and base shear-drift relationship for each column
Figure 2-5: Typical reinforcement for the Resilient Bridge columns
Figure 2-6: Distribution of masses over the Resilient Bridge superstructure
Figure 2-7: Axial stress and the decompression stress at the rocking interface

Figure 2-8: Linear-elastic SAP model for construction sequence design
Figure 2-9: Case 1, Placing the concrete blocks over with timber shoring for the outer bents
**Figure 2-10:** Case 2, post-tensioning the superstructure to 66.67% of the jacking force

**Figure 2-11:** Case 3, placing the first concrete block in each span near the bent caps

**Figure 2-12:** Case 4, placing the concrete second block in each span near the bent caps
Figure 2-13: Case 5, placing the concrete third block in each span near the bent caps

Figure 2-14: Case 6, post-tensioning the superstructure to 100% of the jacking force

Figure 2-15: Case 7, after placing the steel baskets, removing the shoring and grouting the hybrid-bar-socket connection
Figure 2-16: Rendering for the Resilient Bridge after post-tensioning
Figure 2-17: Time-scaled acceleration histories for Motions 14A, 14B, and 14C
Figure 2-18: Scaled spectral accelerations for Motions 14A, 14B2 and 14C using 5% damping ratio

Figure 2-19: Scaled spectral displacements for Motions 14A, 14B2 and 14C using 5% damping ratio
Figure 3-1: Steel cage for Bent 2 column at socket end

Figure 3-2: Prestressing the columns
Figure 3-3: Concrete casting of the column end

Figure 3-4: Bent 3, columns after casting
Figure 3-5: Reduced section duct and reinforcing bar ducts of Bent Cap 2

Figure 3-6: Concrete casting of Bent Cap 2
Figure 3-7: Octagonal shapes at column location to protect the pretension hardware

Figure 3-8: Footing 3 steel cage
Figure 3-9: Lifting the south column of Bent 2

Figure 3-10: Installing the south column of Bent 3
Figure 3-11: Installing the south column of Bent 1

Figure 3-12: Socket connection of the south column of Bent 1 inside the footing over octagonal shaped pocket
Figure 3-13: Bents 2 and 3 after installing the columns with shoring

Figure 3-14: Bent 1 after installing the columns with shoring
Figure 3-15: Bent Cap 2 placement

Figure 3-16: Bent Cap 3 placement
Figure 3-17: Column reduced section and bars inside Bent Cap 2

Figure 3-18: Casting concrete around the south column for Footing 3
**Figure 3-19**: Bent 3 after concrete casting

**Figure 3-20**: Placing the spacer block on spacers and foam
**Figure 3-21:** Setting Bent 3 on shims and foams over spacer block

**Figure 3-22:** Setting Bent 2 on shims and foams over Shake Table 2
Figure 3-23: Bent 1 on shake table with bent cap shoring

Figure 3-24: Alignment of Bent Cap 1 over the columns
Figure 3-25: Adding 4x6 lumber pieces to support the bent cap during the grouting

Figure 3-26: Placement of fiber grout over the top of north column with duct tape confining the grout
Figure 3-27: North column after placing the bent cap over the grout

Figure 3-28: Shoring for Bent 2 before the grout process
Figure 3-29: Lowering and aligning the bent cap over the columns

Figure 3-30: Bent Cap 2 placing over the non-fiber grout
Figure 3-31: Checking the alignment of the Bent Cap 2 after grouting

Figure 3-32: Grouting the first column bar into the bent cap using non fiber grout for Bent 2
Figure 3-33: Grouting the duct between the reduced column section and Bent Cap 2

Figure 3-34: Fully grouting Bent Cap 2 to the columns (30 minutes after the grouting)
Figure 3-35: Aligning the Bent Cap 3 over the columns

Figure 3-36: Fiber grouting of column tops of Bent 3
Figure 3-37: 5x5x1 in. foam pieces used to protect the transverse ducts during pouring the hydrostone

Figure 3-38: Filling the longitudinal joints between the beams by hydrostone
Figure 3-39: Transverse post-tensioning the three beams

Figure 3-40: Beams after completion of transverse post-tensioning of each span
**Figure 3-41:** Lifting the west span

**Figure 3-42:** West span supported by jacks
Figure 3-43: Alignment of the east span

Figure 3-44: U shapes used to protect the duct during the filling process
Figure 3-45: Filling the joints between bent cap and superstructure beams for Bent 1

Figure 3-46: Grouting the Bent 2 footing to Shake Table 2
Figure 3-47: Placement the concrete blocks over the outer bents

Figure 3-48: Removing the jacks at east side of Bent 2
Figure 3-49: Post-tensioning the middle tendon

Figure 3-50: Placement of all concrete blocks after the first stage of post-tensioning
Figure 3-51: Post-tensioning the north tendon after placing all concrete blocks

Figure 3-52: Final bridge condition after post-tensioning
**Figure 3-53:** Grouting the ducts of the north column of Bent 3

**Figure 3-54:** Grouting the ducts of the south column of Bent 1
Figure 3-55: Placement of steel plates at the middle of each span
Figure 4-1: Maximum gap opening for Bent 1, north column, bottom connection, Motion 15 through Motion 21C
Figure 4-2: Maximum gap opening for Bent 1, north column, top connection, Motion 16 through Motion 21C
Figure 4-3: Maximum gap opening for Bent 1, south column, bottom connection, Motion 16 through Motion 20C
Figure 4-4: Maximum gap opening for Bent 1, south column, top connection, Motion 18 through Motion 20C
Figure 4-5: Damage progression for Bent 1, north column, bottom connection, high amplitude motions
Figure 4-6: Damage progression for Bent 1, north column, top connection, high amplitude motions
Figure 4-7: Damage progression for Bent 1, south column, bottom connection, high amplitude motions
Figure 4-8: Damage progression for Bent 1, south column, top connection, high amplitude motions
Figure 4-9: Damage progression for Bent 2, north column, bottom and top connections, high amplitude motions
Figure 4-10: Damage progression for Bent 2, south column, bottom connection, high amplitude motions
Figure 4-11: Damage progression for Bent 2, south column, top connection, high amplitude motions
Figure 4-12: Damage progression for Bent 3, north column, bottom connection, high amplitude motions
**Figure 4-13:** Damage progression for Bent 3, north column, top connection, high amplitude motions
Figure 4-14: Damage progression for Bent 3, south column, bottom connection, high amplitude motions
Figure 4-15: Damage progression for Bent 3, south column, top connection, high amplitude motions
Figure 4.16: Final damage for the north column of Bent 1.
Figure 4-17: Final damage for the south column of Bent 1
Figure 4-18: Final damage for the north column of Bent 2
Figure 4.19: Final damage for the south column of Bent 2

North Side

West Side

South Side

East Side
Figure 4-20: Final damage for the north column of Bent 3
Figure 4.21: Final damage for the south column of Bent 3
Figure 4-22: Average strain ductility envelopes at the interface for each bent
Figure 4-23: Longitudinal reinforcement strain ductility envelopes(158,95),(843,704) for the north bar of Bent 3 at bottom connection of the north column

Figure 4-24: Tensile strain distribution at top connection for the south bar in Bent 1, north column for given drift ratios
Figure 4-25: Average strand strain envelopes for each column without offset
Figure 4.26: Average strain envelopes for each column.
Figure 4-27: Bonded strands inside the south column of Bent 2 at 8 in. from the column interface after the bent demolishing
Figure 4.28: Average end rotation for each bent during Motion 14A
Figure 4.29: Average end rotation for each bent during Motion 18.

(a) Bent 1

(b) Bent 2

(c) Bent 3
Figure 4-30: Average connection rotation-drift contribution for Bents 1 and 3 by the end of Motion 19
Figure 4.31: Motion 14A, drift and acceleration response histories for each bent
Figure 4-32: Motion 14A, transverse deflection plot of superstructure at instant of peak and valley deflections for each bent.
Figure 4-33: Motion 16, drift and acceleration response histories for each bent
Figure 4-34: Motion 16, transverse deflection plot of superstructure at instant of peak and valley deflections for each bent.
Figure 4-35: Motion 19, drift and acceleration response histories for each bent
Figure 4-36: Motion 19, transverse deflection plot of superstructure at instant of peak and valley deflections for each bent
Figure 4-37: Motion 20B, drift and acceleration response histories for each bent
Figure 4-38: Motion 20B, transverse deflection plot of superstructure at instant of peak and valley deflections for each bent.
Figure 4-39: Motion 21C, drift and acceleration response histories for each bent.
Figure 4-40: Motion 21C, transverse deflection plot of superstructure at instant of peak and valley deflections for each bent
Figure 4-41: Maximum drift and residual drift ratios for each bent from Motion 14A to the end of testing
Figure 4-42: Motion 14A, vertical deflection plot of each bent cap at instant of bent peak and valley deflections.
Figure 4-43: Motion 16, vertical deflection plot of each bent cap at instant of bent peak and valley deflections
Figure 4.44: Motion 19, vertical deflection plot of each bent cap at instant of bent peak and valley deflections
Figure 4-45: Motion 20B, vertical deflection plot of each bent cap at instant of bent peak and valley deflections
Figure 4-46: Motion 21C, vertical deflection plot of each bent cap at instant of bent peak and valley deflections.
Figure 4-47: Measured base shear for each bent and total measured base shear for the bridge during Motion 14A
Figure 4-48: Measured base shear for each bent and total measured base shear for the bridge during Motion 14B1
Figure 4-49: Measured base shear for each bent and total measured base shear for the bridge during Motion 14C
Figure 4.50: Measured base shear for each bent and total measured base shear for the bridge during Motion 15.
Figure 4-51: Measured base shear for each bent and total measured base shear for the bridge during Motion 16
Figure 4-52: Measured base shear for each bent and total measured base shear for the bridge during Motion 17
Figure 4.53: Measured base shear for each bent and total measured base shear for the bridge during Motion 18.
Figure 4-54: Measured base shear for each bent and total measured base shear for the bridge during Motion 19.
Figure 4-55: Base shear envelopes measured from accelerometers and inertia masses
Figure 4.56: Response spectra for transverse table acceleration before testing, and after Motions 14C, 16, and 17.
Figure 4-57: Response spectra for transverse table acceleration after Motions 18, 19, and 20B, and at the end of testing
Figure 4-58: FFT plots for each bent before testing, after Motions 14C, 16, and 17.
Figure 4-59: FFT plots for each bent after Motions 18, 19, and 20B, and by the end of testing.
Figure 4-6b: Transfer function plots before testing, and after Motions 14C, 16, and 17.
Figure 4-61: Transfer function plots after Motions 18, 19, and 20B, and by the end of testing
Figure 4-62: Bridge period and damping ratio from transfer function after white noise waves
Figure 4-63: Comparison between table and bent displacement during Motion 20B
Figure 4-64: Damping and period estimation using logarithmic decrement for free vibration part of Motion 20B
Figure 4-65: Comparison between table and bent displacement during Motion 21C
Figure 4-66: Damping and period estimation using logarithmic decrement for free vibration part of Motion 21C
Figure 4-67: Damping and period estimation using logarithmic decrement for free vibration during square wave after Motion 21A.
Figure 4-68: Comparison between the measured displacement and the calculated displacement from accelerometers for each bent during Motion 14A
Figure 4-69: Comparison between the measured displacement and the calculated displacement from accelerometers for each bent during Motion 16
Figure 4.70: Comparison between the measured displacement and the calculated displacement from accelerometers for each bent during Motion 19.
Figure 4-71: Comparison between the measured displacement and the calculated displacement from accelerometers for each bent during Motion 20B.
Figure 4.72: Comparison between the measured displacement and the calculated displacement from accelerometers for each bent during Motion 21C.
Figure 5-1: Rotation of column at north-south direction (transverse direction)

Figure 5-2: Rotation of column at east-west direction (longitudinal direction)
Figure 5-3: Connection geometry during gap opening at north-east components used to perform the equations of the Displacement Method.
Figure 5-4: Displacement Method and strain gauge reading comparison for bottom connection of north column of Bent 1 (1NB) during Motion 15

Figure 5-5: Displacement Method and strain gauge reading comparison for top connection of north column of Bent 1 (1NT) during Motion 15
Figure 5-6: Displacement Method and strain gauge reading comparison for bottom connection of south column of Bent 1 (1SB) during Motion 15

Figure 5-7: Displacement Method and strain gauge reading comparison for top connection of south column of Bent 1 (1ST) during Motion 15
Figure 5-8: Displacement Method and strain gauge reading comparison for bottom connection of north column of Bent 3 (3NB) during Motion 15

Figure 5-9: Displacement Method and strain gauge reading comparison for top connection of north column of Bent 3 (3NT) during Motion 15
Figure 5-10: Displacement Method and strain gauge reading comparison connection of south column of Bent 3 (3SB) during Motion 15

Figure 5-11: Displacement Method and strain gauge reading comparison for top connection of south column of Bent 3 (3ST) during Motion 15
Figure 5-12: Displacement Method and strain gauge maximum strain comparison for north and south bars of Bents 1 and 3 during Motion 15

Figure 5-13: Strains using Displacement Method for 1NB during Motion 17
Figure 5-14: Flow chart for the Low Cycle Fatigue Method for each bar
Figure 5-15: Cumulative fatigue damage index for all longitudinal bars after Motion 15
Figure 5.16: Cumulative fatigue damage index for all longitudinal bars after Motion 16.
Figure 5-17: Cumulative fatigue damage index for all longitudinal bars after Motion 17
Figure 5-18: Cumulative fatigue damage index for all longitudinal bars after Motion 18
Figure 5-19: Cumulative fatigue damage index for all longitudinal bars after Motion 19
Figure 5-20: Cumulative fatigue damage index for all longitudinal bars of Bent 1 after Motion 20A
Figure 5-21: Cumulative fatigue damage index for all longitudinal bars of Bent 1 after Motion 20B
Figure 5-22: Cumulative fatigue damage index for all longitudinal bars of Bent 1 after Motion 21A
Figure 5-23: Cumulative fatigue damage index for all longitudinal bars of Bent 1 after Motion 21B.
Figure 5-24: Cumulative fatigue damage index for all longitudinal bars of Bent 1 after Motion 21C
Figure 5-25: Cumulative fatigue damage index for the bottom connection bars of north column of Bent 1

Figure 5-26: Cumulative fatigue damage index for the top connection bars of north column of Bent 1
Figure 5-27: Cumulative fatigue damage index for the bottom connection bars of south column of Bent 1

Figure 5-28: Cumulative fatigue damage index for the top connection bars of south column of Bent 1
Figure 5-29: Cumulative fatigue damage index for the bottom connection bars of north column of Bent 2

Figure 5-30: Cumulative fatigue damage index for the top connection bars of north column of Bent 2
Figure 5-31: Cumulative fatigue damage index for the bottom connection bars of south column of Bent 2

Figure 5-32: Cumulative fatigue damage index for the top connection bars of south column of Bent 2
Figure 5-33: Cumulative fatigue damage index for the bottom connection bars of north column of Bent 3

Figure 5-34: Cumulative fatigue damage index for the top connection bars of north column of Bent 3
Figure 5-35: Cumulative fatigue damage index for the bottom connection bars of south column of Bent 3

Figure 5-36: Cumulative fatigue damage index for the top connection bars of south column of Bent 3
**Figure 5-37:** Filtered sound using 1NB GoPro slow motion during Motion 17

**Figure 5-38:** Filtered sound using 1NB GoPro slow motion during Motion 18
Figure 5-39: Filtered sound using 3SB GoPro slow motion during Motion 19
Figure 5-40: Connections after demolishing the bent
Figure 6-1: Bent drift comparison using 2-DOF model
Figure 6-2: Damage progression comparison during Motions 16 through 19 (Resilient Bridge on left and Conventional Bridge on right).
Figure 6.3: Bent drifts and bridge COM displacement comparison during Motion 14A.
Figure 6-4: Bent base shear and total base shear comparison during Motion 14A.
Figure 6.5: Bent drifts and bridge COM displacement comparison during Motion 15.
Figure 6.6: Bent base shear and total base shear comparison during Motion 15.
Figure 6-7: Bent drift and Bridge COM displacement comparison during Motion 16.
Figure 6-8: Bent base shear and total base shear comparison during Motion 16.
Figure 6-9: Bent Drifts and COM displacement comparison during Motion 17.
Figure 6-10: Bent Base shear and total base shear comparison during Motion 17.
Figure 6-11: Bent Drifts and COM displacement comparison during Motion 18.
Figure 6-12: Bent Base shear and total base shear comparison during Motion 18.
Figure 6.13: Bent Drifts and COM displacement comparison during Motion 19.
Figure 6.14: Bent Base shear and total base shear comparison during Motion 19.
Figure 6-15: Residual drifts comparison during Motion 19.
Figure 6-16: Base shear vs COM displacement for both the Resilient Bridge and the Conventional Bridge
Figure 6-17: Transfer function comparison for both bridges before testing and after Motions 14A and 17.
**Figure 6-18:** Transfer function comparison for both bridges after Motions 18 and 19.

**Figure 6-19:** Change in fundamental period through the test for both the Resilient Bridge and the Conventional Bridge.
Figure 6-20: Average drift-rotation comparison for Bent 1 during Motions 14A and 18.
Figure 6-21: Average drift-rotation comparison for Bent 2 during Motions 14A and 18.
Figure 6-22: Average drift-rotation comparison for Bent 3 during Motions 14A and 18.
Figure 6-23: Maximum drift ratio vs. corresponding average rotation for Bent 1 in both the Resilient Bridge and the Conventional Bridge.

Figure 6-24: Maximum drift ratio vs. corresponding average rotation for Bent 2 in both the Resilient Bridge and the Conventional Bridge.
Figure 6-25: Maximum drift ratio vs. corresponding average rotation for Bent 3 in both the Resilient Bridge and the Conventional Bridge.

Figure 6-26: Average connection rotation-drift contribution comparison for Bent 1 in both the Resilient Bridge and the Conventional Bridge.
Figure 6-27: Average connection rotation-drift contribution comparison for Bent 3 in both the Resilient Bridge and the Conventional Bridge.

Figure 6-28: Tensile strain distribution comparison at the top connection for the south bar of Bent 3 north column of the Resilient Bridge (1NT) and the East bar of Bent 1 West column of the Conventional Bridge (1WT) for given drift ratios.
Figure 6-29: Longitudinal reinforcement strain envelopes comparison at the both connection for the north bar of Bent 3 north column of the Resilient Bridge (3NB) and the West bar of Bent 3 West column of the Conventional Bridge (3WB).
Figure 6-30: Onsite bent construction time for both the Resilient Bridge and the Conventional Bridge.
Figure 7-1: Three dimensional sketch for the numerical model of the Resilient Bridge
Figure 7-2: Three-dimensional sketch for the rocking connection at the top of the column
Figure 7-3: Detailed sketch for the rocking column modeling
Figure 7-4: Section discretization

Figure 7-5: Depth of neutral axis as a function of the strain in the longitudinal reinforcement (Ranf 2007)
Figure 7-6: Drift ratios and base shear for each bent during Motion 14A

Figure 7-7: C.O.M. displacement and twisting displacement for the bridge during Motion 14A
Figure 7-8: Total base shear for the bridge during Motion 14A

Figure 7-9: Drift ratio vs. base shear for each bent during Motion 14A
Figure 7-10: Drift ratios and base shear for each bent during Motion 14B1

Figure 7-11: C.O.M. displacement and twisting displacement for the bridge during Motion 14B1
Figure 7-12: Total base shear for the bridge during Motion 14B1

Figure 7-13: Drift ratio vs. base shear for each bent during Motion 14B1
Figure 7-14: Drift ratios and base shear for each bent during Motion 14B2

Figure 7-15: C.O.M. displacement and twisting displacement for the bridge during Motion 14B2
**Figure 7-16**: Total base shear for the bridge during Motion 14B2

**Figure 7-17**: Drift ratio vs. base shear for each bent during Motion 14B2
Figure 7-18: Drift ratios and base shear for each bent during Motion 14C

Figure 7-19: C.O.M. displacement and twisting displacement for the bridge during Motion 14C
Figure 7-20: Total base shear for the bridge during Motion 14C

Figure 7-21: Drift ratio vs. base shear for each bent during Motion 14C
Figure 7-22: Drift ratios and base shear for each bent during Motion 15

Figure 7-23: C.O.M. displacement and twisting displacement for the bridge during Motion 15
Figure 7-24: Total base shear for the bridge during Motion 15

Figure 7-25: Drift ratio vs. base shear for each bent during Motion 15
Figure 7-26: Drift ratios and base shear for each bent during Motion 16

Figure 7-27: C.O.M. displacement and twisting displacement for the bridge during Motion 16
Figure 7-28: Total base shear for the bridge during Motion 16

Figure 7-29: Drift ratio vs. base shear for each bent during Motion 16
Figure 7-30: Base shear envelopes for each bent and total base shear for the bridge up to fracture

Figure 7-31: Strain ductility envelopes for each bent
Figure 7-32: Drift ratios for each bent during Motion 17

Figure 7-33: C.O.M. displacement for the bridge during Motion 17
a) fracture included  

b) fracture excluded

**Figure 7-34:** Drift ratios for each bent during Motion 18

a) fracture included  

b) fracture excluded

**Figure 7-35:** C.O.M. displacement for the bridge during Motion 18
Figure 7-36: Drift ratios for each bent during Motion 19

Figure 7-37: C.O.M. displacement for the bridge during Motion 19
Figure 7-38: Drift ratios for each bent during Motion 20B

Figure 7-39: C.O.M. displacement for the bridge during Motion 20B
Figure 7-40: Base shear envelopes for each bent and total base shear for the bridge up to Motion 19

Figure 7-41: Average connection rotation drift contribution for each bent
Figure 7-42: Average strand force envelopes for each column

Figure 7-43: Average strand force envelopes for each column with no offset
Figure 7-44: Average strand strain envelopes for each column

Figure 7-45: Average strand strain envelopes for each column with no offset
Figure 7-46: Cumulative fatigue damage index for Bent 1 from Motions 15 and 16
Figure 7-47: Cumulative fatigue damage index for Bent 1 from Motions 17 and 18
Figure 7-48: Cumulative fatigue damage index for Bent 2 from Motions 16 and 17
Figure 7-49: Cumulative fatigue damage index for Bent 2 from Motions 18 and 19
Figure 7-50: Cumulative fatigue damage index for Bent 2 from Motions 15 and 16
Figure 7-51: Cumulative fatigue damage index for Bent 3 from Motions 17 and 18
Figure 8-1: Percentage of fractured bars using Low Cycle Fatigue, Strain Threshold, Analytical Model, and Acoustic Emissions
Figure 8-2: Cumulative fatigue damage index for each bar after only Motion 17
Figure 8-3: Cumulative fatigue damage index for each bar after only Motion 18
**Figure 8-4**: Cumulative fatigue damage index for each bar after only Motion 20B
Figure 8-5: Drift ratio comparison for Motion 17: Experimental protocol and individual motion

Figure 8-6: Drift ratio comparison for Motion 18: Experimental protocol and individual motion
Figure 8-7: Drift ratio comparison for Motion 20B: Experimental protocol and individual motion

Figure 8-8: Number of fractured bars: experimental protocol and individual motions
Figure 8-9: Cumulative fatigue damage index for Bent 1 after the first three successive design level motion (DE)
Figure 8-10: Cumulative fatigue damage index for Bent 2 after the first three successive design level motion (DE)
Figure 8-11: Cumulative fatigue damage index for Bent 3 after the first three successive design level motion (DE)
Figure 8-12: Maximum numerical drift ratios and number of fractured bars for Bents 1 and 3 for different debonded lengths using bar sizes 3 and 4.
Figure 8-13: Cumulative fatigue damage index in case of using bar # 4 and 24 d#4 as debonded length during Motion 19
Figure 8-14: Drift ratio comparison using bar size # 4 during Motion 19: debonded lengths 16d_{#4} and 24d_{#4}

Figure 8-15: Total base shear envelopes for the bridge case of using bar size # 3 with 9 in. debonded length and using bar size # 4 with 13 in. debonded length during the 221% Design Level Motion.
a) Bent 1 with 24#6 longitudinal reinforcement and 36 0.5"-Strand

b) Bent 2 with 18#7 longitudinal reinforcement and 36 0.5"-Strand

c) Bent 3 with 14#8 longitudinal reinforcement and 36 0.5"-Strand

**Figure 8-16:** Column cross section for prototype bents
**Figure 8-17:** Debonded length to prevent fracture for different bar sizes

**Figure 8-18:** Response spectrum comparison for the 150% Design Level Motion and FEMA ground motions.
Figure 8.19: Response spectrum comparison for the 150% Design Level Motion and design response spectra for multiple cities.
Appendix: A

The design drawings were prepared by UW team, UNR team and Johnson 2006
Figure A-1: Overall specimen dimensions
Figure A-2: Bent dimensions in east-west direction
Figure A-3: Bent dimensions in north-south direction
Figure A-4: Superstructure beam dimensions (Johnson 2006)
Figure A-5: Column dimensions
Figure A-6: Bent Caps 1 and 3, beam reinforcement
Figure A-7: Bent Caps 1 and 3, cantilever reinforcement
Figure A-8: Bent Caps 1 and 3, ledge reinforcement
Figure A-9: Bent Cap 2, beam reinforcement
Figure A-10: Bent Cap 2, ledge reinforcement
Figure A-11: Footing reinforcement of Bents 1 and 2

Notes:
© Axial Ducts 3" Diameter (PVC Sleeve)
Figure A-12: Footing reinforcement of Bent 3
Figure A-13: Spacer block reinforcement (Johnson 2006)
Figure A-14: Superstructure beams reinforcement (Johnson 2006)
Figure A-15: Superstructure beams, ledge reinforcement (Johnson 2006)
Figure A-16: Global coordinate system
Figure A.17: Accelerometer locations

Notes:
- A (Acceleration Measurement)
- T (Torsional measurement)
- L (Longitudinal direction E-W)
- V (Vertical direction U-D)
- 1 (Location 1)

Symbols:
- X (out-of-plane measurement)
- (in-plane measurement)
Figure A-18: Displacement transducer locations
Figure A-19: Strand load cell locations
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**Notes:**

- **Notation:**
  - 1 (Bent Number)
  - N (East Column)
  - T (Top of Column)
  - R (Rotation Measurement)
  - B (Location 1)

- **Symbols:**
  - X: out-of-plane measurement
  - _: in-plane measurement

**Figure A-20:** Curvature rod locations of Bent 1
Table: Curvature Rod Locations

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Notes:

Notation:  
2 (Bent Number)  
N (East Column)  
T (Top of Column)  
R (Rotation Measurement)  
1 (Location 1)

Symbols:

- ×: out-of-plane measurement
- ---: in-plane measurement

Figure A-21: Curvature rod locations of Bent 2
Figure A-22: Curvature rod locations of Bent 3

### Table A-22: Curvature Rod Locations

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Notes:
- **N**: North
- **W**: West
- **Symbols**: 
  - X: out-of-plane measurement
  - _: in-plane measurement
- **Notation**:
  - 3: Bent Number
  - N: North Column
  - R: Rotation Measurement
  - 1: Location 1

**Figure Table**: Bent 3 Curvature Rod Locations
Figure A-23: Strain gage locations of Bent 1
Figure A-24: Strain gage locations of Bent 2
Figure A-25: Strain gage locations of Bent 3
Appendix: B
Figure B-1: Strains using the Displacement Method for 1NB during Motion 17

Figure B-2: Strains using the Displacement Method for 1NT during Motion 17
Figure B-3: Strains using the Displacement Method for 1SB during Motion 17

Figure B-4: Strains using the Displacement Method for 1ST during Motion 17
Figure B-5: Strains using the Displacement Method for 2NB during Motion 17

Figure B-6: Strains using the Displacement Method for 2NT during Motion 17
Figure B-7: Strains using the Displacement Method for 2SB during Motion 17

Figure B-8: Strains using the Displacement Method for 2ST during Motion 17
Figure B-9: Strains using the Displacement Method for 3NB during Motion 17

Figure B-10: Strains using the Displacement Method for 3NT during Motion 17
Figure B-11: Strains using the Displacement Method for 3SB during Motion 17

Figure B-12: Strains using the Displacement Method for 3ST during Motion 17
Figure B-13: Strains using the Displacement Method for 1NB during Motion 18

Motion 18

Figure B-14: Strains using the Displacement Method for 1NT during Motion 18
Figure B-15: Strains using the Displacement Method for 1SB during Motion 18

Figure B-16: Strains using the Displacement Method for 1ST during Motion 18
**Figure B-17:** Strains using the Displacement Method for 2NB during Motion 18

**Figure B-18:** Strains using the Displacement Method for 2NT during Motion 18
Figure B-19: Strains using the Displacement Method for 2SB during Motion 18

Figure B-20: Strains using the Displacement Method for 2ST during Motion 18
Figure B-21: Strains using the Displacement Method for 3NB during Motion 18

Figure B-22: Strains using the Displacement Method for 3NT during Motion 18
Figure B-23: Strains using the Displacement Method for 3SB during Motion 18

Figure B-24: Strains using the Displacement Method for 3ST during Motion 18
Motion 19

Figure B-25: Strains using the Displacement Method for 1NB during Motion 19

Motion 19

Figure B-26: Strains using the Displacement Method for 1NT during Motion 19
Figure B-27: Strains using the Displacement Method for 1SB during Motion 19

Figure B-28: Strains using the Displacement Method for 1ST during Motion 19
Figure B-29: Strains using the Displacement Method for 2NB during Motion 19

Figure B-30: Strains using the Displacement Method for 2NT during Motion 19
Figure B-31: Strains using the Displacement Method for 2SB during Motion 19

Figure B-32: Strains using the Displacement Method for 2ST during Motion 19
Figure B-33: Strains using the Displacement Method for 3NB during Motion 19

Figure B-34: Strains using the Displacement Method for 3NT during Motion 19
Motion 19

Figure B-35: Strains using the Displacement Method for 3SB during Motion 19

Figure B-36: Strains using the Displacement Method for 3ST during Motion 19
Figure B-37: Strains using the Displacement Method for 1NB during Motion 20A

Figure B-38: Strains using the Displacement Method for 1NT during Motion 20A
Figure B-39: Strains using the Displacement Method for 1SB during Motion 20A

Figure B-40: Strains using the Displacement Method for 1ST during Motion 20A
Figure B-41: Strains using the Displacement Method for 2NB during Motion 20A

Figure B-42: Strains using the Displacement Method for 2NT during Motion 20A
Figure B-43: Strains using the Displacement Method for 2SB during Motion 20A

Figure B-44: Strains using the Displacement Method for 2ST during Motion 20A
Figure B-45: Strains using the Displacement Method for 3NB during Motion 20A

Figure B-46: Strains using the Displacement Method for 3NT during Motion 20A
Figure B-47: Strains using the Displacement Method for 3SB during Motion 20A

Figure B-48: Strains using the Displacement Method for 3ST during Motion 20A
Figure B-49: Strains using the Displacement Method for 1NB during Motion 20B

Figure B-50: Strains using the Displacement Method for 1NT during Motion 20B
Figure B-51: Strains using the Displacement Method for 1SB during Motion 20B

Figure B-52: Strains using the Displacement Method for 1ST during Motion 20B
Figure B-53: Strains using the Displacement Method for 2NB during Motion 20B

Figure B-54: Strains using the Displacement Method for 2NT during Motion 20B
Figure B-55: Strains using the Displacement Method for 2SB during Motion 20B

Motion 20B

Figure B-56: Strains using the Displacement Method for 2ST during Motion 20B
Figure B-57: Strains using the Displacement Method for 3NB during Motion 20B

Figure B-58: Strains using the Displacement Method for 3NT during Motion 20B
Figure B-59: Strains using the Displacement Method for 3SB during Motion 20B

Figure B-60: Strains using the Displacement Method for 3ST during Motion 20B
Figure B-61: Strains using the Displacement Method for 1NB during Motion 21A

Figure B-62: Strains using the Displacement Method for 1NT during Motion 21A
Figure B-63: Strains using the Displacement Method for 1SB during Motion 21A

Figure B-64: Strains using the Displacement Method for 1ST during Motion 21A
Figure B-65: Strains using the Displacement Method for 2NB during Motion 21A

Motion 21A

Figure B-66: Strains using the Displacement Method for 2NT during Motion 21A
Figure B-67: Strains using the Displacement Method for 2SB during Motion 21A

Figure B-68: Strains using the Displacement Method for 2ST during Motion 21A
Figure B-69: Strains using the Displacement Method for 3NB during Motion 21A

Figure B-70: Strains using the Displacement Method for 3NT during Motion 21A
Figure B-71: Strains using the Displacement Method for 3SB during Motion 21A

Figure B-72: Strains using the Displacement Method for 3ST during Motion 21A
Figure B-73: Strains using the Displacement Method for 1NB during Motion 21B

Figure B-74: Strains using the Displacement Method for 1NT during Motion 21B
Figure B-75: Strains using the Displacement Method for 1SB during Motion 21B

Figure B-76: Strains using the Displacement Method for 1ST during Motion 21B
Figure B-77: Strains using the Displacement Method for 2NB during Motion 21B

Figure B-78: Strains using the Displacement Method for 2NT during Motion 21B
Figure B-79: Strains using the Displacement Method for 2SB during Motion 21B

Figure B-80: Strains using the Displacement Method for 2ST during Motion 21B
Figure B-81: Strains using the Displacement Method for 3NB during Motion 21B

Figure B-82: Strains using the Displacement Method for 3NT during Motion 21B
Figure B-83: Strains using the Displacement Method for 3SB during Motion 21B

Figure B-84: Strains using the Displacement Method for 3ST during Motion 21B
Figure B-85: Strains using the Displacement Method for 1NB during Motion 21C

Figure B-86: Strains using the Displacement Method for 1NT during Motion 21C
Figure B-87: Strains using the Displacement Method for 1SB during Motion 21C

Figure B-88: Strains using the Displacement Method for 1ST during Motion 21C
Figure B-89: Strains using the Displacement Method for 2NB during Motion 21C

Figure B-90: Strains using the Displacement Method for 2NT during Motion 21C
Figure B-91: Strains using the Displacement Method for 2SB during Motion 21C

Figure B-92: Strains using the Displacement Method for 2ST during Motion 21C
Figure B-93: Strains using the Displacement Method for 3NB during Motion 21C

Figure B-94: Strains using the Displacement Method for 3NT during Motion 21C
Figure B-95: Strains using the Displacement Method for 3SB during Motion 21C

Figure B-96: Strains using the Displacement Method for 3ST during Motion 21C
Figure C-1: Cumulative fatigue damage index for each bar during Motion 18: bar size #3 and 24d₃ debonded length

Figure C-2: Cumulative fatigue damage index for each bar during Motion 18: bar size #4 and 24d₃ debonded length
Figure C-3: Cumulative fatigue damage index for each bar during Motion 18: bar size #3 and 27d₃ debonded length

Figure C-4: Cumulative fatigue damage index for each bar during Motion 18: bar size #4 and 27d₃ debonded length
Figure C-5: Cumulative fatigue damage index for each bar during Motion 18: bar size 

#3 and 30d₃ debonded length

Figure C-6: Cumulative fatigue damage index for each bar during Motion 18: bar size 

#4 and 30d₃ debonded length
Figure C-7: Cumulative fatigue damage index for each bar during Motion 18: bar size #3 and 33d_{#3} debonded length

Figure C-8: Cumulative fatigue damage index for each bar during Motion 18: bar size #4 and 33d_{#3} debonded length
Figure C-9: Cumulative fatigue damage index for each bar during Motion 19: bar size #3 and 24d₃ debonded length

Figure C-10: Cumulative fatigue damage index for each bar during Motion 19: bar size #4 and 24d₃ debonded length
Figure C-11: Cumulative fatigue damage index for each bar during Motion 19: bar size #3 and 27d_{#3} debonded length

Figure C-12: Cumulative fatigue damage index for each bar during Motion 19: bar size #4 and 27d_{#3} debonded length
Figure C-13: Cumulative fatigue damage index for each bar during Motion 19: bar size #3 and 30daₜₚ debonded length

Figure C-14: Cumulative fatigue damage index for each bar during Motion 19: bar size #4 and 30daₜₚ debonded length
Figure C-15: Cumulative fatigue damage index for each bar during Motion 19: bar size #3 and 33d#3 debonded length

Figure C-16: Cumulative fatigue damage index for each bar during Motion 19: bar size #4 and 33d#3 debonded length
Figure C-17: Drift ratio comparison using bar size #3 during Motion 18: debonded lengths $24d_{#3}$ and $27d_{#3}$.

Figure C-18: Drift ratio comparison using bar size #3 during Motion 18: debonded lengths $24d_{#3}$ and $30d_{#3}$.
Figure C-19: Drift ratio comparison using bar size #3 during Motion 18: debonded lengths 24d₃ and 33d₃

Figure C-20: Drift ratio comparison using bar size #4 during Motion 18: debonded lengths 24d₄ and 27d₄
Figure C-21: Drift ratio comparison using bar size #4 during Motion 18: debonded lengths 24d\#3 and 30d\#3

Figure C-22: Drift ratio comparison using bar size #4 during Motion 18: debonded lengths 24d\#3 and 33d\#3
**Figure C-23**: Drift ratio comparison using bar size #3 during Motion 19: debonded lengths 24d₃ and 27d₃

**Figure C-24**: Drift ratio comparison using bar size #3 during Motion 19: debonded lengths 24d₃ and 30d₃
Figure C-25: Drift ratio comparison using bar size #3 during Motion 19: debonded lengths 24d\#3 and 33d\#3

Figure C-26: Drift ratio comparison using bar size #4 during Motion 19: debonded lengths 24d\#3 and 27d\#3
Figure C-27: Drift ratio comparison using bar size #4 during Motion 19: debonded lengths 24d#3 and 30d#3

Figure C-28: Drift ratio comparison using bar size #4 during Motion 19: debonded lengths 24d#3 and 33d#3