Materials modeling for unbonded post-tensioned precast concrete walls

Stacy Marie Horan
Lehigh University

Follow this and additional works at: http://preserve.lehigh.edu/etd

Recommended Citation
Horan, Stacy Marie
Materials Modeling for Unbonded Post-Tensioned Precast Concrete Walls

January 2003
MATERIALS MODELING FOR UNBONDED POST-TENSIONED PRECAST CONCRETE WALLS

by

Stacy Marie Horan

A Thesis
Presented to the Graduate and Research Committee
of Lehigh University
in Candidacy for the Degree of
Master of Science
In
Department of Civil and Environmental Engineering

Lehigh University
Bethlehem, Pennsylvania
December 4, 2002
This thesis is accepted and approved in partial fulfillment of the requirements for the Master of Science.

Dec. 4, 2002
Date

Dr. Richard Sause

Dr. Arup SenGupta
Department of Civil and Environmental Engineering
ACKNOWLEDGEMENTS

This research was funded by the Precast/Prestressed Concrete Institute (PCI) and the Pennsylvania Infrastructure Technology Alliance (PITA).

The author would like to thank Felipe Perez, the Ph.D. doctoral candidate with whom she worked. The author would also like to thank Dr. Richard Sause, the director of the ATLSS Research Center at Lehigh University and the advisor for this project, for his guidance and commitment.

Furthermore, the author would like to thank Mr. John Hoffner, Mr. Gene Matlock, and the rest of the ATLSS Lab and FRITZ Lab personnel for their tremendous assistance during experimental testing. She would also like to thank Dr. Stephen Pessiki, Mr. Ian Hodgson, Mr. Carl Bowman, and her friends and family for their help and support.
TABLE OF CONTENTS

LIST OF TABLES vi
LIST OF FIGURES vii
ABSTRACT 1

CHAPTER 1: INTRODUCTION
1.1 OVERVIEW 2
1.2 OBJECTIVES 3
1.3 SCOPE 3
1.4 SUMMARY OF FINDINGS 4
1.5 ORGANIZATION OF THESIS 5
1.6 NOTATION 6

CHAPTER 2: BACKGROUND
2.1 UNBONDED POST-TENSIONED PRECAST CONCRETE CONSTRUCTION 9
2.2 PREVIOUS RESEARCH ON UNBONDED POST-TENSIONED PRECAST CONCRETE WALLS WITH HORIZONTAL JOINTS 11
2.3 BEHAVIOR OF AXIALLY LOADED SPIRALLY CONFINED CONCRETE 14

CHAPTER 3: EXPERIMENTAL PROCEDURES AND RESULTS FOR POST-TENSIONING SYSTEM SPECIMENS
3.1 EXPERIMENTAL PROGRAM
   3.1.1 Test Specimens 23
   3.1.2 Specimens and Test Fixture Details 24
   3.1.3 Instrumentation 26
3.2 EXPERIMENTAL PROCEDURES AND RESULTS
   3.2.1 Experimental Procedures 28
   3.2.2 Experimental Results 28
   3.2.3 Comparison with Expected Material Properties 30

CHAPTER 4: EXPERIMENTAL PROCEDURES AND RESULTS FOR UNCONFINED CONCRETE CYLINDER SPECIMENS
4.1 EXPERIMENTAL PROGRAM
   4.1.1 Test Specimens 46
   4.1.2 Instrumentation 47
4.2 EXPERIMENTAL PROCEDURES AND RESULTS
   4.2.1 Experimental Procedures 48
   4.2.2 Experimental Results 49
   4.2.3 Comparison with Expected Material Properties 53
CHAPTER 5: EXPERIMENTAL PROCEDURES AND RESULTS FOR
THE SPIRALLY CONFINED CONCRETE PANEL SPECIMENS

5.1 EXPERIMENTAL PROGRAM
   5.1.1 Test Specimens 63
   5.1.2 Specimens Details 64
   5.1.3 Instrumentation 65
   5.1.4 Specimens Fabrication 67
   5.1.5 Material Properties 68

5.2 SPIRALLY CONFINED STUB PANEL SPECIMEN
   5.2.1 Experimental Procedure 68
   5.2.2 Experimental Results 69
   5.2.3 Comparison with Expected Material Properties 75

5.3 SPIRALLY CONFINED BUCKLING PANEL SPECIMEN
   5.3.1 Experimental Procedure 75
   5.3.2 Experimental Results 76
   5.3.3 Comparison with Expected Material Properties 78

CHAPTER 6: MATERIAL MODELING AND LATERAL LOAD ANALYSIS
OF AN UNBONDED POST-TENSIONED PRECAST CONCRETE WALL

6.1 MODELING ASSUMPTIONS 125
6.2 ORIGINAL MATERIAL MODELING
   6.2.1 Modeling of Wall Panels 127
   6.2.2 Modeling of Post-Tensioning Steel 128
6.3 IMPROVED MATERIAL MODELING 130
6.4 ANALYSIS RESULTS 131

CHAPTER 7: CONCLUSIONS
7.1 MAIN CONCLUSIONS 145
7.2 OTHER CONCLUSIONS 147

REFERENCES 149

VITA 151
LIST OF TABLES

Table 3.1  The results from testing of the tension coupons. 32
Table 3.2  The results from testing of the post-tensioning system specimens. 32
Table 3.3  Test report data provided by Dywidag Systems Engineering (2001). 33
Table 4.1  The unconfined concrete cylinder test results. 54
Table 4.2  The variables used for Oh's empirical model (Oh 2002). 54
Table 5.1  Load per strain and strain per load for strain gages from the stub panel specimen test: (a) east end gages, (b) west end gages. 80
Table 5.2  Load per strain and strain per load for strain gages from the buckling panel specimen test: (a) east end gages, (b) west end gages. 81
## LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>The plan view of the prototype structure, a six story office building (adapted from Perez 2001).</td>
<td>18</td>
</tr>
<tr>
<td>2.2</td>
<td>An elevation view comparing the full-scale prototype wall and the scaled wall: (a) full-scale wall, (b) scaled wall (adapted from Perez et al. 2003).</td>
<td>19</td>
</tr>
<tr>
<td>2.3</td>
<td>A cross-sectional view of the bottom wall panel comparing the full-scale wall to the scaled wall: (a) full-scale wall, (b) scaled wall (adapted from Perez et al. 2003).</td>
<td>20</td>
</tr>
<tr>
<td>2.4</td>
<td>An elevation view comparing the scaled wall and the test wall: (a) scaled wall, (b) test wall (adapted from Perez et al. 2003).</td>
<td>21</td>
</tr>
<tr>
<td>2.5</td>
<td>The testing arrangement for the test wall: test frame and test wall specimen (adapted from Perez et al. 2003).</td>
<td>22</td>
</tr>
<tr>
<td>3.1</td>
<td>The upper bearing plate, anchor plate, and nut configuration of a typical post-tensioning system.</td>
<td>34</td>
</tr>
<tr>
<td>3.2</td>
<td>The post-tensioning system safety system.</td>
<td>35</td>
</tr>
<tr>
<td>3.3</td>
<td>A typical post-tensioning system specimen testing arrangement.</td>
<td>36</td>
</tr>
<tr>
<td>3.4</td>
<td>The instrumentation scheme for a post-tensioning system specimen.</td>
<td>37</td>
</tr>
<tr>
<td>3.5</td>
<td>The 4 in. (102 mm) linear potentiometers mounted on the bar for a post-tensioning system specimen. Two strain gages are mounted on the bar next to each potentiometer.</td>
<td>38</td>
</tr>
<tr>
<td>3.6</td>
<td>The 1.5 in. (38 mm) linear potentiometers mounted on the bar for a post-tensioning system specimen.</td>
<td>39</td>
</tr>
<tr>
<td>3.7</td>
<td>Stress versus strain plots for the tension coupons: (a) U505 test specimens, (b) N505 test specimens. The arrow points to the end of the curve for each tension coupon.</td>
<td>40</td>
</tr>
<tr>
<td>3.8</td>
<td>Examples of post-tensioning bar fractures: (a) NAnch_3, (b) NAnch_4.</td>
<td>41</td>
</tr>
</tbody>
</table>
Figure 3.9 Nominal stress versus strain plots for five of the six post-tensioning system specimens.

Figure 3.10 Nominal stress versus anchorage seating deformation plots for the post-tensioning system specimens: (a) all four test specimens, (b) test specimens NAnch_3 and NAnch_4 and the average of the two.

Figure 3.11 Nominal stress versus total deformation plots for two different length post-tensioning bars: (a) 36 in. bar, (b) 360 in. bar.

Figure 3.12 Length for which deformation of the post-tensioning bars not caused by anchorage seating is deducted from the potentiometer measurements.

Figure 4.1 The instrumentation scheme for the cylinder test specimens. The strain gages were used for only specimens UCTrial1, UCTrial2, and UCTrial3.

Figure 4.2 A clip gage mounted on a cylinder specimen.

Figure 4.3 A typical cylinder specimen testing arrangement.

Figure 4.4 An example of a cylinder specimen failure (UCTest3).

Figure 4.5 An example of a cylinder specimen failure (UCTest2).

Figure 4.6 Stress versus strain curves for the unconfined concrete cylinder specimen tests and the average of the three tests.

Figure 4.7 The average stress versus strain curve for the unconfined concrete cylinder specimen tests.

Figure 4.8 Comparison of the experimental data for the unconfined concrete cylinders with two versions of the Mander model (Mander et al. 1988).

Figure 4.9 Comparison of the experimental data for the unconfined concrete cylinders with two versions of Oh's empirical model (Oh 2002).

Figure 4.10 The unconfined concrete cylinder stress versus strain curve. The curve is based on Oh's empirical model (2002) with an elastic range of 1% of the peak stress.
Figure 5.1 The reinforcement scheme of the spirally confined test wall panels used by Perez et al. (2003). The stub panel specimen and buckling panel specimen were both removed from the west end of identical panels (adapted from Perez et al. 2003).

Figure 5.2 The confining beams clamped around the stub panel specimen. This same type of configuration was also used for the buckling panel specimen.

Figure 5.3 The confining beam configuration used for both the stub panel specimen and buckling panel specimen tests.

Figure 5.4 The instrumentation configuration for the stub panel specimen. Three confined concrete strain gages are located within the spirals on the lower west end of the specimen.

Figure 5.5 The instrumentation configuration for the buckling panel specimen.

Figure 5.6 The MM strain gage and DEMEC gage configuration for both the stub and buckling panel specimens.

Figure 5.7 The method for acquiring data with the DEMEC gage for the stub panel and buckling panel specimens.

Figure 5.8 The LVDT configuration for the stub panel and buckling panel specimens: (a) attachment to concrete, (b) arrangement of LVDTs.

Figure 5.9 The attachment locations for the three LVDTs measuring out of plane displacement for the buckling panel specimen.

Figure 5.10 The test wall panel formwork used to make three wall panels. The stub panel specimen and buckling panel specimen were cut out of two of these panels.

Figure 5.11 The testing arrangement for the stub panel specimen: test machine with specimen.

Figure 5.12 The testing arrangement for the stub panel specimen: LVDT arrangement.
Figure 5.13  Spalling of the north face and west face of the stub panel specimen.

Figure 5.14  Spalling of the south face and west face of the stub panel specimen.

Figure 5.15  The north face and west face of the stub panel specimen at peak load.

Figure 5.16  The south face and west face of the stub panel specimen at peak load.

Figure 5.17  The north face and west face of the stub panel specimen at failure.

Figure 5.18  The south face and east face of the stub panel specimen at failure.

Figure 5.19  An example of the fractured spirals of the stub panel specimen.

Figure 5.20  The north face of the stub panel specimen after removing the failed materials.

Figure 5.21  The south face of the stub panel specimen after removing the failed materials.

Figure 5.22  The west face of the stub panel specimen after removing the failed materials. The buckling of the longitudinal steel is evident.

Figure 5.23  Load versus strain for the DEMEC strain gages attached to the stub panel specimen.

Figure 5.24  Load versus strain for the MM strain gages attached to the stub panel specimen.

Figure 5.25  Load versus displacement for the LVDTs located on the east face of the stub panel specimen.

Figure 5.26  Load versus displacement for the LVDTs located on the west face of the stub panel specimen.
Figure 5.27 The average load versus displacement for the LVDTs located on the west face and east face of the stub panel specimen as well as the total average of all the LVDTs mounted on the specimen.

Figure 5.28 Comparison of load versus displacement for the average of LVDT-1E and LVDT-2E with the average of all the LVDTs mounted on the stub panel specimen.

Figure 5.29 Comparison of load versus strain for the average of LVDT-1E and LVDT-2E with the average of all the LVDTs mounted on the stub panel specimen.

Figure 5.30 The unconfined concrete stress-strain curve for the stub panel specimen.

Figure 5.31 Load versus displacement for the confined concrete data of the stub panel specimen.

Figure 5.32 Stress versus strain for the confined concrete data of the stub panel specimen.

Figure 5.33 Stress versus strain for the confined concrete data of the stub panel specimen and the smooth curve model of the confined concrete.

Figure 5.34 Stress versus strain of the confined concrete of the stub panel specimen, the smooth curve model of the confined concrete, and the unconfined concrete.

Figure 5.35 Stress versus strain of the confined concrete of the stub panel specimen, the smooth curve model of the confined concrete, the improved smooth curve model of the confined concrete, and the unconfined concrete.

Figure 5.36 Stress versus strain for the confined concrete data of the stub panel specimen and the improved smooth curve model of the confined concrete.

Figure 5.37 Stress versus strain of the confined concrete of the stub panel specimen. This curve is the improved smooth curve model of the confined concrete data.
Figure 5.38  Comparison of the confined concrete stress-strain curve for the stub panel specimen with the Mander model for confined concrete (Mander et al. 1988).

Figure 5.39  The testing arrangement for the buckling panel specimen: test machine with specimen.

Figure 5.40  The testing arrangement for the buckling panel specimen: LVDT arrangement.

Figure 5.41  The north face and west face of the buckling panel specimen at failure.

Figure 5.42  The south face and west face of the buckling panel specimen at failure.

Figure 5.43  An example of the fractured spirals of the buckling panel specimen.

Figure 5.44  Load versus strain for the DEMEC strain gages attached to the buckling panel specimen.

Figure 5.45  Load versus strain for the MM strain gages attached to the buckling panel specimen.

Figure 5.46  Load versus test machine crosshead displacement for the buckling panel specimen.

Figure 5.47  Load versus displacement for the LVDTs mounted to the buckling panel specimen: (a) LVDT-1E and LVDT-9W, (b) LVDT-2E and LVDT-10W, (c) LVDT-3E and LVDT-11W, (d) LVDT-4E and LVDT-12W.

Figure 5.48  Load versus displacement for the LVDTs mounted to the buckling panel specimen: (a) LVDT-5E and LVDT-13W, (b) LVDT-6E and LVDT-14W, (c) LVDT-7E and LVDT-15W, (d) LVDT-8E and LVDT-16W.

Figure 5.49  The east face of the buckling panel specimen at failure. The LVDTs are labeled (LVDT-1E to LVDT-8E) as well as the areas of tension and compression. The mounting angle between LVDT-3E and LVDT-4E fell upon failure.
Figure 5.50 The west face of the buckling panel specimen at failure. The LVDTs are labeled (LVDT-9W to LVDT-16W) as well as the areas of tension and compression. The mounting angle between LVDT-11W and LVDT-12W fell upon failure.

Figure 6.1 The analytical model: (a) test wall specimen, (b) analytical wall model.

Figure 6.1 The analytical model: (a) fiber element segments, (b) slice of a segment, (c) fibers of a slice.

Figure 6.2 The three types of concrete fibers in a typical cross-section of a wall panel near the base of the wall (adapted from Perez et al. 2003).

Figure 6.3 Concrete compressive stress-strain relationships for confined and unconfined concrete used for the original analytical wall model: (a) spirally confined concrete, (b) unconfined concrete.

Figure 6.4 The stress-strain relationship for a typical post-tensioning bar used for the original analytical wall model: (a) typical stress-strain relationship and the trilinear approximation, (b) bilinear stress-strain relationship.

Figure 6.5 The improved stress-strain relationship for a typical post-tensioning bar and the idealized bilinear relationship.

Figure 6.6 The improved stress-strain relationship for unconfined concrete.

Figure 6.7 The improved stress-strain relationship for spirally confined concrete.

Figure 6.8 Base shear versus loading block displacement for the experimental test and the original analytical wall model.

Figure 6.9 Base shear versus loading block displacement for the experimental test and the original analytical wall model: (a) decompression point, (b) first yield point.

Figure 6.10 Base shear versus loading block displacement for the experimental test and the original and improved analytical wall models.
Figure 6.11 Base shear versus loading block displacement for the experimental test and the original and improved analytical wall models: (a) decompression point, (b) first yield point.

Figure 6.12 Base shear versus loading block displacement for the experimental test and the improved analytical wall model.
ABSTRACT

The use of unbonded post-tensioned precast concrete systems as the primary lateral load resisting system in buildings located in regions of high seismicity has potential economic and performance benefits. However, current building codes restrict the use of these systems. Researchers at Lehigh University have been investigating the seismic behavior of unbonded post-tensioned precast concrete walls with horizontal joints. This ongoing research includes a large-scale experimental study of the lateral load behavior of these walls and associated analytical research.

The research presented in this thesis has the following objectives: (1) to develop more accurate (improved) stress-strain relationships for the materials used in the ongoing experimental research on unbonded post-tensioned precast walls and (2) to use these improved stress-strain relationships in an existing analytical model of the walls in order to improve the predictions of the lateral load behavior of these walls. The research included experimental testing of post-tensioning system specimens, unconfined concrete cylinder specimens, a stub wall panel specimen, and a buckling wall panel specimen.

It was found that improving the material stress-strain relationships in the analytical model for unbonded post-tensioned precast walls significantly improved the lateral load behavior predictions of this model. A conventional concrete stress-strain model (the Mander model) did not adequately represent the unconfined concrete stress-strain behavior obtained from the cylinder tests. A recently developed concrete stress-strain model (Oh’s model), supported by test data from the unconfined concrete cylinder specimens, was used. Furthermore, a conventional stress-strain model (the Mander model) for confined concrete did not adequately represent the confined concrete stress-strain behavior obtained from the stub wall panel specimen test. Indeed, an accurate stress-strain relationship for this confined concrete could not be obtained from existing confined concrete stress-strain models, and could be obtained only by direct application of test results for the concrete confined by the reinforcing details present in the wall panels (i.e., the test results from the stub panel tests).
1.1 OVERVIEW

The use of unbonded post-tensioned precast concrete systems as the primary lateral load resisting system in buildings located in regions of high seismicity has potential economic and performance benefits. To exploit these potential benefits, research is needed. Current building code restrictions on the use of precast systems in regions of high seismicity add to the need for research. The PRESSS (PREcast Seismic Structural Systems) research program and the related research projects at Lehigh University are addressing the need for this research.

As part of the PRESSS program, researchers at the Advanced Technology for Large Structural Systems (ATLSS) Center at Lehigh University have been investigating the behavior of unbonded post-tensioned precast concrete walls with horizontal joints. Kurama (1997) conducted an initial analytical study of these walls, developed a seismic design approach, and, using this approach, designed a series of prototype walls. Perez et al. (2003) are conducting an experimental study of these walls.

Kurama (1997) developed an analytical model for unbonded post-tensioned walls using the DRAIN-2DX computer program (Prakash and Powell 1993). Questions regarding the material stress-strain relationships that should be used in models of unbonded post-tensioned walls exist. This report
describes tests conducted on the materials of the unbonded post-tensioned precast walls tested at Lehigh University. The stress-strain behavior obtained through testing was used to improve the material stress-strain relationships used in an analytical model of an unbonded post-tensioned precast wall, resulting in a better prediction of the lateral load behavior of this wall.

1.2 OBJECTIVES

The objectives of this research are to:

1. Obtain improved stress-strain relationships for the materials used in the unbonded post-tensioned precast walls tested at Lehigh University.

2. Use the improved stress-strain relationships in an analytical model for these walls in order to improve the predictions of the lateral load behavior of these walls.

3. Investigate the affect of anchorage seating deformations of the post-tensioning bars used in the walls tested at Lehigh University to determine the significance of these deformations.

1.3 SCOPE

In order to achieve the objectives stated above, experimental tests were conducted on three of the main components of the unbonded post-tensioned
precast concrete walls tested at Lehigh University. First, post-tensioning system specimens were tested in axial tension in a 600 kip (2669 kN) capacity universal test machine. Tensile coupons were also tested. Next, unconfined concrete cylinder specimens were tested in axial compression in the same universal test machine. Finally, both a stub panel specimen and a buckling panel specimen were tested in a 5000 kip (22240 kN) capacity universal test machine. The stress-strain relationships obtained from these tests were then used in the DRAIN-2DX analytical model of the unbonded post-tensioned walls tested at Lehigh University. The lateral load behavior of this improved analytical model was compared with results of the analytical model with assumed stress-strain relationships and with test results obtained by Perez et al. (2003). The affects of the improved stress-strain relationships on the predictions of the analytical model are shown by these comparisons. During the tests of the post-tensioning systems anchorage seating deformations were measured to investigate the affect of the anchorage seating on the total deformation of the post-tensioning system.

1.4 SUMMARY OF FINDINGS

This research resulted in four main findings. First, improving the material stress-strain relationships in the DRAIN-2DX analytical model for unbonded post-tensioned precast walls improved the predictions of this model. Second, the anchorage seating deformations of the post-tensioning system obtained through testing were larger than expected. However, the seating deformations
were compared with the elongation of the post-tensioning bar due to strain in order to determine the importance of the anchorage seating. For a short bar, such as a 36 in. (914 mm) bar, anchorage seating deformations have a significant contribution to the total deformation of the post-tensioning system. However, for a long bar, such as a 360 in. (9144 mm) bar, anchorage seating deformations have a less influential contribution to the total deformation of the post-tensioning system. Third, a variation of the Mander model (Mander et al. 1988), obtained by setting the steel reinforcement quantities equal to zero, was not a good representation of the unconfined concrete stress-strain behavior obtained from cylinder tests. The Oh model (Oh 2002) with a 1% elastic region provided a good representation of the unconfined concrete stress-strain behavior. And finally, fourth, the Mander model for confined concrete was not a good representation of the confined concrete stress-strain behavior obtained from the stub panel specimen test.

1.5 ORGANIZATION OF THESIS

Chapter 2 describes the relevant background information pertinent to this research including previous research on unbonded post-tensioned precast concrete walls with horizontal joints. The behavior of axially loaded spirally reinforced concrete is also discussed. The experimental procedures and results for the post-tensioning system specimen tests and the unconfined concrete cylinder specimen tests are described in Chapters 3 and 4, respectively. Chapter 5 discusses the experimental procedures and results for
the spirally confined stub panel specimen and the spirally confined buckling panel specimen. Finally, Chapter 6 describes the lateral load analysis of an unbonded post-tensioned precast concrete wall. The lateral load analysis results from the analytical model with stress-strain relationships obtained from material tests are compared with results of the analytical model with assumed stress-strain relationships, and with wall test results obtained by Perez et al. (2003). Chapter 7 presents the conclusions formed from this research.

1.6 NOTATION

The following notation is used in this report.

\[ A_{cc} = \text{area of confined concrete} \]
\[ A_{sp} = \text{area of transverse reinforcement} \]
\[ A_{uc} = \text{area of unconfined concrete} \]
\[ d_s = \text{center-to-center diameter of the spirals} \]
\[ E_{au} = \text{ascending region modulus of concrete for the Oh model} \]
\[ E_c = \text{Young's modulus for concrete} \]
\[ E_{sec} = \text{secant modulus for concrete} \]
\[ f_c = \text{longitudinal compressive concrete stress of spirally confined concrete} \]
\[ f_{cc} = \text{compressive strength of confined concrete} \]
\[
\begin{align*}
\sigma_{co} & = \text{compressive strength of unconfined concrete} \\
\sigma_{l} & = \text{effective lateral pressure from transverse reinforcement} \\
\sigma_{o} & = \text{peak stress of concrete} \\
\sigma_{yh} & = \text{yield strength of the transverse reinforcement for spirally reinforced and circular hoop reinforced concrete} \\
k_{e} & = \text{confinement effectiveness coefficient} \\
P_{cc} & = \text{confined concrete load} \\
P_{t} & = \text{total recorded load from the stub panel test} \\
r & = \text{ratio of the Young's modulus for concrete to the difference between the Young's modulus for concrete and the secant modulus for concrete, i.e.} \frac{E_{c}}{E_{c} - E_{sec}} \\
r_{au} & = \text{ascending region parameter of concrete for the Oh model} \\
r_{du} & = \text{descending region parameter of concrete for the Oh model} \\
s & = \text{pitch of the spirals} \\
s' & = \text{clear vertical spacing between spirals} \\
x & = \text{ratio of the longitudinal compressive concrete strain to the concrete strain corresponding to maximum concrete stress under lateral pressure, } f_{l}, \text{ i.e.} \frac{\varepsilon_{c}}{\varepsilon_{cc}} \\
\varepsilon_{c} & = \text{longitudinal compressive concrete strain}
\end{align*}
\]
\( \varepsilon_{cc} \) = concrete strain corresponding to maximum concrete stress under lateral pressure, \( f_l \)

\( \varepsilon_{co} \) = concrete strain corresponding to peak concrete stress

\( \varepsilon_1 \) = unconfined concrete strain for the Oh model

\( \varepsilon_{ei} \) = concrete strain corresponding to the concrete stress at the end of the elastic region for the Oh model

\( \varepsilon_{10} \) = concrete strain corresponding to peak concrete stress for the Oh model

\( \sigma_{cc} \) = confined concrete stress

\( \sigma_{uc} \) = unconfined concrete stress calculated using the Oh model

\( \sigma_1 \) = unconfined concrete stress for the Oh model

\( \sigma_{10} \) = peak concrete stress for the Oh model

\( \sigma_{ei} \) = concrete stress at the end of the elastic region for the Oh model

\( \rho_{cc} \) = ratio of area of longitudinal reinforcement to area of core of section defined by center lines of the perimeter spirals

\( \rho_s \) = ratio of volume of transverse confining steel to the volume of confined concrete core

\( \omega_u \) = strain ratio
CHAPTER 2
BACKGROUND

This chapter describes the relevant background information pertinent to the present research. Section 2.1 describes previous research on unbonded post-tensioned precast concrete construction. Section 2.2 describes previous research regarding unbonded post-tensioned precast concrete walls with horizontal joints. Finally, Section 2.3 describes the behavior of axially loaded spirally confined concrete.

2.1 UNBONDED POST-TENSIONED PRECAST CONCRETE CONSTRUCTION

Current building codes restrict the use of precast concrete systems in regions of high seismicity in the United States. Codes such as the Uniform Building Code (UBC 1997) and International Building Code (IBC 2000) require what has been termed a “cast-in-place emulation” design philosophy, which requires precast seismic systems to have lateral load resisting characteristics that mimic those of monolithic cast-in-place reinforced concrete systems (Perez 2001).

Precast concrete systems with “wet” connections, which utilize cast-in-place concrete to form a continuous structure, can be designed under the current cast-in-place emulation philosophy of the building codes (Perez 2001). However, these systems lack some of the advantages of precast construction
because of the cast-in-place concrete as well as expensive details in the connections (Perez 2001).

Precast concrete systems with "dry" connections, which utilize bolts, welds, or other mechanical elements, do not emulate cast-in-place systems, and thus, do not satisfy the current cast-in-place emulation philosophy of the building codes. However, these connections are typically less stiff than the precast members due to the natural discontinuities they create within the structure. These discontinuities allow nonlinear deformations to occur in the connections (Perez 2001).

The potential economic and other benefits of using a precast concrete system with dry connections as the primary lateral load resisting system of a building creates a need for research on these systems. Thus, the PRESSS (PREcast Seismic Structural Systems) research program, aimed at the development of these precast systems, began in 1990 (Perez 2001).

PRESSS research has shown that using unbonded post-tensioning to make dry connections between precast members creates systems with a nonlinear elastic load-deformation response. Moments caused by lateral load overcome the precompression of the connections, and consequently, gaps open at the connections. This gap opening causes a nonlinear response which can be essentially elastic (reversible). Further results also show that a delay of yielding of the post-tensioning steel can be provided through the use of unbonded post-tensioned construction. This allows the prestress to be
maintained throughout seismic loading. Bonded post-tensioned systems do not exhibit this behavior (Perez 2001).

2.2 PREVIOUS RESEARCH ON UNBONDED POST-TENSIONED PRECAST CONCRETE WALLS WITH HORIZONTAL JOINTS

Researchers at Lehigh University have been investigating the lateral load behavior of unbonded post-tensioned precast concrete walls with horizontal joints. An initial analytical study conducted by Kurama (1997) included developing a seismic design approach for unbonded post-tensioned precast concrete walls with horizontal joints and using this approach to design a series of prototype walls. These prototype walls were modeled analytically, and the walls were analyzed under static lateral loads and dynamically under earthquake loading (Kurama 1997).

Kurama (1997) developed the analytical wall model utilizing the fiber beam-column element of the DRAIN-2DX program (Prakash and Powell 1993). This model is discussed in Chapter 6.

The Kurama (1997) analytical study showed that large nonlinear lateral displacements of an unbonded post-tensioned precast wall can be obtained without compromising the integrity of the post-tensioning steel. Because of the unbonding, the post-tensioning steel does not yield or fracture. Also, through dynamic analysis under moderate-to-severe earthquakes, the walls exhibited self-centering capacity and large flexural ductility without undergoing excessive drift or damage (Kurama 1997).
In order to confirm the results produced by Kurama (1997) and verify the proposed seismic design approach, Perez et al. (2003) at Lehigh University are currently conducting experimental tests of unbonded post-tensioned precast walls with horizontal joints. The test walls are based on full-scale walls designed for a six-story prototype building. The building is designed in accordance with IBC (2000) and is very similar to the structure designed by Kurama (1997). Ten interior precast walls and four exterior frames comprise the lateral load resisting system in the longitudinal and transverse directions, respectively. The building is shown in Figure 2.1 (Perez 2001).

Six one-story precast panels separated by horizontal joints comprise a full-scale prototype wall. Unbonded post-tensioned steel connects the panels across the horizontal joints and is anchored at the roof and in the foundation. Because the bottom panel sustains the largest compressive strains, it contains regions of highly confined concrete. Each panel has conduits for unbonded post-tensioned strands as well as wire mesh near the front and back sides of each panel. The full-scale wall and a typical cross-section of the bottom panel are shown in Figures 2.2a and 2.3a, respectively (Perez et al. 2003).

Due to laboratory size constraints, the full-scale prototype wall from the proposed structure was reduced to a 5/12-scale wall, referred to as a scaled wall (Perez et al. 2003). The scaled wall retains the same behavioral characteristics as the full-scale wall because the dimensions are proportional. This is shown in Figures 2.2a and 2.2b as well as 2.3a and 2.3b.
For testing purposes, the scaled wall was modified into a test wall configuration. Four wall panels as well as a loading block and two extension panels (used to simulate the other two panels of the scaled wall) comprise each test wall specimen. Highly confined regions exist on both ends of the bottom two panels; however, the confinement in the second panel is only provided to preserve its integrity for re-use in the testing program. The specimens are built on a precast concrete foundation containing a manhole that provides access to the inside of the foundation for anchorage system assembly. The total unbonded height of post-tensioning steel for the scaled wall is preserved in the test wall (Perez et al. 2003).

A lateral load actuator, connected to the loading block, applies the lateral loads during testing. A hydraulic cylinder applies stress to external bars located on either side of the loading block (front and back), thus providing the gravity load. A test wall specimen is shown in comparison to a scaled wall specimen in Figures 2.4a and 2.4b (Perez et al. 2003).

Perez et al. (2003) are performing both cyclic and monotonic lateral load tests, under constant axial load, and comparing the experimental results to those obtained using the analytical model developed by Kurama (1997). Thus far, both one monotonic and one cyclic lateral load test have been performed. A typical testing arrangement is shown in Figure 2.5.
2.3 BEHAVIOR OF AXIALLY LOADED SPIRALLY CONFINED CONCRETE

The ductility requirement for reinforced concrete structures built in seismic regions is very important. In regions of high compression, the concrete must have sufficient transverse reinforcement. This reinforcement not only confines the concrete, but also prevents buckling of the longitudinal bars. The regions on the ends of the bottom panel of an unbonded post-tensioned precast concrete wall are high compression regions which require significant transverse reinforcement.


The Mander model (1988) is widely accepted and used to model confined concrete. The model depends on the type of confinement, considering both circular and rectangular sections, as well as the type of loading (static or dynamic, monotonic or cyclic). The effect of strain rate is also included, and an effective lateral confining stress is defined based on the specific configuration of transverse and longitudinal reinforcement (Mander et al. 1988).
The Mander model for the longitudinal compressive concrete stress of spirally confined concrete, $f_c$, is given below in Equations 2.1 to 2.10. The model considers monotonic loading with a quasi-static strain rate.

$$f_c = \frac{f_{cc}xr}{r-1+x'r}$$ \hspace{1cm} \text{Equation 2.1}

where $f_{cc}$ is the compressive strength of confined concrete (defined in Equation 2.7); and

$$x = \frac{\varepsilon_c}{\varepsilon_{cc}}$$ \hspace{1cm} \text{Equation 2.2}

where $\varepsilon_c$ is the longitudinal compressive concrete strain, and

$$\varepsilon_{cc} = \varepsilon_{co} \left[ 1 + 5 \left( \frac{f_{cc}}{f_{co}} - 1 \right) \right]$$ \hspace{1cm} \text{Equation 2.3}

where $f_{co}$ and $\varepsilon_{co}$ are the unconfined concrete strength and the strain at peak stress of unconfined concrete, respectively, ($\varepsilon_{co}$ is generally taken as 0.002); and

$$r = \frac{E_c}{E_c - E_{sec}}$$ \hspace{1cm} \text{Equation 2.4}

where

$$E_c = 57000\sqrt{f_{co}} \text{ (psi)}$$ \hspace{1cm} \text{Equation 2.5}

(note: This is the ACI code (ACI Committee 318 1999) definition for $E_c$. The Mander model defines the modulus as $E_c = 5000\sqrt{f_{co}}$ (MPa), approximately $60000\sqrt{f_{co}}$ (psi)) and
The compressive strength of the confined concrete is

\[ f_{cc} = f_{co} \left( -1.254 + 2.254 \sqrt{1 + \frac{7.94f'_t}{f_{co}} - 2 \frac{f'_t}{f_{co}}} \right) \]  

where

\[ f'_t = \frac{1}{2} k_e \rho_s f_{yh} \]  

where \( f_{yh} \) is the yield strength of the transverse reinforcement for spirally reinforced or circular hoop reinforced concrete,

\[ \rho_s = \frac{4A_{sp}}{d_s s} \]  

where \( A_{sp} \) is the area of transverse reinforcement, \( d_s \) is the center-to-center diameter of the spirals, and \( s \) is the pitch of the spirals; and

\[ k_e = \frac{1 - \frac{s'}{2d_s}}{1 - \rho_{cc}} \]  

for spirally reinforced concrete where \( s' \) is the clear vertical spacing between spirals and \( \rho_{cc} \) is the ratio of area of longitudinal reinforcement to area of core of section defined by the center lines of the perimeter spirals (Mander et al. 1988).

The Mander model for the longitudinal compressive concrete stress of spirally confined concrete explained above was used by Kurama (1997) and
Perez et al. (2003) in the analytical modeling of the unbonded post-tensioned precast concrete walls with horizontal joints.
Figure 2.1 The plan view of the prototype structure, a six story office building (adapted from Perez 2001).
Figure 2.2 An elevation view comparing the full-scale prototype wall and the scaled wall: (a) full-scale wall, (b) scaled wall (adapted from Perez et al. 2003).
Figure 2.3 A cross-sectional view of the bottom wall panel comparing the full-scale wall to the scaled wall: (a) full-scale wall, (b) scaled wall (adapted from Perez et al. 2003).
Figure 2.4 An elevation view comparing the scaled wall and the test wall: (a) scaled wall, (b) test wall (adapted from Perez et al. 2003).
Figure 2.5  The testing arrangement for the test wall: test frame and test wall specimen (adapted from Perez et al. 2003).
Figure 2.5 The testing arrangement for the test wall: test frame and test wall specimen (adapted from Perez et al. 2003).
CHAPTER 3
EXPERIMENTAL PROCEDURES AND RESULTS FOR POST-TENSIONING SYSTEM SPECIMENS

This chapter describes the procedures and results for tests performed on the post-tensioning system specimens. Section 3.1 and its subsections describe the experimental program performed on the post-tensioning system specimens. Section 3.2 and its subsections present the testing procedures used as well as the test results.

3.1 EXPERIMENTAL PROGRAM
3.1.1 Test Specimens

The six post-tensioning system specimens tested were labeled UAnch_1, UAnch_2, and NAnch_1 to NAnch_4. The post-tensioning system studied was a Dywidag threadbar post-tensioning system including the post-tensioning bar and the anchorage assembly. Each specimen consisted of an anchorage assembly, with a 2.5 in. (64 mm) nut and a 5x8x1.5 in. (127x203x38 mm) anchor plate, at either end of a 1.25 in. (32 mm) nominal diameter post-tensioning bar. The upper end of a post-tensioning system specimen is shown in Figure 3.1.

In addition to the six post-tensioning systems tested, tension coupons were also tested to obtain tensile stress-strain data for the post-tensioning bars. The tension coupons were "505" round specimens labeled U505_1 through
U505_3 and N505_1 through N505_4. Three of the coupons were machined from portions of the post-tensioning bars used for specimens UAnch_1 and UAnch_2. The other four coupons were machined from the post-tensioning bar which was used for specimens NAnch_1 through NAnch_4.

The Dywidag bars were alloy steel, conforming to ASTM A 722 CAN/CSA. The nominal cross sectional area of the 1.25 in. (32 mm) nominal diameter bars was 1.25 in$^2$ (806 mm$^2$), and the minimum specified ultimate stress was 150 ksi (1030 MPa).

3.1.2 Specimens and Test Fixture Details

Each post-tensioning system tested had a bar length of 76 in. (1956 mm) and a clear distance of 51.75 in. (1314 mm) between the anchor plates. The first two post-tensioning systems were previously used in a test of a post-tensioned concrete wall. Specimens UAnch_1 and UAnch_2 used bars cut from post-tensioning bars numbers 5 and 6, respectively, used in a monotonic test performed by Perez et al. (2003). These particular bars (5 and 6) did not yield during the wall test. The other four specimens, NAnch_1 through NAnch_4, had bars cut from an unused bar 413 in. (10490 mm) in length. The openings in the crossheads of the testing machine exceeded the dimensions of the anchor plate, so a 10x18x3 in. (254x457x76 mm) bearing plate was located between each anchor plate and crosshead and bolted to the outside of each crosshead. The bearing plate and anchor assembly are shown in Figure 3.1.
The post-tensioning system specimens as well as the tension coupons were tested in axial tension in a 600 kip (2669 kN) capacity Satec universal testing machine. The post-tensioning bars and anchor assemblies were clamped to the crossheads when the bars were under tension, but were otherwise free. Therefore, a safety system was designed to catch the bar as well as the nut and anchor plate after fracture of the bar. This system is shown in Figure 3.2. The upper part of the system consisted of a 28x10x10 in. (711x254x254 mm) built-up wood block constructed from plywood positioned below a 28x10x2 in. (711x254x51 mm) A36 steel plate. The plate and wood were bolted to two 36 in. (914 mm) long built-up beams located on the top of the upper crosshead by four 42 in. (1067 mm) long, 7/8 in. (22 mm) diameter all-thread rods. The built-up beams consisted of two 36 in. (914 mm) long C5x6.7 sections placed 2 in. (51 mm) apart with a 36 in. (914 mm) long 4.75x2 in. (121x51 mm) A36 steel plate welded to the top flanges of the C-sections. Two 36 in. (914 mm) long W6x15 beams were positioned in weak axis bending on the bottom of the upper crosshead directly below the built-up beams. The W-sections were bolted to the built-up beams by four 28 in. (711 mm) long, 7/8 in. (22 mm) all-thread rods. Also, three 28x10x0.5 in. (711x254x13 mm) A36 steel guiding plates with a 1.75 in. (44 mm) diameter center hole were attached to the upper all-thread rods. The bar from each post-tensioning system test passed through the center hole of these guiding plates to ensure that, after fracture, the bar traveled directly upward into the wood block.
The lower part of the safety system consisted of a 22 in. (559 mm) long W18x40 section positioned in weak axis bending on the bottom platen of the testing machine. Four 28 in. (711 mm) long 2x4 in. (51x102 mm) wood posts were bolted to the edges of the flanges. Four 21 in. (533 mm) wide plywood panels each approximately 44 in. (1118 mm) high were screwed into the wood posts to form a box. Inside the box, a 26 in. (660 mm) tall 6x6 in. (152 mm) oak post stood with a 17.5x22x0.75 in. (445x559x19 mm) piece of plywood attached to the upper end of the post to catch the bar, nut, and anchor plate upon fracture. Also, four wood fins were attached to the oak post to prevent the post from falling over.

Figure 3.3 shows a typical testing arrangement for the post-tensioning system specimens. The bars were anchored on the crossheads of the test machine, and a 20 in. (508 mm) length was left between the upper and lower crossheads.

3.1.3 Instrumentation

Axial load and deformation as well as crosshead displacement were recorded for all six of the post-tensioning system specimens. Axial load and crosshead displacement were recorded from the output signal from the Satec universal test machine. Axial deformations were recorded by strain gages and linear potentiometers. Also, punch marks were made in the post-tensioning bars to estimate the total elongation of the bars.
The typical instrumentation scheme for a post-tensioning system test is shown in Figure 3.4. An extensometer with two 4 in. (102 mm) linear potentiometers was centered on the bar. The potentiometers were positioned 180 degrees from each other on the flat sides of the bar (north and south faces of the bar), as shown in Figure 3.5. Two strain gages were also centered within the 8 in. (203 mm) gage length of the extensometer. Another strain gage was attached 16.5 in. (419 mm) below the centered strain gages on the south face of the bar, and a fourth and final strain gage was attached 16.5 in. (419 mm) above the centered strain gages on the north face of the bar.

Anchorage seating displacements were recorded for specimens NAnch_1 through NAnch_4. Two 1.5 in. (38 mm) linear potentiometers were mounted 180 degrees from each other and 0.75 in. (19 mm) below the bottom of the upper crosshead on the flat sides of the bar, as shown in Figure 3.6.

For the tension coupons, axial load, deformation, and crosshead displacement were recorded. Axial load and crosshead displacement were recorded from the output signal from the test machine. The axial deformations were recorded by an extensometer. Also, punch marks were made in each tension coupon specimen to estimate the total elongation of the specimen. The punch marks were placed 90 degrees from the extensometer.
3.2 EXPERIMENTAL PROCEDURES AND RESULTS

3.2.1 Experimental Procedures

A two-zone loading procedure was utilized for both the post-tensioning system specimens and the tension coupons. For the six post-tensioning system specimens, both loading zones were controlled by displacement. The first loading zone used a rate of 0.0283 inches per minute (0.719 mm per minute) until a load of 179 kips (796 kN) had been obtained. Then, the second and final loading zone began with a rate of 0.14 inches per minute (3.6 mm per minute). The final loading zone was maintained until the post-tensioning bar fractured. For the seven tension coupons, the first loading zone of the procedure was controlled by stress. A rate of 10 ksi per minute (68.9 MPa per minute) was used until the stress reached 100 ksi (689 MPa). Then the final loading zone began. This zone was controlled by displacement with a rate of 0.02 inches per minute (0.51 mm per minute) until the coupon fractured.

3.2.2 Experimental Results

The results from the testing of the tension coupons are shown in Table 3.1 and Figures 3.7a and 3.7b. The seven tension coupons had similar results except for the results of specimen N505_2. Specimen N505_2 had both a larger yield stress and ultimate stress than all of the other specimens. The yield stress of specimen N505_2 was 8.8% larger than the smallest yield stress of the other tension specimens, and its ultimate stress was 7.3% larger.
Figures 3.8a and 3.8b show typical fractures of the post-tensioning bars for the post-tensioning system specimens. Most of the post-tensioning system specimen bars fractured within the upper crosshead of the test machine. The test results are shown in Table 3.2 and Figures 3.9 and 3.10. The yield and ultimate stresses are based on the nominal bar area and are therefore reported as “nominal” in Table 3.2 and Figures 3.9 and 3.10. Due to interference of the lower safety system with the movement of the test machine, specimen UAnch_1 had to be unloaded and then reloaded when the specimen was already well past yield. Therefore, the graph for this specimen is omitted, but the yield and ultimate stress are listed in Table 3.2.

As shown in Table 3.2 and Figure 3.9, the test results for all of the post-tensioning system specimens are similar except for the results of specimen NAnch_2. Specimen NAnch_2 had both a lower nominal yield stress and nominal ultimate stress. Therefore, it was thought that a defect may have existed at the cross-section of the bar where failure occurred, causing the bar to yield and fracture earlier than expected.

Figure 3.10a shows the anchorage seating deformations from NAnch_1 through NAnch_4. The linear potentiometers were attached to the bar, and the deformation of the bar recorded by the potentiometers that was not caused by anchorage seating was deducted from the recorded displacements in order to obtain the anchorage seating deformation. The length for which the bar deformation was deducted from the potentiometer measurements (3.75 in. (95 mm)) is shown in Figure 3.12. Since the total length of bar is typically
considered to be the distance between the anchorage plates, the length of bar over which the bar deformation is deducted from the potentiometer measurements is the bar length between the bottom of the anchor plate and the point of attachment of the potentiometers.

3.2.3 Comparison with Expected Material Properties

Table 3.3 shows the yield stress and ultimate stress reported by Dywidag Systems Engineering from tests on Dywidag post-tensioning bars (2001). The results in Tables 3.1, 3.2, and 3.3, show that both the tension coupons and post-tensioning system specimens had yield and ultimate stress values similar to the results reported by Dywidag Systems Engineering. However, some of the tension coupons and post-tensioning system specimens had large ultimate stress values.

The anchorage seating deformations obtained from the testing of specimens NAnch_1 through NAnch_4 were larger than expected. However, these deformations must be studied further. Deformations that occur due to anchorage seating affect the apparent total elongation (total deformation) of the bar. Therefore, to determine the importance of the anchorage seating, the seating deformations must be compared with the elongation of the bar due to strain. This comparison was done for two different lengths of bars, a 36 in. (914 mm) long bar and a 360 in. (9144 mm) long bar. The anchorage seating deformations were taken as the average seating deformations of specimens NAnch_3 and NAnch_4 as shown in Figure 3.10b. The anchorage seating
deformations at each end of the bar, are added to the bar deformations (bar strain times bar length) and divided by the bar length to show the effect of anchorage seating deformations. The results are shown in Figures 3.11a and 3.11b. From these plots it can be determined that for a short bar such as the 36 in. (914 mm) bar, the anchorage seating deformations have a significant contribution to the total deformation/initial length of the bar. Yet, for a long bar such as the 360 in. (9144 mm) bar, the anchorage seating deformations have a small contribution to the total deformation/initial length of the bar.
<table>
<thead>
<tr>
<th>Specimen</th>
<th>0.2% Yield Stress (ksi)</th>
<th>Modulus of Elasticity (ksi)</th>
<th>Ultimate Stress (ksi)</th>
<th>Strain at Ultimate Stress (in/in)</th>
<th>Maximum Strain (in/in)</th>
<th>Rupture Stress (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>U505_1</td>
<td>141</td>
<td>27700</td>
<td>167</td>
<td>0.075</td>
<td>0.159</td>
<td>138</td>
</tr>
<tr>
<td>U505_2</td>
<td>143</td>
<td>30100</td>
<td>168</td>
<td>0.077</td>
<td>0.144</td>
<td>145</td>
</tr>
<tr>
<td>U505_3</td>
<td>142</td>
<td>28600</td>
<td>168</td>
<td>0.079</td>
<td>0.159</td>
<td>140</td>
</tr>
<tr>
<td>N505_1</td>
<td>137</td>
<td>22200</td>
<td>164</td>
<td>0.078</td>
<td>0.176</td>
<td>132</td>
</tr>
<tr>
<td>N505_2</td>
<td>149</td>
<td>28400</td>
<td>176</td>
<td>0.080</td>
<td>0.181</td>
<td>142</td>
</tr>
<tr>
<td>N505_3</td>
<td>138</td>
<td>35900</td>
<td>165</td>
<td>0.078</td>
<td>0.169</td>
<td>136</td>
</tr>
<tr>
<td>N505_4</td>
<td>137</td>
<td>29700</td>
<td>164</td>
<td>0.074</td>
<td>0.180</td>
<td>129</td>
</tr>
</tbody>
</table>

Table 3.1 The results from testing of the tension coupons. (1 ksi = 6.89 MPa)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Nominal 0.2% Yield Stress (ksi)</th>
<th>Modulus of Elasticity (ksi)</th>
<th>Nominal Ultimate Stress (ksi)</th>
<th>Strain at Ultimate Stress (in/in)</th>
<th>Maximum Strain (in/in)</th>
<th>Nominal Rupture Stress (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>UAnch_1</td>
<td>147</td>
<td>29200</td>
<td>167</td>
<td>0.066</td>
<td>0.072</td>
<td>145</td>
</tr>
<tr>
<td>UAnch_2</td>
<td>138</td>
<td>29000</td>
<td>163</td>
<td>0.073</td>
<td>0.081</td>
<td>141</td>
</tr>
<tr>
<td>NAnch_1</td>
<td>140</td>
<td>29700</td>
<td>166</td>
<td>0.077</td>
<td>0.079</td>
<td>142</td>
</tr>
<tr>
<td>NAnch_2</td>
<td>132</td>
<td>30200</td>
<td>158</td>
<td>0.067</td>
<td>0.069</td>
<td>139</td>
</tr>
<tr>
<td>NAnch_3</td>
<td>142</td>
<td>30800</td>
<td>167</td>
<td>0.072</td>
<td>0.076</td>
<td>146</td>
</tr>
<tr>
<td>NAnch_4</td>
<td>144</td>
<td>30800</td>
<td>168</td>
<td>0.070</td>
<td>0.076</td>
<td>147</td>
</tr>
</tbody>
</table>

Table 3.2 The results from testing of the post-tensioning system specimens. (1 ksi = 6.89 MPa)

32
<table>
<thead>
<tr>
<th>Specimen</th>
<th>0.2% Yield Stress (ksi)</th>
<th>Modulus of Elasticity (ksi)</th>
<th>Ultimate Stress (ksi)</th>
<th>Strain at Ultimate Stress (in/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>137</td>
<td>***</td>
<td>162</td>
<td>***</td>
</tr>
<tr>
<td>Test 2</td>
<td>145</td>
<td>***</td>
<td>165</td>
<td>***</td>
</tr>
</tbody>
</table>

Table 3.3 Test report data provided by Dywidag Systems Engineering (2001).

(1 ksi = 6.89 MPa)
Figure 3.1 The upper bearing plate, anchor plate, and nut configuration of a typical post-tensioning system.
Figure 3.2 The post-tensioning system safety system.
(1 in. = 25.4 mm)
Figure 3.3  A typical post-tensioning system specimen testing arrangement.
Figure 3.4 The instrumentation scheme for a post-tensioning system specimen.
(1 in. = 35.4 mm)
Figure 3.5 The 4 in. (102 mm) linear potentiometers mounted on the bar for a post-tensioning system specimen. Two strain gages are mounted on the bar next to each potentiometer.
Figure 3.6 The 1.5 in. (38 mm) linear potentiometers mounted on the bar for a post-tensioning system specimen.
Figure 3.6 The 1.5 in. (38 mm) linear potentiometers mounted on the bar for a post-tensioning system specimen.
Figure 3.7 Stress versus strain plots for the tension coupons: (a) U505 test specimens, (b) N505 test specimens. The arrow points to the end of the curve for each tension coupon. (1 ksi = 6.89 MPa)
Figure 3.8 Examples of post-tensioning bar fractures: (a) NAnch_3, (b) NAnch_4.
Figure 3.9 Nominal stress versus strain plots for five of the six post-tensioning system specimens. (1 ksi = 6.89 MPa)
Figure 3.10 Nominal stress versus anchorage seating deformation plots for the post-tensioning system specimens: (a) all four test specimens, (b) test specimens NAnch_3 and NAnch_4 and the average of the two. (1 ksi = 6.89 MPa; 1 in. = 25.4 mm)
Figure 3.11 Nominal stress versus total deformation plots for two different length post-tensioning bars: (a) 36 in. bar, (b) 360 in. bar.

(1 ksi = 6.89 MPa)
Figure 3.12 Length for which deformation of the post-tensioning bars not caused by anchorage seating is deducted from the potentiometer measurements. (1 in. = 25.4 mm)
CHAPTER 4

EXPERIMENTAL PROCEDURES AND RESULTS FOR UNCONFINED CONCRETE CYLINDER SPECIMENS

This chapter describes the procedures and results for tests performed on unconfined concrete cylinder specimens. Section 4.1 and its subsections describe the testing program performed on the unconfined concrete cylinder specimens. Section 4.2 and its subsections discuss the procedures used as well as the results of the tests performed on the unconfined concrete cylinder specimens.

4.1 EXPERIMENTAL PROGRAM

4.1.1 Test Specimens

The unconfined concrete cylinders were standard 6x12 in. (152x305 mm) cylinders. They were poured at the same time and from the same concrete batch as the wall panel test specimens used by Perez et al. (2003). The cylinders were prepared according to procedures in ASTM C 31-00 (2000) using a 6000 psi (41 MPa) concrete mixture with #8 limestone C-33 coarse aggregate. The slump was 10 in. (254 mm), and the water to cement ratio was 0.41. The cylinders, poured in July 1998, were cured in wet burlap for 21 days before removal from the molds. Thus, the cylinders were approximately four years old at the time of the tests. The tests were performed to obtain the stress-strain relationship for the unconfined concrete of the wall panel test.
specimens. Prior to the final series of tests, five cylinder specimens (UCTrial1 through UCTrial5) were tested initially to compare unbonded versus bonded end caps as well as the effect of various loading rates. From these initial tests, it was discovered that the unbonded caps allowed too much elastic deformation which was detrimental to determining any part of the post-peak stress-strain curve. Therefore, the three final cylinder specimens (UCTest1, UCTest2, and UCTest3) were capped with a sulfur compound according to ASTM C 617-87 (1987). A loading procedure using five different rates was also established.

4.1.2 Instrumentation

Axial load and deformation as well as crosshead displacement were recorded for three of the five initial cylinders, as well as the three final cylinders. For the other two initial cylinders, axial deformations were not recorded. Axial load was recorded from an output signal from the test machine. The crosshead displacement was recorded from an output signal from the test machine as well as a linear variable displacement transducer, LVDT, mounted between the crosshead and platen of the test machine. Axial deformations were recorded by two external clip gages mounted 180 degrees apart as shown in Figures 4.1 and 4.2. Also, three of the initial cylinders used strain gages to record axial strains. The strain gages were centered under the clip gages.

The clip gages consisted of a full-bridge configuration of strain gages attached to a thin metal strip that was bolted to a 0.75 in. (19 mm) wide
L2x2x1/4 angle. The entire gage pivoted about mounted brackets with a gage length of 7.25 in. (184 mm) (see Figure 4.2).

4.2 EXPERIMENTAL PROCEDURES AND RESULTS

4.2.1 Experimental Procedures

All of the cylinders were tested in axial compression in a 600 kip (2669 kN) capacity Satec universal test machine. A typical testing arrangement is shown in Figure 4.3. The five initial cylinders were labeled UCTrial1 through UCTrial5, and the three final cylinders were labeled UCTest1 through UCTest3. The first three initial specimens had unbonded end caps, while specimens UCTrial4 and UCTrial5 as well as all three final test specimens had bonded sulfur end caps.

In an attempt to capture the descending branch of the load versus displacement curve of the cylinder specimens, various loading rates were utilized. UCTrials 1 and 2 were tested using a two-zone loading procedure. The first zone of the procedure used load control with a rate of 25 kips per minute (111 kN per minute) for the first four minutes. The second and final zone was controlled by displacement with a rate of 0.025 inches per minute (0.635 mm per minute). Next, a four-zone loading procedure was used to test specimens UCTrial3 and UCTrial4. The first zone of this procedure was the same as the first zone of the two-zone procedure. However, the second zone was controlled by displacement with a rate of 0.007 inches per minute (0.178 mm per minute) until a drop in load of 3% from the peak value occurred. The
third zone then began with a displacement rate of 0.003 inches per minute (0.0762 mm per minute) until a drop in load of 5% from the peak value occurred. Finally, the fourth and final zone took over with a displacement rate of 0.001 inches per minute (0.0254 mm per minute). The test was terminated when there was a drop in load of 85% from the peak value.

From these first four tests, it was determined that the testing machine was too flexible to obtain an accurate descending branch of the load versus displacement curve. Therefore, it was decided to obtain an accurate ascending branch from the remaining test specimens and then to use an empirical model (Oh 2002) to estimate the missing descending branch.

The last initial specimen, UCTrial5, was then tested in order to finalize the loading procedure. The same four-zone procedure explained previously was used with one exception. The displacement rate of the second zone was decreased to 0.005 inches per minute (0.127 mm per minute) from the rate of 0.007 inches per minute (0.178 mm per minute) used previously. This improved four-zone procedure provided the best load and displacement measurements. Thus, this procedure was used on the three final test specimens, UCTests 1 through 5.

4.2.2 Experimental Results

Figures 4.4 and 4.5 show typical unconfined concrete cylinder specimen test failures. The results from the tests are shown in Table 4.1 and Figures 4.6
and 4.7. The results from the five initial cylinder specimens are also listed in
the table.

As shown in Figure 4.6, the three final cylinder specimen tests provided
varying peak stresses and strains at the peak stress. Thus, the average of the
three tests was calculated by averaging the stresses at selected strain values
(see Figure 4.7).

The elastic modulus of the concrete was defined both as the value
resulting from the equation given in the ACI code (ACI Committee 318 1999)
\( Ec = 57000 \sqrt{f'c} \) (psi), where \( f'c = 7600 \) psi (52 MPa) the average concrete
compressive strength from the three cylinder specimen tests) and as the
average of the individual slopes of the elastic regions for the three cylinder
specimen tests. The value of the elastic modulus from the ACI code equation
was 4972 ksi (34257 MPa), and the value obtained from the average of the
individual slopes of the elastic regions of the three cylinder specimen tests was
3937 ksi (27126 MPa). The elastic modulus from the ACI code equation is
much larger than the average modulus determined from the data.

Because it was not possible to obtain an experimental descending
branch from the unconfined cylinder specimens, an empirical model was used
to represent this curve. A variation of the Mander model for confined concrete
(Mander et al. 1988) was used by setting the steel reinforcement quantities
equal to zero. However, as shown in Figure 4.8, the Mander model is much
stiffer than the experimental data indicated.
The empirical model created by Oh (2002) was then used to represent the unconfined concrete stress-strain curve. Oh's model designates three main regions to the curve: an elastic region, an ascending region, and a descending region (Oh 2002). The curve is continuous through all three of the regions. The peak stress and the strain at this stress are designated $\sigma_{10}$ and $\varepsilon_{10}$, respectively, and the stress and strain corresponding to the end of the elastic region and the beginning of the ascending region are designated $\sigma_{li}$ and $\varepsilon_{li}$, respectively. Also, the elastic modulus of the concrete is defined as $E_c$.

The equations for all three of the regions are listed below, Equations 4.1 to 4.7.

**Elastic Region:**

$$\sigma_i = E_c \varepsilon_i$$  \hspace{1cm} \text{Equation 4.1}

**Ascending Region:**

$$\sigma_i = (\sigma_{10} - \sigma_{li}) \omega_u \frac{r_{au}}{r_{au} - 1 + \omega_u^r} + \sigma_{li}$$  \hspace{1cm} \text{Equation 4.2}

where

$$\omega_u = \frac{\varepsilon_i - \varepsilon_{li}}{\varepsilon_{10} - \varepsilon_{li}}$$  \hspace{1cm} \text{Equation 4.3}

and

$$r_{au} = \frac{E_c}{E_c - E_{au}}$$  \hspace{1cm} \text{Equation 4.4}

and
\[
E_{av} = \frac{\sigma_{10} - \sigma_{li}}{\varepsilon_{10} - \varepsilon_{li}} \quad \text{Equation 4.5}
\]

Descending Region:

\[
\sigma_1 = \sigma_{10} - \frac{r_{du}}{r_{du} - 1 + \left(\frac{\varepsilon_1}{\varepsilon_{10}}\right)^2 + r_{du}} \quad \text{Equation 4.6}
\]

where

\[
r_{du} = 0.58 + 0.32 f_0 + 0.077 f_0^2 \quad \text{Equation 4.7}
\]

and \( f_0 \) is equal to the peak stress. In Oh’s dissertation (2002), the concrete stress and strain in compression are taken as negative quantities, so \( f_0 = -\sigma_{10} \).

Here compressive stress and strain are taken as positive quantities, so \( f_0 = \sigma_{10} \).

For the unconfined concrete stress-strain curve, the average values of \( \sigma_{10}, \varepsilon_{10}, \sigma_{li}, \varepsilon_{li} \), and \( E_c \) from the three cylinder specimen tests were used to define the input to Oh’s model. These values are listed in Table 4.2. Two different points were used to define the end of the elastic region in order to determine the best representation of the experimental data. Both the stress and corresponding strain at 30% of \( \sigma_{10} \) as well as the stress and corresponding strain at 1% of \( \sigma_{10} \) were used for \( \sigma_{li} \) and \( \varepsilon_{li} \). Curves based on both sets of \( \sigma_{li} \) and \( \varepsilon_{li} \) as well as the experimental data are shown on Figure 4.9. The curve with an elastic region of 1% is closest to the ascending region of the experimental data. Thus, Oh’s model with a 1% elastic region was chosen as
the unconfined concrete stress-strain curve. This curve is shown in Figure 4.10.

4.2.3 Comparison with Expected Material Properties

The elastic modulus from the ACI code equation (ACI Committee 318 1999), 4972 ksi (34257 MPa), is much higher than the modulus calculated from the average of the slopes of the three cylinder specimen tests. Thus, the ACI code predicted a much stiffer concrete than was actually obtained.

Furthermore, the Mander model, when altered to fit the test data, did not model the unconfined concrete cylinder test data well. This model was much stiffer and had a much sharper descending branch than the experimental data showed the cylinder specimens to have. The cylinder specimen tests results showed a much softer and slightly more ductile behavior.

Oh's model (2002) with a 1% elastic region provided a good representation of the unconfined concrete stress-strain behavior for both the ascending and descending regions of the curve. This model was used to represent the unconfined concrete stress-strain curve.
Table 4.1 The unconfined concrete cylinder test results. (1 ksi = 6.89 MPa)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Peak Stress (ksi)</th>
<th>Clip Gage Strain at Peak Stress (in/in)</th>
<th>Strain Gage Strain at Peak Stress (in/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCTrial1</td>
<td>7.8</td>
<td>0.00247</td>
<td>0.00210</td>
</tr>
<tr>
<td>UCTrial2</td>
<td>8.2</td>
<td>0.00226</td>
<td>0.00177</td>
</tr>
<tr>
<td>UCTrial3</td>
<td>7.5</td>
<td>***</td>
<td>***</td>
</tr>
<tr>
<td>UCTrial4</td>
<td>8.0</td>
<td>***</td>
<td>***</td>
</tr>
<tr>
<td>UCTrial5</td>
<td>7.0</td>
<td>0.00271</td>
<td>0.00121</td>
</tr>
<tr>
<td>UCTest1</td>
<td>7.9</td>
<td>0.00291</td>
<td>***</td>
</tr>
<tr>
<td>UCTest2</td>
<td>7.5</td>
<td>0.00293</td>
<td>***</td>
</tr>
<tr>
<td>UCTest3</td>
<td>7.4</td>
<td>0.00260</td>
<td>***</td>
</tr>
</tbody>
</table>

Table 4.2 The variables used for Oh’s empirical model (Oh 2002).
(1 ksi = 6.89 MPa)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th>7.60 ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Peak Stress, $\sigma_{10}$, $f_0$</td>
<td>Average Strain at Peak Stress, $\varepsilon_{10}$</td>
<td>0.0028 in/in</td>
</tr>
<tr>
<td>Average Ec (slope)</td>
<td></td>
<td>3937 ksi</td>
</tr>
<tr>
<td>$\sigma_{11}$ (1% $\sigma_{10}$)</td>
<td>$\varepsilon_{11}$ (1% $\sigma_{10}$)</td>
<td>0.0760 ksi, 0.000019 in/in</td>
</tr>
<tr>
<td>$\sigma_{11}$ (30% $\sigma_{10}$)</td>
<td>$\varepsilon_{11}$ (30% $\sigma_{10}$)</td>
<td>2.28 ksi, 0.00058 in/in</td>
</tr>
</tbody>
</table>
Figure 4.1 The instrumentation scheme for the cylinder test specimens. The strain gages were used for only specimens UCTrial1, UCTrial2, and UCTrial3. (1 in. = 25.4 mm)
Figure 4.2 A clip gage mounted on a cylinder specimen.
Figure 4.2 A clip gage mounted on a cylinder specimen.
Figure 4.3 A typical cylinder specimen testing arrangement.
Figure 4.4 An example of a cylinder specimen failure (UCTest3).
Figure 4.5 An example of a cylinder specimen failure (UCTest2).
Figure 4.6 Stress versus strain curves for the unconfined concrete cylinder specimen tests and the average of the three tests. (1 ksi = 6.89 MPa)

Figure 4.7 The average stress versus strain curve for the unconfined concrete cylinder specimen tests. (1 ksi = 6.89 MPa)
Figure 4.8 Comparison of the experimental data for the unconfined concrete cylinders with two versions of the Mander model (Mander et al. 1988).

\(1 \text{ ksi} = 6.89 \text{ MPa}\)

Figure 4.9 Comparison of the experimental data for the unconfined concrete cylinders with two versions of Oh’s empirical model (Oh 2002).

\(1 \text{ ksi} = 6.89 \text{ MPa}\)
Figure 4.10 The unconfined concrete cylinder stress versus strain curve. The curve is based on Oh's empirical model (2002) with an elastic range of 1% of the peak stress. (1 ksi = 6.89 MPa)
CHAPTER 5
EXPERIMENTAL PROCEDURES AND RESULTS FOR THE SPIRALLY
CONFINED CONCRETE PANEL SPECIMENS

This chapter describes the testing procedures and results for the spirally confined concrete panel specimens. Section 5.1 and its subsections discuss the experimental program for the spirally confined panel specimens. Section 5.2 and its subsections describe the testing procedures and the test results for the first specimen, a stub panel specimen. Section 5.3 and its subsections discuss the testing procedures used and the test results for the second specimen, a buckling panel specimen.

5.1 EXPERIMENTAL PROGRAM

5.1.1 Test Specimens

Two specimens were tested: (1) a spirally confined stub panel specimen and (2) a spirally confined buckling panel specimen. The spirally confined stub panel specimen was cut from a test wall panel that had been previously used as the bottom panel in a monotonic lateral load wall test performed by Perez et al. (2003). As shown in Figure 5.1, the stub panel specimen was removed from the lower west end of the panel. This end of the panel was undamaged during the monotonic test. The stub panel specimen was 28.25x6x41.25 in. (718x152x1048 mm).
The spirally confined buckling panel specimen was also removed from a lateral load test wall panel. This test wall panel was identical to the one used for the stub panel specimen; however, it was previously used as a second story panel for both a monotonic and cyclic test (Perez et al. 2003). The west end of this panel, which was undamaged, was used as the buckling panel specimen. The buckling panel specimen was 28x6x65 in. (711x152x1651 mm)

5.1.2 Specimens Details

As shown in Figure 5.1, the original wall panel contained seven 2.75 in. (70 mm) diameter conduits for unbonded post-tensioned strands. Highly confined regions of concrete existed on both the east and west ends of the panel outside of the conduit region. Each of these confined regions was reinforced by eight interlocking spirals of 4 in. (102 mm) diameter at a pitch of 1.625 in. (41.275 mm). The spiral reinforcement ratio was 7.38%, and the spiral wire diameter was 0.391 in. (9.9 mm). Wire mesh reinforcement was placed within the front and back sides of the panel, and additional wires cut from the mesh were tied to the upper and lower most regions of the panel. Both panel specimens were cut from the spirally reinforced region on the west end of the test panels; however, the stub panel specimen was cut from the lower 41.25 in. (1048 mm) of this region.

Figure 5.2 shows the stub panel in the test configuration. As shown, steel beams were clamped to the top and bottom of the panel to provide additional confinement to these regions. Figure 5.3 shows the details of these
3 ft. (900 mm) long W8x67 beams. For the stub panel specimen, the top and bottom 8.25 in. (210 mm) of the specimen were clamped by the beams, leaving a 24.75 in. (629 mm) long test region. For the buckling specimen, only 6.25 in. (159 mm) of the top and bottom were clamped, leaving a 52.5 in. (1334 mm) long test region. Thin copper strips and 0.1875 in. (4.8 mm) thick neoprene pads were placed between the panel and beams to help develop uniform confinement.

5.1.3 Instrumentation

Both the stub panel and buckling panel specimens had similar instrumentation configurations, as shown in Figures 5.4 and 5.5, respectively. Axial load and deformation as well as crosshead displacement were recorded for both specimens. Axial load was recorded from an output signal from the test machine. The crosshead displacement was recorded from both an output signal from the test machine as well as an LVDT mounted between the crosshead and platen of the test machine. Axial deformations were recorded by bonded strain gages, DEMEC strain gages, and LVDTs.

Six bonded strain gages, referred to as MM strain gages, were attached to both the north and south faces to measure vertical strain. They were located in two vertical lines of gages 4 in. (102 mm) from the east and west edges of the panel. Each vertical line of MM gages consisted of three gages spaced 6 in. (152 mm) apart. For the stub panel specimen, the upper gage of each vertical line was located 6.375 in. (162 mm) below the top of the test region.
For the buckling panel specimen, the upper gages were located 23.25 in. (591 mm) below the top of the test region. Mounting discs for DEMEC strain gage measurements were placed in the same vertical lines as the MM strain gages. The mounting discs were placed 6 in. (152 mm) apart with a MM strain gage in the center of each DEMEC strain gage length. The upper mounting disc of each vertical line was located 3.375 in. (86 mm) and 20.25 in. (514 mm) below the top of the test region of the stub panel and the buckling panel specimens, respectively. Figures 5.6 and 5.7 show photos of the MM strain gage and DEMEC strain gage configuration and the use of the DEMEC strain gage.

Each type of strain gage was numbered 1 through 12 and named for their location on the specimen, north or south face (N or S); east or west end of the face (E or W); and bottom, middle, or top of the face (B, M, or T). An example of one of the gage names is DEMEC-1NEB.

LVDTs having gage lengths of 6 in. (152 mm) were mounted on the east and west ends of the specimens to record axial deformations, three per end for the stub panel specimen (LVDT-1E through LVDT-3E and LVDT-9W through LVDT-11W) and eight per end for the buckling panel specimen (LVDT-1E through LVDT-8E and LVDT-9W through LVDT-12W). The LVDT mounting assembly is shown in Figures 5.8a and 5.8b. For the stub panel specimen, the LVDTs covered an 18 in. (457 mm) region beginning 3.375 in. (86 mm) below the top of the test region. For the buckling panel specimen, the LVDTs covered a 48 in. (1219 mm) region beginning 2.25 in. (57 mm) below the top of the test region.
Each of the panel specimens contained three additional instruments. The stub panel specimen contained three confined concrete strain gages, located inside the first spiral of the spiral reinforcing on the lower west end of the specimen. The gages were attached to a #4 longitudinal steel bar that was centered within the first spiral. The first gage was located 4.5 in. (114 mm) above the bottom of the panel. The second gage was located 8.5 in. (216 mm) above the bottom of the panel, and the third gage was located 18.5 in. (470 mm) above the bottom of the panel. These gages were previously used during the monotonic lateral load testing of the entire test wall and were undamaged during that test. In addition, the buckling panel specimen had three LVDTs positioned in the transverse horizontal direction and attached to the south face to measure out of plane displacements of the panel. These LVDTs were located 12 in. (305 mm) apart beginning 14.25 in. (362 mm) below the top of the test region. These LVDT attachment locations are shown in Figure 5.9.

5.1.4 Specimens Fabrication

The lateral load test wall panels were cast in laminated plywood forms at the ATLSS Engineering Research Center. The forms are shown in Figure 5.10. The forms were oiled before casting to ensure their reusability. Once cast, the panels were cured by covering the top with wet burlap for 21 days. At 21 days the panels were removed from the forms. The concrete strength was obtained from tests of cylinders that were cast from the same batch of concrete as the panels.
The stub and buckling panel specimens were cut from two different lateral load test wall panels after they were used in lateral load tests. The threaded rod inserts used to mount the angles that held the LVDTs were then attached to the east and west ends of the specimens. The MM and DEMEC strain gages were also attached to the north and south faces of the specimens.

5.1.5 Material Properties

The concrete used for the lateral load test wall panels and subsequently the stub and buckling panel specimens was from the same batch of ready-mix concrete as the unconfined cylinders described in Chapter 4. The concrete was a 6000 psi (41 MPa) mixture with #8 limestone C-33 coarse aggregate. The slump was 10 in. (254 mm), and the water to cement ratio was 0.41. The unconfined concrete compressive strength obtained from 6x12 in. (152x305 mm) cylinder tests was 7.6 ksi (52 MPa), as described in Chapter 4.

The spiral wire used to confine the concrete had a specified minimum yield strength of 60 ksi (413 MPa). No tests were performed to verify this value.

5.2 SPIRALLY CONFINED STUB PANEL SPECIMEN

5.2.1 Experimental Procedure

The stub panel specimen was tested in axial compression in a 5000 kip (22240 kN) capacity universal test machine. Figures 5.11 and 5.12 show the specimen in the test machine.
An elastic cycle test was performed on the stub panel specimen before it was loaded to failure. The elastic cycle procedure consisted of ten different load steps: 8 to 50 kips (36 to 222 kN), 50 to 100 kips (222 to 445 kN), 100 to 50 kips (445 to 222 kN), 50 to 100 kips (222 to 445 kN), 100 to 150 kips (445 to 667 kN), 150 to 50 kips (667 to 222 kN), 50 to 100 kips (222 to 445 kN), 100 to 150 kips (445 to 667 kN), 150 to 200 kips (667 to 890 kN), and 200 to 8 kips (890 to 36 kN). The steps were conducted with a rate of 40 to 50 kips per minute (178 to 222 kN per minute) cycled between loads of 8 to 200 kips (36 to 890 kN).

A three zone loading procedure was utilized for the load-to-failure test. The first zone had a load rate of 200 kips per minute (890 kN per minute). While the specimen was behaving elastically, this loading rate was equivalent to a displacement rate of 0.01 inches per minute (0.25 mm per minute). This same displacement rate was then used in the second zone of the procedure when the specimen was beginning to soften. The third and final zone began once the stub panel specimen had reached its ultimate load. This final zone of the procedure had a displacement rate of 0.04 inches per minute (1.02 mm per minute) until failure of the specimen occurred.

5.2.2 Experimental Results

Figures 5.13 through 5.19 show the stub panel specimen test at spalling, peak load, and failure. Also, Figures 5.20 through 5.22 show the specimen
after the fractured material was removed. The test results are shown in Table 5.1 and Figures 5.23 through 5.27.

Although both the MM strain gages and DEMEC strain gages were located at the same places on the stub panel specimen, their readings differed from one another. No discernible pattern existed between the DEMEC and the MM strain gage readings; however, the DEMEC strain gages recorded larger strains per kip (strains per kN) than the corresponding MM strain gages at every location. This is shown in Table 5.1 as well as Figures 5.23 and 5.24. One reason for this difference may be attenuation of strain across the adhesive used to bond the MM strain gages to the stub panel specimen. This attenuation would reduce the strains in the MM strain gages. Another reason for the difference may be the presence of cracks in the stub panel specimen. The presence of cracks within the gage lengths of the DEMEC strain gages would result in larger strain readings for these gages. However, if cracks did not exist where the MM strain gages were attached, then these gages would have smaller strain readings.

Regardless of the differences in the MM and DEMEC strain gage measurements, both types of gages recorded larger strains on the west end of the stub panel specimen than on the east end. This may indicate that the stub panel specimen was loaded unevenly. Figures 5.13 and 5.15 show that when spalling began, cracks formed on the north face of the panel at the upper west end. This uneven loading could be a result of the uneven cover concrete in the stub panel specimen. Because the specimen was removed from a test wall
panel, the sawed face, which became the east face of the stub panel specimen, had more unconfined concrete covering the spirals than the (normal) cover concrete on the west face of the specimen.

Furthermore, the longitudinal bar containing the three confined concrete strain gages was located in the outermost spiral on the west end of the stub panel specimen, but an identical bar did not exist in the outermost spiral on the east end of the specimen. This not only provided additional asymmetry, but also increased the strength of the west end of the specimen causing the initial failure to occur on the east end of the stub panel specimen. This is shown in Figures 5.14, 5.16, and 5.18 as well as in Figures 5.20 to 5.22. The spalling on the south face of the stub panel specimen began at the lower east end of the specimen and expanded as the load increased. Eventually, the spirals on the east end of the specimen fractured and a failure along the entire length of the panel specimen occurred including the buckling of the longitudinal steel bar.

The displacements recorded by the six LVDTs mounted to the east and west faces of the stub panel specimen are shown in Figures 5.25 and 5.26. The steel angles that held the LVDTs were mounted to the panel specimen by threaded rod inserts that were drilled into the core of the outermost spirals of the panel. This configuration was shown in Figure 5.8a and 5.8b. Analysis of the recorded displacements suggested that the steel inserts were not stiff enough and thus the angles and the LVDTs were allowed to move relative to the stub panel specimen. This is clearly shown in the recorded displacements of LVDT-1E and LVDT-2E in Figure 5.25. Under pure compression loading,
LVDT-1E should not show negative (tensile) displacement. Therefore, the middle insert and angle, which is used to mount both LVDT-1E and LVDT-2E, appears to have moved relative to the stub panel specimen. This also helps to explain the large displacements recorded by LVDT-2E.

The suspected movement of the inserts relative to the stub panel specimen required the LVDTs displacements to be averaged to obtain more reasonable results. LVDT-1E, LVDT-2E, and LVDT-3E were averaged to obtain an average displacement of the east end of the stub panel specimen, and LVDT-9W, LVDT-10W, and LVDT-11W were averaged to obtain an average displacement of the west end of the specimen. Also, all six of the LVDTs were averaged to attain an overall displacement of the panel specimen. As shown in Figure 5.27, the east end of the panel specimen recorded 48% more displacement than the west end of the specimen and 24% more displacement than the average of all of the LVDTs. This larger displacement can be explained by the location of the failure region. LVDT-1E and LVDT-2E were mounted at the lower east end of the stub panel specimen where failure first occurred.

Good correlation exists between the displacements of the average of LVDT-1E and LVDT-2E with the overall average of the stub panel specimen. However, when converting these displacements to strains using the appropriate gage lengths, LVDT-1E and LVDT-2E show much greater strains. The strains in these two LVDTs are 36% larger than those obtained by the average all six LVDTs. This is shown in Figures 5.28 and 5.29.
It was determined that the strain of the confined concrete of the stub panel specimen would be best represented by the strains recorded by LVDT-1E and LVDT-2E. However, this is not a good representation of the strains of the unconfined concrete during the initial elastic loading of the specimen. Therefore, it was decided to determine the unconfined concrete strain from the overall displacements of the specimen obtained by averaging all of the LVDTs, but the confined concrete strain was determined from the displacements recorded by LVDT-1E and LVDT-2E.

The unconfined concrete stress-strain curve for the stub panel specimen is shown in Figure 5.30. An empirical model developed by Oh (2002), previously established for the unconfined concrete cylinder specimens, was used to model the unconfined concrete stress-strain behavior. The average strains determined from all six LVDTs was used with Oh’s model to generate the unconfined stress-strain behavior.

Equations 5.1 and 5.2 were used to calculate the confined concrete load, $P_{cc}$, and stress, $\sigma_{cc}$.

\[
P_{cc} = P_t - \sigma_{uc} A_{uc}
\]

Equation 5.1

\[
\sigma_{cc} = \frac{P_{cc}}{A_{cc}}
\]

Equation 5.2

where $P_t$ is the total recorded load from the stub panel test, $\sigma_{uc}$ is the unconfined concrete stress-strain curve from Figure 5.30, $A_{uc}$ is the unconfined
concrete area, equal to 61.85 in\(^2\) (39903 mm\(^2\)), and \(A_{cc}\) is the confined concrete area, equal to 114.75 in\(^2\) (74032 mm\(^2\)).

The confined concrete load-displacement and stress-strain curves for the stub panel specimen are shown in Figures 5.31 and 5.32. The stub panel specimen test was paused several times, and the pauses in loading correspond to the drops in load seen on both graphs. Therefore, polynomial equations were used to calculate a smooth curve model to reproduce the behavior of the confined concrete and eliminate these load drops. Figure 5.33 shows the stress-strain curve of the confined concrete data from the stub panel specimen test as well as the smooth curve model of this behavior. However, this smooth curve model does not capture the correct initial behavior of confined concrete, as shown in Figure 5.34. Confined concrete behavior should be similar to unconfined concrete until a strain of approximately 0.002. Therefore, another smooth curve model was generated which correctly captured the initial stress-strain behavior. This improved smooth curve model is shown in Figure 5.35 compared to the previous smooth curve model. Figure 5.36 shows the stress-strain curve of the confined concrete data from the stub panel specimen test as well as the improved smooth curve model. The improved smooth curve model is taken as the confined concrete stress-strain curve from the stub panel specimen test and used in the analyses given in Chapter 6. This curve is shown in Figure 5.37.
5.2.3 Comparison with Expected Material Properties

Figure 5.38 shows the confined concrete stress-strain curve compared to the stress-strain curve produced by the Mander model (Mander et al. 1988). As shown in the figure, the Mander model does not accurately represent the behavior of the confined concrete from the stub panel specimen test. The Mander model overestimates the peak stress, underestimates the strain at the peak stress, and overestimates the maximum strain. Although the Mander model is widely accept as an accurate representation of the behavior of confined concrete, it does not appropriately depict the behavior of the confined concrete of the stub panel specimen. One possible reason for this result is the influence of residual stresses in the spiral reinforcing steel used to confine the concrete in the panels (Graybeal and Pessiki 1998).

5.3 SPIRALLY CONFINED BUCKLING PANEL SPECIMEN

5.3.1 Experimental Procedure

The buckling panel specimen was tested in axial compression in a 5000 kip (22240 kN) capacity universal test machine. Figures 5.39 and 5.40 show the specimen in the test machine.

An elastic cycle test was performed on the buckling panel specimen before it was loaded to failure. The elastic cycle procedure was divided into six different load steps: 8 to 50 kips (36 to 222 kN), 50 to 100 kips (222 to 445 kN), 100 to 50 kips (445 to 222 kN), 50 to 100 kips (222 to 445 kN), 100 to 50 kips (445 to 222 kN), and 50 to 8 kips (222 to 36 kN). The steps were conducted
with a rate of 35 to 45 kips per minute (156 to 200 kN per minute) cycled between loads of 8 to 100 kips (36 to 445 kN).

A single zone loading procedure was utilized for the load-to-failure test. A displacement rate of 0.01 inches per minute (0.25 mm per minute) was used until failure of the specimen occurred.

5.3.2 Experimental Results

Figures 5.41 through 5.43 show the buckling panel specimen test at failure. The test results are shown in Table 5.2 and Figures 5.44 through 5.48.

Just as for the stub panel specimen, discrepancies existed between the measurements from the MM strain gages and those from DEMEC strain gages that were located in the same region of the buckling panel specimen; however, most of these measured strains were in the range of 25 to 175 με. Only three gages greatly exceeded this range: DEMEC-6NWT, SG-6NWT, and SG-8SWM. Eight of the twelve DEMEC strain gages recorded larger strains per kip (strains per kN) than the corresponding MM strain gages. This is shown in Table 5.2 as well as Figures 5.44 and 5.45. One reason for this difference may be attenuation of strain across the adhesive used to bond the MM strain gages, as discussed for the stub panel specimen. Another reason for the difference may be the presence of cracks in the buckling panel specimen as discussed for the stub panel specimen. For both the DEMEC-6NWT and MM SG-NWT strain gages, a crack may have existed within the gage length of both gages, which could explain the higher measurements recorded by these gages.
Both the MM and DEMEC strain gages recorded larger strains on the west end of the buckling panel specimen than on the east end. This may indicate that the buckling panel specimen was loaded unevenly. As noted for the stub panel specimen, the east face of the specimens had more unconfined concrete covering the spirals than the (normal) cover concrete on the west face of the specimens. However, the difference in cover concrete between the east and west ends of the stub panel specimen was much greater than for the buckling panel specimen.

As shown in Figures 5.44 and 5.45, gages 10SEB and 1NEB for both the DEMEC and MM gages showed consistent strains. Gages 4NWB and 7SWB also showed consistent strains. Therefore, the buckling panel specimen seemed to be straining uniformly throughout the thickness of the specimen.

The load versus displacement of the buckling panel specimen is shown in Figure 5.46, where the load is plotted versus the displacement of the crosshead of the test machine. The curved region at the end of the test signifies the onset of curvature and buckling of the specimen. Spalling accounts for the three earlier decreases in load.

Figures 5.47a through 5.47d and 5.48a through 5.48d show the load versus displacement curves for the LVDTs mounted to both the east and west faces of the buckling panel specimen. Their locations relative to the displaced shape of the panel specimen are shown in Figures 5.49 and 5.50. Letters "T" and "C" indicate areas of tension and compression, respectively, on the panel specimen.
As was shown in Figures 5.8a, 5.8b, 5.39, and 5.40, the LVDTs were staggered on both the east and west faces of the specimen. By comparing the LVDT locations to their recorded displacements, the curvature of the panel can be seen well before the buckling failure occurred. The sign convention for these displacements is positive for compression and negative for tension. LVDTs 2, 3, 6 and 8 on the east face as well as LVDTs 9, 12, and 15 on the west face show decreasing displacements, and all of these LVDTs were located in regions of tension upon buckling of the panel specimen. Conversely, LVDTs 1, 4, and 7 on the east face as well as LVDTs 10, 11, 14, and 16 on the west face show increasing displacements. All of these LVDTs were located in regions of compression upon buckling of the panel specimen. Although LVDT-5E and LVDT-13W both show increasing displacements, LVDT-13W had much larger displacements than LVDT-5E and was located in a region of compression. The reason for the zero displacement readings recorded for several of the LVDTs during the elastic region of loading is unknown.

5.3.3 Comparison with Expected Material Properties

The load at which the buckling panel specimen failed as well as the shape of the specimen upon failure varied from the load and shape obtained from the lateral load wall tests performed by Perez et al. (2003). The buckling panel specimen had different boundary conditions than the panel that buckled in the lateral load test of the entire test wall, and this difference is likely a cause of these discrepancies. Also, the unbraced length of the buckling panel
specimen was less than the unbraced length of the panel in the lateral load test of the entire test wall.
<table>
<thead>
<tr>
<th>Instrument Names</th>
<th>Slope (k/µε)</th>
<th>1/Slope (µε/k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DEMEC-1NEB</td>
<td>0.7776</td>
<td>1.28600823</td>
</tr>
<tr>
<td>SG-1NEB</td>
<td>1.0503</td>
<td>0.95210892</td>
</tr>
<tr>
<td>DEMEC-2NEM</td>
<td>0.7501</td>
<td>1.33315558</td>
</tr>
<tr>
<td>SG-2NEM</td>
<td>0.8548</td>
<td>1.1698643</td>
</tr>
<tr>
<td>DEMEC-3NET</td>
<td>0.8218</td>
<td>1.21684108</td>
</tr>
<tr>
<td>SG-3NET</td>
<td>1.0096</td>
<td>0.99049128</td>
</tr>
<tr>
<td>DEMEC-10SEB</td>
<td>0.8952</td>
<td>1.11706881</td>
</tr>
<tr>
<td>SG-10SEB</td>
<td>1.2854</td>
<td>0.77796795</td>
</tr>
<tr>
<td>DEMEC-11SEM</td>
<td>0.9194</td>
<td>1.08766587</td>
</tr>
<tr>
<td>SG-11SEM</td>
<td>1.0572</td>
<td>0.94589482</td>
</tr>
<tr>
<td>DEMEC-12SET</td>
<td>0.4608</td>
<td>2.17013889</td>
</tr>
<tr>
<td>SG-12SET</td>
<td>1.6879</td>
<td>0.59245216</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Instrument Names</th>
<th>Slope (k/µε)</th>
<th>1/Slope (µε/k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DEMEC-4NWB</td>
<td>0.7101</td>
<td>1.40825236</td>
</tr>
<tr>
<td>SG-4NWB</td>
<td>1.2439</td>
<td>0.80392314</td>
</tr>
<tr>
<td>DEMEC-5NWM</td>
<td>0.6706</td>
<td>1.49120191</td>
</tr>
<tr>
<td>SG-5NWM</td>
<td>0.6534</td>
<td>1.53045608</td>
</tr>
<tr>
<td>DEMEC-6NWT</td>
<td>0.7179</td>
<td>1.39295166</td>
</tr>
<tr>
<td>SG-6NWT</td>
<td>1.0082</td>
<td>0.99186669</td>
</tr>
<tr>
<td>DEMEC-7SWB</td>
<td>0.7110</td>
<td>1.40646976</td>
</tr>
<tr>
<td>SG-7SWB</td>
<td>0.7530</td>
<td>1.32802125</td>
</tr>
<tr>
<td>DEMEC-8SWM</td>
<td>0.7946</td>
<td>1.25849484</td>
</tr>
<tr>
<td>SG-8SWM</td>
<td>0.8647</td>
<td>1.15647045</td>
</tr>
<tr>
<td>DEMEC-9S WT</td>
<td>0.3905</td>
<td>2.56081946</td>
</tr>
<tr>
<td>SG-9SWT</td>
<td>0.9094</td>
<td>1.09962613</td>
</tr>
</tbody>
</table>

Table 5.1 Load per strain and strain per load for strain gages from the stub panel specimen test: (a) east end gages, (b) west end gages. (1 kip = 4.448 kN)
Table 5.2 Load per strain and strain per load for strain gages from the buckling panel specimen test: (a) east end gages, (b) west end gages. 
(1 kip = 4.448 kN)

<table>
<thead>
<tr>
<th>Instrument Names</th>
<th>Slope (k/με)</th>
<th>1/Slope (με/k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DEMEC-1NEB</td>
<td>1.1875</td>
<td>0.84210526</td>
</tr>
<tr>
<td>SG-1NEB</td>
<td>1.3988</td>
<td>0.71489848</td>
</tr>
<tr>
<td>DEMEC-2NEM</td>
<td>1.0179</td>
<td>0.98241478</td>
</tr>
<tr>
<td>SG-2NEM</td>
<td>1.0410</td>
<td>0.96061479</td>
</tr>
<tr>
<td>DEMEC-3NET</td>
<td>0.9500</td>
<td>1.05263158</td>
</tr>
<tr>
<td>SG-3NET</td>
<td>0.9678</td>
<td>1.03327134</td>
</tr>
<tr>
<td>DEMEC-10SEB</td>
<td>0.8906</td>
<td>1.12283854</td>
</tr>
<tr>
<td>SG-10SEB</td>
<td>1.3756</td>
<td>0.72695551</td>
</tr>
<tr>
<td>DEMEC-11SEM</td>
<td>0.8906</td>
<td>1.12283854</td>
</tr>
<tr>
<td>SG-11SEM</td>
<td>0.4924</td>
<td>2.03086921</td>
</tr>
<tr>
<td>DEMEC-12SET</td>
<td>0.5481</td>
<td>1.82448458</td>
</tr>
<tr>
<td>SG-12SET</td>
<td>0.5031</td>
<td>1.98767641</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Instrument Names</th>
<th>Slope (k/με)</th>
<th>1/Slope (με/k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DEMEC-4NWB</td>
<td>0.6786</td>
<td>1.47362216</td>
</tr>
<tr>
<td>SG-4NWB</td>
<td>0.6928</td>
<td>1.44341801</td>
</tr>
<tr>
<td>DEMEC-5NWM</td>
<td>0.6477</td>
<td>1.54392466</td>
</tr>
<tr>
<td>SG-5NWM</td>
<td>0.9686</td>
<td>1.03241792</td>
</tr>
<tr>
<td>DEMEC-6NWT</td>
<td>0.6107</td>
<td>1.6374652</td>
</tr>
<tr>
<td>SG-6NWT</td>
<td>0.6009</td>
<td>1.66417041</td>
</tr>
<tr>
<td>DEMEC-7SWB</td>
<td>0.6196</td>
<td>1.61394448</td>
</tr>
<tr>
<td>SG-7SWB</td>
<td>0.7370</td>
<td>1.3568521</td>
</tr>
<tr>
<td>DEMEC-8SWM</td>
<td>0.7500</td>
<td>1.33333333</td>
</tr>
<tr>
<td>SG-8SWM</td>
<td>0.3323</td>
<td>3.00932892</td>
</tr>
<tr>
<td>DEMEC-9SWT</td>
<td>0.6786</td>
<td>1.47362216</td>
</tr>
<tr>
<td>SG-9SWT</td>
<td>1.1019</td>
<td>0.90752337</td>
</tr>
</tbody>
</table>
One pair of #4 rebars (place inside the spirals)

spiral dia.=4" (center-to-center)
wire dia.=0.207"
spiral length=5'5" (panel height)

2-3/4" dia. duct

stub panel specimen region
top view

8 spirals @ 3.25"

full mesh
additional wires (tie adjacent to the existing wires)

horizontal wires on the inside
vertical wires on the outside

placement and modification of
1 layer of 4x4-W4.0xW4.0 mesh

Figure 5.1 The reinforcement scheme of the spirally confined test wall panels used by Perez et al. (2003). The stub panel specimen and buckling panel specimen were both removed from the west end of identical panels (adapted from Perez et al. 2003). (1 in. = 25.4 mm)
Figure 5.2 The confining beams clamped around the stub panel specimen. This same type of configuration was also used for the buckling panel specimen.
Figure 5.2  The confining beams clamped around the stub panel specimen. This same type of configuration was also used for the buckling panel specimen.
Figure 5.3 The confining beam configuration used for both the stub panel specimen and buckling panel specimen tests. (1 in. = 25.4 mm)
Figure 5.4 The instrumentation configuration for the stub panel specimen. Three confined concrete strain gages are located within the spirals on the lower west end of the specimen. (1 in. = 25.4 mm)
Figure 5.5 The instrumentation configuration for the buckling panel specimen.

(1 in. = 25.4 mm)
Figure 5.6 The MM strain gage and DEMEC gage configuration for both the stub and buckling panel specimens.
Figure 5.7 The method for acquiring data with the DEMEC gage for the stub panel and buckling panel specimens.
Figure 5.8 The LVDT configuration for the stub panel and buckling panel specimens: (a) attachment to concrete, (b) arrangement of LVDTs.
Figure 5.9 The attachment locations for the three LVDTs measuring out of plane displacement for the buckling panel specimen.
Figure 5.9 The attachment locations for the three LVDTs measuring out of plane displacement for the buckling panel specimen.
Figure 5.10 The test wall panel formwork used to make three wall panels. The stub panel specimen and buckling panel specimen were cut out of two of these panels.
Figure 5.11 The testing arrangement for the stub panel specimen: test machine with specimen.
Figure 5.12 The testing arrangement for the stub panel specimen: LVDT arrangement.
Figure 5.13  Spalling of the north face and west face of the stub panel specimen.
Figure 5.13 Spalling of the north face and west face of the stub panel specimen.
Figure 5.14  Spalling of the south face and west face of the stub panel specimen.
Figure 5.15 The north face and west face of the stub panel specimen at peak load.
Figure 5.16 The south face and west face of the stub panel specimen at peak load.
Figure 5.17 The north face and west face of the stub panel specimen at failure.
Figure 5.18 The south face and east face of the stub panel specimen at failure.
Figure 5.19 An example of the fractured spirals of the stub panel specimen.
Figure 5.20 The north face of the stub panel specimen after removing the failed materials.
Figure 5.21 The south face of the stub panel specimen after removing the failed materials.
Figure 5.22 The west face of the stub panel specimen after removing the failed materials. The buckling of the longitudinal steel is evident.
Figure 5.23 Load versus strain for the DEMEC strain gages attached to the stub panel specimen. (1 kip = 4.448 kN)

Figure 5.24 Load versus strain for the MM strain gages attached to the stub panel specimen. (1 kip = 4.448 kN)
Figure 5.25 Load versus displacement for the LVDTs located on the east face of the stub panel specimen. (1 kip = 4.448 kN, 1 in. = 25.4 mm)

Figure 5.26 Load versus displacement for the LVDTs located on the west face of the stub panel specimen. (1 kip = 4.448 kN, 1 in. = 25.4 mm)
Figure 5.27  The average load versus displacement for the LVDTs located on the west face and east face of the stub panel specimen as well as the total average of all the LVDTs mounted on the specimen.

(1 kip = 4.448 kN, 1 in. = 25.4 mm)
Figure 5.28 Comparison of load versus displacement for the average of LVDT-1E and LVDT-2E with the average of all the LVDTs mounted on the stub panel specimen. (1 kip = 4.448 kN, 1 in. = 25.4 mm)

Figure 5.29 Comparison of load versus strain for the average of LVDT-1E and LVDT-2E with the average of all the LVDTs mounted on the stub panel specimen. (1 kip = 4.448 kN)
Figure 5.30 The unconfined concrete stress-strain curve for the stub panel specimen. (1 ksi = 6.89 MPa)
Figure 5.31 Load versus displacement for the confined concrete data of the stub panel specimen. (1 kip = 4.448 kN, 1 in. = 25.4 mm)

Figure 5.32 Stress versus strain for the confined concrete data of the stub panel specimen. (1 ksi = 6.89 MPa)
Figure 5.33 Stress versus strain for the confined concrete data of the stub panel specimen and the smooth curve model of the confined concrete. (1 ksi = 6.89 MPa)

Figure 5.34 Stress versus strain of the confined concrete of the stub panel specimen, the smooth curve model of the confined concrete, and the unconfined concrete. (1 ksi = 6.89 MPa)
Figure 5.35 Stress versus strain of the confined concrete of the stub panel specimen, the smooth curve model of the confined concrete, the improved smooth curve model of the confined concrete, and the unconfined concrete. (1 ksi = 6.89 MPa)

Figure 5.36 Stress versus strain for the confined concrete data of the stub panel specimen and the improved smooth curve model of the confined concrete. (1 ksi = 6.89 MPa)
Figure 5.37 Stress versus strain of the confined concrete of the stub panel specimen. This curve is the improved smooth curve model of the confined concrete data. (1 ksi = 6.89 MPa)
Figure 5.38 Comparison of the confined concrete stress-strain curve for the stub panel specimen with the Mander model for confined concrete (Mander et al. 1988). (1 ksi = 6.89 MPa)
Figure 5.39 The testing arrangement for the buckling panel specimen: test machine with specimen.
Figure 5.40 The testing arrangement for the buckling panel specimen: LVDT arrangement.
Figure 5.41 The north face and west face of the buckling panel specimen at failure.
Figure 5.42 The south face and west face of the buckling panel specimen at failure.
Figure 5.43 An example of the fractured spirals of the buckling panel specimen.
Figure 5.44 Load versus strain for the DEMEC strain gages attached to the buckling panel specimen. (1 kip = 4.448 kN)

Figure 5.45 Load versus MM strain for the strain gages attached to the buckling panel specimen. (1 kip = 4.448 kN)
Figure 5.46 Load versus test machine crosshead displacement for the buckling panel specimen. (1 ksi = 6.89 MPa)
Figure 5.47 Load versus displacement for the LVDTs mounted to the buckling panel specimen: (a) LVDT-1E and LVDT-9W, (b) LVDT-2E and LVDT-10W, (c) LVDT-3E and LVDT-11W, (d) LVDT-4E and LVDT-12W.

(1 kip = 4.448 kN; 1 in. = 25.4 mm)
Figure 5.48 Load versus displacement for the LVDTs mounted to the buckling panel specimen: (a) LVDT-5E and LVDT-13W, (b) LVDT-6E and LVDT-14W, (c) LVDT-7E and LVDT-15W, (d) LVDT-8E and LVDT-16W.

(1 kip = 4.448 kN; 1 in. = 25.4 mm)
Figure 5.49 The east face of the buckling panel specimen at failure. The LVDTs are labeled (LVDT-1E to LVDT-8E) as well as the areas of tension and compression. The mounting angle between LVDT-3E and LVDT-4E fell upon failure.
Figure 5.49  The east face of the buckling panel specimen at failure. The LVDTs are labeled (LVDT-1E to LVDT-8E) as well as the areas of tension and compression. The mounting angle between LVDT-3E and LVDT-4E fell upon failure.
Figure 5.50 The west face of the buckling panel specimen at failure. The LVDTs are labeled (LVDT-9W to LVDT-16W) as well as the areas of tension and compression. The mounting angle between LVDT-11W and LVDT-12W fell upon failure.
Figure 5.50 The west face of the buckling panel specimen at failure. The LVDTs are labeled (LVDT-9W to LVDT-16W) as well as the areas of tension and compression. The mounting angle between LVDT-11W and LVDT-12W fell upon failure.
CHAPTER 6
MATERIAL MODELING AND LATERAL LOAD ANALYSIS OF AN UNBONDED POST-TENSIONED PRECAST CONCRETE WALL

This chapter describes analyses performed using a fiber element-based analytical model for the lateral load behavior of an unbonded post-tensioned precast concrete wall with horizontal joints. The model was originally developed by Kurama (1997). Perez et al. (2003) applied the model to analyze walls tested in an experimental program by varying the geometry and material properties of the model. In this chapter, the effects of the material properties are studied. Section 6.1 explains the modeling assumptions of the DRAIN-2DX unbonded post-tensioned precast wall analytical model. Section 6.2 and its subsections discuss the original modeling of a wall specimen (Perez et al. 2003). Section 6.3 describes the improved modeling of the material properties of this wall. The improved material modeling is based on experimental results presented in Chapters 3 through 5 of this report. Finally, Section 6.4 presents the results of the improved analytical model in comparison with the results of the original analytical model and the results of a lateral load test performed by Perez et al. (2003).

6.1 MODELING ASSUMPTIONS

Developed using the DRAIN-2DX program (Prakash and Powell 1993), the analytical wall model uses fiber elements to represent the precast wall
panels and truss bars to represent the post-tensioning steel. Further
description of the model is given in Section 6.2 and full details are given by
Kurama (1997). Several important assumptions of the analytical wall model are
listed below.

1. Torsional and out-of-plane displacements of a wall are not
modeled. The wall undergoes in-plane axial, flexure, and
shear deformations only.

2. Out-of-plane instability of the wall is not modeled. It is
assumed that the wall is adequately braced against out-of­
plane buckling.

3. Anchorages of post-tensioning tendons remain fully effective
throughout the entire seismic response of the wall.

4. Elastic and inelastic deformations that may occur in the
foundation structure or the supporting ground are not
considered.

5. The concrete in the wall panels does not carry tensile stress,
and, thus, the discrete opening of gaps at the horizontal joints
between panels can be represented as tensile strains in the
panels.
6. Plane sections remain plane in the wall panels.

6.2 ORIGINAL MATERIAL MODELING

6.2.1 Modeling of Wall Panels

The fiber element in DRAIN-2DX was used to model the unbonded post-tensioned precast concrete wall panels. This element captures the nonlinear inelastic axial-flexural behavior of the wall panels. Kurama (1997) describes the element in detail. A brief description of the fiber beam-column element is provided below along with its role in modeling the wall panels.

Figures 6.1a and 6.1b show the elevation of a test wall specimen tested by Perez et al. (2003) and the corresponding analytical wall model, respectively. The four wall panels, loading block, and two extension blocks are modeled using the fiber element. Each of the four panels and the loading block are modeled by two fiber elements. The extension blocks are each modeled by one fiber element.

The fiber element spans from node i to node j, and each element is divided into segments, as shown in Figure 6.1c. Every segment has a slice at the center of the segment (Figure 6.1d), and this slice is subdivided into various fibers (Figure 6.1e). The centroid of each fiber is located relative to the axis of the element. Thus, each fiber is located at a specified distance away from the axis of the element. The fibers model the stress-strain relationship of the cross-section of the wall at their specific locations. Tension strength and stiffness in
the concrete fibers is neglected. The compression stress-strain relationship is discussed below.

**Compression stress-strain relationship for concrete**

Three types of concrete fibers, each having different stress-strain relationships, are identified in the cross-section of the wall. Shown in Figure 6.2, the three types of concrete are unconfined (cover) concrete (i.e., concrete outside the wire mesh), spirally confined concrete (i.e., concrete within the spirals), and mesh-confined concrete (i.e., concrete outside the spirals and inside the wire mesh).

The Mander model (Mander et al. 1988) is used to model the stress-strain relationship of the spirally confined concrete and unconfined concrete of the wall. Idealized curves with five linear segments approximate the smooth stress-strain relationships of the confined and unconfined concrete, as shown in Figures 6.3a and 6.3b. Based on research conducted by Kurama (1997) which showed that modeling the stress-strain relationship of the mesh-confined concrete with the stress-strain relationship of unconfined concrete does not affect the lateral load behavior of a wall, the mesh-confined concrete is assumed to have the same stress-strain behavior as the unconfined concrete.

### 6.2.2 Modeling of Post-Tensioning Steel

The truss element in DRAIN-2DX was used to model the post-tensioning bars in the analytical wall model, as shown in Figure 6.1a. The post-tensioning
bars are anchored to the wall at the roof and at the base. The displacements of each truss element are slaved to the displacements of the fiber element that models the top of the roof panel. The upper node of the fiber element located at the roof of the wall is a master node, and the upper nodes of the truss elements located at the roof of the wall are the slaved nodes. The displacements and rotations of the master node control the displacements and rotations of the slaved nodes. Therefore, the three displacements (two translations and one rotation) of each truss element node at the roof are slaved to the displacements and rotations of the fiber element node of the roof panel. This ensures that the displacements of the truss elements are compatible with the displacements of the fiber element at the roof of the structure. The nodes at the base of the structure are given a fixed boundary condition to model a rigid foundation.

**Stress-strain relationship for post-tensioning steel**

The stress-strain relationship for a typical post-tensioning bar is shown in Figure 6.4a, where $f_{pl}$ and $f_{pu}$ are the stresses corresponding to the linear limit strain and the ultimate strength of the post-tensioning steel, respectively. Although the smooth stress-strain behavior of the post-tensioning steel can be idealized by a trilinear relationship as shown in Figure 6.4a, the truss element in the DRAIN-2DX program uses a bilinear stress-strain relationship, shown in Figure 6.4b. The strains in an unbonded post-tensioning bar typically do not
reach the strain at maximum stress of the tri-linear relationship, \( \varepsilon_{pu} \). Therefore, the bilinear relationship, with the yield strength of the truss element, \( f_{py} \), corresponding to the linear limit of the stress-strain relationship, \( f_{pl} \), is an appropriate approximation of the stress-strain behavior of the post-tensioning bars.

### 6.3 IMPROVED MATERIAL MODELING

The material models used in the analytical wall model are improved by using the experimental results presented in Chapters 3 through 5 of this report. From Chapter 3, the stress-strain behavior of the post-tensioning bars obtained from the testing of six post-tensioning system specimens and seven tension coupons is used to improve the behavior of the truss elements in the analytical wall model. Figure 6.5 shows this improved stress-strain behavior and the idealized bilinear relationship used in the analytical wall model. The anchorages are not included in the model because the anchorage seating deformation results presented in Chapter 3 indicated that these deformations could be neglected. Since the post-tensioning bars of the test wall are approximately 390 in. (9906 mm) long, the influence of the seating deformations is very small.

The unconfined concrete stress-strain relationship presented in Chapter 4 is used to improve the stress-strain relationship of the fibers in the analytical wall model that represent the unconfined (cover) concrete. The improved
relationship was obtained by using Oh's model (Oh 2002) with the parameters based on the test results from the three final cylinder specimens. Figure 6.6 shows the improved stress-strain behavior and the idealized curve composed of five linear segments used in the analytical model.

Finally, the results obtained from the testing of the spirally confined stub panel specimen presented in Chapter 5 are used to improve the stress-strain relationship of the fibers in the analytical wall model that represent the confined concrete. Figure 6.7 shows the improved stress-strain behavior and the idealized curve composed of five linear segments used in the analytical model.

6.4 ANALYSIS RESULTS

Figures 6.8 and 6.9 show the experimental base shear versus loading block displacement for the monotonic lateral load test performed on a test wall and the corresponding results from the original analytical wall model. As shown in the figures, the displacement at failure calculated by the original analytical wall model is much smaller than the displacement obtained from the experimental test. The point when the initial prestressing forces from the post-tensioning system and gravity loads are overcome is called the decompression point. Figure 6.9a indicates the point of decompression for both the experimental test and the original analytical wall model. Yielding of the first post-tensioning bar is indicated in Figure 6.9b. Both decompression and yielding of the first post-tensioning bar occur earlier for the original analytical wall model than for the experimental test.
The results from the improved analytical wall model are compared to results from both the original analytical wall model and the experimental test in Figures 6.10 and 6.11. Figure 6.11a indicates the point of decompression for the experimental test and both analytical models. Yielding of the first post-tensioning bar is indicated in Figure 6.11b. As shown in the figures, the improved analytical wall model, which incorporates the material stress-strain relationships obtained through testing, generally provides a better prediction of the lateral load behavior of the test wall than the original analytical model. It is noted that the decompression point of the experimental test is not predicted well by either model, as shown in Figure 6.11a. However, first yield of a post-tensioning bar is predicted well by the improved analytical model. The point of first yield for the improved analytical model is much closer to the point of first yield of the experimental test than the result obtained from the original analytical model.

Therefore, in summary, the improved analytical wall model provides a better representation of the experimental lateral load behavior of the unbonded post-tensioned precast concrete test wall than the original analytical wall model. By including more accurate material stress-strain relationships in the analytical wall model, the improved model provides better predictions of the experimental behavior, as shown in Figure 6.12.
Figure 6.1 The analytical model: (a) test wall specimen, (b) analytical wall model.
Figure 6.1 (continued) The analytical model: (a) fiber element segments, (b) slice of a segment, (c) fibers of a slice.
Figure 6.2 The three types of concrete fibers in a typical cross-section of a wall panel near the base of the wall (adapted from Perez et al. 2003).
Figure 6.3 Concrete compressive stress-strain relationships for confined and unconfined concrete used for the original analytical wall model: (a) spirally confined concrete, (b) unconfined concrete. (1 ksi = 6.89 MPa)
Figure 6.4 The stress-strain relationship for a typical post-tensioning bar used for the original analytical wall model: (a) typical stress-strain relationship and the trilinear approximation, (b) bilinear stress-strain relationship. (1 ksi = 6.89 MPa)
Figure 6.5 The improved stress-strain relationship for a typical post-tensioning bar and the idealized bilinear relationship. (1 ksi = 6.89 MPa)

Figure 6.6 The improved stress-strain relationship for unconfined concrete. (1 ksi = 6.89 MPa)
Figure 6.7 The improved stress-strain relationship for spirally confined concrete. (1 ksi = 6.89 MPa)
Figure 6.8 Base shear versus loading block displacement for the experimental test and the original analytical wall model.

(1 kip = 4.448 kN; 1 in. = 25.4 mm)
Figure 6.9 Base shear versus loading block displacement for the experimental test and the original analytical wall model: (a) decompression point, (b) first yield point. (1 kip = 4.448 kN; 1 in. = 25.4 mm)
Figure 6.10 Base shear versus loading block displacement for the experimental test and the original and improved analytical wall models.

(1 kip = 4.448 kN; 1 in. = 25.4 mm)
Figure 6.11 Base shear versus loading block displacement for the experimental test and the original and improved analytical wall models: (a) decompression point, (b) first yield point. (1 kip = 4.448 kN; 1 in. = 25.4 mm)
Figure 6.12 Base shear versus loading block displacement for the experimental test and the improved analytical wall model. (1 kip = 4.448 kN; 1 in. = 25.4 mm)
CHAPTER 7
CONCLUSIONS

This chapter presents the conclusions formed from the research described in this thesis. Section 7.1 describes the main conclusions from this research, and Section 7.2 describes the other conclusions from the material tests presented in Chapters 3, 4, and 5.

7.1 MAIN CONCLUSIONS

The main conclusions from this research are the following:

1. Improving the material stress-strain relationships used in the DRAIN-2DX analytical model for unbonded post-tensioned precast walls improved the lateral load response predictions of this model. The improved analytical model provided much better predictions of the experimental lateral load behavior of the test wall.

2. The anchorage seating deformations of the post-tensioning system obtained through testing were larger than expected. However, the seating deformations were compared with the elongation of the post-tensioning bar due to strain in order to determine the importance of the anchorage seating. For a short bar, such as a 36 in. (914 mm) bar, anchorage seating
deformations have a significant contribution to the total deformation of the post-tensioning system. However, for a long bar, such as a 360 in. (9144 mm) bar, anchorage seating deformations have a less influential contribution to the total deformation of the post-tensioning system.

3. A variation of the Mander model (Mander et al. 1988), obtained by setting the transverse steel reinforcement quantities equal to zero, was not a good representation of the unconfined concrete stress-strain behavior obtained from cylinder tests. It was much stiffer and had a much sharper descending branch than the experimental data indicated. The Oh model (Oh 2002) with a 1% elastic region provided a good representation of the unconfined concrete stress-strain behavior and was used to represent the unconfined concrete stress-strain curve.

4. The Mander model (Mander et al. 1988) for confined concrete was not a good representation of the confined concrete stress-strain behavior obtained from the stub panel specimen test. The Mander model overestimated the peak stress, underestimated the strain at peak stress, and overestimated the maximum strain.
7.2 OTHER CONCLUSIONS

Other conclusions formed from the tests of the post-tensioning system specimens, the unconfined concrete cylinder specimens, and the stub panel and buckling panel specimens are as follows:

1. The post-tensioning system stress-strain behavior obtained from the tests agreed with the stress-strain behavior provided by the steel manufacturer.

2. The universal test machine used to perform the unconfined concrete cylinder compression tests was too flexible to obtain an accurate descending branch of the stress-strain curve. Therefore, an empirical model (Oh 2002) was used to represent the stress-strain behavior.

3. For the stub panel and buckling panel specimens, the DEMEC strain gages recorded larger strains per kip (strain per kN) than the corresponding MM strain gages at every location.

4. Both the MM and DEMEC strain gages recorded larger strains on one end of the stub panel specimen and the buckling panel specimen. This may indicate that both panel specimens were loaded unevenly.
5. The steel inserts used to mount the displacement transducers to the east and west faces of the stub panel specimen were not stiff enough and allowed the displacement transducers to move relative to the stub panel specimen.

6. The load at which the buckling panel specimen failed as well as the shape of the specimen upon failure varied from the load and shape obtained from the lateral load test of an entire test wall performed by Perez et al. (2003).
REFERENCES

ACI Committee 318 (1999), "Building Code Requirements for Structural Concrete (318-99) and Commentary (318R-99)," *American Concrete Institute*, June.


VITA

Stacy Marie Horan was born on October 18, 1977 in Valparaiso, IN, to Kathy and Thomas Horan. She has one sibling, an older sister named Melanie. Stacy attended Wheeler High School, graduating in 1996, and continued her education at the University of Notre Dame in Notre Dame, IN. She obtained her Bachelors of Science in Civil Engineering at the University of Notre Dame in June of 2000. She then attended Lehigh University in Bethlehem, PA, where she received a research assistantship. Stacy expects to receive her Master of Science in Civil Engineering in January of 2003. She is currently working for the design firm Halvorson Kaye, Structural Engineers, in Chicago, IL.
END OF TITLE