Title
Analytical and Experimental Studies of the Seismic Performance of Reinforced Concrete Structural Wall Boundary Elements

Permalink
https://escholarship.org/uc/item/5pw520hw

Author
Hilson, Christopher William

Publication Date
2014

Peer reviewed|Thesis/dissertation
UNIVERSITY OF CALIFORNIA
Los Angeles

Analytical and Experimental Studies of the Seismic Performance
of Reinforced Concrete Structural Wall Boundary Elements

A dissertation submitted in partial satisfaction of the
requirements for the degree Doctor of Philosophy
in Civil Engineering

by

Christopher William Hilson

2014
ABSTRACT OF THE DISSERTATION

Analytical and Experimental Studies of the Seismic Performance of Reinforced Concrete Structural Wall Boundary Elements

by

Christopher William Hilson

Doctor of Philosophy in Civil Engineering

University of California, Los Angeles, 2014

Professor John W. Wallace, Chair

Following the February 27, 2010 M\textsubscript{w} 8.8 Maule earthquake, an international effort was undertaken to better understand reasons for observed damage to concrete structural walls in buildings located in the affected region of Chile and to address potential design implications. The Chilean building code for concrete structures is based on the U.S. ACI 318 building code; however, based on the observed performance of over 400 buildings in the March 1985 earthquake-impacted Viña del Mar, Chilean Code NCh433.Of96 included an exception that special boundary elements (SBEs)—which are commonly required for walls in U.S. buildings—need not be provided. By taking exception to the special boundary element detailing provisions, the Chilean code allowed thin wall boundary zones with relatively large (typically 20 cm) spacing of transverse reinforcement (essentially unconfined) to be constructed. Given these differences, the 2010 earthquake is an excellent opportunity to assess the performance of reinforced concrete buildings designed using modern codes similar to those used in the United States. Data from damaged and undamaged buildings, as well as from parametric and experimental studies, are used to provide recommendations to improve the efficacy of U.S. provisions designed to inhibit structural damage at wall boundaries.
Seven Chilean buildings were selected to investigate the performance of boundary elements during the 2010 earthquake. Several walls from each of the seven buildings were chosen to evaluate the ACI 318-11 Section 21.9.6.2 displacement-based trigger equation for determining if SBEs would have been required and if observed damage was consistent with the evaluation result (i.e., SBE required, no damage; SBE required, damage observed). The propensity of boundary longitudinal reinforcement to buckle was also investigated, taking into consideration the influence of boundary transverse reinforcement configuration and longitudinal reinforcement strain history. In conjunction with assessments of in-situ wall performance, parametric studies were conducted on wall sections with various attributes and laboratory tests were performed on prisms representative of wall boundary elements using NEES@UCLA facilities to further investigate structural wall boundary element performance under reverse-cyclic demands.

The evaluation indicated that Chilean wall sections where the neutral axis depth exceeded the limit imposed by ACI 318-11 equation 21-8 and SBE-level detailing was not provided typically suffered significant concrete crushing and longitudinal bar buckling. Rebar buckling studies showed that, under large magnitude strain reversals, longitudinal reinforcement in flanged, tension-controlled walls is susceptible to buckling at non-flanged wall boundaries. Test results demonstrated that spacing-to-bar-diameter ($s/d_b$) ratios of 10.7 (8” spacing) and 8.0 (6” spacing) led to concrete cover loss initiated by longitudinal bar buckling upon reloading into compression from tension strains of approximately 2% or larger, followed by substantial strength loss on the subsequent loading cycle into compression. Specimens that did not exhibit damage initiated by bar buckling eventually failed due to global out-of-plane buckling (lateral instability), or by sudden crushing failure after cover concrete spalling. Results from analyses and experimental studies suggest compression reloading strains greater than 2% may initiate buckling of longitudinal reinforcement if spacing of transverse reinforcement is extended to the maximum permitted by ACI 318-11 for non-SBE configurations, and bar buckling may be possible in SBEs with reloading strains greater than 4%.
The dissertation of Christopher William Hilson is approved.

Jonathan P. Stewart

Ertugrul Taciroglu

Thomas A. Sabol

Anne Lemnitzer

John W. Wallace, Committee Chair

University of California, Los Angeles

2014
ABSTRACT OF THE DISSERTATION ........................................................................................................................................................................ ii
TABLE OF CONTENTS .................................................................................................................................................................................... v
LIST OF FIGURES ......................................................................................................................................................................................... vii
LIST OF TABLES ............................................................................................................................................................................................. xvii
LIST OF SYMBOLS ......................................................................................................................................................................................... xviii
ACKNOWLEDGEMENTS ................................................................................................................................................................................... xxii
VITA ........................................................................................................................................................................................................... xxiv
PUBLICATIONS AND PRESENTATIONS ......................................................................................................................................................... xxv
CHAPTER 1 INTRODUCTION ............................................................................................................................................................................ 1
  1.1 General ......................................................................................................................................................................................................... 1
  1.2 Scope and Objectives ........................................................................................................................................................................ 3
  1.3 Organization .................................................................................................................................................................................................... 4
CHAPTER 2 LITERATURE AND RECONNAISSANCE REVIEW .................................................................................................................. 6
  2.1 Chilean Building Practices Overview .................................................................................................................................................. 6
  2.2 Structural Wall Boundary Element Displacement Based Design in U.S. Codes ............................................................................... 10
  2.3 Previous Research on Bar Buckling and Boundary Elements in Thin Concrete Walls ....................................................................... 12
    2.3.1 Experiments Characterizing Buckling Onset of Isolated Reinforcing Bars .................................................................................. 13
    2.3.2 Out-of-Plane Instability .................................................................................................................................................................. 20
    2.3.3 Bar Buckling Observed in Reinforced Concrete Elements ............................................................................................................. 22
CHAPTER 3 CHILEAN WALL ANALYSIS APPROACH ........................................................................................................................... 31
  3.1 Building Roof Displacement Estimation ............................................................................................................................................... 32
  3.2 Special Boundary Element Detailing Trigger Investigation .............................................................................................................. 44
  3.3 Boundary Element Longitudinal Bar Buckling Investigation .......................................................................................................... 49
  3.4 Boundary Element Longitudinal Bar Buckling Assessment Verification .............................................................................................. 59
CHAPTER 4 ASSESSMENTS OF CHILEAN BUILDINGS ............................................................................................................................ 66
  4.1 Building #1 (Alto Rio) ............................................................................................................................................................................... 68
    4.1.1 Building Description and Wall Selection ........................................................................................................................................... 68
    4.1.2 Special Boundary Element Trigger Results ...................................................................................................................................... 72
    4.1.3 Longitudinal Reinforcement Buckling Predictions ............................................................................................................................. 74
  4.2 Building #2 (Plaza Del Rio B) .................................................................................................................................................................. 78
7.1 Longitudinal Bar Buckling Damage States ........................................................................ 155
7.2 Specimen 8WT-2 (Test #1) .................................................................................................. 156
7.3 Specimen 8NT-2 (Test #2) .................................................................................................. 163
7.4 Specimen 6NT-2 (Test #3) .................................................................................................. 168
7.5 Specimen 6WT-2 (Test #4) .................................................................................................. 172
7.6 Specimen 8WT-3 (Test #5) .................................................................................................. 176
7.7 Specimen 6WT-3 (Test #6) .................................................................................................. 180
7.8 Specimen 45WT-45 (Test #7) ............................................................................................. 183
7.9 Specimen 45WT-1 (Test #8) ............................................................................................... 188
7.10 Experimental Summary .................................................................................................... 192

CHAPTER 8 POTENTIAL BAR BUCKLING IN WALL BOUNDARIES ........................................ 196

CHAPTER 9 SUMMARY AND CONCLUSIONS ........................................................................ 211
9.1 Conclusions ......................................................................................................................... 213

APPENDIX A CHILEAN WALL ANALYSES ............................................................................ 217
APPENDIX B UCLA PRISM EXPERIMENTS .......................................................................... 256
REFERENCES .......................................................................................................................... 289

LIST OF FIGURES

Figure 1.1 Earthquake risk mitigation process (FEMA P-749, 2009). ........................................ 2
Figure 1.2 Typical wall damage in 2010 Chile earthquake. .................................................... 2
Figure 2.1 Representative wall end details of two Chilean buildings (Massone et al. 2012). ........ 8
Figure 2.2 Transverse reinforcement terminated by 90 degree hooks at wall boundary. .............. 9
Figure 2.3 Impact of L/D ratios on symmetric cyclic strain (Monti and Nuti 1992). ...................... 13
Figure 2.4 Initial bar buckling identifier parameter $\varepsilon_p^*$ (Rodriguez et al. 1999). ................. 14
Figure 2.5 Lower bound estimate of initiation of bar buckling from cyclic strain demands .......... 15
Figure 2.6 Deviation strain to identify bar buckling initiation, as defined by Hose (2000) ............. 16
Figure 2.7 Comparison of relationships and data for isolated bar buckling in air ......................... 19
Figure 2.8 Height-to-width ratio, reinforcement ratio influence on critical tension strain for global
buckling relationships. Eqn. 8 refers to the relationship from Chai and Elayer (1999) and Eqn. 9 is
Paulay and Priestley (1993). ................................................................................................. 22
Figure 2.9 (a) Moment curvature of RC element with annotated damage states and (b) average stress-strain ratio of longitudinal reinforcing bar (Suda et al. 1996). .................................................................25
Figure 2.10 Suda et al. (1996) stress-strain model for reinforcing bar post-buckling. ..............................................26
Figure 2.11 Buckled bars in the boundary element of specimen B1 (Oesterle 1976). ..............................................27
Figure 2.12 Force-displacement plots for Walls CLS and CMS (Sittipunt and Wood, 1993).................................28
Figure 2.13 Boundary element damage in 4 wall tests by Thomsen and Wallace (1995). (a) RW1; (b) RW2; (c) TW1; (d) TW2. ........................................................................................................30
Figure 3.1 Summary of Chilean building and recording station orientations. .........................................................35
Figure 3.2 Linear response spectra for Concepción station. Note the large spikes around periods of 1.5-2s. ......................................................................................................................................................36
Figure 3.3 Linear response spectra for Viña Del Mar-Centro station.................................................................37
Figure 3.4 Linear response spectra for Santiago-Centro and Peñalolen stations (E-W left, N-S right)........37
Figure 3.5 Building #1 spectral displacement. Upper and lower period estimates bound a peak in S_d ....38
Figure 3.6 Spectral displacements for Buildings #2 and #3...............................................................................40
Figure 3.7 Building #4 spectral displacement............................................................................................................41
Figure 3.8 Santiago buildings (#5, #6, and #7) spectral displacements..............................................................42
Figure 3.9 BIAx program material models for H25 and H30 unconfined concrete............................................48
Figure 3.10 Lower bound estimate of initiation of bar buckling from cyclic strain demands (Rodriguez et al. 1999)..................................................................................................................................................50
Figure 3.11 Steel reinforcement material models for ACI 318 equation 21-8 trigger and bar buckling analysis. ...............................................................................................................................................51
Figure 3.12 Comparison of various plastic hinge length models............................................................................53
Figure 3.13 Roof Drift Ratio vs. Wall Curvature .......................................................................................................55
Figure 3.14 Influence of plastic hinge length on SBEs (Wallace et al., 2012).......................................................55
Figure 3.15 Steps for assessing ε_p^* from BIAx results for sample wall assuming equal drifts. ....................57
Figure 3.16 Wall 4.2 web stem bar buckling analysis, plus moment capacity vs. drift, to compare the onset of web crushing with the onset of bar buckling.................................................................58
Figure 3.17 Bar buckling analysis of Thomsen Test TW1; web compression in lower left quadrant ....61
Figure 3.18 Bar buckling analysis for Thomsen wall RW1.................................................................................62
Figure 3.19 Bar buckling analysis for Thomsen wall test RW2. .........................................................................63
Figure 3.20 Bar buckling analysis for Thomsen wall test TW2; web compression in lower left quadrant.64
Figure 4.1 Building #1 typical floor plan and walls selected for investigation. .......................................................69
Figure 4.2 Building #1 floor reduction at upper levels. ..........................................................................................70
Figure 4.3 Typical wall setback at north side of Building #1 at podium level. ....................................................70
Figure 4.4 Total collapse of Building #1. .......................................................... 72
Figure 4.5 Damage report for reconstructed wall elevations in Building #1 (IDIEM, 2010). .................. 72
Figure 4.6 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 1.1 ....................... 75
Figure 4.7 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 1.2 ....................... 76
Figure 4.8 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 1.3 ....................... 76
Figure 4.9 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 1.4 ....................... 77
Figure 4.10 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 1.5 ...................... 77
Figure 4.11 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 1.6 ...................... 78
Figure 4.12 Building #2 typical floor plan and selected walls for examination. ................................. 79
Figure 4.13 Building #2 transverse hoop detailing................................................................. 80
Figure 4.14 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 2.1 ....................... 82
Figure 4.15 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 2.2 ....................... 83
Figure 4.16 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 2.3 ....................... 83
Figure 4.17 Building #3 floor plans and walls selected for analysis. .................................................. 84
Figure 4.18 Wall 3.2 horizontal web bar detailing. Note no hoops or crossties at end zones ............... 85
Figure 4.19 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 3.1 ....................... 88
Figure 4.20 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 3.2 ....................... 88
Figure 4.21 Building #4 typical floor plan with investigated walls highlighted in red ....................... 89
Figure 4.22 Wall 4.4 demonstrating typical setback in northern walls of Building #4 ....................... 90
Figure 4.23 Documented damage in Building #4. (a) Crushing and bar buckling in northern walls; (b) story drop at northwest corner with shoring; (c) tension cracks at southern wall .................. 91
Figure 4.24 Damaged walls in Building #4 at ground level (NIST GCR12-917-18). ......................... 93
Figure 4.25 Wall 4.5 with concrete spalling at (plan) south boundary (NIST GCR12-917-18) ............. 93
Figure 4.26 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 4.1 ....................... 95
Figure 4.27 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 4.2 ....................... 95
Figure 4.28 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 4.3 ....................... 96
Figure 4.29 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 4.4 ....................... 96
Figure 4.30 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 4.5 ....................... 97
Figure 4.31 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 4.6 ....................... 97
Figure 4.32 Building #5 typical floor plan with highlighted walls. ................................................. 99
Figure 4.33 Stirrup and horizontal web bar termination at wall end in Building #5 ......................... 99
Figure 4.34 Damage in Building #5. (a) Crushing along length of web and bar buckling; (b) spalling and bar buckling ................................................................. 100
Figure 4.35 Concrete crushing at boundaries in Wall 5.1 (left) and Wall 5.2 (right) ......................... 102
Figure 4.36 Boundary core concrete crushing and bar buckling at non-flanged end of Wall 5.4........... 102
Figure 4.37 Spalling of cover concrete on (plan) north non-flanged end of Wall 5.3....................... 103
Figure 4.38 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 5.1............... 105
Figure 4.39 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 5.2............ 105
Figure 4.40 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 5.3........... 106
Figure 4.41 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 5.4........... 106
Figure 4.42 Building #6 typical floor plan and walls selected for analysis............................... 108
Figure 4.43 Building #6 wall edge detail and photo of unhooked horizontal bars. The wall detail appears to specify hooks on alternating bar sets, but they are not apparent in the wall photo............. 108
Figure 4.44 Setback typical of northern transverse walls at 1st subterranean level in Building #6....... 108
Figure 4.45 Spalled cover concrete at (plan) north boundary of Wall 6.1.................................... 110
Figure 4.46 Concrete cover spalling, bar buckling, and out-of-plane buckling at (plan) north boundary of Wall 6.2 .......................................................... 111
Figure 4.47 Composite image of damage to Wall 6.3 flange (left) and web stem boundary (right) .... 111
Figure 4.48 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 6.1............. 113
Figure 4.49 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 6.2............. 114
Figure 4.50 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 6.3............. 114
Figure 4.51 Typical floor plan for Building #7. Walls investigated highlighted in red.................... 116
Figure 4.52 Wall 7.1 elevation at ground level with barbell section cuts........................................ 117
Figure 4.53 Wall 7.2 elevation (7.3 similar).................................................................................... 118
Figure 4.54 Damage at non-flanged end of wall 7.2................................................................. 119
Figure 4.55 Damage at non-flanged end of Wall 7.2 near ceiling of subterranean level............... 121
Figure 4.56 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 7.1............. 122
Figure 4.57 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 7.2............. 122
Figure 4.58 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 7.3............. 123
Figure 5.1 Demonstration of strain demands in flanged walls and representative test specimens. The red shaded region represents the tension strain field at the boundary, and blue represents the compression strain field................................................................. 132
Figure 5.2 General specimen configuration.................................................................................... 134
Figure 5.3 Test region of specimens with 6" and 8" transverse spacing........................................ 135
Figure 5.4 Specimen hoop and crosstie......................................................................................... 135
Figure 5.5 Headed #6 longitudinal bars and transverse reinforcement........................................ 136
Figure 5.6 Simplified load paths for connection blocks. (Left) Strut-and-tie model for specimen in tension; (right) assumed mat behavior for specimen in compression........................................ 138
Figure 7.7 Force-strain plot for specimen 8NT-2 of 3% and 4.5% tensile strain cycles with damage state indicators. ...................................................................................................................................... 167

Figure 7.8 In-plane loading frame lateral drift for specimen 8NT-2. (Left) Drift ratio vs. axial strain; (right) drift ratio vs. axial force. ..................................................................................................................................... 168

Figure 7.9 Out-of-plane movement and vertical cracking in specimen 6NT-2. ................................................. 169

Figure 7.10 Force-strain plot of 3% and 4.5% tensile strain cycles for specimen 6NT-2. Global buckling captured by mounted LVDTs shortly into compression reloading. ........................................................................... 171

Figure 7.11 In-plane loading frame lateral drift for specimen 6NT-2. (Left) Drift ratio vs. axial strain; (right) drift ratio vs. axial force. .................................................................................................... 171

Figure 7.12 Damage progression associated with longitudinal bar buckling in specimen 6WT-2. (a) No damage associated with bar buckling; (b) cracks associated with DS1; (c) widened cracks associated with DS2; (d) cover push-off associated with DS3; (e) cover loss exposing buckled bars. ............................................................................................................................................... 172

Figure 7.13 Adjacent (left) longitudinal bar buckling in specimen 6WT-2 ............................................................ 173

Figure 7.14 Force-strain plot of 4.5% tensile strain cycle for specimen 6WT-2 .......................................................... 174

Figure 7.15 Specimen 6WT-2 splitting failure after global buckling. ........................................................................ 174

Figure 7.16 In-plane loading frame lateral drift for specimen 6WT-2. (Left) Drift ratio vs. axial strain; (right) drift ratio vs. axial force. .................................................................................................... 174

Figure 7.17 Force-strain plot for specimen 8WT-3 cycles to 3% and 4.5% tension strain .................................. 175

Figure 7.18 Out-of-plane buckling in specimen 8WT-3 (north surface on left). ...................................................... 178

Figure 7.19 In-plane loading frame lateral drift for specimen 8WT-3. (Left) Drift ratio vs. axial strain; (right) drift ratio vs. axial force. .................................................................................................... 179

Figure 7.20 Specimen 8WT-3 after post-test spalled concrete cover removal ......................................................... 179

Figure 7.21 Force-strain plot for specimen 6WT-3 cycles to 3% and 4.5% tension strain ........................................ 181

Figure 7.22 Global buckling deformed shape of specimen 6WT-3 during initial compression reloading cycle from 4.5% tension strain limit ...................................................................................................................... 182

Figure 7.23 In-plane loading frame lateral drift for specimen 6WT-3. (Left) Drift ratio vs. axial strain; (right) drift ratio vs. axial force. .................................................................................................... 183

Figure 7.24 Hairline compression crack at first 0.45% strain cycle (left edge, above mid-height) .................. 185

Figure 7.25 Force-strain plot for specimen 45WT-45 cycles to 3% and 4.5% tension strain ............................... 185

Figure 7.26 Spalling on northeast (left) edge of 45WT-45 ................................................................................... 186

Figure 7.27 Out-of-plane buckling in specimen 45WT-45, first compression reloading from 4.5% tension strain limit ........................................................................................................................................ 187
Figure 7.28 In-plane loading frame lateral drift for specimen 45WT-45. (Left) Drift ratio vs. axial strain; (right) drift ratio vs. axial force. ................................................................. 187
Figure 7.29 Force-strain plots for specimen 45WT-I................................................................................. 190
Figure 7.30 Sudden, brittle failure in 45WT-I on first excursion to 0.6% compressive strain. ................. 191
Figure 7.31 In-plane loading frame lateral drift for specimen 45WT-I. (Left) Drift ratio vs. axial strain; (right) drift ratio vs. axial force. .................................................................................................... 191
Figure 7.32 Reloading strain results versus transverse spacing .............................................................. 194
Figure 8.1 Idealized wall cross section for Eq. 8.1.................................................................................. 196
Figure 8.2 Wall neutral axis depth versus axial load and steel reinforcement ratio ($f'_c = 4$ ksi) .......... 198
Figure 8.3 Wall neutral axis depth versus axial load and steel reinforcement ratio ($f'_c = 6$ ksi) .......... 198
Figure 8.4 Wall neutral axis depth versus axial load and steel reinforcement ratio ($f'_c = 8$ ksi) .......... 199
Figure 8.5 Wall neutral axis depth versus axial load, steel reinforcement ratio, and $f'_c$ ....................... 199
Figure 8.6 Wall neutral axis depth for varying wall end minimum steel ratios ($f'_c = 4$ ksi)................. 200
Figure 8.7 Wall neutral axis depth for varying wall end minimum steel ratios ($f'_c = 6$ ksi)................. 201
Figure 8.8 Wall neutral axis depth for varying wall end minimum steel ratios ($f'_c = 8$ ksi)................. 201
Figure 8.9 Wall neutral axis depth versus boundary longitudinal reinforcement ratio, and $f'/c$ ........ 202
Figure 8.10 Wall neutral axis depth and drift demand impact on compressive strain.......................... 203
Figure 8.11 Tension strain demand at opposite end of wall and Rodriguez et al. (1999) buckling initiation. ................................................................................................................................................ 204
Figure 8.12 Tension strain as a function of drift and neutral axis depth................................................. 206
Figure 8.13 Drift vs. neutral axis depth with ACI 318-14 OBE limits...................................................... 208
Figure 8.14 Revised design chart for bar buckling including 1.5 factor for drift and prism test results... 209
Figure 8.15 Summary of proposed additional check for wall boundary transverse reinforcement detailing. ........................................................................................................................................ 210

Figure A.1 Concepción recording station and candidate buildings (Google Earth, 2014). ................. 217
Figure A.2 Viña del Mar recording station and candidate building (Google Earth, 2014)................. 218
Figure A.3 Santiago recording stations and candidate buildings (Google Earth, 2014)....................... 218
Figure A.4 Critical section for wall 1.1 ................................................................................................. 219
Figure A.5 Critical elevation views for Wall 1.1 .................................................................................. 219
Figure A.6 Moment-curvature for Wall 1.1 (flanged wall, flange compression in negative direction). . 220
Figure A.7 Critical section for Wall 1.2 ................................................................................................. 220
Figure A.8 Critical elevation views for Wall 1.2 .................................................................................. 221
Figure A.9 Moment-curvature for Wall 1.2 (flanged wall, flange compression in positive direction). . 221
Figure B.2 Concrete cylinder compression tests for Specimen 8NT-2 .................................................... 257
Figure B.3 Concrete cylinder compression tests for Specimen 6NT-2 .................................................... 257
Figure B.4 Concrete cylinder compression tests for Specimen 6WT-2 ................................................... 258
Figure B.5 Concrete cylinder compression tests for Specimen 8WT-3 ................................................... 258
Figure B.6 Concrete cylinder compression tests for Specimen 6WT-3 ................................................... 259
Figure B.7 Concrete cylinder compression tests for Specimen 45WT-45 ............................................... 259
Figure B.8 Concrete cylinder compression tests for Specimen 45WT-I ................................................... 260
Figure B.9 Mill certification for Group I specimen longitudinal reinforcement ...................................... 263
Figure B.10 Mill certification for Group II specimen longitudinal reinforcement .................................. 264
Figure B.11 Force-axial strain plots for 8WT-2 (0.05% to 0.2% cycles) ................................................ 265
Figure B.12 Force-axial strain plots for 8WT-2 (0.3% to 1.0% cycles) .................................................. 266
Figure B.13 Force-axial strain plots for 8WT-2 (1.5% to 4.5% cycles) .................................................. 267
Figure B.14 Force-axial strain plots for 8NT-2 (0.05% to 0.2% cycles) ................................................ 268
Figure B.15 Force-axial strain plots for 8NT-2 (0.3% to 1.0% cycles) .................................................. 269
Figure B.16 Force-axial strain plots for 8NT-2 (1.5% to 3.0% cycles) .................................................. 270
Figure B.17 Force-axial strain plots for 6NT-2 (0.05% to 0.2% cycles) ................................................ 271
Figure B.18 Force-axial strain plots for 6NT-2 (0.3% to 1.0% cycles) ................................................... 272
Figure B.19 Force-axial strain plots for 6NT-2 (1.5% to 4.5% cycles) ................................................... 273
Figure B.20 Force-axial strain plots for 6WT-2 (0.05% to 0.2% cycles) ................................................ 274
Figure B.21 Force-axial strain plots for 6WT-2 (0.3% to 1.0% cycles) .................................................. 275
Figure B.22 Force-axial strain plots for 6WT-2 (1.5% to 4.5% cycles) .................................................. 276
Figure B.23 Force-axial strain plots for 8WT-3 (0.05% to 0.2% cycles) ................................................ 277
Figure B.24 Force-axial strain plots for 8WT-3 (0.3% to 1.0% cycles) .................................................. 278
Figure B.25 Force-axial strain plots for 8WT-3 (1.5% to 4.5% cycles) .................................................. 279
Figure B.26 Force-axial strain plots for 6WT-3 (0.05% to 0.2% cycles) ................................................. 280
Figure B.27 Force-axial strain plots for 6WT-3 (0.3% to 1.0% cycles) ................................................... 281
Figure B.28 Force-axial strain plots for 6WT-3 (1.5% to 4.5% cycles) ................................................... 282
Figure B.29 Force-axial strain plots for 45WT-45 (0.05% to 0.2% cycles) ............................................ 283
Figure B.30 Force-axial strain plots for 45WT-45 (0.3% to 1.0% cycles) ............................................. 284
Figure B.31 Force-axial strain plots for 45WT-45 (1.5% to 4.5% cycles) ............................................. 285
Figure B.32 Force-axial strain plots for 45WT-I (0.05% to 0.2% cycles) ............................................... 286
Figure B.33 Force-axial strain plots for 45WT-I (0.3% to 1.0% cycles) ............................................... 287
Figure B.34 Force-axial strain plots for 45WT-I (1.5% to 2.0% cycles) ................................................. 288
LIST OF TABLES

Table 2.1 Measured deviation strains at buckling from single-cycle tests (Hose, 2000) ........................................ 17
Table 3.1 Summary of simple fundamental building period estimates and spectral displacement results .......... 43
Table 3.2 Typical floor weights in Building #1 .................................................................................................. 46
Table 3.3 Design strengths corresponding to cube test strengths (NCh430) .................................................. 47
Table 3.4 Bar buckling analysis results for Thomsen wall specimens ............................................................... 65
Table 4.1 Building #1 wall section geometry. .................................................................................................. 71
Table 4.2 Building #1 SBE trigger evaluation ................................................................................................. 73
Table 4.3 Building #1 bar buckling analysis summary .................................................................................. 75
Table 4.4 Building #2 wall section geometry. .................................................................................................. 80
Table 4.5 Building #2 SBE trigger evaluation ................................................................................................. 81
Table 4.6 Building #2 bar buckling analysis summary .................................................................................. 82
Table 4.7 Building #3 wall section geometry. .................................................................................................. 86
Table 4.8 Building #3 SBE trigger evaluation ................................................................................................. 87
Table 4.9 Building #3 Bar buckling analysis summary .................................................................................. 87
Table 4.10 Building #4 wall section geometry. ............................................................................................... 91
Table 4.11 Building #4 SBE trigger evaluation ............................................................................................... 92
Table 4.12 Building #4 bar buckling analysis summary .................................................................................. 94
Table 4.13 Building #5 wall section geometry. ............................................................................................... 100
Table 4.14 Building #5 SBE trigger evaluation ............................................................................................... 101
Table 4.15 Building #5 bar buckling analysis summary .................................................................................. 104
Table 4.16 Building #6 wall section geometry. ............................................................................................... 109
Table 4.17 Building #6 SBE trigger evaluation ............................................................................................... 110
Table 4.18 Building #6 Bar buckling analysis results .................................................................................... 112
Table 4.19 Building #7 wall section geometry. ............................................................................................... 119
Table 4.20 Building #7 SBE trigger evaluation ............................................................................................... 120
Table 4.21 Building #7 Bar buckling analysis results ..................................................................................... 121
Table 4.22 Chilean wall analyses summary .................................................................................................... 127
Table 5.1 Test matrix with specimen geometry ............................................................................................... 133
Table 6.1 Strain history protocol for Group I. ................................................................................................. 151
Table 6.2 Strain history protocol for first two experiments in Group II. .............................................................. 152
Table 6.3 Strain history protocol for the third experiment in Group II ................................................................ 153
Table 6.4 Strain history protocol for the final experiment in Group II .................................................. 154
Table 7.1 Peak applied load summary for 8WT-2 .............................................................................. 157
Table 7.2 Peak applied load summary for specimen 8NT-2 ............................................................... 165
Table 7.3 Peak applied axial forces for specimen 6NT-2 ................................................................. 169
Table 7.4 Peak applied axial load summary for specimen 6WT-2 ..................................................... 173
Table 7.5 Peak applied forces for loading cycles of specimen 8WT-3 .............................................. 176
Table 7.6 Peak applied axial force for each cycle of specimen 6WT-3 .............................................. 181
Table 7.7 Peak applied axial load for each cycle of specimen 45WT-45 ........................................... 184
Table 7.8 Peak applied axial loads for each cycle of specimen 45WT-1 ............................................ 189
Table B.1 Group I concrete cylinder test summary ............................................................................ 260
Table B.2 Group II concrete cylinder test summary ........................................................................... 261

LIST OF SYMBOLS

\[ A_g = \] gross cross-sectional area of boundary element
\[ A_r = \] wall aspect ratio
\[ b = \] concrete element width for out-of-plane buckling considerations
\[ b_w = \] wall width
\[ c = \] wall neutral axis depth for extreme fiber compression strain of 0.003
\[ C_d = \] building code prescribed deflection amplification factor (ASCE 7-10)
\[ c_{\text{limit}} = \] limiting neutral axis depth for non-SBE detailing assuming 0.003 compressive strain
\[ c_{\text{trig}} = \] maximum neutral axis depth permitted for non-SBE detailing per ACI 318 Eq. (21-8)
\[ d = \] effective depth of concrete member
\[ d_b = \] boundary element longitudinal reinforcement diameter
\[ D = \] steel coupon diameter from Monti and Nuti (1992) and Rodriguez et al. (1999)
\[ E_d = \] steel reinforcement double modulus
\[ E_r = \] steel reinforcement reduced modulus
\[ f_c = \] concrete compression stress
\[ f_c = \text{concrete compression strength} \]
\[ f_{\text{max}} = \text{maximum reinforcement tensile stress} \]
\[ f_y = \text{yield strength of reinforcement} \]
\[ f_p = \text{reinforcing bar compression stress at buckling} \]
\[ h = \text{concrete element height for out-of-plane buckling considerations} \]
\[ h_w = \text{wall height} \]
\[ h_x = \text{maximum horizontal spacing between transverse hoop legs and/or ties in boundary element} \]
\[ I = \text{building importance factor} \]
\[ I_g = \text{wall gross moment of inertia} \]
\[ K = \text{effective length fixity factor} \]
\[ L = \text{steel coupon length from Monti and Nuti (1992)} \]
\[ l_p = \text{wall plastic hinge length} \]
\[ l_w = \text{wall length} \]
\[ l_{\text{web}} = \text{wall web length} \]
\[ m = \text{mechanical reinforcement ratio} \]
\[ M_n = \text{wall nominal moment capacity} \]
\[ M_w = \text{moment magnitude} \]
\[ M_s = \text{surface wave magnitude} \]
\[ M_u = \text{design (factored) moment demand} \]
\[ N = \text{number of stories above grade level} \]
\[ P_u = \text{design (factored) axial load} \]
\[ P_{u,max} = \text{maximum design (factored) axial load} \]
\[ s = \text{vertical spacing between boundary transverse reinforcement} \]
\[ S_d = \text{spectral displacement} \]
\[ S_h = \text{steel coupon length from Rodriguez et al. (1999)} \]
\[ T = \text{building fundamental period} \]
\( t_w \) = wall web thickness
\( z \) = distance
\( \alpha \) = steel strain hardening factor
\( \beta \) = ratio of crack closure width to concrete element width
\( \beta_1 \) = Whitney stress block depth factor
\( \gamma \) = steel overstrength factor
\( \delta_{\text{roof}} \) = roof displacement
\( \delta_{\text{SDOF}} \) = single degree of freedom system displacement response
\( \delta_d \) = design roof displacement
\( \delta_e \) = roof displacement from elastic analysis
\( \varepsilon_c \) = extreme fiber compression strain
\( \varepsilon_{cc} \) = compression strain capacity prior to buckling from Moyer and Kowalsky (2003)
\( \varepsilon_{cu} \) = extreme fiber compression strain at design displacement
\( \varepsilon_{\text{max}} \) = imposed compression strain limit for prism tests
\( \varepsilon_{\text{dev}} \) = deviation strain per Hose (2000), measured from peak tension strain to initial buckling
\( \varepsilon_o^+ \) = compression reloading strain from zero axial force to zero axial strain
\( \varepsilon_p \) = compression strain from zero axial strain to initial reinforcing bar buckling
\( \varepsilon_p^* \) = compression reloading strain from zero axial force to initial reinforcing bar buckling \((\varepsilon_o^+ - \varepsilon_p)\)
\( \varepsilon_{p,\text{calc}}^* \) = compression reloading strain capacity at roof drift estimate (equal drifts assumption)
\( \varepsilon_{p,\text{test}}^* \) = maximum compression reloading strain capacity of prism tests
\( \varepsilon_{p,\text{ana}}^* \) = compression reloading strain at buckling initiation from analysis
\( \varepsilon_{sm} \) = maximum tension strain without developing out-of-plane instability on compression reloading
\( \varepsilon_r \) = reinforcement tension strain
\( \varepsilon_y \) = reinforcement yield strain
\( \theta_p \) = plastic hinge rotation
\( \xi \) = concrete element eccentricity ratio

\( \rho \) = longitudinal reinforcement ratio

\( \rho \) = wall tension end longitudinal reinforcement ratio (Wallace and Moehle, 1992)

\( \rho' \) = wall compression end longitudinal reinforcement ratio (Wallace and Moehle, 1992)

\( \rho'' \) = wall web longitudinal reinforcement ratio (Wallace and Moehle, 1992)

\( \phi_u \) = wall ultimate curvature

\( \phi_y \) = wall first-yield curvature
ACKNOWLEDGEMENTS

I would like to first and foremost thank my advisor, Professor John W. Wallace, for his support and guidance throughout my Ph.D. studies at the University of California, Los Angeles. I would also like to express my sincere appreciation for the advice, insights, and recommendations provided by the members of the Doctoral Committee: Professors Jonathan P. Stewart, Ertugrul Taciroglu, Thomas A. Sabol, and Anne Lemnitzer.

A significant portion of this research was conducted with funding provided by the Consortium of Universities for Researching Earthquake Engineering (CUREE) for the National Earthquake Hazards Reduction Program (NEHRP) Consultants Joint Venture Earthquake Structural and Engineering Research for the National Institute of Standards and Technology (NIST), Task Order #21. Experimental tests conducted in the UCLA Structures/Earthquake Engineering Laboratory were funded by NIST, NEHRP Consultants Joint Venture, Task Order 25, and by the National Science Foundation (NSF) Grant CMMI 1208192. Experimental research was performed in a laboratory renovated with funds provided by the National Science Foundation under Grant No. 0963183, which is an award funded under the American Recovery and Reinvestment Act of 2009 (ARRA).

I would also like to take this opportunity to thank Steve Keowen for his invaluable assistance throughout the preparatory, construction, and installation phases of the specimen tests. I thank Dr. Alberto Salamanca for his guidance and help with specimen test control setup, loading system operation, and data acquisition. I also would like to thank Harold Kasper for his lab equipment training and help with specimen construction and material testing, and Ian Wallace for his help with reinforcement cage fabrication.

xxii
Other individuals that deserve special recognition include colleagues Chris Segura, Chris Motter, Dr. Thien Tran, and Dr. Zhe Qu for all of their help with planning, design, construction, and documentation of the laboratory tests. I very much appreciate the efforts and contributions of NEES summer interns Jasmin Sadegh, Alfredo Pineda, Anna Flintrop, Marissa Shea, Alex Arroyo, and Ramin Cohen through the various phases of my research.

Lastly, I would like to thank my parents and family for their enduring support and encouragement in my academic career. I especially thank my wife, Jaclyn, for her understanding and patience throughout my Ph.D. studies.
VITA

2006 B.S., Structural Engineering
University of California, San Diego

2006-2009 Design Engineer
Ficcadenti and Waggoner Structural Engineers
Irvine, CA

2009 Professional Engineer, Civil
State of California

2010 M.S., Civil Engineering
University of California, Los Angeles

2009-2014 Graduate Student Researcher
Department of Civil and Environmental Engineering
University of California, Los Angeles

2012-2013 Mentor
NEES Summer Intern Program
University of California, Los Angeles

2011-2014 Teaching Assistant
Department of Civil and Environmental Engineering
University of California, Los Angeles
PUBLICATIONS AND PRESENTATIONS


CHAPTER 1    INTRODUCTION

1.1    General

Reinforced concrete structural walls, also known as shear walls, are a prevalent lateral force-resisting system used in buildings in the United States and throughout the world, especially in low to mid-rise buildings (Eberhard and Meigs, 1995). Structural walls can be configured in various thicknesses, heights, and cross-sections (both symmetric and asymmetric) as well as with different reinforcement patterns, depending on the demands and layout used and the design requirements. Reinforced concrete structural walls are typically characterized as having relatively large in-plane stiffness, which limits lateral drifts that are associated with building damage. The generally good performance of concrete structural walls in earthquakes also explains the popularity of this system among structural engineers throughout the world.

Major advances in the understanding of wall behavior and building codes have often come as a result of building performance observations from significant seismic events. As more information becomes available, either from experimental research or from structural investigations following major seismic events, building codes and design practices are updated and modified to address potential deficiencies detrimental to public safety or that lead to prohibitively expensive repair and recovery costs. The flowchart illustrated in Figure 1.1 (FEMA P-749, 2009) demonstrates the iterative process that large earthquakes induce on devising and implementing building code revisions and hazard mitigation efforts. Major earthquakes are infrequent, and abeyances in significant seismic code advancements often coincide with the duration between large seismic events. The prevalence of reinforced concrete structural wall systems throughout the world, however, exposes the building system to more earthquakes, which in turn allows for more opportunities to observe and assess real performance. Variances in building practices and design codes, individual building performance, and ground motion information allow engineers to
differentiate between effective design requirements and potential dangers, with the ultimate goal of contributing to the safer design. The performance of mid-rise buildings with reinforced concrete structural walls in the 2010 $M_w$ 8.8 Maule earthquake in Chile is one such opportunity to investigate U.S. design provisions.

Figure 1.1 Earthquake risk mitigation process (FEMA P-749, 2009).

(a) 18-story building in Santiago (-1 level)  (b) 10-story building in Vina del Mar (ground)

Figure 1.2 Typical wall damage in 2010 Chile earthquake.
Post-earthquake reconnaissance reported significant damage at the boundary elements of many walls in multiple buildings throughout central Chile, as shown in Figure 1.2. Given the large number of modern reinforced concrete buildings that exist in the impacted region, the February 27, 2010, $M_w$ 8.8 Chile earthquake is an excellent opportunity to assess the performance of reinforced concrete buildings designed using modern codes similar to those used in the United States. In conjunction with assessments of in-situ wall performance, a series of laboratory experiments were conducted at UCLA to further investigate structural wall boundary element performance under reverse-cyclic demands. The intent of these analytical and experimental studies is to evaluate and improve the efficacy of U.S. provisions designed to inhibit structural damage at wall boundaries.

1.2 Scope and Objectives

Research presented in the following chapters aims to investigate the performance of boundary elements of concrete structural walls under high seismic demands. This investigation includes analyses of structural components from Chilean buildings whose damage was documented following the 2010 $M_w$ 8.8 earthquake. A chief concern is to determine how concrete shear wall boundary zones performed with respect to fulfilling current U.S. code requirements (namely, confinement detailing requirements in ACI 318 Section 21.9). By taking exception to the special boundary element detailing provisions, Chilean code NCh433.Of96 allowed thin wall boundary zones with large transverse reinforcement spacing to be constructed, leaving wall boundaries essentially unconfined. The availability of ground motion information, construction documents, and detailed building analysis enables an investigation of the reliability and effectiveness of the special boundary detailing provisions in ACI 318 by comparing observed damage in these zones with how U.S. practice would dictate reinforcing requirements. Other issues, including plastic hinge length assumptions and longitudinal bar strain histories are examined as well. Boundary element longitudinal reinforcement strain histories are of interest because some reconnaissance reports identified bar buckling at the ends of damaged structural walls, sometimes in the
absence of concrete core crushing. Very large spacing of boundary transverse reinforcement is common in Chilean walls, which translates into long unsupported lengths for longitudinal reinforcement. This brings into question the stability of those bars, especially when they are subjected to large, reverse-cyclic strain demands. A total of seven Chilean buildings were selected to investigate the performance of boundary elements during the 2010 earthquake. Additional wall tests were analyzed (Wallace and Thomsen, 1995) to investigate and calibrate the potential impact of bar buckling in the boundary zones of thin walls.

In addition to the analyses performed on Chilean walls, eight full-scale boundary element prism tests are presented. Specimens were designed to conform to U.S. code provisions for ordinary boundary elements (ACI 318-11 Section 21.9.6.4) in structural walls and also reflect common Chilean construction. The test prisms were subjected to reverse-cyclical axial loading until strength loss occurred. All eight specimens had the same overall cross-section and same longitudinal reinforcing steel. Horizontal reinforcement and loading histories vary to study parameters impacting the onset of member strength loss, either by concrete crushing, localized member lateral instability, or localized instability initiated by rebar buckling.

1.3 Organization

This dissertation is divided into nine chapters and two appendices. A brief overview of Chilean building practice and code requirements, and a comparison of these practices with current U.S. codes—particularly the displacement based design provisions used in ACI 318-11 Section 21.9.6.2—are presented in Chapter 2. A literature review of relevant research of topics related to reinforcing bar buckling, stability of wall boundaries, and boundary element performance in select reinforced concrete structural wall tests are also included in the chapter. Methods implemented to analyze the response of reinforced concrete structural walls in several Chilean buildings are discussed in Chapter 3. These approaches include modeling
assumptions for determining section properties critical in special boundary element detailing provisions as well as developing strain history predictions for boundary longitudinal reinforcement to assess the likelihood of bar buckling. Chapter 4 includes a review of the seven Chilean buildings chosen for investigation and provides detailed information about geometries and configurations of candidate walls selected for analysis. A summary of analysis results for each building are also contained in Chapter 4. The experimental test setup for the eight specimens examined at UCLA is described in Chapter 5, with information included on the test matrix, specimen design and construction, and material properties. Information about the loading system assembly, loading protocols, and data acquisition system is given in Chapter 6, whereas test results are provided in Chapter 7. Chapter 8 includes an investigation of parameters that affect the behavior at critical sections of structural walls—such as the role of boundary longitudinal reinforcement strain history—and recommendations for spacing of transverse reinforcement needed to restrain rebar buckling based on strain history. Chapter 9 provides an overall summary of research findings and offers recommendations and conclusions. Additional information on the Chilean wall analyses and additional data obtained from the UCLA laboratory tests are summarized in Appendix A and Appendix B, respectively.
CHAPTER 2   LITERATURE AND RECONNAISSANCE REVIEW

2.1   Chilean Building Practices Overview

Massone et al. (2012) and Wallace et al. (2012) describe the evolution of Chilean design and construction practices over the last thirty years, as well as the damage experienced in the 1985 and 2010 events. The March 1985 M₈ 7.8 earthquake in Chile had a significant impact on the design practices and building code provisions at the time. Despite strong shaking, mid-rise concrete wall structures performed extremely well, exhibiting little to no damage in the majority of nearly 400 modernly-detailed reinforced concrete walls. In pre-1985 buildings, structural walls in each principal direction occupied approximately 3% of the building floor plan area, and they were commonly located in nearly every unit-dividing partition. Orthogonal walls were commonly interconnected, forming flanged T- or L-shape cross sections. The numerous walls preclude column presence in most buildings, and these walls would be classified as bearing-type walls in U.S. practice. Most structures were less than 15 stories tall, and walls were typically 20-30cm thick (roughly 8-12 inches). First floor horizontal setbacks (vertical discontinuities) in walls were also fairly common practice. The large amount of walls and relatively low axial load demands probably explain why observed damage in walls was minimal following the 1985 event.

Following the 1985 earthquake, Chile adopted a new building code in 1996: NCh433.Of96. This code adopted earthquake load analyses similar to methods in ASCE 7-10, including equivalent static lateral force and modal response spectrum procedures. Most buildings taller than 5 stories were designed by modal analysis. Design spectra are defined similarly to ASCE 7, where fault zone proximity, lateral force resisting system type, and site conditions influence spectral acceleration and spectral displacement design
values; however, the Chilean spectral shape is much different from U.S. design spectra due to the subduction hazard.

Design requirements for concrete structures in NCh433.Of96 directly referenced ACI 318-95, with relatively few exceptions. One key exception was provisions for special boundary element (SBE) transverse reinforcement to provide concrete confinement and longitudinal bar restraint. Because concrete buildings had performed so well in the 1985 earthquake, the SBE provisions were eliminated. This allowed for significantly less transverse reinforcement at wall ends, which is described below. Another exception (more relevant to ASCE 7) was that vertical structural irregularities, i.e. wall setbacks, were permissible (and prevalent) in Chilean design.

Between 1985 and 2009, more than 1900 new building permits were issued in Chile for reinforced concrete structures with 9 or more stories. While most structures were less than 15 stories tall prior to the 1985 earthquake, the majority of mid-rise buildings built after 1985 consisted of 15 to 25 stories. Additionally, average wall thicknesses decreased: whereas common wall widths were 20 to 30 cm thick pre-1985, typical walls post-1985 were 15 to 20 cm thick. The increase in building height and reduction in wall thickness led to higher axial demand ratios in newer buildings relative to older structures (the wall plan area to total floor plan area ratio remained roughly constant for both pre- and post-1985 construction). Another critical issue in Chilean design was that since special boundary element detailing considerations were not required, nominal transverse reinforcement was provided at wall boundaries. Typical construction consisted of four 18 to 25mm diameter longitudinal bars at wall ends, with web horizontal reinforcement spaced 20 to 25 cm apart terminating with 90 degree hooks around the longitudinal end bars (see Figure 2.1).
The subsequent Chilean building code, NCh430.Of2008, refers to ACI 318-05, and in a departure from NCh433.Of96, did not take exception to the provisions that dictate special boundary detailing. However, due to the delay in the release of the newer code and its use in Chilean practice, very few, if any, existing buildings were built to the new code standards at the time of the 2010 earthquake. Consequently, damage to reinforced concrete structural walls, particularly at or near ground level, was significant and widespread in mid-rise buildings in Santiago, Viña del Mar, and Concepción in the 2010 M$_{w}$ 8.8 earthquake. It is important to note that the spectral displacement amplitudes of the ground motions recorded in Santiago and Viña del Mar were similar for the 1985 and 2010 earthquakes. On the other hand, much larger spectral displacement values were obtained for the ground motions in Concepción (between two to four times the code level spectra, depending on site conditions). In general, crushing and spalling of concrete and buckling of vertical reinforcement were observed, often over nearly the entire wall length. Damage was typically concentrated over a short height equal to one to three times the wall thickness, likely because buckling of vertical bars led to damage concentration. Because structural walls used in Chile are thin—typically 15 to 20cm thick—any spalling of cover concrete (about 2 cm on each side) results in roughly a 25% reduction in the wall thickness. Once cover concrete spalls, the 90-degree
hooks used on transverse reinforcement at wall boundaries would open, becoming ineffective in confining concrete and providing vertical bar stability. The large spacing of transverse reinforcement and the opening of 90-degree hooks after concrete cover spalls likely contributed to buckling of vertical reinforcement.

![Figure 2.2 Transverse reinforcement terminated by 90 degree hooks at wall boundary.](image)

The 2010 Chilean earthquake provides a special opportunity to investigate boundary element detailing provisions used in the United States for concrete buildings designed using ACI 318-11 since the governing Chilean design code for new reinforced concrete shear wall buildings after the 1985 earthquake did not require special detailing. This differentiation from U.S. code design, and the observed performance during a major earthquake in structures designed using approaches very similar to those used in the U.S., offers an opportunity to evaluate the efficacy of U.S. code provisions. In particular, it is of interest to assess whether ACI 318-11 provisions, when applied to walls with and without observed damage due to the 2010 earthquake, would indicate the need for special boundary elements in damaged walls and the lack of need for special boundary elements in undamaged walls.
2.2 Structural Wall Boundary Element Displacement Based Design in U.S. Codes

Current provisions for “Special Structural Walls” are contained in ACI 318-11 §21.9 and include provisions for reinforcement (§21.9.2), shear strength (§21.9.4), design for flexural and axial loads (§21.9.5), and boundary elements of special structural walls (§21.9.6). This section focuses on the provisions of ACI §21.9.6. Wall compression stresses (§21.9.6.3) or neutral axis lengths (§21.9.6.2) are compared to specified limits to determine if special boundary elements are required at wall boundaries of various Chilean buildings. If Special Boundary Elements (SBEs) are required, the quantity and distribution of required transverse reinforcement is based on §21.9.6.4; otherwise requirements of §21.9.6.5 apply. Where SBEs are not required, in this document, we refer to the region at the wall boundary as an “ordinary boundary element”, or “OBEs”.

In the model used to develop ACI 318-11 §21.9.6.2 provisions, the design displacement \( \delta_e \) is related to local plastic hinge rotation \( \theta_p \) and extreme fiber compressive strain \( \varepsilon_c \) as:

\[
\theta_p = \frac{\delta_e}{h_w}; \quad \theta_p = \left( \phi_u = \frac{\varepsilon_c}{c} \right) \left( l_p = \frac{l_w}{2} \right) \therefore \varepsilon_c = 2 \left[ \frac{\delta_e}{h_w} \right] \left[ \frac{c}{l_w} \right]
\]

Where \( l_p \) is the plastic hinge length, \( h_w \) is the wall height, \( c \) is the neutral axis depth for \( (M_n, P_{n,max}) \), and \( l_w \) is the wall length. The plastic hinge length of the wall is assumed to be one-half the length of the wall. If the compressive strain exceeds a limiting value, assumed to be 0.003, then special transverse reinforcement is required. ACI 318-11 Equation (21-8) rearranges the above expression to define a limiting neutral axis depth, instead of a limiting concrete compressive strain, when special transverse reinforcement detailing is necessary:
In this approach, it is obvious that the need for SBEs is sensitive to the values used for the design
displacement and plastic hinge length, as well as the value assumed for limiting concrete compressive
strain. If the neutral axis depth \( c \) computed for the nominal moment and the largest axial load for the
specified load combinations \( (M_n, P_{u,\text{max}}) \) is less than \( c_{\text{limit}} \), then SBEs are not required. The value of \( \delta_u / h_w \)
is not allowed to be taken less than 0.007. This 0.007 minimum drift level can be translated into a neutral
axis depth provision: rearranging equation 2.2 with 0.007 as \( \delta_u / h_w \) requires special boundary detailing if
the neutral axis depth exceeds 0.238 times the overall wall length. It is noted that this “displacement-
based” approach was added to ACI 318-99 based on research following the 1985 M7.8 earthquake in
Chile (Wallace and Moehle, 1992; Wallace, 1996; Wallace and Orakcal, 2002).

Alternatively, ACI 318-11 §21.9.6.3, which uses a nominal value of computed compressive stress at the
wall edge to determine the need for SBEs, may be used. The compressive stress is computed as:

\[
f_c = \frac{P_u}{A_g} + \frac{M_u c}{I_g} \leq 0.2 f_c
\]  

2.3

\( P_u \) and \( M_u \) are the design (factored) axial load and moment, respectively, that produce the largest
compressive stress, \( A_g \) is the wall gross area, \( I_g \) is the gross concrete moment of inertia, and \( c \) is the
distance from the elastic centroid to the wall edge. The ACI commentary notes that (2.3) “is used as an
index value and does not necessarily describe the actual state of stress that may develop at the critical
section.”

Both the displacement-based approach and stress-based approach assume that once the extreme
compressive fiber exceeds a certain limit (stress or strain), additional reinforcement needs to confine the
concrete to prevent significant damage to the wall. If SBEs are required, the quantity and distribution of required transverse reinforcement is defined in §21.9.6.4; otherwise, requirements of §21.9.6.5 apply. The height of the wall over which the SBEs are required is defined in §21.9.6.2 for the displacement-based approach and §21.9.6.3 for the stress-based approach.

As previously noted, the Chilean code NCh433.Of96 refers to ACI 318-95, but includes an exception that SBEs are not required; instead wall boundaries are typically detailed as shown in Figure 2.1. With the absence of special boundary element detailing, the observed damage (or lack thereof) at critical sections of walls in Chilean buildings provides an opportunity to assess if the ACI 318 §21.9.6.2 provisions effectively identify walls potentially susceptible to damage. Chapters 3 and 4 discuss the investigation undertaken to determine if walls in Chilean buildings would have required SBEs according to ACI 318-11 §21.9.6.2, and then this assessment was cross-referenced with the level of damage observed in those walls.

2.3 Previous Research on Bar Buckling and Boundary Elements in Thin Concrete Walls

The provisions of ACI 318-11 section 21.9.6 primarily focus on confining wall boundary elements with large compression demands to prevent or delay concrete core crushing at the wall ends; restraint against rebar buckling is addressed for special boundaries by limiting the spacing of transverse reinforcement to six times the longitudinal bar diameter and for ordinary boundaries by limiting the spacing to eight inches (20 cm). Rebar buckling is not addressed in Chilean code provisions. For OBEs, the wide spacing of transverse reinforcement translates into relatively large unbraced lengths for the longitudinal reinforcing bars. If earthquake shaking imparts significant tensile and compressive strain demands to the longitudinal reinforcement, prior research, discussed in the following section, has shown that the bars are susceptible to buckling, which typically leads to significant loss of wall strength.


2.3.1 Experiments Characterizing Buckling Onset of Isolated Reinforcing Bars

Various experiments have been conducted to explore the stability of longitudinal reinforcement. The literature review presented here focuses on reverse-cyclic experimental studies of reinforcing bars. Many of these experiments were performed on isolated reinforcing bars, that is, the bars were not surrounded by or embedded in concrete. Monti and Nuti (1992) subjected individual reinforcing bars with fixed ends and length-to-diameter ratios ($L/D$) of 5, 8, and 11 to reverse cyclic loading histories. The steel material conformed to the Italian material specification FeB44, with nominal yield strength of 440 MPa (63.8 ksi). The length of tested coupons between fixed ends represented the unsupported span between sufficiently stiff transverse ties in a reinforced concrete element, and the length to diameter ratios chosen represented the most commonly used ratios in construction. They reported inelastic buckling of bars developed for $L/D$ values greater than 5 and up to 11. The post-buckling behavior was characterized as isotropic softening, reduced compression loading branch curvature and slope, and post-buckling stiffness predominantly characterized by flexural stiffness (as opposed to axial stiffness). As the length-to-displacement ratios increased, hysteretic cycle contraction occurred earlier in compression reloading and peak compression strength reduced. Figure 2.3 illustrates the impact of bar buckling on hysteretic loops for increasing L/D ratios.

![Figure 2.3 Impact of L/D ratios on symmetric cyclic strain (Monti and Nuti 1992).](image)

Rodriguez et al. (1999) investigated bar buckling in reverse cyclic tests performed on isolated reinforcing bars (versus embedded in concrete), to identify the onset of bar buckling. Similar to the experiments
performed by Monti and Nuti (1992), a series of coupons with various unsupported lengths were subjected to reverse-cyclic axial strains. 16 mm diameter coupons were milled from 31.8 mm reinforcing bars with stress-strain behavior conforming to the requirements of ASTM 706. Length-to-diameter ratios of 2.5, 4, 6, and 8 were selected for investigation. The onset of bar buckling was defined as when the difference between strains measured on opposite sides of a specimen met or exceeded twenty percent of the difference between the maximum and minimum cyclic strain limits. Rodriguez et al. (1999) defined parameter $\varepsilon_p^*$ to characterize the initiation of buckling; Figure 2.4 illustrates the strain parameter used to track the onset of bar buckling.

![Figure 2.4 Initial bar buckling identifier parameter $\varepsilon_p^*$ (Rodriguez et al. 1999).](image)

The parameter $\varepsilon_o^{+}$ measures the compression reloading strain from the point of zero axial load to the point of zero strain, whereas the parameter $\varepsilon_p$ represents the strain magnitude from zero strain to the point of initial bar buckling. Figure 2.5 shows that the available strain prior to initial bar buckling ($\varepsilon_p^*$) decreases as the ratio of unsupported length to bar diameter increases. The black (solid) line predicts the onset of bar buckling according to the reduced modulus theory Equation 2.4 with average values of parameters defining a monotonic compression curve of reinforcing steel. Use of an effective length fixity factor $K$ of 0.75 resulted in the best fit between the results obtained using the reduced modulus theory and the test results obtained from reverse cyclic loading with unsupported length to bar diameter ratios of 4, 6, and 8.
(Figure 2.5)—buckling was not observed in the tests with length-to-diameter ratios of 2.5.

\[ f_p = \frac{\pi^2 E_s}{16 \left( K \frac{s}{d_b} \right)} \]

The curves in Figure 2.5 define an initial buckling strain for \( s/d_b \) ratios from 2 to 8. Extrapolating the test results to \( s/d_b \) ratios exceeding 8 to determine the initial buckling strain for longer unsupported lengths does not provide rational results (since \( \varepsilon_p^* \) reaches a value of zero at a \( s/d_b \) value of about 9). A value of \( \varepsilon_p^* \) of zero or less implies that bar buckling occurs when the bar is subjected to tensile stress; the primary reason for this inconsistency is that rebar tested in air does not benefit from the lateral support provided by the concrete that surrounds the reinforcing bar in actual construction.

As part of a study to investigate the failure mechanisms associated with reinforced concrete bridge columns, Hose (2000) performed a series of experiments to investigate buckling behavior of longitudinal reinforcement. Like Rodriguez et al. (1999), these tests were performed on isolated bars, versus rebar embedded in concrete, with various lengths between support points. Test specimens were fabricated from
Grade 60 #7 bars with lengths of 3.5", 5.25", and 7.875", representing length-to-diameter ratios (s/dₘ) of 4, 6, and 9, respectively. Three types of experiments were performed: monotonic, single-cycle tension-compression, and reverse cyclic. For the single-cycle tests, a series of reloading cycles from three different strain levels were performed. Each strain level was tested three times for each bar length, for a total of 27 tests. Predetermined tension strain excursion levels were 2%, 4%, and 6%; once the prescribed tension strain was reached, the bars were reloaded in compression until they buckled. Lateral instability was captured by potentiometers attached to the middle of the specimens, and buckling was defined as the point where initial changes in the slope of lateral displacement versus axial load curve were observed. A fourth-order polynomial was fitted to the axial load versus lateral displacement data points in the range where buckling was observed. Hose identified the strain associated with the root of the third derivative of the polynomial fit (the root of the minimum change in slope) with the initiation of bar buckling. The strain reported was identified as the deviation strain, measured as the difference in strain from the peak tension strain to the strain registered at buckling (Figure 2.6).

![Figure 2.6 Deviation strain to identify bar buckling initiation, as defined by Hose (2000).](image)

This approach differs from that used by Rodriguez et al. (1999), who define their compression reloading strain from the strain at zero axial load to the point where initiation of buckling was observed. The strain
values reported by Hose include unloading from the peak stress to zero stress, whereas the strains reported by Rodriguez et al. (1999) are relative to zero stress. Results from the single-cycle tests performed by Hose are presented in Table 2.1. Hose noted that the maximum tension strain level reached prior to compression reloading did not have a significant impact on the deviation strain reported (i.e., the unloading and compression reloading strain range from peak tension to buckling was independent of the previous tension strain magnitude). Reverse cyclic tests were only performed on three specimens with the smallest \( s/d_b \) ratio (\( s/d_b \) of 4), and the results used to calibrate an analytical model of bar buckling. Only results from one of the experiments were presented in Hose (2000): a figure compared the predicted onset of buckling with the experimental measurement. Based on the figure presented in Hose (2000), a deviation strain of about 3.5% is noted between the peak tensile strain and the point identified as bar buckling.

Table 2.1 Measured deviation strains at buckling from single-cycle tests (Hose, 2000).

<table>
<thead>
<tr>
<th>#7 Bar x 3.5&quot; (s/d_b=4)</th>
<th>( \varepsilon_{t, max} )</th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.0314</td>
<td>0.0252</td>
<td>0.032</td>
<td></td>
<td>0.0295</td>
</tr>
<tr>
<td>4</td>
<td>0.0451</td>
<td>0.0434</td>
<td>0.0365</td>
<td></td>
<td>0.0417</td>
</tr>
<tr>
<td>6</td>
<td>0.0409</td>
<td>0.0401</td>
<td>0.0431</td>
<td></td>
<td>0.0414</td>
</tr>
<tr>
<td>Average of 9 Tests</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.0375</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>#7 Bar x 5.25&quot; (s/d_b=6)</th>
<th>( \varepsilon_{t, max} )</th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.0223</td>
<td>0.0156</td>
<td>0.026</td>
<td></td>
<td>0.0213</td>
</tr>
<tr>
<td>4</td>
<td>0.0254</td>
<td>0.025</td>
<td>0.026</td>
<td></td>
<td>0.0255</td>
</tr>
<tr>
<td>6</td>
<td>0.0259</td>
<td>0.0258</td>
<td>0.023</td>
<td></td>
<td>0.0249</td>
</tr>
<tr>
<td>Average of 9 Tests</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.0239</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>#7 Bar x 7.875&quot; (s/d_b=9)</th>
<th>( \varepsilon_{t, max} )</th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.0155</td>
<td>0.01145</td>
<td>0.0147</td>
<td></td>
<td>0.0139</td>
</tr>
<tr>
<td>4</td>
<td>0.0127</td>
<td>0.0144</td>
<td>0.0143</td>
<td></td>
<td>0.0138</td>
</tr>
<tr>
<td>6</td>
<td>0.0207</td>
<td>0.01949</td>
<td>0.0185</td>
<td></td>
<td>0.0196</td>
</tr>
<tr>
<td>Average of 9 Tests</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.0157</td>
</tr>
</tbody>
</table>
In addition to the results in Table 2.1, deviation strains are plotted in Figure 2.7 to compare with results from other studies of cyclic bar buckling. Note that the values reported by Hose inherently include an additional 0.002 strain at minimum, since Hose’s deviation strain includes elastic recovery whereas the other models exclude it (0.002 is an estimate based on recovering yield strain—the discrepancy is probably larger if considering elastic recovery after strain hardening at larger tension excursions). A power fit of the test data for reloading strain versus spacing-to-bar-diameter ratio can be approximated as:

\[
\varepsilon_{\text{dev}} = \frac{1}{6} \left( \frac{s}{d_b} \right)^{-1}
\]

Moyer and Kowalsky (2003) also conducted a study to determine the compressive strain reinforcing bars could withstand prior to buckling. As part of research conducted to characterize buckling in full-scale bridge columns, a series of tests on isolated reinforcing bars were conducted on specimens loaded to different levels of tension strain prior to load reversal into compression. Little difference in compression reloading behavior was noted from tests with peak tension strains ranging from 0.5% to 12%. The objective of these tests was to estimate a modified secant-stiffness to use with a double modulus theory approach to quantify the compression strain achieved prior to buckling for an isolated reinforcing bar tested in air, that is:

\[
\frac{s}{d_b} = \frac{\pi}{4K} \sqrt{\frac{E_d}{f_{\text{max}}}}
\]

In Equation 2.6, \(K\) represents the effective length factor, \(E_d\) represents the double modulus, and \(f_{\text{max}}\) represents the steel stress. Data from a compression reloading test on an isolated reinforcing bar pre-loaded to 6% tension strain were used to fit a function that related bar compression strain capacity prior to buckling, \(\varepsilon_{cc}\), to longitudinal bar diameter and transverse hoop spacing (Equation 2.7, \(K=1\)). Equation 2.7 is plotted against the lower bound estimate of initiation of bar buckling trend established by Rodriguez et al. (1999) in Figure 2.7.
\[ \varepsilon_{cc} = 3 \left( K \frac{s}{d_b} \right)^{-2.5} \]

Figure 2.7 Comparison of relationships and data for isolated bar buckling in air.

The bar buckling trends established by Rodriguez et al. (1999) and Moyer and Kowalsky (2003) presented in Figure 2.7 give similar bar strain capacities prior to buckling, especially for \( s/d_b \) values between 5 and 7. For \( s/d_b \) values greater than 7, the relationship established by Moyer and Kowalsky (2003) gives a larger compression strain capacity prior to buckling than the relationship developed by Rodriguez et al. (1999). While both relationships prove useful in characterizing the initiation of buckling in isolated reinforcing bars, they do not take into account the influence surrounding concrete has on bars in a reinforced concrete element, which is likely important for \( s/d_b \) ratios greater than 8. Interestingly, the strain values reported by Hose (2000) at buckling are smaller than the models established by Rodriguez et al. (1999) and Moyer and Kowalsky (2003, discussed below). This may be explained by the approach used to define buckling of the isolated bars: the polynomial fit was sensitive to the displacements detected by potentiometers, and very-small out-of-plane displacements might have led to (premature) identification.
of bar buckling. Hose acknowledged that the approach was conservative, i.e., it did not necessarily indicate when axial strength loss occurred due to bar instability.

### 2.3.2 Out-of-Plane Instability

The above section provides a review of selected studies on longitudinal reinforcement stability; previous research has also focused on the lateral stability of thin wall boundaries subjected to reverse cyclic loading. Where the longitudinal reinforcement at the boundaries of structural walls experiences high tension strain demands, even with tight hoop spacing to confine concrete and restrain bars from buckling, there is a chance that the end of the wall can displace out-of-plane upon reloading into compression, potentially resulting in lateral instability of the entire wall section, referred to here as “global buckling” of the wall end. Studies by Paulay and Priestley (1993) and Chai and Elayer (1999) explore this issue.

Paulay and Priestley (1993) investigated the mechanism driving out-of-plane displacement and lateral instability of the wall within the plastic hinge region. They noted that out-of-plane buckling is a function of both the geometry (wall story height, wall width) and the longitudinal reinforcement strain history in the wall boundary. Out-of-plane displacements (lateral instability) can develop at wall boundaries where the longitudinal bars have been subjected to tensile strain demands much greater than yield. As reinforcing bars are loaded beyond yield in tension, horizontal cracks develop along the height of the wall. During unloading, bar stress reduces; however, the cracks opened during tensile yielding may not close. If cracks do not close, under reverse loading into compression, the internal compression force must be transferred through the longitudinal reinforcement until crack closure occurs. Due to asymmetry in the location of longitudinal reinforcement and the crack surface, bar yielding in compression and crack closure may occur for one layer of reinforcement prior to the other, creating eccentricities of axial load
that result in out-of-plane curvature and displacement at the boundary element. Paulay and Priestley (1993) derived a criterion for the onset of global instability to address this issue; the resulting relation is given in Equation 2.8 and 2.9:

\[ \xi \leq 0.5 \left( 1 + 2.35m - \sqrt{5.53m^2 + 4.70m} \right) \tag{2.8} \]

\[ \xi = \frac{\varepsilon_{sm} \left( \frac{h}{b} \right)^2}{8\beta} \tag{2.9} \]

\( \xi \) represents an eccentricity ratio and \( m \) is the mechanical reinforcement ratio, \( \rho f_y / f_c \) at the wall boundary. The eccentricity ratio is a function of maximum tension strain, \( \varepsilon_{sm} \), height-to-thickness ratio (\( h/b \)), and the distance from the non-compressing layer of reinforcement to the point of initial crack closure as a ratio of total wall thickness, \( \beta \). The maximum tension strain that can be withstood by the wall prior to out-of-plane instability can be obtained by rearranging Equations 2.8 and 2.9 as:

\[ \varepsilon_{sm} \leq 4\beta \left( \frac{b}{h} \right)^2 \left( 1 + 2.35m - \sqrt{5.53m^2 + 4.70m} \right) \tag{2.10} \]

In general, as the wall height-to-thickness ratio decreases, the maximum tension strain capacity prior to onset of lateral instability increases. Smaller mechanical reinforcement ratios and larger distances between curtain steel and the point of initial crack closure also increase the maximum tension strain longitudinal bars can undergo without global buckling occurring.

Chai and Elayer (1999) also investigated the lateral stability of reinforced concrete columns subjected to reverse-cyclic axial tension and compression demands to determine the maximum longitudinal reinforcement tension strain prior to the onset of lateral (out-of-plane) instability (Equation 2.11).
The relationship advanced by Chai and Elayer (1999) depends on the height-to-thickness ratio, the yield strain of the longitudinal reinforcing bars, the assumed out-of-plane deformed shape, and the same eccentricity ratio established by Paulay and Priestley (1993). Both Equations 2.10 and 2.11 yield similar maximum tensile strain predictions at small height-to-width ratios \((h/b < 8)\), but diverge for larger ratios (Figure 2.8). The critical tension strain predictions were then compared to test results obtained for reinforced concrete prisms representative of boundary elements at the web end of a flanged wall. The strain values leading to lateral instability in tests of reinforced concrete prisms (columns) configured to be representative of conditions at wall are compared with values obtained with the models (Equations 2.10 and 2.11) in Figure 2.8.

\[
\varepsilon_{sm} \leq \frac{\pi^2}{2 \left( \frac{b}{h} \right)^2} \xi + 3\varepsilon_y
\]

Figure 2.8 Height-to-width ratio, reinforcement ratio influence on critical tension strain for global buckling relationships. Eqn. 8 refers to the relationship from Chai and Elayer (1999) and Eqn. 9 is Paulay and Priestley (1993).

### 2.3.3 Bar Buckling Observed in Reinforced Concrete Elements

The research presented in the previous sections establishes upper and lower bounds for strain limits for longitudinal reinforcing, either for the onset of buckling for an isolated reinforcing bar (no surrounding concrete) or for the onset of lateral instability of the member over a prescribed height. Test results on
isolated reinforcing bars in air provide a lower bound estimate of the expected strain reloading capacity of a longitudinal bar in a reinforced concrete member with similar unbraced length-to-diameter ratios. Likewise, predictive models for member lateral instability of a reinforced concrete section provide an upper bound on reloading strain capacity (since the lateral instability occurs prior to local rebar buckling. While there have been numerous studies that examine bar buckling behavior for isolated reinforcing bars and lateral instability of members, relatively few have focused on bar buckling for reinforcement surrounded by concrete. One explanation for the relative lack of dedicated experimental studies of rebar buckling in concrete elements might be that the longitudinal rebar buckling phenomenon has traditionally been associated as a symptom of concrete cover loss and concrete core deterioration (Pantazopoulou 1998), as opposed to an originating source of damage.

Pantazopoulou (1998) investigated results from over 300 column tests to characterize concrete core deformation capacity, with a focus on the role of longitudinal reinforcement stability. As mentioned above, the relatively few tests that reported strains at buckling of longitudinal reinforcement limited the ability to identify deformation capacity by bar strain. In a mathematical model of column deformation capacity, Pantazopoulou acknowledged that the buckling behavior of longitudinal reinforcement was coupled to the mechanics of confined concrete behavior in the column core. The model presumed failure as crushing and buckling of the core concrete as the ductility achieved from transverse reinforcement exhausted, followed by simultaneous buckling of the longitudinal reinforcement due to the outward pressure of the core. While more than a third of the specimens reviewed consisted of reverse-cyclic laterally loaded columns, bar axial strain reversal history was not one of the parameters emphasized. Instead, the reported axial strain limits relative to s/db ratios were the maximum compressive strain the concrete core could achieve prior to longitudinal reinforcement buckling due to outward expansion of the core combined with axial strain demands (compatibility) on the longitudinal reinforcement. This approach suggested much smaller axial compressive strain capacity of the element (and longitudinal
reinforcement, by virtue of being a component of the prisms) than the reloading strain capacity studies performed on isolated bars. Nevertheless, Pantazopoulou observed similar deformation compatibility trends for longitudinal reinforcement unsupported spans as in the isolated bar experiments: deformation capacity decreased with increasing $s/d_b$ ratios. Furthermore, longitudinal bar buckling was identified as accompanied by unrecoverable strength loss in columns, followed by significant strength degradation in post-peak behavior.

One experimental study that specifically tracked damage associated with longitudinal bar buckling behavior in concrete elements (columns) subjected to reverse cyclic laterally loading column tests was performed by Suda et al. (1996). Strain gauges were affixed to induction-hardened regions of longitudinal reinforcement to track strain histories. The induction hardening process increased the reinforcement yield stress at the location where the strain gauges were attached, thereby protecting the gauges by limiting the magnitude of the bar strain at the gauge relative to the rest of the bar. This process enables bar axial strain to be tracked for very large strain demands, because the bar stress is calculated by knowing the material relationship between the hardened material and the unaltered material. Three of the test specimens had $s/d_b$ ratios of 4.5, whereas the fourth had a ratio of 8. Damage observed in compression cycles involved cracking along the longitudinal reinforcing bar prior to buckling, followed by cover splitting and bar buckling, cover spalling, and subsequently loss of flexural strength. The moment-curvature plot shown in Figure 2.9(a) illustrates the section behavior and indicates damage associated with bar instability.
The axial stress-strain relationship of the longitudinal reinforcement shown in Figure 2.9(b) is similar to the behavior captured in the hysteretic loops of large \( s/d_b \) ratios investigated by Monti and Nuti (1992), i.e., shrinking of the hysteretic response post-buckling. It should also be noted that significant compressive strength loss in the longitudinal bar was observed in cycles after bar buckling was observed, to the point that the bar only resisted roughly one-tenth of the stress prior to buckling. The post-buckling stress-strain relationship advanced from the experiments of Suda et al. (1996) is illustrated in Figure 2.10, where subsequent tension strength (points E, F) of the bars post-buckling is not significantly reduced; however, the maximum compressive stress (point G) is significantly reduced from the peak value upon compression reloading, to only roughly 10% of the stress present when the bar buckled.
Relatively few reinforced concrete shear wall panel tests are available where longitudinal bar buckling is associated with significant loss of lateral strength, primarily because many tests were stopped prior to strength loss. Tests where spacing of transverse boundary reinforcement was varied and observed damage may have initiated due to buckling of longitudinal reinforcement include the specimens investigated by Oesterle et al. (1976), Thomsen and Wallace (2004) and Sittipunt and Wood (1993).

Oesterle et al. (1976) tested nine, one-third scale cantilever wall tests subjected to reversed cyclic lateral loading. Wall cross-sectional shapes included rectangular, barbell, and flanged walls. Three of the nine tests—specimens R1, B1, and B3—lost strength due to buckling of the boundary longitudinal reinforcement and spalling of concrete. Boundary transverse reinforcement for specimen R1, with a rectangular boundary shape measuring four inches wide by six inches deep, consisted of deformed wire hoops spaced 4” apart and enclosed four #3 longitudinal bars (for a s/db ratio of 10.7). Concrete clear cover over longitudinal reinforcing bars was one-half of an inch. The outermost longitudinal bars buckled on the second cycle of +3” top-of-specimen deflection level (lateral drift of 1.7%), followed by buckling of ten additional longitudinal reinforcing bars buckling on subsequent cycles. Limited bar strain data exist for large displacement levels. Damage was concentrated over one or two transverse hoop intervals.
at the wall boundary. Boundary transverse reinforcement for specimen B1, with a barbell cross section with 0.5 inches of clear cover and eight #4 boundary longitudinal bars in the boundary, consisted of hoops spaced at 8” on center (s/d₀ of 16). Boundary longitudinal reinforcing bar buckling of a corner bar near the wall base occurred during the first 3” (1.7% drift) deflection cycle, followed by cover spalling and crushing of the concrete core of the boundary column. Figure 2.11 captures the buckled bars in the barbell shaped boundary element of specimen B1.

![Figure 2.11 Buckled bars in the boundary element of specimen B1 (Oesterle 1976).](image)

Specimen B3 had the same cover and longitudinal reinforcement as Specimen B1, but boundary transverse reinforcement (hoops) were spaced 1.33 inches (s/d₀ of 2.7). Bar buckling did not occur in specimen B3 until the 7 inch (4.0% drift) displacement cycles, and this occurred with a sliding shear failure through the compression boundary element. The buckling of bars in Specimen B3 was probably a symptom of the specimen sliding along a shear plane relative to the base rather than due to vertical instability alone.
Sittipunt and Wood (1993) also reported buckling of longitudinal reinforcement as a probable initiator of boundary element damage in two walls with doubly-flanged cross sections subjected to reverse cyclic loading applied parallel to the short web sections (i.e., asymmetric wall section behavior). Wall webs were three inches thick, and six #3 longitudinal reinforcing bars with hoops spaced at two inches on center were used in the web boundary elements ($s/d_h = 5.3$). Variation of the wall web reinforcing ratio was the primary test variable. Concrete spalling along with slight buckling of the boundary longitudinal reinforcement closest to the wall edge was reported during the cycle to +1.5 inch (1.4% drift) in both tests. Sudden, precipitous lateral strength loss was observed for specimen CLS when the boundary transverse reinforcement fractured, causing more longitudinal bars to severely buckle (Figure 2.12). Sittipunt and Wood report that the transverse reinforcement rupture was due to longitudinal bars buckling in opposite directions. Loss of lateral strength for specimen CMS occurred at an earlier cycle as extensive buckling of boundary longitudinal reinforcing bars manifested. Transverse reinforcement did not fracture and strength loss was less abrupt because adjacent longitudinal bars buckled in the same direction; still, applied lateral loads only reached approximately 60% of the maximum applied load prior to strength loss.

![Figure 2.12 Force-displacement plots for Walls CLS and CMS (Sittipunt and Wood, 1993).](image-url)
Thomsen and Wallace tested four wall specimens: two walls with rectangular cross section, RW1 and RW2, and two walls with T-shaped cross sections, TW1 and TW2. The rectangular walls and the web stems of the T-shaped walls were four inches thick, with eight #3 boundary longitudinal reinforcing bars. Hoops and crossties in the boundary elements of RW1 and TW1 were spaced at 3 inches ($8d_b$). Boundary transverse reinforcement for specimen RW2 consisted of a single hoop spaced at 2 inches on center ($s/d_b = 5.3$), whereas hoops and crossties in the web stem of TW2 were spaced at 1.25 inches ($s/d_b = 3.3$). Minor spalling of concrete at the wall edge of RW1 occurred during the 1% lateral drift cycles, and significant loss of lateral load capacity was observed at around 2.5% drift due to buckling of boundary longitudinal reinforcement. RW2 attained larger drifts prior to strength degradation compared to RW1, which the authors attributed to closer hoop spacing delaying the onset of buckling of the boundary longitudinal reinforcement. Specimen TW1 experienced abrupt strength loss when the web stem was loaded in compression at approximately 1.25% lateral drift due to abrupt buckling of all boundary longitudinal reinforcement and several pairs of web longitudinal reinforcement. Specimen TW2, where detailing was determined using a displacement-based approach that accounted for the T-shaped cross section, did not exhibit bar buckling due to the tight spacing of its boundary transverse reinforcement; strength loss for TW2 was initiated due to out-of-plane instability in the highly compressed wall edge after substantial spalling of cover concrete. Figure 2.13 illustrates the wall damage at the critical boundary elements of all four test specimens performed by Thomsen and Wallace (1995).
The experiments performed by Thomsen and Wallace (1995, 2004) are investigated later (Chapter 3) to validate the approach developed to assess buckling of wall longitudinal reinforcement. Longitudinal reinforcement axial strain predictions are compared with measured bar strain to assess the accuracy of the approach developed.

Based on the observed wall damage in Chilean buildings following the 2010 earthquake, a review of research reported for buckling of isolated longitudinal reinforcement and buckling of boundary longitudinal reinforcement in tests of isolated structural walls, and a review of U.S. code provisions, several key issues were identified. Items of concern include whether the provisions of ACI 318 21.9.6 are sufficient to assess whether boundary transverse reinforcement is needed (i.e., does the provision serve as an effective indicator for damage if special boundary element detailing is required but not provided); can longitudinal bar buckling occur prior to spalling of concrete cover, i.e., can buckling of wall boundary longitudinal reinforcement initiate lateral strength degradation; and what loading history or mechanisms lead to localized lateral instability and subsequent member strength degradation. The following chapters document efforts to answer these questions.
CHAPTER 3   CHILEAN WALL ANALYSIS APPROACH

With significant damage to reinforced concrete walls observed after the 2010 Chilean earthquake, a chief concern is to determine how concrete shear wall boundary zones performed with respect to fulfilling current U.S. code requirements (namely, ACI 318-11 Section 21.9). By taking exception to the special boundary element detailing provisions, Chilean concrete codes prior to 2008 allowed use of relatively light transverse reinforcement at wall boundaries compared to U.S. practice (e.g., 20 cm vertical spacing in Chilean walls. Information from reconnaissance efforts offers an excellent opportunity to assess the performance of reinforced concrete buildings designed using modern codes similar to those used in the US. This chapter describes in detail the processes undertaken to (1) evaluate the ACI 318-11 Section 21.9.6.2 trigger equation for determining if special boundary elements (SBE) are required for candidate walls, and compare those assessments to wall damage documented in post-earthquake reconnaissance; and (2) determine the likelihood of longitudinal reinforcement undergoing cyclic axial strain demands capable of forcing bar buckling between wall transverse reinforcement intervals. The investigations presented in Chapters 3 and 4 were conducted as part of NEHRP Consultants Joint Venture Task Order #21 for the National Institute of Standards and Technology (NIST).

In total, seven mid-rise buildings ranging from 10 to 22 stories in height—some with underground parking and others built at grade level—were selected for investigation: three structures in Concepción, one from Viña del Mar, and three in Santiago. Damage ranged from total collapse or concrete crushing (cover and core) and bar buckling across entire wall lengths to minor concrete spalling or cracking. Basic information regarding structural heights, number of stories, and site conditions are presented in this chapter as part of a displacement based approach; Chapter 4 presents additional information on wall configuration, geometry, and sustained damage available from design documents and for damage reports each candidate building and summarizes results from the approaches developed herein for studies on
confinement trigger assessments and the potential for longitudinal bar buckling at the boundaries of 27 reinforced concrete structural walls of varying geometry, level of axial load, and reinforcement quantities and detailing. Both SBE trigger checks and longitudinal reinforcement buckling assessments have components based on estimates of roof-level displacement; therefore, prior to presenting the analytical approaches, the procedure used to estimate Chilean building drifts is reviewed.

3.1 Building Roof Displacement Estimation

Expected roof drifts for the candidate buildings were estimated using a simple method consistent with the “target displacement approach” in Chapter 3 of ASCE-41 (2006) and recommendations of Shimazaki and Sozen (1985). First, the ground motion record closest to a given building site was identified (for consistent soil conditions). A critical damping ratio of 2% was selected for generating linear displacement response spectra for the horizontal components of recorded ground motions. Although a larger damping ratio (e.g., 5%) could have been used, a rationale for using 2% is provided in the following paragraph. The impact of using a damping ratio of 2% versus 5% on the results obtained with the assessments approach presented in this Chapter is discussed later.

PEER-ATC 72-1 (2010) summarizes modeling criteria for seismic design and analysis of tall buildings, and presents research findings that suggest critical damping ratios for mid rise structures around 2% to 3% are more appropriate than the typically assumed 5% for modeling structural response. Various studies reviewed in PEER-ATC 72-1 (2010) report that initial, undamaged response of structures with reinforced concrete lateral force resisting systems typically have damping near 2%. It is assumed that the buildings selected for investigation were undamaged at the beginning of the Maule earthquake. As displacement demands exceed the lateral force resisting system elastic limits, damping ratio increases and damage
accrues. ASCE 41-06 and Shibata and Sozen (1974) offer relations for scaling spectral response for different damping ratios: the spectral response with 2% damping is approximately 1.23 times larger than the response with 5% damping according to ASCE 41, and 1.38 times larger according to Shibata and Sozen (1974), in regions of the spectrum controlled by velocity and displacement (i.e., not at very short periods). It should also be noted that as damage accrues, stiffness is lost and the fundamental period of the structure increases, and this period increase typically results in higher spectral displacements. Shimazaki and Sozen (1985) show that for the damage ratio that would typically generate 5% damping, the fundamental period would increase by more than 17%. The displacement response spectra in Concepción, Viña del Mar, and Santiago increase in spectral values at the period ranges associated with mid-rise buildings, so the decrease in spectral displacement associated with larger damping ratios due to damage is mitigated by the increase in response due to larger fundamental periods. It is also acknowledged that soil-structure interaction affects damping, but it is difficult to quantify: while the spectral response would decrease with increased damping, overall displacement tends to increase with soil-structure interaction (SSI) modeling (FEMA P-750, 2009). SSI is not a consideration in the approach presented in this section.

Four recording stations were selected based on their proximity to the seven structures chosen for investigation. Orientations, locations, and geotechnical information for recording stations were reported by Boroschek et al. (2012). Building orientations were obtained through reports (NIST GCR 12-917-18, Lemnitzer et al., 2014), photos from post-earthquake reconnaissance from ATC investigators, or by visual inspection of maps using the program Google Earth. The station selected in Concepción was relatively close to all three buildings (Buildings #1, #2, and #3)—within 0.7 mi (1.1 km). The recording channel orientations coincided with the general city plan (street layout). Channel 1 was oriented about 60 degrees clockwise from north; channel 3 was perpendicular to channel 1. The soil conditions at Building #1 were classified as Soil Classification SII, although a post-collapse investigation suggested SIII was more
appropriate for the soil conditions present at the site. Soil conditions for both Buildings #2 and #3 were designated SIII in design documents. The soil conditions at the recording station were characterized as silty sand and had a $V_{S30}$ of roughly 230 m/s. This information, along with the soil conditions at surrounding locations, would suggest the site conditions at the Concepción recording station would be classified as SIII in Chilean code, and Site Class D under NEHRP provisions.

At a distance of only 0.43 mi (0.7 km), the Viña del Mar-Centro station was relatively close to Building #4. Channel 1 of this station was oriented in the East-West direction, and channel 2 was aligned in the North-South direction. Site class conditions at Building #4 were classified as SIII (NIST GCR 12-917-18), and information about soil conditions at the Viña del Mar-Centro station—alluvium and sand material, $V_{S30}$ of 273 m/s—suggest SIII or NEHRP Site Class D conditions.

Most of the structures selected from Santiago were not as close to recording stations as the buildings in Concepción or Viña del Mar, with distances ranging from 5 to 15 km from a recording station. The two locations selected for earthquake records were Santiago-Centro and Santiago-Peñalolen, based on their closer proximity to the case-study structures. Both of these stations had their channels aligned with cardinal directions. Channel 1 for the Centro recorder was oriented East-West, and channel 3 was North-South. For the Peñalolen recording station, channel 1 recorded East-West shaking and channel 2 monitored North-South shaking. Site conditions at Buildings #5, #6, and #7 were all classified as SII. Soil conditions at the Peñalolen station were clay and gravel, with two different $V_{S30}$ values reported (390 m/s and 276 m/s). These reported $V_{S30}$ values correspond to either Site Class C or D according to NEHRP provisions. The Centro station was classified as gravel and Site Class C.

Building and recording station orientations are summarized along with the locations of earthquake recording stations in Figure
3.1. Axes labeled “L” and “T” represent the principal longitudinal (major) and transverse (minor) axes, respectively, of the selected buildings.

Figure 3.1 Summary of Chilean building and recording station orientations.

Figure 3.2 through Figure 3.4 show linear response spectra with 2% damping generated from the records of the four recording stations. As the walls investigated in this study were all oriented in the transverse direction of the buildings (observed wall boundary damage was found in walls with lengths oriented predominantly in buildings’ transverse direction), the recordings used for determining spectra corresponded to the direction most closely matching the wall orientation. Spectra from both Santiago recording stations were considered for determining displacement demands for Buildings #5, #6, and #7:
an average value obtained using both spectra was used. Spectral displacements obtained from the Concepción recording dramatically increase for periods greater than approximately 1.2 seconds. Both the city-grid north-south and east-west directions exhibit this jump in spectral displacement, but the north-south direction response spectrum has a notably larger maximum displacement than the east-west direction. Neither the Viña del Mar nor the Santiago spectra exhibit such a large climb in spectral displacement values beyond 1.2 seconds. The displacement spectrum generated from the Viña del Mar station has increasing spectral values up to approximately 1 second, and remains relatively constant for periods greater than 1 second. Response spectra from both east-west recordings in Santiago show generally increasing spectral displacements as periods increase. Santiago displacement spectra in the north-south direction increase for periods up until approximately 0.5 seconds, and then values remain relatively constant up to periods around 1.1 seconds before increasing again for periods beyond 1.1 seconds.

Figure 3.2 Linear response spectra for Concepción station. Note the large spikes around periods of 1.5-2s.
Building fundamental periods including the influence of concrete cracking were estimated. Uncracked periods (or periods at low-amplitude shaking) for each structure were calculated as \( T = \frac{N}{20} \), where \( N \) is the number of stories above ground level (Massone et al., 2012). A comparison of periods determined using \( N/20 \) and approximate periods calculated using ASCE 7 is provided in Table 3.1. In general, the values obtained by \( N/20 \) were usually close to the values obtained by the approximate fundamental period calculated following ASCE 7-10 section 12.8.2.1. Uncracked periods were amplified by \( \sqrt{2} \) to account...
for wall section cracking (effectively modeling cracked wall stiffness as 50% of gross section stiffness), and then used to determine spectral displacement of a single-degree-of-freedom oscillator $\delta_{\text{sdof}}$. This increased period is similar to N/15 estimates of fundamental periods during larger-amplitude vibrations for Chilean structures (Massone et al. 2012).

Figure 3.5 through Figure 3.8 illustrate the spectral displacement values obtained from periods associated with N/20 and cracked wall section periods estimated by $\sqrt{2}$ N/20. In general, longer periods are usually associated with higher displacements; however, if the cracked section period fell into a trough in the associated displacement spectrum, the largest spectral displacement value bounded by the N/20 and $\sqrt{2}$ N/20 period values were used for analysis. For the structures selected from Santiago, the largest spectral displacements bounded by the two period estimates from both response spectra (Centro and Peñalolen) were averaged.

Figure 3.5 through Figure 3.8 illustrate the spectral displacement values obtained from periods associated with N/20 and cracked wall section periods estimated by $\sqrt{2}$ N/20. In general, longer periods are usually associated with higher displacements; however, if the cracked section period fell into a trough in the associated displacement spectrum, the largest spectral displacement value bounded by the N/20 and $\sqrt{2}$ N/20 period values were used for analysis. For the structures selected from Santiago, the largest spectral displacements bounded by the two period estimates from both response spectra (Centro and Peñalolen) were averaged.

Figure 3.5 Building #1 spectral displacement. Upper and lower period estimates bound a peak in $S_D$.
Building #1’s period estimates straddle a peak spectral displacement value of about 8 inches. The cracked period estimate is located right where spectral displacements begin to climb dramatically to approximately 40 inches at 1.5 seconds and more than 60 inches at 2.0 seconds. It is possible that the structure’s effective fundamental period increased beyond 1.1 seconds during the event, which may have drastically increased roof drift demands beyond the simple estimate above.

Most linear displacement spectra for ground motions recorded in Chile (in both the 1985 and 2010 earthquakes) become relatively “flat” (or constant) for periods beyond about 1.0 second, except for the ground motion recorded in downtown Concepción about 1 km from Building #1 (Massone et al, 2012). For Building #1, a two-dimensional nonlinear response history analysis was conducted on a “slice” of the building in the collapse direction to assess the potential impact of spectral shape on the roof displacement estimate (Tuna and Wallace, 2014). A modal analysis in Perform 3D using material properties based on core tests and cracked section properties yielded a period of 0.69 s in the transverse direction. This was similar to the N/20 value of 0.75 for this structure. Non-linear response history plots of roof displacement using the Maule event record showed a forced frequency of roughly 3 cycles every 5 seconds as damage accrued, suggesting a period much higher than 0.69 seconds as shaking progressed. This likely would have pushed the roof displacements much higher than what was anticipated by the simple model employed in this study.
Buildings #2 and #3 were oriented perpendicularly to Building #1. Spectral displacement associated with Building #2’s estimated cracked period was around 4.5 inches. The range of periods bounded by Building #3’s elastic period estimate ($N/20$) and cracked period estimate ($\sim N/14$) coincide with the start of a large spike in spectral displacement for the earthquake record component oriented in the same direction as Building #3’s transverse displacement. The elastic period was associated with a relatively small 8 inch spectral displacement, whereas the cracked period estimate was significantly larger, at around 42 inches. The elastic response spectrum associated with the transverse roof displacement yields very large increases in spectral displacement even for very small increases in period estimates greater than the $N/20$ estimate; when spectral displacements for the cracked period estimate are transformed into a spectral roof translation, drift demands are almost 3% for the cracked section period estimate. As this is a very large displacement demand that would more than likely result in notable damage, and there was relatively no damage reported after the earthquake suggests the displacement demand associated with the upper bound period estimate over-predicts the actual displacement experienced by the structure. Furthermore, the large setback occurring halfway up the height of Building #3 (creating a significant
change in mass and stiffness, as opposed to the relatively uniform mass and stiffness associated with the other buildings in the investigation) probably creates a natural response of the building that differs from the 1\textsuperscript{st} mode dominant, N/20 fundamental period behavior used to characterize the other six structures in this study. Nevertheless, for consistency, the same approach for characterizing building behavior as was used for the other structures was applied to Building #3.

Building #4, located in Viña del Mar, had a cracked section period estimate of approximately 0.7 seconds following the $\sqrt{2}N/20$ estimate scaled for cracking. An ETABS model of the entire structure was analyzed as part of NIST GCR 12-917-18 (2012), and the fundamental period associated with transverse shaking was 0.65 seconds using gross (uncracked) section properties and 0.91 seconds using effective (cracked, 0.5 times gross) section properties, both slightly larger than the N/20 and $\sqrt{2}N/20$ period estimates, respectively. The associated maximum spectral displacement bounded by periods using the N/20 approach was 6.31 inches, but would have been 15\% larger (7.27 inches) if the considered period range was expanded to 0.9 seconds.

Figure 3.7 Building #4 spectral displacement.
The two stations closest to the Santiago Buildings #5, #6, and #7 yielded similar 2% damping spectral displacement values for the period ranges considered. The response spectrum from the Santiago-Centro station suggested a slightly larger spectral displacement value for Building #7 than Santiago-Penalolen (about 3.2 inches versus 2.6 inches), whereas the Penalolen station spectrum yielded a cracked section period displacement for Building #5 almost twice that of the Centro station. Both stations had similar spectral value estimates for Building #6. The period limits for building #7 were expanded beyond the √2*N/20 limit based on building responses to aftershocks captured after the main Maule earthquake (Lemnitzer et al. 2014). Building #7 in Santiago was instrumented with several accelerometers on multiple story levels and the roof after the 2010 Maule event. As reported by Lemnitzer et al. (2014), more than twenty aftershocks were recorded in a one month period. Amplitudes of transfer functions of relative displacements on multiple floors indicate that the main frequency of building motion in the transverse direction (north-south) was approximately 1.09 Hz, corresponding to a period of about 0.92 seconds. A computer model of the mid-rise structure including the subterranean parking was analyzed and reported a fundamental period of 0.89 seconds. Both estimates fall in between cracked, amplified period estimates of N/20 (0.71 s) and N/15 (0.94 s) for a 10-story structure such as Building #7. The 0.92
second period was obtained from relatively small aftershocks, so this value might reflect a period for a wall not quite fully cracked, but also not uncracked.

After spectral displacement values were predicted for the candidate buildings, the building roof displacement was estimated as $\delta_{\text{roof}} = 1.5 \delta_{\text{sdof}}$, where the 1.5 factor accounts for the difference between the mode shape for a multi-story building and a single-degree-of-freedom oscillator (e.g., see ASCE 41-06, Chapter 3; Wallace and Moehle 1992). Calculated roof drift ratios typically fell between 0.5% and 1.0%, with the exception of Building #3. Table 3.1 reviews parameters for the displacement based approach summarized in the preceding paragraphs and the resulting roof drift ratios for the buildings investigated.

<table>
<thead>
<tr>
<th>Building ID</th>
<th>Location</th>
<th>Nearest Recording Station, Channel</th>
<th>Stories</th>
<th>Roof Height (ft)</th>
<th>ASCE 7 Approx Period (s)</th>
<th>N/20 Period Estimate (s)</th>
<th>Cracked Period Estimate (s)</th>
<th>Spectral Displacement (in)</th>
<th>Roof Drift Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Concepción, Ch 1</td>
<td>15 126</td>
<td>0.75</td>
<td>0.75</td>
<td>1.06</td>
<td>8</td>
<td>0.0082</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Concepción, Ch 3</td>
<td>13 105</td>
<td>0.66</td>
<td>0.65</td>
<td>0.92</td>
<td>6</td>
<td>0.0053</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Concepción, Ch 3</td>
<td>22 183</td>
<td>0.99</td>
<td>1.10</td>
<td>1.56</td>
<td>42</td>
<td>0.029</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Viña del Mar, Ch 2</td>
<td>10 86</td>
<td>0.56</td>
<td>0.50</td>
<td>0.71</td>
<td>7</td>
<td>0.009</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Stgo-Centro, Ch 1; Stgo-Penalolen, Ch 1</td>
<td>12 100</td>
<td>0.63</td>
<td>0.60</td>
<td>0.85</td>
<td>6</td>
<td>0.0075</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Stgo-Centro, Ch 1; Stgo-Penalolen, Ch 1</td>
<td>20 166</td>
<td>0.93</td>
<td>1.00</td>
<td>1.41</td>
<td>8</td>
<td>0.0062</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Stgo-Centro, Ch 3; Stgo-Penalolen, Ch 2</td>
<td>10 92</td>
<td>0.60</td>
<td>0.50</td>
<td>0.92</td>
<td>3.9</td>
<td>0.0053</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1Cracked period estimate based on actual building response to recorded aftershocks (Lemnitzer et al. 2014)

The roof drift estimates systematically obtained above were critical to assessing performance at the boundary elements of the structural walls. Section 3.2 discusses the use of the roof drift directly in determining if special boundary element detailing requirements would have been required according to...
ACI 318-11 Section 21.9 requirements, and Section 3.3 investigates the relationship of longitudinal bar tensile strain history and roof drift levels.

### 3.2 Special Boundary Element Detailing Trigger Investigation

The objective of these studies was to determine if special boundary elements would have been required according to ACI 318-11 Section 21.9.6.2. Since special boundary detailing was not required for concrete structural wall construction in Chile, comparing the damage observed at the boundary elements with how the U.S. code would dictate detailing offers insight into the effectiveness of the ACI trigger in predicting damage in the absence of special boundary element detailing. The following paragraphs outline the approach taken to model and analyze the wall critical sections of the Chilean walls, wall axial load, wall lateral displacement demand, and material properties. Critical sections were typically near the building base, either at the ground line or one level below ground line if the wall length was reduced (i.e., a significant wall setback existed).

Moment-curvature analyses were used to characterize wall behavior, i.e., to determine neutral axis depth for use in assessing the ACI trigger and for determining the distribution of axial strain in longitudinal reinforcing bars for the bar buckling studies. Moment-curvature relations were computed using BIAX (1996) to determine the neutral axis depth corresponding to the development of ACI 318-11 nominal flexural strength of each wall. This approach was used because it was determined that simplified equations (i.e., Wallace 1994) did not produce sufficiently accurate results, especially for wall sections with flanges. For asymmetric wall cross sections (e.g., T-shaped), two moment-curvature analyses were performed: one corresponding to the flange in compression, and the other corresponding to the flange in tension. Information regarding the axial load demands on wall cross sections and the material models used for the concrete and steel reinforcement are presented in this section. Wall axial loads were factored
in a manner consistent with U.S. practice using IBC 2012 Eq. 16-5 and ASCE 7-10 §2.3.2, i.e. $P_u = 1.2D + 0.5L + 1.0E_v$. Dead loads, represented as $D$, were estimated based on construction documents, typical architectural features for Chilean buildings, available photos, and input from local engineers (collaborators from Chile). Live loads, represented by $L$, were based on recommendations from U.S. code provisions (IBC, ASCE 7) for residential occupancy along with input from Chilean engineers. Vertical earthquake load $E_v$ was estimated as $0.2D$.

Weight takeoffs for floor slabs and roofs were estimated based on information available from structural drawings, architectural feature assumptions, and building occupancy. Weights for structural elements including walls, slabs, and beams were calculated based on structural drawings specific to the building being analyzed. Vertical element (beam, wall) weights were distributed evenly across the floor plate for simple weight tabulation. Assumptions were made for the weight of architectural, mechanical, and electrical features and these assumptions were consistently applied for the buildings analyzed, unless weight information was specifically provided in building documents. A residential live load of 40 psf was used across floor slabs (corresponding to IBC Table 1607.1 item 28: private rooms and corridors serving them) and a 20 psf live load was used on roofs. Table 3.2 is an example tabulation of dead and live loads for a typical floor in Building #1. After calculating dead and live loads a quick analysis of tributary areas was performed on walls of interest. Axial loads were then calculated for the various walls of interest in the building study set.
Table 3.2 Typical floor weights in Building #1

<table>
<thead>
<tr>
<th>Typ Floor Loads (15 cm Concrete Slab):</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SLAB DEAD LOADS</strong></td>
<td></td>
</tr>
<tr>
<td>Flooring (Carpet and Pad)</td>
<td>3.0 psf</td>
</tr>
<tr>
<td>15 cm (5.9&quot;) Concrete Slab</td>
<td>73.8 psf</td>
</tr>
<tr>
<td>Concrete Beams (32.4m 20W X 93D on 515 s.m.)</td>
<td>5.8 psf</td>
</tr>
<tr>
<td>HVAC</td>
<td>1.0 psf</td>
</tr>
<tr>
<td>Plumbing/Fixtures/Electrical</td>
<td>1.0 psf</td>
</tr>
<tr>
<td>Ceiling/Plaster</td>
<td>1.0 psf</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>5.0 psf</td>
</tr>
<tr>
<td>Estimated Dead Load from Horizontal Elements</td>
<td>90.6 psf</td>
</tr>
<tr>
<td>Use</td>
<td>90.6 psf</td>
</tr>
<tr>
<td></td>
<td>(4.34 kPa)</td>
</tr>
<tr>
<td><strong>LIVE LOADS</strong></td>
<td></td>
</tr>
<tr>
<td>Live Load (Private Rooms, Reducible)</td>
<td>40.0 psf</td>
</tr>
<tr>
<td>Partition Load (Negligible)</td>
<td>0.0 psf</td>
</tr>
<tr>
<td>Estimated Live Load on Horizontal Elements</td>
<td>40.0 psf</td>
</tr>
<tr>
<td></td>
<td>(1.92 kPa)</td>
</tr>
<tr>
<td><strong>ADDITIONAL WEIGHT</strong></td>
<td></td>
</tr>
<tr>
<td>Columns</td>
<td>0.0 psf</td>
</tr>
<tr>
<td>Nearly None</td>
<td></td>
</tr>
<tr>
<td>Walls (Concrete w/ Plaster EA Side)</td>
<td>88.5 psf</td>
</tr>
<tr>
<td>162.8m of 20 cm Walls (2.37m Tall) on 515 s.m. Floor</td>
<td></td>
</tr>
<tr>
<td>~1.5 cm Plaster Finish EA Side</td>
<td></td>
</tr>
<tr>
<td>Additional Considerations</td>
<td>0.0 psf</td>
</tr>
<tr>
<td>Total Estimated Floor Dead Weight</td>
<td>179.1 psf</td>
</tr>
<tr>
<td>Use</td>
<td>180.0 psf</td>
</tr>
<tr>
<td></td>
<td>(8.62 kPa)</td>
</tr>
</tbody>
</table>

Based on this review, typical unit floor weights varied between 150 and 200 psf (0.75 to 1.0 T/m²), depending on variations in slab thickness and structural member distribution (e.g., beams, walls). A unit weight of 150 to 200 psf is consistent with values reported in the literature (e.g., Massone et al., 2012). Construction documents (drawings) were used to determine wall axial loads based on estimates of floor plate tributary areas. As the numerous walls in Chilean buildings provide the majority of partitions between room units, partition loads were not considered. With tributary areas calculated and gravity loads estimated, the next step was to determine the total load at the wall critical section. Most often, the
critical section was located either at the ground or podium level, or the 1st subterranean level. Wall axial stress was found to vary widely, from as low as $0.07A_f f'_c$ to as high as $0.43A_f f'_c$.

Material models for concrete and steel were based on information available in construction documents for the buildings investigated. The most common callout for concrete quality was type H-30 (Buildings #1, #2, #3, and #7), with a few instances of H-25 (Buildings #5 and #6). Concrete strength, $f'_c$, is typically established through concrete cube tests in Chile, and concretes designated H-30 must demonstrate cube test strength of at least 30MPa according to NCh170 specifications. Since design strength in ACI 318 (and therefore NCh430) is based off of cylinder tests, adjustments are made in 5.1.2 of NCh430 to translate cube test strength into cylinder strength (and thus $f'_c$) as follows:

<table>
<thead>
<tr>
<th>$f'_c$ (MPa)</th>
<th>Concrete grade (NCh170 with 10% deficient fraction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>H20</td>
</tr>
<tr>
<td>20</td>
<td>H25</td>
</tr>
<tr>
<td>25</td>
<td>H30</td>
</tr>
<tr>
<td>30</td>
<td>H35</td>
</tr>
<tr>
<td>35</td>
<td>H40</td>
</tr>
<tr>
<td>40</td>
<td>H45</td>
</tr>
<tr>
<td>&gt;45*</td>
<td>For concrete compressive strength greater than H45 the value of $f'_c$ shall be determined using standard cylindrical samples.</td>
</tr>
</tbody>
</table>

Therefore, where documents call out H-30, the design strength used in analysis is 25 MPa (3625 psi). The design strength for H-25 is 20 MPa (2900 psi). Because the walls studied are so thin, have large vertical spacing between hoops (if provided) and horizontal web reinforcement that is typically bent around the wall boundary longitudinal reinforcement—see Figure 2.1—and is not anchored into the wall core, it is unlikely that concrete contained inside boundary vertical reinforcement (and hoops, if present) should be considered as “confined” concrete. Therefore, for this study, all concrete is treated as
“unconfined” and modeled following the Saatcioglu and Razvi (1992) relationship. Design documents specified H-30 quality concrete for the walls in Buildings #1, #2, #3, and #7, and specified H-25 quality concrete for the walls in Buildings #5 and #6. The concrete stress-strain relationship is presented in Figure 3.9.

![Concrete Material Stress-Strain Model](image.png)

**Figure 3.9 BIAX program material models for H25 and H30 unconfined concrete.**

Steel reinforcement in all structures was specified as A63-42. According to the Chilean standard NCh204, which governs hot-rolled reinforcing bars, A63-42 must have a minimum yield strength of 420 MPa (60.9 ksi) and a minimum ultimate strength of 630 MPa (91.4 ksi). For analyses used to check the special boundary element triggers, an elasto-plastic model was used for steel reinforcement to remain consistent with design assumptions prescribed in section 10.2.4 of ACI 318. Minimum design strengths, without material overstress considerations, were used to model both the concrete and reinforcing steel stress-strain relationships to remain consistent with typical design practice according to ACI 318.

Material models, axial load demands, and wall section geometries were input into BIAX, a computer program that generates moment-curvature, neutral axis depth, and extreme tension fiber strain data at specified levels of extreme compressive fiber strain. A BIAX model was created for each wall examined.
in this study, for lateral loading in both a positive and negative directions. Results for neutral axis depths at compression strains of 0.003 were compared against the limiting neutral axis depth in ACI 318 equation 21-8, and are summarized in the following chapter.

3.3 Boundary Element Longitudinal Bar Buckling Investigation

The special boundary element assessment presented above is critical in designing structural walls subjected to large seismic demands to prevent boundary concrete from crushing. However, it is essentially a monotonic evaluation, i.e., it does not consider cyclic loading history. Research and reconnaissance have suggested that bar buckling after reloading from large tensile strain excursions might lead to strength loss due to bar buckling, especially at larger ratios of transverse reinforcement spacing to longitudinal bar diameter. Common ratios of vertical spacing of horizontal reinforcement or hoops/ties (if present) to longitudinal bar diameter \((s/d_b)\) in Chilean wall boundaries were between 8 and 11. With such large \(s/d_b\) ratios along with the thin walls commonly used in Chile, concrete confinement is modest (at best) and longitudinal reinforcement is susceptible to instability (buckling). One possibility considered was whether bar buckling prior to concrete spalling might initiate damage concentration at wall boundaries. The issue of bar buckling is addressed in the following section.

In order to investigate if buckling of vertical reinforcement at wall boundaries was expected in some of the Chilean walls previously evaluated to assess the ACI trigger for SBEs, an analysis adopting the strain identifier presented by Rodriguez et al. (1999) was implemented. As discussed previously in Chapter 2, Rodriguez et al. (1999) identified a trend for the onset of bar buckling (parameter \(\varepsilon_p^*\)) relative to the unbraced length of a reinforcing bar tested in air \((S_h/D\) or \(s/d_b\)). Rodriguez introduces parameter \(\varepsilon_p^*\), which measures the amount of bar strain from the point of reloading bars in compression to the point
when bars start to buckle, referred to as the onset of buckling (Figure 2.4). Figure 2.5 (repeated below as Figure 3.10) illustrates the trend of earlier onset of buckling for larger \( s/d_b \) ratios: the value of the strain indicator parameter \( e_p^* \) drops substantially for large \( s/d_b \) ratios, suggesting very low \( e_p^* \) values for typical \( s/d_b \) ratios used at wall boundaries in Chilean buildings (e.g. \( s/d_b \) 8 to 11). In this investigation, analyses were conducted to estimate \( e_p^* \) and then assess the likelihood of longitudinal bars buckling at the wall boundaries of selected Chilean walls.

![Figure 3.10 Lower bound estimate of initiation of bar buckling from cyclic strain demands (Rodriguez et al. 1999).](image)

First, a set of moment-curvature analyses were performed for the critical section for each wall section selected. Nonlinear material stress-strain relations, best estimates for the quantity and distribution of vertical (boundary and web) and boundary transverse reinforcement, and expected axial loads based on D+0.25L (ASCE 41) were used for the analyses. Because the walls studied were so thin, had large vertical spacing between hoops and horizontal bar bends, and hooks at boundary zones were typically 90 degree bends instead of 135 degrees (Figure 2.1), it is unlikely that the concrete core should be considered confined. Therefore, concrete is considered unconfined and modeled following the Saatcioglu and Razvi (1992) unconfined relationship in all cases. The analyses here for the onset of bar buckling in boundary
elements differ from neutral axis depth studies in that they used a non-linear stress-strain relationship that included strain hardening and post-ultimate-strength softening before rupture. The elasto-plastic (used in the ACI neutral axis trigger investigations) and nonlinear (used in the bar buckling analyses) relationships for steel are shown in Figure 3.11.

Figure 3.11 Steel reinforcement material models for ACI 318 equation 21-8 trigger and bar buckling analysis.

Moment-curvature relations were computed using the BIAX program (Wallace, 1996), including associated values of extreme fiber compression strains and maximum reinforcement tensile strains. Strain history parameter $\varepsilon_p^*$ (from here on referred to as “buckling strain indicator”) was determined for the outermost layer of boundary longitudinal bars. Since response-history analyses were not performed, rebar strain histories were not available to obtain the buckling strain indicator: instead this parameter was estimated using the following process. First, it was assumed that the building would experience approximately equal roof displacements for positive and negative loading parallel to the wall web, which typically is one of the principal directions for the building. The equation relating drift-curvature that was used to relate moment-curvature results to drift demands was derived by Wallace and Moehle (1992):

$$\frac{\delta}{h_w} = \frac{11}{40} \phi_h h_w + l_p (\phi_u - \phi_f)$$

3.1
Note that this drift-curvature relationship differs from the drift-curvature assumption inherent in ACI 318-11 Equation 21-8 (Equation 2.2 in this document). Whereas the relationship in ACI 318 assumes roof drifts are dominated by plastic rotations, the model implemented in these bar buckling investigations includes the drift component associated with elastic curvature. Elastic curvature and drift were based on assumptions on the yield curvature (curvature associated with first yield of outermost longitudinal reinforcement), seismic loading pattern experienced by the structure (reverse triangular distribution on a cantilever wall, representative of 1st mode behavior), and plastic hinge length ($l_p$ equal to the smaller of $l_w/2$ or story height to remain consistent with other analyses performed in this project as well as design assumptions integrated into U.S. building code equations). Ultimate drift ratios were determined from spectral displacement as in Section 3.1.

The plastic hinge model used in Equation 3.1 has a significant impact on drift ductility (and determining axial tensile strain demands on longitudinal bars due to drift levels). As noted in Chapter 2, the provisions of ACI 318 §21.9.6.2 (Equations 2.1 and 2.2) are based on an assumed plastic hinge length of one-half the wall web length ($l_p = l_w/2$). ASCE 41-06 also recommends using a plastic hinge length of one-half the wall length. Given wall damage observed in Chile, which tended to concentrate over a relatively short height of 2 to 3 wall thicknesses, it is important to reassess this assumption and consider providing alternative values or imposing limitations on the using a value of ($l_p = l_w/2$). Recommended values for wall plastic hinge length vary widely, although typical recommended values range between $0.5l_w$ and $1.0l_w$, but less than one story height. Figure 3.12 illustrates hinge models obtained with various relations; most of the models were developed from tests on concrete columns and beams subjected to a variety of loading conditions. The relationship implicit in ACI 318 §21.9.6.2 and recommended by ASCE 7 is the heavy black line.
As noted above, ACI 318-11 21.9.6.2 implies \( l_p = \frac{l_w}{2} \), and ASCE 41-06 (2007) and FEMA 356 (2000) recommend the same value, but not more than one story height (this story height limitation is not included in the ACI code). Kabeyasawa et al. (1983) assumed uniform axial strains at wall boundaries over one story height, which is a practical limit given that walls in Japanese buildings tend to be long and with low aspect ratio \( \left( \frac{h_w}{l_w} \right) \). Shorter lengths also have been suggested, including \( h_w/8 \) (Orakcal and Wallace, 2006), \( 0.3l_w \) (Tabata et al., 2003), \( 2.5t_w \) (Takahashi et al., 2013)—the value recommended by Takahashi et al. (2013) is intended to be used for walls with light transverse reinforcement at wall boundaries, which might apply to a Special Structural Wall detailed according to 21.9.6.5 or an Ordinary Structural Wall. Wallace (2012) also suggested a value of 2 to 3\( t_w \) for walls with modest detailing.

Slightly more complex relationships for wall plastic hinge length have been recommended, such as by Baker and Amarakone (1964), derived from beam and column tests, which considers both member length and member depth, i.e., \( l_p = C_2^{0.25} d^{0.75} \). Paulay and Priestley (1993) recommend a simpler plastic hinge length relationship.
length model that scales depending on both wall length and aspect ratio: \( l_p = (0.2 + 0.044A_y)l_w \). Other relationships, most of them derived from beam and column tests, include: \( l_p = 0.25d + 0.75z \) (Sawyer, 1964); \( l_p = 0.5d + 0.05z \) (Mattock, 1967); \( l_p = 0.5l_w + 0.022d_b f_y \) (Hines et al., 2004); and \( l_p = 0.12z + 0.014d_b f_y \) (Panagiotakis and Fardis, 2001).

The value selected for plastic hinge length may have a significant impact on the results computed with the Equation (3.1). Figure 3.13, which presents drift versus curvature relations for various plastic hinge lengths for a wall with an L-shaped cross section, demonstrates that computed curvature values obtained using (3.5) are very sensitive to the assumed plastic hinge length. For small estimates of plastic hinge length, damage concentrates over a short height and strength loss occurs at relatively low deformation (drift) demands; improved behavior (strength loss at higher inelastic deformations) occurs where inelastic deformations are spread out over greater heights. Figure 3.14 relates drift ratio \( \frac{\delta}{h_w} \) to the limiting concrete compressive strain (\( \varepsilon_c = 0.003 \)) for various plastic hinge lengths and reveals that the drift ratio at which the limiting compressive strain is reached is significantly impacted by the assumed plastic hinge length.
Figure 3.13 Roof Drift Ratio vs. Wall Curvature.

Figure 3.14 Influence of plastic hinge length on SBES (Wallace et al., 2012).
Information summarized in the preceding paragraphs demonstrates that the expression used to assess the need for SBEs at wall boundaries in ACI 318-11, Equation (2.2) assumes $l_p = l_u / 2$, that recommended values for plastic hinge length vary widely (Figure 3.12), and that use of different plastic hinge lengths in (2.2) would produce wide variations in the limiting neutral axis depth ($c_{lim}$) (Figure 3.13 and 3.14). One approach considered in this study to address this issue was to develop a wall database and to compare test and analytical results to develop more robust relationships for wall plastic hinge lengths, e.g., compare test results for lateral load versus top displacement with similar relations derived from analytical results using different plastic hinge lengths. Although these results would be of interest, insufficient time and likely insufficient test data exist to implement this approach at this time.

With these relationships and assumptions, buckling strain indicator for boundary longitudinal reinforcement was estimated using the following three steps exhibited using Figure 3.15. For clarity, these steps are described below for an asymmetric (flanged) wall:

1. Use M-ϕ analysis results to calculate the web boundary compression strain associated with an arbitrary level of drift causing flange tension,

2. Use M-ϕ analysis to calculate the maximum tensile strain in web boundary reinforcement associated with an equal (but in the reverse direction) level of drift causing flange compression, and

3. To determine $\varepsilon_p$, adjust for (subtract out) elastic strain recovery in the reinforcing bars from the maximum tensile strain calculated, and reduce the level of compressive strain expected in the bars by considering the compressive strain gradient between the extreme compressive fiber and the neutral axis depth.
Repeating this process for a range of drift levels generates a relation for $\varepsilon_p^*$, and can easily be repeated for the opposite wall boundary zone. Once $\varepsilon_p^*$ values are evaluated for a set of drift ratios, the relationship suggested by Rodriguez et al. (1999) is used to highlight the range in bar strains where rebar buckling might be expected. The results from this process are depicted in Figure 3.16, which illustrates the trend for rebar buckling established by Rodriguez et al. (1999) along with a moment capacity versus drift relation for the web boundary in compression (flange in tension) of a flanged wall in Building #4. The relations plotted for this particular wall suggest rebar buckling would occur prior to concrete crushing, as described in the following paragraph.
The hatched region in Figure 3.16 corresponds to the range in which isolated bar buckling would be expected to initiate according to Figure 13 of the Rodriguez et al. (1999) paper. The moment-drift curve does not include any buckling limit state, it only addresses when concrete in compression crushes or when reinforcement in tension ruptures. By comparing the two relations, it is possible to assess at what drift ratios bar buckling and concrete crushing are anticipated, and which is more likely to occur first. For this wall, the onset of bucking may have occurred as early as 0.5% drift, and is likely to have occurred between roof drift ratios between 1 and 1.2%. Crushing of the concrete is not predicted until the roof drift ratio reaches approximately 1.6%. The buckling indicator strain for this wall at the target drift level is almost 3%; this is well beyond the expected onset of buckling strain according to the Rodriguez trend (inset on Figure 3.16).
One potential shortcoming of using this approach is that it tries to relate the onset of buckling for an isolated rebar tested in air to significant loss of lateral strength of structural walls; these events are not necessarily well correlated. Bucking of an individual bar or pair of bars at a wall boundary with a fairly large number of longitudinal bars distributed along the wall may not lead to significant (or observed) strength loss. Rectangular wall RW1 tested by Thomsen and Wallace (1995, 2004) demonstrated this behavior, as bar buckling was observed to initiate at 1.0 to 1.5% drift, yet the wall was able to sustain several cycles of 2.0% drift prior to significant loss in lateral strength during the first cycle to 2.5% drift. Another limitation of this approach is that for large ratios of $s/d_b$ concrete cover spalling may be necessary to enable bar buckling. The possibility of longitudinal reinforcement buckling prior to concrete cover spalling was investigated by conducting tests of prisms representative of wall boundary elements subjected to cyclic tension and compression, and is presented in Chapter 7.

3.4 Boundary Element Longitudinal Bar Buckling Assessment Verification

The reliability of the approach used above, i.e., assuming equal drift levels for loading in each direction, was assessed by comparing analytical results to experimental results obtained from the walls tested by Thomsen and Wallace (1995, 2004). The approach presented above is essentially a monotonic analysis of a reverse-cyclic problem, so calibration to reverse-cyclic data is necessary to determine the validity of the application in predicting when bar buckling onset would be expected. Some of the walls tested by Thomsen and Wallace (1995, 2004) were detailed similar to Chilean walls, displayed damage similar to that observed in Chilean buildings after the 2010 earthquake, and included drift and strain data for walls with both rectangular and T-shaped cross sections. As such, results for all four wall specimens tested by Thomsen and Wallace are presented here in an effort to verify the prediction approach for the Chilean walls with actual strain data from similar walls.
The drift cycle history used in the test program was well documented, eliminating the need for the equal displacement assumption; however, it is noted that the test specimens were subjected to equal drift levels for positive and negative loading under displacement control. Knowing the previous half cycle’s maximum drift permitted calculating indicator strains for the range of each drift level cycle, rather than assuming values for the prior drift cycle and bar strain history.

Figure 3.17 shows the indicator strain values calculated for TW1 for each half-cycle beyond ±1.0% drift, as well as the imposed lateral force to achieve a given drift level. Plots of experimental reverse-cyclic force-displacement data, monotonic force-displacement BIAx analysis, and longitudinal bar compression reloading strains associated with web compression are in the lower left quadrant of Figure 3.17, and curves related to flange compression are in the upper right quadrant. Similar to Figure 3.16, the hatched region indicates the range where the onset of bar buckling would be expected according to the data fit presented by Rodriguez et al. (1999). TW1 had relatively large hoop spacings at the web boundary element, with the length to bar diameter ratio equal to 8. As such, a relatively low level of bar buckling indicator strain would be expected prior to bar instability. Instead of a single line tracking $\varepsilon_p$ for equal roof drifts, each hatched line tracks the respective longitudinal bar indicator strain history for a different level of drift. For example, the plot shows that for the 1% drift cycle longitudinal bar strains entered the lower bound range of when onset of buckling might be expected. By the time the experiment reached the 1.5% drift cycle, bar strains were beyond the upper bound of when buckling would initiate. Analyses also indicated that when the flange was in compression, bar strain history never exceeded 0.5%. This was much lower than the anticipated level where buckling might initiate, and no damage associated with bar buckling was observed at the flanged end of the wall.
TW1 experienced sudden loss of lateral strength during the first (negative) cycle to 1.5% drift, which loaded the web boundary (opposite the flange) in compression. At approximately -1% drift, based on video data, minor concrete spalling occurred, corner bars appeared to grow unstable, and then a brittle, explosive failure occurred as all eight vertical boundary bars and several pairs of web vertical bars buckled, thrusting off the concrete cover. The calculated $\varepsilon_p$ at the drift level of -1.0%, after considering the web longitudinal bars had already experienced tensile strains associated with the +1.5% drift cycle, was about 0.023. Longitudinal bar strain history plots (Thomsen and Wallace, 1995) revealed that maximum tensile strain for boundary vertical bars was about 0.025, and the extreme fiber concrete compression strain at the wall web boundary opposite the flange at the 1.5% drift level was about 0.005, producing an $\varepsilon_{p,test}^*$ of 0.03, suggesting the calculated $\varepsilon_{p,analysis}^*$ of 0.023 from moment-curvature analysis was fairly conservative (predicted onset of buckling for lower strains than measured). One possible reason for this discrepancy is the role of concrete cover, which might provide lateral support and delay the
onset of bar buckling. Another possible reason for the discrepancy is that the onset of bar buckling occurred prior to observed strength loss. Despite the minor discrepancies, the prediction approach from the monotonic moment-curvature analyses agreed well with the overall performance of the longitudinal bars in the test specimen web end boundary element.

Longitudinal bar strain analysis of wall RW1, with rectangular cross section, is illustrated in Figure 3.18. Monotonic force-displacement analysis from BIAx and experimental force-displacement data are also provided as references. The hatched region indicates the range where the onset of bar buckling would be expected. The results indicate that the onset of bar buckling would be expected for a drift ratio as low as 1.0%, would be quite likely for a drift ratio of 1.5%, and expected for a drift ratio of 2.0%. These results are consistent with test observations, where initial (onset) of buckling was noted between 1.0 and 1.5% drift (orange curve), visible and significant buckling of boundary edge bars was observed for 1.5% and 2.0% drift cycles (red curve), and significant strength loss was noted for the first cycle to 2.5% drift.

Figure 3.18 Bar buckling analysis for Thomsen wall RW1.
Two more walls with tighter hoop spacing from the Thomsen tests (RW2 and TW2) were also analyzed using this same approach and suggested initiation of bar buckling was not as likely as in the previous two tests examined. Investigation of bar strain data and testing observations confirmed these results. As exhibited in Figure 3.19 and Figure 3.20, indicator strain values did not reach the range where bar buckling would be expected to initialize. Wall RW2 had a $s/d_s$ ratio of 5.3; the Rodriguez curve suggests bar buckling would initiate around 4% strain. Calculated bar strains in the boundary zone were higher than that in both RW1 and TW1, but they did not exceed the predicted minimum threshold for bar buckling to occur. The calculated bar indicator strain for the largest drift cycle (2.5%) of RW2 reached about 3.5% strain, approaching the lower bound curve where bar buckling might initiate. Concret cover had already spalled earlier in the experiment, and by the end of the positive 2.5% drift cycle, several longitudinal bars buckled simultaneously.

![Figure 3.19 Bar buckling analysis for Thomsen wall test RW2.](image-url)
The hoop spacing in TW2 was even smaller than in RW2, approximately 3.3 times the boundary zone longitudinal bar diameter. Web compression, in the bottom right quadrant of Figure 3.20, shows that the bar buckling indicator strain never approached the 7%-9% strain predicted to be necessary to initialize bar buckling. Longitudinal reinforcement buckling was not observed in this experiment—instead, the damaged region of the web end boundary buckled out-of-plane.

![Figure 3.20 Bar buckling analysis for Thomsen wall test TW2; web compression in lower left quadrant.](image)

Table 3.4 provides a summary of analytical results for the four Thomsen and Wallace (1995) wall tests. For both RW1 and TW1, the bar buckling approach predicted strain values higher than the estimated strains needed to buckle bars for the given $s/d_b$ ratios; RW1 and TW1 both suffered damage at the boundary elements due to bars buckling. Analyses of RW2 and TW2 indicated that bar buckling indicator strains would not have exceeded the strains required to buckle reinforcement between transverse hoops. This was affirmed by the lack of bar buckling observed for both specimens.
The procedure developed to assess the onset of buckling for structural walls according to Rodriguez et al. (1999) was calibrated by comparing results with test results for cantilever walls with both rectangular and T-shaped cross sections. The comparison with test results indicated, for the range of strain values expected at the onset of bar buckling, that: (1) onset of buckling (very minor out-of-plane displacement of vertical bar) with no observed loss in lateral strength typically was associated with the lower-bound of the range, (2) bar buckling with noticeable out-of-plane displacements and significant concrete cover spalling was associated with the mid- to upper-range of the indicator, and (3) significant loss in lateral strength was generally associated with the high-range of the indicator (or slightly above the high range). Calibration of this approach with a larger set of test data is recommended.

Chapter 4 presents detailed information on the candidate buildings from Chile selected for investigation using the assessments described in Chapter 3. Building layouts, wall geometry and reinforcement, and pertinent irregularities with the lateral force resisting system are discussed. Results from the SBE trigger assessment and longitudinal bar buckling prediction approaches from this chapter are presented for each selected wall of each candidate building.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>RW1</td>
<td>10.0</td>
<td>8.0</td>
<td>0.01</td>
<td>0.02</td>
<td>0.03</td>
<td>Yes</td>
</tr>
<tr>
<td>RW2</td>
<td>7.0</td>
<td>5.3</td>
<td>0.035</td>
<td>0.045</td>
<td>0.035</td>
<td>Yes</td>
</tr>
<tr>
<td>TW1</td>
<td>9.0</td>
<td>8.0</td>
<td>0.01</td>
<td>0.02</td>
<td>0.025</td>
<td>Yes</td>
</tr>
<tr>
<td>TW2</td>
<td>7.5</td>
<td>3.3</td>
<td>0.07</td>
<td>0.09</td>
<td>0.05</td>
<td>No</td>
</tr>
</tbody>
</table>
CHAPTER 4  
ASSESSMENTS OF CHILEAN BUILDINGS

A total of seven mid-rise Chilean buildings, varying in height, layout and damage suffered, were selected in an effort to investigate the structural wall performance in the 2010 earthquake. Three of the selected buildings were located in Concepción. These included a 15-story structure that completely collapsed during the earthquake (Building #1—Alto Rio), a 13-story structure that was part of a two-building complex and was relatively undamaged (Building #2—Plaza del Río B), and 22-story tower that was also relatively undamaged (Building #3—Concepto Urbano). A 10-story structure in Viña del Mar with major wall damage at ground level also was also studied (Building #4—Toledo). Three additional buildings located in Santiago were selected for study, including a 12-story structure with damaged walls in the 1st subterranean level (Building #5), a 19-story structure that also experienced damage in walls in subterranean levels (Building #6—Emerald), and a 10-story building with some wall damage but relatively little boundary zone damage (Building #7—Magnolio).

Building loads, roof heights, tributary areas, and wall section design information for each structure were based on available construction documents and damage reports, and are reported in the following section on a building-by-building basis. In addition to building heights ranging from 10 to 22 stories—plays a significant role in axial loads (Massone et al., 2012) and roof drift ratio demands—various wall cross sections (T-, L-, and rectangular) were selected to identify important differences in expected performance of planar versus flanged walls. The expected wall behavior is substantially different for the cases with flange in tension versus flange in compression. Detailed descriptions of building plans, heights and wall configurations are presented in the following subsections.

Results from the SBE trigger assessments and longitudinal reinforcement buckling predictions are also offered in this chapter. After describing the layout of each structure and characteristics of the lateral force
resisting system, each section presents the results of assessments for each wall selected for analysis. Tables report neutral axis depths for extreme fiber at 0.003 compression strains normalized by web length for easier comparison among various walls and buildings. ACI 318-11 equation 21-9 has a minimum roof drift ratio of 0.007 that can be considered, effectively requiring SBE detailing for walls with neutral axis depths exceeding approximately one-quarter of the wall web length. In cases where building analysis predicted roof drifts less than 0.007, two trigger assessments were made: one abiding by the code, using the minimum 0.007 drift ratio to calculate the limiting neutral axis (0.238lw), and the other determining the SBE-triggering neutral axis depth if the calculated drift ratio less than 0.007 was used (as indicated by an asterisk in the following tables). Results discussions for each building will indicate where estimated roof drift ratios did not exceed the 0.007 drift ratio limit.

Neutral axis analysis results are compared to trigger neutral axis depths as a ratio (for both the code specified trigger depth and the modified equation/asterisk depth). Information regarding the degree of damage in the critical level of the wall is also provided. Buildings with wall boundary element detailing exceeding the minimum level typically provided in Chile, e.g., horizontal web bars terminating with 90 degree bends wrapped around the wall boundary longitudinal bars as shown in Figure 2.1) are noted in cases where this information is available (either from construction documents or observations from post-earthquake reconnaissance). Considerations for stress/strain concentrations due to wall discontinuities are not considered in the analyses.

Bar buckling strain history values were obtained over a range of roof drifts for each wall. This section summarizes results using tables and figures for each building. Vertical spacing of boundary transverse reinforcement, based on construction documents and/or field reconnaissance data, is provided for each wall. In such cases, the reported spacing-to-bar-diameter (s/ds) value represents the spacing of the
horizontal web bars that wrap around the boundary zone longitudinal bars (see Figure 2.1 and Figure 2.2), and is noted in the summary tables presented below. Upper and lower bound estimates of when buckling might initiate for a particular $s/d_b$ value are given in the tables, which are derived from the curves developed by Rodriguez et al. (1999) for isolated bars (Figure 2.5). Longitudinal bar strain values provided in the tables are results from the equal drift bar buckling indicator strain approach described in Section 3.3 at the roof drift level obtained in Section 3.1. Strain results for drift estimates slightly larger and smaller are included in wall figures to see how this value changes for lower or higher roof drift estimates, refer to the plots provided for the individual walls.

Individual wall plots show several important calculated wall properties. Each figure presents moment capacity versus drift, represented by the solid grey line. Moment capacity values are shown on the right vertical axis. Bar buckling indicator strains are illustrated by a dotted line and circled data points. The buckling indicator strain values are reported on the left vertical axis. A hatched area stretches across the main figure, corresponding to the range where buckling is expected for that particular wall’s $s/d_b$ ratio. Figure 2.5 from Rodriguez et al. (1999), as presented in Chapter 2, is inset as a reference to identify the bar buckling indicator strain prediction versus $s/d_b$ ratio.

4.1 Building #1 (Alto Rio)

4.1.1 Building Description and Wall Selection

As mentioned above, Building #1 was a 15-story structure located in Concepción. Two levels of subterranean parking were located under a podium slab at ground level, and 12 levels of a typical floor plan measuring 40 m (130 ft) long by 12 m (40 ft) wide (with some variations due to balconies) rose above the podium before the top three floor levels progressively tapered off in plan length. The roof
height of Building #1 was 38.3 m (126 ft) above grade level. The lateral force resisting system (LFRS) consisted of transverse and longitudinal concrete shear walls, 20 cm (~8 in) thick, interconnected along a central corridor. Structural wall boundary longitudinal reinforcement diameters ranged from 18 mm (0.71 in) to 25 mm (0.98 in). Web reinforcement consisted of two curtains, with either 8 mm (0.31 in) or 10 mm (0.39 in) diameter reinforcing bars, typically spaced 20 cm (~8 in) on center. There are no hoop or crosstie details at wall boundary elements. Figure 4.1 presents a representative floor plan of the building and highlights the walls being investigated in this study. Soil conditions at the site were classified as Soil Type II for the original design; post-collapse investigation reported that soil properties were characteristic of both Soil Type II and III, and suggested that the soil should have been identified as Soil Type III (IDIEM 2010).

Figure 4.1 Building #1 typical floor plan and walls selected for investigation.

Figure 4.2 illustrates the reduction in floor plan at the top levels via a building section cut along the corridor southern walls. Walls 1.1 and 1.2 end at the 13th level above grade, and walls 1.3 and 1.4 go up to the 14th level. Walls 1.5 and 1.6 extend all the way to the main roof level. Wall terminations and floor plate reductions at upper stories are not uncommon and generally do not adversely impact structural performance; however, several discontinuities also existed at the base of the building that probably did
contribute to structural collapse (Tuna and Wallace, 2014). Many of the transversely oriented walls located on the (plan) northern side of the building are “flag” shaped and lose their small returns at the podium level. These setbacks were made as an accommodation for parking at the podium level and below, and are primarily located in transverse walls along the northern edge of the structure. Figure 4.3 is an elevation view of the typical “flag” trait characteristic of many of the northern walls in Building #1.

Figure 4.2 Building #1 floor reduction at upper levels.

Figure 4.3 Typical wall setback at north side of Building #1 at podium level.
Table 4.1 provides general information for the six walls considered in Building #1. Critical section web lengths and thicknesses, flange widths and thicknesses, aspect ratios, and total wall areas are tabulated. Aspect ratios are calculated using the building roof height, under the assumption that walls shorter than the total building height still behave the same as if they extended all the way to the roof.

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>General Shape</th>
<th>Vertical Discont- inuity</th>
<th>Web Length (ft)</th>
<th>Web Thickness (in)</th>
<th>Roof Height (ft)</th>
<th>Aspect Ratio</th>
<th>Web Area (in²)</th>
<th>N Flange Width (ft)</th>
<th>N Flange Thickness (in)</th>
<th>S Flange Width (ft)</th>
<th>S Flange Thickness (in)</th>
<th>Total Wall Area (in²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>N. Web Stem, S. Flange</td>
<td>Yes, Flag</td>
<td>16'-5&quot;</td>
<td>7.9&quot;</td>
<td>117'-6&quot;</td>
<td>7.2</td>
<td>1550</td>
<td>NA</td>
<td>NA</td>
<td>5'-11&quot;</td>
<td>7.9&quot;</td>
<td>2170</td>
</tr>
<tr>
<td>1.2</td>
<td>N. Flange, S. Barbell</td>
<td>No</td>
<td>17'-9&quot;</td>
<td>7.9&quot;</td>
<td>117'-6&quot;</td>
<td>6.6</td>
<td>1674</td>
<td>13'-3&quot;</td>
<td>7.9&quot;</td>
<td>2'-11&quot;</td>
<td>7.9&quot;</td>
<td>3209</td>
</tr>
<tr>
<td>1.3</td>
<td>N. Web Stem, S. Flange</td>
<td>Yes, Flag</td>
<td>16'-5&quot;</td>
<td>7.9&quot;</td>
<td>117'-6&quot;</td>
<td>7.2</td>
<td>1550</td>
<td>NA</td>
<td>NA</td>
<td>11'-8&quot;</td>
<td>7.9&quot;</td>
<td>2713</td>
</tr>
<tr>
<td>1.4</td>
<td>N. Flange, S. Barbell</td>
<td>No</td>
<td>17'-9&quot;</td>
<td>7.9&quot;</td>
<td>117'-6&quot;</td>
<td>6.6</td>
<td>1674</td>
<td>22'-4&quot;</td>
<td>7.9&quot;</td>
<td>2'-11&quot;</td>
<td>7.9&quot;</td>
<td>4061</td>
</tr>
<tr>
<td>1.5</td>
<td>N. Web Stem, S. Flange</td>
<td>Yes, Flag</td>
<td>16'-5&quot;</td>
<td>7.9&quot;</td>
<td>117'-6&quot;</td>
<td>7.2</td>
<td>1550</td>
<td>NA</td>
<td>NA</td>
<td>9'-8&quot;</td>
<td>7.9&quot;</td>
<td>2527</td>
</tr>
<tr>
<td>1.6</td>
<td>N. Flange, S. Barbell</td>
<td>No</td>
<td>17'-9&quot;</td>
<td>7.9&quot;</td>
<td>117'-6&quot;</td>
<td>6.6</td>
<td>1674</td>
<td>9'-8&quot;</td>
<td>7.9&quot;</td>
<td>1'-12&quot;</td>
<td>7.9&quot;</td>
<td>2775</td>
</tr>
</tbody>
</table>

Ultimately, the podium/ground level was the critical level of the structure. Complete collapse of the structure occurred, with the upper portion of the building rigidly overturning in the direction of the flagged northern wall ends and disconnecting from the podium at the south (Figure 4.4). Post-earthquake investigations reconstructed the podium level walls and plotted the observed wall damage over elevation views of the original wall configuration (Figure 4.5).
4.1.2 Special Boundary Element Trigger Results

Factored axial load demand estimates for the six walls selected from Building #1 were typically between 9 and 10 percent of $A_g f'_c$. Neutral axis depths calculated for the six walls’ where web stem ends were loaded in compression exceeded the depth that would have required SBEs according to ACI 318-11 for the associated target roof drift ratio (Table 4.2). The three walls evaluated with non-flanged ends matching the wall web thickness (i.e., no flag or return at the wall edge—Wall 1.1, 1.3, and 1.5) exceeded
the ACI trigger neutral axis depth by at least 80%. These walls’ web stem ends were all located on the (plan) north side of the structure and oriented such that the wall web stems were on the outside edge of the building; Building #1 collapsed in this direction. None of the neutral axis depths associated with wall flange compression triggered special detailing requirements. The walls that exceeded the neutral axis trigger as smaller ratios or did not require special detailing in the web ends typically had a small flange/barbell (larger compression width), and their web ends were oriented towards the south side of the structure (such that they would have experienced large tension demands as the building collapsed to the north). The roof drift ratio exceeded 0.007 for this structure, so the trigger evaluation was performed with the estimated roof drift ratio.

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>Design Load $1.2D+0.5L+E_c$ (%$A_g f'_c$)</th>
<th>Roof Drift Ratio</th>
<th>Trigger Depth $c_{rig/1_{web}}$</th>
<th>North End $c_{rig/c_{rig}}$</th>
<th>North BE Damage?</th>
<th>South End $c_{rig/c_{rig}}$</th>
<th>South BE Damage?</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>11.8</td>
<td>0.0085</td>
<td>0.196</td>
<td>1.94</td>
<td>Collapse</td>
<td>0.21</td>
<td>Collapse</td>
</tr>
<tr>
<td>1.2</td>
<td>10.1</td>
<td>0.0085</td>
<td>0.196</td>
<td>0.17</td>
<td>Collapse</td>
<td>1.01</td>
<td>Collapse</td>
</tr>
<tr>
<td>1.3</td>
<td>14.0</td>
<td>0.0085</td>
<td>0.196</td>
<td>2.40</td>
<td>Collapse</td>
<td>0.12</td>
<td>Collapse</td>
</tr>
<tr>
<td>1.4</td>
<td>9.3</td>
<td>0.0085</td>
<td>0.196</td>
<td>0.06</td>
<td>Collapse</td>
<td>1.59</td>
<td>Collapse</td>
</tr>
<tr>
<td>1.5</td>
<td>10.1</td>
<td>0.0085</td>
<td>0.196</td>
<td>1.86</td>
<td>Collapse</td>
<td>0.11</td>
<td>Collapse</td>
</tr>
<tr>
<td>1.6</td>
<td>9.2</td>
<td>0.0085</td>
<td>0.196</td>
<td>0.11</td>
<td>Collapse</td>
<td>1.33</td>
<td>Collapse</td>
</tr>
</tbody>
</table>

Since this building completely collapsed—overturning at the ground level—there was extensive damage to the ground level walls. Wall elevations shown in Figure 4.5 reconstruct the wall to its pre-collapse shape and overlay observed damage from post-earthquake reconnaissance. Concrete crushing is noted in the non-flanged (north ends) wall boundaries of Walls 1.1, 1.3, and 1.5, consistent with the large neutral axis depths calculated. The damage report elevation also indicates concrete crushing damage at the flag-shaped (south end) wall boundaries of Walls 1.2, 1.4, and 1.6. Neutral axis depths from analyses for each of these aforementioned wall boundaries exceeded the ACI 318-11 critical neutral axis depth that would
trigger special detailing requirements in the U.S. This suggests the concrete crushing damage observed in walls in Building #1 was to be expected since the more demanding provisions of ACI 318-11 Section 21.9.6.4 were not met. Even if the spectral displacement responses were obtained for a higher damping ratio (e.g., 5% instead of 2%), the revised neutral axis depths would have exceeded the trigger value in ACI 318-11 that requires the use of special boundary elements.

### 4.1.3 Longitudinal Reinforcement Buckling Predictions

The horizontal web bar spacing relative to longitudinal bar diameter in the wall web ends of Building #1 was large. Almost every wall web end has indicator strain levels that at least enter the range where buckling initiation might be expected (Table 4.3). Figure 4.6 through Figure 4.11 below show the bar strain indicators and moment capacity-drift curves associated with wall web stems in compression. For even-numbered walls (1.2, 1.4, 1.6), web compression translates to drift to the “north” of the building and the direction of collapse. Web compression in odd-numbered walls (1.1, 1.3, and 1.5) corresponds to drift to the opposite direction. At the drift demand calculated for Building #1, all walls are approaching their peak moment capacity associated with web compression. Additionally, all walls have bar buckling indicator strains within levels that suggest buckling initiation at the calculated roof drift demands. If drift levels exceeded the calculated demands, both crushing and bar buckling were more likely to occur.
Table 4.3 Building #1 bar buckling analysis summary.

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>Expected Load $D + 0.25L$ (% $A_d f'_c$)</th>
<th>Web $s/d_b$ Ratio$^1$</th>
<th>$\varepsilon_{pf}$ Lower Estimate</th>
<th>$\varepsilon_{pf}$ Upper Estimate</th>
<th>Calc. Strain $\varepsilon_{pf,calc}$</th>
<th>Long. Bar Buckling Expected?</th>
<th>Buckling Observed?</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>8.8</td>
<td>9.1</td>
<td>0.005</td>
<td>0.015</td>
<td>0.009</td>
<td>Yes</td>
<td>Collapse$^2$</td>
</tr>
<tr>
<td>1.2</td>
<td>7.6</td>
<td>8.0</td>
<td>0.005</td>
<td>0.02</td>
<td>0.009</td>
<td>Yes</td>
<td>Collapse$^2$</td>
</tr>
<tr>
<td>1.3</td>
<td>10.5</td>
<td>9.1</td>
<td>0.005</td>
<td>0.015</td>
<td>0.009</td>
<td>Yes</td>
<td>Collapse$^2$</td>
</tr>
<tr>
<td>1.4</td>
<td>7.0</td>
<td>9.1</td>
<td>0.005</td>
<td>0.015</td>
<td>0.011</td>
<td>Yes</td>
<td>Collapse$^2$</td>
</tr>
<tr>
<td>1.5</td>
<td>7.5</td>
<td>11.1</td>
<td>0.005</td>
<td>0.015</td>
<td>0.008</td>
<td>Yes</td>
<td>Collapse$^2$</td>
</tr>
<tr>
<td>1.6</td>
<td>6.9</td>
<td>11.1</td>
<td>0.005</td>
<td>0.015</td>
<td>0.009</td>
<td>Yes</td>
<td>Collapse$^2$</td>
</tr>
</tbody>
</table>

$^1$Based on horizontal web reinforcement spacing. No hoops or ties provided at boundary.

$^2$Bar buckling is presumed to have occurred as part of the collapse of the structure.

Figure 4.6 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 1.1.
Figure 4.7 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 1.2.

Figure 4.8 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 1.3.
Figure 4.9 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 1.4.

Figure 4.10 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 1.5.
Building #2 (Plaza Del Rio B)

4.2.1 Building Description and Wall Selection

Building #2, also located in Concepción, is a 13-story, 32.1 m (105 ft) tall structure that was part of a two-structure complex. The building, permitted in 2004 and completed in 2006 (Westenenk et al. 2012), was seismically separated and oriented 90 degrees from an adjacent tower (and Building #1). There was no podium or subterranean parking at this site. The typical floor plan was approximately rectangular, measuring roughly 38 m long by 13 m wide. A typical floor plate is shown in Figure 4.12, along with the walls selected for analysis highlighted in red (and labeled). The building experienced relatively little damage in the walls, with only minor cracking reported, whereas the adjacent tower suffered major damage (and was eventually demolished). These observations on building orientation, along with the lack of damage in Building #3 and its major axis orientation similar to Building #2, suggest that directionality
may have had a significant impact on building performance in this geographic area. Building #2 was selected over its counterpart building due to its relative lack of vertical discontinuities compared to the adjacent structure and because of the lack of major damage. The lack of damage in the structural walls presents an opportunity to investigate if minor damage can result in moderately detailed walls subjected to earthquake demands even if U.S. design codes would have required special reinforcement detailing.

Walls in Building #2 were thinner than those in Building #1—typically only 15 cm thick (~6 in). Boundary longitudinal reinforcement was specified on drawings as 18 mm diameter bars. Web reinforcement consisted of two curtains of 8 mm diameter bars spaced at 20 cm (~8 in) on center in each direction. In addition to the typical web steel termination hooks at wall boundaries, 8 mm (0.31 in) diameter transverse hoops, with 135 degree returns that encompass the longitudinal reinforcement (see Figure 4.13), were specified in wall elevations as spaced 20 cm on center. No crossties were specified with the hoops at any boundary. Soil conditions at the site of Building #2 were classified as Soil Type III (Birely 2012, Westenenk et al. 2012).

Figure 4.12 Building #2 typical floor plan and selected walls for examination.
Figure 4.13 Building #2 transverse hoop detailing.

There are a few discontinuities in the walls selected for review. Wall 2.1 had flanges at both web ends at ground level, but neither continued up into the second story. Also, the small northern flange and part of the web that exists on upper floor levels discontinued at the ground level. As such, a rectangular wall model representative of only the continuous portion of the wall web was analyzed. Similarly, Wall 2.3 had flange discontinuities between the first and second stories, although there is no web discontinuity. As was done for Wall 2.1, the model (cross section) for wall 2.3 only includes the continuous elements of web and flange. Table 4.4 summarizes the general geometric properties of the critical wall sections.

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>General Shape</th>
<th>Vertical Discont.-inity</th>
<th>Web Length (ft)</th>
<th>Web Thickness (in)</th>
<th>Root Height (ft)</th>
<th>Aspect Ratio</th>
<th>Web Area (in²)</th>
<th>N Flange Width (ft)</th>
<th>N Flange Thickness (in)</th>
<th>S Flange Width (ft)</th>
<th>S Flange Thickness (in)</th>
<th>Total Wall Area (in²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Rectangular</td>
<td>Yes, Flag and Web Reduction</td>
<td>9'-10&quot;</td>
<td>5.9&quot;</td>
<td>105'-4&quot;</td>
<td>10.7</td>
<td>698</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>698</td>
</tr>
<tr>
<td>2.2</td>
<td>N. Web Stem, S. Flange</td>
<td>No</td>
<td>19'-8&quot;</td>
<td>5.9&quot;</td>
<td>105'-4&quot;</td>
<td>5.4</td>
<td>1395</td>
<td>NA</td>
<td>NA</td>
<td>4'-2&quot;</td>
<td>5.9&quot;</td>
<td>1725</td>
</tr>
<tr>
<td>2.3</td>
<td>N. and S. Flanges</td>
<td>Yes, Flange Reduction</td>
<td>20'-4&quot;</td>
<td>5.9&quot;</td>
<td>105'-4&quot;</td>
<td>5.2</td>
<td>1442</td>
<td>2'-0&quot;</td>
<td>5.9&quot;</td>
<td>1'-10&quot;</td>
<td>5.9&quot;</td>
<td>1711</td>
</tr>
</tbody>
</table>
4.2.2 Special Boundary Element Trigger Results

Estimated roof drift ratios for Building #2 did not exceed 0.007, the current minimum drift ratio considered for ACI 318-11 Equation 21-9. Neutral axis depths were compared to the trigger equation using this minimum drift ratio (0.007), as well as what the trigger neutral axis would be with the estimated drift ratio (indicated by an asterisk in Table 4.5). The three walls chosen for investigation were transverse walls. Analysis shows that only the stem end of Wall 2.1 would have required special detailing (Table 4.5), and, adjusting the trigger equation for the smaller than 0.007 drift ratio shows the neutral axis depth would have only exceeded the trigger by 9%.

Table 4.5 Building #2 SBE trigger evaluation.

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>Design Load 1.2D+0.5L+E, (%A.f')</th>
<th>Roof Drift Ratio</th>
<th>Trigger Depth c_nig/A_web</th>
<th>Trigger Depth c_nig/A_web</th>
<th>North End c/c_nig</th>
<th>North End c/c_nig</th>
<th>North BE Damage?</th>
<th>South End c/c_nig</th>
<th>South End c/c_nig</th>
<th>South BE Damage?</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>10.5</td>
<td>0.0053</td>
<td>0.24</td>
<td>0.31</td>
<td>0.83</td>
<td>0.63</td>
<td>No</td>
<td>0.96</td>
<td>0.73</td>
<td>No</td>
</tr>
<tr>
<td>2.2</td>
<td>12.3</td>
<td>0.0053</td>
<td>0.24</td>
<td>0.31</td>
<td>1.437</td>
<td>1.09</td>
<td>No</td>
<td>0.18</td>
<td>0.14</td>
<td>No</td>
</tr>
<tr>
<td>2.3</td>
<td>10.7</td>
<td>0.0053</td>
<td>0.24</td>
<td>0.31</td>
<td>0.937</td>
<td>0.71</td>
<td>No</td>
<td>0.53</td>
<td>0.40</td>
<td>No</td>
</tr>
</tbody>
</table>

4.2.3 Longitudinal Reinforcement Buckling Predictions

Building #2 had low drift relative to the other two structures investigated in Concepción. This structure did not suffer the severe damage experienced by its sister building. According to analysis, despite having large vertical spacing of boundary transverse reinforcement to boundary longitudinal bar diameter ratios (above 11 for all walls considered), the bar buckling indicator strains did not reach the levels where onset of bar buckling would be expected. This is consistent with the lack of reported wall damage in this structure. Table 4.6 and Figure 4.14 through Figure 4.16 illustrate the bar strain values obtained in the equal-displacement assumption analysis.

81
Table 4.6 Building #2 bar buckling analysis summary

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>Expected Load $D+0.25L$ (%$A_gf'_c$)</th>
<th>Web $s/d_h$ Ratio</th>
<th>$\varepsilon_p^*$ Lower Estimate</th>
<th>$\varepsilon_p^*$ Upper Estimate</th>
<th>Calc. Strain $\varepsilon_{p,calc}^*$</th>
<th>Long. Bar Buckling Expected?</th>
<th>Buckling Observed?</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>7.7</td>
<td>11.1</td>
<td>0.005</td>
<td>0.015</td>
<td>0.0006</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>2.2</td>
<td>9.0</td>
<td>11.1</td>
<td>0.005</td>
<td>0.015</td>
<td>0.003</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>2.3</td>
<td>7.8</td>
<td>11.1</td>
<td>0.005</td>
<td>0.015</td>
<td>0.001</td>
<td>No</td>
<td>No</td>
</tr>
</tbody>
</table>

Figure 4.14 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 2.1.
Figure 4.15 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 2.2.

Figure 4.16 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 2.3.
4.3 Building #3 (Concepto Urbano)

4.3.1 Building Description and Wall Selection

The last structure from Concepción that was considered for analysis was Building #3, a 22-story tower with two levels of subterranean parking. The building was under construction from 2008-2010, and was unoccupied at the time of the earthquake (Birely 2012). The soil classification per construction drawings was Soil Type III. Major damage was not reported in any of the structural walls of Building #3 (Birely 2012). Total height of the building from the podium level was 55.7 m (183 ft). The typical floor plate from the roof to the 12th floor measured about 26.5 m long by 17.7 m wide. From the 11th floor to the podium, the floor expands into an almost L-shape plan, adding a large amount of space to the west and south of the upper tower area (Figure 4.17). Two walls that extend from the base of the structure to the tower roof were selected for investigation, and are highlighted in Figure 4.17. One of the selected walls has a rectangular configuration (3.1) and the other wall has a large flange that made up part of the corridor wall (3.2).

The walls in Building #3 at the podium level were much larger than the walls at the base of Buildings #1 and #2: Walls 3.1 and 3.2 were 30 cm (~12 in) wide, compared to 20 cm (~8 in) in Building #1 walls and
15 cm (~6 in) in Building #2. Longitudinal bars in the boundary zone of wall 3.1 were 22 mm in diameter, and the longitudinal reinforcement in wall 3.2 was 25 mm diameter bars. Web zone bars were also much larger than those found in Buildings #1 and #2. The web bars specified in walls 3.1 and 3.2 were 16 mm diameter reinforcement and were spaced tighter than the 20 cm center to center spacing found in the other two buildings. No hoops or crossties were specified at wall boundary elements. Instead, the horizontal web bars wrapped around the boundary zone longitudinal bars and terminated with a 135 degree bend on the face opposite the web bar curtain (see Figure 4.18). Critical section geometry is provided in Table 4.7.

![Figure 4.18 Wall 3.2 horizontal web bar detailing. Note no hoops or crossties at end zones.](image)

Building #3 was the tallest structure assessed as part of this investigation, and, as such, had the largest fundamental period estimate of all seven buildings. The period estimate corresponded to the response range where spectral displacements increased significantly, to the point that the spectral displacement at the upper bound period estimate (40 inches) was five times the spectral displacement associated with the lower bound period estimate (8 inches). Since the structure also had a significant change in mass and stiffness halfway up the height of the structure, it is possible that higher mode effects had a larger participation in the structure’s dynamic response, and the simplified period estimate may not be appropriate for a structure with a non-uniform mass and stiffness distribution with respect to height.
Table 4.7 Building #3 wall section geometry.

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>General Shape</th>
<th>Vertical Discontinuity</th>
<th>Wall Length (ft)</th>
<th>Web Thickness (in)</th>
<th>Roof Height (ft)</th>
<th>Aspect Ratio</th>
<th>Web Area (in²)</th>
<th>N Flange Width (ft)</th>
<th>N Flange Thickness (in)</th>
<th>S Flange Width (ft)</th>
<th>S Flange Thickness (in)</th>
<th>Total Wall Area (in²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1</td>
<td>Rectangular</td>
<td>Yes, Web Reduction</td>
<td>12'-10&quot;</td>
<td>11.8&quot;</td>
<td>182'-9&quot;</td>
<td>14.3</td>
<td>1814</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>1814</td>
</tr>
<tr>
<td>3.2</td>
<td>N. Web Stem, S. Flange</td>
<td>Yes, Flag</td>
<td>22'-8&quot;</td>
<td>11.8&quot;</td>
<td>182'-9&quot;</td>
<td>8.1</td>
<td>3209</td>
<td>NA</td>
<td>NA</td>
<td>9'-5&quot;</td>
<td>11.8&quot;</td>
<td>4683</td>
</tr>
</tbody>
</table>

4.3.2 Special Boundary Element Trigger Results

No significant damage was reported in Building #3. Building #3’s major axes were oriented in the same direction as Building #2. Analysis indicates that the stem end of the flanged wall and the rectangular wall boundary zones would have required special detailing as the resulting neutral axis depths were more than twice the trigger depth. The roof drift ratio obtained by simple spectral displacement analysis was exceptionally large (almost 50% greater than the 2% drift limit of ASCE 7-10), as the cracked period adjustment moved the spectral response to the large “hump” that ramped up spectral displacements for fundamental periods larger than about 1.25 seconds. With such large drift demands, neutral axis depths triggering special detailing requirements were only roughly 6% of the wall web length. The calculated neutral axis depths at 0.003 compressive strain in the extreme concrete fiber greatly exceeded the trigger depth, as shown in Table 4.8. Based on the generally good performance of the walls and lack of reported damage, the roof drift estimation for Building #3 may not be representative of the true building response during the 2010 earthquake. Furthermore, live load demands may have been overestimated, as information obtained following the wall analyses revealed the building was not yet fully occupied (Birely, 2012). The lower axial load demands would have decreased the critical wall section neutral axis depth for an extreme compression strain of 0.003, but this adjustment alone likely would not have reduced the critical wall section neutral axis to a depth less than the SBE triggering value for the calculated roof drift.
Table 4.8 Building #3 SBE trigger evaluation.

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>Design Load 1.2D+0.5L+E&lt;sub&gt;c&lt;/sub&gt; (%&lt;i&gt;A_0&lt;/i&gt;f'c)</th>
<th>Roof Drift Ratio</th>
<th>Trigger Depth &lt;i&gt;c&lt;sub&gt;trig&lt;/sub&gt;/I&lt;sub&gt;web&lt;/sub&gt;&lt;/i&gt;</th>
<th>North End &lt;i&gt;c/c&lt;sub&gt;trig&lt;/sub&gt;&lt;/i&gt;</th>
<th>North BE Damage?</th>
<th>South End &lt;i&gt;c/c&lt;sub&gt;trig&lt;/sub&gt;&lt;/i&gt;</th>
<th>South BE Damage?</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1</td>
<td>26.8</td>
<td>0.0285</td>
<td>0.058</td>
<td>6.3</td>
<td>No</td>
<td>6.3</td>
<td>No</td>
</tr>
<tr>
<td>3.2</td>
<td>14.4</td>
<td>0.0285</td>
<td>0.058</td>
<td>2.71</td>
<td>No</td>
<td>0.365</td>
<td>No</td>
</tr>
</tbody>
</table>

4.3.3 Longitudinal Reinforcement Buckling Predictions

Results for Building #3 are not consistent with the observed behavior of the building. According to the simple procedure used to estimate roof displacement (drift) demands, the expected roof drift for this structure was nearly 3%. Analysis suggests the walls should have crushed once drifts exceeded about 1.2%. Horizontal web reinforcement terminated by wrapping around the wall ends and hooking 135 degrees into the core of the boundary element. Since no hoops or ties were specified for walls in this building, the spacing of horizontal web reinforcement is used to determine ratios of <i>s/d<sub>b</sub></i>. Vertical spacing of horizontal web reinforcement was 15 cm in wall 3.1 and 17 cm in wall 3.2. This configuration translated to <i>s/d<sub>b</sub></i> ratios of 6.8 in the two walls examined in Building #3, smaller than in Building #1 or #2. The walls evaluated in Building #3 would not be expected to exhibit bar buckling until the walls crushed (so long as the horizontal web bars remained engaged with the boundary element core).

Table 4.9 Building #3 Bar buckling analysis summary

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>Expected Load &lt;i&gt;D+0.25L&lt;/i&gt; (%&lt;i&gt;A_0&lt;/i&gt;f'c)</th>
<th>Web &lt;i&gt;s/d&lt;sub&gt;b&lt;/sub&gt;&lt;/i&gt; Ratio&lt;sup&gt;1&lt;/sup&gt;</th>
<th>&lt;i&gt;ε&lt;sub&gt;p&lt;/sub&gt;&lt;/i&gt;* Lower Estimate</th>
<th>&lt;i&gt;ε&lt;sub&gt;p&lt;/sub&gt;&lt;/i&gt;* Upper Estimate</th>
<th>Calc. Strain &lt;i&gt;ε&lt;sub&gt;p&lt;/sub&gt;&lt;/i&gt;*&lt;sub&gt;calc&lt;/sub&gt;</th>
<th>Long. Bar Buckling Expected?</th>
<th>Buckling Observed?</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1</td>
<td>19.6</td>
<td>6.8</td>
<td>0.02</td>
<td>0.03</td>
<td>0.015</td>
<td>No&lt;sup&gt;2&lt;/sup&gt;</td>
<td>No</td>
</tr>
<tr>
<td>3.2</td>
<td>10.5</td>
<td>6.8</td>
<td>0.02</td>
<td>0.03</td>
<td>0.025</td>
<td>No&lt;sup&gt;2&lt;/sup&gt;</td>
<td>No</td>
</tr>
</tbody>
</table>

<sup>1</sup>Based on horizontal web reinforcement spacing. No hoops or ties provided at boundary.

<sup>2</sup>Web crushing likely to occur prior to estimated roof drift demand.

87
Figure 4.19 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 3.1.

Figure 4.20 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 3.2.
4.4 Building #4 (Toledo)

4.4.1 Building Description and Wall Selection

Building #4 was the only structure from Viña del Mar selected for examination in this study. It was built in 1996 and is a 10-story building measuring 26.1 m (85 ft) tall that had one level of subterranean parking. The building site classification was classified as Soil Type III (NIST GCR 12-917-18). Similar to Building #1, Building #4 had setbacks in transverse walls at the podium level on the north side of the building to accommodate parking spaces. Like Building #1, there was significant concrete crushing and rebar buckling in walls at grade level in this structure. The typical floor plan was 37.1 m long by 14.5 m wide. Figure 4.21 shows the typical floor plan with analyzed walls highlighted and labeled. Wall critical sections at ground level have major setbacks from the level above; Figure 4.22 illustrates the wall web reduction between the podium and second floor levels.

Figure 4.21 Building #4 typical floor plan with investigated walls highlighted in red.
Walls in Building #4 were typically 20 cm thick (~8 in) with two curtains of web reinforcement. Vertically oriented web bars were 8mm diameter bars spaced 25 cm apart. Horizontal bars were typically either 8 or 10 mm bars, with vertical spacing ranging between 20 and 25 cm on center. Boundary element longitudinal bar sizes and configurations varied dramatically from wall to wall: specified sizes ranged from 18mm bars in one wall to 32mm in another, and while most walls had evenly spaced bar groups, wall 4.4 had 25mm bars adjacent to each other in the boundary zone. No hoops or crossties were specified in wall boundary zones at the podium level of Building #4. Critical wall section geometries are summarized in Table 4.1.

Extensive crushing damage was documented in most transverse walls on the north side of the building (Walls 4.1 through 4.4) at the ground level, along with story height shortening (Figure 4.23(a)). The (plan) northwest corner of the structure dropped significantly, and shoring at the ground level was required to stabilize the building (Figure 4.23(b)). Walls on the south side of Building #4 had extensive horizontal cracking, indicating significant tension demands to keep the building upright (Figure 4.23(c)).
Table 4.10 Building #4 wall section geometry.

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>General Shape</th>
<th>Vertical Discontinuity</th>
<th>Web Length (ft)</th>
<th>Web Thickness (in)</th>
<th>Roof Height (ft)</th>
<th>Aspect Ratio</th>
<th>Web Area (in²)</th>
<th>N Flange Width (ft)</th>
<th>N Flange Thickness (in)</th>
<th>S Flange Width (ft)</th>
<th>S Flange Thickness (in)</th>
<th>Total Wall Area (in²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1</td>
<td>Rectangular</td>
<td>Yes, Flag and Web Loss</td>
<td>9'-8&quot;</td>
<td>7.9&quot;</td>
<td>85'-10&quot;</td>
<td>8.9</td>
<td>915</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>915</td>
</tr>
<tr>
<td>4.2</td>
<td>N. Web Stem, S. Flange</td>
<td>Yes, Web Loss</td>
<td>17'-9&quot;</td>
<td>7.9&quot;</td>
<td>85'-10&quot;</td>
<td>4.8</td>
<td>1674</td>
<td>NA</td>
<td>NA</td>
<td>9'-7&quot;</td>
<td>7.9&quot;</td>
<td>2626</td>
</tr>
<tr>
<td>4.3</td>
<td>Rectangular</td>
<td>Yes, Flag</td>
<td>8'-8&quot;</td>
<td>7.9&quot;</td>
<td>85'-10&quot;</td>
<td>9.9</td>
<td>822</td>
<td>NA</td>
<td>NA</td>
<td>0'-0&quot;</td>
<td>0.0&quot;</td>
<td>822</td>
</tr>
<tr>
<td>4.4</td>
<td>N. Web Stem, S. Flange</td>
<td>Yes, Web Loss</td>
<td>17'-9&quot;</td>
<td>7.9&quot;</td>
<td>85'-10&quot;</td>
<td>4.8</td>
<td>1674</td>
<td>NA</td>
<td>NA</td>
<td>17'-7&quot;</td>
<td>7.9&quot;</td>
<td>3364</td>
</tr>
<tr>
<td>4.5</td>
<td>N. Flange, S. Web Stem</td>
<td>Yes, Flag</td>
<td>10'-8&quot;</td>
<td>7.9&quot;</td>
<td>85'-10&quot;</td>
<td>8.0</td>
<td>1008</td>
<td>2'-7&quot;</td>
<td>7.9&quot;</td>
<td>NA</td>
<td>NA</td>
<td>1194</td>
</tr>
<tr>
<td>4.6</td>
<td>N. Flange, S. Web Stem</td>
<td>Yes, Flag</td>
<td>10'-8&quot;</td>
<td>7.9&quot;</td>
<td>85'-10&quot;</td>
<td>8.0</td>
<td>1008</td>
<td>11'-10&quot;</td>
<td>7.9&quot;</td>
<td>NA</td>
<td>NA</td>
<td>2062</td>
</tr>
</tbody>
</table>

Figure 4.23 Documented damage in Building #4. (a) Crushing and bar buckling in northern walls; (b) story drop at northwest corner with shoring; (c) tension cracks at southern wall.

4.4.2 Special Boundary Element Trigger Results

Photos of the ground level parking area on the north side of Building #4 revealed significant damage to all wall webs (walls 4.1-4.4). Despite being shorter than Buildings #1 and #2, the target drift ratio calculated for this building was higher than for those other two structures; thus, the trigger neutral axis depth ratio was smaller for the walls in this building. In fact, of four flanged walls and two rectangular walls, all
non-flanged ends showed neutral axis depths at least twice the level that would dictate special detailing (Table 4.11). No neutral axis depth associated with flange compression triggered the more stringent detailing requirements.

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>Design Load ($1.2D+0.5L+E_v$) (%$A_{f'}$)</th>
<th>Roof Drift Ratio</th>
<th>Trigger Depth ($c_{trig}/I_{web}$)</th>
<th>North End ($c/c_{trig}$)</th>
<th>North BE Damage?</th>
<th>South End ($c/c_{trig}$)</th>
<th>South BE Damage?</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1</td>
<td>15.5</td>
<td>0.009</td>
<td>0.185</td>
<td>1.59</td>
<td>Yes</td>
<td>1.59</td>
<td>Yes</td>
</tr>
<tr>
<td>4.2</td>
<td>10.7</td>
<td>0.009</td>
<td>0.185</td>
<td>1.39</td>
<td>Yes</td>
<td>0.19</td>
<td>No</td>
</tr>
<tr>
<td>4.3</td>
<td>28.4</td>
<td>0.009</td>
<td>0.185</td>
<td>2.57</td>
<td>Yes</td>
<td>2.57</td>
<td>Yes</td>
</tr>
<tr>
<td>4.4</td>
<td>10.4</td>
<td>0.009</td>
<td>0.185</td>
<td>2.80</td>
<td>Yes</td>
<td>0.07</td>
<td>No</td>
</tr>
<tr>
<td>4.5</td>
<td>12.6</td>
<td>0.009</td>
<td>0.185</td>
<td>0.44</td>
<td>No</td>
<td>1.73</td>
<td>Yes</td>
</tr>
<tr>
<td>4.6</td>
<td>6.5</td>
<td>0.009</td>
<td>0.185</td>
<td>0.10</td>
<td>No</td>
<td>1.90</td>
<td>No</td>
</tr>
</tbody>
</table>

Photos of walls 4.1 through 4.4 revealed there were no hoops or ties at the boundary elements, only horizontal web bars terminating with hooks around the longitudinal bars. Crushed boundary concrete was documented at both ends of rectangular walls 4.1 and 4.3, and at the non-flanged ends of asymmetric walls 4.2 and 4.4 (Figure 4.24). Figure 4.25 shows concrete spalling at the (plan) south boundary, although damage was not as severe as in Walls 4.1, 4.2, 4.3, and 4.4. Damage at the south end boundary of Wall 4.6 was not reported. In five of the six wall boundaries assessed to need special boundary detailing according to U.S. code provisions, photographic evidence shows that damage to concrete occurred.
4.4.3 Longitudinal Reinforcement Buckling Predictions

Walls in Building #4 had various diameters of boundary longitudinal reinforcement and also variations in the vertical spacing between web horizontal reinforcement, which is the reason that each wall has a different $s/d_b$ ratio. No hoops or ties were present within the boundary zone of these walls. Analysis of some walls, like Wall 4.1 and Wall 4.4, did not indicate onset of bar buckling was likely to happen before the target drift level was achieved; however, other walls (e.g., Wall 4.2, and possibly Wall 4.4) indicate...
the onset of bar buckling could occur prior to concrete crushing at the wall boundary or before the target drift level was reached. It appears Wall 4.4 was especially susceptible to concrete crushing at the target drift level, and it is possible that load redistribution following damage to Wall 4.4 could have led to further damage in other walls. Analyses of Walls 4.5 and 4.6 on the south side of Building #4 indicate that longitudinal bar buckling was possible in the non-flanged boundaries; however, post-earthquake reconnaissance did not note buckled reinforcement at these locations. A post-earthquake photo taken of Wall 4.5 on the south side of the structure (unidentified) showed large tension cracks and spalling of concrete vertically roughly over a distance equal to three to four intervals between web horizontal reinforcement at the wall boundary without a flange (Figure 4.23c). As mentioned in Chapter 3, the north side of the second floor dropped almost 16 inches due to walls crushing; the residual tension in the southern walls to resist structure overturning could have straightened out bars that had previously buckled and had pushed off cover concrete at the south wall boundary of Wall 4.5.

Table 4.12 Building #4 bar buckling analysis summary

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>Expected Load D+0.25L (%Agf'c)</th>
<th>Web s/db Ratio¹</th>
<th>ε* Lower Estimate</th>
<th>ε* Upper Estimate</th>
<th>Calc. Strain ε*calc</th>
<th>Long. Bar Buckling Expected?</th>
<th>Buckling Observed?</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1</td>
<td>11.8</td>
<td>6.4</td>
<td>0.02</td>
<td>0.035</td>
<td>0.002</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>4.2</td>
<td>8.1</td>
<td>8.0</td>
<td>0.005</td>
<td>0.02</td>
<td>0.011</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>4.3</td>
<td>21.9</td>
<td>11.1</td>
<td>0.005</td>
<td>0.015</td>
<td>0.001</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>4.4</td>
<td>7.8</td>
<td>6.3</td>
<td>0.02</td>
<td>0.035</td>
<td>0.018</td>
<td>No²</td>
<td>Yes</td>
</tr>
<tr>
<td>4.5</td>
<td>9.5</td>
<td>11.4</td>
<td>0.005</td>
<td>0.015</td>
<td>0.007</td>
<td>Yes</td>
<td>No³</td>
</tr>
<tr>
<td>4.6</td>
<td>5.1</td>
<td>10.0</td>
<td>0.005</td>
<td>0.015</td>
<td>0.008</td>
<td>Yes</td>
<td>No</td>
</tr>
</tbody>
</table>

¹Based on horizontal web reinforcement spacing. No hoops or ties provided at boundary. ²Web crushing likely to occur prior to achieving estimated roof drift demand. ³Corner spalling possibly indicates bar instability (Figure 4.25)
Figure 4.26 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 4.1.

Figure 4.27 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 4.2.
Figure 4.28 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 4.3.

Figure 4.29 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 4.4.
Figure 4.30 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 4.5.

Figure 4.31 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 4.6.
4.5 Building #5

4.5.1 Building Description and Wall Selection

Building #5 is a 12-story building, 30.3 m (100 ft) tall above the podium level, with two levels of underground parking, and was built circa 2005 in Santiago. A rectangular floor plan measuring 35m long by 21m wide remains relatively unchanged from the second level to the roof. Site conditions were categorized as Soil Type II in construction documents. Walls in Building #5 were among the thinnest encountered of all the buildings in this investigation, typically only 15 cm (5.9 in) wide. Four walls in Building #5 in Santiago were examined. Two of the walls were rectangular with symmetric vertical reinforcement; the other two walls were L-shaped walls. The walls chosen were specified in an independent damage report as suffering some level of damage: exploration of the first subterranean level of parking found significant damage to walls 5.1, 5.2, and 5.4, along with spalling and bar buckling in wall 5.3. Figure 4.32 shows the typical floor plan above the podium level and identifies the walls with damage at subterranean levels. Like with other buildings discussed earlier, many of the walls at the parking levels in Building #5 have significant setbacks relative to the residential levels above.

Confining hoops were specified in Building #5, typically spaced at the same vertical interval as the horizontal web bars and usually located at web stems of flanged walls or rectangular wall ends. Post-earthquake photos showed that the locations of these hoops were staggered between web bar bends in some walls (Figure 4.33, Figure 4.34(a)), but either not present or not located intermediately between horizontal web terminations in other walls (Figure 4.34(b)). Figure 4.33 shows the stirrup callout and exposed reinforcement in a damaged wall boundary zone. Longitudinal bar sizes in boundary zones varied from wall to wall, typically between 18, 22, 25, and 28 mm. However, some walls had longitudinal bars as little as 12 mm in diameter (walls 5.3 and 5.4). Web steel distribution was fairly
standard across walls in the basement level, typically 8 mm or 10 mm bars spaced either 15 cm or 20 cm on center.

Figure 4.32 Building #5 typical floor plan with highlighted walls.

Figure 4.33 Stirrup and horizontal web bar termination at wall end in Building #5.
Damage to walls at the first subterranean level was similar in type (along the entire web length, over a few transverse reinforcement spacing intervals) to that encountered in Building #4 (Figure 4.33, Figure 4.34(a)). One interesting damage observation was at the perimeter wall 5.3 next to the subterranean parking ramp. Damage at the web end of the wall was limited to the height of one transverse reinforcement spacing interval, with cover spalled and exposed bars visibly buckled between the horizontal reinforcement (Figure 4.34(b)). The damage at this location suggests bar buckling without concrete cover spalling, and that bar buckling led to loss of concrete cover.

Damage in Building #5. (a) Crushing along length of web and bar buckling; (b) spalling and bar buckling.
4.5.2 Special Boundary Element Trigger Results

The rectangular walls (5.1 and 5.2) had high axial load demands (up around 40% of $A_g f'_c$) and required special boundary detailing according to ACI 318 Equation 21-9. These walls suffered crushing and buckling of bars at the wall boundaries and into the wall web (Figure 4.35). The flanged walls (5.3 and 5.4) were slightly less loaded than the rectangular walls but still significantly loaded (around 25% of $A_g f'_c$). The flanged walls also triggered special boundary detailing requirements at the non-flanged ends. Wall 5.4 exhibited significant crushing damage at the (plan) south non-flanged boundary (Figure 4.36) similar that found on walls 5.1 and 5.2. Wall 5.3 had some spalling at the (plan) north non-flanged end (Figure 4.37), but did not suffer the severe boundary core crushing seen in Walls 5.1, 5.2, and 5.4. Figure 4.37 reveals that the as-built Wall 5.3 had a wider thickness than specified in construction documents—the photo shows an additional layer of wall to the left of the structural wall, and there also appears to be infill to the right of the wall end—which may explain the reduction in crushing damage despite the large ratio of neutral axis depth to trigger depth (2.67). Assessments show that the wall edges exhibiting significant damage typically had neutral axis depths at 0.003 compression strain that exceeded the trigger neutral axis depth by at least 100% (Table 4.14).

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>Design Load 1.2D+0.5L+E_r (%A_gf'_c)</th>
<th>Roof Drift Ratio</th>
<th>Trigger Depth $c_{trig}/l_{web}$</th>
<th>North End $c/c_{trig}$</th>
<th>North BE Damage?</th>
<th>South End $c/c_{trig}$</th>
<th>South BE Damage?</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.1</td>
<td>39.0</td>
<td>0.0075</td>
<td>0.22</td>
<td>2.045</td>
<td>Yes</td>
<td>2.045</td>
<td>Yes</td>
</tr>
<tr>
<td>5.2</td>
<td>43.3</td>
<td>0.0075</td>
<td>0.22</td>
<td>2.285</td>
<td>Yes</td>
<td>2.285</td>
<td>Yes</td>
</tr>
<tr>
<td>5.3</td>
<td>24.1</td>
<td>0.0075</td>
<td>0.22</td>
<td>2.669</td>
<td>Slight</td>
<td>0.455</td>
<td>No</td>
</tr>
<tr>
<td>5.4</td>
<td>20.2</td>
<td>0.0075</td>
<td>0.22</td>
<td>0.076</td>
<td>No</td>
<td>2.976</td>
<td>Yes</td>
</tr>
</tbody>
</table>
Figure 4.35 Concrete crushing at boundaries in Wall 5.1 (left) and Wall 5.2 (right).

Figure 4.36 Boundary core concrete crushing and bar buckling at non-flanged end of Wall 5.4.
4.5.3 Longitudinal Reinforcement Buckling Predictions

Table 4.15 presents the results for assessing buckling of wall longitudinal reinforcement at the boundaries opposite the flange (also referred to as web stem boundary) for Building #5. All of the walls selected for analysis exhibited some level of bar buckling at the subterranean level according to damage reports. Walls 5.1, 5.2, and 5.4 had extensive damage, characterized by concrete loss, buckled vertical reinforcement, and out-of-plane displacement at the wall boundaries and along the length of the wall web. Wall 5.3 showed some concrete cover spalling over one hoop interval with slight buckling of boundary vertical reinforcement, but not the extensive damage observed in the other three walls of Building #5. The $s/d_v$ ratio for wall 5.3, at a value of 16.7, is the largest encountered among all the walls reviewed for this study.
Table 4.15 Building #5 bar buckling analysis summary

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>Expected Load $D+0.25L$ ($%A_{gf'}c'$)</th>
<th>Web $s/d_b$ Ratio</th>
<th>$\varepsilon_p^*$ Lower Estimate</th>
<th>$\varepsilon_p^*$ Upper Estimate</th>
<th>Calc. Strain $\varepsilon_p^*_{calc}$</th>
<th>Long. Bar Buckling Expected?</th>
<th>Buckling Observed?</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.1</td>
<td>28.5</td>
<td>6.0</td>
<td>0.03</td>
<td>0.04</td>
<td>0.0015</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>5.2</td>
<td>31.7</td>
<td>11.1</td>
<td>0.005</td>
<td>0.015</td>
<td>0.0014</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>5.3</td>
<td>17.6</td>
<td>16.7</td>
<td>0.005</td>
<td>0.01</td>
<td>0.0065</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>5.4</td>
<td>14.8</td>
<td>16.7</td>
<td>0.005</td>
<td>0.01</td>
<td>0.010</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>

Section analysis revealed that most of these walls were compression controlled (i.e., the strain at the compression end of the wall reaches 0.003 prior to tensile yielding of boundary longitudinal reinforcement). It is less likely that compression-controlled walls can develop large plastic hinge lengths, such as the $0.5l_w$ assumed in the methodology used to obtain bar strains, meaning the calculated moment-drift curves in the following figures may overestimate the ductility/drift capacity of these walls (and therefore underestimate the longitudinal bar indicator strains). Walls 5.1 and 5.2 both had fairly large axial loads (about 30% of $A_{gf'}c'$) and probably experienced web end crushing. Another issue with analyses of walls in Building #5 was that Wall 5.4 had an extremely large amount of longitudinal reinforcement in its flanged end, to a degree that the longitudinal bars do not yield prior to web crushing according to section analysis (BIAX) results. In this case, “first yield”, as defined in the approach in Section 3.4, was modified to be the curvature at which extreme fiber concrete began to degrade.
Figure 4.38 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 5.1.

Figure 4.39 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 5.2.
Figure 4.40 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 5.3.

Figure 4.41 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 5.4.
4.6 Building #6 (Emerald)

4.6.1 Building Description and Wall Selection

Building #6, built in 2006, is a 20 story structure with 4 subterranean levels of parking in Santiago. The main structure had a roof height of 50.7m (166 ft) above the podium level. Site conditions for Building #6 were classified as Soil Type II. The typical floor plan was roughly trapezoidal, with a width of approximately 8m on one end increasing to 14m wide at the opposite end, and a length of 66m. Figure 4.42 shows the typical floor plan for levels above the podium and highlights the walls chosen for this investigation. Three damaged walls (two severely damaged) were selected from Building #6 and analyzed: all selected walls had flanges, and were located on the (plan) north side of the structure. Web thickness varies from wall to wall, ranging from 17cm (6.8 in) at the narrowest in wall 6.3 up to 25cm (9.8 in) at the widest in wall 6.1. No hoops or crossties are provided at wall end zones at the subterranean level; horizontal web steel wraps around the boundary element longitudinal bars and terminates with 90 degree bends. Figure 4.43 illustrates the typical wall edge horizontal web bar termination detail. Walls 6.1, 6.2 and 6.3 all had vertical discontinuities (setbacks) at the first subterranean level. Figure 4.44 illustrates the setback in wall 6.1. Damage was located in walls at the first level of underground parking. Spalling of cover concrete, bar buckling and fracture, horizontal bar bends unfurling and concrete crushing at wall ends were documented in several transverse walls at the north side of the building at this level. Note that although crossties are shown at alternating sets of bars per Figure 4.43, photos of damaged end zones suggest that these ties were not provided.
Figure 4.42 Building #6 typical floor plan and walls selected for analysis.

Figure 4.43 Building #6 wall edge detail and photo of unhooked horizontal bars. The wall detail appears to specify hooks on alternating bar sets, but they are not apparent in the wall photo.

Figure 4.44 Setback typical of northern transverse walls at 1st subterranean level in Building #6.
### Table 4.16 Building #6 wall section geometry.

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>General Shape</th>
<th>Vertical Discontinuity</th>
<th>Web Length (ft)</th>
<th>Web Thickness (in)</th>
<th>Roof Height (ft)</th>
<th>Aspect Ratio</th>
<th>Web Area (in²)</th>
<th>N Flange Width (ft)</th>
<th>N Flange Thickness (in)</th>
<th>S Flange Width (ft)</th>
<th>S Flange Thickness (in)</th>
<th>Total Wall Area (in²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.1</td>
<td>N. Web Stem, S. Flange</td>
<td>Yes, Flag</td>
<td>19'-2&quot;</td>
<td>9.8</td>
<td>166'-2&quot;</td>
<td>8.7</td>
<td>2259</td>
<td>NA</td>
<td>NA</td>
<td>16'-9&quot;</td>
<td>6.7</td>
<td>3669</td>
</tr>
<tr>
<td>6.2</td>
<td>N. Web Stem, S. Flange</td>
<td>Yes, Flag</td>
<td>15'-10&quot;</td>
<td>7.9</td>
<td>166'-2&quot;</td>
<td>10.5</td>
<td>1494</td>
<td>NA</td>
<td>NA</td>
<td>11'-6&quot;</td>
<td>6.7</td>
<td>2469</td>
</tr>
<tr>
<td>6.3</td>
<td>N. Web Stem, S. Flange</td>
<td>Yes, Flag</td>
<td>15'-10&quot;</td>
<td>6.7</td>
<td>166'-3&quot;</td>
<td>10.5</td>
<td>1270</td>
<td>NA</td>
<td>NA</td>
<td>7'-4&quot;</td>
<td>6.7</td>
<td>1905</td>
</tr>
</tbody>
</table>

Similar to Buildings #1, #4, and #5, Building #6 had wall damage primarily on one side of the structure. Damage was focused almost exclusively in transverse walls with setbacks to accommodate subterranean parking on the (plan) north side of the building. Transverse walls on the (plan) south side of the structure generally did not have vertical discontinuities (reduced wall area relative to the floor above), and did not have the extensive damage observed in the northern part of the building.

### 4.6.2 Special Boundary Element Trigger Results

Factored axial load demands were large (23%-45% of $A_g f'_c$) in the walls in Building #6. All walls investigated had neutral axis depths associated with 0.003 extreme fiber compression strain that significantly exceeded both the unaltered trigger neutral axis depth via ACI 318-11 equation 21-8 and the modified neutral axis depth using the drift ratio less than 0.007 (Table 4.17). Wall 6.1 exhibited some minor spalling near the ceiling (Figure 4.45), but did not have the level of damage observed in the other two walls selected for investigation. Walls 6.2 and 6.3 had crushed boundary elements and buckled longitudinal bars at the web ends, with damage extending through the web and into the flanged end (Figure 4.46 and Figure 4.47). The damage in the flanged end of these walls was probably not due to
boundary element crushing at that end, but probably due to damage propagation along the length of the wall after the web stem boundary zones crushed.

Table 4.17 Building #6 SBE trigger evaluation.

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>Design Load $1.2D + 0.5L + E_v$ (%$A_f f'_c$)</th>
<th>Roof Drift Ratio</th>
<th>Trigger Depth $c_{trig}/I_{web}$</th>
<th>Trigger Depth $c_{trig}/I_{web}$</th>
<th>North End $c/c_{trig}$</th>
<th>North End $c/c_{trig}$</th>
<th>North BE Damage?</th>
<th>South End $c/c_{trig}$</th>
<th>South End $c/c_{trig}$</th>
<th>South BE Damage?</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.1</td>
<td>22.7</td>
<td>0.006</td>
<td>0.24</td>
<td>0.27</td>
<td>2.32</td>
<td>2.06</td>
<td>Slight</td>
<td>0.15</td>
<td>0.13</td>
<td>No</td>
</tr>
<tr>
<td>6.2</td>
<td>28.1</td>
<td>0.006</td>
<td>0.24</td>
<td>0.27</td>
<td>2.59</td>
<td>2.30</td>
<td>Yes</td>
<td>0.43</td>
<td>0.38</td>
<td>Yes</td>
</tr>
<tr>
<td>6.3</td>
<td>45.1</td>
<td>0.006</td>
<td>0.24</td>
<td>0.27</td>
<td>3.34</td>
<td>2.96</td>
<td>Yes</td>
<td>1.79</td>
<td>1.58</td>
<td>Yes</td>
</tr>
</tbody>
</table>

Figure 4.45 Spalled cover concrete at (plan) north boundary of Wall 6.1.
Figure 4.46 Concrete cover spalling, bar buckling, and out-of-plane buckling at (plan) north boundary of Wall 6.2.

Figure 4.47 Composite image of damage to Wall 6.3 flange (left) and web stem boundary (right).
4.6.3 Longitudinal Reinforcement Buckling Predictions

Table 4.18 summarizes the results for Building #6. Damage concentrated in the 1st subterranean level and revealed that no crossties or hoops were provided at the wall boundary; therefore, for the $s/d_h$ ratio reported, the spacing reflects the vertical distance between horizontal web reinforcement. Web horizontal reinforcement terminated with 90 degree bends around the boundary longitudinal reinforcement, with no engagement to the concrete core within the longitudinal bars. Critical wall section analyses presented in Chapter 4 resulted in neutral axis depths exceeding twice the limiting value of the ACI 318-11 trigger depth, indicating concrete cover crushing and spalling was likely for these walls. Loss of concrete cover and the lack of horizontal reinforcement engaging the concrete core likely diminished the effectiveness of said horizontal reinforcement in restraining boundary longitudinal reinforcement, likely precipitating bar buckling. The walls in Building #6 likely crushed prior to boundary longitudinal reinforcement buckling, but once cover concrete was lost, bar buckling developed.

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>Expected Load $D+0.25L$ (%$A_{g,f}$)</th>
<th>Web $s/d_h$ Ratio$^1$</th>
<th>$\varepsilon_{p*}$ Lower Estimate</th>
<th>$\varepsilon_{u*}$ Upper Estimate</th>
<th>Calc. Strain $\varepsilon_{p*}^{calc}$</th>
<th>Long. Bar Buckling Expected?</th>
<th>Buckling Observed?</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.1</td>
<td>16.6</td>
<td>5.2</td>
<td>0.045</td>
<td>0.06</td>
<td>0.0015</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>6.2</td>
<td>20.6</td>
<td>5.2</td>
<td>0.02</td>
<td>0.03</td>
<td>0.0014</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>6.3</td>
<td>33.0</td>
<td>4.8</td>
<td>0.045</td>
<td>0.06</td>
<td>0.0018</td>
<td>No</td>
<td>Yes</td>
</tr>
</tbody>
</table>

$^1$Based on horizontal web reinforcement spacing. No hoops or ties provided at boundary.

Expected loads in the walls investigated from Building #6 were quite large, and none of the walls were tension-controlled when loaded such that the webs were in compression. For Walls 6.2 and 6.3, the calculated tensile strains in boundary longitudinal reinforcement barely surpassed yield strain (6.2) or
never yielded (6.3). Since the curvature-to-drift relationship used in predicting longitudinal reinforcement strains relied on a first-yield curvature parameter, the curvature associated with a compressive strain of 0.003 was instead chosen as a substitute for the yield curvature of the wall. The resulting moment-drift curves probably overestimate the wall deformation capacity prior to strength loss since the relations are derived from assuming a plastic hinge length equal to one-half the wall length (which is too large for a compression controlled wall with boundary element detailing that does not satisfy SBE requirements). Figure 4.48 through Figure 4.50 illustrate the approximated compression reloading strain for the equal drift in each building direction assumption, and compares the strain to moment-drift curves for wall ends opposite a flange in compression of all three walls investigated from Building #6.

Figure 4.48 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 6.1.
Figure 4.49 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 6.2.

Figure 4.50 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 6.3.
Equal displacement strain values for the three walls investigated in Building #6 do not suggest that bar buckling was the initiator of failure of Walls 6.2 and 6.3. Due to such large axial loads on the walls, and consistent with the analysis, concrete crushing probably occurred first, followed by bar instability once the restraint provided by the cover concrete was lost. As noted previously, loss of concrete cover would allow the 90-degree bends around boundary longitudinal reinforcement to open, effectively increasing the s/db ratio and reducing the amount of strain reloading necessary to buckle the longitudinal reinforcement.

4.7 Building #7 (Magnolio)

4.7.1 Building Description and Wall Selection

Building #7, also located in Santiago, is a 10-story structure (9 stories and a large penthouse level) with an underground parking level. It was oriented perpendicular to Buildings #5 and #6, with the transverse axis roughly coinciding with North-South. Figure 4.51 presents the typical floor plan above the podium level. General notes from design drawings specified that seismic analysis of the building was based on a soil classification of soil type II. An independent report (Lemnitzer et al., 2014) documented shear wall web damage at the podium level, and crushed columns and concrete cover spalling with minor bar buckling at several wall boundaries in the subterranean level. The damaged podium-level structural wall connected two smaller wall piers above the second level, and it exhibited diagonal cracking in the wall web between those piers, indicative of high shear demands. It did not suffer damage to the wall boundaries, and, because of its shape linking two smaller walls above the second floor, was not selected for investigation. Instead, an adjacent wall was chosen (Wall 7.1): it was neither damaged at the ground floor nor at the subterranean parking level. The ground level configuration was modeled as the critical section, since at the subterranean level its flanges framed into the perimeter wall and a long corridor wall. On the south side of the structure, three boundary elements at the parking level were documented as being damaged. All three damaged walls had very similar L-shaped geometry, and two of those walls had the
same (but mirrored) geometry. Damage was located at the wall boundary over approximately one to two spacings of horizontal web reinforcement, and seemed to be exacerbated by a mechanical duct cutout in the wall web near the end of the walls.

![Figure 4.51 Typical floor plan for Building #7. Walls investigated highlighted in red.](image_url)

As mentioned above, Wall 7.1 did not suffer boundary damage at the basement level: at the parking level it joined into the north perimeter wall at one end and a long perpendicular flange (corridor wall) at the other end. Instead, the critical wall section was at the ground level. Wall 7.1 had a vertical discontinuity at the south flange at the ground floor, but it is best described as a barbell shaped wall at the critical level: the south flange terminates at the podium level, and the region below was a thickened column-like region approximately twice as wide as the wall web contains the web edge. An example section cut is provided in Figure 4.52. These wall edge barbells had hoops that surrounded the boundary zone longitudinal bars and the entire thickened area, providing more robust section detailing (some confinement of a thicker boundary zone) than investigated wall boundaries from other buildings. Since this wall did not suffer
boundary damage at this level, it offers an opportunity to see if studies can predict that the wall would not have been damaged.

![Figure 4.52 Wall 7.1 elevation at ground level with barbell section cuts.](image)

Walls 7.2 and 7.3 had some concrete cover spalling and one wall had what appeared to be minor longitudinal reinforcement buckling. Concrete spalling and longitudinal reinforcement buckling extended from the wall edge into the wall web until it intersected a mechanical duct cutout in the web. Wall 7.2 is a flanged wall that has a considerable setback below the podium level (a web reduction of approximately 45%). Unlike other walls with significant setbacks, additional diagonal reinforcement was placed as a strut to transfer load from the wall edge above the critical level (Figure 4.53). Table 4.1 provides general wall dimensions for the selected walls in Building #7. Wall elevation details indicate hoops at the wall boundaries selected for investigation, and photos of the damaged walls confirm their inclusion. General notes from the construction documents specify that all stirrups/hoops terminate with hooks at 135 degree angles, which was an uncommon detail for the buildings selected for this investigation. Figure 4.54
shows that the exposed horizontal web bars began to unfurl, but there is no evidence of the boundary transverse reinforcement losing shape.

Figure 4.53 Wall 7.2 elevation (7.3 similar).
Figure 4.54 Damage at non-flanged end of wall 7.2.

Table 4.19 Building #7 wall section geometry.

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>General Shape</th>
<th>Vertical Discontinuity</th>
<th>Web Length (ft)</th>
<th>Web Thickness (in)</th>
<th>Roof Height (ft)</th>
<th>Aspect Ratio</th>
<th>Web Area (in²)</th>
<th>N Flange Width (ft)</th>
<th>N Flange Thickness (in)</th>
<th>S Flange Width (ft)</th>
<th>S Flange Thickness (in)</th>
<th>Total Wall Area (in²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.1</td>
<td>Barbell</td>
<td>Yes, Flange Reduction</td>
<td>30'-1&quot;</td>
<td>6.7&quot;</td>
<td>92'-4&quot;</td>
<td>3.1</td>
<td>2419</td>
<td>1'-4&quot;</td>
<td>24.6&quot;</td>
<td>1'-4&quot;</td>
<td>29.9&quot;</td>
<td>3313</td>
</tr>
<tr>
<td>7.2</td>
<td>North Flange</td>
<td>Yes, Web Reduction</td>
<td>7'-11&quot;</td>
<td>9.8&quot;</td>
<td>92'-4&quot;</td>
<td>11.6</td>
<td>1125</td>
<td>4'-11&quot;</td>
<td>6.7&quot;</td>
<td>--</td>
<td>--</td>
<td>1261</td>
</tr>
<tr>
<td>7.3</td>
<td>North Flange</td>
<td>Yes, Web Reduction</td>
<td>7'-11&quot;</td>
<td>11.8&quot;</td>
<td>92'-4&quot;</td>
<td>11.6</td>
<td>937</td>
<td>4'-11&quot;</td>
<td>6.7&quot;</td>
<td>--</td>
<td>--</td>
<td>1447</td>
</tr>
</tbody>
</table>

4.7.2 Special Boundary Element Trigger Results

Low level drift ratio demands (0.005) were obtained from simple analysis of Building #7, which was below the 0.007 limit of the ACI equation. As such, neutral axis depths obtained from moment-curvature analyses were compared to the regular ACI trigger depth assuming a minimum drift and a trigger depth.
using the drift obtained by analysis. No significant damage was reported at either end of Wall 7.1. The neutral axis depth on one side of Wall 7.1 exceeded the trigger limit using the code minimum drift of 0.007, but when using the estimated roof drift, the neutral axis depth would not exceed the trigger neutral axis depth, $c_{trig}^*$. Of the two damaged walls examined, both exceed the trigger value using the code minimum drift ratio by at least 50%, and both walls trigger the special boundary provisions if the smaller drift ratio is used in lieu of the minimum drift required by ACI 318 equation 21-8 (Table 4.20) by at least 15%. Boundary element damage was limited to concrete cover spalling in these walls.

Table 4.20 Building #7 SBE trigger evaluation.

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>Design Load 1.2D+0.5L+E_r (%$A_g f'_c$)</th>
<th>Roof Drift Ratio</th>
<th>Trigger Depth $c_{trig}/A_{web}$</th>
<th>Trigger Depth $c_{trig}/A_{web}$</th>
<th>North End $c/c_{trig}$</th>
<th>North End $c/c_{trig}$</th>
<th>North BE Damage?</th>
<th>South End $c/c_{trig}$</th>
<th>South End $c/c_{trig}$</th>
<th>South BE Damage?</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.1</td>
<td>9.4</td>
<td>0.0053</td>
<td>0.24</td>
<td>0.31</td>
<td>0.64</td>
<td>0.48</td>
<td>No</td>
<td>1.26</td>
<td>0.95</td>
<td>No</td>
</tr>
<tr>
<td>7.2</td>
<td>17.5</td>
<td>0.0053</td>
<td>0.24</td>
<td>0.31</td>
<td>0.35</td>
<td>0.27</td>
<td>No</td>
<td>1.69</td>
<td>1.28</td>
<td>Yes</td>
</tr>
<tr>
<td>7.3</td>
<td>15.4</td>
<td>0.0053</td>
<td>0.24</td>
<td>0.31</td>
<td>0.33</td>
<td>0.25</td>
<td>No</td>
<td>1.52</td>
<td>1.15</td>
<td>Yes</td>
</tr>
</tbody>
</table>

4.7.3 Longitudinal Reinforcement Buckling Predictions

Expected axial loads in the walls selected from Building #7 were considerably lower than those calculated in Building #6. Use of a plastic hinge length equal to $L_{w}/2$ for Wall 7.1 exceeds the story height; therefore, the plastic hinge length was taken as equal to the story height in the drift-curvature calculation. No adjustments were necessary for Walls 7.2 and 7.3. Results are presented in Table 4.21:
Wall 7.2 (and walls with similar geometry) exhibited signs of slight buckling of longitudinal reinforcement at the basement level (Figure 4.55—second bar from the right). The following plots in Figure 4.56 through Figure 4.58 show that neither strength loss due to crushing of the boundary element nor buckling of the boundary longitudinal reinforcement was expected at the estimated roof drift demand for Building #7. This was consistent with observations after the earthquake: the walls investigated did not show signs of significant structural damage.

![Figure 4.55 Damage at non-flanged end of Wall 7.2 near ceiling of subterranean level.](image)

Table 4.21 Building #7 bar buckling analysis results.

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>Expected Load $D+0.25L$ ($%A_f f'_c$)</th>
<th>Web $s/d_b$ Ratio</th>
<th>$\varepsilon^*_p$ Lower Estimate</th>
<th>$\varepsilon^*_p$ Upper Estimate</th>
<th>Calc. Strain $\varepsilon^*_p$ calc</th>
<th>Long. Bar Buckling Expected?</th>
<th>Buckling Observed?</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.1</td>
<td>6.9</td>
<td>8.3</td>
<td>0.005</td>
<td>0.02</td>
<td>0.0044</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>7.2</td>
<td>12.5</td>
<td>6.8</td>
<td>0.01</td>
<td>0.02</td>
<td>0.0009</td>
<td>No</td>
<td>Slight$^1$</td>
</tr>
<tr>
<td>7.3</td>
<td>11.0</td>
<td>11.1</td>
<td>0.005</td>
<td>0.015</td>
<td>0.0009</td>
<td>No</td>
<td>No</td>
</tr>
</tbody>
</table>

$^1$One vertical bar, not at extreme edge, displayed slight bending between hoops.
Figure 4.56 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 7.1.

Figure 4.57 Moment capacity and bar buckling strain indicator vs. roof drift for Wall 7.2.
Summary of Chilean Building Analyses

Midrise Buildings #1, #4, #5, and #6 exemplify the potential for damage in walls when either excessive axial loads are present or large displacement demands are expected. Buildings #1 and #4 had walls with factored axial loads typically between $0.09 - 0.15A_gf'_c$, and had target drifts approaching 1% based on spectra from nearby ground motion recording stations. Neither building had hoops confining the longitudinal bars at the wall boundary where damage accrued. All walls selected for investigation in Buildings #5 and #6 had design axial loads exceeding $0.2A_gf'_c$ (and up to $0.45A_gf'_c$ in several walls). Calculated drift demands were close to the ACI 318 minimum value used in Equation 21-8 (~0.007). Neither building had transverse reinforcement that satisfied ACI 318-11 SBE detailing requirements; both buildings had wall damage indicative of boundary element crushing and bar buckling.
The damage observed in Buildings #1, #4, #5, and #6 along with the trigger analyses indicate that if the ACI 318 trigger neutral axis is exceeded and special boundary detailing is not provided, then significant and concentrated damage is likely. Results for Building #2 and Wall 7.1 of Building #7, where damage was not observed in boundary elements and trigger limits were not exceeded (or marginally exceeded), provide evidence for the converse. In contrast to the axial loads calculated for the buildings with reported damage, design axial load estimates in selected walls from these buildings did not exceed $0.17A_g f'_c$, and drift ratio demands were estimated to be around 0.005 for each building. Relatively low axial demands coupled with small roof drifts yielded results indicating special boundary elements would not have been required in four of the six chosen walls. Hoops with seismic hooks were provided at wall edges, but not to a level that would have satisfied special boundary element detailing requirements of ACI 318-11. Damage in walls of Building #7 (7.2 and 7.3) was limited to spalling of concrete cover at the boundary zones, and not the excessive core crushing observed in Buildings #1, #4, #5, and #6.

In general, the higher the roof-drift ratio demand, the more likely the walls would have 1) been damaged at the wall boundary zones, and 2) required special boundary detailing if ACI 318-11 SBE provisions were applied without exception. Buildings #1, #4, #5, and #6 had four of the five highest roof drift ratio estimates calculated in this study, whereas Buildings #2 and #7 had the two lowest roof drift ratio demands. The exception was Building #3, which had a roof drift ratio demand of almost 3%, had walls that would have triggered special boundary detailing provisions for this drift level, but did not reportedly suffer major damage.

The trigger analyses results summarized here have three important implications for US (and Chile) practice: (1) the impact of axial load on wall deformation capacity and the need for SBEs should be reviewed, (2) the potential impact of wall cross section configuration should be reviewed since wall web
boundaries opposite flanges are identified as being very susceptible to concrete crushing and rebar buckling, and (3) the impact (or consequence) of displacement demands exceeding values used for design should be evaluated. The greater level of wall damage reported in Chile following the February 2010 earthquake relative to the March 1985 earthquake was very likely impacted by all three of these factors. Other factors that likely contributed to the greater level of observed damage in 2010 included the use of thinner walls (15 to 20 cm in 2010 versus 25 to 30 cm in 1985) and reduction to wall lengths (setbacks) at or just below the ground line.

In tension-controlled wall sections, bar strain analyses at the target drift level generally resulted in large boundary longitudinal reinforcement strains at non-flanged wall edges (rectangular wall configurations and the web stem ends of T- or L-shaped walls). Buckling analyses of longitudinal bars in the flanged areas of wall sections suggested lower axial strains in reinforcement and onset of bar buckling is unlikely at these locations. For flanged wall sections, relatively deep neutral axis depths when the non-flanged edges are loaded in compression would limit tensile strains in bars in the flanged area; likewise very shallow neutral axis depths associated with flange compression would yield very small compressive strains in these bars.

Analyses of several compression-controlled wall sections with high axial loads ($\geq 0.2A_f f'_c$) indicated that neither the onset of bar buckling nor strength loss due to concrete crushing was likely at the expected roof drift limit. Following the earthquake, all of these walls were damaged near the wall base where wall setbacks occurred. This discrepancy between analysis and observed performance may be due to limitations in the drift-curvature relationship approach employed and the assumption that plane sections remain plane after loading. Due to the relatively large neutral axis depths for compression-controlled walls, moment-curvature analyses results indicated that extreme tension fibers would not exceed yield
strains significantly, which in turn would minimize the reloading strain prior to returning to a zero-drift state. Using a plastic hinge length of one-half the wall web length is probably not appropriate for these walls since compression controlled walls have less ductility than tension controlled walls, and because of the lack of confining hoops and ties does not allow nonlinear concrete compressive strains to be distributed over the assumed plastic hinge height. Furthermore, stress/strain concentrations due to re-entrant corners at wall setbacks would cause higher strain levels than predicted by the drift-curvature relationship.

Table 4.22 provides a summary of the SBE trigger analysis results, longitudinal bar buckling predictions, observed damage, and general observations for each wall investigated. Of the 27 walls selected for analysis, 20 walls would have triggered SBE provisions per ACI 318-11 and suffered some crushing and spalling of concrete; 3 walls did not trigger SBE provisions per ACI 318 and did not experience crushing at the boundaries; 2 walls marginally exceeded the trigger limit using the code minimum drift, but did not exceed the trigger using the estimated drift (lower than the ACI 318 limit), and showed no signs of crushing damage; and 2 walls far exceeded the SBE limit but were undamaged. Of the 27 wall longitudinal reinforcement buckling investigations, only 15 walls matched observed damage with buckling predictions (56%). If cases where significant concrete crushing occurred and bars would have had to buckle to maintain compatibility as the wall shortened (e.g., several of the walls from Building #4), the percentage of wall analyses that matched observations jumps to 71%.
<table>
<thead>
<tr>
<th>Wall ID</th>
<th>SBE Triggered</th>
<th>Crushing Damage Reported</th>
<th>Buckling Predicted</th>
<th>Buckling Reported</th>
<th>Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>No boundary hoops; Flag shape terminates at podium level; Building collapsed in direction of crushed web; Bar buckling presumed to have occurred as part of the collapse</td>
</tr>
<tr>
<td>1.2</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>No boundary hoops; Flag continues to podium; Building collapse pulled wall apart; Bar buckling presumed to have occurred as part of the collapse</td>
</tr>
<tr>
<td>1.3</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>No boundary hoops; Flag shape terminates at podium level; Building collapsed in direction of crushed web; Bar buckling presumed to have occurred as part of the collapse</td>
</tr>
<tr>
<td>1.4</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>No boundary hoops; Flag continues to podium; Building collapse pulled wall apart; Bar buckling presumed to have occurred as part of the collapse</td>
</tr>
<tr>
<td>1.5</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>No boundary hoops; Flag shape terminates at podium level; Building collapsed in direction of crushed web; Bar buckling presumed to have occurred as part of the collapse</td>
</tr>
<tr>
<td>1.6</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>No boundary hoops; Flag continues to podium; Building collapse pulled wall apart; Bar buckling presumed to have occurred as part of the collapse</td>
</tr>
<tr>
<td>2.1</td>
<td>N</td>
<td>N</td>
<td>N</td>
<td>N</td>
<td>Boundary hoops with seismic ties; Not damaged</td>
</tr>
<tr>
<td>2.2</td>
<td>Y*</td>
<td>N</td>
<td>N</td>
<td>N</td>
<td>Boundary hoops with seismic ties; SBE trigger limit exceeded by 9% with drift ratio limit (0.007), but did not exceed with drift ratio estimate (0.0053); Not damaged</td>
</tr>
<tr>
<td>2.3</td>
<td>N</td>
<td>N</td>
<td>N</td>
<td>N</td>
<td>Boundary hoops with seismic ties; Not damaged</td>
</tr>
<tr>
<td>Wall ID</td>
<td>SBE Triggered</td>
<td>Crushing Damage Reported</td>
<td>Buckling Predicted</td>
<td>Buckling Reported</td>
<td>Commentary</td>
</tr>
<tr>
<td>---------</td>
<td>---------------</td>
<td>--------------------------</td>
<td>--------------------</td>
<td>------------------</td>
<td>------------</td>
</tr>
<tr>
<td>3.1</td>
<td>Y</td>
<td>N</td>
<td>N*</td>
<td>N</td>
<td>No boundary hoops; Web reinforcement had 135 degree hooks; Not damaged; Walls lack drift capacity calculated</td>
</tr>
<tr>
<td>3.2</td>
<td>Y</td>
<td>N</td>
<td>N*</td>
<td>N</td>
<td>No boundary hoops; Web reinforcement had 135 degree hooks; Not damaged; Walls lack drift capacity calculated</td>
</tr>
<tr>
<td>4.1</td>
<td>Y</td>
<td>Y</td>
<td>N</td>
<td>Y</td>
<td>No boundary hoops; Considerable setback at podium; Severe crushing and buckling of reinforcement</td>
</tr>
<tr>
<td>4.2</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>No boundary hoops; Considerable setback at podium; Severe crushing and buckling of reinforcement</td>
</tr>
<tr>
<td>4.3</td>
<td>Y</td>
<td>Y</td>
<td>N</td>
<td>Y</td>
<td>No boundary hoops; Considerable setback at podium; Severe crushing and buckling of reinforcement</td>
</tr>
<tr>
<td>4.4</td>
<td>Y</td>
<td>Y</td>
<td>N</td>
<td>Y</td>
<td>No boundary hoops; Considerable setback at podium; Severe crushing and buckling of reinforcement</td>
</tr>
<tr>
<td>4.5</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>N*</td>
<td>No boundary hoops; Setback at podium; Edge spalled; Wall in residual tension to prevent overturning</td>
</tr>
<tr>
<td>4.6</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>N</td>
<td>No boundary hoops; No visual evidence of damage; Assumed to be in residual tension</td>
</tr>
<tr>
<td>5.1</td>
<td>Y</td>
<td>Y</td>
<td>N</td>
<td>Y</td>
<td>Boundary hoops provided; High axial load demands; Considerable setback below podium; Extensive crushing and buckling</td>
</tr>
<tr>
<td>5.2</td>
<td>Y</td>
<td>Y</td>
<td>N</td>
<td>Y</td>
<td>Boundary hoops provided; High axial load demands; Considerable setback below podium; Extensive crushing and buckling</td>
</tr>
<tr>
<td>5.3</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Boundary hoops provided; High axial load demands; Part of &quot;pants&quot;-type wall; Considerable setback below podium; Extensive crushing and buckling; Ruptured longitudinal boundary bars</td>
</tr>
</tbody>
</table>
Table 4.22 Chilean wall analyses summary (continued)

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>SBE Triggered</th>
<th>Crushing Damage Reported</th>
<th>Buckling Predicted</th>
<th>Buckling Reported</th>
<th>Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.4</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Boundary hoops provided; High axial load demands; Parts of &quot;pants&quot;-type wall; Considerable setback below podium; Web possibly thicker than specified; Spalling and cracking, but no core crushing; Buckled edge longitudinal bars</td>
</tr>
<tr>
<td>6.1</td>
<td>Y</td>
<td>Y</td>
<td>N</td>
<td>N</td>
<td>No boundary hoops; Ties specified but may not exist; Concrete cover spalled; No buckling reported</td>
</tr>
<tr>
<td>6.2</td>
<td>Y</td>
<td>Y</td>
<td>N</td>
<td>Y</td>
<td>No boundary hoops; Ties specified but do not exist; Extensive crushing and bar buckling; Out of plane buckling</td>
</tr>
<tr>
<td>6.3</td>
<td>Y</td>
<td>Y</td>
<td>N</td>
<td>Y</td>
<td>No boundary hoops; Ties specified but do not exist; Extensive crushing and bar buckling; Out of plane buckling</td>
</tr>
<tr>
<td>7.1</td>
<td>Y*</td>
<td>N</td>
<td>N</td>
<td>N</td>
<td>Barbell boundaries with boundary hoops; SBE trigger limit exceeded by 9% with drift ratio limit (0.007), but did not exceed with drift ratio estimate (0.0053); Not damaged</td>
</tr>
<tr>
<td>7.2</td>
<td>Y</td>
<td>Y</td>
<td>N</td>
<td>N</td>
<td>Boundary hoops provided; Considerable setback below podium; Internal diagonal strut provided at podium level; Concrete cover spalled; Single bar, not at extreme edge, buckled</td>
</tr>
<tr>
<td>7.3</td>
<td>Y</td>
<td>Y</td>
<td>N</td>
<td>Y</td>
<td>Boundary hoops provided; Considerable setback below podium; Internal diagonal strut provided at podium level; Concrete cover spalled; No bars buckled</td>
</tr>
</tbody>
</table>

Based on the analyses of the various walls, there are several instances that clearly identify concrete crushing due to large drift demands, high axial loads, and relatively sparse boundary transverse reinforcement to confine boundaries, as well as instances where combinations of ratios of spacing of boundary transverse reinforcement relative to longitudinal reinforcement diameters ($s/d_b$) and longitudinal
reinforcement strain history estimates that indicate bar buckling was possible. These conditions overlap in several walls, and the extreme, long-duration shaking obfuscates whether longitudinal reinforcement buckling can initiate damage and failure in a wall, if it occurs concurrently with boundary concrete spalling and crushing, or if it is a symptom of concrete crushing. Research presented in Chapters 5, 6, and 7 aims to address this issue: prisms representative of wall boundaries are designed and tested under reverse-cyclic loading to attempt to generate buckling of longitudinal reinforcement prior to concrete cover loss.
CHAPTER 5  LABORATORY TEST DESIGN & CONSTRUCTION

Results presented in Chapter 4, assessing the need for special boundary elements and the potential role of buckling of boundary longitudinal reinforcement for walls in seven buildings in Chile, provide insight into reasons for the wall damage observed following the 2010 earthquake impacting central Chile. However, since damage assessments were obtained after strong, long-duration shaking, it is impossible to determine conclusively whether section (member) damage was initiated by concrete crushing (followed by bar buckling) or by bar buckling that led to loss of concrete cover and damage to the concrete core. To investigate this issue, a series of eight tests were performed in the UCLA Structures/Earthquake Engineering Laboratory on reinforced concrete prisms (or columns) representative of structural wall ordinary boundary elements. The eight tests were constructed and tested in two groups: Group I was funded as part of a NIST research project, and Group II was funded as part of a NEESR project. The information presented in this chapter focuses on the development of the test matrix, specimen design and construction, and material properties. Information pertaining to the loading system devised to carry out the tests, data acquisition, and loading history protocols is discussed in Chapter 7.

5.1  Test matrix

ACI 318-11 section 21.9.6.5 permits vertical spacing of boundary transverse reinforcement up to eight inches if wall extreme fiber compressive stress is less than 0.2$f_c'$ or the wall neutral axis depth is less than a critical value (to represent the condition where concrete compressive strain at the wall boundary is less than approximately 0.003; see Wallace and Orakcal, 2002). NIST Technical Brief No. 6 (2011) refers to boundary regions of US walls with eight inch spacing as “Ordinary Boundary Elements”, or OBES. It is noted that the eight inch spacing limit is analogous to the spacing typically used in Chile (20 cm). Therefore, the first series of component tests (Group I) were conducted to investigate the role of large
spacing of boundary transverse reinforcement initiating section and member failure in thin wall boundaries detailed to satisfy ACI 318-11 provisions. The second series (Group II) was constructed after preliminary results were reviewed from Group I, with the same prism geometry as the first four specimens (but with different \( s/d_b \) ratios in some cases, and different loading histories). Rectangular prisms (columns) were constructed to represent OBEs at a non-flanged end of a flanged structural wall, as shown in Figure 5.1. Under such conditions, the prisms are subjected to large, nearly uniform, tensile and compressive strains; the laboratory specimens idealize the strain profile as uniform uniaxial tension and compression. In order to remain consistent with U.S. design practice and also represent the typical, damaged walls documented in Chile, four specimens were designed and constructed to satisfy ACI 318-11 Section 21.9.6.5 provisions with relatively large spacing of boundary transverse reinforcement. Test variables included spacing of transverse reinforcement and the presence of crossties supporting longitudinal bars not at corners of the boundary element prism specimens.

All specimens had one inch of clear concrete cover on all sides of the reinforcement, slightly more than the ACI 318-11 Section 7.7 minimum cover of 0.75 inches for walls not exposed to weather or in contact with ground. Longitudinal reinforcement in each prism consisted of six \#6 headed bars anchored into post-tensioned capital (top) and pedestal (bottom) support blocks, and transverse hoops and crossties were constructed of \#3 bars. Reinforcement was configured to satisfy the requirements of ACI 318-11 section 21.9.6.5 (special walls, but without transverse reinforcement required by section 21.9.6.4; referred to here...
as OBEs), but also reflect typical wall thickness and spacing of boundary transverse observed in Chile (20 cm). Three variables were considered in the test program: the vertical spacing of horizontal hoops, the presence of crossties supporting longitudinal bars along the long face of the cross section (Figure 5.2), and cyclic strain history (loading protocol). Table 5.1 outlines differences between the eight test specimens. Specimen labels identify the transverse hoop spacing in the first digit (8”, 6”, 4.5”), the presence of crossties (NT—no ties, WT—with ties), and the compressive strain limit imposed on the loading cycles (e.g., -2 for 0.002, -3 for 0.003). The last test, with suffix –I, had an imposed strain history with increasing compressive strain limits as the test moved into later cycles. Loading protocols are described in greater detail later in Chapter 7.

<table>
<thead>
<tr>
<th>ID</th>
<th>$b_w$ (in.)</th>
<th>$l$ (in.)</th>
<th>$h$ (in.)</th>
<th>$f'_c$ (ksi)</th>
<th>Cover (in.)</th>
<th>Long. bars</th>
<th>Trans. bars</th>
<th>s/d_b</th>
<th>Code provisions</th>
<th>$h_s$ (in.)</th>
<th>$\varepsilon_{c,max}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>8NT-2</td>
<td>6</td>
<td>15</td>
<td>30</td>
<td>4</td>
<td>1</td>
<td>(6) #6 ($\rho=0.029$)</td>
<td>#3 @ 8&quot; hoop only</td>
<td>10.7</td>
<td>21.9.6.5(a)</td>
<td>6.9</td>
<td>0.002</td>
</tr>
<tr>
<td>8WT-2</td>
<td>6</td>
<td>15</td>
<td>30</td>
<td>4</td>
<td>1</td>
<td>(6) #6 ($\rho=0.029$)</td>
<td>#3 @ 8&quot; hoop &amp; tie</td>
<td>10.7</td>
<td>21.9.6.5(a)</td>
<td>12.6</td>
<td>0.002</td>
</tr>
<tr>
<td>6NT-2</td>
<td>6</td>
<td>15</td>
<td>30</td>
<td>4</td>
<td>1</td>
<td>(6) #6 ($\rho=0.029$)</td>
<td>#3 @ 6&quot; hoop only</td>
<td>8</td>
<td>21.9.6.5(a)</td>
<td>12.6</td>
<td>0.002</td>
</tr>
<tr>
<td>6WT-2</td>
<td>6</td>
<td>15</td>
<td>30</td>
<td>4</td>
<td>1</td>
<td>(6) #6 ($\rho=0.029$)</td>
<td>#3 @ 6&quot; hoop &amp; tie</td>
<td>8</td>
<td>21.9.6.5(a)</td>
<td>6.9</td>
<td>0.002</td>
</tr>
<tr>
<td>8WT-3</td>
<td>6</td>
<td>15</td>
<td>30</td>
<td>4</td>
<td>1</td>
<td>(6) #6 ($\rho=0.029$)</td>
<td>#3 @ 8&quot; hoop &amp; tie</td>
<td>10.7</td>
<td>21.9.6.5(a)</td>
<td>6.9</td>
<td>0.003</td>
</tr>
<tr>
<td>6WT-3</td>
<td>6</td>
<td>15</td>
<td>30</td>
<td>4</td>
<td>1</td>
<td>(6) #6 ($\rho=0.029$)</td>
<td>#3 @ 6&quot; hoop &amp; tie</td>
<td>8</td>
<td>21.9.6.5(a)</td>
<td>6.9</td>
<td>0.003</td>
</tr>
<tr>
<td>45WT-45</td>
<td>6</td>
<td>15</td>
<td>30</td>
<td>4</td>
<td>1</td>
<td>(6) #6 ($\rho=0.029$)</td>
<td>#3 @ 4.5&quot; hoop &amp; tie</td>
<td>6</td>
<td>21.9.6.5(a)</td>
<td>6.9</td>
<td>0.0045</td>
</tr>
<tr>
<td>45WT-1</td>
<td>6</td>
<td>15</td>
<td>30</td>
<td>4</td>
<td>1</td>
<td>(6) #6 ($\rho=0.029$)</td>
<td>#3 @ 4.5&quot; hoop &amp; tie</td>
<td>6</td>
<td>21.9.6.5(a)</td>
<td>6.9</td>
<td>0.006*</td>
</tr>
</tbody>
</table>

*Specimen failed prior to 0.006 compression strain.
5.2 Specimen Design & Fabrication

Four specimens were constructed in Group I between September and October 2012. Each specimen had a thin, rectangular cross section (6” x 15”) representative of an OBE satisfying the provisions of ACI 318-11 Section 21.9.6.5. The four specimens in Group 2 were constructed in April and May 2013.

The test region of the prisms was located at the center of the 30 in. height, where hoop and crosstie (if provided) were spaced as indicated in Table 5.1. Above and below this region, and into the connection blocks, a tighter spacing of transverse reinforcement was used to force damage into the region with the larger (prescribed) spacing of transverse reinforcement (Figure 5.3).

Detailing #4 hoops and crossties was not practical given the 6 inch specimen thickness ($t_w = $wall thickness), the 1 inch clear cover, and the $6d_b$ extensions on hoops and crossties required by ACI 318-11 Section 21.6.4.2. Minimum radii on bends and returns of hoops would have required the longitudinal bars at the corners of the specimens to be located further into the central core of the specimen. Also, the radius of the hoop bends prohibited having both bar-end terminations to wrap around the same bar: if both returns were located at the same corner bar, the hook extension and the radius of the hoop bend would not...
have allowed room for two #6 diameter longitudinal bars to fit adjacent to each other at the specimen narrow edge. As such, both terminating legs of the transverse hoop wrapped both edge bars before the 135-degree bend engaged the concrete core (Figure 5.4). Hook extensions, particularly for the crossties, typically exceeded the minimum length specified by ACI 318 due to issues with constructability. The configuration of the machine that bent rebar shapes prohibited fabricating the hook on the 90 degree bend of the crossties without some extra material; ultimately the extra hook development length probably had little influence on the outcome of the tests.

Figure 5.3 Test region of specimens with 6" and 8" transverse spacing.

Figure 5.4 Specimen hoop and crosstie.
Longitudinal reinforcement consisted of six #6 headed bars made of steel material conforming to ASTM A706. In the first group of specimens, the headed longitudinal bars had a total length of 66 inches. This allowed the longitudinal bars to run continuously through the test prism and embed 16 inches into both the capital and pedestal connection blocks. Compared to the provisions for minimum development length in ACI 318 Section 12.6, 16 inches was more than the required minimum length assuming reinforcement with yield strength of 60 ksi. Yield strength distribution for #6 reinforcing bars conforming to ASTM A706 show that yield stress could be as high as 100 ksi (Bournonville et al. 2004); calculations of the required development length assuming 100 ksi as a yield strength suggested an embedment depth of almost 19 inches, more than the available 16 inches. However, the inclusion of hoops confining the bars inside the connection blocks, post-tensioning bars providing additional confinement to the connection blocks, and the low probability that the bars would actually achieve 100 ksi led to the decision that 16 inches was sufficiently long for bar development. This was less of a concern in the second group of specimens, since the total length of the longitudinal bars in the second group of specimens was 75 inches, which allowed for development lengths of 22.5 inches in both connection blocks. Even though this exceeded the 19 inch embedment length, additional transverse reinforcement and post-tensioning was provided as an additional safeguard against bar pullout.

Figure 5.5 Headed #6 longitudinal bars and transverse reinforcement.
A strut-and-tie model was developed to determine the necessary reinforcement for the top and bottom connection blocks. Each model assumed that the concrete strength would not exceed 4ksi, while the maximum force that could be developed by the longitudinal bars was more than 110ksi (after strain hardening), i.e., a load factor of about 1.1 on ultimate strength was used. As an extra precaution, the blocks were post-tensioned with 1.25” diameter DYWIDAG rods to create extra strength and prevent early onset of cracking within the connection blocks.

Connection blocks were not the same above and below the specimen due to connection constraints. The pedestal block had a 36” x 36” footprint to accommodate anchorage into a strong floor with anchor holes spaced 24” apart. The capital block measured 36” long by 24” wide in order to connect to a bolt hole pattern in the loading frame spaced 24” by 14”. Both blocks were 24” tall. The connection blocks were designed for tension via a three-dimensional strut-and-tie model as mentioned above. The blocks engaged the strong floor and loading beam via four tensioned vertical DYWIDAG rods running through the connection blocks, bearing on thick steel plates in direct contact with the blocks. The tension force demand from the longitudinal bars was idealized as a point load acting within the blocks at the headed ends, resisted by four struts linking to nodes at the four DYWIDAG rods connecting the blocks to the strong floor and loading frame (Figure 5.6). Results from the strut and tie model showed that neither the struts nor compression nodes were overloaded, and sufficient reinforcing steel was included in the design to provide the necessary tie forces for equilibrium. In addition, post-tensioned horizontal DYWIDAG rods ran through the connection blocks at roughly the same elevation as the tie reinforcement as an additional source of tie force and crack prevention. The target peak compressive strength of the concrete was 4 ksi; however, considerations for compression demands on the connection blocks assumed the gross area of the test prisms could resist up to 6 ksi based on research on concrete overstrength distribution (Nowak et al., 2008). Since the top surface of the capital block and bottom surface of the pedestal block
in a compression scenario would be in full contact with the loading frame and reaction blocks, both the top and bottom blocks were designed as mat foundations.

![Diagram](image)

**Figure 5.6 Simplified load paths for connection blocks.** (Left) Strut-and-tie model for specimen in tension; (right) assumed mat behavior for specimen in compression.

Cages for the top block, bottom block, and test prism were constructed separately. Once the individual component cages were completed, the three cages for each specimen were assembled together and placed into formwork (Figure 5.7). The cage assembly rested on the sides of the long dimension of the connection blocks in the formwork (Figure 5.8a). The specimen was positioned this way to allow the specimen and blocks to be poured in one continuous session, instead of having different stages of construction for the blocks and specimen. The centerlines of the blocks and specimen were aligned such that the specimen formwork was elevated above the “base” formwork for the connection blocks. The desired clear cover for the test prisms and the connection blocks were achieved by placing wired dobies and plastic seats between the cages and the formwork. Large diameter holes were drilled into the formwork and PVC tubes ran through the steel cages to provide a clear path for post-tensioning rods. 3/8” diameter threaded rods were positioned along the length of the test prism to provide mounting nodes for the LVDT sensors. The rods were positioned one inch above or below the transverse hoops (Figure 6.4), and to the inside of the outermost longitudinal bars such that if concrete cover spalled, the position of the rods would not be compromised.
The volume of each specimen only required approximately 1.2 cubic yards of concrete. All four specimens in each group were poured from the same truck delivery. A high slump achieved by plasticizer was specified for the concrete mix delivery since the concrete had to be pumped into the lab through almost 200’ of hose. Both concrete deliveries had measured slumps between 7 and 8 inches. Concrete was first poured into the connection blocks until the top surface reached the elevation of the specimen top edge (Figure 5.8b). Concrete was then smoothed and a plywood cap was secured over the test prism (Figure 5.8c). The pour then continued in each connection block form until filled (Figure 5.8d). Plastic sheeting was draped over the curing concrete, and specimens set in the forms for approximately one week before being stripped (Figure 5.9).
Figure 5.8 Concrete pour progression. (a) Cages in formwork; (b) concrete poured up to top edge of test prism; (c) prism capped; (d) completed pour.

Figure 5.9 Upright test prism and connection blocks after formwork removed.
5.3 Construction Materials

The desired target concrete strength after 28 days was 4000 psi due to concerns regarding actuator loading capacity. A mix with 3/8” coarse aggregate, design strength of 2500 psi, and vendor-tested 28-day strength of 4100 psi was selected for construction for both batches. As mentioned previously, concrete had to be pumped approximately 200 feet from the concrete truck to the specimen formwork; a high slump (8” ± 1”) was specified to ensure the mix would not set up inside the pump hose. 6” diameter by 12” tall concrete cylinders were cast throughout the pours for both test batches to obtain material properties. Concrete cylinder tests were typically performed within a day or two of, and no more than a week later than, the specimen tests to characterize the concrete stress-strain relationship. Figure 5.10 and Figure 5.11 capture the peak compressive stresses and their associated strain levels, grouped by the batches from which they were cast. Table 6.1 summarizes the peak values and gives averages for each batch. Plots of the entire stress-strain curve for each cylinder test are available in the appendix. Averaging peak strengths of all cylinder tests coinciding with specimen tests in Group I resulted in a mean $f'_c$ of 4010 psi, very close to the target 4000 psi. Averaging cylinder compression test results for Group II specimens (a different truck delivery but same mix specifications) yielded a $f'_c$ value of 3810 psi, slightly less than strengths from the first delivery (95% of $f'_c$ from Group I).
Two types of reinforcing steel were used in the construction of the specimens. Capital and pedestal (connection) block and specimen transverse reinforcement (hoops and ties) were constructed from material conforming to Grade 60 ASTM A615. Reinforcement in the connection blocks were fabricated from #4 and #5 bars, and specimen transverse reinforcement was constructed from #3 bars. Specimen longitudinal reinforcement conformed to Grade 60 ASTM A706 material since the bars were expected to elongate considerably past yield strain. All Group I specimen longitudinal bars were #6 bars and came from the same heat; longitudinal bars in the second batch came from a different source but were from the
same heat within that different source. Mill certifications for each batch of longitudinal material confirmed that yield stress, ultimate stress, and elongation conformed to ASTM A706. Longitudinal bars used in Group I specimens had a mill-tested yield strength of 67.5 ksi, ultimate strength of 95.3 ksi, and an elongation of 18%. Longitudinal bars in Group II specimens were spare bars repurposed from another laboratory project, and had mill-tested yield strength of 69.1 ksi, ultimate strength of 93.2 ksi, and elongation of 14%. Mill certifications are provided in Appendix B.

With specimen design and fabrication completed, and construction material properties obtained, focus shifted to additional setup considerations external to the test specimens. Chapter 6 documents preparations for the loading system setup, loading protocols, and data acquisition for the experiments.
CHAPTER 6  EXPERIMENTAL PROGRAM

This chapter describes in detail the setup for the loading system used to impart loads and displacements on the specimens, the data acquisition sensor configuration, and the applied loading protocols for the eight prism tests.

6.1  Loading System

The test prisms were positioned upright such that forces transferred into the capital connection block loaded the test prism axially. Applied axial forces were delivered by two 400 kip-capacity hydraulic actuators oriented vertically. Each actuator was positioned such that its centerline of force acted six feet on either side of the specimen centerline and the actuators positioned between the loading frame and the strong floor (Figure 6.1). The test specimens, including both top and bottom blocks, measured 6’-6” tall, which was smaller than the actuators’ minimum stroke of approximately 11 feet; therefore, the test specimens were placed on two reaction blocks measuring a combined 5’ tall so that the height at the top of the specimen permitted sufficient elongation and contraction in the loading actuators. The specimens and the actuators were connected via a loading frame, a W36x256 deep beam reinforced by a 2.5” thick by 20” wide plate welded to the bottom beam flange (Figure 6.1). The loading frame would not undergo plastic deformations under the expected loads delivered by the actuators. Post-tensioned DWYIDAG rods clamped the capital connection block and actuator connector plate to the loading frame. Similarly at the base of the specimen, post-tensioned DWYIDAG rods anchored into the strong-floor, extended through the reaction blocks and pedestal connection block, and kept the base block in continuous contact with the reaction blocks. Hydrostone was poured between the reaction blocks and the pedestal connection block to maximize the contact area between the surfaces, and also used at various locations between the loading frame and capital connection block when the block top surface was uneven (Figure 6.2).
Because the actuators, loading frame, and test specimen were all aligned in a single plane, restraints were provided to prevent the loading frame from moving out-of-plane. Bracing of the loading frame horizontal member was provided by two rectangular tube steel struts attached to concrete beams otherwise isolated from the test setup. The strut-frame bracing connections were located to roughly coincide with the lines of action of the actuators, and the connections created pinned restraints for the brace ends (Figure 6.3a and 6.3b). Pinned connections allowed the struts to pivot as the frame translated vertically during the
experiments. An additional brace was provided at the base of the vertical component (see Figure 6.1) of the loading frame. Two steel channels reinforced by diagonal square tube steel restrained the flange of the vertical member of the loading frame (Figure 6.3c and 6.3d). The clevises at the ends of the actuators also had resistance against out of plane movement, but they were not relied upon due to concerns that out of plane movement might damage the actuator connections.

In-plane horizontal movement was not strictly restrained, but mechanisms were installed to limit translations. The steel channels used as out of plane restraint doubled as guides limiting in plane horizontal translation. Rollers were installed on both channels on either side of the outer flange of the loading frame vertical member (Figure 6.3d). A steel wide flange column with a tube steel kicker support was located at the other end of the loading frame, restricting excessive in-plane horizontal translation at the top of the loading frame (Figure 6.3a).

Figure 6.3 Loading frame restraints. (a) Column restricting in-plane translation & strut restricting out-of-plane translation; (b) out-of-plane strut connecting to loading frame; (c) channel and roller assembly restricting in-plane and out-of-plane translation; (d) top-down view of rollers on either side of steel flange.
6.2 Sensor Setup & Data Acquisition

Full-height specimen strains were determined by two linear variable differential transformers (LVDTs) supported between the pedestal and capital connection blocks on either side of the test prisms. The two LVDTs were positioned along the major axis of the specimen cross section, typically 4-7 inches from the specimen edge (depending on the connection blocks’ concrete surface conditions), and served as the control sensors during the experiments (Figure 6.4). The average displacement registered by the two full-height LVDTs corresponded to the average axial displacement applied along the centerline of the specimen cross section parallel to the 15 inch dimension of the cross section; the average axial strain value was used to determine when the imposed strain limits had been reached for each loading cycle. A third full-height, capital-to-pedestal LVDT was positioned south of the test specimen on the cross section minor axis. Combined with readings from the two control sensors, the data from the three block-to-block LVDTs capture planar movement of the section over the specimen height (between end blocks). It is assumed that the pedestal block is essentially rigid and the three full-height LVDT points are used to calculate the planar movement relative to the pedestal connection block. Calculating planar movement in turn allows determining axial strain at any point within the test prism.

In addition to the three block-to-block sensors, twelve LVDTs were mounted directly to each specimen with 8 or 6 inch hoop spacing: six on each long (15-inch) face, aligned into two vertical columns of three sensors each. Five LVDTs were fit into each sensor column in specimens with 4.5” hoop spacing. The columns were positioned four inches to each side of specimen centerline. In each column of sensors, one LVDT spanned the approximate height of one hoop spacing interval, mounted to threaded rods located just above or below the horizontal reinforcement (Figure 6.4). The objective of providing these sensors was to monitor local axial strains across intervals between transverse reinforcement, since block-to-block sensors include displacements associated with bar slip at the prism-block interfaces. Axial force versus
axial strain plots presented in Chapter 8 and the appendix compare the block-to-block axial strain at the locations of longitudinal reinforcement to assess the variation of axial strain over the cross section. Discussions on variations of documented axial strains between readings from block-to-block sensors and mounted sensors are presented in each test summary in Chapter 8.

Figure 6.4 Positions of LVDT mounting rods on boundary element specimens: (top left) elevation view rod positioning schematic, (top right) mounted LVDT assembly, (bottom) cross-section view schematic.

Two additional LVDTs were positioned horizontally, perpendicular to the long surface of the specimen to detect out-of-plane bulging of the concrete due to bar buckling (Figure 6.5). These sensors did not capture substantial results in the first experiment and were not used for the remaining tests. Longitudinal bars in the specimens were not instrumented with strain gauges, because (1) budget constraints precluded the time and labor necessary for installation, (2) strain gauges only report local strains and therefore a large number of gauges would have been required to obtain meaningful data, and (3) use of a large number of gauges might impact the phenomena (rebar buckling) being studied.
Since the loading frame was not entirely restricted from horizontal translation in-plane, a horizontally oriented LVDT was mounted directly to the end of the steel loading frame to capture translation at the top of the loading frame parallel to the 15 inch face of the test specimen (Figure 6.6). One end of the LVDT attached to a stiffener plate on the loading frame, and the opposite end attached to a rigid reference frame supported on the strong floor. Plots recording the horizontal movement of the loading frame (and capital connection blocks) are included in Chapter 7.
Each load actuator had a self-contained load-cell to measure applied axial load and an axially-oriented LVDT that tracked shaft elongation and contraction. Load cells were capable of registering up to 400 kips of axial force, and the LVDTs had sufficiently large stroke to accommodate the anticipated actuator movement needed to result in bar buckling. The loading system was set to displacement control, with the actuator-mounted LVDTs providing the system with the means to dictate and limit actuator displacements. The east actuator was “slaved” to the west to ensure a symmetric load application. The longitudinal axis of the actuators were symmetrically located six feet on either side of the specimen cross section centerline (Figure 6.1), and the loads imparted by the actuators created large moment demands (and thus rotations) on the loading frame. The loading frame had sufficient elastic strength to avoid any permanent deformations under the applied loads, but the flexural deformations translated into actuator displacements that were significantly different from axial displacements measured along the vertical axis of the test specimens. Since the displacements registered at the actuators did not reflect the displacements as the specimen, axial displacements of the actuators were carefully monitored to achieve the desired strain history for the test specimen. That is, specimen block-to-block displacement was continuously monitored by the loading system operator, and actuator elongation or contraction limits were meticulously adjusted in small increments in real-time until the prescribed specimen block-to-block strain limits were achieved.

6.3 Loading Protocols

Steps were taken prior to applying loads to the specimens to zero out the self-weight of the loading frame. The actuators were attached to the loading frame without connecting to the test specimens, and readings from the load cells were recorded to determine the weight distribution of the frame. Once the specimens had been installed, before beginning the loading protocols the recorded reactions by the actuators without the specimen in place were restored to remove any preload on the specimen. After the impact of the
weight of the loading frame had been cancelled out, the loading protocols for each specimen were applied.

A reverse cyclic loading history was applied to each test specimen (Figure 6.7, Table 6.1). Test cycles started with the specimen being loaded in axial tension, followed by axial compression. Each full cycle was performed at least twice before moving on to the next axial strain level. Peak axial strains for the first loading level were 0.05% in both tension and compression. Peak axial strains were then increased by 0.05% increments (equal in both directions) until the peak strains reached 0.2% in both tension and compression. In order to avoid concrete cover from crushing and spalling prior to rebar buckling (since the test specimens were detailed as OBEs), the peak compressive strain was limited to 0.2% for all subsequent cycles in Group I experiments. While the compression strain limit was held constant at 0.2%, tension strains were increased in prescribed increments (Table 6.1), with the increase between peak values for different strain levels limited to a multiplier of 1.5.

<table>
<thead>
<tr>
<th>Cycle #</th>
<th>Compression Strain Limit (%)</th>
<th>Tension Strain Limit (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1, 2</td>
<td>-0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>3, 4</td>
<td>-0.10</td>
<td>0.10</td>
</tr>
<tr>
<td>5, 6</td>
<td>-0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>7, 8</td>
<td>-0.20</td>
<td>0.20</td>
</tr>
<tr>
<td>9, 10</td>
<td>-0.20</td>
<td>0.30</td>
</tr>
<tr>
<td>11, 12</td>
<td>-0.20</td>
<td>0.50</td>
</tr>
<tr>
<td>13, 14</td>
<td>-0.20</td>
<td>0.75</td>
</tr>
<tr>
<td>15, 16</td>
<td>-0.20</td>
<td>1.00</td>
</tr>
<tr>
<td>17, 18</td>
<td>-0.20</td>
<td>1.50</td>
</tr>
<tr>
<td>19, 20</td>
<td>-0.20</td>
<td>2.00</td>
</tr>
<tr>
<td>21, 22*</td>
<td>-0.20</td>
<td>3.00</td>
</tr>
<tr>
<td>23, 24</td>
<td>-0.20</td>
<td>4.50</td>
</tr>
</tbody>
</table>

*Specimen 8WT-2 was loaded 3 cycles at -0.2%/+2.0% strain limits.
Table 6.2 Strain history protocol for first two experiments in Group II.

<table>
<thead>
<tr>
<th>Cycle #</th>
<th>Compression Strain Limit (%)</th>
<th>Tension Strain Limit (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1, 2</td>
<td>-0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>3, 4</td>
<td>-0.10</td>
<td>0.10</td>
</tr>
<tr>
<td>5, 6</td>
<td>-0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>7, 8</td>
<td>-0.20</td>
<td>0.20</td>
</tr>
<tr>
<td>9, 10</td>
<td>-0.30</td>
<td>0.30</td>
</tr>
<tr>
<td>11, 12</td>
<td>-0.30</td>
<td>0.50</td>
</tr>
<tr>
<td>13, 14</td>
<td>-0.30</td>
<td>0.75</td>
</tr>
<tr>
<td>15, 16</td>
<td>-0.30</td>
<td>1.00</td>
</tr>
<tr>
<td>17, 18</td>
<td>-0.30</td>
<td>1.50</td>
</tr>
<tr>
<td>19, 20</td>
<td>-0.30</td>
<td>2.00</td>
</tr>
<tr>
<td>21, 22</td>
<td>-0.30</td>
<td>3.00</td>
</tr>
<tr>
<td>23, 24</td>
<td>-0.30</td>
<td>4.50</td>
</tr>
</tbody>
</table>

Three of the experiments in Group II underwent similar loading histories, except the compressive strain limit for specimens 8WT-3 and 6WT-3 was increased to 0.3% (Table 6.2). Therefore, specimens 8WT-3 and 6WT-3 underwent one additional series of cycles of equal tension-compression strain beyond 0.2% before being subjected to asymmetric loading cycles. Similarly, specimen 45WT-45 was subjected to equal tension-compression strain cycles up to 0.45% (Table 6.3) prior to application of asymmetric
loading. After strain cycles to 0.45%, tension strain limits increased in successive cycles, while the compression strain limit was held to 0.45% for all subsequent cycles.

Table 6.3 Strain history protocol for the third experiment in Group II.

<table>
<thead>
<tr>
<th>Cycle #</th>
<th>Compression Strain Limit (%)</th>
<th>Tension Strain Limit (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1, 2</td>
<td>-0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>3, 4</td>
<td>-0.10</td>
<td>0.10</td>
</tr>
<tr>
<td>5, 6</td>
<td>-0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>7, 8</td>
<td>-0.20</td>
<td>0.20</td>
</tr>
<tr>
<td>9, 10</td>
<td>-0.30</td>
<td>0.30</td>
</tr>
<tr>
<td>11, 12</td>
<td>-0.45</td>
<td>0.45</td>
</tr>
<tr>
<td>13, 14</td>
<td>-0.45</td>
<td>0.75</td>
</tr>
<tr>
<td>15, 16</td>
<td>-0.45</td>
<td>1.00</td>
</tr>
<tr>
<td>17, 18</td>
<td>-0.45</td>
<td>1.50</td>
</tr>
<tr>
<td>19, 20</td>
<td>-0.45</td>
<td>2.00</td>
</tr>
<tr>
<td>21, 22</td>
<td>-0.45</td>
<td>3.00</td>
</tr>
<tr>
<td>23, 24</td>
<td>-0.45</td>
<td>4.50</td>
</tr>
</tbody>
</table>

The final experiment from the Group II tests was subjected to a different load history pattern than the prior seven test prisms. The initial loading cycles up to 0.2% tension and compression strain were the same as in previous experiments; however, in subsequent cycles, both tension and compression strain limits increased asymmetrically to explore the influence of increasing compression strain on OBE behavior (the loading history mimics what would happen to a cantilever wall with T-shaped wall cross section where equal lateral drifts are applied at the tip of the cantilever in positive and negative directions). Following the cycles to 0.2% tension and compression strain, the maximum compression strain limit was maintained at 0.2% as tension strain limits incrementally increased to 1.0%. After the cycle of +1.0%/-0.2% axial strain, the compression strain limit increased to 0.3%, while the tension strain limit was held to 1.0%. After two cycles at these strain limits were completed, the tension strain limit
was increased to 1.5% and compression strain was limited to 0.3%. This pattern of cycle limit combinations alternating between increasing the tension strain or compression strain limit by 50% while holding the strain limit in the opposite direction constant continued until the compression strain limit was increased to 0.6% (Table 6.4).

<table>
<thead>
<tr>
<th>Cycle #</th>
<th>Compression Strain Limit (%)</th>
<th>Tension Strain Limit (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1, 2</td>
<td>-0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>3, 4</td>
<td>-0.10</td>
<td>0.10</td>
</tr>
<tr>
<td>5, 6</td>
<td>-0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>7, 8</td>
<td>-0.20</td>
<td>0.20</td>
</tr>
<tr>
<td>9, 10</td>
<td>-0.20</td>
<td>0.30</td>
</tr>
<tr>
<td>11, 12</td>
<td>-0.20</td>
<td>0.50</td>
</tr>
<tr>
<td>13, 14</td>
<td>-0.20</td>
<td>0.75</td>
</tr>
<tr>
<td>15, 16</td>
<td>-0.20</td>
<td>1.00</td>
</tr>
<tr>
<td>17, 18</td>
<td>-0.30</td>
<td>1.00</td>
</tr>
<tr>
<td>19, 20</td>
<td>-0.30</td>
<td>1.50</td>
</tr>
<tr>
<td>21, 22</td>
<td>-0.45</td>
<td>1.50</td>
</tr>
<tr>
<td>23, 24</td>
<td>-0.45</td>
<td>2.00</td>
</tr>
<tr>
<td>25*</td>
<td>-0.60</td>
<td>2.00</td>
</tr>
</tbody>
</table>

*Specimen failed prior to reaching compressive strain limit of -0.60%.
CHAPTER 7 EXPERIMENTAL RESULTS

This chapter describes detailed observations from each prism experiment performed in the laboratory at UCLA. Three damage states are established to describe observed damage due to buckling of longitudinal reinforcement. Axial force versus axial strain plots are used to describe the loading history over the cycles leading up to significant loss in axial capacity for all specimens. Where buckling of longitudinal reinforcement was observed to initiate damage, close-up photographs are presented to capture the progression of damage. LVDT data from all mounted sensors from all experimental cycles are provided in Appendix B.

7.1 Longitudinal Bar Buckling Damage States

To quantify strain histories and force levels associated with observed rebar buckling, a series of damage states were defined. Damage state 1 (DS1), showing the first visible signs of bar distress, was categorized as vertical cracks located near longitudinal bars. These cracks usually delineate a block or wedge of concrete cover that is being pushed by unstable longitudinal bars. Damage state 2 (DS2) is based on surface cracks expanding and cover wedges bulging out from the core of the test specimen, indicating that a longitudinal bar or bars have buckled to the point that the cover is no longer providing restraint nor is it in contact with the core concrete of the prism cross-section. Damage state 3 (DS3) is defined by a clear separation of cover, with buckled bars visible within cracks separating the cover from the specimen. DS3 is typically followed by cover loss exposing the buckled bars in subsequent cycles (cover spalling typically occurs during the subsequent tension cycle).

Timestamps from time-lapse photos, videos, crack catalog photos, and data acquisition files were used to coordinate visual damage assessments with force and displacement (strain) data. Once force-
displacement records were synchronized with the different visual media, data points corresponding to the different damage states were identified and indicated on force-strain plots. Corresponding photos of the observed damage are included with the force-strain plots for tests that exhibited buckling of longitudinal reinforcement as the damage initiator.

7.2 Specimen 8WT-2 (Test #1)

The first specimen tested was 8WT-2; a specimen with 8” spacing of transverse reinforcement, with cross ties, and a maximum compressive strain limit of 0.002. As the experiment proceeded, the loading protocol was periodically paused after each half cycle to track developing concrete cracks and to ensure that the data collected were consistent with expectations. Time-lapse photography captured deformations of the specimen over the duration of the experiment, and high resolution video was taken as the specimen approached failure. The progression of damage is tracked in Figure 7.1. Figure 7.1a provides a visual reference of the test prism before damage associated with bar buckling manifests.

Horizontal tension cracks were first observed during the first 0.05% tension strain cycle at heights where transverse reinforcement was located. These tensile cracks circumscribed the specimen at the peak of the first 0.1% tension strain cycle. Vertical cracks under compressive loading first developed in the first half-cycle following the first cycle reaching 3% peak tensile strain. Vertical cracks spanning roughly three hoop spacing intervals appeared on the 15 inch surfaces of the specimen approximately 1.5 to 2 inches from the edge, providing the first indication of bar buckling (Figure 7.1b). Despite indications that the longitudinal bars were developing instability, the specimen completed the full compression half-cycle with no observed strength loss.
Upon reloading the specimen into tension, the concrete cover bounded by horizontal tension cracks near the transverse hoops and the vertical cracks started to separate from the rest of the specimen. Once the specimen underwent compression reloading for the second time from the same peak tension strain level (3%), longitudinal bars buckled between hoop supports and completely separated the edge blocks of concrete cover from the remaining specimen (Figure 7.1d). The separated cover was removed since it began interfering with one of the control sensors, and it was observed that both longitudinal edge bars buckled between hoop intervals immediately above and below the center spacing (Figure 7.1e). A reduction in the peak applied compression force at the compression strain limit by more than 10% relative to the previous cycle (356.7 kips versus 402.8 kips) is consistent with the amount of cross sectional area lost due to cover push-off by the buckled longitudinal bars. The peak axial compression force applied to specimen 8WT-2 corresponds to 112% of \( A_{gf} ^{c} \) using the average of all cylinder tests from Group I. Table 7.1 summarizes the peak forces associated with the strain limits of each cycle for specimen 8WT-2.

Table 7.1 Peak applied load summary for 8WT-2.

<table>
<thead>
<tr>
<th>Cycle #</th>
<th>Loading Protocol</th>
<th>Initial Cycle</th>
<th>Reloading Cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension Strain Limit (%)</td>
<td>Compression Strain Limit (%)</td>
<td>Tension Force (kip)</td>
</tr>
<tr>
<td>1, 2</td>
<td>0.05</td>
<td>-0.05</td>
<td>42.59</td>
</tr>
<tr>
<td>3, 4</td>
<td>0.10</td>
<td>-0.10</td>
<td>61.09</td>
</tr>
<tr>
<td>5, 6</td>
<td>0.15</td>
<td>-0.15</td>
<td>84.98</td>
</tr>
<tr>
<td>7, 8</td>
<td>0.20</td>
<td>-0.20</td>
<td>106.83</td>
</tr>
<tr>
<td>9, 10</td>
<td>0.30</td>
<td>-0.20</td>
<td>146.07</td>
</tr>
<tr>
<td>11, 12</td>
<td>0.50</td>
<td>-0.20</td>
<td>172.48</td>
</tr>
<tr>
<td>13, 14</td>
<td>0.75</td>
<td>-0.20</td>
<td>176.69</td>
</tr>
<tr>
<td>15, 16</td>
<td>1.00</td>
<td>-0.20</td>
<td>180.74</td>
</tr>
<tr>
<td>17, 18</td>
<td>1.50</td>
<td>-0.20</td>
<td>183.75</td>
</tr>
<tr>
<td>19, 20</td>
<td>2.00</td>
<td>-0.20</td>
<td>193.97</td>
</tr>
<tr>
<td>21, 22(^1)</td>
<td>3.00</td>
<td>-0.20</td>
<td>208.42</td>
</tr>
<tr>
<td>23, 24</td>
<td>4.50</td>
<td>-0.20</td>
<td>220.69</td>
</tr>
</tbody>
</table>

\(^1\) Loss of cover due to longitudinal bar buckling occurred in compression reloading
\(^2\) Maximum compression force before degradation due to global buckling
Observations from photos and LVDT data indicated that there was some slippage of the longitudinal bars at the block-specimen interface. Overall specimen axial displacements (between top and bottom connection blocks) were monitored by the two in-plane full-height LVDTs; however, the reported displacements include slip and extension (or yield penetration) of reinforcement at the prism-block interfaces. Readings obtained in the LVDTs mounted within the test region (with larger spacing of transverse reinforcement, see Figure 5.3) were used to provide the best estimate of axial strains within the test region with larger hoop spacing; average strains within the test region were typically less than the control strain for the overall specimen height (in tension) due to the crack widths observed at the prism-block interface. Because of the non-uniform distribution of tension cracks and tension crack widths over the specimen height, deformations recorded by LVDTs in the same column line within the test region were not uniform. Therefore, LVDT data within the test region nearest the distressed longitudinal bars were averaged to obtain a representative strain between threaded rod nodes above and below the hoops bounding the test region (Figure 5.3). A comparison between the strain obtained with the block-to-block
sensors and average strain within the test region nearest the buckled bars is shown in Figure 7.2 and indicates that, for the 3% control tension strain cycle, the average axial tension strain within the test region is only about 2.20%.

The buckling strain indicator suggested by Rodriguez et al. (1999), $\varepsilon^*_p$, also is shown in Figure 7.2. For this study, $\varepsilon^*_p$ was taken as the largest strain range prior to specimen instability, caused by either local buckling of longitudinal reinforcement or some other failure mode (discussed subsequently). For tests where local rebar buckling was not observed prior to failure, the $\varepsilon^*_p$ determined from test results provides a lower bound estimate of the tensile strain required to cause rebar buckling.

![Figure 7.2 Force-strain plot for specimen 8WT-2 of 3% and 4.5% tensile strain cycles with damage state indicators.](image)

An interesting but subtle phenomenon captured in the force-strain plots for 8WT-2 was a slight indication of out-of-plane movement following the second cycle to a peak value of 3% tension strain, upon reloading in compression (immediately following Damage State 3). The slight out-of-plane deformations occurred only for a short range of loading due to the presence of open cracks; once tension cracks started to close, the specimen was able to reestablish stability and the out-of-plane deformations were eliminated. Visual observations confirmed these findings. In the subsequent cycle to larger tensile strain, the
specimen was unable to reestablish stability. Out-of-plane failures were observed in two additional specimens in the first group of experiments, as noted in the following paragraphs for discussion related to specimens 6WT-2 and 6NT-2.

Tension strain was increased to 4.5% in the subsequent half-cycle for specimen 8WT-2, leading to straightening of the previously buckled longitudinal reinforcement. The maximum tension force applied to the specimen occurred during the excursion to 4.5% strain, 220.7 kips (83.6 ksi average stress on the six longitudinal bars). As loading transitioned from tension to compression, the previously-buckled bars again bent out away from the concrete core, and then out-of-plane deformations were observed. Unlike the 3% tension cycle, the out-of-plane deformations were too large to correct, and the specimen could not reestablish stability (Figure 7.3); this failure mode is referred to here as “global” buckling (or global instability), although it is noted it was preceded by rebar buckling. The compression force in the reloading region from the 4.5% tension strain cycle reached only 99.6 kips, 25% of the peak compressive load applied in prior cycles before the global instability failure occurred. The instability failure is captured in Figure 7.2 where the strain readings from the mounted LVDTs (solid blue line) begin to indicate test region extension while the control sensors (red dashed line) show overall compression of the specimen. The average strain reading from LVDTs was from the convex side of global buckled shape, and it captures the growth over the gauge length as the specimen bent out-of-plane. The prism response captured by the other onboard LVDT columns can be found in the appendix.
In-plane lateral drifts were captured at the loading frame. Initially it was assumed that the pedestal base would remain relatively immobile with respect to lateral translation during the experiment due to the grouted, post-tensioned connection to reaction blocks and the strong floor. For that reason, no sensor measured pedestal block rotation or lateral translations during the experiments. However, it became clear from time-lapse videos that this assumption was incorrect. Unquantifiable, but noticeable—especially at large tension strain demands—small lateral movements of the pedestal block were observed in video recordings. Data for base-block rotation and translation were not recorded, so the plots of capital block drift ratios reported herein are calculated as the in-plane translation of the loading beam divided by the height of the test prism. These represent upper-bound estimates of the actual lateral drift ratios experienced by the prisms, since rotations are unaccounted for and translations are assumed to occur over a smaller height than what was actually observed.

Lateral drifts tend to be larger where the specimen was loaded in tension. The cracked concrete section in tension essentially relies only on the longitudinal bars to supply lateral resistance, as opposed to the full concrete prism cross section when loaded in compression. Drift plots are configured such that the line
traces the path of the top of the specimen as viewed from the north of the specimen (lines leading to the left side of the plots represent movement to the left relative to an observer looking from the north).

As evident in the left plot of Figure 7.4, drifts associated with large tension strain demands were significantly larger than drifts where the specimen was compressed. The largest recorded drift (assuming the pedestal connection block was completely immobile) was approximately 1.7% at the tension strain limit of 4.5%. In general, frame drift ratios at peak compression strain limits do not exceed 0.2% throughout the experiment duration. Significant horizontal translation occurred when the applied loads transition from tension to compression and vice versa, shown in the right plot of Figure 7.4. This concentration of movements at the transition from tension to compression loading is likely due to loading system feedback as the loading frame transitioned from positive moments and curvature to negative moments and curvature and the system controlling the actuators tried to maintain equal forces in the actuator load cells. While the relatively large horizontal movement in the tension regime of the loading cycles is not believed to have negatively impacted the prism performance, the behavior was undesirable, and greater attention in the setup of subsequent tests was made to prevent excessive drifts. This included
repositioning steel shim plates between the vertical frame element and the rolling restraint assembly, and verifying how plumb the loading frame was relative to the specimen before and after installation.

7.3 Specimen 8NT-2 (Test #2)

The second specimen, 8NT-2, was tested 25 days after specimen 8WT-2; the difference between the specimens being the crosstie on the longitudinal bars along the long face of the prism. Concrete cylinder tests revealed no significant difference in material properties between the two specimens. Cracking patterns for 8NT-2 were very similar to 8WT-2, i.e., predominantly horizontal cracks at heights where transverse reinforcement was located. Hairline vertical cracks on the south/rear face (15 inch side) near the prism edge (Figure 7.5c, d) were observed at the end of the first compressive cycle following the tension cycle to a peak strain of 3%. On the subsequent compression cycle, the vertical cracks expanded, eventually to the point where spalling at the corner revealed a single buckled bar in the upper 8-inch hoop interval on the rear face of the specimen (Figure 7.5g, i). In the same reloading cycle, predominantly vertical cracks developed on the north/front face adjacent to the distressed longitudinal bar on the south side of the specimen (Figure 7.5a).
An adjustment to the loading protocol was made at this point: instead of increasing the tension to 4.5% strain, a third cycle at 3% tensile strain was applied to the specimen. Upon reloading into compression, the exposed longitudinal bar buckled at a much lower reloading strain level, and the adjacent longitudinal bar also exhibited signs of buckling in the same hoop interval. However, unlike specimen 8WT-2, which exhibited buckling of longitudinal reinforcement followed by “global” out-of-plane failure with gradual strength loss, abrupt strength loss occurred in specimen 8NT-2. Just prior to brittle failure vertical cracks expanded at the northwest longitudinal bar, but cover was not completely separated by the bar before sudden crushing failure.

Figure 7.5 Damage progression associated with longitudinal bar buckling for specimen 8NT-2. (a) & (b) No damage associated with bar buckling; (c) & (d) vertical cracking associated with DS1; (e) & (f) widened cracks associated with DS2; (g) cover push-off associated with DS3; (h) & (i) cover loss exposing buckled bars.
Table 7.2 Peak applied load summary for specimen 8NT-2.

<table>
<thead>
<tr>
<th>Cycle #</th>
<th>Loading Protocol</th>
<th>Initial Cycle</th>
<th>Reloading Cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension Strain Limit (%)</td>
<td>Compression Strain Limit (%)</td>
<td>Tension Force (kip)</td>
</tr>
<tr>
<td>1, 2</td>
<td>0.05</td>
<td>-0.05</td>
<td>45.64</td>
</tr>
<tr>
<td>3, 4</td>
<td>0.10</td>
<td>-0.10</td>
<td>69.48</td>
</tr>
<tr>
<td>5, 6</td>
<td>0.15</td>
<td>-0.15</td>
<td>91.38</td>
</tr>
<tr>
<td>7, 8</td>
<td>0.20</td>
<td>-0.20</td>
<td>111.97</td>
</tr>
<tr>
<td>9, 10</td>
<td>0.30</td>
<td>-0.20</td>
<td>144.93</td>
</tr>
<tr>
<td>11, 12</td>
<td>0.50</td>
<td>-0.20</td>
<td>172.51</td>
</tr>
<tr>
<td>13, 14</td>
<td>0.75</td>
<td>-0.20</td>
<td>180.92</td>
</tr>
<tr>
<td>15, 16</td>
<td>1.00</td>
<td>-0.20</td>
<td>183.91</td>
</tr>
<tr>
<td>17, 18</td>
<td>1.50</td>
<td>-0.20</td>
<td>184.73</td>
</tr>
<tr>
<td>19, 20</td>
<td>2.00</td>
<td>-0.20</td>
<td>193.87</td>
</tr>
<tr>
<td>21, 22</td>
<td>3.00</td>
<td>-0.20</td>
<td>208.11</td>
</tr>
<tr>
<td>23¹</td>
<td>3.00</td>
<td>-0.20</td>
<td>--</td>
</tr>
</tbody>
</table>

¹A second reloading cycle (3rd total) to 3% tension strain was performed instead of increasing to 4.5%
²Maximum compression force at sudden failure

Applied compression force peaked at 413.95 kips (1.15Agfc) at 0.002 compression strain following the first excursion to 0.02 tension strain (Table 7.2). The maximum applied compression force following the first cycle to 0.03 tension strain was slightly smaller, measuring 411.52 kips (1.14Agfc). Compression forces decreased on reloading cycles after the first excursion to 3.0% axial strain: the first reloading cycle achieved a peak compression force of 384.15 kips (1.06Agfc), and immediately prior to brittle failure forces only reached 344.37 kips (0.95Agfc). Peak applied tension force occurred in the tension loading cycles to 3.0% strain: the maximum tension force recorded was 210.5 kips (an average of 79.8 ksi tension stress in each longitudinal bar, or 1.18Afy). The compression strength loss observed in the reloading cycles is most likely due to the loss of gross cross section associated with longitudinal bars pushing off cover concrete. Concrete cover violently exploded off both ends (the 6 inch faces) of the specimen in the brittle failure event. The hoop spacing interval that had two bars buckle prior to failure had four buckled longitudinal bars (two at each end of the specimen) post-failure due to instantaneous axial shortening, and
a noticeable diagonal failure plane had developed through the core of the prism (Figure 7.6) within that same transverse hoop spacing.

![Figure 7.6 Diagonal failure plane in specimen 8NT-2 as indicated by red dashed line.](image)

Prior to strength loss, some LVDTs detected out-of-plane deformations, with LVDTs mounted on the convex side of the buckling specimen capturing vertical expansion despite overall reloading in compression. This was evidenced by the increasing tension strains registered as compression forces increased. Similar to specimen 8WT-2, even though noticeable out-of-plane deformation was recorded in both the second and third cycles to a peak tensile strain of 3%, the specimen eventually recovered to a planar shape (no out-of-plane deformations indicative of global buckling) as compression reloading continued.

Figure 7.7 illustrates the load-strain history and damage states for the cycles to 3% peak tensile strain. Initial signs of out-of-plane movement manifested in the second reloading cycle at a tension strain of approximately 1.8%, and spalling (DS3) occurred at roughly 0% strain. After reloading into compression for a third time, exposed bars buckled much sooner and brittle failure occurred just prior to reaching 0.2% axial compressive strain.
As mentioned in Section 7.2, extra precautions were made to mitigate in-plane frame drift after the first specimen test. Figure 7.8 presents the resulting frame drift ratios versus specimen axial strain and axial force. The largest drift the specimen underwent was in the 3% tension cycles, similar in behavior to the first experiment. However, the maximum drift ratio was just over 0.75%, a significant reduction from the 1.7% drift measured in the first specimen experiment. Furthermore, specimen drift ratios when in compression did not exceed 0.1%, also a decrease relative to the first experiment. Whereas in the first experiment a significant amount of drifting occurred as the imposed loads transitioned from compression to tension or vice versa, the second experiment exhibited far less horizontal movement at low levels of force.
Figure 7.8 In-plane loading frame lateral drift for specimen 8NT-2. (Left) Drift ratio vs. axial strain; (right) drift ratio vs. axial force.

7.4 Specimen 6NT-2 (Test #3)

The third specimen, 6NT-2, was tested sixteen days after specimen 8NT-2. Concrete material properties obtained from concrete cylinder tests were similar to the previous two specimens. The loading protocol used for specimen 8WT-2 was also used for specimen 6NT-2. Like the previous tests, horizontal cracks were identified in the same general locations—that is, near the transverse reinforcement. No identifiable signs of individual bar instability were noted in the cycles to peak tension strain of 2%, 3%, or 4.5% (due to the tighter spacing of hoops). As noted in prior tests, indications of out-of-plane movement were observed on both cycles on reloading from peak tensile strain to 3% into compression; however, the specimen recovered, returning to a planar shape, and was loaded to the compression strain limit of 0.2%.
Table 7.3 Peak applied axial forces for specimen 6NT-2.

<table>
<thead>
<tr>
<th>Cycle #</th>
<th>Loading Protocol</th>
<th>Initial Cycle</th>
<th>Reloading Cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension Strain Limit (%)</td>
<td>Compression Strain Limit (%)</td>
<td>Tension Force (kip)</td>
</tr>
<tr>
<td>1, 2</td>
<td>0.05</td>
<td>-0.05</td>
<td>45.87</td>
</tr>
<tr>
<td>3, 4</td>
<td>0.10</td>
<td>-0.10</td>
<td>68.74</td>
</tr>
<tr>
<td>5, 6</td>
<td>0.15</td>
<td>-0.15</td>
<td>82.04</td>
</tr>
<tr>
<td>7, 8</td>
<td>0.20</td>
<td>-0.20</td>
<td>102.2</td>
</tr>
<tr>
<td>9, 10</td>
<td>0.30</td>
<td>-0.20</td>
<td>139.39</td>
</tr>
<tr>
<td>11, 12</td>
<td>0.50</td>
<td>-0.20</td>
<td>179.05</td>
</tr>
<tr>
<td>13, 14</td>
<td>0.75</td>
<td>-0.20</td>
<td>182.52</td>
</tr>
<tr>
<td>15, 16</td>
<td>1.00</td>
<td>-0.20</td>
<td>183.9</td>
</tr>
<tr>
<td>17, 18</td>
<td>1.50</td>
<td>-0.20</td>
<td>185.73</td>
</tr>
<tr>
<td>19, 20</td>
<td>2.00</td>
<td>-0.20</td>
<td>195.24</td>
</tr>
<tr>
<td>21, 22</td>
<td>3.00</td>
<td>-0.20</td>
<td>208.95</td>
</tr>
<tr>
<td>23</td>
<td>4.50</td>
<td>-0.20</td>
<td>221.41</td>
</tr>
</tbody>
</table>

<sup>1</sup>The reported force was the maximum applied prior to strength loss due to global buckling.

Compression force applied in the first cycle to 0.002 compression strain reached 328.5 kips ($0.91A_f'$), which was similar in magnitude to the forces generated at the same cycles in specimens 8WT-2 and 8NT-2 (343.1 kips and 348.4 kips, respectively). The maximum compressive force imparted on the specimen (396.7 kips, $1.10A_f'$) occurred in the compression reloading cycle following the first excursion to 3.0%
tension strain. Applied compression force on the following cycle was identical to the first cycle applied force. The maximum applied tension force was imparted on the cycle to 4.5% tension strain, measuring 221.4 kips ($1.24A_f$, an average stress of 83.9 ksi per longitudinal bar).

Global buckling of the specimen was the ultimate failure mode for 6NT-2, as the specimen displaced out-of-plane during the first compression reloading half-cycle following the half cycle to peak tensile strain of 4.5%. Large vertical cracks along the edges developed over approximately two-thirds the specimen height (Figure 7.9), and the peak load in compression obtained in previous cycles was not achieved prior to strength loss. The peak load reached only 160.9 kips ($0.45A_f'$), only 40% of the overall maximum applied compression force. Diverging strains captured by LVDTs mounted on the south (concave) surface and north (convex) surface in Figure 7.10 indicate a global buckling failure mode. As the central region of the specimen moved out of plane to the north, the convex side LVDTs recorded elongation, while LVDTs on the concave side captured an increased rate of contraction. The results appear to indicate that, whereas the 8-inch spacing was large enough to allow local rebar buckling prior to global buckling, the 6-inch spacing restrained local rebar buckling and instead led to a global instability failure (after peak tensile strains were applied). Surface-mounted LVDTs were removed to prevent damage to the equipment once it became clear the out of plane movement was irrecoverable.
Measured frame drift ratios in specimen 6NT-2 were less than both 8WT-2 and 8NT-2 at peak tension strains. As seen in Figure 7.11, loading frame drifts at peak tension strains measured less than 0.45%, and drifts at compression peak limits were less than 0.2% drift ratio. The compression drift ratios were comparable to the drift ratio levels in the previous two experiments.
7.5 Specimen 6WT-2 (Test #4)

Specimen 6WT-2, the last of the four prisms in Group I tests, was tested approximately six weeks after 6NT-2. Horizontal cracks developed similar to those in the previous three tests. No signs of local instability (rebar buckling) were observed in cycles up to a peak tensile strain of 3.0% (Figure 7.12a). The maximum tension force applied to 6WT-2, 219.9 kips, occurred in the first cycle to 4.5% strain, which corresponded to an average stress of 83.3 ksi per longitudinal bar (see Table 7.4). Out-of-plane deformations occurred in the compression reloading cycles following tension cycles to a peak strain of 4.5%, but stability was regained once cracks closed. The out-of-plane deformations were, however, larger than observed in previous tests. After stability was regained, continued reloading in compression led to the observation that concrete cover at one corner of the specimen was displacing away from the prism core at the 6 inch hoop spacing closest to the bottom of the test, revealing a buckled longitudinal bar (Figure 7.12e). As compression reloading continued, the adjacent corner longitudinal bar buckled and pushed off the concrete cover over the same hoop spacing interval (Figure 7.13).

![Figure 7.12](image.png)

Figure 7.12 Damage progression associated with longitudinal bar buckling in specimen 6WT-2. (a) No damage associated with bar buckling; (b) cracks associated with DS1; (c) widened cracks associated with DS2; (d) cover push-off associated with DS3; (e) cover loss exposing buckled bars.
Figure 7.13 Adjacent (left) longitudinal bar buckling in specimen 6WT-2.

Table 7.4 Peak applied axial load summary for specimen 6WT-2.

<table>
<thead>
<tr>
<th>Cycle #</th>
<th>Loading Protocol</th>
<th>Initial Cycle</th>
<th>Reloading Cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension Strain Limit (%)</td>
<td>Tension Force (kip)</td>
<td>Compression Force (kip)</td>
</tr>
<tr>
<td>1, 2</td>
<td>0.05 -0.05</td>
<td>32.8 -53.82</td>
<td>30.18 -53.08</td>
</tr>
<tr>
<td>3, 4</td>
<td>0.10 -0.10</td>
<td>60.91 -123.73</td>
<td>57.37 -123.53</td>
</tr>
<tr>
<td>5, 6</td>
<td>0.15 -0.15</td>
<td>82.18 -210.49</td>
<td>79.2 -199.91</td>
</tr>
<tr>
<td>7, 8</td>
<td>0.20 -0.20</td>
<td>102.41 -291.55</td>
<td>98.34 -279.62</td>
</tr>
<tr>
<td>9, 10</td>
<td>0.30 -0.20</td>
<td>135.27 -273.10</td>
<td>133.64 -279.67</td>
</tr>
<tr>
<td>11, 12</td>
<td>0.50 -0.20</td>
<td>177.06 -310.16</td>
<td>171.16 -298.25</td>
</tr>
<tr>
<td>13, 14</td>
<td>0.75 -0.20</td>
<td>184.83 -327.34</td>
<td>182.55 -328.12</td>
</tr>
<tr>
<td>15, 16</td>
<td>1.00 -0.20</td>
<td>187.01 -345.16</td>
<td>184.14 -348.41</td>
</tr>
<tr>
<td>17, 18</td>
<td>1.50 -0.20</td>
<td>185.55 -348.54</td>
<td>187.85 -357.76</td>
</tr>
<tr>
<td>19, 20</td>
<td>2.00 -0.20</td>
<td>192.88 -363.81</td>
<td>193.35 -363.29</td>
</tr>
<tr>
<td>21, 22</td>
<td>3.00 -0.20</td>
<td>206.91 -382.27</td>
<td>207.1 -391.96</td>
</tr>
<tr>
<td>23, 24</td>
<td>4.50 -0.20</td>
<td>219.93 -398.41</td>
<td>219.17 -200.20(^{1})</td>
</tr>
</tbody>
</table>

\(^{1}\)The reported force was the maximum applied prior to strength loss due to global buckling

A second cycle to a peak tensile strain of 4.5% was attempted (Figure 7.14), but global instability (buckling) led to significant strength loss: applied axial compression load peaked at 200.2 kips (0.55\(Ag'\)) just before strength degradation, which was only 50% of the maximum compression force of 398.4 kips.
(1.10\sqrt{f_{c}'}) applied in the first 4.5% cycle (Table 7.4). Out-of-plane buckling worsened during compression reloading until a splitting failure developed over the height of the specimen (Figure 7.15). Data from the mounted LVDT columns end prior to the full-height LVDTs because the sensors were removed once the prism buckled out of plane and lost strength to prevent damage.

Figure 7.14 Force-strain plot of 4.5% tensile strain cycle for specimen 6WT-2.

Figure 7.15 Specimen 6WT-2 splitting failure after global buckling.
It is interesting to note that progression of damage leading to strength loss for specimens 6NT-2 (global buckling) and 6WT-2 (local rebar buckling followed by global buckling) were quite different. Also, based on results for the first four tests, the added crosstie for specimens 6WT-2 and 8WT-2 did not appear to have any substantial impact on test results (except possibly for the diagonal failure plane noted for test 8NT-2).

In-plane horizontal movement was relatively small. The largest frame drift ratio was approximately 0.6%, and coincided with peak tension limits at the 0.75% tension strain cycles. Subsequent cycles to larger tension strain limits exceeded 0.5% drift ratio, but did not exceed the 0.6% reached in the 0.75% cycles. In-plane frame drifts associated with specimen compression were very small, with most cycles experiencing no more than 0.05% drift ratio at the compression strain limit.

Figure 7.16 In-plane loading frame lateral drift for specimen 6WT-2. (Left) Drift ratio vs. axial strain; (right) drift ratio vs. axial force.
7.6 Specimen 8WT-3 (Test #5)

The first specimen tested from the Group II prisms was Specimen 8WT-3: transverse hoops were spaced 8 inches apart and crossties were provided; the same configuration as in Specimen 8WT-2. However, specimen 8WT-3 was loaded to a maximum compressive strain limit of 0.003 rather than the 0.002 value used for specimen 8WT-2. The experiment was performed six weeks after the concrete was poured.

Table 7.5 Peak applied forces for loading cycles of specimen 8WT-3.

<table>
<thead>
<tr>
<th>Cycle #</th>
<th>Loading Protocol</th>
<th>Initial Cycle</th>
<th>Reloading Cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension Strain Limit (%)</td>
<td>Compression Strain Limit (%)</td>
<td>Tension Force (kip)</td>
</tr>
<tr>
<td>1, 2</td>
<td>0.05</td>
<td>-0.05</td>
<td>32.55</td>
</tr>
<tr>
<td>3, 4</td>
<td>0.10</td>
<td>-0.10</td>
<td>61.19</td>
</tr>
<tr>
<td>5, 6</td>
<td>0.15</td>
<td>-0.15</td>
<td>83.37</td>
</tr>
<tr>
<td>7, 8</td>
<td>0.20</td>
<td>-0.20</td>
<td>105.22</td>
</tr>
<tr>
<td>9, 10</td>
<td>0.30</td>
<td>-0.30</td>
<td>141.83</td>
</tr>
<tr>
<td>11, 12</td>
<td>0.50</td>
<td>-0.30</td>
<td>175.99</td>
</tr>
<tr>
<td>13, 14</td>
<td>0.75</td>
<td>-0.30</td>
<td>178.68</td>
</tr>
<tr>
<td>15, 16</td>
<td>1.00</td>
<td>-0.30</td>
<td>178.17</td>
</tr>
<tr>
<td>17, 18</td>
<td>1.50</td>
<td>-0.30</td>
<td>182.2</td>
</tr>
<tr>
<td>19, 20</td>
<td>2.00</td>
<td>-0.30</td>
<td>188.63</td>
</tr>
<tr>
<td>21, 22</td>
<td>3.00</td>
<td>-0.30</td>
<td>200.81</td>
</tr>
<tr>
<td>23, 24²</td>
<td>4.50</td>
<td>-0.30</td>
<td>215.36</td>
</tr>
</tbody>
</table>

¹The reported force was the maximum applied prior to strength loss due to global buckling
²Tension reloading to 4.5% occurred after incomplete compression cycle due to strength loss
³Peak residual compression force after it became apparent out-of-plane movement was worsening

Like the previous four tests, specimen 8WT-3 developed essentially horizontal tension cracks that circumscribed the prism near the horizontal hoops starting at low tension cycles (as early as 0.001 strain cycles). No spalling or crushing was observed as the imposed compression strain reached 0.003, and the peak strength of the specimen at 0.003 compressive strain was 438 kips (1.28A_g\(f'_c\)). As tension strain demands increased to 3%, the specimen exhibited little evidence of damage beyond tension cracks. After tension strain demands reached a limit 4.5%, where tension force peaked at 215 kips (1.18A_fy), the
specimen developed out of plane instability and lost strength under compression reloading during the first cycle to 4.5%. Compression capacity of the specimen reached 130 kips (30% of peak compressive strength of 438 kips, $0.38A_f'c$) before strength was lost and out of plane instability worsened. Once it became clear the specimen was losing strength and was incapable of correcting itself from out of plane displacement, the compression cycle was halted prior to reaching the 0.003 compression strain limit. An extra tension cycle was performed to the 4.5% tension strain limit, and compression reloading was performed again on the specimen. Again, the specimen exhibited out of plane displacement, and the peak compression capacity was less than in the first compression reloading half-cycle—only 63 kips (14% of peak compressive strength of 438 kips).

![Force-strain plot for specimen 8WT-3 cycles to 3% and 4.5% tension strain.](image)

Figure 7.17 captures the global buckling phenomenon upon compression reloading from the 4.5% tension strain limit. The green dash-dot curve and blue solid line represent the average strains measured on the south and north LVDT columns west of the specimen centerline. The two curves diverge just after compression reloading begins, and strength loss is apparent shortly thereafter. Mounted LVDTs were removed upon specimen strength loss, and as such, there is no data from these sensors for the second
reloading cycle. The buckled shape after the initial 4.5% tension strain cycle is shown in Figure 7.18. Exposed longitudinal bars and the specimen buckled shape are visible in Figure 7.20.

![Figure 7.18 Out-of-plane buckling in specimen 8WT-3 (north surface on left).](image)

Similar to the previous four experiments, a single LVDT was mounted to the loading frame connected to the capital block on specimen 8WT-3 to capture in-plane horizontal displacements. The plots in Figure 7.19 show that drifts did not exceed 0.5% throughout the duration of the test, and drifts at the compression strain limit were less than 0.2%. Peak horizontal displacements occurred during compression reloading while the longitudinal bars were the only source of axial resistance (i.e. concrete cracks not yet closed). This behavior differs from the previous four tests, where the largest drifts occurred at the tension loading half-cycles. Like in previous tests, a large concentration of movement occurred as loads transitioned from tension to compression.
Figure 7.19 In-plane loading frame lateral drift for specimen 8WT-3. (Left) Drift ratio vs. axial strain; (right) drift ratio vs. axial force.

Figure 7.20 Specimen 8WT-3 after post-test spalled concrete cover removal.
7.7 Specimen 6WT-3 (Test #6)

Specimen 6WT-3 was the second specimen tested from the Group II prisms. It had the same configuration as 6WT-2, but the applied strain history in 6WT-3 had an increased compressive strain limit of 0.003 instead of 0.002. Similar to 8WT-3, no significant damage except tension cracks was observed in cycles through the completion of the 2% tension strain limit/0.3% compression limit cycles. Slight out of plane movement was visually observed and captured by the mounted LDVTs in the first cycle of compression reloading from 3% tension strain. LVDT columns recorded extension in average strains on the south face of the prism while undergoing compression, indicating out of plane movement to the south. The specimen was able to reestablish stability (once tension cracks closed) and the compression half-cycle was completed with no significant new observed damage. It is noted that the average peak tension strains recorded by LVDT columns mounted on the north and south faces of the specimen differed by approximately 0.5% in the 2% and 3% peak tension strain cycles, with the south face exhibiting the larger of the strain recordings. This difference in average strain from one face of the specimen to the other was an indicator of impending out-of-plane instability upon load reversal. The maximum applied tension was 211 kips (1.16$f_y$, an average stress of 79.9 ksi per longitudinal par), and the maximum applied compressive force was 428 kips (1.25$f'_c$). Figure 7.21 shows the force-strain plots for the northwest and southwest longitudinal bars based on the strain field calculated by the three full-height LVDTs, as well as the average strains recorded by the four columns of mounted LVDTs.
Table 7.6 Peak applied axial force for each cycle of specimen 6WT-3.

<table>
<thead>
<tr>
<th>Cycle #</th>
<th>Tension Strain Limit (%)</th>
<th>Compression Strain Limit (%)</th>
<th>Tension Force (kip)</th>
<th>Compression Force (kip)</th>
<th>Tension Force (kip)</th>
<th>Compression Force (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1, 2</td>
<td>0.05</td>
<td>-0.05</td>
<td>33.63</td>
<td>-68.69</td>
<td>37.64</td>
<td>-68.85</td>
</tr>
<tr>
<td>3, 4</td>
<td>0.10</td>
<td>-0.10</td>
<td>65.45</td>
<td>-155.94</td>
<td>61.96</td>
<td>-153.17</td>
</tr>
<tr>
<td>5, 6</td>
<td>0.15</td>
<td>-0.15</td>
<td>88.32</td>
<td>-238.38</td>
<td>85.25</td>
<td>-232.45</td>
</tr>
<tr>
<td>7, 8</td>
<td>0.20</td>
<td>-0.20</td>
<td>106.56</td>
<td>-310.28</td>
<td>104.32</td>
<td>-294.61</td>
</tr>
<tr>
<td>9, 10</td>
<td>0.30</td>
<td>-0.30</td>
<td>144.41</td>
<td>-427.71</td>
<td>139.93</td>
<td>-411.56</td>
</tr>
<tr>
<td>11, 12</td>
<td>0.50</td>
<td>-0.30</td>
<td>177.22</td>
<td>-394.24</td>
<td>170.8</td>
<td>-388.32</td>
</tr>
<tr>
<td>13, 14</td>
<td>0.75</td>
<td>-0.30</td>
<td>178.25</td>
<td>-396.85</td>
<td>175.72</td>
<td>-390.71</td>
</tr>
<tr>
<td>15, 16</td>
<td>1.00</td>
<td>-0.30</td>
<td>177.22</td>
<td>-390.60</td>
<td>175.9</td>
<td>-390.07</td>
</tr>
<tr>
<td>17, 18</td>
<td>1.50</td>
<td>-0.30</td>
<td>180.14</td>
<td>-395.57</td>
<td>179.3</td>
<td>-388.60</td>
</tr>
<tr>
<td>19, 20</td>
<td>2.00</td>
<td>-0.30</td>
<td>187.4</td>
<td>-397.62</td>
<td>186.19</td>
<td>-396.54</td>
</tr>
<tr>
<td>21, 22</td>
<td>3.00</td>
<td>-0.30</td>
<td>201.45</td>
<td>-416.65</td>
<td>198.36</td>
<td>-411.24</td>
</tr>
<tr>
<td>23</td>
<td>4.50</td>
<td>-0.30</td>
<td>211.31</td>
<td>-133.18&lt;sup&gt;1&lt;/sup&gt;</td>
<td>--</td>
<td>--</td>
</tr>
</tbody>
</table>

<sup>1</sup>The reported force was the maximum applied prior to strength loss due to global buckling.

Figure 7.21 Force-strain plot for specimen 6WT-3 cycles to 3% and 4.5% tension strain.

Irrecoverable out of plane instability manifested in the first cycle of compression reloading after achieving 4.5% average tension strain. At the peak tension strain limit, the strain readings on the south
face LVDT columns averaged 0.75% more tension strain than those recorded by the north LVDT columns. Applied compression force reached 87 kips (20% of peak compression) before the south-face mounted LVDTs began registering extension (indicating the south face was the convex surface of the buckling prism), and a local maximum compression force of 133 kips (31% of peak compression) was achieved prior to specimen strength loss. The specimen exhibited out-of plane instability (global buckling) to the south, the side exhibiting larger average strains relative to the north face. Figure 7.22 captures the out of plane deformed shape (south is to the right in the figure) and shows the gap closure difference in horizontal cracks in the central region of the specimen. The experiment was terminated once strength loss became apparent, and the 0.3% compression strain limit was not reached following the first tension cycle at 4.5% tension strain.

Figure 7.22 Global buckling deformed shape of specimen 6WT-3 during initial compression reloading cycle from 4.5% tension strain limit.
Drift plots in Figure 7.23 show that the largest in-plane loading frame movement occurred at large tension strains. Maximum drift ratios did not exceed 0.45% throughout the duration of the experiment, and at peak compression strains the lateral drift was less than 0.18%. As in previous tests, a large amount of horizontal movement occurred during the transitions of load reversal.

![Figure 7.23 In-plane loading frame lateral drift for specimen 6WT-3. (Left) Drift ratio vs. axial strain; (right) drift ratio vs. axial force.](image)

### 7.8 Specimen 45WT-45 (Test #7)

Neither specimen 8WT-3 nor specimen 6WT-3 showed signs of compression failure due to concrete crushing or cover spalling when loaded up to compression strains of 0.003. Based on the performance of those two specimens and the fact that the transverse reinforcement in 45WT-45 provided more confinement by virtue of the tighter 4.5 inch vertical spacing of hoop and tie than in 8WT-3 and 6WT-3, the maximum compression strain limit was increased to 0.45%. All other loading protocols remained consistent with the previous six experiments. Observed behavior during the low-level strain cycles (e.g., 0.05% to 0.2% strain cycles) was consistent with the other prism tests. A vertically inclined hairline crack approximately two inches long developed on the north face at the first excursion to the 0.45% compression strain limit (Figure 7.24), but no concrete crushing nor cover spalling occurred. The peak compressive force applied to Specimen 45WT-45 was 485 kips ($1.41A_g f'_c$), approximately 1.72 and 1.22
times the peak values for the 0.2% and 0.3% peak strain cycles, respectively. As imposed tension strain limits increased beyond 1%, average measured strains on the eastern LVDT columns began registering tension strains larger than the average of the western LVDT columns. Also, LVDT columns on the north face of the test prism recorded average compression strains of roughly 0.003, or approximately 50% less than the 0.0046 average compressive strain detected by LVDT columns on the south face (Figure 7.25). These measured strain differences, especially at the compression strain limit, may indicate subtle out-of-plane curvature of the specimen occurring even when cracks were completely closed.

Table 7.7 Peak applied axial load for each cycle of specimen 45WT-45.

<table>
<thead>
<tr>
<th>Cycle #</th>
<th>Loading Protocol</th>
<th>Initial Cycle</th>
<th>Reloading Cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension Strain Limit (%)</td>
<td>Compression Strain Limit (%)</td>
<td>Tension Force (kip)</td>
</tr>
<tr>
<td>1, 2</td>
<td>0.05</td>
<td>-0.05</td>
<td>40.32</td>
</tr>
<tr>
<td>3, 4</td>
<td>0.10</td>
<td>-0.10</td>
<td>63.29</td>
</tr>
<tr>
<td>5, 6</td>
<td>0.15</td>
<td>-0.15</td>
<td>86.67</td>
</tr>
<tr>
<td>7, 8</td>
<td>0.20</td>
<td>-0.20</td>
<td>109.77</td>
</tr>
<tr>
<td>9, 10</td>
<td>0.30</td>
<td>-0.30</td>
<td>143.76</td>
</tr>
<tr>
<td>11, 12</td>
<td>0.45</td>
<td>-0.45</td>
<td>171.00</td>
</tr>
<tr>
<td>13, 14</td>
<td>0.75</td>
<td>-0.45</td>
<td>175.20</td>
</tr>
<tr>
<td>15, 16</td>
<td>1.00</td>
<td>-0.45</td>
<td>175.58</td>
</tr>
<tr>
<td>17, 18</td>
<td>1.50</td>
<td>-0.45</td>
<td>179.98</td>
</tr>
<tr>
<td>19, 20</td>
<td>2.00</td>
<td>-0.45</td>
<td>188.42</td>
</tr>
<tr>
<td>21, 22</td>
<td>3.00</td>
<td>-0.45</td>
<td>199.09</td>
</tr>
<tr>
<td>23, 24</td>
<td>4.50</td>
<td>-0.45</td>
<td>211.49</td>
</tr>
</tbody>
</table>

1The reported force was the maximum applied prior to strength loss due to global buckling
2The reported force was the maximum applied prior to test termination due to concerns for stability
The maximum tension force applied to specimen 45WT-45 was 211.5 kips (1.16$A_fy$, approximately 80.1 ksi per longitudinal bar) when the tension strain limit of 4.5% was reached for the first time. Concrete spalling on the northeastern edge of the prism was observed in the compression cycle following the first
excursion to a tension strain of 3.0%, but no longitudinal reinforcement was exposed (Figure 7.26). Ultimately, specimen 45WT-45 developed out of plane instability upon reloading from the first tension cycle to 4.5% strain. Mounted LVDTs were removed as the buckling intensified. The maximum compression force applied on the reloading cycle from 4.5% tension strain was 169 kips (34.8% of the maximum compression force applied over the entire course of the experiment). At this point the specimen was clearly buckling out-of-plane, and strength loss quickly developed. A residual compression strength of 60 kips (12.4% of the maximum applied compression force) was reached as the experiment continued to undergo compression until the 0.45% compression strain limit was achieved. A second tension cycle to 4.5% strain was completed, with the applied tension force reaching 201.2 kips (95% of the peak tension force achieved in the first 4.5% tension strain cycle). A second compression reloading cycle was attempted, but the specimen buckled almost immediately as compression forces were applied, and the experiment was ended.

Figure 7.26 Spalling on northeast (left) edge of 45WT-45.
Figure 7.27 Out-of-plane buckling in specimen 45WT-45, first compression reloading from 4.5% tension strain limit.

Figure 7.28 In-plane loading frame lateral drift for specimen 45WT-45. (Left) Drift ratio vs. axial strain; (right) drift ratio vs. axial force.
Loading frame in-plane translation measurements were small relative to the previous six tests. Drift ratios when the specimen was in tension did not exceed 0.25%, and drifts at compression strain limits were less than 0.1%. Figure 7.28 also shows the in-plane drift ratio dramatically increased at the end of the experiment as the specimen developed out-of-plane instability.

### 7.9 Specimen 45WT-I (Test #8)

Specimen 45WT-I was the final experiment in the Group II tests. The specimen performed similarly to the previous seven tests up to applied tension and compression cycles to 0.2% strain. Beyond 0.2% strain, where tension cycle limits increased incrementally up to 1% strain while the compression cycle limits were maintained at 0.2% strain, the prism developed more pronounced horizontal tension cracks near horizontal reinforcement locations. After the cycles with 1% tension strain and 0.2% compression strain, both tension and compression strains were increased. No damage, other than the predominantly horizontal tension cracks mentioned above, was observed. Additionally, no signs of concrete cover spalling were observed in cycles with compression strain demands up to 0.45%. Large jumps in peak applied compression force at compression strain limits coincided with compressive strain limit increases from 0.2% to 0.3%, and 0.3% to 0.45%. Peak compression in cycles limiting compressive strain to 0.2% was 338.1 kips ($0.99A_{g'c}$). Cycles with compression strain limits of 0.3% had a peak compression force of 466.9 kips ($1.36A_{g'c}$), and cycles with compression strain limits set to 0.45% had a peak compression force of 537.1 kips ($1.57A_{g'c}$).
Table 7.8 Peak applied axial loads for each cycle of specimen 45WT-I.

<table>
<thead>
<tr>
<th>Cycle #</th>
<th>Loading Protocol</th>
<th>Initial Cycle</th>
<th>Reloading Cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tension Strain Limit (%)</td>
<td>Compression Strain Limit (%)</td>
<td>Tension Force (kip)</td>
</tr>
<tr>
<td>1, 2</td>
<td>0.05</td>
<td>-0.05</td>
<td>28.20</td>
</tr>
<tr>
<td>3, 4</td>
<td>0.10</td>
<td>-0.10</td>
<td>54.15</td>
</tr>
<tr>
<td>5, 6</td>
<td>0.15</td>
<td>-0.15</td>
<td>75.70</td>
</tr>
<tr>
<td>7, 8</td>
<td>0.20</td>
<td>-0.20</td>
<td>98.79</td>
</tr>
<tr>
<td>9, 10</td>
<td>0.30</td>
<td>-0.20</td>
<td>142.05</td>
</tr>
<tr>
<td>11, 12</td>
<td>0.50</td>
<td>-0.20</td>
<td>176.28</td>
</tr>
<tr>
<td>13, 14</td>
<td>0.75</td>
<td>-0.20</td>
<td>178.59</td>
</tr>
<tr>
<td>15, 16</td>
<td>1.00</td>
<td>-0.20</td>
<td>179.27</td>
</tr>
<tr>
<td>17, 18</td>
<td>1.00</td>
<td>-0.30</td>
<td>176.30</td>
</tr>
<tr>
<td>19, 20</td>
<td>1.50</td>
<td>-0.30</td>
<td>181.44</td>
</tr>
<tr>
<td>21, 22</td>
<td>1.50</td>
<td>-0.45</td>
<td>181.23</td>
</tr>
<tr>
<td>23, 24</td>
<td>2.00</td>
<td>-0.45</td>
<td>187.86</td>
</tr>
<tr>
<td>25</td>
<td>2.00</td>
<td>-0.60</td>
<td>186.72</td>
</tr>
</tbody>
</table>

¹Brittle failure and strength loss occurred as strains exceeded 0.45%
²Post-failure buckled bars straightened out at tension strain limit
³Maximum post-failure compression prior to test termination due to stability concerns

Figure 7.29 captures the force-strain behavior of specimen 45WT-I at cycles near the end of the experiment. The average compression strains registered by the mounted LVDT columns were slightly less in magnitude than the specimen average strains calculated by block-to-block sensors, with the exception of the southeast set of sensors. LVDTs mounted to the southeast column nodes recorded compression strains less than half the strains measured by the block-to-block sensors. This is apparent in the green curve in Figure 7.29.
Sudden, brittle failure occurred in the first excursion to 0.6% compression strain. No signs of concrete or crushing occurred before the compression strain reached 0.45%. However, just as the average compression strain surpassed 0.45%, immediate failure ensued. Slow-motion review of high-definition video recording showed minor cracking develop on the south side of the specimen, followed by failure less than 3 seconds later. Figure 7.30 contains screen captures of the failure event, progressing from left to right. The picture in Figure 7.30a shows the test prism with no signs of damage due to compression. Figure 7.30b is taken only seconds later, showing the development of vertical cracks at the southeast and southwest edges of the specimen. Taken less than two seconds after the test prism status shown in Figure 7.30b, Figure 7.30c captures the explosive crushing failure of specimen 45WT-I.
In-plane lateral drift measurements of the loading frame are shown in Figure 7.31. Lateral drift ratios associated with peak tension strain limits did not exceed 0.3%, and ratios associated with peak compression strain limits did not exceed 0.1% (with the exception of the final cycle to 0.6% compression strain, which experienced sudden failure and registered a maximum in-plane drift ratio of 0.14%). The largest drifts occurred during compression reloading from large tension strain excursions while the bars were the only source of compressive strength and lateral resistance (i.e., before tension cracks closed...
completely and loads could be redistributed into concrete). Frame drifts in compression reloading prior to the concrete engaging were as large as 0.4% in the latter cycles.

7.10 Experimental Summary

Observations from the eight boundary element prism tests demonstrate that longitudinal bar instability (buckling) can initiate concrete cover spalling that leads to strength loss in 6” by 15” cross sections with one inch of cover. In the first group of four experiments, axial compressive strains were limited to 0.2% to avoid post-peak crushing of (essentially unconfined) concrete, and to verify that damage could be initiated by unstable longitudinal reinforcement buckling prior to concrete crushing. Three of four experiments exhibited local bar buckling resulting in push-off of cover prior to any observed concrete crushing. In each of these three tests, once localized damage occurred, global lateral instability failure or through-section diagonal failure (8NT-2) occurred in the subsequent loading cycle. The final, damaged state of the tests with longitudinal reinforcement buckle appeared similar to several of the Chilean walls investigated, and could be interpreted as a section crushing failure or lateral instability failure if inspected without knowledge of the damage progression.

For a 6” by 15” cross section with one inch of clear cover over longitudinal reinforcing bars, bar buckling pushing off cover concrete represented a significant loss in section compression capacity and in lateral stability. The presence of a crosstie did not seem to impact the performance of longitudinal reinforcement placed at the centerline of the longer side of the test specimen, although sudden, brittle through-section failure (following bar buckling) was observed only in the 8NT-2 specimen, which did not have a crosstie.
The second group of test prisms did not exhibit the surface vertical cracking or spalling associated with longitudinal bar buckling instability, but still established limitations for thin wall boundary elements. Out-of-plane instability of the prism led to the ultimate failure of three of the four prism tests in Group II. As the specimens reloaded into compression from a peak tension strain of 3% or 4.5%, tension cracks closed unevenly and the prisms could not reestablish stability. The tension strain excursions at which the specimen failed to reload from into compression are remarkably close to the models established by Paulay and Priestley (1993) and Chai and Elayer (1999), which show another mechanism thin boundary elements are susceptible to in an extreme seismic event. According to the models advanced by both research groups, the critical reloading strain that would generate out-of-plane instability associated with the aspect ratio, reinforcement ratios, mechanical properties, and reinforcement configuration of the UCLA tests was approximately 2.7-2.9%. Specimens from Group II had a lower critical strain associated with global buckling due to concrete compression strength slightly lower and longitudinal reinforcement yield strength slightly higher than in the Group I tests.

Figure 7.32 summarizes the results of the seven prisms where failure was initiated by either buckling of longitudinal reinforcement, or by out-of-plane instability. The largest reloading strain from a complete, stable compression cycle is plotted against the specimen transverse hoop spacing to longitudinal bar diameter. Prisms that exhibited localized buckling of longitudinal reinforcement prior to concrete degradation are represented by the red diamonds, and prisms that failed due to out-of-plane, global buckling are represented by the green icons. For reference, trends from isolated reinforcement buckling studies are also plotted, as well as the calculated critical reloading strain limit for the prisms as described above. Specimen 45WT-I was not included in this figure since both the loading history and failure mechanism differed substantially from the other experiments. A recommended strain versus \( s/d_h \) ratio is established for \( s/d_h \) ratios from 8 to 10.7, based on a linear fit for the six tests with 8” or 6” spacing of transverse reinforcement. As expected, these values show additional stability for larger reloading strains.
than the results for isolated bars. The specimens that failed due to global (lateral) instability without exhibiting individual bar buckling approached the theoretical limit for reloading strain for global (lateral) instability as established by Paulay and Priestley (1993) and Chai and Elayer (1999). Although they did not exhibit individual bar buckling, they demonstrate that the hoop spacing to bar diameter ratio is capable of reaching the reported compression reloading strains prior to buckling. The single test reported for \( s/d_b \) ratio of 6 reached the reloading strain limit for out-of-plane instability. The recommended curve between \( s/d_b \) of 6 and 8 connects the fitted point at 8 and intersects the curves for isolated bars at a ratio of 6. While it is logical to assume that the available reloading strain of bars embedded in concrete would exceed the reloading strains for isolated bars, additional information is needed to confirm this. It is recommended to test additional specimens at \( s/d_b \) ratios of 6 and 8 with geometric configurations that do not limit reloading due to global instability, to further populate a curve for bar buckling in concrete elements.

Figure 7.32 Reloading strain results versus transverse spacing.
Brittle concrete crushing failure was observed in specimen 45WT-I as the compressive strain reached approximately 0.5%, indicating the ultimate compressive strain capacity was only marginally above a limit equal to 1.5 times the commonly used unconfined concrete strain limit of 0.3%. Potential uncertainties associated with the displacement-based design approach of ACI 318-11 21.9.6.2 should be assessed to determine if this margin is acceptable.

The test results indicate that brittle failure and rapid strength loss are likely for thin wall boundary elements with large spacing of transverse reinforcement; therefore, the use of this configuration should be discouraged in critical regions, e.g., plastic hinging regions. Either local crushing of concrete or local bar buckling is likely to cause lateral instability on subsequent cycles, and thus, rapid and significant strength loss with little to no ductility. ACI 318-11 has no minimum wall thickness requirement and, where special boundary elements are not required, Section 21.9.6.5 allows ordinary boundary element transverse spacing to be up to 8 inches regardless of cross-section geometry or longitudinal bar diameter. The reinforcing details in all eight tests conformed to OBE requirements, but results of these tests show the potential for damage at ratios of transverse hoop spacing to longitudinal bar diameter of 10.7 and 8, and at compression strain demands that slightly exceed 0.45%.
CHAPTER 8  POTENTIAL BAR BUCKLING IN WALL BOUNDARIES

Previous research has shown that longitudinal bars can buckle after smaller reloading compression strains as the unsupported length, typically taken equal to the spacing of transverse reinforcement, increases. A study was conducted to identify trends in potential for rebar buckling for various wall configurations to identify configurations likely to lead to large tension strain demands in longitudinal reinforcing bars, which when combined with large hoop spacing (e.g., as used for ordinary boundary elements), could potentially lead to longitudinal bar buckling in structural wall boundary elements. The section analysis equation derived by Wallace and Moehle (1992) was used to determine trends for the neutral axis depth of wall configurations with variable reinforcement ratios and axial load demands:

\[
\frac{c}{l_w} = \left( \frac{\rho + \rho'' - \frac{\gamma}{\alpha} \rho'}{\rho} \right) \frac{af_y}{f'_c} + \frac{P}{0.85 \beta_i + 2 \rho'' \frac{af_y}{f'_c}}
\]

8.1

![Figure 8.1 Idealized wall cross section for Eq. 8.1.](image)

In Equation 8.1, \(c/l_w\) is the neutral axis depth normalized by the total wall length in the direction of the wall web; \(t_w\) is the wall thickness; \(\rho, \rho', \text{ and } \rho''\) are the steel reinforcement ratios at the tension end of the wall, the compression end of the wall, and the web steel reinforcement ratio, respectively; \(f_y\) and \(f'_c\) are the steel yield strength and the concrete compressive strength, respectively; \(\gamma\) and \(\alpha\) are factors adjusting for overstrength of steel reinforcement and strain hardening (\(\alpha = 1.50\) and \(\gamma = 1.25\) in the proceeding
analysis); $P$ is the axial load demand on the wall; and $\beta_i$ is from the assumed Whitney stress block. While using the Whitney stress block term for internal equilibrium technically is only valid for extreme fiber compression strains of 0.003, the equation yields good results over a range of extreme fiber compressive strains from roughly 0.002 to 0.006. Assuming plane sections remain plane, smaller neutral axis depths will drive up tension strain demands on longitudinal reinforcement relative to compression demands.

A number of parameters were varied to observe trends in neutral axis depth for planar walls. Studies use material strengths of 4, 6, and 8 ksi for concrete and 60ksi for reinforcing steel. For the first study, web steel reinforcement was set to the minimum reinforcement ratio permitted by ACI 318, 0.0025, and the minimum steel reinforcement ratio at either end of the wall was initially set at 0.002, but allowed to increase on one end or another (but not both). The 0.002 value is the ratio of reinforcement only in the wall boundary divided by the entire wall area—a boundary element encompassing 20% of the wall length with a local reinforcement ratio of 0.01 would correspond to 0.002 with respect to the entire wall length. Figure 8.2 through Figure 8.4 show that larger axial load demands and tension steel relative to compression steel drive up the neutral axis depth. Increasing compression steel relative to tension steel decreases the neutral axis depth. Increasing the concrete compression strength reduces the slope of the curves, indicating that, relative to walls with lesser concrete strength, more tension steel is necessary to increase the neutral axis depth in walls with higher $f'_c$ (Figure 8.5).
Figure 8.2 Wall neutral axis depth versus axial load and steel reinforcement ratio ($f'_c = 4$ ksi).

Figure 8.3 Wall neutral axis depth versus axial load and steel reinforcement ratio ($f'_c = 6$ ksi).
The second part of this study was to vary the amount of boundary element longitudinal reinforcement, while keeping axial load demands fixed. In Figure 8.6 through Figure 8.9, the wall axial load and web reinforcement ratio are held constant, and end steel ratios increase, both in baseline amount ($\rho_{\text{min}}$, $\rho'_{\text{min}}$).
and relative to each other. The blue dashed line in Figure 8.6 represents the same information as in Figure 8.2, showing the influence of one boundary element with increasing longitudinal reinforcement while the other end remains constant. The other curves in Figure 8.6 show the same concept as the blue dashed lines, but for increasing amounts of boundary element longitudinal reinforcement. At symmetry (both compression and tension ends have the same amount of steel) it is apparent that, as the balanced amount of steel in the wall increases, the neutral axis depth also increases. For walls with larger baseline amounts of longitudinal reinforcement, the change in neutral axis depth is more rapid (both with increasing compression steel and increasing tension steel) than for walls with smaller minimum amounts of boundary element longitudinal reinforcement. Increasing concrete compression strength has the same effect as in the prior group of plots: adding tension steel in walls with relatively large compressive strengths has less impact on the neutral axis depth than in walls with lower compressive strength (Figure 8.9).

![Figure 8.6 Wall neutral axis depth for varying wall end minimum steel ratios ($f'_c = 4$ ksi).](image)

200
Figure 8.7 Wall neutral axis depth for varying wall end minimum steel ratios ($f'_{c} = 6$ ksi).

Figure 8.8 Wall neutral axis depth for varying wall end minimum steel ratios ($f'_{c} = 8$ ksi).
These figures show that the most likely wall candidate for a small neutral axis depth has low axial load demands, with lower boundary longitudinal reinforcement ratios, and with one end of the wall having significantly more longitudinal steel than the other. Asymmetrically flanged walls—i.e., L-shaped or T-shaped—may fit the criteria for large differences between boundary element longitudinal reinforcement. Non-bearing type walls or lightly axially loaded bearing-type walls might also have low values of neutral axis depth. The numerous, flanged-shaped walls with minimal web-end longitudinal reinforcement ubiquitous in Chilean mid-rise construction fit the description mentioned above, i.e., are likely to have boundary longitudinal reinforcement that is subjected to large tensile strains.

Another factor that influences the magnitude of longitudinal bar strain demand is roof drift demand. There are multiple relationships that show how plastic hinge curvature increases with increasing roof drift demands, such as Equation 8.2 (Wallace and Moehle, 1992) and Equation 8.3 (implied in ACI 318-11).

\[
\frac{\delta_u}{h_w} = \frac{11}{40} \phi \phi_h^2 + \left(\phi_u - \phi_h\right) \phi_w \left(h_w - \frac{l_p}{2}\right) 
\]
\[ \frac{\Delta_u}{h_w} = \phi_u l_p h_w \] 8.3

In general, wall neutral axis depths do not change significantly for moderate levels of extreme fiber compressive strain (including the investigation scope between roughly 0.002 and 0.006). Curvature can be defined as in Equation 8.4, and if a fixed neutral axis depth is assumed, compression strain demands become proportional to ultimate curvature demands (i.e., plane sections are assumed to remain plane).

\[ \phi_u = \frac{\varepsilon_{cu}}{c} \] 8.4

Therefore, as drift demands increase, so must the maximum extreme fiber compression strain. Figure 8.10 illustrates how, for a given neutral axis depth, compression strains are driven to larger values as roof drift demands increase. Each curve represents a contour where neutral axis depth and drift demand require the same compressive strain.

Figure 8.10 Wall neutral axis depth and drift demand impact on compressive strain.
The figure also includes the code imposed drift limit for reinforced concrete walls per ASCE 7-10, as well as a visual representation of the ACI 318-11 special boundary element detailing trigger equation. Essentially, for combinations of wall neutral axis depths less than $0.238l_w$ and drift ratios that fall to the left of the 0.003 compressive strain contours, special boundary element detailing is not necessary, and larger spacing of transverse reinforcement is permitted (and less volume). Assuming plane sections remain plane, tension strain demands in longitudinal reinforcement must also increase proportional to the extreme fiber compression strain. Figure 8.11 shows the extreme tension strain demand in longitudinal bars on the opposite end of the wall (assuming plane sections) given the compressive strain contours from Figure 8.10 and the neutral axis depth.

Initial buckling strains for different length-to-bar diameter ratios per Rodriguez et al. (1999) are also plotted in Figure 8.11, to illustrate the potential for longitudinal bar buckling on load reversal. Estimated tension steel strains above 1.0% (e.g., for $c/l_w$ less than 0.15 with maximum compression strain of 0.002) might produce buckling in bars with hoop spacing equal to eight times the longitudinal bar diameter;
tension strains greater than 3.2% (e.g., for $c/l_w$ less than 0.09 with maximum compression strain of 0.003) could cause buckling in bars with hoop spacing equal to six times longitudinal bar buckling.

Knowing drift demands from building analyses and neutral axis depths from wall section analyses allows for determining if compression demands warrant special confining details per ACI 318-11 section 21.9.6, but they can also show whether longitudinal bar strains are at levels high enough that bar buckling could initiate on load reversal (expected to occur in an earthquake, especially under a significantly long duration of shaking).

A minor manipulation of the simplified expression relating drifts to curvature and extreme compression fiber strain can relate the extreme longitudinal bar tension strain with drift demands. Plane section analysis relates tension strain to compression strain by the following:

$$
\varepsilon_{st} = \varepsilon_{cu} \left( \frac{l_w}{c} - 1 \right)
$$

This relationship can be combined with Equations 8.3 and 8.4, and assuming a plastic hinge length $l_p$ of half the wall length produces:

$$
\varepsilon_{st} = 2 \left( \frac{\delta_p}{h_w} \right) \left( 1 - \frac{c}{l_w} \right)
$$

The expression in Equation 8.6 holds an advantage over determining bar strains from the compression strain contours discussed above and shown in Figure 8.11, at least from a design viewpoint. Assuming the wall behavior (i.e., tension yielding of boundary longitudinal reinforcement and sufficient transverse reinforcement to prevent crushing failure) validates the plastic hinge model assumption inherent in the...
above relationship, and knowing that the neutral axis depth to wall length ratio does not change significantly for compressive strains less than approximately 0.006 (and maybe as large as 0.01), bar tension strain demands can be determined without calculating the compression strain at a given drift ratio. This simplification is useful for practicing engineers, who in most applications assume a Whitney stress block model ($\varepsilon_c=0.003$) in determining neutral axis depths. Figure 8.12 illustrates how for a given neutral axis depth, tension strains at the opposite wall boundary increase with larger roof drift ratios. Alternatively, it shows that for a specified roof drift ratio, walls with deeper neutral axis depths (larger $c/l_w$ ratios) have smaller tension strain demands due to plane section assumptions. In particular, Figure 8.12 demonstrates that a flanged wall with the flange loaded in compression—and an associated neutral axis depth that can easily be less than 10% of the overall wall length—can generate large tension strain demands at higher drift levels.

Figure 8.12 Tension strain as a function of drift and neutral axis depth

Figure 8.12 also incorporates the bar buckling initiation strains derived by Rodriguez et al. (1999) into the plot. It shows the strain limits associated with different $s/d_b$ ratios relative to strain demands obtained
from section analysis and building drift demands. The yellow area shows where bar strain exceeds the reloading strain associated with buckling onset for isolated bars with an \( s/d_b \) ratio of 6. The orange region highlights where reloading strains would exceed the reloading strain limit for longitudinal reinforcement supported by hoops and ties spaced at 7 times the bar diameter, and light red shows where the strains would exceed the bar buckling initiation strain for \( s/d_b \) ratios of 8. White indicates strains where buckling is not expected for configurations with hoops and ties spaced no more than 8 times the longitudinal bar diameter (generally less than tensile strain of 0.01). By having an estimate of the longitudinal reinforcement strains for displacement in one direction, the designer has additional information to determine if the transverse hoop spacing specified for the boundary element in compression in the opposite direction of loading is satisfactory to prevent longitudinal bar buckling.

The upcoming release of ACI 318-14 modifies several requirements for both SBE and OBE configurations. The intent of current codes is to provide 90% confidence of non-collapse for MCE-level shaking; however, the confinement trigger (Equation 2.2) in ACI 318-11 is based on 50% confidence of not exceeding the concrete crushing limit (set to 0.003 strain) in the Design Basis Earthquake. The 2014 iteration of the concrete building code (ACI 318-14), applies a factor of 1.5 to the drift ratio in the denominator of ACI 318-11 Equation (21-8) to be more consistent with the building code performance intent (Equation 8.7). The factor is based on MCE spectra being approximately 1.5 times DBE spectra (ASCE 7-10). The intent of using the higher displacement estimate would be to relax detailing requirements only when the displacement estimate, taken as 1.5 times the design displacement, has a low probability of being exceeded.

\[
c = \frac{l_w}{600(1.5 \delta_u / h_w)}
\]  \hspace{1cm} 8.7
Figure 8.13 illustrates the difference in equations from the 2011 code to the 2014 code. Under the 2014 relationship, there is a reduced field of neutral axis depth and design roof drift combinations that allow for the wall boundaries to be designed as OBEs. The new trigger limit essentially follows the drift-neutral axis depth contour associated with 0.002 compressive strain (versus the 0.003 contour for ACI 318-11, also see Figure 8.10). In addition to the restriction of cases where OBEs may be used, the maximum transverse reinforcement spacing allowed for OBEs is changed from an absolute value of 8 inches to a ratio of 8 times the longitudinal reinforcement diameter ($8d_b$) and 8 inches. However, if section yielding is detected, the maximum transverse reinforcement spacing allowed for OBEs is limited to 6 times the longitudinal reinforcement diameter ($6d_b$) and 6 inches.

Brittle failures with precipitous strength loss associated with longitudinal bar buckling have been observed in past wall experiments (e.g., Thomsen and Wallace, 2004; flanged wall TW1) as well as in the prism tests conducted at UCLA (specimen 8NT-12), which exhibits the potential for strength loss if longitudinal bars buckle on load reversal. To remain consistent with the above change regarding design
roof drifts and code intent to prevent collapse, the drift contours, as plotted in Figure 8.12, were factored by 1.5 in Figure 8.14 and Equation 8.8 to also account for the potential hazard associated with bar buckling.

\[ \varepsilon_{st} = 2 \left( 1.5 \times \frac{\delta_e}{h_w} \right) \left( 1 - \frac{c}{l_w} \right) \]

8.8

The change effectively results in larger longitudinal bar strains that must be accounted for with tighter spacing of transverse reinforcement, and limits the cases where the maximum spacing of transverse reinforcement of 8 times the longitudinal bar diameter may be used. The buckling initiation strain fields have been updated from Rodriguez et al. (1999) to include the results from the UCLA experiments presented in Chapter 7. The revised figure illustrates that for low drift demands (e.g., all walls with less than 0.5% drift ratio), spacing of transverse reinforcement less than 8 times the longitudinal reinforcement diameter is not necessary to prevent bar buckling upon load reversal. It is recommended that additional tests be performed investigating buckling of longitudinal reinforcement embedded in concrete at spacing less than \(8d_b\) and 8 inches.

Figure 8.14 Revised design chart for bar buckling including 1.5 factor for drift and prism test results.
Based on the results and observations presented above, it is recommended to amend the process of determining the required spacing of transverse reinforcement in current ACI 318 provisions. Requirements for boundary transverse reinforcement detailing depends on the neutral axis depth associated with the boundary element considered in compression. An additional check should be included to determine if the vertical spacing of transverse reinforcement as required by either OBE or SBE provisions is sufficient to delay the onset of buckling of longitudinal reinforcement. If the calculated longitudinal reinforcement axial strain from reverse loading exceeds the compression-reloading strain capacity for the $s/d_b$ ratio provided after following OBE or SBE detailing provisions, transverse reinforcement spacing should be further reduced. Figure 8.15 provides a simplified summary of the proposed additional boundary spacing requirement.

![Figure 8.15 Summary of proposed additional check for wall boundary transverse reinforcement detailing.](image-url)
The analyses and experiments presented in Chapters 3, 4 and 7 were tasked with identifying and characterizing damage mechanisms at the boundaries of thin reinforced concrete structural walls, with the underlying objective of evaluating the efficacy of current U.S. seismic code requirements. Many recently constructed (post-1985) mid-rise structures in Chile, whose design was adapted from ACI 318-95, suffered extensive wall boundary damage in the 2010 Maule earthquake. A total of 27 walls from seven structures in three Chilean cities were selected for detailed investigation, with reported conditions ranging from essentially undamaged to extensive damage, characterized by crushed boundaries and webs. Neutral axis depth assessments of walls that suffered concrete crushing at the wall boundary typically resulted in depths more than twice the ACI 318 value that requires special boundary elements (well-detailed wall boundary).

The large unsupported lengths of boundary longitudinal reinforcement common in Chilean structural walls also raised concerns whether buckling bars or crushing of concrete initiated failure in these walls and whether the same type of damage could occur in boundary elements designed following modern U.S. design provisions. An approach based on estimated roof drift history and reinforcement strain histories was implemented to assess the likelihood of longitudinal bar buckling at wall boundaries for the 27 walls investigated, and revealed that bar strains in many walls approached or surpassed the strains associated with buckling of isolated reinforcing bars. However, due to the long duration of the earthquake, and because reinforcing bars that indicated likelihood of buckling were often located at wall boundaries that also were susceptible to concrete crushing (due to lack of confinement), it was unclear whether concrete crushing or rebar buckling were the initiators of wall damage, or symptomatic of each other.
Therefore, in addition to analyses of in-situ walls, a series of eight reverse-cyclic reinforced concrete prism (column) tests were conducted at UCLA. These walls represented thin wall boundaries satisfying ACI 318-11 provisions 21.9.6.5 (similar to typical detailing found in Chilean walls), and were tested with the objective of inducing buckling of longitudinal reinforcement before compromising the concrete cover due to crushing. Spacing of transverse reinforcement and applied axial strain history were selected as primary test variables to identify potential areas where ACI 318 section 21.9.6 code provisions could be improved in ACI 318-14. Damage initiated by buckling of longitudinal reinforcement occurred in prisms representing boundaries with spacing of transverse reinforcement of 8” \( (s/d_b = 10.7) \) and 6” \( (s/d_b = 8) \); both spacing values are permitted by ACI 318-11, which does not include a spacing limit for boundary reinforcement relative to longitudinal bar diameter. Applied axial compressive strains in the last two specimens (45WT-45, 45WT-I) exceeded 0.003 to investigate the role of higher compression strains on longitudinal bar stability and variability in boundary element concrete compressive strain limits prior to strength loss. Both specimens had 4.5” spacing of transverse reinforcement \( (s/d_b = 6) \), which was smaller than the previous six tests. Gradual, out of plane instability developed in 45WT-45, where average compression strain was limited to 0.0045; however, brittle failure with abrupt strength loss was observed in 45WT-I as the compressive strain approached 0.5%.

Finally, a parametric study of expressions relating drift, axial load, reinforcement ratios, and curvature was performed in Chapter 8 to identify wall configurations that might subject boundary longitudinal reinforcement to large axial strain reversals during earthquake shaking. The investigation showed that asymmetric walls with flanges with large drift demands can generate large axial strain demands in longitudinal reinforcement, and recommendations were proposed to be included in ACI 318 that could address buckling upon drift reversal.
9.1 Conclusions

Based on the above summary, the following conclusions can be drawn from the presented research:

1) Analyses of Chilean mid-rise buildings with reinforced concrete structural walls indicate that the majority of the damaged walls investigated would have required special boundary element detailing per ACI 318-11 Section 21.9.6.2 and 21.9.6.4. Estimates of axial loads and seismic roof drifts for most walls would have yielded neutral axis depths that exceed the critical depth established in ACI 318-11 Equation 21-8 that triggers special boundary elements in structural walls, often by a factor of two or more. With respect to U.S. design, the 2010 $M_w$ 8.8 Maule earthquake could be considered a maximum considered earthquake (MCE) described by ASCE 7-10. The severe damage suffered in Chilean buildings and the excessive neutral axis depths relative to the SBE trigger limit of ACI 318-11—particularly in walls with neutral axis lengths in excess of approximately twice the trigger limit—indicates that review of the trigger equation in Section 21.9.6.2 is warranted. In current US codes, the intent is to provide 90% confidence of non-collapse for MCE shaking. In contrast, the current ACI confinement trigger (ACI 318-11 Equation 21-8, also Equation 2.2 in this document) is based on 50% confidence of not exceeding the concrete crushing limit in the Design Basis Earthquake (which is much lower shaking intensity than the MCE). To address this issue, it is suggested to adjust ACI 318-11 Equation (21-8), to be more consistent with the building code performance intent. In the 2014 version of the concrete building code (ACI 318-14), a factor of 1.5 has been applied to the drift ratio in the denominator of ACI 318-11 Equation (21-8). The factor is based on MCE spectra being approximately 1.5 times DBE spectra (ASCE 7-10). The intent of using the higher displacement estimate would be to relax detailing requirements only when the displacement estimate, taken as 1.5 times the design displacement, has a low probability of being exceeded.
2) Brittle concrete crushing failure was observed in specimen 45WT-I as the compressive strain reached approximately 0.5%, indicating the ultimate compressive strain capacity was only marginally above 0.45%, or a limit equal to 1.5 times the commonly used unconfined concrete strain limit of 0.3% (for the Design Earthquake level shaking). The transverse hoop spacing in 45WT-I was less than the maximum allowed for non-special (ordinary) boundary elements, but did not meet the requirements of 21.9.6.4 to satisfy special boundary detailing. As with the results from the analysis of walls in Chilean buildings, test results highlight potential uncertainties associated with the displacement-based design approach of ACI 318-11 21.9.6.2. Although the test results indicate that current provisions may have sufficient conservatism to address the uncertainty in design displacement, other factors, including variation in wall axial load, are often not evaluated. Given the brittle failures and abrupt strength loss noted in several tests, the added conservatism associated with using $1.5(\delta_u/h_w)$ in the denominator of ACI 318-11 Equation 21-8 appears justified.

3) ACI 318-11 Section 21.9.6.4 limits spacing of boundary transverse reinforcement to a maximum of six times the diameter of the boundary longitudinal reinforcement, independent of wall demands and wall configuration, to provide restraint against longitudinal bar buckling. However, in wall configurations where longitudinal reinforcement experiences large axial strain reversals from tension to compression (e.g., the non-flanged end of a T- or L-shaped wall) at the appropriate target drift, the strain reversals may be sufficiently large to initiate buckling of longitudinal reinforcement, either in special boundary elements ($s < 6d_b$) or ordinary boundary elements ($s < 8''$). Results presented indicate that, if section analysis produces extreme fiber tension strains in excess of 4%, six times the longitudinal bar diameter may be too large a spacing to prevent buckling of longitudinal reinforcement. The relationship between configuration/spacing of transverse reinforcement and longitudinal bar diameter ($s/d_b$) is currently
not a consideration for the deformation capacity of walls where special boundary elements per ACI 318-11 Section 21.9.6.4 are required. Provisions to address this issue could be added, as discussed in Chapter 8: a simplified equation like Equation 8.8 that relates tension strain to drift demand and neutral axis depth can be used to determine if compression reloading necessitates tighter spacing than $6d_b$ to prevent bar buckling. Given the variation in reloading strain required to produce rebar buckling in tests reported in the literature, additional tests should be performed on configurations with $s/d_b$ less than 6 to confirm the relationship between buckling of reinforcement, compressive reloading strain, and $s/d_b$ ratios.

4) Chilean wall investigations and laboratory tests of concrete prisms indicate that current detailing provisions of ACI 318-11 Section 21.9.6.5 may produce wall sections that are susceptible to bar buckling and abrupt strength loss associated with brittle failure. Flanged walls (i.e. T- or L-shaped walls) have attributes that require special attention, i.e., large web end (opposite the flange) longitudinal bar tensile strains when the flange is loaded in compression and large compression strains (and large neutral axis depths) when the web (opposite the flange) is compressed. Permitting eight inches as the maximum spacing of transverse reinforcement allows for very large spacing to bar diameter ratios, especially where small diameter boundary longitudinal reinforcement is used, and aggravates the potential for wall damage caused by bar buckling. Specimens 8NT-2 and 8WT-2, both satisfying the Section 21.9.6.5 spacing limit of transverse reinforcement, exhibited initiation of damage due to buckling of longitudinal bars leading to strength loss (via loss of concrete cover).

5) As with the SBE conclusions presented in item 3) above, an approach was presented in Chapter 8 to incorporate checks on longitudinal reinforcement buckling into the ACI 318 code section for boundary element detailing. Reloading strains greater than 2% may initiate buckling of
longitudinal reinforcement if spacing of boundary transverse reinforcement is extended to the maximum permitted by ACI 318-11 for OBEs, and may be possible in SBEs with reloading strains greater than 4%. In addition to the current limits on spacing of transverse reinforcement for SBEs and OBEs in ACI 318-11 Sections 21.9.6.4 and 21.9.6.5, respectively, a strain limit for boundary longitudinal reinforcement should be considered using information that is readily available during design (i.e., neutral axis depth or depths).

6) For a 6” by 15” cross section with one inch of clear cover over longitudinal reinforcing bars, bar buckling pushing off cover concrete represented a significant loss in section compression capacity and led to lateral instability on subsequent loading cycles. The presence of a crosstie did not seem to impact the performance of longitudinal reinforcement placed at the centerline of the longer side of the test specimen, although sudden, brittle through-section failure (following bar buckling) was observed in the 8NT-2 specimen, which did not have a crosstie.

7) Several of the laboratory specimens exhibited out-of-plane instability upon reloading into compression from very large tension strains (3% and 4.5% tension strain excursions). These experiments provide additional data for modeling the relationship between strain history of longitudinal reinforcement and out-of-plane stability of thin walls. The maximum stable compression reloading cycles in four experiments, where out-of-plane movement limited specimen stability, approached strain levels predicted by relations developed by Paulay and Priestley (1993) and Chai and Elayer (1999). The test results also indicate that buckling of longitudinal reinforcement led to lateral instability in subsequent cycles; this observation indicates that assessing what initiates section or member failure (strength loss) may not be apparent by inspection of walls following moderate- to long-duration shaking.
APPENDIX A   CHILEAN WALL ANALYSES

This appendix provides additional information on the Chilean wall analyses not included in Chapters 3 and 4. This includes maps showing building orientations and locations relative to the recording stations used to generate response spectra; plans, elevations, and cross-sections generated from design documents obtained in the reconnaissance efforts; and moment-curvature analyses for the selected walls.

![Figure A.1 Concepción recording station and candidate buildings (Google Earth, 2014).](image-url)
Figure A.2 Viña del Mar recording station and candidate building (Google Earth, 2014).

Figure A.3 Santiago recording stations and candidate buildings (Google Earth, 2014).
Figure A.4 Critical section for wall 1.1

Figure A.5 Critical elevation views for Wall 1.1
Figure A.6 Moment-curvature for Wall 1.1 (flanged wall, flange compression in negative direction).

Figure A.7 Critical section for Wall 1.2.
Figure A.8 Critical elevation views for Wall 1.2.

Figure A.9 Moment-curvature for Wall 1.2 (flanged wall, flange compression in positive direction).
Figure A.10 Critical section for Wall 1.3.

Figure A.11 Critical elevation views for Wall 1.3.
Figure A.12 Moment-curvature for Wall 1.3 (flanged wall, flange compression in negative direction).

Figure A.13 Critical section for Wall 1.4.
Figure A.14 Moment-curvature for Wall 1.4 (flanged wall, flange compression in positive direction).
Figure A.15 Critical section for Wall 1.5.

Figure A.16 Moment-curvature for Wall 1.5 (flanged wall, flange compression in negative direction).
Figure A.17 Critical section for Wall 1.6.

Figure A.18 Moment-curvature for Wall 1.6 (flanged wall, flange compression in positive direction).
Figure A.19 Critical section and elevation for Wall 2.1.

Figure A.20 Moment-curvature for Wall 2.1.
Figure A.21 Critical section for Wall 2.2.

Figure A.22 Critical elevation views for Wall 2.2.
Figure A.23 Moment-curvature for Wall 2.2.

Figure A.24 Critical section for Wall 2.3.
Figure A.25 Critical elevation views for Wall 2.3.

Figure A.26 Moment-curvature for Wall 2.3.
Figure A.27 Critical section and elevation for Wall 3.1

Figure A.28 Detailed cross-section of Wall 3.1
Figure A.29 Moment-curvature for Wall 3.1.

Figure A.30 Critical section for Wall 3.2.
Figure A.31 Critical elevation views for Wall 3.2.

Figure A.32 Detailed cross-section for Wall 3.2.
Figure A.33 Moment-curvature for Wall 3.2.

Figure A.34 Critical section and elevation view for Wall 4.1
Figure A.35 Moment-curvature for Wall 4.1.

Figure A.36 Critical section for Wall 4.2.
Figure A.37 Critical elevation views for Wall 4.2

Figure A.38 Moment-curvature for Wall 4.2.
Figure A.39 Critical section and elevation for Wall 4.3

Figure A.40 Moment-curvature for Wall 4.3.
Figure A.41 Critical section for Wall 4.4

Figure A.42 Critical elevation views for Wall 4.4.
Figure A.43 Moment-curvature for Wall 4.4.

Figure A.44 Critical section for Wall 4.5.
Figure A.45 Critical elevation views for Wall 4.5.

Figure A.46 Moment-curvature for Wall 4.5.
Figure A.47 Critical section for Wall 4.6.

Figure A.48 Critical elevation views for Wall 4.6.
Figure A.49 Moment-curvature for Wall 4.6.

Figure A.50 Critical section and elevation for Wall 5.1
Figure A.51 Moment-curvature for Wall 5.1.

Figure A.52 Critical section and elevation view for Wall 5.2
Figure A.53 Moment-curvature for Wall 5.2.

Figure A.54 Critical section for Wall 5.3
Figure A.55 Critical elevation views for Wall 5.3

Figure A.56 Moment-curvature for Wall 5.3.
Figure A.57 Critical wall section for Wall 5.4

Figure A.58 Critical elevation views for Wall 5.4
Figure A.59 Moment-curvature for Wall 5.4.

Figure A.60 Critical section for Wall 6.1.
Figure A.61 Critical elevation views for Wall 6.1.

Figure A.62 Moment-curvature for Wall 6.1.
Figure A.63 Critical section for Wall 6.2.

Figure A.64 Critical elevation views for Wall 6.2.
Figure A.65 Moment-curvature for Wall 6.2.

Figure A.66 Critical section for Wall 6.3.
Figure A.67 Critical elevation views for Wall 6.3.

Figure A.68 Moment-curvature for Wall 6.3.
Figure A.69 Critical section and elevation for Wall 7.1.

Figure A.70 Moment-curvature for Wall 7.1.
Figure A.71 Critical section for Wall 7.2.

Figure A.72 Critical elevation views for Wall 7.2
Figure A.73 Moment-curvature for Wall 7.2.

Figure A.74 Critical section for Wall 7.3.
Figure A.75 Critical elevation views for Wall 7.3.

Figure A.76 Moment-curvature for Wall 7.3.
This section provides additional information regarding the eight prism tests performed at UCLA. Data include individual stress-strain curves of concrete cylinder compression tests, specimen longitudinal reinforcement mill certifications, and force-strain plots for each mounted sensor from each experiment. Figures B.1 through B.10 present material properties of the concrete and longitudinal reinforcement used in the eight specimens.

Figure B.1 Concrete cylinder compression tests for Specimen 8WT-2.
Figure B.2 Concrete cylinder compression tests for Specimen 8NT-2.

Figure B.3 Concrete cylinder compression tests for Specimen 6NT-2.
Figure B.4 Concrete cylinder compression tests for Specimen 6WT-2.

Figure B.5 Concrete cylinder compression tests for Specimen 8WT-3.
Figure B.6 Concrete cylinder compression tests for Specimen 6WT-3.

Figure B.7 Concrete cylinder compression tests for Specimen 45WT-45.
Figure B.8 Concrete cylinder compression tests for Specimen 45WT-I.

<table>
<thead>
<tr>
<th>Event</th>
<th>Cylinder No.</th>
<th>Cylinder Test Group</th>
<th>Cylinder Test Date</th>
<th>Days after Pour</th>
<th>Days from Experiment</th>
<th>$f'_c$ (ksi)</th>
<th>$\varepsilon_{peak}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.5wk</td>
<td>1</td>
<td>1</td>
<td>10/8/2012</td>
<td>18</td>
<td>-</td>
<td>3.233</td>
<td>0.00190</td>
</tr>
<tr>
<td>2.5wk</td>
<td>2</td>
<td>1</td>
<td>10/8/2012</td>
<td>18</td>
<td>-</td>
<td>3.184</td>
<td>0.00170</td>
</tr>
<tr>
<td>4wk</td>
<td>3</td>
<td>2</td>
<td>10/19/2012</td>
<td>29</td>
<td>-</td>
<td>3.577</td>
<td>0.00199</td>
</tr>
<tr>
<td>4wk</td>
<td>4</td>
<td>2</td>
<td>10/19/2012</td>
<td>29</td>
<td>-</td>
<td>3.786</td>
<td>0.00191</td>
</tr>
<tr>
<td>8WT-2</td>
<td>5</td>
<td>3</td>
<td>10/31/2012</td>
<td>41</td>
<td>-1</td>
<td>3.864</td>
<td>0.00245</td>
</tr>
<tr>
<td>8WT-2</td>
<td>6</td>
<td>3</td>
<td>10/31/2012</td>
<td>41</td>
<td>-1</td>
<td>3.808</td>
<td>0.00208</td>
</tr>
<tr>
<td>8WT-2</td>
<td>7</td>
<td>3</td>
<td>10/31/2012</td>
<td>41</td>
<td>-1</td>
<td>3.926</td>
<td>0.00224</td>
</tr>
<tr>
<td>8WT-2</td>
<td>8</td>
<td>3</td>
<td>10/31/2012</td>
<td>41</td>
<td>-1</td>
<td>3.874</td>
<td>0.00233</td>
</tr>
<tr>
<td>8WT-2 Average</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.868</td>
<td>0.00227</td>
</tr>
<tr>
<td>8NT-2</td>
<td>9</td>
<td>4</td>
<td>11/28/2012</td>
<td>69</td>
<td>+2</td>
<td>4.008</td>
<td>0.00222</td>
</tr>
<tr>
<td>8NT-2</td>
<td>10</td>
<td>4</td>
<td>11/28/2012</td>
<td>69</td>
<td>+2</td>
<td>3.996</td>
<td>0.00208</td>
</tr>
<tr>
<td>8NT-2</td>
<td>11</td>
<td>4</td>
<td>11/28/2012</td>
<td>69</td>
<td>+2</td>
<td>4.156</td>
<td>0.00231</td>
</tr>
<tr>
<td>8NT-2</td>
<td>12</td>
<td>4</td>
<td>11/28/2012</td>
<td>69</td>
<td>+2</td>
<td>4.467</td>
<td>0.00294</td>
</tr>
<tr>
<td>8NT-2 Average</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4.157</td>
<td>0.00239</td>
</tr>
</tbody>
</table>
Table B.1 continued

<table>
<thead>
<tr>
<th>Event</th>
<th>Cylinder No.</th>
<th>Cylinder Test Group</th>
<th>Cylinder Test Date</th>
<th>Days after Pour</th>
<th>Days from Experiment</th>
<th>$f'_c$ (ksi)</th>
<th>$\varepsilon_{peak}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>6NT-2</td>
<td>13</td>
<td>5</td>
<td>12/14/2012</td>
<td>85</td>
<td>+1</td>
<td>3.524</td>
<td>0.00157</td>
</tr>
<tr>
<td>6NT-2</td>
<td>14</td>
<td>5</td>
<td>12/14/2012</td>
<td>85</td>
<td>+1</td>
<td>3.789</td>
<td>0.00183</td>
</tr>
<tr>
<td><strong>6NT-2 Average</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>3.657</strong></td>
<td><strong>0.00170</strong></td>
</tr>
<tr>
<td>6WT-2</td>
<td>15</td>
<td>6</td>
<td>2/4/2013</td>
<td>137</td>
<td>+5</td>
<td>4.674</td>
<td>0.00270</td>
</tr>
<tr>
<td>6WT-2</td>
<td>16</td>
<td>6</td>
<td>2/4/2013</td>
<td>137</td>
<td>+5</td>
<td>3.774</td>
<td>0.00186</td>
</tr>
<tr>
<td>6WT-2</td>
<td>17</td>
<td>6</td>
<td>2/4/2013</td>
<td>137</td>
<td>+5</td>
<td>4.264</td>
<td>0.00241</td>
</tr>
<tr>
<td>6WT-2</td>
<td>18</td>
<td>6</td>
<td>2/4/2013</td>
<td>137</td>
<td>+5</td>
<td>4.354</td>
<td>0.00251</td>
</tr>
<tr>
<td><strong>6WT-2 Average</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>4.267</strong></td>
<td><strong>0.00237</strong></td>
</tr>
<tr>
<td><strong>Group I Specimens Average</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>4.010</strong></td>
<td><strong>0.00223</strong></td>
</tr>
</tbody>
</table>

Table B.2 Group II concrete cylinder test summary

<table>
<thead>
<tr>
<th>Event</th>
<th>Cylinder No.</th>
<th>Cyl Test Group</th>
<th>Date</th>
<th>Days after Pour</th>
<th>Days from Experiment</th>
<th>$f'_c$ (ksi)</th>
<th>$\varepsilon_{peak}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2wk</td>
<td>21</td>
<td>7</td>
<td>6/4/2013</td>
<td>14</td>
<td>--</td>
<td>2.861</td>
<td>0.00151</td>
</tr>
<tr>
<td>2wk</td>
<td>22</td>
<td>7</td>
<td>6/4/2013</td>
<td>14</td>
<td>--</td>
<td>2.846</td>
<td>0.00148</td>
</tr>
<tr>
<td>2wk</td>
<td>23</td>
<td>7</td>
<td>6/4/2013</td>
<td>14</td>
<td>--</td>
<td>2.908</td>
<td>0.00183</td>
</tr>
<tr>
<td>6wk</td>
<td>24</td>
<td>8</td>
<td>7/1/2013</td>
<td>41</td>
<td>--</td>
<td>3.739</td>
<td>0.00223</td>
</tr>
<tr>
<td>6wk</td>
<td>25</td>
<td>8</td>
<td>7/1/2013</td>
<td>41</td>
<td>--</td>
<td>3.771</td>
<td>0.00240</td>
</tr>
<tr>
<td>8WT-3</td>
<td>26</td>
<td>9</td>
<td>7/10/2013</td>
<td>50</td>
<td>+1</td>
<td>3.846</td>
<td>0.00225</td>
</tr>
<tr>
<td>8WT-3</td>
<td>27</td>
<td>9</td>
<td>7/10/2013</td>
<td>50</td>
<td>+1</td>
<td>3.941</td>
<td>0.00222</td>
</tr>
<tr>
<td>8WT-3</td>
<td>28</td>
<td>9</td>
<td>7/10/2013</td>
<td>50</td>
<td>+1</td>
<td>3.716</td>
<td>0.00147</td>
</tr>
<tr>
<td>8WT-3</td>
<td>29</td>
<td>9</td>
<td>7/10/2013</td>
<td>50</td>
<td>+1</td>
<td>3.462</td>
<td>0.00157</td>
</tr>
<tr>
<td><strong>8WT-3 Average</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>3.741</strong></td>
<td><strong>0.00188</strong></td>
</tr>
<tr>
<td>6WT-3</td>
<td>30</td>
<td>10</td>
<td>7/19/2013</td>
<td>59</td>
<td>+1</td>
<td>3.737</td>
<td>0.00170</td>
</tr>
<tr>
<td>6WT-3</td>
<td>31</td>
<td>10</td>
<td>7/19/2013</td>
<td>59</td>
<td>+1</td>
<td>3.934</td>
<td>0.00237</td>
</tr>
<tr>
<td>6WT-3</td>
<td>32</td>
<td>10</td>
<td>7/19/2013</td>
<td>59</td>
<td>+1</td>
<td>3.572</td>
<td>0.00195</td>
</tr>
<tr>
<td>6WT-3</td>
<td>33</td>
<td>10</td>
<td>7/19/2013</td>
<td>59</td>
<td>+1</td>
<td>3.808</td>
<td>0.00180</td>
</tr>
<tr>
<td><strong>6WT-3 Average</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>3.763</strong></td>
<td><strong>0.00195</strong></td>
</tr>
<tr>
<td>Event</td>
<td>Cylinder No.</td>
<td>Cyl Test Group</td>
<td>Date</td>
<td>Days after Pour</td>
<td>Days from Experiment</td>
<td>$f'_c$ (ksi)</td>
<td>$\varepsilon_{peak}$</td>
</tr>
<tr>
<td>--------</td>
<td>--------------</td>
<td>----------------</td>
<td>------------</td>
<td>-----------------</td>
<td>----------------------</td>
<td>-------------</td>
<td>------------------------</td>
</tr>
<tr>
<td>4.5WT-4.5</td>
<td>34</td>
<td>11</td>
<td>7/25/2013</td>
<td>65</td>
<td>0</td>
<td>3.888</td>
<td>0.00184</td>
</tr>
<tr>
<td>4.5WT-4.5</td>
<td>35</td>
<td>11</td>
<td>7/25/2013</td>
<td>65</td>
<td>0</td>
<td>3.95</td>
<td>0.00259</td>
</tr>
<tr>
<td>4.5WT-4.5</td>
<td>36</td>
<td>11</td>
<td>7/25/2013</td>
<td>65</td>
<td>0</td>
<td>3.664</td>
<td>0.00156</td>
</tr>
<tr>
<td>4.5WT-4.5</td>
<td>37</td>
<td>11</td>
<td>7/25/2013</td>
<td>65</td>
<td>0</td>
<td>3.949</td>
<td>0.00203</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.863</td>
<td>0.00201</td>
</tr>
<tr>
<td>4.5WT-I</td>
<td>38</td>
<td>12</td>
<td>8/2/2013</td>
<td>73</td>
<td>+1</td>
<td>3.892</td>
<td>0.00228</td>
</tr>
<tr>
<td>4.5WT-I</td>
<td>39</td>
<td>12</td>
<td>8/2/2013</td>
<td>73</td>
<td>+1</td>
<td>3.952</td>
<td>0.00187</td>
</tr>
<tr>
<td>4.5WT-I</td>
<td>40</td>
<td>12</td>
<td>8/2/2013</td>
<td>73</td>
<td>+1</td>
<td>3.870</td>
<td>0.00227</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.905</td>
<td>0.00214</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.812</td>
<td>0.00198</td>
</tr>
</tbody>
</table>

45WT-I Average  | 3.905       | 0.00214 |

Group II Specimens Average  | 3.812       | 0.00198 |
Figure B.9 Mill certification for Group I specimen longitudinal reinforcement.
Table of Physical and Chemical Test Report:

<table>
<thead>
<tr>
<th>Mill Co.</th>
<th>Heat #</th>
<th>Size</th>
<th>Grade</th>
<th>C</th>
<th>Mn</th>
<th>P</th>
<th>Si</th>
<th>Cr</th>
<th>Ni</th>
<th>Cu</th>
<th>Mo</th>
<th>V</th>
<th>CE</th>
<th>Yield</th>
<th>Tensile</th>
</tr>
</thead>
<tbody>
<tr>
<td>CMC Steel</td>
<td>4001882</td>
<td>4/15</td>
<td>A766</td>
<td>.27</td>
<td>1.08</td>
<td>.010</td>
<td>.225</td>
<td>.060</td>
<td>.080</td>
<td>.000</td>
<td>.002</td>
<td>.470</td>
<td>69,200</td>
<td>91,600</td>
<td>170</td>
</tr>
<tr>
<td>CMC Steel</td>
<td>4001906</td>
<td>4/16</td>
<td>A766</td>
<td>.26</td>
<td>1.15</td>
<td>.008</td>
<td>.029</td>
<td>.265</td>
<td>.110</td>
<td>.080</td>
<td>.259</td>
<td>.020</td>
<td>.004</td>
<td>.080</td>
<td>60,600</td>
</tr>
<tr>
<td>CMC Steel</td>
<td>4001959</td>
<td>4/15</td>
<td>A766</td>
<td>.25</td>
<td>1.05</td>
<td>.007</td>
<td>.028</td>
<td>.210</td>
<td>.140</td>
<td>.120</td>
<td>.228</td>
<td>.020</td>
<td>.002</td>
<td>.460</td>
<td>69,100</td>
</tr>
</tbody>
</table>

The above material described herein has been made and tested in accordance with ASTM A615 and or ASTM A-706. We hereby certify the above data to be correct as contained in our company records. We further certify all manufacturing processes for reinforcing steel, in these materials occurred in the United States of America.

By: [Signature]

Report Date: 8/11/2011

Material: REINFORCING STEEL
Figures B.11 through B.35 plot the axial force versus axial strain measurements for each LVDT mounted on the specimens. For reference, the average strain across all nodes of the LVDT column and the block-to-block of the nearest longitudinal bar as calculated from the strain field generated by the three full-height LVDT columns are also provided.

![Force-axial strain plots for 8WT-2 (0.05% to 0.2% cycles).](image)

**Figure** B.11 Force-axial strain plots for 8WT-2 (0.05% to 0.2% cycles).
Figure B.12 Force-axial strain plots for SWT-2 (0.3% to 1.0% cycles).
Figure B.13 Force-axial strain plots for 8WT-2 (1.5% to 4.5% cycles).
Figure B.14 Force-axial strain plots for SNT-2 (0.05% to 0.2% cycles).
Figure B.15 Force-axial strain plots for 8NT-2 (0.3% to 1.0% cycles).
Figure B.16 Force-axial strain plots for 8NT-2 (1.5% to 3.0% cycles).
Figure B.17 Force-axial strain plots for 6NT-2 (0.05% to 0.2% cycles).
Figure B.18 Force-axial strain plots for 6NT-2 (0.3% to 1.0% cycles).
Figure B.19 Force-axial strain plots for 6NT-2 (1.5% to 4.5% cycles).
Figure B.20 Force-axial strain plots for 6WT-2 (0.05% to 0.2% cycles).
Figure B.21 Force-axial strain plots for 6WT-2 (0.3\% to 1.0\% cycles).
Figure B.22 Force-axial strain plots for 6WT-2 (1.5% to 4.5% cycles).
Figure B.23 Force-axial strain plots for SWT-3 (0.05% to 0.2% cycles).
Figure B.24 Force-axial strain plots for SWT-3 (0.3% to 1.0% cycles).
Figure B.25 Force-axial strain plots for 8WT-3 (1.5% to 4.5% cycles).
Figure B.26 Force-axial strain plots for 6WT-3 (0.05% to 0.2% cycles).
Figure B.27 Force-axial strain plots for 6WT-3 (0.3% to 1.0% cycles).
Figure B.28 Force-axial strain plots for 6WT-3 (1.5% to 4.5% cycles).
Figure B.29 Force-axial strain plots for 45WT-45 (0.05% to 0.2% cycles).
Figure B.30 Force-axial strain plots for 45WT-45 (0.3% to 1.0% cycles).
Figure B.31 Force-axial strain plots for 45WT-45 (1.5% to 4.5% cycles).
Figure B.32 Force-axial strain plots for 45WT-I (0.05% to 0.2% cycles).
Figure B.33 Force-axial strain plots for 45WT-I (0.3% to 1.0% cycles).
Figure B.34 Force-axial strain plots for 45WT-I (1.5% to 2.0% cycles).
REFERENCES

1. ACI Committee 318 (1995), “Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95),” American Concrete Institute, Farmington Hills, Michigan, 369 pp.

2. ACI Committee 318 (2005), “Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05),” American Concrete Institute, Farmington Hills, Michigan, 430 pp.


4. ACI Committee 318 (2014), “Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14),” American Concrete Institute, Farmington Hills, Michigan


293


