ABSTRACT

GOODNIGHT, JASON CHAD. The Effects of Load History and Design Variables on Performance Limit States of Circular Bridge Columns. (Under the direction of Dr. Mervyn Kowalsky and Dr. James Nau).

This report discusses a research program aimed at defining accurate limit state displacements which relate to specific levels of damage in reinforced concrete bridge columns subjected to seismic hazards. Bridge columns are designed as ductile elements which form plastic hinges to dissipate energy in a seismic event. To satisfy the aims of performance based design, levels of damage which interrupt the serviceability of the structure or require more invasive repair techniques must be related to engineering criteria. For reinforced concrete flexural members such as bridge columns, concrete compressive and steel tensile strain limits are very good indicators of damage.

Serviceability limit states such as concrete cover crushing or residual crack widths exceeding 1mm may occur during smaller, more frequent earthquakes. While the serviceability limit states do not pose a safety concern, the hinge regions must be repaired to prevent corrosion of internal reinforcing steel. At higher ductility demands produced by larger less frequent earthquakes, reinforcing bar buckling may lead to permanent elongation in the transverse steel, which diminishes its effectiveness in confining the concrete core. Bar buckling and significant damage to the core concrete represent the damage control limit states, which when exceeded lead to significant repair costs. Furthermore, rupture of previously buckled bars during subsequent cycles of loading leads to rapid strength loss. The life safety or collapse prevention limit state is characterized by fracture of previously buckled bars.

The goal of the experimental program is to investigate the impact of load history and other design variables on the relationship between strain and displacement, performance strain limits, and the spread of plasticity. The main variables for the thirty circular bridge column tests included: lateral displacement history, axial load, longitudinal steel content, aspect ratio, and transverse steel detailing. A key feature of the experiments is the high fidelity strain data obtained through the use of an optical 3D position measurement system.
Column curvature distributions and fixed-end rotations attributable to strain penetration of reinforcement into the footing were quantified.

The following sequence of damage was observed in all of the cyclically loaded experiments: concrete cracking, longitudinal steel yielding, cover concrete crushing, confinement steel yielding, longitudinal bar buckling, and fracture of previously buckled reinforcement. The first significant loss in strength occurred when previously buckled reinforcement fractured. The measured data was used to refine strain limit recommendations. Particular attention was paid to the limit state of longitudinal bar buckling, since it limited the deformation capacity of all of the cyclically loaded specimens. Empirical expression were developed to predict the compressive strain at cover crushing, the compressive strain at spiral yielding, and the peak tensile strain prior to visible buckling after reversal of loading.

In design, limit state curvatures are converted to target displacements using an equivalent curvature distribution. The Modified Plastic Hinge Method was developed to improve the accuracy of strain-displacement predictions. Key aspects of the proposed model which differentiate it from the current method include: (1) a decoupling of column flexure and strain penetration deformation components, (2) a linear plastic curvature distribution which emulates the measured curvature profiles, and (3) separate plastic hinge lengths for tensile and compressive strain-displacement predictions.

In the experiments, the measured extent of plasticity was found to increase due to the combined effects of moment gradient and tension shift. The proposed tension hinge length was calibrated to match the upper bound of the measured spread of palsticity. The proposed compressive hinge length only contains a term related to the moment gradient effect. Expressions which describe the additional column deformation due to strain penetration of reinforcement into the adjoining member were developed. When compared to the current technique, the Modified Plastic Hinge Method improved the accuracy of both tensile and compressive strain-displacement predictions.
The Effects of Load History and Design Variables on Performance Limit States of Circular Bridge Columns

by

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A dissertation submitted to the Graduate Faculty of North Carolina State University in partial fulfillment of the requirements for the degree of Doctor of Philosophy

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BIOGRAPHY

Jason C. Goodnight was born in Charlotte, NC on March 30, 1987 and was raised by his parents, Jack and Sherrie Goodnight, in Mooresville, NC. He attended North Carolina State University, where he graduated Suma Cum Laude with a Bachelor of Science in Civil Engineering. Prior to graduation, he began work at the Constructed Facilities Laboratory (cfl), helping graduate students with their structural testing projects.

He accepted a project on performance limit states of circular bridge columns as part of his en-route Master of Civil Engineering and Doctorate of Philosophy degrees. His research work throughout graduate school consisted of experimental and analytical components focusing on structural behavior in regards to seismic performance. Following the completion of his Doctorate degree, he plans to continue the experimental program as a Post-Doctoral Researcher. The continuation of the project will evaluate the influence of bi-directional seismic load history on column performance.
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during Reversal from $\mu 10.014.26 \ sec = 6.70"$  and (Right) Buckled Deformation in
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LIST OF SELECTED NOTATIONS

$A_{st} = \text{Total Area of Longitudinal Steel in the Column Cross Section}$

$A_g = \text{Gross Area of the Column Cross Section}$

$\rho_t = \frac{A_{st}}{A_g} = \text{Longitudinal Reinforcement Ratio}$

$P = \text{Compressive Axial Load Applied to the Specimen}$

$f'_c = \text{Measured Column Concrete Strength at the Day of Testing}$

$f'_{cf} = \text{Measured Footing Concrete Strength at the Day of Testing}$

$P / (f'_c A_g) = \text{Column Axial Load Ratio}$

$L = \text{Column Length from the Top of the Footing to the Center of the Applied Lateral Load}$

$L_c = \text{Column Cantilever Length, } L_c = L \text{ for Single Bending, } L_c = L/2 \text{ for Double Bending}$

$D = \text{Column Diameter}$

$M = \text{Moment in the Column, Specifically the Maximum Value at the Base Section}$

$V = \text{Shear Force in the Column}$

$L/D = \text{Column Aspect Ratio, Equivalent to } M/(VD)$

$f_y \text{ and } \varepsilon_y = \text{Yield Stress and Strain of Longitudinal Reinforcement}$

$E_s = \text{Elastic Modulus of Longitudinal Reinforcement}$

$f_h \text{ and } \varepsilon_h = \text{Stress and Strain at Strain Hardening of Longitudinal Reinforcement}$

$f_u \text{ and } \varepsilon_u = \text{Stress and Strain at Maximum Stress of Longitudinal Reinforcement}$

$d_{bl} \text{ and } A_{bl} = \text{Diameter and Area of the Longitudinal Bar}$
\( A_{sp} \) = Cross Sectional Area of a Single Spiral Transverse Steel Bar

\( s \) = Centerline Spacing between Transverse Steel Layers

\( D' \) = Diameter of the Confined Core Measured Between Spiral Centerlines

\( \rho_s = \frac{4A_{sp}}{D's} \) = Transverse Volumetric Steel Ratio

\( \rho_{eff} = \rho_s \frac{f_{yh}}{f'_c} \) = Effective Confinement Ratio

\( \Delta \) = Column Displacement at the Center of the Applied Lateral Load

\( \Delta'_y \) = Column Displacement at First Yield of Longitudinal Reinforcement

\( \phi'_y \) = Base Section Curvature at First Yield of Longitudinal Reinforcement

\( F'_y \ and \ M'_y \) = Shear Force and Moment at First Yield of Longitudinal Reinforcement

\( L_{sp} \) = Equivalent Strain Penetration Length

\( L_{eff} \) = Effective Column Length

\( k \) = Moment Gradient Component of Plastic Hinge Length Expression

\( \varepsilon_s \) = Extreme Fiber Bar Longitudinal Steel Strain

\( \varepsilon_c \) = Extreme Fiber Cover Concrete Strain

\( \varepsilon_{core} \) = Core Concrete Strain Measured at the Centerline of the Transverse Steel

\( M_n \) = Nominal Moment Capacity Defined by First Occurrence of Either \( \varepsilon_s = 0.015 \) or \( \varepsilon_c = -0.004 \)

\( \phi_y = \phi'_y \frac{M_n}{M'_y} \) = Equivalent Yield Curvature at the Base Section

\( \Delta_y = \Delta'_y \frac{M_n}{M'_y} \) = Equivalent Yield Displacement of the Column
\[ \mu_\Delta = \text{Displacement Ductility} \]

\[ \mu_\phi = \text{Curvature Ductility} \]

\[ \phi_p = \phi - \phi_y \frac{M}{M_y} = \text{Plastic Curvature at the base Section} \]

\[ \Delta_p = \text{Column Plastic Displacement} \]

\[ L_p = \text{Equivalent Plastic Hinge Length from Priestley, Calvi, and Kowaksy (2007)} \]

\[ \Delta_e = \text{Elastic Column Displacement} \]

\[ Lpr = \text{Extent of Plasticity, Measured at Intersection of Elastic and Plastic Curvature Profiles} \]

\[ Lpr_t = \text{Tension Hinge Length Based on Triangular Distribution} \]

\[ Lpr_c = \text{Compression Hinge Length Based on Triangular Distribution} \]

\[ Lp_t = \text{Tension Hinge Length Based on Rectangular Distribution} \]

\[ Lp_c = \text{Compression Hinge Length Based on Rectangular Distribution} \]
Chapter 1: Introduction

1.1 Background – Performance Limit States

Bridge columns are designed as ductile elements which form plastic hinges to dissipate energy in a seismic event. The goal of performance based seismic engineering is to design structures to achieve a predictable level of performance under a specific earthquake hazard within definable levels of reliability, as defined by the Structural Engineering Association of California (SEAOC 1999). To satisfy the aims of performance based design, levels of damage which interrupt the serviceability of the structure or require more invasive repair techniques must be related to engineering criteria.

For reinforced concrete flexural members such as bridge columns, concrete compressive and steel tensile strain limits are good indicators of damage. Serviceability limit states such as concrete cover crushing or residual crack widths exceeding 1mm may occur during smaller, more frequent earthquakes (Priestley et al. 1996). While the serviceability limit states do not pose a safety concern, the hinge regions must be repaired to prevent corrosion of internal reinforcing steel. This repair is typically accomplished through removal of damaged concrete, epoxy injection of cracks, and subsequent patching to restore the damaged concrete.

At higher ductility demands produced by larger less frequent earthquakes, reinforcing bar buckling may lead to permanent elongation in the transverse steel, which diminishes its effectiveness in confining the concrete core. Bar buckling and significant damage to the core concrete represent the damage control limit states, which when exceeded lead to significant repair costs (Priestley et al. 1996). The damage control limit states of reinforcement bar buckling and significant damage to the core concrete represent the limit of economical repair. This being said, (Rutledge et al. 2013) demonstrated that the displacement capacity of columns with buckled reinforcement could be rehabilitated through plastic hinge relocation.
Chapter 1. Introduction

This procedure involved capacity protection of damaged column regions through FRP wraps and anchors to relocate of the new plastic hinge to an undamaged section of the column.

Rupture of previously buckled bars or confining steel during subsequent cycles of loading leads to rapid strength loss. The life safety or collapse prevention limit state is characterized by fracture of previously buckled bars or confining steel. A summary of damage observations from a reinforced concrete bridge column tested at NCSU as part of this research appears in Figure 1.1. Stable hysteretic response was observed until the first fracture of a previously buckled longitudinal bar.

![Displacement History, Hysteretic Response, and Performance Limit States](image)

Figure 1.1 Displacement History, Hysteretic Response, and Performance Limit States
1.2 The Need for Research

While the progression of damage in flexural bridge columns has been investigated in the past for both uniaxial and biaxial deformation demands, traditional methods of deriving material strains from measured curvatures do not assess strains at the locations of interest, namely the longitudinal reinforcement and core concrete. These methods utilize an array of linear potentiometers placed on the ends of threaded rods embedded in the core concrete to calculate changes in displacement outside of the cover concrete. An example of this instrumentation technique from experiments by (Hose et al. 1997) appears in Figure 1.2. The instrumentation diagram for the test conducted by (Hose et al. 1997) appeared in a report by (Hines et al. 2003).

These linear potentiometer displacement readings, and their associated location within the cross section, are used to compute the cross-section curvature and neutral axis depth. Material strains at locations of interests are then calculated based on this strain profile. The accuracy of computed strain histories are influenced by rotations in the curvature rods themselves. It is for this reason that the current performance strain limit recommendations, summarized in Table 1.1 from (Kowalsky 2000), lack adequate experimental basis.
Chapter 1. Introduction

Figure 1.2 Curvature Rod and Linear Potentiometer Instrumentation from Tests by (Hose et al. 1997), Figure appears in (Hines et al. 2003)

Table 1.1 Performance Strain Limits from (Kowalsky 2000)

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Concrete Compressive Strain Limit</th>
<th>Steel Tensile Strain Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Serviceability</td>
<td>0.004 Cover Concrete Crushing</td>
<td>0.015</td>
</tr>
<tr>
<td>Damage Control</td>
<td>≈0.018, (Mander et al. 1988) $\varepsilon_{cu}$</td>
<td>0.060 Tension Based Bar Buckling</td>
</tr>
<tr>
<td></td>
<td>Limit of Economical Concrete Repair</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Residual Crack Widths Exceed 1mm</td>
<td></td>
</tr>
</tbody>
</table>
1.3 Research Goals and Scope

The 2009 AASHTO Seismic provisions are a displacement-based document and as a consequence, accurate estimates of displacement at key performance limit states are essential. The current code provisions are focused on life safety or collapse prevention under the ultimate limit state defined by loss or significant reductions to the lateral or axial capacity of the columns. The scope of the research presented in this dissertation aims to identify engineering demand parameters which can be used to predict the occurrence of key limit states, and provide engineers with an accurate method of relating these parameters to column deformations needed in design. Particular focus is given to the damage control limit states which lack adequate experimental basis. This includes yielding of confinement steel, significant damage to the core concrete, and buckling of longitudinal reinforcement.

An experimental program was devised to assess the performance of thirty circular well-confined bridge columns. A key feature of the experiments is the high fidelity strain data obtained using an optical 3D position measurement system. The goal of the experimental program is to investigate the impact of load history and other design variables on the relationship between strain and displacement, performance strain limits, and the spread of plasticity. The main variables for the thirty tests include: (1) lateral displacement history, (2) axial load, (3) longitudinal steel content, (4) aspect ratio, and (5) transverse steel detailing.
Chapter 2: Test Setup, Instrumentation, Construction, and Text Matrix

2.1 Test Setup

The specimen was designed to represent a single degree of freedom bridge column subjected to lateral and axial load. The test specimen consists of a footing, column, and loading cap, Figure 2.1 and Figure 2.2. The footing is a capacity protected member which secures the specimen to the lab strong floor using post tensioned bars placed through four perimeter holes. The footing was elevated above the lab floor by ½” black board pads placed under the six floor holes. Gypsum cement was poured in the gap between the specimen and the lab floor to create a level bonded connection surface. A 200kip hydraulic actuator, with a 40in stroke capacity, applies lateral load to the loading cap of the specimen, Figure 2.3 and Figure 2.8. The actuator was connected to the lab strong wall through a wall plate with hole patterns customized for various column lengths. The actuator end connections allow for rotation about the horizontal axis, which places the column under single bending.

A spreader beam, two hydraulic jacks, and a load cell are placed above the loading cap to apply a constant axial compressive load, Figure 2.7. Rectangular ducts near in the middle of the footing allowed for movement of the axial post tensioning bars. A self-regulating axial load system was utilized with a third hydraulic jack in a force controlled uniaxial testing machine to regulate the pressure, and thus the load, of two jacks on top of the specimen to maintain a constant axial load throughout testing. This technique of axial load application does not replicate true P-Δ effects induced by a vertical gravity load. Instead, the axial force follows the direction of the post tensioning bar which is hinged at the floor and centered above the column. Examples of test setups which recreate P-Δ effects may be found in (Dutta et. al. 1999) and (Esmaeily and Xiao 2002).
Chapter 2: Test Setup, Instrumentation, and Construction Process

The procedure used to setup the self-regulating axial load system is described below. The two hydraulic jacks at the top of the specimen and the third jack placed into the uniaxial testing machine were elongated to half of their total stroke capacity. The locking nuts on the post tensioning bars were tightened beneath the lab floor. Under displacement control, the platens for the universal testing machine were brought into contact with both ends of the jack. A hydraulic pump was added to the closed hydraulic system connecting the three jacks to increase the pressure, and thus the load, until each jack reached half of the target column axial load. The two jacks placed above the column apply the total column axial load. The valve for the hydraulic pump was then closed creating an initially pressurized, but locked off system between the three hydraulic jacks. At this point, the MTS platens under displacement control have not moved. The universal testing machine was then placed into force control to maintain that prescribed level of axial load in the three jack system as the column undergoes lateral displacements. This process was done in reverse to remove the axial load from the column. The technique can be done with both single acting and double acting jacks. For specimens subjected to the highest axial loads double acting jacks were utilized, and a separate hydraulic reservoir was placed above the column and the MTS machine, Figure 2.7.
Figure 2.1 General Dimensions for 24” and 18” Specimens with 8ft Cantilever Lengths

Figure 2.2 General Dimensions for 24” and 18” Specimens with 8ft Cantilever Lengths
Figure 2.3 Front View of Test Specimen
Figure 2.4 Side and Top Views of the Test Setup
Figure 2.5  Test Setup Overview

Figure 2.6  Specimen Under Lateral Deformation
Figure 2.7 Self-Regulating Axial Load with 3rd Jack in Force Controlled MTS Machine

Figure 2.8 Test Setup for 11’ and 13’ Cantilever Length Specimens
2.2 Test Matrix

The main variables for the thirty tests include: (1) lateral displacement history, (2) axial load, (3) longitudinal steel content, (4) aspect ratio, and (5) transverse steel detailing. The specimens were constructed in sets of six columns. The first six specimens, which are excluded from the body of this report and design recommendations, were constructed by a local contractor. The standards of detailing were not sufficient to isolate the impact of individual variables. The transverse steel spacing was not uniform, which influenced the restraint of longitudinal bars and subsequent bar buckling. A brief review of the first six tests is included in Chapter 10. To improve the quality of construction, all subsequent specimens were constructed at NCSU by the research team. Test 7 used the same instrumentation technique as the first six experiments. This technique involved tack welding steel post extensions to longitudinal reinforcement, Figure 2.13. Test 7 was ultimately excluded from design recommendations, because the A706 steel was influenced by the surface tack welds. The strain at maximum stress was reduced, and this shifted the failure mechanism for the column to brittle fracture of reinforcement without prior bar buckling. The results for Test 7 and a discussion of weldability if A706 reinforcing steel is included in Chapter 9.

An improved instrumentation system was devised in which instrumentation was directly applied to the reinforcing steel. All design recommendations and strain data presented in this report come from Tests 8-30 which utilized the direct application instrumentation method. This technique is discussed in greater detail in the following section. Nominally identical Specimens 8-12 investigated the impact of lateral displacement history on column performance. The test matrix for these experiments appears in Table 2.1, while a full observation summary appears in Section 3.1. Specimens 13-18, in Table 2.2 and Section 3.2, investigated the transverse steel detailing and lateral displacement history. Specimens 19-24, in Table 2.3 and Section 3.3, had axial load and aspect ratio as the primary variable. The final test series, Specimens 25-30 in Table 2.4 and Section 3.4, investigated the impact of longitudinal steel content and axial load on column performance.
### Table 2.1 Column Property Summary for Load History Variable Tests 8-12

<table>
<thead>
<tr>
<th>Test</th>
<th>Load History</th>
<th>D (in)</th>
<th>L/D</th>
<th>Long. Steel ($\rho_l$)</th>
<th>Spiral Detailing ($\rho_s$)</th>
<th>$f'_c$ (psi)</th>
<th>$P/f'_c*Ag$</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>Chile 2010</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>6988</td>
<td>5.4%</td>
</tr>
<tr>
<td>8b</td>
<td>Cyclic Aftershock</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>6988</td>
<td>5.4%</td>
</tr>
<tr>
<td>9</td>
<td>Three Cycle Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>6813</td>
<td>5.5%</td>
</tr>
<tr>
<td>10</td>
<td>Chichi 1999</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>5263</td>
<td>7.1%</td>
</tr>
<tr>
<td>10b</td>
<td>Cyclic Aftershock</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>5263</td>
<td>7.1%</td>
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<tr>
<td>11</td>
<td>Kobe 1995</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>6070</td>
<td>6.2%</td>
</tr>
<tr>
<td>12</td>
<td>Japan 2011</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>6100</td>
<td>6.2%</td>
</tr>
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</table>

### Table 2.2 Column Property Summary for Load History Variable Tests 13-18

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<th>Test</th>
<th>Load History</th>
<th>D (in)</th>
<th>L/D</th>
<th>Long. Steel ($\rho_l$)</th>
<th>Spiral Detailing ($\rho_s$)</th>
<th>$f'_c$ (psi)</th>
<th>$P/f'_c*Ag$</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>Three Cycle Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#4 at 2.75” (1.3%)</td>
<td>6097</td>
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<tr>
<td>14</td>
<td>Three Cycle Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 4” (0.5%)</td>
<td>6641</td>
<td>5.7%</td>
</tr>
<tr>
<td>15</td>
<td>Three Cycle Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2.75” (0.7%)</td>
<td>7232</td>
<td>5.2%</td>
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<tr>
<td>16</td>
<td>Three Cycle Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 1.5” (1.3%)</td>
<td>6711</td>
<td>5.6%</td>
</tr>
<tr>
<td>17</td>
<td>Llolleo 1985</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 1.5” (1.3%)</td>
<td>7590</td>
<td>5.0%</td>
</tr>
<tr>
<td>17b</td>
<td>Cyclic Aftershock</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 1.5” (1.3%)</td>
<td>7590</td>
<td>5.0%</td>
</tr>
<tr>
<td>18</td>
<td>Darfield 2010</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 1.5” (1.3%)</td>
<td>7807</td>
<td>4.8%</td>
</tr>
<tr>
<td>18b</td>
<td>Cyclic Aftershock</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 1.5” (1.3%)</td>
<td>7807</td>
<td>4.8%</td>
</tr>
</tbody>
</table>
### Table 2.3  Column Summary for Aspect Ratio and Axial Load Variable Tests 19-24

<table>
<thead>
<tr>
<th>Test</th>
<th>Load History</th>
<th>D (in)</th>
<th>L/D</th>
<th>Long. Steel ($\rho_l$)</th>
<th>Spiral Detailing ($\rho_s$)</th>
<th>$f'_c$ (psi)</th>
<th>$P/f'_c*Ag$</th>
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</thead>
<tbody>
<tr>
<td>19</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2&quot; (1.3%)</td>
<td>6334</td>
<td>10%</td>
</tr>
<tr>
<td>20</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2&quot; (1.3%)</td>
<td>6467</td>
<td>5%</td>
</tr>
<tr>
<td>21</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>7.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2&quot; (1.3%)</td>
<td>6390</td>
<td>5%</td>
</tr>
<tr>
<td>22</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>7.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2&quot; (1.3%)</td>
<td>6530</td>
<td>10%</td>
</tr>
<tr>
<td>23</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>8.67</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2&quot; (1.3%)</td>
<td>6606</td>
<td>5%</td>
</tr>
<tr>
<td>24</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>8.67</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2&quot; (1.3%)</td>
<td>6473</td>
<td>10%</td>
</tr>
</tbody>
</table>

### Table 2.4  Column Summary for Steel Content and Axial Load Variable Tests 25-30

<table>
<thead>
<tr>
<th>Test</th>
<th>Load History</th>
<th>D (in)</th>
<th>L/D</th>
<th>Long. Steel ($\rho_l$)</th>
<th>Spiral Detailing ($\rho_s$)</th>
<th>$f'_c$ (psi)</th>
<th>$P/f'_c*Ag$</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>Three Cycle Set</td>
<td>24</td>
<td>4</td>
<td>16 #7 bars (2.1%)</td>
<td>#3 at 2&quot; (1%)</td>
<td>6289</td>
<td>5%</td>
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<td>26</td>
<td>Three Cycle Set</td>
<td>24</td>
<td>4</td>
<td>16 #7 bars (2.1%)</td>
<td>#3 at 2&quot; (1%)</td>
<td>5890</td>
<td>10%</td>
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<tr>
<td>27</td>
<td>Three Cycle Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2&quot; (1%)</td>
<td>6149</td>
<td>10%</td>
</tr>
<tr>
<td>28</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2&quot; (1.3%)</td>
<td>6239</td>
<td>15%</td>
</tr>
<tr>
<td>29</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2&quot; (1.3%)</td>
<td>5912</td>
<td>20%</td>
</tr>
<tr>
<td>30</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #8 bars (3.1%)</td>
<td>#3 at 2&quot; (1.3%)</td>
<td>6050</td>
<td>15%</td>
</tr>
</tbody>
</table>
2.3 Instrumentation

Instrumentation diagrams for the specimen appear in Figure 2.9 and Figure 2.10. A string potentiometer, placed at the center of the applied lateral load, was used to monitor the column displacement. The longitudinal and transverse rotations of the cap were monitored with inclinometers. The lateral load and stroke of the 200kip MTS hydraulic actuator were measured through an integrated load cell and temposonic. An axial load cell monitored the contribution of a single jack to the total compressive axial load of the column.

The experimental program utilized an innovative technique of applying a commercially available instrumentation system to measure large strains at the level of the reinforcement with multiple Optotrak Certus HD 3D position sensors produced by Northern Digital Inc. The Optotrak position monitoring system can read the location of target markers placed on the specimen in three dimensional space during a test, Figure 2.11 and Figure 2.12. By calculating the change in three dimensional distance between adjacent target markers, strains can be determined with respect to their original unloaded gage lengths. The 3D accuracy of the Optotrak Certus system reported by Northern Digital Inc. is 0.1mm with a resolution of 0.01mm.

Over the course of the study three different techniques were used to monitor material strains with the Optotrak system: (1) a single positon monitor with post extensions, (2) two position monitors with vertical cover concrete blockouts, and (3) three position monitors with a complete cover blockout. The single position monitor method was utilized in the first seven specimens, which were ultimately excluded from design recommendations in this report. The single positon monitor technique, Figure 2.13, had target markers applied to the ends of tack welded steel posts. Vertical cover concrete blockout strips over six extreme fiber bars were created by blocking out the cover concrete with insulation foam during casting. The measured strains from the target markers at the ends of the post extensions suffer from the same issues of the traditional curvature rod method. Strains are not measured
at the location of interest; therefore the recorded values are influenced by rotations of the rods themselves.

The second method utilized two Optotrak position monitors facing extreme fiber regions of the specimen and direct application of target markers to six reinforcing bars, Figure 2.11. The same technique of blocking out vertical strips of cover concrete to the outside surface of the longitudinal bars was utilized, Figure 2.11. Target markers were applied directly to longitudinal bars in the extreme fiber regions of the column to obtain strain hysteresis, vertical strain profiles, cross section curvatures, curvature distributions, and fixed-end rotations attributable to strain penetration.

The third method used three Optotrak position monitors and a complete cover concrete blockout to the depth of the outside surface of the longitudinal reinforcement. Spiral layers were always in complete contact with the core, as shown in Figure 2.12. Target markers were applied to longitudinal and transverse reinforcement on the two extreme fiber regions and on a shear face. The additional target markers on the transverse steel were used to compute spiral strains based on the change in arc-length between adjacent target markers. This technique allowed for strain measurements in ten reinforcing bars at different depths within the cross section to analyze the plane sections hypothesis.

Strain gages were applied to layers of transverse steel overlaying the extreme fiber longitudinal reinforcement to observe the interaction between compressive demand, transverse steel strain, and buckling restraint. An illustration of the accuracy of the Optotrak system compared to traditional measurement techniques appears in Figure 2.14. A tensile test on a reinforcing bar was conducted with a 2” Optotrak gage length, a 2” extensometer gage length, and a centrally placed strain gage. Closer inspection demonstrates that the Optotrak strains oscillate around the measurements predicted by the conventional instrumentation, but the general trend is captured throughout the entire tensile test. Electrical resistance strain gages fail to remain attached at large inelastic strain levels, which are of interest to this study.
Chapter 2: Test Setup, Instrumentation, and Construction Process

Figure 2.9 Instrumentation Placement on Test Setup

Figure 2.10 Overlapping Measurement Volume for the Optotrak Position Monitors
Figure 2.11 Dual Optotrak Method with Vertical Cover Concrete Blockout Strips
Figure 2.12 Triple Optotrak Method with Complete Cover Concrete Blockout
Chapter 2: Test Setup, Instrumentation, and Construction Process

Figure 2.13 Single Optotrak Method Utilizing Post Extensions (Ultimately Abandoned)

Figure 2.14 Comparison of Measurement Techniques for Tensile Rebar Test
2.4 Construction Process

The first six specimens, which are excluded from the body of this report and design recommendations, were constructed by a local contractor. The standards of detailing were not sufficient to isolate the impact of individual variables. The transverse steel spacing was not uniform, which influenced the restraint of longitudinal bars and subsequent observations of bar buckling. A brief review of the first six tests is included in the Chapter 10, but the data is excluded from design recommendations because the direct application instrumentation method produced superior strain measurements. To improve the quality of construction, all subsequent specimens were constructed at NCSU by the research team. The process through which these specimens were constructed is described below, along with the method of preparing the specimens for instrumentation with the Optotrak position monitoring system.

The general reinforcement layout for the footing, column, and cap appear in Figure 2.15 through Figure 2.19. Two column variants were utilized in the study, (1) a 24” diameter column with 16 longitudinal bars and ½” cover the outside of the transverse steel and (2) a 18” diameter column with 10 longitudinal bars and ½” cover the outside of the transverse steel. The longitudinal bar sizes, transverse steel detailing, and column lengths varied depending on the test series. The test variables and detailing for individual specimens is addressed in Chapter 3. The footing is a capacity protected member designed to remain elastic while the full flexural capacity of the column was developed at the footing-column interface. Specifically, the required footing steel was allocated based on the maximum flexural overstrength of the column plastic hinge. The same transverse steel spacing was utilized over the entire column length. Longitudinal reinforcement was inclined through the footing-column joint to provide joint shear resistance. Additional vertical J-hooks were provided outside the joint for shear resistance. The column reinforcement had ninety degree end bends in which extreme fiber bars turned inside the joint to take advantage of the additional support provided by the inclined compression strut. Individual horizontal hoops were provided in the footing joint for additional confinement and joint shear resistance.
2.4.1 Construction Sequence

The first step in the column construction process was to tie the column reinforcing cages. Wooden support templates with the specific locations of longitudinal bars in the cross section were created to maintain consistent reinforcement placement throughout the column length, Figure 2.20 through Figure 2.22. Individual longitudinal bars were tied in their proper location to the wooden exterior support templates, while interior templates maintained the correct bar placement over the column length. An interior support was provided to minimize the sag created by self-weight of the column cage. The column reinforcement had ninety degree end bends in which extreme fiber bars turned inside the joint to take advantage of the additional support provided by the inclined compression strut, Figure 2.22. The footing and cap locations of the column were left untied with a double overlapping ring of transverse steel at the termination points, Figure 2.20. A constant transverse steel spacing was utilized over the column length, and temporary diagonal supporting bars were added to provide stability until the cage was placed into the footing.

For the second stage of construction, the column cage was placed horizontally onto supports, and the inclined footing longitudinal steel was added to the joint, Figure 2.23 and Figure 2.24. Once the footing longitudinal steel was in place, individual 2/3 circumference overlapping transverse steel hoops were tied over column reinforcement in the footing joint, Figure 2.25. Once the additional joint hoop steel was added, individual J-hooks were tied over the side-by-side footing longitudinal steel outside of the joint, Figure 2.26.

The horizontal column and joint steel assemblages were picked up by a crane, rotated vertically, and placed into the footing formwork. Additional inclined and straight footing longitudinal reinforcement was placed outside of the joint, and later secured using transverse bars, Figure 2.17, Figure 2.18, Figure 2.27, and Figure 2.28. The reinforcement diagram and photos are shown for the most congested 24” and 18” configurations. These columns had the largest steel content and axial load ratio, increasing the demands on the footing. For many of the test series, the size of the footing inclined longitudinal steel was decreased and the
number of J-hooks was reduced. End-capped PVC tubes were used to block out the footing concrete in the four perimeter holes. The two rectangular axial load ducts near the middle of the footing were created using insulation foam.

A concrete mix with a target 28-day strength of 5ksi and a ½” maximum aggregate size was utilized for the footing concrete. Columns were constructed in sets of six, one concrete truck was used to pour three of the footings and a second truck poured the remaining three specimens. Cylinders for the separate trucks were kept separate so that they could be attributed to the correct columns. A seven day covered cure with damp burlap was utilized before removing the footing formwork. The excess concrete which covered some of the lower column reinforcement was removed, and the surface of the footing to column joint was roughened using an air needle scaler, Figure 2.29.

The second instrumentation technique using the Optotrak position monitoring system had vertical strips of cover concrete which were blocked out during construction. This was accomplished using two layers of ½” thick insulation foam over seven extreme fiber bars, Figure 2.31. The first layer was made from individual squares cut and placed directly over the longitudinal bar between each spiral layer. The second layer was a single vertical strip which was adhered to the first layer using construction adhesive. Tie wire was placed between each transverse steel spacing to prevent either layer from slipping during construction. Using this technique, the foam never shifted during casting.

The third method Optotrak instrumentation method required a complete cover concrete blockout to the depth of the outside surface of the longitudinal steel. Strips of ½” insulation foam were cut to match the clear spacing between spiral layers. After wrapping the strips in clear packaging tape to minimize the bond to the core concrete, the strips were wound around the column. At every intersection of the strip with a longitudinal bar tie wire was used to prevent the foam from shifting, Figure 2.30. An outside layer of ¼” thick insulation foam was wound tightly around the column to block out the region outside of the spiral.
A sonotube was placed over the foam blockout and column cage before being wrapped with plastic to keep it dry prior to casting. Formwork was erected to level and stabilize the column and cap as shown in Figure 2.32 and Figure 2.33. The column formwork was freestanding; scaffolding was only used as a work platform. Three panels of the cap formwork were erected before placing the cap reinforcement, Figure 2.19. The rectangular cap ties were fit above and below each of the four actuator attachment holes formed using PVC pipe. The ends of the PVC were fit over circular disks drilled into the formwork panels to prevent movement during casting.

For the 8ft cantilever length columns a forklift and 1/3 cubic yard concrete hopper was used to cast the specimen. A 13ft attachment for a concrete vibrator was used to consolidate individual lifts. Each column consisted of around four lifts, where each lift was connected to the prior lift by consolidating the interface with the concrete vibrator. A crane was used to pour lifts with the concrete hopper for the 11ft and 13ft cantilever length columns. After removing the formwork and sonotube, the insulation foam which was used to block out the cover concrete was left in place to prevent corrosion of the internal reinforcing steel.
Figure 2.15 General Reinforcement Details for 24” Diameter Specimens
Figure 2.16 General Reinforcement Details for 18” Diameter Specimens
Figure 2.17 Footing Reinforcement Details for 24” Diameter Specimens
Figure 2.18  Footing Reinforcement Details for 18” Diameter Specimens
Figure 2.19  Column Cap Reinforcing Details for 18” and 24” Diameter Specimens
Figure 2.20 Column Reinforcing Cage Construction Method with Rebar Templates
Figure 2.21 External and Internal Column Reinforcement Cage Templates

Figure 2.22 Longitudinal Column Rebar Bends in the Footing
Figure 2.23 Footing Longitudinal and Joint Shear Z-Bar Installation

Figure 2.24 Install Footing Longitudinal and Inclined Joint Shear Steel
Figure 2.25 Individual 2/3 Circumference Overlapping Single Hoops in Footing Joint

Figure 2.26 Install Individual J-Hooks over the Back-to-Back Z-Bars
Figure 2.27  Footing Formwork, Reinforcement, Tie Down PVC, and Axial Blockouts

Figure 2.28  Footing Formwork, Reinforcement, Tie Down PVC, and Axial Blockouts
Figure 2.29 Casting of the Footing Concrete with Rented Formwork

Figure 2.30 Installation of 2-Layer Full Cover Concrete Blockout
Figure 2.31 After Casting, 2-Layer Longitudinal Cover Concrete Blockout Strips

Figure 2.32 Column Stabilization and Cap Formwork
Figure 2.33 Details for Column Stabilization and Cap Formwork

Figure 2.34 Casting with Fork Lift, 1/3 Cubic Yard Hopper, and 13’ Concrete Vibrator
2.4.2 Optotrak Target Marker Application Method

Now that the method of blocking out cover concrete during construction has been reviewed for both instrumentation techniques, Figure 2.30 and Figure 2.31, the method of surface preparation and application of Optotrak target markers deserves some discussion. After casting the columns, a small layer of concrete formed over the outside surface of the longitudinal reinforcement since the foam is compressible. An air needle scaler was used to remove this small layer of concrete over the reinforcing steel since the procedure is non-invasive and does not damage the underlying concrete. The remaining concrete debris was removed using a wire brush drill attachment.

Uniaxial tests on reinforcing steel bars were used to identify the best surface preparation technique and adhesive. The first step involved removing the layer of mill scale over the longitudinal reinforcement by polishing the bar with a wire brush. This layer of mill scale would otherwise flake off at high strains, debonding the instrumentation. The surface of the reinforcement was then cleaned with a degreaser. Disposable plastic caps which house the Optotrak target markers were adhered to the reinforcement using several different adhesives. Instrumentation adhered with hot glue and five minute epoxy failed to remain attached at large inelastic strain levels. A flexible silicone-based RTV adhesive provided the best results since it could accommodate the large strains in the steel while retaining strength.

The plastic caps had a center hole which filled with adhesive forming a shear key. A photo of a specimen with the disposable caps applied appears in Figure 2.35. The tape plugging the hole in the plastic cap was removed and the reusable target markers were snapped into place. Tape was used to route wires away from the field of view of the positon monitor. The wire management technique needs to accommodate deformation during the test and allow for the underlying concrete material to crush. The technique ultimately proved successful, and instrumentation remained attached to the latest stages of damage where previously buckled reinforcement fractured, Figure 2.36.
Chapter 2: Test Setup, Instrumentation, and Construction Process

Figure 2.35 Surface Preparation and Target Marker Plastic Cap Application

Figure 2.36 Instrumentation Remained Attached to the Latest Stages of Damage
Chapter 3: Experimental Observations

3.1 Load History Variable Tests 8-12

The load history variable specimens had nominally identical geometry and longitudinal steel content, and were subjected to different quasi-static unidirectional lateral displacement histories. The 24” (610mm) diameter bridge columns, Figure 3.2, contained 16 #6 (19mm) A706 bars for longitudinal reinforcement ($A_{st}/A_g = 1.6\%$) and a #3 (9.5mm) A706 spiral at 2” (51mm) ($4A_{sp}/(D’s) = 1\%$) on center. The shear span for the cantilever columns was 8ft (244cm), and they had a moment to shear ratio of ($M/VD = 4$). The specimens were subjected to a constant axial load of 170kips (756kN), ($P/(f_c’A_g) \approx 5\%$) depending on the concrete compressive strength. The test matrix for the eight columns is shown in Table 3.1, and the material properties of the reinforcement appear in Table 3.2. Monotonic and cyclic stress-strain curves for the longitudinal steel appear in Figure 3.3 and Figure 3.4.

The specimens were subjected to various unidirectional top-column displacement histories including standardized laboratory reversed cyclic loading, and recreations of the displacement responses obtained from non-linear time history analysis of multiple earthquakes with distinct characteristics. The experiments utilized a quasi-static displacement controlled loading procedure. The symmetric three-cycle-set load history is commonly used to evaluate the seismic performance of structural components. The load history begins with elastic cycles to the following increments of the analytically predicted first yield force: $\frac{1}{4} F’_y$, $\frac{1}{2} F’_y$, $\frac{3}{4} F’_y$, and $F’_y$. The experimental first yield displacement is then determined by taking the average of the recorded displacements during the first yield push and pulls cycles. The equivalent yield displacement, used to determine the displacement ductility levels ($\mu_{\Delta_1} = (1 \times \Delta_y)$, is then calculated as $\Delta_y = \Delta’_y (M_n/M’_y)$. The symmetric three-cycle-set load history resumes with three balanced cycles at each of the following displacement ductility levels: 1, 1.5, 2, 3, 4, 6, 8, 10, 12, etc.
Figure 3.1 Target Marker Application and Optotrak Spatial Coordinate Output

Figure 3.2 (Left) Symmetric Three Cycle Set Load History from Test 9 and (Right) Column Cross Section and Bar Designation
For earthquake time-history tests, the analytical top column displacement history is determined using non-linear time history analysis (NLTHA). The original acceleration input of the earthquake record is multiplied by a constant scale factor to produce a peak displacement response suitable for the experimental test. This is necessary because the amplitude of peak response is an important variable when comparing the performance of the columns subjected to different load histories. The goal of the experimental load history is not to re-produce the exact displacement response which the specific acceleration record may have created, but rather to compare the performance of columns subjected to specific characteristics in the displacement histories obtained from NLTHA. Specific earthquake top-column displacement response characteristics were chosen including: the number and amplitude of cycles prior to the peak, degree of symmetry, and peak displacement in each direction of loading.

The symmetric three-cycle-set experiment, Test 9, was conducted prior to earthquake tests to establish the displacement ductility levels. The scaling factors of the acceleration input used in NLTHA of the earthquake load histories were determined based on the displacement capacity of Test 9, which had bar buckling during displacement ductility eight. Two earthquake records (Tests 8 and 10) were scaled approximately displacement ductility nine while two records (Tests 11 and 12) were scaled to ductility ten. The strains at the first yield displacement of each earthquake test were verified to confirm that the ductility levels from Test 9 remained appropriate. Specimens which had un-buckled reinforcement during the earthquake load histories were subjected to a symmetric three-cycle-set displacement history to evaluate the columns post-earthquake performance, Tests 8b and 10b.
### Table 3.1 Column Property Summary for Load History Variable Tests 8-12

<table>
<thead>
<tr>
<th>Test</th>
<th>Load History</th>
<th>D (in)</th>
<th>L/D</th>
<th>Long. Steel ($\rho_l$)</th>
<th>Spiral Detailing ($\rho_s$)</th>
<th>f'c (psi)</th>
<th>P/f'c*Ag</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>Chile 2010</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>6988</td>
<td>5.4%</td>
</tr>
<tr>
<td>8b</td>
<td>Cyclic Aftershock</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>6988</td>
<td>5.4%</td>
</tr>
<tr>
<td>9</td>
<td>Three Cycle Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>6813</td>
<td>5.5%</td>
</tr>
<tr>
<td>10</td>
<td>Chichi 1999</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>5263</td>
<td>7.1%</td>
</tr>
<tr>
<td>10b</td>
<td>Cyclic Aftershock</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>5263</td>
<td>7.1%</td>
</tr>
<tr>
<td>11</td>
<td>Kobe 1995</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>6070</td>
<td>6.2%</td>
</tr>
<tr>
<td>12</td>
<td>Japan 2011</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>6100</td>
<td>6.2%</td>
</tr>
</tbody>
</table>

### Table 3.2 Reinforcement Material Property Summary for Columns 8-12

<table>
<thead>
<tr>
<th>Longitudinal Reinforcement</th>
<th>$\varepsilon_y$</th>
<th>fy (ksi)</th>
<th>$\varepsilon_h$</th>
<th>fh (ksi)</th>
<th>$\varepsilon_u$</th>
<th>fu (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tests 8-12 (#6 Bar)</td>
<td>0.00235</td>
<td>68.1</td>
<td>0.0131</td>
<td>68.2</td>
<td>0.1189</td>
<td>92.8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Transverse Steel</th>
<th>Yield Stress, $f_y$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tests 8-12 (#3 Spiral)</td>
<td>74.1</td>
</tr>
</tbody>
</table>
Chapter 3: Experimental Observations

Figure 3.3 Long, Steel Stress-Strain Curve with Different Measurement Techniques

Figure 3.4 Cyclic Stress-Strain Curve for Longitudinal Reinforcement
3.1.1 Test 9 – Symmetric Three Cycle Set Load History

Table 3.3 Observational Summary for Test 9 – Symmetric Three Cycle Set LH

VALUES OF INTEREST:

Concrete Compressive Strength: \( f'_c = 6814 \text{ psi} \)
Axial Load: \( P = 170 \text{ kips} \)
Analytical First Yield Force: \( F'_y = 46.9 \text{ kips} \)
Experimental First Yield Displacement: \( \Delta'_y = 0.63" \)
Analytical Nominal Moment Capacity: \( M_n = 503.6 \text{ kip} \times \text{ ft} \)
Equivalent Yield Displacement: \( \Delta_y = 0.84" \)
Maximum Lateral Force: \( 70.3 \text{ kips} \)
Failure Mode: Fracture of Previously Buckled Reinforcement

DAMAGE OBSERVATIONS:

First Cracking North: \( 3/4 F y' = 0.40" \)
First Cracking South: \( -3/4 F y' = -0.38" \)
Cover Concrete Crushing North: \( \mu_2^{-2} = -1.67" \)
Cover Concrete Crushing South: \( \mu_2^{+2} = 1.69" \)
Transverse Steel Yield North: At \(-0.22"\) during pull to \( \mu_6^{-1} = -5.05" \)
Transverse Steel Yield South: At \(-3.69"\) during push to \( \mu_6^{+1} = 6.72" \)
Longitudinal Bar Buckling North: Reversal from \( \mu_8^{+1} = 6.72" \)
Longitudinal Bar Buckling South: Reversal from \( \mu_8^{-2} = -6.70" \)
Longitudinal Bar Fracture North: At \(5.18"\) during push to \( \mu_{10}^{+1} = 8.38" \)
Longitudinal Bar Fracture South: At \(-4.56"\) during pull to \( \mu_{10}^{-2} = -8.42" \)

*\( \mu_8^{+1} = 6.64" \) represents the first push cycle of displacement ductility eight
### Table 3.4 Strain Data Summary for Test 9 – Symmetric Three Cycle Set

<table>
<thead>
<tr>
<th>MATERIAL STRAINS:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North:</td>
<td>$\varepsilon_s = 0.0041$ (compression)</td>
</tr>
<tr>
<td>Cover Concrete Crushing South:</td>
<td>$\varepsilon_s = 0.0032$ (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield North:</td>
<td>$\varepsilon_s = 0.0139$ (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield South:</td>
<td>$\varepsilon_s = 0.0163$ (compression)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North:</td>
<td>$\varepsilon_s = 0.053$ (peak tension prior to $bb$)</td>
</tr>
<tr>
<td></td>
<td>$\varepsilon_s = 0.018$ (peak comp. prior to $bb$)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South:</td>
<td>$\varepsilon_s = 0.051$ (peak tension prior to $bb$)</td>
</tr>
<tr>
<td></td>
<td>$\varepsilon_s = 0.015$ (peak comp. prior to $bb$)</td>
</tr>
<tr>
<td>Mander (1988) Ultimate Concrete Compression Strain, $\varepsilon_{cu} = 0.0175$</td>
<td></td>
</tr>
</tbody>
</table>

**Figure 3.5** T9 – Cross Section Bar Designation – North in Tension for Push Cycles
Chapter 3: Experimental Observations

Figure 3.6 T9 – Symmetric Three Cycle Set Load History

Figure 3.7 T9 – Force vs. Deformation Hysteretic Response
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Figure 3.8 T9 – Compressive Axial Load from Each Jack, Total = 2xValue

Figure 3.9 T9 – Two Optotrak Position Sensors with Vertical Cover Blockout Strips
Test 9 Symmetric Three Cycle Set – Experimental Observations:

The first yield force for the tested material and geometric properties was determined using moment curvature analysis (Cumbia $F'_y = 46.9 \text{kips}$ with $f'_c = 6814 \text{psi}$). The initial elastic portion of the symmetric three cycle set load history contains reversals of loading at $\frac{1}{4}F'_y$, $\frac{1}{2}F'_y$, $\frac{3}{4}F'_y$, and $F'_y$. After the specimen has reached the first yield force in each direction, the first yield displacement is obtained as an average ($\Delta'_y = 0.63''$). The equivalent yield displacement, used to determine the displacement ductility levels ($\mu_{\Delta n} = n \times \Delta_y$), is then calculated as $\Delta_y = \Delta'_y(M_n/M'_y) = 0.84''$. The symmetric three cycle set load history continues with three complete cycles at each ductility level, as shown in Figure 3.6. The resulting lateral force vs. top column displacement response appears in Figure 3.7. The compressive axial load applied by one of the two self-regulating hydraulic jacks placed above the loading cap is shown in Figure 3.8. Since the pressure in the two jacks is equal, the total axial load ($170 \text{kips}$) is obtained by multiplying the recorded value by two. The extreme fiber vertical strain profiles for the initial elastic cycles appear in Figure 3.10.

![Bar S3](image1.png) ![Bar N3](image2.png)

Figure 3.10  T9 – Vertical Strain Profiles to First Yield
The first cracks on the North side of the specimen appeared during the \((3/4Fy' = 0.40")\) push cycle had a measured crack width of 0.1mm and were spaced at approximately 7", Figure 3.11. The first cracks on the on the South side of the specimen measured 0.2mm at approximate 8” spacing during \((-3/4Fy' = -0.38")\). During the first yield cycles the cracks on the North side measured 0.3mm at 6” spacing and the cracks on the South side measured 0.35mm at 5”. The vertical strain profiles in Figure 3.10 show that the yield strain, marked by the gray dashed line, was reached during the first yield push and pull cycles.

![Figure 3.11 T9 – (Left) North and South Cracking during 3/4 Fy’ and (Right) during Fy’](image)

At \((\mu_1^{+1} = 0.84")\) the cracks on the North side of the specimen measured 0.35mm at approximate 6” spacing. The cracks on the South side of the specimen expanded to 0.4mm at 5” during \((\mu_1^{-1} = -0.84")\). During \((\mu_1^{+\frac{1}{2}} = 1.26")\), the cracks on the North measured 0.5mm at 6” spacing. The cracks on the South side of the specimen reached 0.75mm at 5” spacing during \((\mu_{1.5}^{-1} = -1.26")\). The North cracks expanded to 1.6mm at 5” spacing during \((\mu_2^{+1} = 1.69")\) as shown in Figure 3.13. The cracks on the South side of the specimen measured 1.7mm at 5” spacing during \((\mu_2^{-1} = -1.68")\). The cover concrete on both sides of the
specimen showed signs of visible flaking, which precedes crushing, during \( \mu_2^{+2} = 1.69'' \) and \( \mu_2^{-2} = -1.67'' \) as shown in Figure 3.12. During \( \mu_2^{+1} = 2.51'' \), the extent of crushing on the South side of the column reached 17” above the footing and 2.5mm crack widths were measured on the North side of the column. The extent of crushing on the North side of the specimen reached 13” above the footing during \( \mu_3^{-1} = -2.51'' \), as shown in Figure 3.14.

Figure 3.12 T9 – Cover Concrete Flaking (Left) South during \( \mu_2^{+2} = 1.69'' \) and (Right) North during \( \mu_2^{-2} = -1.67'' \)
Figure 3.13 T9 – (Left) North Side during ($\mu^+_2 = 1.69\"$) and (Right) South Crack Pattern during ($\mu^-_2 = -1.68\")

Figure 3.14 T9 – Cover Concrete Crushing (Left) South Side of the Specimen during ($\mu^+_3 = 2.51\"$) and (Right) North Side of the Specimen during ($\mu^-_3 = -2.51\")
The test progressed through ($\mu_8^{+1} = 6.72''$) without incident. The progression of cracking on the shear faces of the column appears in Figure 3.15. As the ductility level increased, the cracks became more numerous, increased in inclination, and linked up with cracks formed during loading in the opposite direction. The North extreme fiber reinforcing bar buckled after reversal from ($\mu_8^{+1} = 6.72''$), as shown in Figure 3.16. Additional North reinforcing bars N2 and N4 buckled after reversal from ($\mu_8^{+2} = 6.71''$). The extreme fiber South reinforcing bar S3 buckled after reversal from ($\mu_8^{-2} = -6.70''$), as shown in Figure 3.17. During ($\mu_1^{+1} = 8.38''$), previously buckled bars N3 and N4 ruptured and bar S2 buckled as shown in Figure 3.18. Two additional North reinforcing bars outside of the instrumented region buckled during ($\mu_1^{-1} = -8.48''$). During ($\mu_1^{+2} = 8.39''$), previously buckled bar N2 ruptured and bars S1 and S4 buckled. The test was concluded after the pull cycle to ($\mu_1^{-2} = -8.42''$) when previously buckled bars S3 and S2 ruptured. Rupture of previously buckled reinforcing bars limited the displacement capacity of the bridge column as shown in Figure 3.20.
Figure 3.15 T9 – Crack Progression with Increasing Ductility Demands
Figure 3.16  T9 – (Left) Buckling of Extreme Fiber Bar N3 during \( \mu_8^{-1} = 6.78" \) and (Right) Additional Buckling of Bars N2 and N4 during \( \mu_8^{-2} = 6.70" \)

Figure 3.17  T9 – (Left) Buckling of Extreme Fiber Bar S3 after Reversal from \( \mu_8^{-2} = -6.70" \) and (Right) Additional deformation in N2, N3, and N4 during \( \mu_8^{-3} = 6.73" \)
Figure 3.18 T9 – Photos during \( \mu_{10}^{+1} = 8.38''\) (Left) Rupture of N3 and N4 and (Right) Buckling of S2 and Additional Deformation in Previously Buckled Bar S3

Figure 3.19 After the Test (Left) South Side and (Right) North Side
Test 9 Symmetric Three Cycle Set – Strain Data Analysis:

South Reinforcement:

The vertical strain profile for north extreme fiber bar N3 placed into tension during push cycles appear in the right half of Figure 3.21. This figure shows both extreme fiber bars on the same graph to illustrate the effects of tension shift on strain profiles. As the hinge rotates about inclined flexural shear cracks, compressive strains are concentrated at the base and tensile strains are fanned out to a greater height following the crack distribution. Near the footing cracks remain effectively horizontal, but above this base section the flexural shear cracks are inclined as shown in Figure 3.15. The effects of tension shift increase as the cracks become more inclined at higher ductility levels. Due to the effects of tension shift, the tensile strains at the beginning of an inclined flexural shear crack do not coincide with the perceived moment demand at that location based on its height above the footing and the
applied lateral load. Since the tensile strains are fanned out over a greater distance, the measured tensile strains above the base section are increased. The initial vertical tensile strain profiles are highly influenced by individual crack locations, but later profiles past displacement ductility three are smoother. The compressive vertical strain profile for north extreme fiber bar N3 during pull cycles appears in the left half of Figure 3.22.

A peak tensile strain of 0.053 was measured 2.50” above the footing at \( \mu_{6}^{+1} = 6.72'' \), before the North extreme fiber bar visibly buckled during the subsequent reversal of load. The relationship between tension strain and displacement for this gage length appears in Figure 3.23. Solid lines represent push cycles to the peak displacement while dashed lines correspond to the subsequent displacement reversal. The tensile strain-displacement relationship matches the moment curvature prediction well for cycles under displacement ductility three for the Priestley, Calvi, and Kowalsky (2007) plastic hinge method abbreviated as PCK (2007) Lp. As the displacement increases, moment curvature analysis begins to over predict the reinforcement tensile strains at an increasing rate. As part of this study, a new equivalent curvature distribution is recommended, the result of which is abbreviated Tensile Lpr in Figure 3.23. The intersection of the dashed unloading line with the vertical axis at zero displacement represents the residual growth strain measured over this gage length. The relationship between compression strain-displacement for the bar N3 gage length centered 4.38” above the footing appears in Figure 3.24. The recorded strains match the PCK (2007) Lp prediction well, with the exception of the second and third pull cycles of ductility six.

The compressive strain profile for bar N3, in Figure 3.25, shows that the compressive strains measured 4.38” above the footing increased with each additional cycle during displacement ductility six. This observation, combined with lower strains measured over the first gage length during these cycles suggests measurable deformation occurred before bar buckling. Six spiral layers closest to the footing-column interface were instrumented with strain gages at the location where they overlaid the extreme fiber reinforcement on each side of the specimen. The spiral strains measured on the North side of the specimen appear in
Figure 3.26. The spiral layer 3” above the footing entered the inelastic range during \((\mu_6^{-1} = -5.05")\). During the next two pull cycles of ductility six, the spiral strains continued to rise as the apparent measurable deformation increased. The North extreme fiber bar N3 visibly buckled after reversal from \((\mu_8^{+1} = 6.72")\) at the location of the previously inelastic spiral layer, as shown in Figure 3.16. The inelastic spiral layer, alone, did not lead to bar buckling during ductility six. Instead, the peak tensile strain of 0.053 sustained during \((\mu_8^{+1} = 6.72")\), combined with inelastic transverse steel restraint were sufficient to produce bar buckling upon reversal of load.

The strain hysteresis centered 2.50” above the footing on extreme fiber bar N3 appears in Figure 3.27 with a color bar that represents elapsed time while testing. During the first pull cycle of ductility eight, bar N3 begins to buckle at the location of the data label (X-Displacement, Y-Strain, and Z-Time). During pull cycles the strain in bar N3 should decrease, but the recorded strain begins increasing after the data label due to the outward deformation over the buckled region shown in Figure 3.16. The stain hysteresis also shows a small amount of deformation during each successive pull cycle of ductility six prior to visible buckling. The deformation over the first gage length above the footing causes an increase in strain with each successive cycle while the second gage length contracts causing larger compression strains, as shown in Figure 3.24.

The transverse steel strain hysteresis over the North buckled region appears in Figure 3.28. The transverse steel strain sharply increases upon reversal from the first push cycle of ductility eight, which is marked by the data label. The increase in transverse steel strain occurred before the increase in deformation of longitudinal bar N3 which signified the beginning of visible bar buckling. The measurable deformation in bar N3 during ductility six also caused small increases in the transverse steel strain prior to visible bar buckling.
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Figure 3.21 T9 – Extreme Fiber Vertical Strain Profiles During Push Cycles

Figure 3.22 T9 – Extreme Fiber Vertical Strain Profiles During Pull Cycles
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Figure 3.23  T9 – Tensile Strain-Displacement for Bar N3 during Push Cycles

Figure 3.24  T9 – Compressive Strain-Displacement for Bar N3 during Pull Cycles
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Figure 3.25 T9 – Vertical Strain Profile for Bar N3 with All Cycles during Ductility Six

Figure 3.26 T9 – Transverse Steel Strain for the Lowest Six North Spiral Layers
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Figure 3.27  T9 – Bar N3 Strain Hysteresis, Gage Length Centered 2.50” above Footing

Figure 3.28  T9 – Spiral Strain Hysteresis for North Spiral Layer 4.88” Above Footing
South Reinforcement:

The vertical strain profile for south extreme fiber bar S3 which is placed into tension during pull cycles appears in the right half of Figure 3.22. The compressive strain profiles during push cycles are shown in the left half of Figure 3.21. The extreme fiber South reinforcing bar buckled during the third push cycle of ductility eight after sustaining a tension strain of 0.051 centered 2.95” above the footing at \((\mu_b^{-1} = -6.78")\). The tension strain over the same gage length during \((\mu_b^{-2} = -6.70")\) was 0.050. The compressive vertical strain profile for bar S3 during \((\mu_b^{+2} = 6.71")\) shows measurable deformation 2.95” and 8.47” above the footing before visible buckling occurred in the third push cycle. A peak compressive strain of -0.0177 was measured 4.38” above the footing on South extreme fiber bar S3 during \((\mu_c^{-3} = -5.03")\).

The relationship between tension strain and displacement from when the column was vertical to the peak of significant pull cycles for bar S3, 2.95” above the footing, appears in Figure 3.29. The relationship between compression strain and displacement for push cycles 4.84” above the footing on bar S3 appears in Figure 3.30. Moment curvature analysis with the PCK (2007) Lp hinge method does a good job of predicting the compressive strains, but the tensile strains are over predicted significantly at higher displacements.

The strain hysteresis 2.95” above the footing for extreme fiber south reinforcing bar S3 appears in Figure 3.32. The graph includes a color bar which represents elapsed time while testing to track the progression of the experiment. The strain hysteresis for bar S3 indicates that buckling occurred after reversal from \((\mu_c^{-2} = -6.70")\), which agrees with the test observations. While the South reinforcement should be in compression during the push cycle to \((\mu_b^{+3} = 6.71")\), the outward deformation of bar S3 during bar buckling causes elongation over the Optotrak gage length placed on the outside surface of the bar. The transverse steel strain hysteresis for the spiral layer overlaying the outward buckled region of bar S3 appears in Figure 3.33. A data label shows when the transverse steel strain begins to sharply increase during bar buckling. A similar data label is shown in the strain hysteresis for bar S3 at the
same displacement when the spiral strain began to increase. As extreme fiber bar S3 began to visibly buckle, it placed a larger strain demand on the transverse steel. Measurable deformation occurred before visible buckling of bar S3 during \( \mu_8^2 = 6.71'' \), resulting in increased transverse steel strains.

![Figure 3.29 T9 – Tensile Strain-Displacement for Bar S3 during Pull Cycles](image)

Figure 3.29 T9 – Tensile Strain-Displacement for Bar S3 during Pull Cycles
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Figure 3.30 T9 – Compressive Strain-Displacement for Bar S3 during Push Cycles

Figure 3.31 T9 – Transverse Steel Strain for the Lowest Six South Spiral Layers
Chapter 3: Experimental Observations

Figure 3.32  T9 – Bar S3 Strain Hysteresis, Gage Length Centered 2.95” above Footing

Figure 3.33  T9 – Spiral Strain Hysteresis for South Layer 3.13” Above the Footing
Test 9 – Curvature and Strain Penetration Data:

The cross section curvature profiles are plotted by connecting the measured strains from all six instrumented bars on a given horizontal cross section with a least squared error line. The curvature is then extracted from the slope of the least squared error line, see Figure 3.34. Vertical curvature profiles are plotted for push and pull cycles in Figure 3.35 and Figure 3.36 respectively. These figures show that plastic curvatures have a linear distribution at higher displacement ductility levels. The extent of plastic curvatures above the footing can be calculated by determining where the linear plastic curvature distribution intersects the triangular yield curvature distribution, shown as a grey dashed line. The dashed lines for each curvature distribution represent a least squared error linear fit to the plastic portion of the measured curvatures. The data points used to create the least squared error lines appear as circle data markers. The measured spread of plasticity for Test 9 is shown in Figure 3.44 as a function of base section curvature ductility. The extent of plasticity is computed as the intersection of the linear plastic curvature regression and the elastic curvature profile.

The target marker on each bar placed closest the footing-column interface can be used to create slip hysteresis and horizontal slip profiles attributable to strain penetration. The slip hysteresis for extreme fiber bars N3 and S3 appear in Figure 3.37 and Figure 3.38 respectively. The peak tensile slip of each bar exceeds 0.4in during ductility eight. If the measured slip of all of the instrumented bars is plotted along the cross section depth, the base rotation attributable to strain penetration may be calculated. The slip profiles for push and pull cycles appear in Figure 3.39 and Figure 3.40 respectively. The rotation of the base section can be extracted from the slope of the least squared error line connecting all six measured bar slips.

The displacement at the center of the lateral load may be calculated by combining the measured curvatures over the instrumented region (3ft above the footing), base rotation due to strain penetration, and an elastic curvature assumption above the instrumented region. This process is shown graphically in Figure 3.41. This integrated displacement calculated from the Optotrak system is compared to the measured string potentiometer displacement at
the center of loading in Figure 3.42. The calculated displacements match well over the entire range of response indicating that shear displacements are negligible in comparison to flexural displacements for this column. A bar chart which plots the components of top column displacement for each displacement ductility level appears in Figure 3.43. Strain penetration accounts for between 25-35% of the top column displacement throughout the entire range of response.

Figure 3.34  T9 – Base Section Curvature Profiles during Push Cycles
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Figure 3.35  T9 – Push Cycle Curvature Profiles with Linear Plastic Dashed Regression

Figure 3.36  T9 – Pull Cycle Curvature Profiles with Linear Plastic Dashed Regression
Figure 3.37  T9 – Bar N3 Base Section Slip Hysteresis due to Strain Penetration

Figure 3.38  T9 – Bar S3 Base Section Slip Hysteresis due to Strain Penetration
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Figure 3.39 T9 – Base Section Rotation due to Strain Penetration during Push Cycles

Figure 3.40 T9 – Base Section Rotation due to Strain Penetration during Pull Cycles
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Figure 3.41 T9 – Integration Method for Flexural Displacements

Figure 3.42 T9 – Comparison of String Potentiometer and Integrated Displacements
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Figure 3.43 T9 – Components of Integrated Deformation

Figure 3.44 T9 – Measured Spread of Plasticity (Circle Data Points)
### 3.1.2 Tests 8 and 8b – Chile 2010 Earthquake and Cyclic Aftershock LH

#### Table 3.5 Observational Summary for Test 8 – Chile 2010 Earthquake Load History

**VALUES OF INTEREST:**

- **Concrete Compressive Strength:** \( f_c' = 6988 \text{ psi} \)
- **Axial Load:** \( P = 170 \text{ kips} \)
- **Analytical First Yield Force:** \( F_y' = 47.0 \text{ kips} \)
- **Experimental First Yield Displacement:** \( \Delta_y' = 0.63" \)
- **Analytical Nominal Moment Capacity:** \( M_n = 503.8 \text{ kip} \times \text{ft} \)
- **Equivalent Yield Displacement:** \( \Delta_y = 0.84" \)
- **Maximum Lateral Force:** \( 69.2 \text{ kips} \)
- **Maximum Lateral Displacement:** \( \mu_{26.34 \text{ sec}} = 7.25" \)
- **Failure Mode:** No Significant Damage from Earthquake LH

**DAMAGE OBSERVATIONS:** During Chile 2010 Earthquake LH

- **First Cracking North:** \( \mu_{0.35 \text{ sec}} = 0.21" \)
- **First Cracking South:** \( \mu_{-0.2 \text{ sec}} = -0.20" \)
- **Cover Concrete Crushing North:** *During cycle to \( \mu_{-4.0 \text{ sec}} = -4.03" \)
- **Cover Concrete Crushing South:** \( \mu_{2.92 \text{ sec}} = 2.42" \)
- **Transverse Steel Yield North:** At \(-0.34" \) otwt \( \mu_{26.90 \text{ sec}} = -2.65" \)
- **Transverse Steel Yield South:** At \(5.98" \) otwt \( \mu_{8.7} = 7.25" \)

\( \mu_{8.7} = 7.25" \) represents a push cycle 26.34 seconds into the earthquake load history which reached a peak displacement of 7.25” and a displacement ductility of 8.7
### Table 3.6 Observational Summary for Test 8b – Symmetric Three Cycle Set Aftershock LH

<table>
<thead>
<tr>
<th>DAMAGE OBSERVATIONS: During Symmetric Three Cycle Set Post Earthquake LH</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal Bar Buckling North: Reversal from $\mu_8^{+1} = 6.64''$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South: Reversal from $\mu_8^{-1} = -6.65''$</td>
</tr>
<tr>
<td>Failure Mode: Specimen Saved as a Repair Candidate after Each Extreme Fiber Longitudinal Bar Buckled</td>
</tr>
</tbody>
</table>

$^{*}\mu_8^{+1} = 6.64''$ represents the first push cycle of displacement ductility eight

### Table 3.7 Strain Data Summary for Test 8 and 8b – Chile 2010 Earthquake and Cyclic Aftershock LH

<table>
<thead>
<tr>
<th>MATERIAL STRAINS:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North: N/A, During cycle to $\mu_{-4.0}^{9.69\text{sec}} = -4.03''$</td>
</tr>
<tr>
<td>Cover Concrete Crushing South: $\epsilon_s = 0.006$ (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield North: N/A, Spiral Yielded during reversal from peak tensile. Not a function of compression strain.</td>
</tr>
<tr>
<td>Transverse Steel Yield South: $\epsilon_s = 0.0183$ (compression)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North: $\epsilon_s = 0.051$ (<em>peak tension prior to $bb$</em>) $\epsilon_s = 0.013$ (<em>peak comp. prior to $bb$</em>)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South: $\epsilon_s = 0.048$ (<em>peak tension prior to $bb$</em>) $\epsilon_s = 0.032$ (<em>peak comp. prior to $bb$</em>)</td>
</tr>
<tr>
<td>Mander (1988) Ultimate Concrete Compression Strain, $\epsilon_{cu} = 0.0172$</td>
</tr>
</tbody>
</table>
Figure 3.45  T8 – Chile 2010 Earthquake Load History

Figure 3.46  T8 – Chile 2010 Lateral Force vs. Top Column Displacement Response
Figure 3.47  T8b – Symmetric Three Cycle Set Aftershock Load History

Figure 3.48  T8b – Cyclic Aftershock Hysteretic Response
Figure 3.49  T8 and T8b – Hysteretic Response with Elapsed Time Color Bar

Figure 3.50  T8 and T8b – Compressive Axial Load from One Jack (Total = 2*Value)
Test 8 Chile 2010 Earthquake Load History:

Fiber based analytical modeling in OpenSees was used to determine the top column displacement history using a scaled version of the 2010 Chile earthquake. The acceleration values from the Chile time history were scaled until the peak displacement was equal to 7.25” as shown in Figure 3.45. This peak displacement was chosen based on engineering judgment and the results of the first six specimens. For the first six tests, a displacement ductility of ten produced buckling for specimens with transverse steel spacing in the plastic hinge region closest to 2”. The Chile displacement history includes a large amount of high ductility reversals before the peak cycle. With the exception of the peak displacement cycle, the load history is symmetric with similar ductility demands in each direction.

The analytical displacement history has a series of small cycles within the first eight seconds. The experimental load history began during the first cycle which exceeded the displacement at half yield from Test 7 (same as Test 9). Since the concrete begins to crack around half yield, leaving out smaller cycles at the beginning of the load history should not impact the response. Every intermediate cycle in the analytical displacement history was matched in the experimental test from 6.95sec to 39.24sec when the last meaningful cycle past ductility one was concluded. The displacement history was recreated in the lab using a displacement controlled quasi-static loading procedure with displacement rates below 6 in/min.

The resulting experimental lateral force vs. displacement response for the Chile 2010 earthquake record appears in Figure 3.46. The Chile load history scaled to a peak displacement of 7.25” was concluded without buckling of reinforcement on either side of the specimen and without any loss of strength. A symmetric three cycle set aftershock study was then conducted to determine when reinforcement buckling would occur in a column with degraded stiffness and strain accumulation, but without loss of strength. The cyclic aftershock load history and hysteretic response appear in Figure 3.47 and Figure 3.48 respectively. The entire response for the column including both load histories is shown in Figure 3.49 with an elapsed time colorbar to track the progression of the experiment.
Test 8 Chile 2010 Earthquake Load History – Experimental Observations:

The first cracks on the North side of the specimen were measured at 0.1mm during $(\mu_{0.3}^{6.95\text{ sec}} = 0.21\text{")}. The cycle annotation represents a push cycle 6.95 second into the Chile load history to 0.21\text{"}, which is equivalent to displacement ductility 0.3. During the next cycle, the South side of the specimen had cracks measuring 0.1mm at $(\mu_{-0.2}^{7.14\text{ sec}} = -0.17\text{")}. Cracks on the South side of the specimen measured 0.75mm at approximate 6\text{"} spacing during $(\mu_{-1.2}^{8.12\text{ sec}} = -1.00\text{")}, see Figure 3.51. Cracks on the North side of the specimen measured 1mm at approximate 5\text{"} spacing during $(\mu_{1.6}^{8.40\text{ sec}} = 1.35\text{")}. The first signs of cover concrete crushing over the bottom 5\text{"} of the South side of the column occurred during $(\mu_{2.9}^{9.17\text{ sec}} = 2.42\text{")}, as shown in Figure 3.52, while cracks on the tension side of the column were measured at 1/8\text{"} with approximate 4\text{’’} spacing. Crushing of the cover concrete on the North side of the specimen extended 15\text{”} above the top of the footing during $(\mu_{-4.0}^{9.59\text{ sec}} = -3.35\text{") while cracks on the tension side measured 1/8\text{’’}, Figure 3.52. Crushing on the south side of the specimen reached 12\text{”} above the footing during $(\mu_{2.2}^{14.13\text{ sec}} = -1.84\text{")}, Figure 3.53. The crack Distribution on the front shear face of the specimen during the largest pull cycle to $(\mu_{3.3}^{18.99\text{ sec}} = -4.43\text{")} appears in Figure 3.53.

The peak displacement of $(\mu_{8.7}^{26.34\text{ sec}} = 7.25\text{")} from the scaled Chile 2010 load history was reached with a lateral force of 69.18 kips. Photos of extreme fiber regions of the column at the peak displacement are shown in Figure 3.54. During subsequent reversals of loading the reinforcement remained visibly straight. Bar buckling or rupture did not occur, therefore, the load history was completed with degraded stiffness but no large losses in strength.
Figure 3.51 T8 – Crack Patterns on the (Left) South Side at \( \mu_{12}^{8.12 \text{ sec}} = -1.00'' \) and (Right) North Side at \( \mu_{1.6}^{8.40 \text{ sec}} = 1.35'' \)

Figure 3.52 T8 – Crushing on the (Left) South Side at \( \mu_{2.9}^{9.17 \text{ sec}} = 2.42'' \) and (Right) North Side at \( \mu_{4.0}^{9.59 \text{ sec}} = -3.35'' \)
Figure 3.53 T8 – (Left) South Crack Distribution at $(\mu_{2.2}^{14.13 \text{ sec}} = -1.84")$ and (Right) Front of the Specimen during $(\mu_{5.3}^{18.99 \text{ sec}} = -4.43")$

Figure 3.54 T8 – Peak Displacement $(\mu_{8.7}^{26.34 \text{ sec}} = 7.25")$ (Left) South & (Right) North
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Test 8 Chile 2010 Earthquake Load History – Strain Data Analysis:

Due to the random cyclic nature of the earthquake load histories, specific observation points along the backbone curve of cyclic response were chosen for data analysis in Figure 3.55. The tensile and compressive vertical strain profiles for bar S3 appear in the right half of Figure 3.57 and the left half of Figure 3.56 respectively. The transverse steel strains in the lowest six spiral layers for the South and North extreme fiber regions in compression appear in Figure 3.58 and Figure 3.59. A peak tension strain of 0.031 was measured 2.02” above the footing on bar S3 during ($\mu_{18.99^{sec}} = -4.42"$). The relationship between tensile strain and displacement for this gage length appears in Figure 3.62. The maximum compression strain of -0.02 in reinforcing bar S3 occurred 5.75” above the footing during ($\mu_{26.34^{sec}} = 7.25"$). The relationship between compression strain and displacement for bar S3 for this gage length appears in Figure 3.63.

Vertical strain profiles for extreme fiber bar N3 appear in Figure 3.56 and Figure 3.57 for push tension strains and pull compression strains respectively. The largest tensile strain of 0.051, located 2.09” above the footing, was measured on bar N3 at ($\mu_{26.34^{sec}} = 7.25"$). The relationship between tensile strain and displacement for this gage length appears in Figure 3.60. The error in strain prediction by moment curvature analysis with the PCK (2007) hinge method becomes larger with increasing displacement. The largest compression strain value of -0.013 for extreme fiber bar N3 occurred 5.85” above the footing at ($\mu_{18.99^{sec}} = -4.42"$). The relationship between compression strain and displacement for significant pull cycles, 5.85” above the footing, for bar N3 appears in Figure 3.61.
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Figure 3.55 T8 – Strain Data Observation Points along the Envelope Curve

Figure 3.56 T8 – Extreme Fiber Vertical Strain Profiles during Push Cycles
Figure 3.57 T8 – Extreme Fiber Vertical Strain Profiles during Pull Cycles

Figure 3.58 T8 – Transverse Steel Strains over the South Extreme Fiber Bar S3
Figure 3.59  T8 – Transverse Steel Strains over the North Extreme Fiber Bar N3

Figure 3.60  T8 – Tension Strain-Displacement for Bar N3 during Push Cycles
Figure 3.61 T8 – Compression Strain-Displacement for Bar N3 during Pull Cycles

Figure 3.62 T8 – Tensile Strain-Displacement for Bar S3 during Pull Cycles
Test 8 Chile 2010 Earthquake Load History – Strain Penetration and Curvature Data:

The vertical strain profiles for observation points along the backbone curve of cyclic response, see Figure 3.55, for push and pull cycles appear in Figure 3.64 and Figure 3.65. The slip hysteresis for extreme fiber bars N3 and S3 due to strain penetration of the reinforcement into the footing appear in Figure 3.78 and Figure 3.79 respectively. The slip hystereses contained data from the Chile and Cyclic Aftershock load histories up until each reinforcing bar buckled. The base section rotation attributable to strain penetration of reinforcing bars appears in Figure 3.66 and Figure 3.67 for push and pull cycles respectively. The total deformation calculated by integrating the measured curvature profiles and extrapolating the base section rotation to the center of loading appear in Figure 3.68. The integrated curvatures match well throughout the entire range of response. Circle data points in Figure 3.69 track the spread of plasticity as a function of curvature ductility for the measured curvature profiles.
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Figure 3.64 T8 – Push Cycle Curvature Profiles with Linear Plastic Dashed Regression

Figure 3.65 T8 – Pull Cycle Curvature Profiles with Linear Plastic Dashed Regression
Figure 3.66 T8 – Base Section Rotation due to Strain Penetration during Push Cycles

Figure 3.67 T8 – Base Section Rotation due to Strain Penetration during Pull Cycles
Figure 3.68  T8 – Comparison of Integrated and Measured Lateral Displacements

Figure 3.69  T8 – Measured Spread of Plasticity (Circle Data Points)
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**Test 8b Symmetric Three-Cycle-Set Aftershock LH – Experimental Observations:**

Since the Chile load history scaled to a peak displacement of 7.25” was concluded without buckling reinforcement on either side of the specimen, a symmetric three-cycle-set aftershock study was then conducted to determine when buckling would occur to the column with degraded stiffness and accumulated strains. The top column displacement history and resultant force vs. displacement response for the cyclic aftershock study appear in Figure 3.47 and Figure 3.48 respectively. The complete hysteretic response for Tests 8 and 8b appears in Figure 3.49 with a color bar which represents elapsed time during the experiment. Since the largest cycle in the Chile load history exceeded the peak displacement of the aftershock study in the push direction, there is more strength degradation in the push direction of loading.

The symmetric three-cycle-set load history progressed through ductility six without incident. After the North reinforcement was exposed to tension during \((\mu^+_8 = 6.64”)\), the extreme fiber bar N3 buckled over the first and second gage lengths during the subsequent reversal, as shown in Figure 3.70. Remember that the North reinforcement had already been subjected to larger displacements placing the bars in tension during \((\mu^{26.34}_8 = 7.25”)\) in the Chile load history. After being exposed to tension during \((\mu^-_8 = -6.65”)\), extreme fiber bar S3 buckled over the first and fourth gage lengths as shown in Figure 3.70. The experiment was concluded with buckled reinforcement on each side of the specimen to save the column as a repair candidate. A photo of the damaged regions of the column after removal of the instrumentation appears in Figure 3.71.
Figure 3.70 T8b – (Left) Buckling of Bar N3 after Reversal from ($\mu^{-1}_b = 6.64''$) and (Right) Buckling of Bar S3 after Reversal from ($\mu^{-1}_b = 6.65''$)

Figure 3.71 T8b – (Left) Buckling of N3 after Experiment and (Right) Buckling of S3
Test 8b Symmetric Three-Cycle-Set Aftershock LH – Strain Data Analysis:

Extreme fiber vertical strain profiles for push and pull cycles of the symmetric three cycle set aftershock load history are shown in Figure 3.72 and Figure 3.73 respectively. The strain profiles shape is controlled by the crack distribution set in place during high ductility cycles of the original Chile load history. The compressive vertical strain profiles for each extreme fiber reinforcing bar show significant deformation prior to visible bar buckling. If the reinforcing bar where to outwardly deform, the gage length over the deformation would increase in tensile strain while the gage lengths above and below would further increase in compressive strain. The strain values measured when this deformation occurred do not represent engineering strains, but they are shown to highlight the progression of damage.

South Reinforcement:

Visible buckling of Bar S3 was not observed until the second push cycle of ductility eight. The South reinforcing bar buckled over the first and fourth gage lengths, see Figure 3.71, which matches the problematic areas of the compressive vertical strain profile in Figure 3.72. The tension strain sustained by bar S3 prior to visible buckling during the aftershock study was 0.048, for the gage length 2.02” above the footing, during ($\mu_{8^{-1}} = 6.65$”). The largest strain sustained by bar S3 during the Chile load history was 0.032 located 2.02” above the footing at ($\mu_{18.99 sec} = -4.42$”).

The complete strain hysteresis for extreme fiber bar S3 appears in Figure 3.76 for the gage length 7.76” above footing in the upper buckled region. The strain hysteresis obtained from a strain gage located on the transverses steel overlaying the upper buckled region of bar S3 appears in Figure 3.77. The strains in bar S3 increase during each successive push cycle of ductility six during the aftershock study even though visible buckling was not observed. For the second and third push cycles of ductility six the peak strain increases with each successive cycle indicating measureable deformation prior to buckling. The trend continues as the first push cycle of ductility eight produces an even larger tensile strain in the South reinforcement even though this region should be in compression during push cycles. The
tension strain during push cycles becomes much larger after reversal from \((\mu_8^{-1} = 6.65\text{")})
which coincides with visible buckling of bar S3. When buckling occurs, the reinforcing bar places additional
demand on the transverse steel, which can be seen in Figure 3.77. During each successive cycle of ductility
six the measured strains on the transverse steel in the upper buckled region of bar S3 become larger. During
the first push cycle of ductility eight, prior to visible buckling, the strain in the transverse steel sharply
increases to the point where the strain gage goes off scale preventing further measurement. The longitudinal
and transverse strain hystereses show that buckling may be a more gradual process with measurable
def ormation prior to visible buckling.

*North Reinforcement:*

The extreme fiber bar N3 was exposed to 0.043 during \((\mu_8^{+1} = 6.64\text{")}) which is less than
the strain which occurred during the largest cycle of the Chile load history 0.051 at \((\mu_8^{26.34 \text{ sec} } = 7.25\text{")}). The compressive strain vertical profile in Figure 3.73 for bar N3 during
pull cycles shows measurable deformation during \((\mu_6^{-3} = -4.99\text{")}) before visible buckling. The complete
strain hysteresis, for the same gage length 4.02” above the footing on bar N3 is shown in Figure 3.74. After each successive pull cycle of ductility six the deformation in the
buckled region of bar N3 increases, as indicated by positive strain when the reinforcement
should be in compression. Similarly, the strain rises sharply after reversal from \((\mu_8^{+1} = 6.64\text{")}) when visible buckling was observed. The transverse steel strain hysteresis over the
buckled region of bar N3 is shown in Figure 3.75. Again, each cycle of ductility six produces a greater strain demand on the transverse steel which is restraining bar N3. After
reversal from \((\mu_8^{+1} = 6.64\text{")}), when the bar visibly buckled, the transverse steel strain gage
goes off scale preventing further measurement.
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Figure 3.72  T8b – Cyclic Aftershock Vertical Strain Profiles during Push Cycles

Figure 3.73  T8b – Cyclic Aftershock Vertical Strain Profiles during Pull Cycles
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Figure 3.74  T8 and T8b – Bar N3 Strain Hysteresis (4.02” Above Footing)

Figure 3.75  T8 and T8b – Spiral Strain Hysteresis over North Buckled Region
Figure 3.76 T8 and T8b – Bar S3-4 Strain Hysteresis (7.76” Above Footing)

Figure 3.77 T8 and T8b – Spiral Strain Hysteresis over South Buckled Region
Figure 3.78  T8 and T8b – North Extreme Fiber Bar N3 Slip Hysteresis

Figure 3.79  T8 and T8b – South Extreme Fiber Bar S3 Slip Hysteresis
3.1.3 Tests 10 and 10b – Chichi Earthquake and Cyclic Aftershock LH

Table 3.8 Observational Summary for Test 10 – Chichi Earthquake Load History

<table>
<thead>
<tr>
<th>VALUES OF INTEREST:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Compressive Strength: ( f'_c = 5263) psi</td>
</tr>
<tr>
<td>Axial Load:</td>
</tr>
<tr>
<td>( P = 170) kips</td>
</tr>
<tr>
<td>Analytical First Yield Force: ( F'_y = 45.9) kips</td>
</tr>
<tr>
<td>Experimental First Yield Displacement: ( \Delta'_y = 0.62&quot; ) (Same as Test 9)</td>
</tr>
<tr>
<td>Analytical Nominal Moment Capacity: ( M_n = 505.6) kip * ( ft )</td>
</tr>
<tr>
<td>Equivalent Yield Displacement: ( \Delta_y = 0.84&quot; )</td>
</tr>
<tr>
<td>Maximum Lateral Force: ( 70.6) kips</td>
</tr>
<tr>
<td>Maximum Lateral Displacement: ( \mu_{8.9\ sec}^{17.31} = 7.40&quot; )</td>
</tr>
<tr>
<td>Failure Mode:</td>
</tr>
<tr>
<td>No Significant Damage from Earthquake LH</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DAMAGE OBSERVATIONS: During Chichi Earthquake LH</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Cracking North: ( \mu_{0.3\ sec}^{7.02\ sec} = 0.25&quot; )</td>
</tr>
<tr>
<td>First Cracking South: ( \mu_{-0.27\ sec}^{6.80\ sec} = -0.22&quot; )</td>
</tr>
<tr>
<td>Cover Concrete Crushing North: ( \mu_{-1.7\ sec}^{13.72\ sec} = -1.39&quot; )</td>
</tr>
<tr>
<td>Cover Concrete Crushing South: ( \mu_{2\ sec}^{13.40\ sec} = 1.70&quot; )</td>
</tr>
<tr>
<td>Transverse Steel Yield South: ( \text{At 4.47&quot; owt } \mu_{8.9\ sec}^{17.31\ sec} = 7.40&quot; )</td>
</tr>
</tbody>
</table>

\( *\mu_{8.9\ sec}^{17.31\ sec} = 7.40" \)represents a push cycle 17.31 seconds into the earthquake load history which reached a peak displacement of 7.40” and a displacement ductility of 8.9
Table 3.9 Observational Summary for Test 8b – Symmetric Three Cycle Set Aftershock LH

<table>
<thead>
<tr>
<th>Damage Observations:</th>
<th>During Symmetric Three Cycle Set Post Earthquake LH</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse Steel Yield North:</td>
<td>At $-4.57''$ otwt $\mu_6^{-3} = -4.98''$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North:</td>
<td>No Visible Buckling Observed</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South:</td>
<td>Reversal from $\mu_6^{-1} = -5.01''$</td>
</tr>
<tr>
<td>Failure Mode:</td>
<td>Specimen Saved as a Repair Candidate with Buckled South Reinforcement</td>
</tr>
</tbody>
</table>

$\mu_6^{-1} = -5.01''$ represents the first pull cycle of displacement ductility six

Table 3.10 Strain Data Summary for Test 8 and 8b – Chile 2010 Earthquake and Cyclic Aftershock LH

<table>
<thead>
<tr>
<th>Material Strains:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North: $\varepsilon_s = 0.0026$ (compression)</td>
</tr>
<tr>
<td>Cover Concrete Crushing South: $\varepsilon_s = 0.0039$ (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield North: $\varepsilon_s = 0.0092$ (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield South: $\varepsilon_s = 0.0151$ (compression)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North: No Visible Buckling Observed</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South: $\varepsilon_s = 0.038$ ($peak$ $tension$ $prior$ $to$ $bb$) $\varepsilon_s = 0.045$ ($peak$ $comp$. $prior$ $to$ $bb$)</td>
</tr>
<tr>
<td>Mander (1988) Ultimate Concrete Compression Strain, $\varepsilon_{cu} = 0.0204$</td>
</tr>
</tbody>
</table>
Chapter 3: Experimental Observations

Figure 3.80 T10 – Chichi Earthquake Displacement History

Figure 3.81 T10 – Hysteretic Response for the Chichi Earthquake Load History
Figure 3.82 T10b – Symmetric Three Cycle Set Aftershock Load History

Figure 3.83 T10b – Symmetric Three Cycle Set Aftershock Hysteretic Response
Figure 3.84  T10 and T10b – Complete Response with an Elapsed Time Color Bar

Figure 3.85  T10 and T10b – Compressive Axial Load from One Jack (Total = 2*Value)
Test 10 Chichi Earthquake Load History – Experimental Observations:

To determine possible effects of different load history characteristics on the relationship between strain and displacement, an asymmetric displacement history from the 1999 Chichi Earthquake in Taiwan was used. The Chichi record, see Figure 3.80, produced a one sided response with a displacement ductility demand of 8.9 in one direction of loading and a ductility demand of only 2.5 in the opposing direction. The resulting lateral force vs. top column displacement response appears in Figure 3.81. Buckling did not occur during the Chile or Chichi load histories even though the peak displacements exceeded ductility eight which produced buckling during the symmetric three cycle set load history of Test 9. The purpose of running the Chichi load history was to determine if the asymmetric characteristic has an impact on the relationship between strain and displacement. The asymmetric load history produces significantly different tensile demands on the North and South sides of the specimen.

Cracks measuring 0.1mm at approximate 6” spacing on the South side of the specimen first occurred at \( \mu_{-0.3}^{sec} = -0.22" \). On the North side of the specimen cracks measuring 0.1mm at approximate 9” spacing where observed at \( \mu_{0.3}^{7.02 sec} = 0.25" \). The cracks on the North side of the specimen increased to 0.3mm at approximate 8” spacing during \( \mu_{0.6}^{7.90 sec} = 0.49" \). The cracks on the South side of the specimen increased to 0.3mm at approximate 5” spacing during \( \mu_{-0.7}^{9.06 sec} = 0.60" \). Crushing of the cover concrete 8” above the footing on the South side of the specimen began during \( \mu_{2.0}^{13.40 sec} = 1.70" \) while cracks on the North side of the specimen measured 1.25mm at approximate 4” spacing, Figure 3.86. The cover concrete on the North side of the specimen crushed 5” above the footing during \( \mu_{-1.7}^{13.72 sec} = -1.39" \), as shown in Figure 3.87. The extent of crushing on the South side of the specimen extended 15” above the footing during \( \mu_{2.6}^{15.18 sec} = 2.20" \), Figure 3.88. The peak cycle of the load history at \( \mu_{6.9}^{7.31 sec} = 7.40" \), with a lateral force of 69.98 kips, was completed without additional visible damage, Figure 3.89 and Figure 3.90. Bar Buckling did not occur in subsequent cycles of the Chichi earthquake load history.
Figure 3.86 T10 – (Left) South Concrete Crushing ($\mu_{2.0}^{13.40\ sec} = 1.70''$), (Right) North Side

Figure 3.87 T10 – (Left) North Crushing ($\mu_{-1.7}^{13.72\ sec} = -1.39''$), (Right) South Side
Figure 3.88 T10 – (Left) South ($\mu_{2.7 \text{ sec}}^{15.18} = 2.23''$), (Right) ($\mu_{5.8 \text{ sec}}^{16.02} = 4.85''$)

Figure 3.89 T10 – Specimen at the Peak Disp. of the Chichi EQ ($\mu_{8.9 \text{ sec}}^{17.31} = 7.40''$)
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Test 10 Chichi Earthquake Load History – Strain Data Analysis:

Due to the random cyclic nature of the Chichi earthquake load history, specific data observation points along the backbone curve of cyclic response were chosen in Figure 3.91. The vertical strain profiles for each extreme fiber bar during push and pull cycles are shown in Figure 3.92 and Figure 3.93. The strain profiles for cycles exceeding displacement ductility three are smoother and are influenced less by individual crack locations. The maximum recorded compression strain in the extreme fiber bar S3 during \(\mu_{8.917.31\text{sec}} = 7.40"\) was 0.032 measured 7.64” above the footing. A peak tensile strain of 0.052, centered 4.33” above the footing, was measured on bar N3 during the push cycle to \(\mu_{8.917.3\text{sec}1} = 7.40"\). Tests 8, 9, and 10 (Chile 2010, Symmetric Three Cycle Set, and Chichi) were

Figure 3.90 T10 – (Left) North and (Right) South at \(\mu_{8.917.31\text{sec}} = 7.40"\)
subjected to similar values of peak tensile strain (0.051, 0.053, and 0.052 respectively) at different levels of displacement ductility (8.7, 8, and 8.9 respectively), but buckling only occurred during the Symmetric Three Cycle Set load history of Test 9.

The relationship between tensile strain and displacement for North extreme fiber bar N3, centered 4.33” above the footing, appears in Figure 3.94. The Cumbia moment curvature analysis with the PCK (2007) Lp hinge method prediction significantly over predicts the tensile strains at higher displacements. The relationship between compressive strain and displacement for extreme fiber bar N3 during significant pull cycles appears in Figure 3.95. The ductility demands in the pull direction after the peak cycle were not large enough to place the North reinforcement back into compression due to the large residual growth strains.

The relationship between compressive strain and displacement, for gage length centered 1.82” above the footing on extreme fiber bar S3, from when the column was vertical to the peak of significant push cycles appears in Figure 3.97. The moment curvature prediction with the PCK (2007) Lp hinge method for compressive strains matches the recorded strains well. The graph shows compressive strains over the first gage length above the footing, even though measured strains in the fourth gage length were larger. The recorded strains during the $\mu_{8.9}^{17.31sec} = 7.40”$ push cycle exceed the moment curvature prediction. Strains recorded during later cycles of the load history are similarly under predicted by moment curvature analysis. The relationship between tensile strain and displacement for extreme fiber bar S3 placed into tension during pull cycles appears in Figure 3.96.
Figure 3.91  T10 – Strain Data Observation Points along the Backbone Curve

Figure 3.92  T10 – Extreme Fiber Vertical Strain Profiles During Push Cycles
Figure 3.93 T10 – Extreme Fiber Vertical Strain Profiles During Pull Cycles

Figure 3.94 T10 – Tensile Strain-Displacement for Bar N3 during Push Cycles
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Figure 3.95 T10 – Compressive Strain-Displacement for Bar N3 during Pull Cycles

Figure 3.96 T10 – Tensile Strain-Displacement for Bar S3 during Pull Cycles
Test 10 Chichi Earthquake Load History – Strain Penetration and Curvature Data:

Vertical curvature profiles obtained for points along the backbone curve of cyclic response during push and pull cycles appear in Figure 3.98 and Figure 3.99 respectively. Linear plastic curvature least squared error lines show that the curvatures are linearly distributed after displacement ductility three when the profiles smooth out. Initial cycles below ductility three are highly influenced by individual crack locations. The base section rotation attributable to strain penetration of reinforcing bars appears in Figure 3.100 and Figure 3.101 for push and pull cycles respectively. The total deformation calculated by integrating the measured curvature profiles and extrapolating the base section rotation to the center of loading appear in Figure 3.102. The integrated curvatures match well throughout the entire range of displacements. The measured spread of plasticity as a function of curvature ductility appears in Figure 3.103.
Figure 3.98 T10 – Vertical Curvature Profiles during Push Cycles

Figure 3.99 T10 – Vertical Curvature Profiles during Pull Cycles
Figure 3.100  T10 – Base Section Rotation due to Strain Penetration during Push Cycles

Figure 3.101  T10 – Base Section Rotation due to Strain Penetration during Pull Cycles
Figure 3.102 T10 – Comparison of Measured and Optotrak Integrated Displacements

Figure 3.103 T10 – Measured Spread of Plasticity (Circle Data Points)
Test 10b Symmetric Three-Cycle-Set Aftershock LH – Experimental Observations:

Since buckling did not occur during the Chichi load history, a second symmetric three cycle set load history was conducted on the specimen with degraded stiffness but no losses in strength, similar to Test 8b conducted after the Chile 2010 load history. The extreme fiber South reinforcement bar S3 buckled after reversal from \((\mu^\text{6-1} = -5.01")\), Figure 3.104. Due to the asymmetric nature of the Chichi load history, the South side of the specimen was subjected to low tensile demands but high compressive demands during the peak displacement cycle to \((\mu^{17.31\text{sec}} = 7.40")\). The purpose of the cyclic aftershock study shifted to determine if continued cycling at ductility six would rupture the previously buckled South reinforcement. After reversal from \((\mu^\text{6-3} = -4.98")\), bar S2 buckled as shown in Figure 3.104. Six complete cycles of ductility six where completed without rupturing previously buckled reinforcement on the South side of the specimen, so the load history continued to ductility eight as shown in Figure 3.82. As the load history progressed, visible deterioration of the core concrete on the South side of the specimen over the buckled region occurred due to loss of confinement, which is evident in the left photo in Figure 3.105 taken at \((\mu^\text{6+1} = 6.64")\).

Additionally, South reinforcement bar S4 buckled during \((\mu^\text{6+1} = 6.64")\). Previously buckled reinforcing bar S3 ruptured in tension during \((\mu^\text{6-1} = -6.63")\). Bar S1 buckled during \((\mu^\text{5+2} = 6.62")\), and previously buckled Bar S2 ruptured during \((\mu^\text{5+2} = -6.67")\). At \((\mu^\text{5+3} = 6.63")\), the fifth reinforcing bar on the South side of the specimen buckled. Previously buckled reinforcing bar S4 ruptured during \((\mu^\text{5+3} = -6.67")\). The test was concluded with five buckled bars on the South side of the specimen and intact reinforcing bars on the North side of the specimen, see Figure 3.106 and Figure 3.105 for photos of South and North sides of the specimen after testing.

The complete hysteretic response for Tests 10 and 10b is shown in Figure 3.84 with an elapsed time color bar to track the progression of the response through both load histories. Buckling of multiple bars during ductility six produced minimal losses in strength during pull
cycles when previously buckled reinforcement was placed into tension, but push cycles of ductility six did not suffer from losses in strength. During each pull cycle of ductility eight, a previously buckled bar on the South side of the specimen ruptured leading to losses of strength in both the push and pull directions of loading. When the response from Test 10b is compared to the moment curvature prediction, in Figure 3.83, it is clear that there is a larger amount of stiffness degradation at lower ductility cycles in the push direction of loading due to the original asymmetric Chichi load history. The hysteretic response for Tests 9 and 10b are shown in Figure 3.107, and the response for Tests 8b and 10b are compared in Figure 3.108. The hysteretic response for Tests 8b and 10b are similar up to ductility six except test 10b has larger forces in the pull direction due to lower stiffness degradation during the asymmetric Chichi load history compared to the symmetric Chile 2010 load history of Test 8.

Figure 3.104 T10b – (Left) Buckling of Bar S3 after Reversal from \( \mu_6^{-1} = -5.01" \) and (Right) Buckling of S2 after Reversal from \( \mu_6^{-3} = -4.98" \)
Figure 3.105 T10b – (Left) South at ($\mu_8^{+1} = 6.64''$) and (Right) North Side after Test

Figure 3.106 T10b – South Side after the Test (5 Buckled and 3 Ruptured Bars)
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Figure 3.107  T10b – Cyclic Aftershock Response Compared to Initially Undamaged T9

Figure 3.108  Comparison of Post-Earthquake Cyclic Response of T8b and T10b
Test 10b Symmetric Three-Cycle-Set Aftershock LH – Strain Data Analysis:

South Reinforcement:

The extreme fiber vertical strain profiles for push and pull cycles of the cyclic aftershock load history are shown in Figure 3.109 and Figure 3.110 respectively. The compressive vertical strain profiles for Bar S3, left half of Figure 3.109, indicate a large amount of measurable deformation prior to visible buckling occurred after reversal from \(\mu_6^{-1} = -5.01''\). Tensile strains were measured in the third gage length above the footing, while large compression strains were recorded over adjacent gage lengths above and below, which indicates outward deformation of the reinforcement. Visually, this is supported by the left photo in Figure 3.104 that shows outward deformation over the third gage length when visible buckling was observed during the second push cycle of ductility six. The vertical strain profile shows that measurable deformation over second, third, and fourth gage lengths were recorded over the entire cyclic aftershock test. The measured deformation increased during the fourth ductility level before visible buckling was observed. Once outward deformation of the longitudinal steel occurs, the magnitude of recorded strains is no longer representative of engineering strain. Instead, the vertical strain profiles are shown until visible buckling to highlight the location and propagation of damage.

The complete strain hysteresis for South extreme fiber bar S3 for the gage length centered 5.72” above the footing for Tests 10 and 10b appear in Figure 3.113. This particular gage length captures the outwards deformation of the buckled bar which increases the distance between target markers. The strain hysteresis shows the peak cycle of the Chichi load history with bar S3 in compression, and upon reversal many small ductility cycles failed to place the gage length back into large compression due to the effects of small deformation prior to visible buckling. The recorded strains over the South reinforcement gage length should be in compression after reversal from pull cycles; instead the apparent strain increases as the column is pushed due to the outward deformation. The opposite phenomenon was observed over adjacent gage lengths where increasing compression strains were observed at locations where the bar begins to straighten back out. The final push cycle in Figure 3.113
represents the push to \( \mu_6^{+2} = 4.97'' \) when visible buckling occurred. The recorded data at this stage of the strain hysteresis is affected by measurable deformations prior to buckling.

The strain gage hysteresis for the transverse steel layer overlaying the portion of the extreme fiber south reinforcing bar that later buckled outwards appears in Figure 3.114. After reversal from the peak cycle, the transverse steel maintained a large residual strain over 0.01, even during low ductility cycles. The increased residual strain in the transverse steel affects the column behavior in two distinct ways: (1) Inelastic strains in the transverse steel decrease its effectiveness as a boundary condition restraining buckling of the longitudinal steel, which explains small measurable deformation prior to visible buckling, and (2) Large residual strains in the transverse steel result in reduced confinement of the core concrete which concentrates further damage at that location. Presumably, if the effectiveness of the transverse steel in confining the core concrete was reduced, repeated cycles could lead to deterioration of the core concrete in the localized region critical to reinforcement buckling. If even small regions of the confined core were to crush, the effect of this crushing is analogous to increasing the demand on the longitudinal steel while cracks are closing since the longitudinal steel would be required to maintain compression zone stability until portions of the core concrete were engaged at potentially greater displacements.

North Reinforcement:

The tensile and compressive vertical strain profiles for extreme fiber bar N3 during push and pull cycles appear in Figure 3.109 and Figure 3.110 respectively. A peak tensile strain of 0.048 was measured 4.33” above the footing on bar N3 during \( \mu_8^{+3} = 6.63'' \). This value is lower than the peak tensile strain of 0.052 measured over the same gage length during the original Chichi record at \( \mu_{8.9}^{17.31 \text{ sec}} = 7.40'' \). Initial strain profiles at low ductility levels are strongly influenced by residual tension strains from previous high ductility cycles during the Chichi record. The compression strains for bar N3 up to ductility six follow the same trend with increasing strain at greater displacements with no sign of measurable deformation.
The complete strain hysteresis for extreme fiber north reinforcing bar N3 appears in Figure 3.111 with an elapsed time color bar to follow the test progression. Since the North reinforcement did not buckle during either load history, stable hysteretic loops were observed for the gage length centered 4.33” above the footing. The transverse steel strain gage hysteresis for the spiral layer which experienced the highest tensile strains overlaying the North unbuckled region appears in Figure 3.112. Large transverse steel strains were not recorded until displacement ductility eight of the cyclic aftershock study.

Figure 3.109 T10b – Extreme Fiber Vertical Strain Profiles during Push Cycles (Significant Measurable Deformation in Bar S3)
Figure 3.110  T10b – Extreme Fiber Vertical Strain Profiles during Pull Cycles

Figure 3.111  T10 and T10b – Bar N3 Strain Hysteresis for Gage Length (4.33” Above)
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Figure 3.112  T10 and T10b – Spiral Strain Hysteresis over North “Unbuckled” Region

Figure 3.113  T10 and T10b – Bar S3 Strain Hysteresis for Gage Length 5.72” Above
Strain Gage Off Scale before Visible Buckling of Bar S3

Figure 3.114 T10 and T10b – Spiral Strain Hysteresis over South Buckled Region

Figure 3.115 T10 and T10b – Bar N3 Base Section Slip Hysteresis
Figure 3.116  T10 and T10b – Bar S3 Base Section Slip Hysteresis
Chapter 3: Experimental Observations

3.1.4 Test 11 – Kobe 1995 Earthquake Load History

Table 3.11 Observational Summary for Test 11 – Kobe 1995 Earthquake Load History

<table>
<thead>
<tr>
<th>VALUES OF INTEREST:</th>
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<tbody>
<tr>
<td>Concrete Compressive Strength:</td>
<td>$f_c' = 6070\ psi$</td>
</tr>
<tr>
<td>Axial Load:</td>
<td>$P = 170\ kips$</td>
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<tr>
<td>Analytical First Yield Force:</td>
<td>$F_y' = 46.5\ kips$</td>
</tr>
<tr>
<td>Experimental First Yield Displacement:</td>
<td>$\Delta_y' = 0.62''$</td>
</tr>
<tr>
<td>Analytical Nominal Moment Capacity:</td>
<td>$M_{n} = 495.58\ kip\ast\ ft$</td>
</tr>
<tr>
<td>Equivalent Yield Displacement:</td>
<td>$\Delta_y = 0.83''$</td>
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<tr>
<td>Maximum Lateral Force:</td>
<td>68.0\ kips</td>
</tr>
<tr>
<td>Maximum Lateral Displacement:</td>
<td>$\mu_{3.86sec} = 8.28''$</td>
</tr>
<tr>
<td>Failure Mode:</td>
<td>Specimen Saved as a Repair Candidate</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DAMAGE OBSERVATIONS:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>First Cracking North:</td>
<td>Unknown $\Delta$ during Push to $\mu_{10}^{3.86sec} = 8.28''$</td>
</tr>
<tr>
<td>First Cracking South:</td>
<td>Unknown $\Delta$ during Pull to $\mu_{-1.5}^{3.44sec} = -1.24''$</td>
</tr>
<tr>
<td>Cover Concrete Crushing North:</td>
<td>Unknown $\Delta$ during Pull to $\mu_{-6.1}^{4.42sec} = -5.08''$</td>
</tr>
<tr>
<td>Cover Concrete Crushing South:</td>
<td>Unknown $\Delta$ during Push to $\mu_{10}^{3.86sec} = 8.28''$</td>
</tr>
<tr>
<td>Transverse Steel Yield North:</td>
<td>At 1.47'' during pull to $\mu_{-2.7}^{7.16sec} = -2.22''$</td>
</tr>
<tr>
<td>Transverse Steel Yield South:</td>
<td>At 3.96'' during push to $\mu_{10}^{3.86sec} = 8.28''$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North:</td>
<td>Reversal from $\mu_{5.3}^{6.56sec} = 7.75''$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South:</td>
<td>Reversal from $\mu_{-6.1}^{4.42sec} = -5.08''$</td>
</tr>
</tbody>
</table>

$\mu_{10}^{3.86sec} = 8.28''$ represents a ductility ten push cycle 3.86 seconds into the Kobe EQ LH
Table 3.12 Strain Data Summary for Test 11 – Kobe 1995 Earthquake Load History

<table>
<thead>
<tr>
<th>MATERIAL STRAINS:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North:</td>
<td>N/A, During cycle to $\mu_{-6.1}^{4.42sec} = -5.08''$</td>
</tr>
<tr>
<td>Cover Concrete Crushing South:</td>
<td>N/A, During cycle to $\mu_{10}^{3.86sec} = 8.28''$</td>
</tr>
<tr>
<td>Transverse Steel Yield North:</td>
<td>N/A, Due to Reversal From Tensile Strain</td>
</tr>
<tr>
<td>Transverse Steel Yield South:</td>
<td>$\epsilon_s = 0.0163$ (compression)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North:</td>
<td>$\epsilon_s = 0.059$ (peak tension prior to bb)</td>
</tr>
<tr>
<td></td>
<td>$\epsilon_s = 0.012$ (peak comp. prior to bb)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South:</td>
<td>$\epsilon_s = 0.033$ (peak tension prior to bb)</td>
</tr>
<tr>
<td></td>
<td>$\epsilon_s = 0.037$ (peak comp. prior to bb)</td>
</tr>
<tr>
<td>Mander (1988) Ultimate Concrete Compression Strain, $\epsilon_{cu} = 0.0187$</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.117 T11 – Compressive Axial Load from One Jack (Total = 2*Value)
Chapter 3: Experimental Observations

Figure 3.118 T11 – Kobe 1995 Earthquake Load History

Figure 3.119 T11 – Kobe Earthquake Hysteretic Response
Test 11 – Kobe Earthquake Load History:

The analytical top column displacement history for the scaled Kobe earthquake, see Figure 3.118, was determined using fiber-based numerical simulation in OpenSees. A 1.13x scaled version of the 1995 Kobe Japan earthquake was selected because it contains a near monotonic cycle to the peak displacement ductility of ten in one direction followed by the largest reversal to the peak cycle in the opposing direction of loading. In previous time history based tests, buckling did not occur during the Chile or Chichi records scaled to displacement ductility 8.7 and 8.9 respectively. The results from the asymmetric Chichi record suggest that high ductility cycles can decrease the effectiveness of transverse steel as a boundary condition restraining the longitudinal steel. A peak displacement level consistent with ductility ten was chosen to increase the level of tension strain in the steel to evaluate the steel tensile strain limit. The Kobe displacement history is unique since the peak cycle occurs early, without previous cyclic ramp up in a near monotonic fashion. The resulting lateral force vs. top column displacement history for the Kobe earthquake is shown in Figure 3.119.

The North reinforcement was exposed to a tensile strain of 0.059 during the peak cycle, but did not initially buckle after the first large reversal of loading. Instead, the North extreme fiber bar buckled after the second largest push cycle with elastic transverse steel restraint prior to bar buckling. The transverse steel on the South side of the specimen experienced inelastic strains over 0.015 during the largest push cycle of the load history. Since the transverse steel was less effective as a boundary condition restraining buckling, the South reinforcement buckled after reversal from the largest pull cycle with a tensile strain of only 0.033. The Kobe earthquake points out the effects of load history on the longitudinal steel buckling mechanism.
Test 11 – Kobe Earthquake Load History Experimental Observations:

The first cycle of loading consisted of small pull cycle to ductility 1.5, where cracks were measured at 0.75mm at approximate 5” spacing on the South side of the specimen. The Chile and Chichi records contained a cyclic ramp up to the peak cycle in contrast to the near monotonic push cycle to the peak displacement in the Kobe load history ($\mu_{10} = 8.28\text{"}$). The crack distribution on the North side of the specimen can be seen in the left photo of Figure 3.121, while the extent of crushing on the South side of the specimen appears in the middle photo. During the largest pull cycle at ($\mu_{6.1} = 5.08\text{"}$), the concrete on the North side of the specimen crushed and the reinforcement did not show signs of visible buckling even though large tensile strains occurred during the near monotonic push cycle, Figure 3.121.

The South side of the specimen was exposed to large compressive strains during ($\mu_{10} = 8.28\text{"}$). The dilation of the core concrete caused large strains in the transverse steel which decrease its effectiveness as a boundary condition restraining longitudinal bar buckling during subsequent push cycles. The extreme fiber South reinforcing bar buckled after reversal from the peak cycle in the pull direction ($\mu_{6.1} = 5.08\text{"}$), as shown in the left photo of Figure 3.122. Buckling on the South side of the specimen after reversal from such a low level of displacement required diminished lateral restraint from the transverse steel and sufficient tensile strains to induce buckling upon reversal. The extreme fiber North reinforcing bar visibly buckled after reversal from the second largest push cycle at ($\mu_{6.1} = 5.08\text{"}$), as shown in the right photo of Figure 3.122. Additional deformation in previously buckled bars S3 and N3 occurred during the remainder of the load history as shown in Figure 3.123. The specimen was saved as a repair candidate with a single buckled extreme fiber bar on each side of the specimen, but without significant loss in strength.
Figure 3.120 T12 – Crack Distribution at Peak Displacement ($\mu_{10}^{3.86\, sec} = 8.28''$)

Figure 3.121 T11 – (Left and Middle) North and South Sides of Specimen during ($\mu_{10}^{3.86\, sec} = 8.28''$) and (Right) North Side of the Specimen during ($\mu_{-6.1}^{4.42\, sec} = -5.08''$)
Figure 3.122 T11 – (Left) Buckling of Bar S3 after Reversal from $(\mu_{-6.1}^{44.2\sec} = -5.08")$ and (Right) Buckling of Bar N3 after Reversal from $(\mu_{9.3}^{5.6\sec} = 7.75")$

Figure 3.123 T11 – Additional Deformation in the (Left) North and (Right) South Buckled Regions
Test 11 – Kobe Earthquake Load History Strain Data Analysis

North Reinforcement

The extreme fiber vertical strain profiles for push and pull cycles appear in Figure 3.124 and Figure 3.125. The vertical strain profiles in the push direction are all from the backbone curve of the near monotonic push cycle which occurred 3.86 seconds into the Kobe load history, the data observation points appear in Figure 3.140. The lowest gage lengths on each side of the specimen were blocked by debris for most of the push cycle. The pull cycle vertical strain profiles mainly show the reversal from the peak displacement, and are therefore highly influenced by residual strains. The spiral layer placed closed to the footing-column interface remained elastic due to the additional confinement provided by the footing. The spiral strains on the North side of the specimen, see Figure 3.127, remained elastic during the peak pull cycle to \(\mu_{6.1} = -5.08"\).

Since bar buckling happened so early into the load history, only a few cycles contain usable strain data. The relationship between tensile strain and displacement for Bar N3 during push cycles before bar buckling is shown in Figure 3.128. Moment curvature analysis with the PCK (2007) Lp hinge method significantly over predicts the measured tensile strains at higher displacements. The relationship between compression strain and displacement for the gage length centered 3.33” above the footing appears in Figure 3.129.

The strain hysteresis for the buckled region of the North extreme fiber bar, 3.33” above the footing, appears in Figure 3.134. The peak tensile strain over the North buckled region is slightly lower than the maximum tensile strain sustained by bar N3 since they occur over different gage lengths. The strain values after reversal from \(\mu_{9.3} = 7.75"\) no longer represent engineering strain since visible bar buckling occurred. After this point, the reinforcement is never placed back into compression, indicating an outward deformation of the reinforcement over this location which matches test observations. The transverse steel strain gage hysteresis for the spiral layer overlaying the North buckled region appears in Figure 3.135. The transverse steel restraining the North reinforcement did not yield until
reversal from \((\mu_{9.3}^{6.56\,sec} = 7.75\text{"})\), which was when visible buckling was observed in the test. Since the transverse steel on the North side of the specimen did not yield during the largest pull cycle, the inelastic spiral layers are attributed to bar buckling.

**South Reinforcement**

The measured compressive strains in bar S3 during the peak cycle, see Figure 3.131, are under predicted by moment curvature analysis with the PCK (2007) Lp hinge method. This is likely due to the inelastic layers of transverse steel in this region. The measured strains in the lowest six spiral layers on the South side of the specimen during the push cycle to \((\mu_{10}^{3.86\,sec} = 8.28\text{"})\) are shown in Figure 3.126. A single layer of transverse steel entered the inelastic range at a displacement ductility of five. The compressive demand continued to increase during the push to \((\mu_{10}^{3.86\,sec} = 8.28\text{"})\) until five layers of transverse steel were inelastic.

The longitudinal steel strain hysteresis over the South buckled region, 7.13" above the footing, appears in Figure 3.132. While the entire strain hysteresis is shown, only the data before buckling occurred, upon reversal from \((\mu_{-6.1}^{4.42\,sec} = −5.08\text{"})\), represents engineering strains. This particular gage length was over the outward buckled region of the bar that expands during buckling. This explains the erroneous tensile strains measured during a cycle which should have placed the reinforcement in compression. The transverse steel strain gage hysteresis over the South buckled region appears in Figure 3.133. Transverses steel strains over 0.015 were measured during the peak push cycle to \((\mu_{10}^{3.86\,sec} = 8.28\text{"})\). The measured spiral strains sharply increase after reversal from \((\mu_{-6.1}^{4.42\,sec} = −5.08\text{"})\) when visible buckling was observed. The strain gage quickly goes off scale and no longer provides meaningful data.
Figure 3.124 T11 – Extreme Fiber Vertical Strain Profiles during Push Cycles

Figure 3.125 T11 – Extreme Fiber Vertical Strain Profiles during Pull Cycles
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Figure 3.126  T11 – Spiral Strains for Lowest Six Spiral Layers on the South Side

Figure 3.127  T11 – Spiral Strains for Lowest Six Spiral Layers on the North Side
Figure 3.128  T11 – Tensile Strain-Displacement for Bar N3 during Push Cycles

Figure 3.129  T11 – Compressive Strain-Displacement for Bar N3 during Pull Cycles
Figure 3.130  T11 – Tensile Strain-Displacement for Bar S3 during Pull Cycles

Figure 3.131  T11 – Compressive Strain-Displacement for Bar S3 during Push Cycles
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Figure 3.132  T11 – Bar S3 Strain Hysteresis over South Buckled Region (7.13" Above)

Figure 3.133  Test 11 – Transverse Steel Strain Hysteresis over South Buckled Region
Figure 3.134  T11 – Bar N3 Strain Hysteresis over North Buckled Region (3.33" Above)

Figure 3.135  T11 – Transverse Steel Strain Hysteresis over North Buckled Region
Test 11 – Kobe Earthquake Load History Curvature and Strain Penetration Data

Vertical curvature profiles for push and pull cycles before bar buckling appear in Figure 3.136 and Figure 3.137. The curvature profiles during pull cycles seem to be affected by residual strains from the peak displacement cycle to (\( \mu^{3.86 \text{ sec}} = 8.28'' \)). The plastic portions of the curvature profiles during pull cycles are less linear when compared to profiles in other tests. The curvatures measured in the lowest 18” above the footing remained effectively constant during the pull cycle to (\( \mu^{−6.142 \text{ sec}} = −5.08'' \)). The base section rotations attributable to strain penetration during push and pull cycles appear in Figure 3.138 and Figure 3.139 respectively. A comparison of the measured top column displacements and the integrated displacements from the curvature data and base rotation profiles appear in Figure 3.140. The Optotrak integrated displacements match the measured string potentiometer displacements well throughout the entire range of response. Circle data points in Figure 3.141 plot the measured spread of plasticity as a function of base section rotation.

![Figure 3.136 T11 – Vertical Curvature Profiles during Push Cycles](image-url)
Figure 3.137  T11 – Vertical Curvature Profiles during Pull Cycles

Figure 3.138  T11 – Base Rotation due to Strain Penetration during Push Cycles
Figure 3.139  T11 – Base Rotation due to Strain Penetration during Pull Cycles

Figure 3.140  T11 – Comparison of Measured and Integrated Displacements
Figure 3.141 T11 – Measured Spread of Plasticity (Circle Data Points)
3.1.5 Test 12 – Japan 2011 Earthquake Load History

Table 3.13 Observational Summary for Test 12 – Japan 2011 EQ Load History

VALUES OF INTEREST:

Concrete Compressive Strength: \( f'_c = 6100 \text{ psi} \)
Axial Load: \( P = 170 \text{ kips} \)
Analytical First Yield Force: \( F'_y = 46.5 \text{ kips} \)
Experimental First Yield Displacement: \( \Delta'_y = 0.62" \)
Analytical Nominal Moment Capacity: \( M_n = 494.5 \text{ kip} \times \text{ ft} \)
Equivalent Yield Displacement: \( \Delta_y = 0.83" \)
Maximum Lateral Force: \( 72.6 \text{ kips} \)
Maximum Lateral Displacement: \( \mu_{9.9}^{68.62 sec} = 8.22" \)
Failure Mode: Specimen Saved as a Repair Candidate with Buckled Reinforcement

DAMAGE OBSERVATIONS:

First Cracking North: \( \mu_{0.5}^{44.26 sec} = 0.39" \)
First Cracking South: \( \mu_{-0.3}^{43.98 sec} = -0.26" \)
Cover Concrete Crushing North: \( \mu_{-2.2}^{61.80 sec} = -1.85" \)
Cover Concrete Crushing South: \( \mu_{2.1}^{48.83 sec} = 1.77" \)
Transverse Steel Yield North: At \(-5.02"\) during pull to \( \mu_{-7.9}^{66.88 sec} = -6.53" \)
Transverse Steel Yield South: At \(5.70"\) during push to \( \mu_{9.9}^{68.62 sec} = 8.22" \)
Longitudinal Bar Buckling North: Reversal from \( \mu_{9.9}^{68.62 sec} = 8.22" \)
Longitudinal Bar Buckling South: *Deformation during \( \mu_{-5.1}^{70.55 sec} = -4.17" \)

*\( \mu_{9.9}^{68.62 sec} = 8.22" \) represents a ductility 9.9 push cycle 68.62 seconds into the Japan EQ LH
Table 3.14 Strain Data Summary for Test 12 – Japan 2011 Earthquake Load History

<table>
<thead>
<tr>
<th>MATERIAL STRAINS:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North:</td>
<td>( \varepsilon_s = 0.0047 ) (compression)</td>
</tr>
<tr>
<td>Cover Concrete Crushing South:</td>
<td>( \varepsilon_s = 0.0044 ) (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield North:</td>
<td>( \varepsilon_s = 0.0165 ) (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield South:</td>
<td>( \varepsilon_s = 0.0176 ) (compression)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North:</td>
<td>( \varepsilon_s = 0.058 ) (peak tension prior to ( bb ))  ( \varepsilon_s = 0.021 ) (peak comp. prior to ( bb ))</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South:</td>
<td>*( \varepsilon_s = 0.044 ) (peak tension prior to ( bb ))</td>
</tr>
<tr>
<td>*Deformation, No Visible Buckling</td>
<td>*( \varepsilon_s = 0.032 ) (peak comp. prior to ( bb ))</td>
</tr>
<tr>
<td>Mander (1988) Ultimate Concrete Compression Strain, ( \varepsilon_{cu} ) = 0.0187</td>
<td></td>
</tr>
</tbody>
</table>
Figure 3.143 T12 – Experimental Portion of the Japan 2011 Earthquake Load History

Figure 3.144 T12 – Japan 2011 Lateral Force vs. Top Column Displacement Response
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Figure 3.145  T12 – Compressive Axial Load from One Jack (Total = 2*Value)

Test 12 – Japan 2011 Earthquake Load History Experimental Observations:

The analytical top column displacement history for the scaled Japan 2011 earthquake, which appears in Figure 3.142, was determined using fiber-based numerical simulation in OpenSees. A 1.25x scaled version of the 2011 Japan earthquake was selected to reach a displacement ductility of ten during the largest cycle. In previous time history based tests, buckling did not occur during the Chile or Chichi records scaled to displacement ductility 8.7 and 8.9 respectively. The results from the Chichi and Kobe records suggest that high ductility cycles can decrease the effectiveness of transverse steel as a boundary condition restraining the longitudinal steel. A peak displacement ductility level of ten was chosen to increase the level of tension strain in the steel to evaluate the steel tensile strain limit which leads to buckling of longitudinal steel upon reversal of loading. The initial portion of the Japan 2011 earthquake contained reversals around ductility one which have little large impact on the remainder of the test. The portion of the load history recreated in the
experiment is shown in Figure 3.143. The resulting lateral force vs. top column displacement response for the Japan 2011 record appears in Figure 3.144.

The first cycle for the experimental test reached \( \mu_{0.3}^{43.9\text{sec}} = -0.26'' \), as shown in Figure 3.143. The first cracks on the South side of the specimen measured 0.1mm at a lateral force of -24.63 kips which is over half of the first yield force. The load history prior to this point contained many cycles of loading around a displacement ductility of one, which were not included in the experimental test. The beginning cycles omitted from the experimental displacement history should not have a large impact on the relationship between strain and displacement or damage within the section. Cracks measuring 0.2mm on the North side of the specimen occurred during \( \mu_{0.5}^{44.26\text{sec}} = 0.39'' \), which had a lateral force of 35.69 kips. This equates to around 75% of the analytical first yield force.

Cracks on the South side of the specimen were measured at 0.4mm during \( \mu_{1.3}^{47.53\text{sec}} = -1.10'' \) as shown in Figure 3.146. The crack distribution on the North side of the specimen during \( \mu_{2.1}^{48.83\text{sec}} = 1.77'' \) appears in the middle and right photos of Figure 3.146. Crack widths on the North side of the specimen measured 2mm and the cover concrete on the South side of the specimen began to crush as shown in the left photo of Figure 3.147. Crushing on the South side of the specimen extended 10” above the footing during \( \mu_{2.4}^{61.36\text{sec}} = 2.02'' \), while crack widths on the North side of the column measured 2.5mm. In previous tests, cover crushing began after visual flaking. This flaking was observed on the North side of the specimen during \( \mu_{2.2}^{61.89\text{sec}} = -1.85'' \), as shown in the right photo of Figure 3.147. The extent of crushing on the North side of the specimen extended 7” above the footing during \( \mu_{2.1}^{65.83\text{sec}} = -1.71'' \) as shown in the right photo of Figure 3.148.

The largest cycle in the pull direction of loading occurred during \( \mu_{7.9}^{66.98\text{sec}} = -6.53'' \) with additional crushing on the North side of the specimen, see Figure 3.149. The peak cycle in the push direction at \( \mu_{9.9}^{68.62\text{sec}} = -8.22'' \) was concluded without visible buckling on the South side of the specimen, Figure 3.150. A peak lateral load of 72.1 kips was recorded during the largest cycle of the Japan 2011 load history. Upon reversal of loading from
(\(\mu_{9,9}^{68.62 \text{ sec}} = 8.22"\)), which placed the North side of the specimen under large tensile strains, the extreme fiber North reinforcing bar N3 buckled on the way to (\(\mu_{2,0}^{68.95 \text{ sec}} = 1.68"\)), Figure 3.151. Even though the reversal only brought the specimen to a lower ductility in the same direction of loading as the peak cycle, a lateral load of -27.40 kips was recorded due to hysteretic offset from the peak displacement cycle. Therefore, visible buckling was observed while the cracks on the North side of the specimen remained open and the North reinforcement was the sole source of compression zone stability. After a small push cycle to (\(\mu_{3,4}^{69.12 \text{ sec}} = 2.80"\)), a second reinforcing bar N4 on the North side of the specimen buckled on the way to (\(\mu_{-1,4}^{69.41 \text{ sec}} = -1.14"\)), as shown in the right photo of Figure 3.151. The rest of the load history progressed without any additional buckled reinforcement or rupture of buckled reinforcement. The deformation in the previously buckled bars increased and the core concrete over the North buckled region began to deteriorate as the load history progressed, see Figure 3.152. Visible buckling of the South reinforcement was never observed, although very slight deformation over the bottom three transverse steel spacing was noticed. This deformation never visibly increased with additional cycles.

**Figure 3.146** T12 – (Left) South Crack Distribution during (\(\mu_{-1,3}^{47.53 \text{ sec}} = -1.10"\)) and (Middle & Right) North Crack Distribution during (\(\mu_{2,1}^{48.83 \text{ sec}} = 1.77"\))
Figure 3.147 T12 – First Signs of Crushing (Left) South Side at ($\mu_{2.1}^{48.83 \text{ sec}} = 1.77''$) and (Right) North Side at ($\mu_{-2.2}^{61.80 \text{ sec}} = -1.85''$)

Figure 3.148 T12 – (Left) South side at ($\mu_{2.4}^{61.36 \text{ sec}} = 2.02''$) and (Right) North side at ($\mu_{-2.1}^{65.83 \text{ sec}} = -1.71''$)
Figure 3.149  T12 – Peak Pull Cycle at ($\mu_{-7.9}^{66.88\ sec} = -6.53\"$) – (Left) Back Side of the Specimen, (Middle) South Side, and (Right) North Side

Figure 3.150  T12 – Peak Push Cycle in the Japan 2011 Load History ($\mu_{9.9}^{68.62\ sec} = 8.22\"$)
Figure 3.151 T12 – (Left) Buckling of Extreme Fiber Bar N3 after Reversal from $(\mu_{9.92 \text{ sec}}^{68.62} = 8.22")$ and (Right) Buckling of Bar N4 at $(\mu_{-1.41 \text{ sec}}^{69.41} = -1.14")$

Figure 3.152 T12 – Increased Deformation in the Buckled bars toward the End of LH
Test 12 – Japan 2011 Earthquake Load History Strain Data Analysis:

North Reinforcement

Specific strain data observation points along the backbone curve of cyclic response were chosen for analysis, see Figure 3.153. The extreme fiber vertical strain profiles for push and pull cycles appear in Figure 3.154 and Figure 3.155 respectively. Transverse steel strains for the lowest six spiral layers on the North side of the column appear in Figure 3.157 for compressive pull cycles.

Bar N3 buckled after reversal from a peak tensile strain of 0.058, measured 3.57” above the footing, during the peak push cycle (\(\mu_{9.9 \text{ sec}}^{68.62} = 8.22\)). The peak compressive strain of -0.021 measured 3.57” above the footing in bar N3 during (\(\mu_{-7.9 \text{ sec}}^{66.88} = -6.53\)) preceded the peak tensile cycle which caused buckling upon reversal of loading. The location of the largest tensile and compressive strains coincides with the location of outward buckling later in the test. The relationship between tensile strain and displacement from when the column was vertical to the peak of push cycles for extreme fiber bar N3 appears in Figure 3.158. The relationship between compressive strain and displacement for bar N3 appears in Figure 3.159. During initial pull cycles, the moment curvature prediction with the PCK (2007) Lp Hinge Method matches the recorded compressive strains well, but during the peak pull cycle to (\(\mu_{-7.9 \text{ sec}}^{66.88} = -6.53\)) the recorded strains begin to exceed the prediction at an increasing rate.

The strain hysteresis for the buckled region of bar N3, 3.57” above the footing, appears in Figure 3.163 with an earthquake time color bar to track the progression of the test. The peak tensile and compressive strains for bar N3 were measured over this gage length during the largest push and pull cycles respectively. The transverse steel strain gage hysteresis for the spiral layer overlaying the North buckled region appears in Figure 3.164. The strain in the transverse steel went into the inelastic range during the largest pull cycle to (\(\mu_{-7.9 \text{ sec}}^{66.88} = -6.53\)). A data marker was placed at the location when the transverse steel strain began to sharply increase during the reversal from (\(\mu_{9.9 \text{ sec}}^{68.62} = 8.22\)), indicating outward
deformation over the buckled extreme fiber bar. A similar data label is shown on the bar N3 strain hysteresis. Measured strains past this point no longer represent engineering strain, but are included to illustrate the progression of damage. Similarly, the strain gage placed over the transverse steel quickly debonds, preventing further measurement.

South Reinforcement

A peak compressive strain of -0.032 was measured 7.88” above the footing for extreme fiber bar S3. The relationship between compressive strain and displacement from when the column was vertical to the peak of push cycles appears in Figure 3.161 for bar S3. The recorded compressive strains exceed the moment curvature prediction with the PCK (2007) Lp Hinge Method during the peak cycle at ($\mu_{9.9}^{68.62 \text{ sec}} = 8.22$”). The relationship between tensile strain and displacement is shown in Figure 3.160.

The strain hysteresis for extreme fiber bar S3 appears in Figure 3.165 with an earthquake time color bar to track the progression of the test. The transverse steel strain hysteresis for the spiral layer restraining the potential outward deformed region of bar S3 is shown in Figure 3.166. The transverse steel strain sharply increased during the peak push cycle at ($\mu_{9.9}^{68.62 \text{ sec}} = 8.22$”). Since visible buckling was not observed for the South reinforcement, this sharp increase is largely attributed to compressive demand in the region. The strain hysteresis in Figure 3.165 would suggest that measurable deformation occurred after the second largest pull cycle. This particular gage length was never placed back into compression due to outward deformation during push cycles. The potential deformation cannot be visually verified by test results since bar buckling on the South side of the specimen was not observed. In previous tests, the measurable deformation was verified by buckling in the same region later in the test.
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Figure 3.153 T12 – Strain Data Observation Points along the Backbone Curve

Figure 3.154 T12 – Extreme Fiber Vertical Strain Profiles during Push Cycles
Figure 3.155  T12 – Extreme Fiber Vertical Strain Profiles during Pull Cycles

Figure 3.156  T12 – Spiral Strains for Six Lowest Spiral Layers on the South Side
Figure 3.157 T12 – Spiral Strains for Six Lowest Spiral Layers on the North Side

Figure 3.158 T12 – Tensile Strain-Displacement for Bar N3 during Push Cycles
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Figure 3.159 T12 – Compressive Strain-Displacement for Bar N3 during Pull Cycles

Figure 3.160 T12 – Tensile Strain-Displacement for Bar S3 during Pull Cycles
Figure 3.161 T12 – Compressive Strain-Displacement for Bar S3 during Push Cycles

Figure 3.162 T12 – Hysteretic Response Japan 2011 Record with an EQ Time Colorbar
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Figure 3.163  T12 – North Extreme Fiber Bar N3 Strain Hysteresis (3.57" Above)

Figure 3.164  T12 – Transverse Steel Strain Hysteresis over North Buckled Region
Figure 3.165  T12 – South Extreme Fiber Bar S3 Strain Hysteresis (4.03" Above)

Deformation in Bar S3

Figure 3.166  T12 – Spiral Strain Hysteresis for South “Measurable Deformation”
Test 12 – Japan 2011 Earthquake LH - Strain Penetration and Curvature Data:

Vertical curvature profiles for push and pull cycles along the backbone curve of cyclic response appear in Figure 3.167 and Figure 3.168. The least squared error lines show that higher ductility cycles have a linear distribution of plastic curvature similar to previous tests. The base rotations attributable to strain penetration of longitudinal reinforcement into the footing are shown in Figure 3.169 and Figure 3.170 respectively. The measured displacement of the base section was obtained from the LED placed closest to the footing-column interface. The slip hysteresis for extreme fiber bars N3 and S3 appear in Figure 3.171 and Figure 3.172 respectively. The top column displacement from the Optotrak may be determined by integrating the measured curvature distribution, extrapolating the base rotation to the center of loading, and assuming a linear distribution of curvature above the instrumented region which aligns the equivalent yield curvature at the base section. A comparison of the Optotrak integrated and measured top column displacements, in Figure 3.173, shows that the two methods agree throughout the entire range of displacements. The measured spread of plasticity as a function of base section curvature ductility appears in Figure 3.174. The extent of plasticity is computed as the intersection of the linear plastic curvature regression and the elastic curvature profile.
Figure 3.167 T12 – Vertical Curvature Profiles during Push Cycles

Figure 3.168 T12 – Vertical Curvature Profiles during Pull Cycles
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Figure 3.169 T12 – Base Rotation due to Strain Penetration during Push Cycles

Figure 3.170 T12 – Base Rotation due to Strain Penetration during Pull Cycles
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Figure 3.171  T12 – Bar N3 Base Section Slip Hysteresis due to Strain Penetration

Figure 3.172  T12 – Bar S3 Base Section Slip Hysteresis due to Strain Penetration
Figure 3.173 T12 – Comparison of Measured and Optotrak Integrated Displacements

Figure 3.174 T12 – Measured Spread of Plasticity (Circular Data Points)
3.2 Load History and Transverse Steel Variable Tests 13-18

The effect of transverse steel detailing on restraint of longitudinal bars was the main variable for Tests 13-18. The test matrix for the eight columns is shown in Table 3.15, and the material properties of the reinforcement appear in Table 3.16. The 24” (610mm) diameter bridge columns, Figure 3.178, contained 16 #6 (19mm) A706 bars for longitudinal reinforcement \( (A_{st}/A_g = 1.6\%) \) and either a #3 (9.5mm) or #4 (12.7mm) A706 spiral at variable spacing. The shear span for the cantilever columns was 8ft (244cm), and they had a moment to shear ratio of \( (M/VD = 4) \). The specimens were subjected to a constant axial load of 170kips (756kN), \( (P/(f'cA_g) \approx 5\%) \) depending on the concrete compressive strength. Load history was maintained as a variable for Tests 16-18 which had the same transverse steel detailing. The following transverse volumetric steel ratios were investigated: \( (4A_{sp}/(D's)) = 0.5\% \) (6d₀-spacing), 0.7%, 1% (previous test series), and two separate detailing arrangements for 1.3%. Both the volumetric ratio and spacing of the transverse steel are important when describing confinement and bar buckling restraint. Two columns were tested with 1.3% transverse steel, one with a #3 spiral at 1.5” spacing and another with a #4 spiral at 2.75” spacing. For comparison, a specimen was tested with a #3 spiral at 2.75” spacing.

An engineer has the most control over the size and spacing of transverse steel to improve buckling resistance. Previously tested specimens 8-12 utilized a #3 spiral at 2” pitch \( (4A_{sp}/(D's) = 1\%) \). During the Kobe and Japan 2011 load histories, a peak displacement ductility of ten was necessary to produce sufficient tensile strain to buckle reinforcement upon reversal in an earthquake load history. The compressive demand at ductility ten resulted in server layers of inelastic transverse steel, which decreased their effectiveness in restraining the longitudinal reinforcement during the remainder of the load history. Even though this side of the specimen was subjected to lower levels of tensile strain, the reinforcement still buckled due to the inelastic transverse steel.
Table 3.15 Column Property Summary for Load History Variable Tests 13-18

<table>
<thead>
<tr>
<th>Test</th>
<th>Load History</th>
<th>D (in)</th>
<th>L/D</th>
<th>Long. Steel ($\rho_l$)</th>
<th>Spiral Detailing ($\rho_s$)</th>
<th>$f_c$ (psi)</th>
<th>$P/f_c*Ag$</th>
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</thead>
<tbody>
<tr>
<td>13</td>
<td>Three Cycle Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#4 at 2.75” (1.3%)</td>
<td>6097</td>
<td>6.2%</td>
</tr>
<tr>
<td>14</td>
<td>Three Cycle Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 4” (0.5%)</td>
<td>6641</td>
<td>5.7%</td>
</tr>
<tr>
<td>15</td>
<td>Three Cycle Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2.75” (0.7%)</td>
<td>7232</td>
<td>5.2%</td>
</tr>
<tr>
<td>16</td>
<td>Three Cycle Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 1.5” (1.3%)</td>
<td>6711</td>
<td>5.6%</td>
</tr>
<tr>
<td>17</td>
<td>Llolleo 1985</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 1.5” (1.3%)</td>
<td>7590</td>
<td>5.0%</td>
</tr>
<tr>
<td>17b</td>
<td>Cyclic Aftershock</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 1.5” (1.3%)</td>
<td>7590</td>
<td>5.0%</td>
</tr>
<tr>
<td>18</td>
<td>Darfield 2010</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 1.5” (1.3%)</td>
<td>7807</td>
<td>4.8%</td>
</tr>
<tr>
<td>18b</td>
<td>Cyclic Aftershock</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 1.5” (1.3%)</td>
<td>7807</td>
<td>4.8%</td>
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</table>

Table 3.16 Reinforcement Material Property Summary for Columns 13-18

<table>
<thead>
<tr>
<th>Longitudinal Reinforcement</th>
<th>$\varepsilon_y$</th>
<th>$f_y$ (ksi)</th>
<th>$\varepsilon_h$</th>
<th>$f_h$ (ksi)</th>
<th>$\varepsilon_u$</th>
<th>$f_u$ (ksi)</th>
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</thead>
<tbody>
<tr>
<td>Tests 8-12 (#6 Bar)</td>
<td>0.00235</td>
<td>68.1</td>
<td>0.0131</td>
<td>68.2</td>
<td>0.1189</td>
<td>92.8</td>
</tr>
<tr>
<td>Tests 13-18 (#6 Bar)</td>
<td>0.00235</td>
<td>68.1</td>
<td>0.0146</td>
<td>68.2</td>
<td>0.1331</td>
<td>94.8</td>
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</table>

<table>
<thead>
<tr>
<th>Transverse Steel</th>
<th>Yield Stress, $f_t$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tests 8-12 (#3 Spiral)</td>
<td>74.1</td>
</tr>
<tr>
<td>Tests 13-18 (#3 Spiral)</td>
<td>64.6</td>
</tr>
<tr>
<td>Tests 13-18 (#4 Spiral)</td>
<td>69.9</td>
</tr>
</tbody>
</table>
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Figure 3.175 Long. Steel Stress-Strain Response with Three Measurement Methods

Figure 3.176 #3 Bar Transverse Steel Stress-Strain Response and 0.2% Offset Method
Figure 3.177 #4 Bar Transverse Steel Stress-Strain Response and 0.2% Offset Method

Figure 3.178 (Left) Vertical Cover Concrete Blockout Strips and Target Marker Application, (Right) Cross Section Bar Designation
3.2.1 Test 13 – Three Cycle Set Load History with #4 Spiral at 2.75” (1.3%)

Table 3.17 Observational Summary for Test 13 – Cyclic with #4 Spiral at 2.75” (1.3%)

<table>
<thead>
<tr>
<th>VALUES OF INTEREST:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Compressive Strength:</td>
<td>$f'_c = 6097 \text{ psi}$</td>
</tr>
<tr>
<td>Axial Load:</td>
<td>$P = 170 \text{ kips}$</td>
</tr>
<tr>
<td>Analytical First Yield Force:</td>
<td>$F'_y = 46.5 \text{ kips}$</td>
</tr>
<tr>
<td>Experimental First Yield Displacement:</td>
<td>$\Delta'_y = 0.60$&quot;</td>
</tr>
<tr>
<td>Analytical Nominal Moment Capacity:</td>
<td>$M_n = 498.7 \text{ kip ft}$</td>
</tr>
<tr>
<td>Equivalent Yield Displacement:</td>
<td>$\Delta_y = 0.81&quot;$</td>
</tr>
<tr>
<td>Maximum Lateral Force:</td>
<td>$70.9 \text{ kips}$</td>
</tr>
<tr>
<td>Failure Mode:</td>
<td>Fracture of Previously Buckled Reinforcement</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DAMAGE OBSERVATIONS:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>First Cracking North:</td>
<td>$1/2Fy' = 0.17&quot;$</td>
</tr>
<tr>
<td>First Cracking South:</td>
<td>$-1/2Fy' = -0.16&quot;$</td>
</tr>
<tr>
<td>Cover Concrete Crushing North:</td>
<td>$\mu_2^{-1} = -1.61&quot;$</td>
</tr>
<tr>
<td>Cover Concrete Crushing South:</td>
<td>$\mu_2^{+1} = 1.60&quot;$</td>
</tr>
<tr>
<td>Transverse Steel Yield North:</td>
<td>At $-4.78&quot;$ during pull to $\mu_6^{-3} = -4.85&quot;$</td>
</tr>
<tr>
<td>Transverse Steel Yield South:</td>
<td>At $4.17&quot;$ during push to $\mu_8^{+2} = 6.46&quot;$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North:</td>
<td>Reversal from $\mu_8^{+1} = 6.46&quot;$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South:</td>
<td>Reversal from $\mu_8^{-1} = 6.48&quot;$</td>
</tr>
<tr>
<td>Longitudinal Bar Fracture North:</td>
<td>At $-0.79&quot;$ during push to $\mu_{10}^{+1} = 8.06&quot;$</td>
</tr>
<tr>
<td>Longitudinal Bar Fracture South:</td>
<td>At $-2.51&quot;$ during pull to $\mu_{10}^{-2} = -8.12&quot;$</td>
</tr>
</tbody>
</table>

* $\mu_8^{-1} = 6.48"$ represents the first pull cycle of displacement ductility eight
Table 3.18 Strain Data Summary for Test 13 – Cyclic with #4 Spiral at 2.75” (1.3%)

<table>
<thead>
<tr>
<th>MATERIAL STRAINS:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North: $\varepsilon_s = 0.046$ (compression)</td>
</tr>
<tr>
<td>Cover Concrete Crushing South: $\varepsilon_s = 0.036$ (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield North: $\varepsilon_s = 0.0166$ (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield South: $\varepsilon_s = 0.0162$ (compression)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North: $\varepsilon_s = 0.047$ (peak tension prior to $bb$) $\varepsilon_s = 0.017$ (peak comp. prior to $bb$)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South: $\varepsilon_s = 0.047$ (peak tension prior to $bb$) $\varepsilon_s = 0.017$ (peak comp. prior to $bb$)</td>
</tr>
</tbody>
</table>

Mander (1988) Ultimate Concrete Compression Strain, $\varepsilon_{cu} = 0.0211$

Figure 3.179 Cross Section Bar Designation for Tests 13-18
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**Figure 3.180** T13 – Symmetric Three Cycle Set Load History

**Figure 3.181** T13 – Hysteretic Response with PCK (2007) Lp Prediction
Figure 3.182 T13 – Compressive Axial Load from One Jack (Total = 2*Value)

Figure 3.183 T13 – Bar Fracture History of Previously Buckled Reinforcement
Test 13 – Symmetric Three Cycle Set (#4 @ 2.75") Experimental Observations

The first yield force for the tested material and geometric properties was determined using moment curvature analysis (Test 13: Cumbia Fy’ = 46.5 kips with f’c = 6097 psi) compared to (Test 9: Cumbia Fy’ = 46.9 kips with f’c = 6814 psi). The predicted first yield force for both test series, 7-12 and 13-18, are remarkably similar due to the near identical longitudinal reinforcement properties for both batches of steel. The first yield displacement for the thirteenth test was obtained as an average for the first yield push and pull cycles (Δ’y = 0.60") compared to (Δ’y = 0.63") for the ninth test. Vertical strain profiles for both push and pull cycles up to the first yield force appear in Figure 3.184 with a dashed line representing the yield strain of the longitudinal reinforcement. The equivalent yield displacement, used to determine the displacement ductility levels (μΔ1 = 1 * Δy), is then calculated as Δy = Δ’y(Mn/Mn’y) = 0.81" for Test 13 compared to Δy = 0.84" for Test 9. The full symmetric three-cycle-set load history appears in Figure 3.180 and the resulting lateral force vs. top column displacement hysteresis is shown in Figure 3.181.

![Figure 3.184 T13 – Vertical Strain Profiles for Extreme Fiber Bars (Dashed Yield Line)](image-url)
The first cracks on the North side of the specimen where measured at 0.1mm at approximate 8” spacing during the push cycle to \( \frac{1}{2}F_y \). Cracks of the same width and approximate spacing where measured on the South side of the specimen during the \(-\frac{1}{2}F_y\) pull cycle. The cracks on the North side of the specimen increased to 0.2mm at 4” spacing during the push cycle to \( \frac{3}{4}F_y \). Larger 0.3mm crack widths at a greater spacing of 8” were measured on the South side of the specimen during \(-\frac{3}{4}F_y\). Up until the first yield force was reached, the cracks were all horizontal without any inclination on the sides of the specimen with greater shear stress. The change in orientation of flexural shear cracks with increased ductility demands appears in Figure 3.190. Cracks on the North side of the specimen increased to 0.3mm width at approximate 4” spacing during the first yield push cycle. During the first yield pull cycle cracks increased to 0.4mm width at 5” spacing.

During \((\mu_{1}^{+3} = 0.81")\), crack widths measured 0.4mm at 4” spacing on the North side of the specimen. On the opposite side of the specimen crack widths were measured at 0.5mm at 5” spacing during \((\mu_{1}^{-3} = -0.80")\). Visible flaking which occurs just before cover concrete crushing was apparent on both sides of the specimens after the first push and pull cycles of ductility 1.5. This flaking did not lead to crushing during subsequent cycles at displacement ductility 1.5. Cracks on the North side of the specimen measured 1.1mm, while the South side measured 1.25mm during the third push and pull cycles of ductility 1.5 respectively. Concrete cover crushing 2” above the footing occurred on the South side of the specimen during \((\mu_{2}^{+1} = 1.60")\). Similarly, crushing over 2” on the North side of the specimen was observed during \((\mu_{2}^{-1} = -1.61")\). Cracks on the North and South sides of the specimens measured 1.5mm and 2mm during the third push and pull cycles of ductility two respectively. After three complete cycles at ductility three, the extent of crushing increased to 10” on the North and 7” on the South side of the specimen. The load history continued through ductility six with additional inclined flexural shear cracks and increased extent of crushing, but without buckling of the longitudinal steel.

After reversal from \((\mu_{8}^{+1} = 6.46")\), extreme fiber bar N3 and adjacent bar N2 buckled as shown in the left photo of Figure 3.187. During the second push cycle of ductility eight,
South reinforcing bar S4 visibly buckled, see the left photo of Figure 3.188. The South extreme fiber bar S3 did not show signs of visible buckling while adjacent bar S4 deformed out of plane at the location where more prominent buckling later occurred. During the second and third pull cycles of ductility eight the deformation in the North buckled bars increased and an additional bar N4 buckled as shown in the right photo of Figure 3.187. The buckled deformation of both the North and South reinforcement occurred between layers of transverse steel (#4 at 2.75” spacing). Buckling over two to three layers of transverse steel was observed in previous tests with a #3 spiral at 2” spacing.

During \( \mu_{10}^{+1} = 8.07" \), two of the previously buckled North reinforcing bars ruptured and South reinforcing bars S2 and S3 buckled, see Figure 3.188 and Figure 3.189. Rupture of the North reinforcing bar N3 occurred before the bar straightened out in tension. While this has never been observed in previous tests, it is likely a consequence of the more severe buckled profile between layers of transverse reinforcement, see Figure 3.187. Losses in strength from reinforcement ruptures are shown in Figure 3.183 on the hysteretic response. Three additional North reinforcing bars ruptured during the \( \mu_{10}^{+2} = 8.06" \). Three previously buckled reinforcing bars on the South side of the specimen ruptured during \( \mu_{10}^{-2} = -8.12" \). The test was concluded with a total of eight ruptured reinforcing bars and a considerable loss of strength in each direction of loading.
Figure 3.185  T13 – Concrete Cover Crushing at the End of Ductility Two (Left) North and (Right) South

Figure 3.186  T13 – Crushing at the End of Ductility Six (Left) North and (Right) South
Figure 3.187 T13 – (Left) Buckling of Reinforcing Bars N2 and N3 during \( \mu_0^{-1} = -6.48" \) and (Right) Increased deformation in North Buckled Bars \( \mu_0^{-3} = -6.50" \)

Figure 3.188 T13 – Buckling of South Reinforcing Bar S4 during \( \mu_0^{+2} = 6.46" \) and (Right) Buckling of Bar S2 and S3 during \( \mu_{10}^{+1} = 8.07" \)
Figure 3.189  T13 – (Left) Rupture of North Reinforcement Bars N2 and N3 during \( (\mu_{10}^+ = 8.07") \) and (Right) Additional Deformation in South Bars during \( (\mu_{10}^+ = 8.06") \)
Figure 3.190  T13 – Crack Progression on the Back Side of the Specimen
Test 13 – Symmetric Three Cycle Set (#4 @ 2.75”) Strain Data Analysis

North Reinforcement

Extreme fiber vertical strain profiles during push and pull cycles appear in Figure 3.191 and Figure 3.192 respectively. These figures show both extreme fiber bars on the same graph to illustrate the effects of tension shift on strain profiles. As the hinge rotates about inclined flexural shear cracks, compressive strains are concentrated at the base and tensile strains are fanned out to a greater height following the crack distribution. Just above the footing cracks remain horizontal, but above this base section the flexural shear cracks are inclined as shown in Figure 3.190. Due to the effects of tension shift, the tensile strains at the beginning of an inclined flexural shear crack do not coincide with the perceived moment demand at that location based on its height above the footing and the applied lateral load.

A peak tensile strain of 0.047, at a height of 2.03” above the footing, was measured in North extreme fiber bar N3 during ($\mu_6^{+1} = 6.46”$). It is notable that a higher peak tensile strain of 0.050 was measured 2.2” above the footing in the adjacent North reinforcement bar N4. Bar N2 and N3 visually buckled after reversal from ($\mu_6^{+1} = 6.46”$), leaving bar N4 intact. During the next pull cycle bar N4 visually buckled. The largest compressive strain in bar N3 of -0.017, located 2.03” above the footing, was measured during ($\mu_6^{+3} = 4.85”$). The relationship between tensile strain and displacement for bar N3 appears in Figure 3.195 for the largest tensile gage length 2.03” above the footing. Each curve in the graph represents the tensile strains measured from when the column was vertical to the peak of the given cycle of the load history. The gray line represents the moment curvature prediction for the relationship between strain and displacement from using the PCK (2007) Lp Hinge Method. During higher displacement ductility cycles, the measured tensile strains are significantly lower than the moment curvature prediction. The relationship between compressive strain and displacement for bar N3 appears in Figure 3.196. Buckling of bar N3 during the first pull cycle of ductility eight did not have a large impact on the relationship between compressive strain and displacement for this gage length, see the left photo of Figure 3.187.
The strain hysteresis for the buckled region of extreme fiber north reinforcing bar N3 appears in Figure 3.199. The transverse steel strain hysteresis for a layer of transverse steel close to the buckled region is shown in Figure 3.200. The peak displacement cycle at \( \mu_{8}^{+1} = 6.46" \), which preceded visible buckling, appears as a small red circle in both figures. A data label at the same displacement appears in both figures which represents the time when the buckled bar began to rapidly increase the tensile strain in the transverse steel restraint. The compressive demand during \( \mu_{6}^{+3} = 4.85" \) was not enough to cause the transverse steel to enter the nonlinear range.

*South Reinforcement*

The peak compressive strain in bar S3 of -0.0174 was measured 1.82" above the footing during \( \mu_{8}^{+1} = 6.46" \). A peak tensile strain of 0.047, centered 7.18" above the footing, was measured in bar S3 at \( \mu_{8}^{-1} = 6.48" \). The lowest tensile gage length for bar S3 was blocked by debris during ductility eight, so larger tensile strains may have occurred over this region. To illustrate this point, the vertical strain profile for adjacent bar S4 appears in Figure 3.201. The largest tensile strain in bar S4 of 0.051 was measured 1.82" above the footing during \( \mu_{8}^{-1} = 6.48" \). Bar S4 was the first South reinforcement to visibly buckle after reversal from \( \mu_{8}^{-1} = 6.48" \). The strain hysteresis for the buckled region of bar S4 can be seen in Figure 3.202. Buckling of the extreme fiber south bar S3 was delayed until \( \mu_{10}^{+1} = 8.07" \), which is confirmed by the measured longitudinal and transverse steel strain hysteresis in Figure 3.203 and Figure 3.204. The relationship between strain and displacement for push and pull cycles for extreme fiber bar S3 appears in Figure 3.198 and Figure 3.197 respectfully.
Figure 3.191 T13 – Extreme Fiber Vertical Strain Profiles during Push Cycles

Figure 3.192 T13 – Extreme Fiber Vertical Strain Profiles during Pull Cycles
Figure 3.193  T13 – Spiral Strains for the Lowest Six Layers on the South Side

Figure 3.194  T13 – Spiral Strains for the Lowest Six Layers on the North Side
Figure 3.195  T13 – Tensile Strain-Displacement for Bar N3 during Push Cycles

Figure 3.196  T13 – Compressive Strain-Displacement for Bar N3 during Pull Cycles
Figure 3.197 T13 – Tensile Strain-Displacement for Bar S3 during Pull Cycles

Figure 3.198 T13 – Compressive Strain-Displacement for Bar S3 during Push Cycles
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Figure 3.199  T13 – Bar N3 Strain Hysteresis for Buckled Region (7.48" Above Footing)

Figure 3.200  T13 – Spiral Strain Hysteresis for Layer over the North Buckled Region
Figure 3.201 T13 – Tensile Strain Profile for Bar S4 (First South Bar to Buckle)

Figure 3.202 T13 – Bar S4 Strain Hysteresis for Buckled Region (1.82" Above Footing)
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Figure 3.203  T13 – Bar S3 Strain Hysteresis for Buckled Region (4.41" Above Footing)

Figure 3.204  T13 – Spiral Strain Hysteresis for Layer over the South Buckled Region
Test 13 – Three Cycle Set (#4 @ 2.75”) Curvature and Strain Penetration Data:

The cross section curvature for each horizontal section above the footing is determined by connecting the strain measurements from all six instrumented bars with a least squared error line. The curvature is then extracted from the slope of the least squared error line as shown in Figure 3.205. Vertical curvature profiles are plotted for push and pull cycles as shown in Figure 3.206 and Figure 3.207 respectively. These figures show that plastic curvatures have a linear distribution at higher displacement ductility levels. The extent of plastic curvatures above the footing can be calculated by determining where the linear plastic curvature distribution intersects the triangular yield curvature shown as a grey dashed line. The dashed lines for each curvature distribution represent a least squared error linear fit to the plastic portion of the measured curvatures. The data points used to create the least squared error lines appear as circle data markers.

LEDs placed closest to the footing-column interface on the six reinforcing bars can track the base section rotation due to strain penetration of longitudinal reinforcement into the footing. The measured base rotations for push and pull cycles appear in Figure 3.210 and Figure 3.211 respectively. Compared to previous tests, the bar slip profiles are shifted down slightly. Inspection of the measured slip hysteresis for extreme fiber bars N3 and S3 in Figure 3.208 and Figure 3.209 shows that each bar shifted downwards after the tests began. A possible explanation for why this occurred is not available, since this was not observed in any of the other experiments. A comparison of the measured top column displacement and the Optotrak integrated displacements appear in Figure 3.212. The Optotrak displacement was obtained by integrating the measured curvature profile, extrapolating the base rotation to the center of loading, and assuming an elastic curvature distribution above the instrumented region. The measured spread of plasticity as a function of base section curvature ductility is shown in Figure 3.213. The extent of plasticity is calculated as the intersection of the linear plastic curvature regression with the elastic curvature profile, Figure 3.206.
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Figure 3.205  T13 – Push Cycle Strain Profiles Used to Find Cross Section Curvatures

Figure 3.206  T13 – Vertical Curvature Profiles for Push Cycles with Plastic Regression
Figure 3.207  T13 – Vertical Curvature Profiles for Pull Cycles with Plastic Regression

Figure 3.208  T13 – Bar N3 Base Section Slip Hysteresis due to Strain Penetration
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Figure 3.209  T13 – Bar S3 Base Section Slip Hysteresis due to Strain Penetration

Figure 3.210  T13 – Base Rotations during Push Cycles due to Strain Penetration
Figure 3.211 T13 – Base Rotations during Pull Cycles due to Strain Penetration

Figure 3.212 T13 – Comparison of Measured and Optotrak Integrated Displacements
Figure 3.213  T13 – Measured Spread of Plasticity (Circular Data Points)
3.2.2 Test 14 – Three Cycle Set Load History with #3 Spiral at 4” (0.5%) 

Table 3.19 Observational Summary for Test 14 – Cyclic with #3 Spiral at 4” (0.5%) 

VALUES OF INTEREST:

- Concrete Compressive Strength: \( f'_c = 6641 \text{ psi} \)
- Axial Load: \( P = 170 \text{ kips} \)
- Analytical First Yield Force: \( F'_y = 47.0 \text{ kips} \)
- Experimental First Yield Displacement: \( \Delta'_y = 0.60" \) *From Test 13, See Discussion
- Analytical Nominal Moment Capacity: \( M_n = 499.7 \text{ kip} \ast \text{ ft} \)
- Equivalent Yield Displacement: \( \Delta_y = 0.80" \)
- Maximum Lateral Force: \( 69.1 \text{ kips} \)
- Failure Mode: Fracture of Previously Buckled Reinforcement 

DAMAGE OBSERVATIONS:

- First Cracking North: **Intended Cycle to \( \frac{1}{2} F y' = 0.42" \)
- First Cracking South: **Intended Cycle to \( -\frac{1}{2} F y' = -0.45" \)
- Cover Concrete Crushing North: \( \mu^{3}_{1.5} = -1.19" \)
- Cover Concrete Crushing South: \( \mu^{3}_{1.5} = 1.20" \)
- Transverse Steel Yield North: At \( -0.25" \) during pull to \( \mu^{-1}_{6} = -4.80" \)
- Transverse Steel Yield South: At \( 3.84" \) during push to \( \mu^{+1}_{6} = 4.80" \)
- Longitudinal Bar Buckling North: Reversal from \( \mu^{+1}_{6} = 4.80" \)
- Longitudinal Bar Buckling South: Reversal from \( \mu^{-1}_{6} = -4.80" \)
- Longitudinal Bar Fracture North: At \( -3.46" \) during push to \( \mu^{-2}_{8} = 6.40" \)
- Longitudinal Bar Fracture South: At \( -2.81" \) during pull to \( \mu^{-1}_{8} = -6.39" \)

\( *\mu^{-1}_{6} = -4.80" \) represents the first pull cycle of displacement ductility six
Table 3.20 Strain Data Summary for Test 14 – Cyclic with #3 Spiral at 4” (0.5%)

<table>
<thead>
<tr>
<th>MATERIAL STRAINS:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North:</td>
<td>$\varepsilon_s = 0.0029 \text{ (compression)}$</td>
</tr>
<tr>
<td>Cover Concrete Crushing South:</td>
<td>$\varepsilon_s = 0.003 \text{ (compression)}$</td>
</tr>
<tr>
<td>Transverse Steel Yield North:</td>
<td>N/A, Due to Reversal From Tensile Strain</td>
</tr>
<tr>
<td>Transverse Steel Yield South:</td>
<td>$\varepsilon_s = 0.0152 \text{ (compression)}$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North:</td>
<td>$\varepsilon_s = 0.035 \text{ (peak tension prior to bb)}$</td>
</tr>
<tr>
<td></td>
<td>$\varepsilon_s = 0.011 \text{ (peak comp. prior to bb)}$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South:</td>
<td>$\varepsilon_s = 0.035 \text{ (peak tension prior to bb)}$</td>
</tr>
<tr>
<td></td>
<td>$\varepsilon_s = 0.015 \text{ (peak comp. prior to bb)}$</td>
</tr>
<tr>
<td>Mander (1988) Ultimate Concrete Compression Strain, $\varepsilon_{cu} = 0.0109$</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.214 T14 – Cross Section Bar Designation and Target Marker Application
Figure 3.215 T14 – Cyclic Load History (Initial Cycles Influenced by Load Cell Error)

Figure 3.216 T14 – Lateral Force vs. Top Column Displacement Response
Figure 3.217  T14 – Appropriate Scale Factor for Initial Cycles with Load Cell Error

Figure 3.218  T14 – Compressive Axial Load from One Jack (Total = 2*Value)
Test 14 – Symmetric Three Cycle Set (#3 @ 4") Experimental Observations

The first yield force for the tested material and geometric properties was determined using moment curvature analysis (Test 14: Cumbia Fy’ = 46.97 kips with $f_c = 6641$ psi) compared to (Test 13: Cumbia Fy’ = 46.48 kips with $f_c = 6097$ psi). During the early cycles of the Test 14 load history, it became apparent that the actuator load cell was not functioning properly. This was noticed because the forces were considerably lower than expected at small displacements. Initially, this problem was not attributed to the actuator load cell and several small cycles were conducted to try and pin down the specific cause of the problem. During these cycles, the specimen was pushed past the first and equivalent yield displacements in both directions of loading, see Figure 3.215. In the push direction of loading the specimen went past displacement ductility 1.5 and in the pull direction the displacement was just past ductility one. It was determined that the only thing that could have caused this issue is an incorrect actuator load cell reading, and upon inspection a
damaged cable connection was found. The cable was replaced and the actuator load cell began recording the correct lateral force for the remainder of the test.

At this point of the test, there was no way to go back and redo the elastic cycles to the first yield displacement due to the slight stiffness degradation from inelastic cycles in each direction of loading. For this reason, the first yield displacement from Test 13 was used for Test 14. The first yield displacement for the thirteenth test was obtained as an average for the first yield push and pull cycles ($\Delta_y' = 0.60''$). Extreme fiber vertical strain profiles, for Test 14, at the first yield displacement appear in Figure 3.220 for push and pull cycles respectively. The first yield displacement from Test 13 is also appropriate for Test 14 based on the vertical strain profiles which have strains just past yield at large crack locations.

The equivalent yield displacement, used to determine the displacement ductility levels ($\mu_{\Delta n} = n * \Delta_y$), is then calculated as $\Delta_y = \Delta_y' (M_n/M_{\Delta y}) = 0.80''$ for Test 14 compared to $\Delta_y = 0.81''$ for Test 13. The full symmetric three-cycle-set load history appears in Figure 3.215 and the resulting lateral force vs. top column displacement hysteresis is shown in Figure 3.216. Past tests in the load history research program suggest that cycles at lower displacement ductility levels, such as those prior to fixing lateral load issue, should not have an impact on later cycles at larger displacements. The concrete cover on the South side of the specimen remained intact during the largest overload cycle, Figure 3.221. These initial cycles caused stiffness degradation which decreased the force during lower displacement ductility levels in the three cycle set load history.

Since the actuator load is calibrated based on a linear curve relating voltage to lateral force which passes through the origin, a constant scale factor can be used to transform the incorrect data to a better approximation of the actual lateral force. The backbone curves of reinforced concrete bridge columns with similar material properties should remain similar, so this was used to calibrate the scale factor to relate the incorrect lateral force of Test 14 to the backbone curve of Test 13. As shown in Figure 3.217, a constant scale factor of 1.64 gave
the best approximation of the actual lateral force during early cycles affected by the damaged load cell cable.

**Figure 3.220** T14 – North and South Bar Strain Profiles during $\Delta_y'$ (Test 13) and $\Delta_y$
Even though the initial cycles were did not reach the proper level of force expected in a symmetric three-cycle-set load history, information on the crack location, width, and orientation were still taken at the peak of each cycle. The first half cycle was intended to reach \( \frac{1}{4} F_y \), but since the actuator load cell cable was damaged, the actual displacement at this intended lateral force was greater. This initial cycle and the subsequent reversal to \(-\frac{1}{4} F_y \) were not large enough to cause cracking in the specimen. The third half cycle intended to reach \( \frac{1}{2} F_y \) pushed the specimen to 0.42” and 0.3mm cracks at 8” spacing were observed. As expected, the cracks occurred at the level of the transverse steel. The same crack width and spacing was observed during the pull cycle to \(-\frac{1}{2} F_y \). The cycle intended for \( \frac{3}{4} F_y \) reached a displacement of 0.91”. Note that this is larger than the equivalent yield displacement of 0.80”. The largest crack width measured 0.6mm at 4” spacing which followed the locations of transverse steel. During the intended pull cycle to \(-\frac{3}{4} F_y \) at -0.94”, 0.75mm crack widths were measured at 4” spacing.

The next cycle was intended to reach \( F_y \), but the specimen was paused and the actuator load cell connection problem was determined. The peak displacement reached prior to pausing and reversing the load to zero force was 1.38”. Note that this is equal to a displacement ductility of 1.73, therefore latter cycles in the load history to ductility 1 and 1.5 in the push direction have a loss of stiffness due to this overload. In the pull direction of loading only the ductility one cycles are affected by stiffness degradation from the displacement ductility -1.13 cycle.

During \( (\mu_1^{+3} = 0.79") \), 0.75mm crack widths at 4” spacing were measured. This is very similar to the crack widths and spacing observed during the intended \( \frac{3}{4} F_y \) cycle earlier in the load history to ductility 1.13. The same crack width and spacing were measured during \( (\mu_1^{-3} = -0.79") \). At \( (\mu_1^{+1} = 1.19") \) visible flaking of the cover concrete was observed on the South side of the specimen which usually occurs just before crushing. Crushing over the bottom two inches of the cover concrete on the south side occurred during \( (\mu_1^{+3} = 1.20") \), as shown in the left photo of Figure 3.222. During this same cycle cracks on the North side of the specimen were measured at 1.25mm at 4” spacing. The extent of crushing on the North
side of the specimen reached 7” above the footing during \((\mu_{1,5}^{-3} = 1.19”)\), see the right photo of Figure 3.222. Here, the cracks on the South side of the specimen increased to 1.5mm width at 4” spacing. Cracks on the North side of the specimen measured 1.5mm at 4” spacing at \((\mu_{2}^{+3} = 1.58”)\) while the extent of crushing on the South side of the specimen reached 7” above the footing. The extent of crushing on the North side of the specimen did not increase during ductility two, but it spread to other uncrushed locations near the base of the column. The extent of crushing on the South side of the specimen increased to 10” above the footing and widened during \((\mu_{3}^{+3} = 2.40”)\). The crushing on North side of the specimen widened, but did not increase in height during ductility three.

Figure 3.221 T14 – Crack Distribution after All Cycles with Actuator Load Cell Errors (No Notable Limit States Reached)
Figure 3.222  T14 – (Left) South Cover Crushing during ($\mu_{1.5}^3 = 1.20''$) and (Right) Cover Crushing on the North Side of the Specimen at ($\mu_{1.5}^3 = 1.19''$)

The extent of crushing on the North and South sides of the specimen during the third cycle of ductility four is shown in Figure 3.223. Extreme fiber North reinforcing bar N3 buckled after reversal from ($\mu_6^{+1} = 4.80''$), see the left photo of Figure 3.224. All three instrumented bars on the South side of the specimen (S2, S3, and S4) buckled after reversal from ($\mu_6^{-1} = -4.80''$), as shown in the right photo of Figure 3.224. During the reversal from ($\mu_6^{+2} = 4.82''$), additional North reinforcement bars N2 and N4 buckled, Figure 3.225. North reinforcing bars N1 and N5 buckled during ($\mu_6^{-3} = -4.80''$). At the end of ductility six there was 9% strength loss in the push direction of loading and 12% strength loss in the pull direction due to buckled reinforcing bars and loss of confinement prior to rupture. On the way to ($\mu_8^{+1} = 6.40''$), South reinforcing bar S1 buckled. Separation of the deformed spiral North reinforcing bars, while they were in tension, is shown in Figure 3.226. During ($\mu_8^{-1} = -6.39''$), previously buckled South reinforcing bars S2, S3, and S4 ruptured causing a 48% loss in strength, Figure 3.227. On the way to ($\mu_8^{+2} = 6.40''$), previously buckled North reinforcing bars N2 and N3 ruptured causing a 45% loss in strength, as shown in the
left photo of Figure 3.228. North reinforcement bars N4 and N5 ruptured on the way to $(\mu_8^{+2} = 6.40\)'' causing a 67% loss in strength. The test was concluded at this time and photos which show the specimen after instrumentation and debris were removed appear in Figure 3.228. A photo progression of the crack propagation on the back side of the specimen is shown in Figure 3.229.

![Figure 3.223 T14 – (Left) Extent of Crushing on the South Side of the Specimen during $(\mu_4^{+3} = 3.19\)'' and (Right) Extent of Crushing on the North Side during $(\mu_4^{-3} = 3.20\)''](image-url)
Figure 3.224  T14 – (Left) Buckling of Bar N3 during \( \mu_{\delta}^1 = -4.80'' \) and (Right) Buckling of South Reinforcing Bars S2, S3, and S4 during \( \mu_{\delta}^{+2} = 4.82'' \)

Figure 3.225  T14 – (Left) Buckling of North Bars N2 and N4 during \( \mu_{\delta}^{-2} = -4.80'' \) and (Right) Deformation in Buckled Bars S2, S3, and S4 during \( \mu_{\delta}^{+3} = 4.83'' \)
Figure 3.226 T14 – (Left) Separation of Deformed Spiral Layer from Buckled Bar N3 at \( \mu_B^{+1} = 6.40" \) and (Right) Additional Deformation in Buckled South Bars S1, S2, S3, and S4 during \( \mu_B^{+1} = 6.40" \)

Figure 3.227 T14 – (Left) Deformation in North Buckled Bars during \( \mu_B^{-1} = -6.39" \) and (Right) Rupture of South Buckled Bars S2, S3, and S4 during \( \mu_B^{-1} = -6.39" \)
Figure 3.228  T14 – (Left) Rupture of North Buckled Bars N2 and N3 during ($\mu_{8}^{+2} = 6.40''$) and (Right) Front of the Specimen after the Conclusion of the Test
Figure 3.229  T14 – Crack Propagation and Orientation on the Back of the Specimen
Test 14 – Symmetric Three Cycle Set (#3 @ 4”) Strain Data Analysis

North Reinforcement

Vertical strain profiles for each extreme fiber bar during push and pull cycles appear in Figure 3.230 and Figure 3.231 respectively. Measured strains at the first occurrence of ductility 1 and 1.5 are also shown due to the initial overload cycles while the actuator load cell problems were being resolved. A peak tensile strain of 0.0348, at a height of 6.95” above the footing, was measured for extreme fiber bar N3 during ($\mu_6^{+1} = 4.80”$) before the bar buckled after reversal of load. The relationship between tensile strain and displacement for this gage length appears in Figure 3.234. Similar to previous tests, the moment curvature prediction with the PCK (2007) Lp Hinge Method begins to over predict the measured tensile strains at higher displacements at an increasing rate. The largest compressive strain of -0.011, located 4.92” above the footing, was measured during ($\mu_4^{-3} = -3.20”$). The relationship between compressive strain and displacement for bar N3 during pull cycles appears in Figure 3.235 for the gage length 4.92” above the footing. Here the measured compressive strains are slightly larger than the moment curvature prediction, but the overall trend is captured through displacement ductility three.

The strain hysteresis for the largest tensile gage length on extreme fiber bar N3 is shown in Figure 3.238 with a color bar to track the progression of the test. The strain hysteresis is plotted through ($\mu_6^{-1} = -4.80”$) when the bar buckled. Prior to bar buckling, the spiral layers on the North side of the specimen remained elastic, as shown in Figure 3.233. After reversal from ($\mu_6^{+1} = 4.80”$), the relationship between strain and displacement begins to break away from the trend at around 1”, which coincides with the visual buckling observation. This gage length is centered over a layer of transverse steel with the largest tensile crack. Since the outward buckling of bar N3 occurred between layers of transverse steel, this particular gage length just above the outward buckled region shortens as the deformation increases. To illustrate this point, the strain hysteresis over the outward buckled region of bar N3, located 4.92” above the footing, appears in Figure 3.239. The transverse
steel strain gage hysteresis for a spiral layer restraining buckled bar N3 is shown in Figure 3.240. In all three graphs, buckling looks like it occurred between 0-1” of displacement after reversal from ($\mu_6^+ = 4.80$”). Here the gage length over the outward buckled region begins to rapidly elongate and the transverse steel restraint tensile strain sharply increases.

**South Reinforcement**

A peak tensile strain of 0.035 in extreme fiber bar S3 was measured 3.61” above the footing during ($\mu_6^- = -4.80$”). Debris was blocking the lowest gage length of bar S3 during this cycle, so it is unclear whether higher strains occurred. The relationship between tensile strain and displacement for bar S3 is shown in Figure 3.236 for the gage length located 3.61” above the footing. A peak compression strain of -0.0152 was measured 7.62” above the footing during ($\mu_6^+ = 4.80$”). This particular gage length did not have the largest compressive strains during earlier cycles. The relationship between compressive strain and displacement for the gage length 3.61” above the footing appears in Figure 3.237. The measured strains match the moment curvature prediction with the PCK (2007) Lp Hinge Method. For the gage length 7.62” above the footing, the compression strain sharply increased during the first push cycle of ductility six.

Transverse steel strains in the lowest six spiral layers on the South side of the specimen are shown in Figure 3.232. During ($\mu_6^+ = 4.80$”), compressive demands the South side of the specimen caused the transverse steel to enter the inelastic range. The strain hysteresis for the gage length overlaying the outward buckled region of bar S3, 3.61” above the footing, appears in Figure 3.241. After reversal from a peak tensile strain of 0.035 at ($\mu_6^- = -4.80$”), the relationship between strain and displacement begins to break from the trend around 1” which agrees with visible buckling observations during the test. The transverse steel strain gage hysteresis for a spiral layer over the South buckled region is shown in Figure 3.242.
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Figure 3.230 T14 – Extreme Fiber Vertical Strain Profiles for Push Cycles

Figure 3.231 T14 – Extreme Fiber Vertical Strain Profiles during Pull Cycles
Chapter 3: Experimental Observations

Figure 3.232 T14 – Spiral Strains for the Lowest Six Layers on the South Side

Figure 3.233 T14 – Spiral Strains for the Lowest Six Layers on the North Side
Figure 3.234  T14 – Tensile Strain-Displacement for Bar N3 during Push Cycles

Figure 3.235  T14 – Compressive Strain-Displacement for Bar N3 during Pull Cycles
Figure 3.236 T14 – Tensile Strain-Displacement for Bar S3 during Pull Cycles

Figure 3.237 T14 – Compressive Strain-Displacement for Bar S3 during Push Cycles
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Figure 3.238 T14 – Bar N3 Strain Hysteresis (6.95” Above Footing)

Figure 3.239 T14 – Bar N3 Strain Hysteresis (4.92” Above Footing)
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Figure 3.240  T14 – Spiral Strain Gage Hysteresis over North Buckled Region

Figure 3.241  T14 – Bar S3 Strain Hysteresis (3.61” Above Footing)
Figure 3.242 T14 – Spiral Strain Gage Hysteresis over South Buckled Region

Test 14 – Symmetric Three Cycle Set (#3 @ 4”) Curvature and Strain Penetration Data

Vertical curvature profiles for push and pull cycles appear in Figure 3.243 and Figure 3.244 respectively. Plastic curvatures at higher ductility levels have a linear distribution as shown by the linear least squared error lines. The base section reinforcement slip measured at the footing-column interface can be monitored using the Optotrak system. The slip hysteresis for North and South extreme fiber bars appears in Figure 3.245 and Figure 3.246. The base section rotation due to strain penetration during push and pull cycles is shown in Figure 3.247 and Figure 3.248 respectively. The top column displacement can be calculated by integrating the measured curvature profiles, extrapolating the base section rotation to the center of loading, and assuming an elastic curvature distribution above the instrumented region. A comparison of measured and integrated top column displacements appears in Figure 3.249. The measured spread of plasticity for Test 14 is shown in Figure 3.250. The extent of plasticity is the intersection of the plastic regression and elastic curvature profiles.
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Figure 3.243 T14 – Vertical Curvature Profiles during Push Cycles

Figure 3.244 T14 – Vertical Curvature Profiles during Pull Cycles
Figure 3.245  T14 – Bar N3 Base Section Slip Hysteresis due to Strain Penetration

Figure 3.246  T14 – Bar S3 Base Section Slip Hysteresis due to Strain Penetration
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Figure 3.247 T14 – Base Section Rotation due to Strain Penetration during Push Cycles

Figure 3.248 T14 – Base Section Rotation due to Strain Penetration during Pull Cycles
Figure 3.249  T14 – Comparison of Measured and Optotrak Integrated Displacements

Figure 3.250  T14 – Measured Spread of Plasticity (Circular Data Points)
3.2.3 Test 15 – Three Cycle Set Load History with #3 Spiral at 2.75” (0.7%) 

Table 3.21 Observational Summary for Test 15 – Cyclic with #3 Spiral at 2.75” (0.7%) 

VALUES OF INTEREST:

Concrete Compressive Strength: \( f'_c = 7232 \text{ psi} \)
Axial Load: \( P = 170 \text{ kips} \)
Analytical First Yield Force: \( F'_y = 47.1 \text{ kips} \)
Experimental First Yield Displacement: \( \Delta'_y = 0.62" \)
Analytical Nominal Moment Capacity: \( M_n = 506.9 \text{ kip} \times \text{ ft} \)
Equivalent Yield Displacement: \( \Delta_y = 0.84" \)
Maximum Lateral Force: \( 68.6 \text{ kips} \)
Failure Mode: Fracture of Previously Buckled Reinforcement 

DAMAGE OBSERVATIONS:

First Cracking North: \( 1/2Fy' = 0.16" \)
First Cracking South: \( -1/2Fy' = -0.20" \)
Cover Concrete Crushing North: \( \mu_{1.5}^3 = -1.25" \)
Cover Concrete Crushing South: \( \mu_{2}^1 = 1.68" \)
Transverse Steel Yield North: At \(-1.89"\) during pull to \( \mu_{6}^{-1} = -5.00" \)
Transverse Steel Yield South: At \(2.08"\) during push to \( \mu_{4}^{+2} = 3.33" \)
Longitudinal Bar Buckling North: Reversal from \( \mu_{6}^{+2} = 5.00" \)
Longitudinal Bar Buckling South: Reversal from \( \mu_{6}^{-1} = -5.00" \)
Longitudinal Bar Fracture North: At \(3.91"\) during push to \( \mu_{8}^{+2} = 6.67" \)
Longitudinal Bar Fracture South: At \(-2.54"\) during pull to \( \mu_{8}^{-1} = -6.69" \)

\(*\mu_{6}^{-1} = -5.00"\) represents the first pull cycle of displacement ductility six
### Table 3.22 Strain Data Summary for Test 15 – Cyclic with #3 Spiral at 2.75” (0.7)%

<table>
<thead>
<tr>
<th>MATERIAL STRAINS:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North:</td>
<td>$\varepsilon_s = 0.0027$ (compression)</td>
</tr>
<tr>
<td>Cover Concrete Crushing South:</td>
<td>$\varepsilon_s = 0.0041$ (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield North:</td>
<td>$\varepsilon_s = 0.0199$ (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield South:</td>
<td>$\varepsilon_s = 0.0125$ (compression)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North:</td>
<td>$\varepsilon_s = 0.037$ (peak tension prior to $bb$) $\varepsilon_s = 0.020$ (peak comp. prior to $bb$)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South:</td>
<td>$\varepsilon_s = 0.038$ (peak tension prior to $bb$) $\varepsilon_s = 0.023$ (peak comp. prior to $bb$)</td>
</tr>
<tr>
<td>Mander (1988) Ultimate Concrete Compression Strain, $\varepsilon_{cu} = 0.0129$</td>
<td></td>
</tr>
</tbody>
</table>

![Figure 3.251 T15 – Cross Section Bar Designation and Target Marker Application](image-url)
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Figure 3.252 T15 – Symmetric Three Cycle Set Load History

Figure 3.253 T15 – Lateral Force vs. Top Column Displacement Response
Figure 3.254  T15 – Compressive Axial Load from One Jack (Total = 2*Value)

Figure 3.255  T15 – Rupture History of Previously Buckled Reinforcement
Test 15 – Symmetric Three Cycle Set (#3 @ 2.75”) Experimental Observations

The first yield force for the tested material and geometric properties was determined using moment curvature analysis (Test 15: Cumbia Fy’ = 47.11 kips with f’c = 7232 psi). The first yield displacement was obtained as an average for the first yield push and pull cycles (Δy’ = 0.62”). Vertical strain profiles for both extreme fiber bars during push and pull cycles up to the first yield force appear in Figure 3.256 with a dashed line representing the yield strain of the longitudinal reinforcement. The equivalent yield displacement, used to determine the displacement ductility levels (μΔn = n * Δy), is then calculated as Δy = Δy’(Mn/My’) = 0.84” for Test 15. The full symmetric three-cycle-set load history appears in Figure 3.252 and the resulting lateral force vs. top column displacement hysteresis is shown in Figure 3.253.

The test began with cycles in ¼ Fy’ (first yield force) increments in each direction of loading until the first yield force was reached. The first cracks on the North side of the specimen measured 0.1mm at approximate 10” spacing at (1/2Fy’ = 23.27 kips). Cracks on the South Side of the specimen measured 0.1mm at approximate 10” spacing during
Chapter 3: Experimental Observations

\((-1/2Fy' = -23.47 kips)\). The largest crack widths on the North side of the specimen reached 0.2mm at approximate 5” spacing at \((3/4Fy' = 35.23 kips)\). Cracks measured at 0.3mm with 5” spacing were observed on the South side of the specimen during \((-3/4Fy' = -35.16 kips)\). During the first yield push cycle \((Fy' = 47.13 kips, 0.62")\), the largest crack widths measured 0.35mm at approximate 5” spacing. During the subsequent pull cycle \((-Fy' = -46.81 kips, -0.63")\), crack widths reached 0.4mm on the South side of the specimen. The crack distribution at first yield is shown in Figure 3.257. The progression of the crack distribution on the back side of the specimen is shown in Figure 3.267. Cracks on the North side of the specimen measured 0.5mm at approximate 4” spacing during \((\mu_1^3 = 0.85")\). After reversal, cracks on the South side reached 0.55mm at 5” spacing during \((\mu_1^{-3} = -0.84")\). Visible flaking of the cover concrete in compression, which is a precursor to crushing, was observed on the South side of the specimen during \((\mu_{1,5}^{+1} = 1.24")\). A similar observation on the North side of the specimen occurred during \((\mu_{1,5}^{-1} = -1.24")\).

Cracks on the North side of the specimen measured 1mm during \((\mu_{1,5}^{+3} = 1.25")\). Crushing of the cover concrete on the North side of the specimen was observed during \((\mu_{1,5}^{-3} = -1.25")\), see the left photo of Figure 3.258. Here, the largest crack width on the South side of the specimen reached 1.25mm. Crushing on the South side of the specimen did not occur until \((\mu_2^{+1} = 1.68")\), as shown in the right photo of Figure 3.258. The largest crack width on the North side of the specimen measured 1.25mm during \((\mu_2^{+3} = 1.66")\). The extent of crushing on the South side of the specimen increased to 13” above the footing during \((\mu_{3,5}^{+3} = 2.49")\), as shown in the right photo of Figure 3.259. On the North side of the specimen, the extent of crushing reached 10 ¾” above the footing during \((\mu_3^{-3} = -2.51")\). The extent of crushing on the South side of the specimen reached 24 ½” above the footing during \((\mu_6^{+1} = 5.01")\). Crushing on the North side of the specimen reached 16 ¼” above the footing at \((\mu_6^{-1} = -5.00")\), as shown in the left photo of Figure 3.260. The first push and pull cycles of ductility six were concluded without visible buckling on either side of the specimen.
South reinforcing bars S2 and S3 visibly buckled on the way to \((\mu_6^{+2} = 5.00")\), as shown in Figure 3.261. Buckling of the two South bars caused a 5.5\% loss of strength from the peak load of 68.37 kips measured during \((\mu_6^{+1} = 5.01")\). North reinforcing bars N2 and N3 visibly buckled on the way to \((\mu_6^{-2} = -5.01")\), as shown in Figure 3.262. Buckling of the two North bars caused a 5\% loss of strength from the peak load of -68.51 kips, which occurred during \((\mu_6^{-1} = -5.00")\). An additional South reinforcing bar S4 buckled during \((\mu_6^{+3} = 4.99)\) and the outward deformation in bars S2 and S3 increased, which lead to a 9.3\% loss in strength. North reinforcing bar N4 buckled during \((\mu_6^{-3} = -5.00")\) causing an 8.1\% loss in strength relative to the peak load in the pull direction. During the first push cycle of ductility eight, an 11.75\% loss of strength was observed without additional buckling or rupture of reinforcement. The effect of buckling on confinement loss is highlighted by observed permanent deformation in spiral layers over the North reinforcement when the bar was placed back into tension, see Figure 3.263.

Previously buckled bars S2 and S3 ruptured in tension during \((\mu_8^{-1} = -6.69")\), see Figure 3.264. Rupture of the two South bars lead to a 32.2\% total loss in strength, as shown in Figure 3.255 on the force vs. displacement response. During \((\mu_8^{+2} = 6.67")\), South bars S1 and S5 buckled and previously buckled North reinforcing bars N2 and N3 ruptured in tension, as shown in Figure 3.265. This caused a 40.72\% total loss of strength in the push direction of loading. During \((\mu_8^{-2} = -6.71")\), North bars N1 and N5 buckled and an additional bar S4 ruptured in tension leading to a 50.1\% loss in strength. North bar N4 ruptured during \((\mu_8^{+3} = 6.70")\) causing a 55.9\% loss in strength. During the final cycle of the load history \((\mu_8^{-3} = -6.66")\), South bar S1 ruptured leading to a total loss in strength of 65.4\%. Photos of the specimen after the test was concluded appear in Figure 3.266.
Figure 3.257 T15 – Crack Distribution at First Yield (Left) North and (Right) South

Figure 3.258 T15 – (Left) Crushing on the North Side of the Specimen during ($\mu_{1.5} = -1.25''$) and (Right) Crushing on the South Side during ($\mu_{2.1} = 1.68''$)
Figure 3.259: T15 – (Left) Crushing on the North Side of the Specimen during ($\mu_3^{-3} = -2.51''$) and (Left) Crushing on the South Side during ($\mu_3^{+3} = 2.49''$)

Figure 3.260: T15 – (Left) Extent of Crushing on the North Side of the Specimen during ($\mu_6^{-1} = -5.00''$) and (Right) Crushing on the South Side during ($\mu_6^{+1} = 5.01''$)
Figure 3.261  T15 – Buckling of Reinforcing Bars S2 and S3 during ($\mu_6^+ = 5.00\,$)
Figure 3.263 T15 – (Left) Permanent Deformation in North Spiral Layers at \( \mu_8^+ = 6.68'' \) and (Right) Deformation of Buckled Bars S2, S3, and S4 during \( \mu_8^+ = 6.68'' \)

Figure 3.264 T15 – (Left) Rupture of Previously Buckled Bars S2 and S3 during \( \mu_8^- = -6.69'' \) and (Right) Deformation in Bars N2, N3, and N4 at \( \mu_8^- = -6.69'' \)
Figure 3.265 T15 – (Left) Rupture of Previously Buckled Bars N2 and N3 during $(\mu_B^+ = 6.67")$ and (Right) Buckling of Bar S1 and S5 during $(\mu_B^+ = 6.67")$

Figure 3.266 T15 – After the Test (Left) North, (Middle) Front, and (Right) South Side
Figure 3.267 T15 – Crack Progression on the Back Side of the Specimen
Test 15 – Symmetric Three Cycle Set (#3 @ 2.75”) Strain Data Analysis

North Reinforcement

Extreme fiber vertical strain profiles for push and pull cycles appear in Figure 3.268 and Figure 3.269 respectively. As the hinge rotates about inclined flexural shear cracks, compressive strains are concentrated at the base and tensile strains are fanned out to a greater height following the inclined crack distribution. Near the footing cracks remain effectively horizontal, but above the base section flexural shear cracks are inclined as shown in Figure 3.267. The effects of tension shift increase as the cracks become more inclined at higher ductility levels. A peak tensile strain of 0.0372 was measured 2.31” above the footing for bar N3 during ($\mu_6^{+1} = 5.01$”). Bar N3 did not buckle until reversal from ($\mu_6^{+2} = 5.00”$), when the peak tensile strain was 0.0365. The relationship between tensile strain and displacement for this gage length appears in Figure 3.272. The solid line contains data during the push cycle loading up to the peak displacement and the dashed line represents the subsequent reversal of load. Similar to previous tests, the moment curvature prediction with the PCK (2007) Lp Hinge Method over predicts the tensile strain-displacement relationship at an increasing rate. The largest compressive strain of -0.0199, located 7.89” above the footing, was measured during ($\mu_6^{-1} = -5.00”$). The peak compressive strain of -0.0199 measured in bar N3 is 54.3% larger than the original Mander ultimate concrete compressive strain of -0.0129. The relationship between compressive strain and displacement for bar N3, gage length centered 2.31” above the footing, during pull cycles appears in Figure 3.273. Here the measured compressive strains are slightly larger than the moment curvature prediction with the PCK (2007) Lp Hinge Method, but the overall trend is captured. At the section 7.89” above the footing, the relationship between strain and displacement does not match as well at higher ductility levels.

The strain hysteresis for the largest tensile gage length, 2.31” above the footing, on extreme fiber bar N3 is shown in Figure 3.276 with an elapsed time color bar to track the progression of the test. The strain hysteresis is plotted through ($\mu_6^{-2} = -5.01”$) when the bar
visibly buckled. After reversal from \((\mu_6^{+2} = 5.00")\), the relationship between strain and displacement begins to break away from the trend at around zero displacement. This gage length is centered over a layer of transverse steel where the largest crack was located. Since the outward buckled deformation of bar N3 occurred between layers of transverse steel, this particular gage length just below the outward buckled region shortens with increased deformation. To illustrate this point, the strain hysteresis over the outward buckled region of bar N3, located 5.11" above the footing, appears in Figure 3.277. The transverse steel strain gage hysteresis for a spiral layer restraining buckled bar N3 is shown in Figure 3.278. In all three graphs, buckling looks like bar buckling occurred between 0-2” of displacement after reversal from \((\mu_6^{+2} = 5.00")\). Here the gage length over the outward buckled region begins to elongate as the overlaying spiral strain increases. The transverse steel restraint on the North side of the specimen went into the inelastic range during \((\mu_6^{-1} = -5.00")\), as shown in Figure 3.271. Even though the transverse steel was inelastic during this cycle, visibly buckling was not observed. The strain hysteresis for the gage length located 5.11” above the footing on bar N3, in Figure 3.277, shows that some measurable deformation occurred during \((\mu_6^{-1} = -5.00")\). The curve which represents the reversal from \(\mu_6^{+1}\) to \(\mu_6^{-1}\) breaks away from the trend set by previous cycles.

*South Reinforcement*

A peak tensile strain of 0.0347 was measured 4.64” above the footing on bar S3 during \((\mu_6^{-1} = -5.00")\). When the loading of the specimen was paused at \(\mu_6^{-1}\), debris was removed and the peak tensile strain over the base gage length measured 0.0378. The relationship between tensile strain and displacement for bar S3 is shown in Figure 3.274 for the gage length located 4.64” above the footing. A peak compression strain of -0.0233 was measured 7.47” above the footing on bar S3 during \((\mu_6^{+1} = 5.01")\). The peak value is 80.6% larger than the original Mander (1988) ultimate concrete compressive strain of -0.0129. The relationship between compressive strain and displacement for the gage length 2.03” above the footing on bar S3 during push cycles appears in Figure 3.275. This gage length
represents the base section, where a peak compression strain of -0.0115 was measured during $(\mu_{6}^{+1} = 5.01")$.

The strain hysteresis for the gage length overlaying the outward buckled region of bar S3, 4.64” above the footing, appears in Figure 3.279. After reversal from a peak tensile strain of 0.0378 at $(\mu_{6}^{-1} = -5.00")$, the relationship between strain and displacement begins to break from the trend at around -3”, which agrees with the visible buckling observation. The transverse steel strain gage hysteresis for a spiral layer restraining the top portion of the outward buckled region is shown in Figure 3.280. The strain hysteresis for the spiral layer restraining the lower portion of the outward buckled region appears in Figure 3.281. The second spiral layer above the footing was inelastic by the time the specimen reached $(\mu_{6}^{+1} = 5.01")$, see Figure 3.270. The South reinforcing bars S2 and S3 buckled during the push cycle to $(\mu_{6}^{+2} = 5.00")$. The measured strain in the upper spiral layer continued to rapidly increase while the lower spiral layer entered the inelastic range for the first time. The data suggests that buckling of bar S3 began at around -3”. 
Chapter 3: Experimental Observations

Figure 3.268 T15 – Extreme Fiber Vertical Strain Profiles during Push Cycles

Figure 3.269 T15 – Extreme Fiber Vertical Strain Profiles during Pull Cycles
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Figure 3.270  T15 – Spiral Strains on the South Side during Push Cycles

Figure 3.271  T15 – Spiral Strains on the North Side during Pull Cycles
Chapter 3: Experimental Observations

Figure 3.272 T15 – Tensile Strain-Displacement for Bar N3 during Push Cycles

Figure 3.273 T15 – Compressive Strain-Displacement for Bar N3 during Pull Cycles
Figure 3.274  T15 – Tensile Strain-Displacement for Bar S3 during Pull Cycles

Figure 3.275  T15 – Compressive Strain-Displacement for Bar S3 during Push Cycles
Visible Buckling of Bar N3

Figure 3.276  T15 – Strain Hysteresis over the Buckled Region of Bar N3 (2.31” Above)

Visible Buckling of Bar N3

Figure 3.277  T15 – Strain Hysteresis over the Buckled Region of Bar N3 (5.11” Above)
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Figure 3.278  T15 – Spiral Strain Hysteresis over the Buckled Region of Bar N3

Figure 3.279  T15 – Strain Hysteresis over the Buckled Region of Bar S3 (4.64” Above)
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Figure 3.280  T15 – Spiral Strains over the Buckled Region of Bar S3 (2\textsuperscript{nd} Layer)

Figure 3.281  T15 – Spiral Strains over the Buckled Region of Bar S3 (1\textsuperscript{st} Layer)
Test 15 – Cyclic Load History (#3 @ 2.75”) Curvature and Strain Penetration Data

Vertical curvature profiles are plotted for push and pull cycles as shown in Figure 3.282 and Figure 3.283 respectively. These figures show that plastic curvatures have a linear distribution at higher displacement ductility levels. As the displacements increase, the base curvatures become larger and the extent of plastic curvatures reach higher above the footing. The effects of strain penetration of longitudinal reinforcement into the footing can be measured with the LEDs placed closest to the footing-column interface. The slip hysteresis for the North and South extreme fiber bars appear in Figure 3.284 and Figure 3.285. The base rotation attributable to strain penetration is obtained by looking at the measured slip of all six instrumented bars, as shown in Figure 3.286 and Figure 3.287 for push and pull respectively. The base rotation is equal to the slope of the least squared error line connecting the measured values.

The top column displacement can be determined using the Optotrak system by integrating the measured curvatures, extrapolating the base rotation to the center of loading, and assuming an elastic distribution of curvature above the instrumented region. A comparison of the measured top column displacements and the Optotrak integrated displacements appears in Figure 3.288. The two methods agree well throughout the entire test. The measured spread of plasticity for Test 14 is shown in Figure 3.289 as a function of base section curvature ductility. The extent of plasticity is the intersection of the linear regression for the plastic curvature profile and the elastic curvature distribution.
Figure 3.282 T15 – Vertical Curvature Profiles during Push Cycles

Figure 3.283 T15 – Vertical Curvature Profiles during Pull Cycles
Figure 3.284  T15 – Bar N3 Base Section Slip Hysteresis due to StrainPenetration

Figure 3.285  T15 – Bar S3 Base Section Slip Hysteresis due to StrainPenetration
Figure 3.286  T15 – Base Rotation due to Strain Penetration during Push Cycles

Figure 3.287  T15 – Base Rotation due to Strain Penetration during Pull Cycles
Chapter 3: Experimental Observations

Figure 3.288 T15 – Comparison of Measured and Optotrak Integrated Displacements

Figure 3.289 T15 – Measured Spread of Plasticity (Circular Data Points)
### 3.2.4 Test 16 – Three Cycle Set Load History with #3 Spiral at 1.5” (1.3%)  

**Table 3.23 Observational Summary for Test 16 – Cyclic with #3 Spiral at 1.5” (1.3%)**

<table>
<thead>
<tr>
<th>VALUES OF INTEREST:</th>
<th></th>
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</thead>
<tbody>
<tr>
<td>Concrete Compressive Strength:</td>
<td>$f'_c = 6711 \text{ psi}$</td>
</tr>
<tr>
<td>Axial Load:</td>
<td>$P = 170 \text{ kips}$</td>
</tr>
<tr>
<td>Analytical First Yield Force:</td>
<td>$F'_y = 46.8 \text{ kips}$</td>
</tr>
<tr>
<td>Experimental First Yield Displacement:</td>
<td>$\Delta'_y = 0.62&quot;$</td>
</tr>
<tr>
<td>Analytical Nominal Moment Capacity:</td>
<td>$M_n = 503.2 \text{ kip } \ast \text{ ft}$</td>
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<tr>
<td>Equivalent Yield Displacement:</td>
<td>$\Delta_y = 0.83&quot;$</td>
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<tr>
<td>Maximum Lateral Force:</td>
<td>$70.7 \text{ kips}$</td>
</tr>
<tr>
<td>Failure Mode:</td>
<td>Fracture of Previously Buckled Reinforcement</td>
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</table>

<table>
<thead>
<tr>
<th>DAMAGE OBSERVATIONS:</th>
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</thead>
<tbody>
<tr>
<td>First Cracking North:</td>
<td>$1/2Fy' = 0.17&quot;$</td>
</tr>
<tr>
<td>First Cracking South:</td>
<td>$-1/2Fy' = -0.19&quot;$</td>
</tr>
<tr>
<td>Cover Concrete Crushing North:</td>
<td>$\mu_2^{-3} = -1.65&quot;$</td>
</tr>
<tr>
<td>Cover Concrete Crushing South:</td>
<td>$\mu_2^{+1} = 1.66&quot;$</td>
</tr>
<tr>
<td>Transverse Steel Yield North:</td>
<td>At $-4.98&quot;$ during pull to $\mu_6^{-1} = -4.98&quot;$</td>
</tr>
<tr>
<td>Transverse Steel Yield South:</td>
<td>At $3.80&quot;$ during push to $\mu_6^{+1} = 4.99&quot;$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North:</td>
<td>Reversal from $\mu_6^{+2} = 5.00&quot;$</td>
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<tr>
<td>Longitudinal Bar Buckling South:</td>
<td>Reversal from $\mu_6^{-1} = -4.98&quot;$</td>
</tr>
<tr>
<td>Longitudinal Bar Fracture North:</td>
<td>At $3.68&quot;$ during push to $\mu_{10}^{+2} = 8.32&quot;$</td>
</tr>
<tr>
<td>Longitudinal Bar Fracture South:</td>
<td>At $-2.64&quot;$ during pull to $\mu_{10}^{-1} = -8.34&quot;$</td>
</tr>
</tbody>
</table>

* $\mu_6^{-1} = -4.98"$ represents the first pull cycle of displacement ductility six.
Table 3.24 Strain Data Summary for Test 16 – Cyclic with #3 Spiral at 1.5” (1.3%)

MATERIAL STRAINS:

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<thead>
<tr>
<th>Material Type</th>
<th>Strain Value</th>
<th>Description</th>
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<tbody>
<tr>
<td>Cover Concrete Crushing North</td>
<td>$\varepsilon_s = 0.0048$</td>
<td>(compression)</td>
</tr>
<tr>
<td>Cover Concrete Crushing South</td>
<td>$\varepsilon_s = 0.0038$</td>
<td>(compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield North</td>
<td>$\varepsilon_s = 0.0120$</td>
<td>(compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield South</td>
<td>$\varepsilon_s = 0.0152$</td>
<td>(compression)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North</td>
<td>$\varepsilon_s = 0.056$</td>
<td>(peak tension prior to $bb$)</td>
</tr>
<tr>
<td></td>
<td>$\varepsilon_s = 0.019$</td>
<td>(peak comp. prior to $bb$)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South</td>
<td>$\varepsilon_s = 0.052$</td>
<td>(peak tension prior to $bb$)</td>
</tr>
<tr>
<td></td>
<td>$\varepsilon_s = 0.030$</td>
<td>(peak comp. prior to $bb$)</td>
</tr>
<tr>
<td>Mander (1988) Ultimate Concrete Compression Strain, $\varepsilon_{cu} = 0.0193$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.290 T16 – Cross Section Bar Designation and Target Marker Application
Figure 3.291  T16 – Symmetric Three Cycle Set Load History

Figure 3.292  T16 – Lateral Force vs. Top Column Displacement Response
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Figure 3.293  T16 – Compressive Axial Load from One Jack (Total = 2*Value)

Figure 3.294  T16 – Bar Fracture History of Previously Buckled Reinforcement
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Test 16 – Symmetric Three Cycle Set (#3 @ 1.5”) Experimental Observations

The test began with cycles in $\frac{1}{4}$ Fy’ (first yield force) increments in each direction of loading until the first yield force was reached. The first cracks on the North side of the specimen measured 0.1mm at approximate 9” spacing at ($1/2Fy’ = 23.30$ kips). Cracks on the South Side of the specimen measured 0.1mm at approximately 7” spacing during ($-1/2Fy’ = -23.40$ kips). The largest crack widths on the North side of the specimen reached 0.2mm at approximate 6” spacing at ($3/4Fy’ = 35.06$ kips). Crack widths measuring 0.3mm with 6” spacing were observed on the South side of the specimen during ($-3/4Fy’ = -34.46$ kips). During the first yield push cycle ($Fy’ = 46.85$ kips, 0.61”), the largest crack widths measured 0.3mm at approximate 6” spacing. During the subsequent pull cycle ($-Fy’ = -46.92$ kips, -0.63”), crack widths reached 0.4mm at approximate 5” spacing on the South side of the specimen. The crack distribution at first yield is shown in Figure 3.295. The progression of the crack distribution on the back side of the specimen is shown in Figure 3.303. Cracks on the North side of the specimen measured 0.6mm at approximate 4” spacing during ($\mu_1^+ = 0.83”$). After reversal, cracks on the South side reached 0.6mm at 6” spacing at ($\mu_1^- = -0.84”$).

Visible flaking of the cover concrete in compression, which is a precursor to crushing, was observed on the South side of the specimen during ($\mu_{1}^{+3} = 1.34”$). While the displacement for this cycle was intended to reach 1.25”, a slight overload to 1.34” occurred. The largest crack width on the North side of the specimen measured 0.9mm, located 10” above the footing, during ($\mu_{1}^{+3} = 1.24”$). Crushing on the South side of the specimen 2” above the footing was observed during ($\mu_2^{+1} = 1.66”$), see the left photo of Figure 3.296. Visible cover concrete flaking on the North side of the specimen did not occur until ($\mu_2^{-1} = -1.66”$). During ($\mu_2^{-3} = -1.65”$), the largest crack width on the South side of the specimen measured 1.5mm and cover concrete crushing on the North side of the specimen reached 5” above the footing as shown in the right photo of Figure 3.296. The extent of crushing on the South side of the specimen reached 15” above the footing during ($\mu_3^{+3} = 2.50”$), as shown in the left photo of Figure 3.297. Crushing on the North side of the specimen extended 11”
above the footing during \((\mu_3^{-3} = -2.50\text{")\), see the right photo of Figure 3.297. During
\((\mu_4^{+3} = 3.33\text{")\) and \((\mu_4^{-3} = -3.33\text{")\) the extent of crushing on the South and North side of the
specimen reached 15” and 13” above the footing respectively.

Crushing on the North and South sides of the specimen both reached 25” above the
footing during \((\mu_6^{+3} = 5.00\text{")\) and \((\mu_6^{-3} = -4.99\text{")\), as shown in Figure 3.298. After reversal
from \((\mu_6^{-1} = -6.68\text{")\), south extreme fiber bar S3 buckled as shown in the left and middle
photos of Figure 3.299. After reversal from \((\mu_9^{+3} = 6.65\text{")\), north extreme fiber bar N3 and
adjacent bar N2 buckled, see the right photo of Figure 3.299. Even though rupture of the
North reinforcement did not occur during \((\mu_{10}^{+2} = 8.32\text{")\), a 5.7% loss in strength was
observed due only to buckled bars on each side of the specimen during \((\mu_{10}^{+1} = 8.29\text{")\). An
additional South reinforcing bar S2 buckled during \((\mu_{10}^{+1} = 8.29\text{")\), as shown in the left photo
of Figure 3.300.

During \((\mu_{10}^{-1} = 8.34\text{")\), previously buckled South reinforcing bar S3 ruptured causing a
19.5% loss in strength, see the right photo of Figure 3.300. North reinforcing bars N1 and
N4 also buckled during \((\mu_{10}^{-1} = 8.34\text{")\), see the left photo of Figure 3.301. Previously
buckled North bars N2 and N3 ruptured during \((\mu_{10}^{+2} = 8.32\text{")\), leading to a 33.4% total loss
in strength, see the right photo of Figure 3.301. Additional South reinforcing bars S1 and S4
buckled during \((\mu_{10}^{+2} = 8.32\text{")\), as shown in Figure 3.302. Previously buckled South bars S2
and S4 ruptured during \((\mu_{10}^{-2} = -8.39\text{")\), causing a 49.7% total loss in strength. During
\((\mu_{10}^{+3} = 8.32\text{")\), North bars N1 and N5 ruptured leading to a total 64.7% loss in strength. At
this time the test was concluded. A graph plotting the rupture locations and corresponding
losses in strength on the hysteretic response appears in Figure 3.294.
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Figure 3.295 T16 – Crack Distribution at First Yield (Left) North and (Right) South

Figure 3.296 T16 – (Left) South Crushing during ($\mu_2^1 = 1.66''$) and (Right) North Crushing during ($\mu_2^3 = -1.65''$)
Figure 3.297 T16 – (Left) South Side at \( (\mu_3^+ = 2.50\)”, (Right) North at \( (\mu_3^- = -2.50\)”

Figure 3.298 T16 – (Left) South Crushing during \( (\mu_6^+ = 5.00\)” and (Right) North Crushing during \( (\mu_6^- = -4.99\)”)
Figure 3.299  T16 – (Left and Middle) Buckling of Bar S3 during ($\mu_8^{+2} = 6.64''$) and (Right) Buckling of Bar N2 and N3 during ($\mu_8^{-3} = 6.66''$)

Figure 3.300  T16 – (Left) Buckling of Bar S2 during ($\mu_{10}^{+1} = 8.29''$), (Middle) Deformation in Bar S3 at ($\mu_{10}^{+1} = 8.29''$), and (Right) Rupture of Bar S3 during ($\mu_{10}^{-1} = -8.34''$)
Figure 3.301  T16 – (Left) Buckling of N1 and N4 during ($\mu_{10}^1 = -8.34''$) and (Right) Rupture of N2 and N3 during ($\mu_{10}^2 = 8.32''$)

Figure 3.302  T16 – Buckling of S1 and S4 during ($\mu_{10}^2 = 8.32''$)
Figure 3.303 T16 – Crack Progression on the Back Side of the Specimen (North has Black Crack Markings while South has Red)
Test 16 – Symmetric Three Cycle Set (#3 @ 1.5”) Strain Data Analysis

North Reinforcement

Extreme fiber vertical strain profiles for push and pull cycles appear in Figure 3.304 and Figure 3.305 respectively. These figures show both extreme fiber bars on the same graph to illustrate the effects of tension shift on strain profiles. Compressive strains are concentrated near the base of the column while tensile strains are fanned out to a greater height above the footing following the inclined crack distribution shown in Figure 3.303. The compressive vertical strain profile for north extreme fiber bar N3 during pull cycles appears in the left half of Figure 3.305. While the overall shape of the compressive strain profile matches past observations, a single gage length located 14.8” above the footing shows tensile strains during compressive cycles. The calculations for this gage length have been checked, and an explanation for why this may occur is not available. At this same height on adjacent bars N2 and N4, compressive strains were measured as expected.

A peak tensile strain of 0.056, located 3.40” above the footing, was measured for North extreme fiber bar N3 during \( \mu_\text{8}^+ = 6.65” \). Bar N3 buckled after reversal from this peak tensile strain. The relationship between tensile strain and displacement for this gage length appears in Figure 3.308. The solid line contains data during the push cycle loading up to the peak and the dashed line represents the subsequent reversal of load. Similar to previous tests, the moment curvature prediction for the relationship between strain and displacement using the PCK (2007) Lp Plastic Hinge Method begins to over predict the tensile strains at higher displacements at an increasing rate. The largest compressive strain of -0.0187, located 7.70” above the footing, was measured during \( \mu_\text{8}^- = -6.68” \). The relationship between compressive strain and displacement for bar N3, gage length centered 3.4” above the footing, during pull cycles appears in Figure 3.309. Here the measured compressive strains deviate above or below the prediction depending on the displacement range, but the overall trend is captured.
The strains in the lowest six transverse steel layers restraining North extreme fiber bar N3 are plotted in Figure 3.307. The individual data points are from strain gages attached to each spiral layer at a specific height above the footing. The data points are connected with lines only to show trends for the particular displacement level. The vertical grey dashed line represents the yield strain of the transverse reinforcement. A single transverse steel layer, located 3.5” above the footing, entered the inelastic range during \( \mu_6^{-3} = -4.99" \). Compressive demands during \( \mu_8^{-1} = -6.68" \) led to three layers of transverse steel going into the inelastic range. Prior to buckling, the strain in the three inelastic spiral layers increased during \( \mu_8^{-2} = -6.64" \), even though the displacement level remained the same. When bar N3 latter buckled during \( \mu_8^{-3} = -6.66" \) the tensile strain for these spiral layers rapidly increased as they accommodated the outward deformation of the bar.

The strain hysteresis over the outward buckled region of bar N3, gage length located 3.40” above the footing, appears in Figure 3.312. It is clear that there was some measurable outward deformation during \( \mu_8^{-2} = -6.64" \), as shown by the blue arrow in Figure 3.312. Visible Buckling occurred after reversal from \( \mu_8^{+3} = 6.65" \), here the outward deformation begins to rapidly increase as indicated by the red arrow. The transverse steel strain gage hysteresis for the layer over the outward buckled region of bar N3 appears in Figure 3.313. The measurable deformation during \( \mu_8^{-2} = -6.64" \), shown by the blue arrow, increases the inelastic tensile strain in the spiral layer. Visible bar buckling after reversal from \( \mu_8^{+3} = 6.65" \) leads to a rapid increase in the spiral strain causing the gage to go beyond its measurable range.

**South Reinforcement**

A peak tensile strain of 0.052 on bar S3 was measured 7.76” above the footing during \( \mu_8^{-1} = -6.68" \). The relationship between tensile strain and displacement for bar S3 is shown in Figure 3.310 for the gage length located 7.75” above the footing. The same comments on the accuracy of the moment curvature prediction with the PCK (2007) Lp Hinge Method for the North reinforcement bar N3 also apply to bar S3. A peak compression
strain of -0.0303 was measured 4.89” above the footing during \( (\mu_{8}^{+1} = 6.64") \). The relationship between compressive strain and displacement for the gage length 4.89” above the footing on bar S3 during push cycles appears in Figure 3.311. The measured strains match the moment curvature prediction with the PCK (2007) Lp Hinge Method well through ductility two, but at higher ductility levels the measured compressive strains are significantly larger than the prediction. The peak compressive strain of -0.0303 measured in bar S3 is 57% larger than the original Mander (1988) ultimate concrete compressive strain of -0.0193.

The strains in the lowest six transverse steel layers restraining south extreme fiber bar S3 are plotted in Figure 3.306. Compressive demands during \( (\mu_{6}^{+3} = 6.64") \), led to two layers of transverse steel exceeding the yield strain. Prior to buckling, the strain in the two inelastic spiral layers increased and a third layer entered the inelastic range during \( (\mu_{8}^{+1} = 6.64") \). The strain hysteresis for the outward buckled region of extreme fiber bar S3, gage length located 3.37” above the footing, appears in Figure 3.314. The strain gage hysteresis for the spiral layer overlaying the outward buckled region of bar S3 appears in Figure 3.315. Visible buckling of bar S3 occurred after reversal from \( (\mu_{8}^{-1} = -6.68") \). During this reversal, measurable outward deformation over bar S3 occurred as shown by the increased tensile strains in Figure 3.314. As the bar deformed outwards, the spiral restraint tensile strain began to rapidly increase until the strain gage exceeded its maximum value by going off scale.
Figure 3.304  T16 – Extreme Fiber Vertical Strain Profiles during Push Cycles

Figure 3.305  T16 – Extreme Fiber Vertical Strain Profiles during Pull Cycles
Figure 3.306  T16 – Transverse Steel Strains on the South Side during Push Cycles

Figure 3.307  T16 – Transverse Steel Strains on the North Side during Pull Cycles
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Figure 3.308  T16 – Tensile Strain-Displacement for Bar N3 during Push Cycles

Figure 3.309  T16 – Compressive Strain-Displacement for Bar N3 during Pull Cycles
Figure 3.310  T16 – Tensile Strain-Displacement for Bar S3 during Pull Cycles

Figure 3.311  T16 – Compressive Strain-Displacement for Bar S3 during Push Cycles
Figure 3.312 T16 – Strain Hysteresis over the Buckled Region of Bar N3 (3.4” Above)

Figure 3.313 T16 – Spiral Strain Gage Hysteresis over the Buckled Region of Bar N3
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Figure 3.314 T16 – Strain Hysteresis over the Buckled Region of Bar S3 (3.37” Above)

Figure 3.315 T16 – Spiral Strain Gage Hysteresis over the Buckled Region of Bar S3
Test 16 – Cyclic Load History (#3 @ 1.5") Curvature and Strain Penetration Data

The cross section curvature for each horizontal section above the footing is determined by connecting the strain measurements from all six instrumented bars with a least squared error line. The curvature is then extracted from the slope of the least squared error line, see Figure 3.316. Vertical curvature profiles are plotted for push and pull cycles as shown in Figure 3.317 and Figure 3.318 respectively. These figures show that plastic curvatures have a linear distribution at higher displacement ductility levels. The extent of plastic curvatures above the footing can be calculated by determining where the linear plastic curvature distribution intersects the triangular yield curvature distribution, shown as a grey dashed line. The dashed lines for each curvature distribution represent a least squared error linear fit to the plastic portion of the measured curvatures. The measured spread of plasticity as a function of curvature ductility appears in Figure 3.121.

The target marker on each bar placed closest the footing-column interface can be used to create slip hysteresis and horizontal slip profiles attributable to strain penetration. The slip hysteresis for extreme fiber bars N3 and S3 appear in Figure 3.319 and Figure 3.320 respectively. If the tensile and compressive slip of all of the instrumented bars is plotted along the cross section depth, the base rotation attributable to strain penetration may be calculated. The slip profiles for push and pull cycles appear in Figure 3.321 and Figure 3.322 respectively. The rotation of the base section can be extracted from the slope of the least squared error line connecting all six measured bar slips.

Combining the curvatures over the instrumented region (4ft above the footing), bar slip profiles, and an elastic curvature assumption above the instrumented region, the top column displacement can be calculated. This top column displacement calculated from the Optotrak system is compared to the top column displacement measured with a string potentiometer at the center of loading in Figure 3.323. The calculated displacements match well over the entire range of response indicating that shear displacements are negligible in comparison to flexural displacements.
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Figure 3.3.16 T16 – Cross Section Curvature from Slope of Regression Line

Figure 3.3.17 T16 – Curvature Profiles during Push Cycles with Plastic Regression
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Figure 3.318 T16 – Curvature Profiles during Pull Cycles with Plastic Regression

Figure 3.319 T16 – Extreme Fiber Bar N3 Slip Hysteresis due to Strain Penetration
Figure 3.320  T16 – Extreme Fiber Bar S3 Slip Hysteresis due to Strain Penetration

Figure 3.321  T16 – Fixed End Rotation due to Strain Penetration during Push Cycles
Figure 3.322  T16 – Fixed End Rotation due to Strain Penetration during Pull Cycles

Figure 3.323  T16 – Comparison of Measured and Optotrak Integrated Displacements
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Figure 3.324 T16 – Optotrak Integrated Deformation Components

Figure 3.325 T16 – Measured Spread of Plasticity (Circular Data Points)
3.2.5 Test 17 – Chile 1985 Earthquake LH with #3 Spiral at 1.5” (1.3%)

Table 3.25 Observational Summary for Test 17 – Chile 1985 Earthquake LH with #3 Spiral at 1.5” (1.3%)

VALUES OF INTEREST:

Concrete Compressive Strength: \( f'_c = 7590 \text{ psi} \)
Axial Load: \( P = 170 \text{ kips} \)
Analytical First Yield Force: \( F'_y = 47.5 \text{ kips} \)
Experimental First Yield Displacement: \( \Delta'_y = 0.62" \) *From Test 16
Analytical Nominal Moment Capacity: \( M_n = 509.2 \text{ kip } \cdot \text{ ft} \)
Equivalent Yield Displacement: \( \Delta_y = 0.83" \)
Maximum Lateral Force: \( 72.0 \text{ kips} \)
Maximum Lateral Displacement: \( \mu^{32.76\text{ sec}}_{9.0} = 7.49" \)
Failure Mode: No Significant Damage from Earthquake LH.

DAMAGE OBSERVATIONS:

First Cracking North: During cycle to \( \mu^{10.50\text{ sec}}_{1.0} = 0.84" \)
First Cracking South: During cycle to \( \mu^{10.29\text{ sec}}_{0.6} = -0.49" \)
Cover Concrete Crushing North: \( \mu^{16.27\text{ sec}}_{-1.9} = -1.60" \)
Cover Concrete Crushing South: \( \mu^{15.32\text{ sec}}_{2.0} = 1.67" \)
Transverse Steel Yield North: At \(-4.02" \) otwt \( \mu^{18.52\text{ sec}}_{-5.4} = -4.49" \)
Transverse Steel Yield South: At \(4.5" \) otwt \( \mu^{32.76\text{ sec}}_{3.0} = 7.49" \)

*\( \mu^{32.76\text{ sec}}_{9.0} = 7.49" \) represents a push cycle 32.76 seconds into the earthquake load history which reached a peak displacement of 7.49” and a displacement ductility of 9.0
### Table 3.26 Observational Summary for Test 17b – Symmetric Three Cycle Set Aftershock LH

<table>
<thead>
<tr>
<th>DAMAGE OBSERVATIONS:</th>
<th>During Symmetric Three Cycle Set Post Earthquake LH</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal Bar Buckling North:</td>
<td>Reversal from $\mu_6^{+2} = 4.99''$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South:</td>
<td>Reversal from $\mu_6^{-2} = -5.00''$</td>
</tr>
<tr>
<td>Failure Mode:</td>
<td>Specimen Saved as a Repair Candidate after Each Extreme Fiber Longitudinal Bar Buckled</td>
</tr>
</tbody>
</table>

* $\mu_6^{-2} = -5.00''$ represents the second pull cycle of displacement ductility six

### Table 3.27 Strain Data Summary for Test 17 and Test 17b – Chile 1985 EQ LH and Cyclic Aftershock LH

<table>
<thead>
<tr>
<th>MATERIAL STRAINS:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North: $\varepsilon_s = 0.0043$ (compression)</td>
</tr>
<tr>
<td>Cover Concrete Crushing South: $\varepsilon_s = 0.0043$ (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield North: $\varepsilon_s = 0.0148$ (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield South: $\varepsilon_s = 0.0168$ (compression)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North: $\varepsilon_s = 0.055$ (peak tension prior to bb) $\varepsilon_s = 0.039$ (peak comp. prior to bb)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South: $\varepsilon_s = 0.039$ (peak tension prior to bb) $\varepsilon_s = 0.043$ (peak comp. prior to bb)</td>
</tr>
<tr>
<td>Mander (1988) Ultimate Concrete Compression Strain, $\varepsilon_{cu} = 0.0179$</td>
</tr>
</tbody>
</table>
Figure 3.326  T17 – Cross Section Bar Designation and Target Marker Application

Figure 3.327  T17 – Llolleo Chile 1985 Earthquake Load History
Figure 3.328 T17 – Lateral Force vs. Top Column Displacement Hysteretic Response

Figure 3.329 T17b – Symmetric Three Cycle Set Aftershock Load History
Figure 3.330  T17b – Lateral Force vs. Top Column Displacement Hysteretic Response

Figure 3.331  T17 – Compressive Axial Load from One Jack (Total = 2*Value)
Test 17 – Llolleo Chile 1985 Earthquake Load History (#3 @ 1.5”)

A scaled version of the Llolleo 1985 Chile earthquake load history, with a peak displacement ductility of nine, was chosen for Test 17. The top column displacement history, in Figure 3.327, was obtained using numerical analysis in OpenSees with a force-based fiber element to model the column and a zero-length strain penetration element to model the effects of strain penetration. The acceleration input of the Llolleo 1985 Chile earthquake was multiplied by 2.16 to produce a peak displacement ductility of nine. The resulting experimental lateral force vs. top column displacement response for the Llolleo 1985 Chile earthquake load history appears in Figure 3.328. The first yield displacement for Test 16, which contained the same spiral detailing as Test 17, was obtained as an average of the experimental first yield push and pull cycles ($\Delta y' = 0.62"$). To determine if this first yield displacement is applicable to Test 17, the tensile strain profile at ($\Delta y' = 0.62"$) for each extreme fiber bar appears in Figure 3.332. At the first yield displacement, the tensile strains in both extreme fiber reinforcing bars reached yield. The equivalent yield displacement, used to determine the displacement ductility levels ($\mu \Delta n = n \cdot \Delta y$), was then calculated as $\Delta y = \Delta y'(M_n/M_y') = 0.84"$ for Test 16. The displacement ductility levels for Test 16, see Figure 3.332, are also applicable for Test 17.

Previous Tests 8-12 focused on the effects of load history on reinforcement buckling. For the detailing of Tests 8-12 (#3 @ 2”, 1% volumetric ratio), it was found that reinforcement bar buckling occurred during displacement ductility eight of a three cycle set laboratory load history. Subsequent earthquake load history based tests scaled to displacement ductility (Test 8 - Chile 2010, 8.7) and (Test 10 - Chi-Chi 1999, 8.9) did not produce buckling of longitudinal steel. Instead, earthquake load histories scaled to ductility ten (Test 11 - Kobe 1995, 10) and (Test 12 – Japan 2011, 9.9) buckled reinforcing bars. The balanced repeated cycles of increasing ductility of the symmetric three-cycle-set load history appear to be more damaging than the load histories produced by historical earthquake records. To buckle reinforcing bars, the earthquake load histories were required to reach larger peak displacement ductility.
For the previous Test 16 (also #3 @ 1.5”, 1.3% volumetric ratio), a column with the same transverse steel detailing produced bar buckling during ductility eight of a symmetric three cycle set load history. The Llolleo 1985 Chile load history for Test 17 was scaled to displacement ductility nine to further evaluate the effect of load history on accumulated strains in the longitudinal and transverse steel. Based on previous test observations, an earthquake load history scaled to ductility nine is not expected to produce bar buckling. The Llolleo 1985 Chile top column displacement history contains a large number of inelastic reversals of generally high amplitude both before and after the peak displacement. The push direction of loading is dominated by a single large push cycle to ductility nine with many smaller reversals which range between ductility four and six. In the opposing direction of loading, there are a large number of reversals within the range of ductility four to six which appear both before and after the peak displacement.

After conclusion of the Llolleo 1985 Chile load history, the specimen had crushed cover concrete and degraded stiffness, but the longitudinal steel had not visibly buckled. The state of the specimen resembled Tests 8 and 10 where the reinforcement did not visibly buckle during the earthquake load history. Specimens 8, 10, and now 17 were subjected to a symmetric three cycle set aftershock to study the effect of degraded stiffness and strain accumulation on post-earthquake performance during a controlled load history. The displacement history and hysteretic response for the symmetric three-cycle-set aftershock study for Test 17 appear in Figure 3.329 and Figure 3.330 respectively. Visible bar buckling was observed on both sides of the specimen during ductility six of the cyclic aftershock study of Test 17b.
Figure 3.332  T17 – (Left) Tensile Strain Profiles at the First Yield Displacement of Test 16 and (Right) Displacement Ductility Levels from Test 16 (Also Apply for Test 17)

Test 17 – Lolleo Chile 1985 Earthquake (#3 @ 1.5”) Experimental Observations

The Lolleo 1985 Chile earthquake load history begins with a series of cycles below the first yield displacement, followed by cycles below ductility two as shown in Figure 3.327. Since the initial elastic cycles are not expected to affect the relationship between strain and displacement during later inelastic cycles, they were excluded from the experimental load history to save time. Crack widths on the North side of the specimen reached 0.45mm at approximate 6” spacing during $(\mu_{1.0}^{10.50 \text{ sec}} = 0.84”)$, as shown in the left photo of Figure 3.333. The format for the cycle naming system is as follows: $(\mu_{1.0}^{10.50 \text{ sec}} = 0.84”)$ represents the peak of the push cycle 10.50 seconds into the Lolleo earthquake load history which reached a displacement of 0.84” and a displacement ductility of 1.0. During $(\mu_{1.5}^{11.91 \text{ sec}} = 1.22”)$, the largest crack width on the North side of the specimen increased to 1mm. Crack widths on the South side of the specimen reached 0.5mm in width and approximate 6” spacing at $(\mu_{-1.2}^{12.25 \text{ sec}} = -0.96”)$, see the right two photos of Figure 3.333.
Visible flaking of cover concrete, which precedes crushing, was observed on the South side of the specimen during \((\mu_{1.6}^{12.50\ sec} = 1.32\")\), as shown in the left photo of Figure 3.334. Cover concrete crushing over the lowest 5” of the South side of the column occurred during \((\mu_{2.0}^{15.32\ sec} = 1.67\")\), see the right photo of Figure 3.334. Also during this cycle, crack widths on the North side of the specimen reached 1.5mm at approximate 6” spacing. Cover concrete crushing on the North side of the specimen over 3.5” occurred during \((\mu_{1.9}^{16.27\ sec} = -1.60\")\), see the left photo of Figure 3.335. The extent of crushing on the North side increased to 18.5” above the footing during \((\mu_{3.5}^{17.66\ sec} = -2.87\")\), as shown in the right photo of Figure 3.335. The extent of crushing on the South side of the specimen reached 24” above the footing during \((\mu_{5.4}^{19.52\ sec} = -4.49\")\), see the left photo of Figure 3.336. The extent of crushing on the South side of the specimen reached 21.5” above the footing during \((\mu_{3.5}^{21.36\ sec} = 2.89\")\). The crack distribution on the South and back sides of the specimen appear in the middle and right photos of Figure 3.336.

During \((\mu_{4.7}^{30.52\ sec} = 3.95\")\) and \((\mu_{6.0}^{31.34\ sec} = -4.96\")\) crushing on the South and North sides of the specimen did not increase in height, but rather widened to previously uncrushed areas around the column base as shown in Figure 3.337. At the peak cycle of the load history \((\mu_{0.0}^{32.76\ sec} = 7.49\")\), the extent of crushing on the South side of the specimen reached 25” above the footing. Photos of each side of the specimen during the peak cycle of the Llolleo earthquake load history appear in Figure 3.338. The remainder of the earthquake load history contained a large number of cycles below ductility six. Visible bar buckling was not observed during the remainder of the load history.
Figure 3.333 T17 – (Left) North Crack Distribution during ($\mu_{1.0}^{10.50\text{sec}} = 0.84''$), (Mid-Left) Back Side during ($\mu_{1.5}^{11.91\text{sec}} = 1.22''$), (Mid-Right) South Side during ($\mu_{-1.2}^{12.25\text{sec}} = -0.96''$), and (Right) Back Side during ($\mu_{-1.2}^{12.25\text{sec}} = -0.96''$)

Figure 3.334 T17 – (Left) Cover Concrete Flaking Preceding Crushing on the South Side during ($\mu_{1.0}^{12.50\text{sec}} = 1.32''$), (Right) Cover Concrete Crushing on the South Side at ($\mu_{2.0}^{15.32\text{sec}} = 1.67''$)
Figure 3.335  T17 – (Left) North Cover Concrete Crushing during \( \mu_{1.9}^{16.27 \text{ sec}} = -1.60" \) and (Right) Extent of Crushing on the North Side during \( \mu_{3.5}^{17.66 \text{ sec}} = -2.87" \)

Figure 3.336  T17 – (Left) Extent of Crushing on the North Side during \( \mu_{-5.4}^{18.52 \text{ sec}} = -4.49" \), (Middle) Extent of Crushing on the South Side during \( \mu_{3.5}^{21.36 \text{ sec}} = 2.89" \), and (Right) Crack Distribution on the Back Side during \( \mu_{3.5}^{21.36 \text{ sec}} = 2.89" \)
Figure 3.337 T17 – (Left) Extent of Crushing on the South during \( \mu_{3.7}^{30.52 \, sec} = 3.95'' \), (Middle) Crushing on the North Side during \( \mu_{-6.0}^{31.34 \, sec} = -4.96'' \), and (Right) Crack Distribution on the Back Side during \( \mu_{-6.0}^{31.34 \, sec} = -4.96'' \)

Figure 3.338 T17 – (Left, Middle, and Right) South, Back, and North Side of the Specimen during \( \mu_{3.0}^{32.76 \, sec} = 7.49'' \) Respectively
Test 17b – Cyclic Aftershock Load History (#3 @ 1.5”) Experimental Observations

Since bar buckling did not occur during the earthquake record, a symmetric three cycle set load history was conducted to determine the effect of degraded stiffness and strain accumulation on column behavior. The displacement ductility levels for the cyclic aftershock matched those from the symmetric three cycle set load history of Test 16. No notable damage was observed during cycles from displacement ductility one to four. The extreme fiber reinforcement remained visibly straight without noticeable outward deformation. Visible buckling of the North extreme fiber bar N3 occurred during \((\mu_6^{−2} = −5.00”)\), as shown in Figure 3.339. Visible outward deformation was observed 3.5” above the footing on bar N3 as well as slight rotation of LEDs above and below where the bar begins to straighten back out.

During the subsequent push cycle to \((\mu_6^{+3} = 5.00”)\), the South extreme fiber bar S3 visibly buckled as shown in Figure 3.340. Outward deformation was observed 8” above the footing over the highest transverse steel layer instrumented with a strain gage. During the next pull cycle to \((\mu_6^{−3} = −5.00”)\), the deformation in buckled bar N3 increased as shown in the left photo of Figure 3.341. Permanent deformation in spiral layers overlaying bar N3 was observed during \((\mu_6^{+4} = 5.00”)\), see the middle photo of Figure 3.341. During this cycle, the outward deformation in buckled bar S3 increased as shown in the right photo of Figure 3.341. A fourth cycle at ductility six was conducted to verify that the outward deformation in bar S3 would increase over the same location giving a stronger indication of observable bar buckling during the previous cycle. After this cycle, the test was concluded with buckling of each extreme fiber bar, but without any strength loss or rupture of reinforcement. The specimen was saved as a repair candidate.
Figure 3.339  T17b – (Left and Right) Buckling of North Reinforcing Bar N3 during $(\mu_6^2 = -5.00^\circ)$ of the Cyclic Aftershock Load History

Figure 3.340  T17b – (Left and Right) Buckling of South Reinforcing Bar S3 during $(\mu_6^{+3} = 5.00^\circ)$ of the Cyclic Aftershock
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Figure 3.341  T17 – (Left) Additional Deformation in Buckled Bar N3 during \( \mu_9^3 = -5.00" \), (Middle) Spiral Deformation over North Buckled Region at \( \mu_9^4 = 5.00" \), and (Right) Additional Deformation in Buckled Bar S3 during \( \mu_9^4 = 5.00" \)

Test 17 – Llolleo Chile 1985 Earthquake (#3 @ 1.5") Strain Data Analysis

North Reinforcement

Since the peaks of cycles during the earthquake load history do not align with the ductility levels of a traditional three cycle set load history, intermediate cycles along the backbone curve were selected for strain data analysis, see Figure 3.342. Extreme fiber vertical strain profiles for push and pull cycles appear in Figure 3.343 and Figure 3.344 respectively. A peak tensile strain of 0.055, located 3.56” above footing, was measured for Bar N3 during \( \mu_{9.0}^{32.76 \text{ sec}} = 7.49" \). The relationship between tensile strain and displacement for this gage length appears in Figure 3.347. Each line represents a single push cycle which began with the column at zero displacement and ended at the peak during a continuous push cycle. The solid line contains data during the push cycle loading up to the peak displacement and the dashed line represents the subsequent reversal of load. The peak tensile strain of
0.055 was not sufficient to produce visible bar buckling after reversal of load. Similar to previous tests, the moment curvature prediction for the relationship between strain and displacement using the PCK (2007) Lp Hinge Method begins to over predict the tensile strains at higher displacements at an increasing rate. The largest compressive strain of -0.023 was measured over the bar N3 gage length 2.05” above the footing during ($\mu_{-6.0}^{41.20 \text{ sec}} = -5.02”$). The relationship between compressive strain and displacement for this gage length appears in Figure 3.348. The recorded strains match the trend predicted by moment curvature analysis and the PCK (2007) Lp Hinge Method through ($\mu_{-3.5}^{17.66 \text{ sec}} = -2.87”$), but during later cycles the measured strains are larger than expected.

The transverse steel strains measured over the lowest six spiral layers overlaying the North reinforcement were plotted in Figure 3.346. Even though the peak compressive strains were measured 2.05” and 6.45” above the footing, the layer of transverse steel located 3.6” above the footing went furthest into the inelastic range during the Llolleo load history. The outward buckled region of bar N3 latter formed at this location during the ductility six of the cyclic aftershock, as shown in Figure 3.339. The peak tensile strains for bar N3 were located 3.56” above the footing. As previously mentioned, the residual growth strains measured for this gage were large, as shown in Figure 3.347. One possible explanation for the observations noted above is that measurable outward deformation occurred over the gage length 3.56” above the footing on bar N3 prior to visible bar buckling. It is not immediately obvious that this occurred because a large amount of growth strain could, perhaps, outweigh future compressive strains during subsequent cycles. Some amount of measurable outward deformation would increase the residual growth strain, increase the demand on the layer of transverse steel overlaying the bar, and agree with the location of visible bar buckling observations during the cyclic aftershock study.
South Reinforcement

A peak tensile strain of 0.0387 on bar S3 was measured 2.26” above the footing during $(\mu_{2.0}^{1.0} \text{sec} = -5.02”)$. The relationship between tensile strain and displacement for this gage length is shown in Figure 3.349. The same comments on the accuracy of the moment curvature prediction for bar N3 also apply to bar S3. A peak compression strain of -0.0392 on bar S3 was measured 9.53” above the footing during $(\mu_{9.0}^{3.76} \text{sec} = 7.49”)$. The measured peak compression strain is 2.2 times the calculated Mander (1988) ultimate concrete compressive strain of -0.0179. The relationship between compressive strain and displacement for the gage length 5.12” above the footing on bar S3 during push cycles appears in Figure 3.350. The measured compressive strains begin to deviate away from the prediction after a displacement ductility of 3.5. The gage length centered 9.53” above the footing with the largest compressive strain during $(\mu_{9.0}^{3.76} \text{sec} = 7.49”)$ appears in Figure 3.351. The relationship between compressive strain and displacement matches well until 5” of displacement during the push cycle to $(\mu_{9.0}^{3.76} \text{sec} = 7.49”)$, when the measured compression strains begin to sharply increase. Closer inspection of the transverse steel strains for spiral layers restraining the South bar during push cycles, in Figure 3.345, provides an explanation for measured increase in compressive strains. The transverse steel layer 8” above the footing first goes inelastic during ductility six, at approximately 5”, during the push cycle to $(\mu_{9.0}^{3.76} \text{sec} = 7.49”)$. It appears that the transverse steel layer entering the inelastic range influenced the relationship between compressive strain and displacement for the gage length 9.53” above the footing, localizing further compressive demand at this location.
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Figure 3.342 T17 – Strain Data Observation Points on the Hysteretic Response

Figure 3.343 T17 – Extreme Fiber Vertical Strain Profiles during Push Cycles
Figure 3.344 T17 – Extreme Fiber Vertical Strain Profiles during Pull Cycles

Figure 3.345 T17 – Transverse Steel Strains on the South Side during Push Cycles
Figure 3.346  T17 – Transverse Steel Strains on the North Side during Pull Cycles

Figure 3.347  T17 – Tensile Strain-Disp. during Push Cycles (Bar N3, 3.56” Above)
Figure 3.348 T17 – Compressive Strain-Disp. for Pull Cycles (Bar N3, 2.05” Above)

Figure 3.349 T17 – Tensile Strain-Disp. during Pull Cycles (Bar S3, 2.26” Above)
Figure 3.350  T17 – Compressive Strain-Disp. for Push Cycles (Bar S3, 5.12” Above)

Figure 3.351  T17 – Compressive Strain-Disp. for Push Cycles (Bar S3, 9.53” Above)
Test 17 – Lolleo Chile 1985 (#3 @ 1.5”) Curvature and Strain Penetration Data

Vertical curvature profiles are plotted for push and pull cycles in Figure 3.352 and Figure 3.353 respectively. These figures show that plastic curvatures have a linear distribution at higher displacement ductility levels. The dashed lines for each curvature distribution represent a least squared error linear fit to the plastic portion of the measured curvatures. The data points used to create the least squared error lines appear as circle data markers. The target marker on each bar placed closest the footing-column interface can be used to create slip hysteresis and horizontal slip profiles attributable to strain penetration. The slip hysteresis for extreme fiber bars N3 and S3 appear in Figure 3.354 and Figure 3.355 respectively. The peak tensile slip of North extreme fiber bar N3 exceeds 0.45” during displacement ductility nine.

If the tensile and compressive slip of all of the instrumented bars is plotted along the cross section depth, the base rotation attributable to strain penetration may be calculated. The slip profiles for push and pull cycles appear in Figure 3.356 and Figure 3.357 respectively. The rotation of the base section can be extracted from the slope of the least squared error line connecting all six measured bar slips. The total displacement could be calculated as the addition of the column flexure, strain penetration, and shear displacement components. The measured string potentiometer displacements from Test 17 were compared to the displacements obtained from curvature diagram integration and slip profile extrapolation to the center of loading in Figure 3.358. The measured and integrated top column displacements match well throughout the entire range of displacements indicating that shear displacements, which were not directly accounted for, must be small and thus negligible. The measured spread of plasticity as a function of base section curvature ductility appears in Figure 3.359. The circular data points represent the measured extent of plasticity, determined as the intersection of the linear plastic regression and the elastic curvature profile.
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Figure 3.352  T17 – Curvature Profiles during Push Cycles with Plastic Regression

Figure 3.353  T17 – Curvature Profiles during Pull Cycles with Plastic Regression
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Figure 3.354 T17 and T17b – Bar N3 Slip Hysteresis at the Footling-Column Interface

Figure 3.355 T17 and T17b – Bar S3 Slip Hysteresis at the Footling-Column Interface
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Figure 3.356 T17 – Base Rotation due to Strain Penetration during Push Cycles

Figure 3.357 T17 – Base Rotation due to Strain Penetration during Pull Cycles
Figure 3.358  T17 – Comparison of Measured and Optotrak Integrated Displacements

Figure 3.359  T17 – Measured Spread of Plasticity (Circular Data Points)
Test 17b – Cyclic Aftershock Load History (#3 @ 1.5”) Strain Data Analysis

North Reinforcement

Extreme fiber vertical strain profiles for the cyclic aftershock load history appear in Figure 3.360 and Figure 3.361 for push and pull cycles respectively. The shape of the tensile strain profiles during the cyclic aftershock resemble each other since the specimen rotates about crack profiles induced during higher ductility cycles of the Llolleo load history. The compressive vertical strain profiles are highly influenced by the measureable outward deformation 3.56” above the footing. The height of potential outward deformation coincides with the location where the bar later visually buckled. The gage lengths above and below 3.56” show greater compressive strains at the location where the bar begins to straighten back out. The recorded strains over these gage lengths are not accurate representations of engineering strains due to the deformation. The graphs are plotted in order to show the location and severity of the deformation.

The complete strain hysteresis for the outward buckled region of bar N3 appears in Figure 3.362 for the gage length 3.56” above the footing. A peak tensile strain of 0.055, located 3.56” above footing, was measured for North extreme fiber bar N3 during $(\mu^2 = 7.49")$. The peak tensile strain is marked by a small blue circle along with a blue arrow after reversal which represents the beginning of the measurable outward deformation. The transverse steel strain gage hysteresis for the spiral layer over the outward buckled region of bar N3 appears in Figure 3.363. After reversal from the peak displacement, while the cracks on the north side still remained open, the transverse steel strain starts to increase indicating additional demand caused by restraint of bar N3. The peak displacement prior to reinforcement buckling during the cyclic after shock study of $(\mu_6 = -5.00")$ is marked by a small red circle on the longitudinal and transverse steel strain hysteresis. Following the red arrow in both hystereses, the measured strains in the longitudinal steel deviate further deviate from prior trends, at the same time the measured transverse steel restraint strain increased rapidly.
**South Reinforcement**

The compressive strain profiles for bar S3 indicate measurable outward deformation 8.06” above the footing. At this height the outward deformation increased the measured tensile strain during cycles where the South reinforcing bar should be placed into compression. Gage lengths above and below the outward deformations reached higher compressive strains where the bar straightens back out. The longitudinal steel strain hysteresis for bar S3, over the outward buckled region 8.06” above the footing, appears in Figure 3.364. A strain hysteresis for the gage length above the outward buckled region 9.5” above the footing is shown in Figure 3.365. The transverse steel strain hysteresis, 8” above the footing, for the spiral layer over the outward buckled region of bar S3 appears in Figure 3.366. A blue data point marker on all three hysteresis marks the point at which the measured compression strain 9.5” above the footing started to rapidly increase during the push to ($\mu_{9.0^{sec}}^{32.76} = 7.49”$). As the gage length at 9.5” increased in compressive strain, the measured strains for the gage length below at 8.06” decreased. Coinciding with these two observations the transverse steel layer 8” above the footing entered the inelastic range. The strain hysteresis for the gage length 9.5” above the footing, in Figure 3.365, operates about a permanent downward shift decreasing the strain at a given displacement for the remainder of the test. Deviation after the blue data point for the gage length 8.06” above the footing, in Figure 3.364, indicates some measurable outward deformation. For the portion of the load history between ($\mu_{9.0^{sec}}^{32.76} = 7.49”$) of the Llolleo earthquake and ($\mu_{6}^{+1} = 4.99”$) of the cyclic aftershock, the strain in the transverse steel layer 8” above the footing in Figure 3.366 did not sharply increase indicating that the measurable deformation remained small prior to visible bar buckling. Over multiple cycles at ductility six the transverse steel strain gradually increased during each cycle, before rapidly increasing during ($\mu_{6}^{+3} = 5.00”$) when the bar visibly buckled.
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Figure 3.360  T17b – Extreme Fiber Vertical Strain Profiles during Push Cycles

Figure 3.361  T17b – Extreme Fiber Vertical Strain Profiles during Pull Cycles
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Figure 3.362  T17 and T17b – Bar N3 Strain Hysteresis Located 3.56” Above Footing

Figure 3.363  T17 and T17b – Spiral Strain Hysteresis over North Buckled Region
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Figure 3.364 T17 and T17b – Bar S3 Strain Hysteresis Located 8.06" Above Footing

Figure 3.365 T17 and T17b – Bar S3 Strain Hysteresis Located 9.53" Above Footing
Figure 3.366  T17 and T17b – Spiral Strain Hysteresis over South Buckled Region
3.2.6 Test 18 – Darfield NZ 2010 Earthquake LH with #3 Spiral at 1.5” (1.3%)

Table 3.28 Observational Summary for Test 17 – Darfield NZ Earthquake LH with #3 Spiral at 1.5” (1.3%)

VALUES OF INTEREST:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Compressive Strength: $f'_c$</td>
<td>$7807 \text{ psi}$</td>
</tr>
<tr>
<td>Axial Load: $P$</td>
<td>$170 \text{ kips}$</td>
</tr>
<tr>
<td>Analytical First Yield Force: $F_y'$</td>
<td>$47.6 \text{ kips}$</td>
</tr>
<tr>
<td>Experimental First Yield Displacement: $\Delta_y'$</td>
<td>$0.62&quot;$</td>
</tr>
<tr>
<td>Analytical Nominal Moment Capacity: $M_n$</td>
<td>$510.4 \text{ kip} \cdot \text{ft}$</td>
</tr>
<tr>
<td>Equivalent Yield Displacement: $\Delta_y$</td>
<td>$0.83&quot;$</td>
</tr>
<tr>
<td>Maximum Lateral Force:</td>
<td>$72.7 \text{ kips}$</td>
</tr>
<tr>
<td>Maximum Lateral Displacement: $\mu_{24.40 \text{sec}}$</td>
<td>$7.46&quot;$</td>
</tr>
</tbody>
</table>

DAMAGE OBSERVATIONS:

<table>
<thead>
<tr>
<th>Damage</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Cracking North:</td>
<td>$\mu_{0.2}^{18.12 \text{sec}} = 0.17&quot;$</td>
</tr>
<tr>
<td>First Cracking South:</td>
<td>$\mu_{-0.3}^{18.12 \text{sec}} = -0.23&quot;$</td>
</tr>
<tr>
<td>Cover Concrete Crushing North:</td>
<td>During the pull to $\mu_{-2.7}^{23.72 \text{sec}} = -2.21&quot;$</td>
</tr>
<tr>
<td>Cover Concrete Crushing South:</td>
<td>During the push to $\mu_{9.0}^{24.40 \text{sec}} = 7.46&quot;$</td>
</tr>
<tr>
<td>Transverse Steel Yield North:</td>
<td>At $-5.49&quot;$ otw $\mu_{7.3}^{25.00 \text{sec}} = -6.05&quot;$</td>
</tr>
<tr>
<td>Transverse Steel Yield South:</td>
<td>At $3.70&quot;$ otw $\mu_{9.0}^{24.40 \text{sec}} = 7.46&quot;$</td>
</tr>
<tr>
<td>Bar Buckling South:</td>
<td>After Reversal from $\mu_{-7.3}^{25.00 \text{sec}} = -6.05&quot;$</td>
</tr>
</tbody>
</table>

* $\mu_{9.0}^{24.40 \text{sec}} = 7.46"$ represents a push cycle 24.40 seconds into the earthquake load history which reached a peak displacement of 7.46” and a displacement ductility of 9.0
Table 3.29 Observational Summary for Test 17b – Symmetric Three Cycle Set
Aftershock LH

DAMAGE OBSERVATIONS: During Symmetric Three Cycle Set Post Earthquake LH

<table>
<thead>
<tr>
<th>Observation</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal Bar Fracture South</td>
<td>Reversal from $\mu_6^{-2} = -4.99''$</td>
</tr>
<tr>
<td>Failure Mode</td>
<td>Fracture of Previously Buckled Reinforcement</td>
</tr>
</tbody>
</table>

*$\mu_6^{-2} = -4.99''$ represents the second pull cycle of displacement ductility six

Table 3.30 Strain Data Summary for Test 17 and Test 17b – Darfield NZ EQ LH and
Cyclic Aftershock

MATERIAL STRAINS:

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Strain Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North</td>
<td>$\epsilon_s = 0.068$</td>
<td>compression</td>
</tr>
<tr>
<td>Cover Concrete Crushing South</td>
<td>N/A</td>
<td>During the push to $\mu_5^{24,40 sec} = 7.46''$</td>
</tr>
<tr>
<td>Transverse Steel Yield North</td>
<td>$\epsilon_s = 0.0147$</td>
<td>compression</td>
</tr>
<tr>
<td>Transverse Steel Yield South</td>
<td>$\epsilon_s = 0.0136$</td>
<td>(compression)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North</td>
<td>North Bar Never Visibly Buckled</td>
<td></td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South</td>
<td>$\epsilon_s = 0.047$ (peak tension prior to bb)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\epsilon_s = 0.048$ (peak comp. prior to bb)</td>
<td></td>
</tr>
</tbody>
</table>

Mander (1988) Ultimate Concrete Compression Strain, $\epsilon_{cu} = 0.0176$
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Figure 3.367 T18 – Cross Section Bar Designation and Target Marker Application

Figure 3.368 T18 – Darfield NZ 2010 Earthquake Load History
Figure 3.369  T18 – Darfield NZ Lateral Force vs. Top Column Displacement Response

Figure 3.370  T18b – Symmetric Three Cycle Set Aftershock Load History
Figure 3.371 T18b – Cyclic Aftershock Lateral Force vs. Displacement Response

Figure 3.372 T18 – Compressive Axial Load from One Jack (Total = 2*Value)
Test 18 – Darfield NZ 2010 Earthquake Load History (#3 @ 1.5”)

A scaled version of the Darfield 2010 New Zealand earthquake load history, with a peak displacement ductility of nine, was chosen for Test 18. The top column displacement history, in Figure 3.368, was obtained using numerical analysis in OpenSees with a force-based fiber element to model the column and a zero-length strain penetration element. The acceleration input of the Darfield 2010 earthquake record was multiplied by 0.97 to produce a peak displacement ductility of nine. The analytical top column displacement history was recreated in the lab with a quasi-static loading procedure. The resulting experimental lateral force vs. top column displacement response for the Darfield 2010 load history appears in Figure 3.369. The first yield displacement for Test 16, which had same detailing as Tests 17 and 18, was obtained as an average for the experimental first yield push and pull cycles ($\Delta y' = 0.62"$). To determine if this first yield displacement is applicable to Test 18, the tensile strain profile at ($\Delta y' = 0.62"$) for each extreme fiber bar appears in Figure 3.373. At the first yield displacement, the tensile strains in both extreme fiber reinforcing bars reached yield. The equivalent yield displacement, used to determine the displacement ductility levels ($\mu_{\Delta n} = n \star \Delta y$), was then calculated as $\Delta y = \Delta y' (M_n/M'_y) = 0.83"$. The displacement ductility levels for Test 16, see Figure 3.373, are also applicable for Tests 17 and 18.

Three columns detailed with a #3 spiral at 1.5” spacing ($4A_{sp}/(D') = 1.3\%$) were chosen for Tests 16-18. Reinforcement buckling occurred during ductility eight of a symmetric three cycle set load history in Test 16. A scaled version of the 1985 Llolleo Chile earthquake record did not produce bar buckling even though the peak response reached displacement ductility nine. The Llolleo 1985 Chile top column displacement history contains a large number of inelastic reversals of high amplitude both before and after the peak displacement. The top column displacement history for Test 18, which utilized a scaled version of the Darfield 2010 New Zealand record, appears in Figure 3.368. In comparison, the Darfield load history contains only a few high ductility cycles. The peak cycle in the opposing direction of the maximum response reaches displacement ductility 7.3. Bar buckling is not expected to occur after reversal from the peak displacement ductility of 9.0,
however, it offers the opportunity to study the influence of inelastic transverse steel restraint on potential buckling of longitudinal steel placed into tension during the ductility 7.3 reversal.

After conclusion of the Darfield 2010 New Zealand load history, the specimen had crushed cover concrete, degraded stiffness, and a single buckled reinforcing bar on the South side of the specimen. The specimen was subjected to a symmetric three cycle set load history to evaluate the effect of additional cycles on the buckled region on the South side of the column, and to determine what level of displacement is required to induce buckling of the North reinforcement. During ductility six, two additional South reinforcing bars buckled. The tensile demand sustained during repeated cycles at displacement ductility six was sufficient to rupture the extreme fiber South reinforcing bar which buckled during the Darfield load history. The test was concluded with three buckled bars and a single ruptured bar on the South side and unbuckled reinforcement on the North. The specimen was saved as a repair candidate.

Figure 3.373  T18 – (Left) Tensile Strain Profiles at the First Yield Displacement of Test 16 and (Right) Displacement Ductility Levels from Test 16 (Also Apply for Test 17)
The beginning of the Darfield 2010 New Zealand load history contains a large number of elastic reversals. The first cracks on the North side of the specimen measured 0.1mm at approximate 9” spacing during \(\mu_{0,2}^{12.12 \text{sec}} = 0.17"\), as shown in the left photo of Figure 3.374. Crack widths reached 0.1mm at approximate 9” spacing on the South side of the specimen during \(\mu_{0,3}^{18.30 \text{sec}} = -0.23"\), see the middle photo of Figure 3.374. During the pull cycle to \(\mu_{0,6}^{19.54 \text{sec}} = -0.50"\), in the right photo of Figure 3.374, crack widths on the South side reached 0.3mm at approximate 5” spacing. Cracks on the North side reached 0.4mm at 5” spacing during the pull cycle to \(\mu_{0,6}^{19.74 \text{sec}} = 0.51"\), see the left photo of Figure 3.375. The first cycles exceeding yield for the Darfield load history occurred during \(\mu_{0,8}^{21.72 \text{sec}} = 0.66"\), when cracks on the North side of the specimen increased to 0.45mm at approximate 5” spacing. Crack widths reached 0.5mm at 5” spacing on the South side of the specimen during \(\mu_{0,8}^{22.02 \text{sec}} = -0.66"\), see the right two photos of Figure 3.375.

The first cycle exceeding the equivalent yield displacement in the push direction occurred during \(\mu_{1,3}^{22.50 \text{sec}} = -1.24"\), where crack widths reached 0.8mm at approximate 3-4” spacing. The crack distribution on the front side of the specimen at \(\mu_{1,3}^{22.78 \text{sec}} = 1.04"\) appears in the left photo of Figure 3.376. During the pull cycle to \(\mu_{2,7}^{23.72 \text{sec}} = -2.21"\), crushing on the North side of the specimen extended 16” above the footing. Crack widths on the South side of the specimen reached 2.5mm at 3-4” spacing, as shown in Figure 3.376. The displacement when crushing first occurred was not recorded. The following reversal of loading pushed the specimen to the peak displacement of \(\mu_{0,0}^{24.40 \text{sec}} = 7.46"\). The extent of crushing on the South side of the specimen reached 21” above the footing, see the middle photo of Figure 3.377. Additional photos of the specimen at the peak displacement appear in Figure 3.378. Crushing on the North side of the specimen reached 22” above the footing during the pull cycle to \(\mu_{7,3}^{25.00 \text{sec}} = -6.05"\), Figure 3.379.

North reinforcement exposed to tension during \(\mu_{0,0}^{24.40 \text{sec}} = 7.46"\) did not visibly buckle during the large reversal to \(\mu_{7,3}^{25.00 \text{sec}} = -6.05"\). Large compressive demand during
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\( \mu_{0.0}^{24.40 \text{ sec}} = 7.46" \) caused several layers of transverse steel on the South side of the specimen to enter the inelastic range. Inelastic transverse steel layers combined with large tensile strains during \( \mu_{-7.3}^{25.00 \text{ sec}} = -6.05" \) buckled the extreme fiber South reinforcing bar during the following reversal of load. Visible buckling of Bar S3 is shown in the right two photos of Figure 3.379 at \( \mu_{4.0}^{25.58 \text{ sec}} = 3.32" \). Outward bar buckling occurred over the second and third transverse steel spacings above the footing. The remainder of the Darfield load history contained lower ductility cycles which did not produce any notable damage beyond increasing the buckled deformation in Bar S3.

Figure 3.374 T18 – (Left) First Cracking on the North Side during \( \mu_{0.2}^{18.12 \text{ sec}} = 0.17" \), (Middle) First Cracking on the South Side during \( \mu_{-0.3}^{18.30 \text{ sec}} = -0.23" \), and (Right) South Crack Distribution during \( \mu_{-0.6}^{19.54 \text{ sec}} = -0.50" \)
Figure 3.375 T18 – (Left) Cracks on the North Side at \( \mu^{19.74 \text{ sec}}_{0.6} = 0.51" \), (Middle and Right) Crack Distribution on the South and Back Sides at \( \mu^{22.02 \text{ sec}}_{-0.8} = -0.66" \)

Figure 3.376 T18 – (Left) Crack Distribution on the Front Side during \( \mu^{22.78 \text{ sec}}_{1.3} = 1.04" \), (Middle) Cracking on the South Side during \( \mu^{23.72 \text{ sec}}_{-2.7} = -2.21" \), and (Right) Extent of Cover Concrete Crushing on the North Side at \( \mu^{23.72 \text{ sec}}_{-2.7} = -2.21" \)
Figure 3.377  T18 – (Left) Crack Distribution on the Front Side during the Peak Cycle to \( \mu_{24.40}^{sec} = 7.46" \), (Middle) Crushing on the South Side, and (Right) Crack on the Top of the Footing on the North Side of the Column

Figure 3.378  T18 – Crack Distribution at Peak Displacement \( \mu_{9.0}^{24.40 \sec} = 7.46" \)
Figure 3.379  T18 – (Left) Extent of Crushing on the North Side during \( \mu = -7.3 \), (Middle and Right) Visible Buckling of Bar S3 at \( \mu = 3.32'' \)

Test 18b – Cyclic Aftershock Load History (#3 @ 1.5””) Experimental Observations:

After conclusion of the Darfield 2010 New Zealand load history, the specimen had crushed cover concrete, degraded stiffness, and a single buckled reinforcing bar on the South side of the specimen. Previous earthquake load histories scaled to approximately ductility nine failed to produce visible buckling. The specimen was subjected to a symmetric three cycle set load history, see Figure 3.370, to evaluate the effect of additional cycles on the buckled region on the South side of the column, and to determine what level of displacement is required to induce buckling of the North reinforcement. No notable damage occurred through ductility four of the cyclic aftershock study.

During the first push cycle of displacement ductility six, \( \mu^1 = 4.99'' \), additional South reinforcing bars S2 and S4 buckled as shown in the left photo of Figure 3.380. The
outward deformation of previously buckled bar S3 was more severe than at any other point of the load history. On the way to \( \mu_6^{-2} = -4.99'' \), previously buckled South reinforcing bar S3 ruptured in tension. The ruptured bar and deformations in several spiral layers is shown in Figure 3.380. The test was concluded with three buckled bars and a single ruptured bar on the South side and unbuckled reinforcement on the North. Cross section equilibrium was distorted beyond the use of additional buckling data for North reinforcement if test were to continue. The specimen was saved as a repair candidate.

![Figure 3.380](image-url)

Figure 3.380 T18 – (Left) Buckling of Bars S2 and S4 during \( \mu_6^{+1} = 4.99'' \) and (Right) Rupture of Previously Buckled Bar S3 during \( \mu_6^{-2} = -4.99'' \)
Test 18 – Darfield NZ 2010 Earthquake Load History (#3 @ 1.5”) Strain Data Analysis

North Reinforcement

Since the peaks of cycles during the earthquake load history do not align with the ductility levels of a traditional three cycle set load history, intermediate cycles along the backbone curve were selected for strain data analysis, see Figure 3.382. Extreme fiber vertical strain profiles for push and pull cycles appear in Figure 3.383 and Figure 3.384 respectively. A peak tensile strain of 0.062, located 3.19” above footing, was measured for extreme fiber bar N3 during \( \mu_{9.0}^{24.40 \text{ sec}} = 7.46" \). The relationship between tensile strain and displacement for this gage length appears in Figure 3.387. The solid line contains data during the push cycle loading up to the peak displacement and the dashed line represents the subsequent reversal of load. The peak tensile strain of 0.062 was not sufficient to produce visible bar buckling after reversal of load. The peak tensile gage length overlaid the largest crack on the north side of the specimen in Figure 3.381. Similar to previous tests, the moment curvature prediction for the relationship between strain and displacement using the PCK (2007) Lp Hinge Method begins to over predict the tensile strains at higher displacements at an increasing rate.

The largest compressive strain of -0.021 was measured in bar N3 for the gage length located 1.63” above the footing during \( \mu_{-7.3}^{25.00 \text{ sec}} = -6.05" \). The relationship between compressive strain and displacement for the gage length 1.63” above the footing appears in Figure 3.388. The transverse steel strains measured for the lowest six spiral layers overlaying the North reinforcement are plotted in Figure 3.386. The figure depicts tensile strains in the spiral layers on the North side of the specimen placed into compression during pull cycles. During the peak pull cycle to \( \mu_{-7.3}^{25.00 \text{ sec}} = -6.05" \), two spiral layers entered the inelastic range. The strain data for the North reinforcement does not give any indication of measurable deformation during the Darfield load history.
South Reinforcement

A peak tensile strain of 0.0466 on bar S3 was measured 3.31” above the footing during \((\mu_{7.3}^{25.00\,sec} = -6.05")\). The relationship between tensile strain and displacement for this gage length is shown in Figure 3.389. The same comments on the accuracy of the moment curvature prediction for the North reinforcement also apply to bar S3. The blue dashed line, after reversal from \((\mu_{7.3}^{25.00\,sec} = -6.05")\), no longer represents engineering strain due to the observed outward buckling over the gage length depicted in Figure 3.379. A peak compression strain of -0.0481 was measured 1.78” above the footing during \((\mu_{9.0}^{24.40\,sec} = 7.46")\). The relationship between compressive strain and displacement for this gage length appears in Figure 3.390. At large displacements, the measured compressive strains are significantly larger than the moment curvature prediction. The measured compression strain of -0.0481 is 2.7 times larger than the Mander ultimate concrete compressive strain of -0.0176. The effect of the large compressive demand on the South side of the specimen can be seen in the transverse steel layers overlaying the extreme fiber bar in Figure 3.385. Two layers of transverse steel enter the inelastic range at displacement ductility six during the push cycle to \((\mu_{9.0}^{24.40\,sec} = 7.46")\). By displacement ductility eight, four transverse steel layers went into the inelastic range. The strain gage on the spiral layer 3.56” above the footing went off scale during \((\mu_{9.0}^{24.40\,sec} = 7.46")\).

The strain hysteresis for extreme fiber Bar S3 appears in Figure 3.391 for the gage length 3.31” above the footing which outwardly deformed as the bar buckled. The strain hysteresis for the gage length located 4.83” above the footing appears in Figure 3.392. This gage length coincides with the region where the bar begins to straighten back out. Both graphs are shown to illustrate the effect compressive localization over inelastic spiral layers. The transverse steel strain hysteresis for the spiral layers located 2.06” and 3.56” above the footing appear in Figure 3.393 and Figure 3.394 respectively. During \((\mu_{9.0}^{24.40\,sec} = 7.46")\), the strain in the transverse steel layer 3.56” above the footing increased beyond 0.016, where the strain gages goes off scale. As this occurred, the instrumentation indicated measurable deformation in bar S3, leading to higher compressive strains in the gage length 4.83” above
the footing and lower compressive strains 3.31” above the footing. This location agrees with the location of bar buckling.

Figure 3.381  T18 – Largest Cracks on North Side at \( \mu_{24.40 \ sec} = 7.46” \)

Figure 3.382  T18 – Strain Data Observation Points along the Backbone Curve
Figure 3.383  T18 – Extreme Fiber Vertical Strain Profiles during Push Cycles

Figure 3.384  T18 – Extreme Fiber Vertical Strain Profiles during Pull Cycles
Figure 3.385  T18 – Transverse Steel Strains on the South Side during Push Cycles

Figure 3.386  T18 – Transverse Steel Strains on the North Side during Pull Cycles
Figure 3.87  T18 – Tensile Strain-Displacement for Bar N3 (3.19” Above Footing)

Figure 3.88  T18 – Compressive Strain-Displacement for Bar N3 (1.63” Above)
Figure 3.389  T18 – Tensile Strain-Displacement for Bar S3 (3.31” Above Footing)

Figure 3.390  T18 – Compressive Strain-Displacement for Bar S3 (1.78” Above)
Figure 3.391  T18 – Bar S3 Strain Hysteresis for the Gage Length 3.31” Above Footing

Figure 3.392  T18 – Bar S3 Strain Hysteresis for the Gage Length 4.83” Above Footing
Figure 3.393  T18 – Spiral Strain Hysteresis over Buckled Bar S3 (Layer 2.06” Above)

Figure 3.394  T18 – Spiral Strain Hysteresis over Buckled Bar S3 (Layer 3.56” Above)
Test 18 – Darfield NZ 2010 EQ LH (#3 @ 1.5”) Curvature and Strain Penetration Data

Vertical curvature profiles are plotted for push and pull cycles in Figure 3.395 and Figure 3.396 respectively. These figures show that plastic curvatures have a linear distribution at higher displacement ductility levels. As the displacements increase, the base curvatures become larger and the extent of plastic curvatures reach higher above the footing. The target marker on each reinforcing bar placed closest the footing-column interface can be used to create slip hysteresis and horizontal slip profiles attributable to strain penetration. The slip hysteresis for North extreme fiber bar N3 appears in Figure 3.399. The peak tensile slip bar N3 exceeded 0.34” at ($\mu_{9,24.40} = 7.46”$). If the tensile and compressive slip of all of the instrumented bars is plotted along the cross section depth, the base rotation attributable to strain penetration may be calculated. The slip profiles for push and pull cycles appear in Figure 3.397 and Figure 3.398 respectively.

The measured string potentiometer displacements from Test 18 were compared to the displacement obtained from curvature diagram integration and slip profile extrapolation to the center of loading in Figure 3.400. The measured and integrated top column displacements match well with the exception of high ductility data points near ($\mu_{7.3} = -6.05”$). The measured spread of plasticity as a function of base section curvature ductility appears in Figure 3.401. The circular data points plot the measured extent of plasticity, obtained as the intersection of the linear plastic curvature regression and the elastic curvature profiles.
Figure 3.395  T18 – Curvature Profiles during Push Cycles with Plastic Regression

Figure 3.396  T18 – Curvature Profiles during Pull Cycles with Plastic Regression
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Figure 3.397 T18 – Base Rotation during Push Cycles due to Strain Penetration

Figure 3.398 T18 – Base Rotation during Pull Cycles due to Strain Penetration
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Figure 3.399 Test 18 – Bar N3 Slip Hysteresis at the Footing-Column Interface

Figure 3.400 T18 – Comparison of Measured and Optotrak Integrated Displacements
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Test 18b – Cyclic Aftershock Load History (#3 @ 1.5”) Strain Data Analysis:

Vertical strain profiles for extreme fiber Bar N3 during push and pull cycles of the cyclic aftershock load history appear in Figure 3.402 and Figure 3.403 respectively. The strain profiles for bar N3 follow a similar shape because the crack distribution was previously set in place during peak cycles of the Darfield load history. The transverse steel strains measured over the lowest six spiral layers overlaying the North reinforcement are plotted in Figure 3.404. During the first and second pull cycles of ductility six, transverse steel strains reached 0.0038 and 0.0039 respectively. The complete strain hysteresis for bar N3, for the gage length 3.19” above the footing, appears in Figure 3.405. A similar strain hysteresis for the gage length 4.63” above the footing is shown in Figure 3.406. The gage length 3.19” above the footing is directly crossed by the largest crack on the North side of the specimen as shown in Figure 3.381. This explains the larger residual strain after the peak cycle evident in the gage length 3.19” above the footing.
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Figure 3.402 T18b – Vertical Strain Profiles for Bar N3 during Push Cycles

Figure 3.403 T18b – Vertical Strain Profiles for Bar N3 during Pull Cycles
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Figure 3.404  T18b – Spiral Strains for Layers overlaying Bar N3 during Pull Cycles

Figure 3.405  T18 and T18b – Bar N3 Strain Hysteresis 3.19” Above the Footing
Figure 3.406  T18 and T18b – Bar N3 Strain Hysteresis 4.63” Above the Footing

Figure 3.407  Spiral Strain Hysteresis for the Layer over Bar N3 (3.44” Above)
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3.3 Aspect Ratio and Axial Load Variable Tests 19-24

The effects of aspect ratio and axial load ratio on column performance were the main variables for Tests 19-24. The test matrix for the eight columns is shown in Table 3.31, and the material properties of the reinforcement appear in Table 3.32. The 18” (457mm) diameter bridge columns, Figure 3.410, contained 10 #6 (19mm) A706 bars for longitudinal reinforcement \( \frac{A_{st}}{A_g} = 1.7\% \) and a #3 (9.5mm) A706 spiral at 2” spacing \( 4A_{sp}/D's = 1.3\% \). The shear span for the cantilever columns was either 8ft (244cm), 11ft (335cm), or 13ft (396cm), resulting in moment to shear ratios of \( (M/VD = 5.33, 7.33, \) or \( 8.67) \). For each aspect ratio, one specimen was subjected to \( (P/(f'_cA_g) = 5\%) \) and the other was subjected to 10% axial load. Photos of the test setup for the tallest aspect ratio columns appear in Figure 3.411. Stress-strain curves for the longitudinal and transverse steel are shown in Figure 3.408 and Figure 3.409. The test series used the full cover concrete blockout method with target markers applied to both longitudinal and transverse steel, Figure 3.410.

In design, an equivalent curvature distribution such as the Plastic Hinge Method from Priestley, Calvi, and Kowalsky (2007) is used to translate the curvature at specific material strain limits to column deformations. The moment gradient component of the plastic hinge length is dependent on the column length. Aspect ratio also influences shear in the column, which impacts the additional spread in plasticity due to tension shift. Aspect ratio is not expected to influence bar buckling behavior, but the tests are included to evaluate its effect on the spread of plasticity.

Axial load influences the distribution of forces within the cross section. Columns with higher levels of axial load are expected to have a reduced deformation capacity but higher lateral forces. Limit states governed by compression are influenced by the increased axial load. Tests 19-24 evaluate columns subjected to 5% and 10% axial load, while future specimens are subjected to 15% and 20% axial load.
Table 3.31  **Column Summary for Aspect Ratio and Axial Load Variable Tests 19-24**

<table>
<thead>
<tr>
<th>Test</th>
<th>Load History</th>
<th>D (in)</th>
<th>L/D</th>
<th>Long. Steel ($p_l$)</th>
<th>Spiral Detailing ($p_s$)</th>
<th>$f'c$ (psi)</th>
<th>$P/f'c*Ag$</th>
</tr>
</thead>
<tbody>
<tr>
<td>19</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2&quot; (1.3%)</td>
<td>6334</td>
<td>10%</td>
</tr>
<tr>
<td>20</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2&quot; (1.3%)</td>
<td>6467</td>
<td>5%</td>
</tr>
<tr>
<td>21</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>7.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2&quot; (1.3%)</td>
<td>6390</td>
<td>5%</td>
</tr>
<tr>
<td>22</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>7.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2&quot; (1.3%)</td>
<td>6530</td>
<td>10%</td>
</tr>
<tr>
<td>23</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>8.67</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2&quot; (1.3%)</td>
<td>6606</td>
<td>5%</td>
</tr>
<tr>
<td>24</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>8.67</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2&quot; (1.3%)</td>
<td>6473</td>
<td>10%</td>
</tr>
</tbody>
</table>

Table 3.32  **Reinforcement Material Property Summary for Columns 19-24**

<table>
<thead>
<tr>
<th><strong>Longitudinal Reinforcement</strong></th>
<th>$\varepsilon_y$</th>
<th>$f_y$ (ksi)</th>
<th>$\varepsilon_h$</th>
<th>$f_h$ (ksi)</th>
<th>$\varepsilon_u$</th>
<th>$f_u$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tests 19-24 (#6 Bar)</td>
<td>0.00250</td>
<td>68.1</td>
<td>0.0153</td>
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<th><strong>Transverse Steel</strong></th>
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<th>$f_y$ (ksi)</th>
<th>$\varepsilon_u$</th>
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<td>Tests 19-24 (#3 Spiral)</td>
<td>0.00465</td>
<td>65.6</td>
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Figure 3.408 Test 19-24 – Longitudinal Steel Tensile Test Results

Figure 3.409 Test 19-24 – Transverse Steel Tensile Test Results
Figure 3.410 Tests 19-24 Cross Section and Bar Designation for Smaller Section

Figure 3.411 Test Setup for 11’ and 13’ Cantilever Length Specimens
3.3.1 Test 19 – Aspect Ratio of 5.33 and 10% Axial Load

Table 3.33 Observations for Test 19 – Aspect Ratio of 5.33 and 10% Axial Load

<table>
<thead>
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<th>VALUES OF INTEREST:</th>
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<tr>
<td>Concrete Compressive Strength:</td>
<td>$f'_c = 6334 \text{ psi}$</td>
</tr>
<tr>
<td>Axial Load:</td>
<td>$P = 144 \text{ kips } (P/(f'_c A_g) = 10%)$</td>
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<tr>
<td>Column Length and Aspect Ratio:</td>
<td>8 ft ($L/D = 5.33$)</td>
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<tr>
<td>Analytical First Yield Force:</td>
<td>$F'_y = 21.90 \text{ kips}$</td>
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<tr>
<td>Experimental First Yield Displacement:</td>
<td>$\Delta'_y = 0.87''$</td>
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<tr>
<td>Analytical Nominal Moment Capacity:</td>
<td>$M_n = 230.93 \text{ kip } \ast \text{ ft}$</td>
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<tr>
<td>Equivalent Yield Displacement:</td>
<td>$\Delta_y = 1.15''$</td>
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<tr>
<td>Maximum Lateral Force:</td>
<td>29.81 kips</td>
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</table>

<table>
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<th>DAMAGE OBSERVATIONS:</th>
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</thead>
<tbody>
<tr>
<td>First Cracking North:</td>
<td>$3/4Fy' = 0.48''$</td>
</tr>
<tr>
<td>First Cracking South:</td>
<td>$-3/4Fy' = -0.58''$</td>
</tr>
<tr>
<td>Cover Concrete Crushing North:</td>
<td>$\mu_3^{-2} = -3.44''$</td>
</tr>
<tr>
<td>Cover Concrete Crushing South:</td>
<td>$\mu_3^{+2} = 3.43''$</td>
</tr>
<tr>
<td>Transverse Steel Yield North:</td>
<td>At $-3.17''$ during pull to $\mu_3^{-1} = -3.42''$</td>
</tr>
<tr>
<td>Transverse Steel Yield South:</td>
<td>At $0.23''$ during push to $\mu_3^{+3} = 3.43''$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North:</td>
<td>Reversal from $\mu_5^{+3} = 5.74''$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South:</td>
<td>Reversal from $\mu_5^{-2} = -5.71''$</td>
</tr>
<tr>
<td>Longitudinal Bar Fracture North:</td>
<td>At $4.10''$ during push to $\mu_6^{+4} = 6.89''$</td>
</tr>
<tr>
<td>Longitudinal Bar Fracture South:</td>
<td>At $-4.22''$ during pull to $\mu_6^{-3} = -6.88''$</td>
</tr>
</tbody>
</table>

* $\mu_5^{+3} = 5.74''$ represents the third push cycle of displacement ductility five
Table 3.4 Strain Data Summary for Test 19 – Aspect Ratio of 5.33 and 10% Axial

MATERIAL STRAINS:

Cover Concrete Crushing North: \( \varepsilon_s = 0.0060 \) (compression) *\( \mu_{\Delta 2} = -2.29" \)

Cover Concrete Crushing South: \( \varepsilon_s = 0.0065 \) (compression) *\( \mu_{\Delta 2} = 2.29" \)

Transverse Steel Yield North: \( \varepsilon_s = 0.0103 \) (compression)

Transverse Steel Yield South: \( \varepsilon_s = 0.0119 \) (compression)

Longitudinal Bar Buckling North: \( \varepsilon_s = 0.037 \) (peak tension prior to \( bb \))

\( \varepsilon_s = 0.024 \) (peak comp. prior to \( bb \))

Longitudinal Bar Buckling South: \( \varepsilon_s = 0.032 \) (peak tension prior to \( bb \))

\( \varepsilon_s = 0.022 \) (peak comp. prior to \( bb \))

Mander (1988) Ultimate Concrete Compression Strain, \( \varepsilon_{cu} = 0.0205 \)
Figure 3.413  T19 – Symmetric Three Cycle Set Load History

Figure 3.414  T19 – Lateral Force vs. Top Column Displacement Response
Test 19 Aspect Ratio of 5.33 and 10% Axial Load – Experimental Observations

Specimens 19-24 focus on the effects of axial load and aspect ratio on column behavior. The 18” diameter columns contain 10 #6 (A706) bars for longitudinal reinforcement ($A_{st}/A_g = 1.7\%$) and a #3 A706 spiral at 2” on center ($4A_{sp}/(D’s) = 1.3\%$). The specific specimen chosen for Test 19 has an 8ft cantilever length ($L/D = 5.33$), and was subjected to ($P/(f'_cA_g) = 10\%$) axial load. The symmetric three-cycle-set laboratory load history was used for Tests 19-24. The load history begins with elastic cycles to the following increments of the analytically predicted first yield force: $\frac{1}{4}F_y'$, $\frac{1}{2}F_y'$, $\frac{3}{4}F_y'$, and $F_y'$. The first yield force for the tested material and geometric properties was determined using moment curvature analysis (Test 19: Cumbia $Fy' = 21.90$ kips with $f'_c = 6334$ psi). The first yield displacement for the nineteenth test was obtained as an average for the experimental first yield push and pull cycles ($\Delta'_y = 0.87''$). Vertical strain profiles for both push and pull
cycles up to the first yield force appear in Figure 3.416 with a dashed line representing the yield strain of the longitudinal reinforcement. The equivalent yield displacement, used to determine the displacement ductility levels ($\mu_{\Delta 1} = 1 \cdot \Delta_\gamma$), is then calculated as $\Delta_\gamma = \Delta_{\gamma}'(M_R/M'_{\gamma}) = 1.15''$ for Test 19. The symmetric three-cycle-set load history resumes with three balanced cycles at each of the following ductility levels: 1, 1.5, 2, 3, 4, 5, 6, 7, 8, etc. The full symmetric three-cycle-set load history appears in Figure 3.413 and the resulting lateral force vs. top column displacement hysteresis is shown in Figure 3.414. The monotonic moment curvature prediction does not include P-$\Delta$ effects.

![Figure 3.416 T19 – Strain Profiles before Yield (Left) Bar N3 and (Right) Bar S3](image)

The test began with cycles in ¼ $F_{y'}$ (first yield force) increments in each direction of loading until the first yield force was reached. The first cracks on the North and South sides of the specimen were observed during ($3/4F_{y'} = 15.95$ kips) and ($-3/4F_{y'} = -16.96$ kips) respectively. The first cracks appeared at the location of the spirals, therefore they were difficult to locate and measure. Crack widths measured on the outside surface of the concrete core have little meaning when describing a serviceability limit state which applies to columns with cover concrete. The crack distribution on all sides of the specimen at first yield, ($F_{y'} = 21.39$ kips) and ($-F_{y'} = -22.29$ kips), appears in Figure 3.417.
Similarly, the crack progression at displacement ductility 1, 1.5, and 2 appear in Figure 3.418, Figure 3.419, and Figure 3.420 respectively. During these cycles the cracks became more numerous and increased in inclination on the shear faces of the specimen. A small amount of core concrete crushed on the North side of the specimen during \(\mu_3^{-2} = -3.44\), as shown in the left photo of Figure 3.421. A similar observation was made, see the right photo of Figure 3.421, on the South side of the specimen during \(\mu_3^{+3} = 3.43\)". The crushing on each side of the specimen during ductility three was not severe, and it appeared that only a thin layer of concrete flaked off between spiral layers. The crack distribution on the front side of the specimen during \(\mu_3^{-3} = -3.43\)" and \(\mu_4^{-3} = -4.59\)" appears in the left and right photos of Figure 3.422.

The South extreme fiber bar S3 visibly buckled after reversal from \(\mu_5^{-2} = -5.71\)"), as shown in Figure 3.423. After reversal from \(\mu_5^{+3} = 5.74\)"), the North extreme fiber bar N3 visibly buckled as shown in Figure 3.424. The additional deformation in previously buckled bars S3 during \(\mu_6^{+1} = 6.86\)" and N3 during \(\mu_6^{-1} = -6.88\)" is shown in Figure 3.425. The deformed spiral layers over the outward buckled region of bars S3 and N3 allow for further cycle to cycle degradation of the core concrete. Previously buckled South extreme fiber bar S3 ruptured during the pull cycle to \(\mu_6^{-3} = -6.88\)"), as shown in the left photo of Figure 3.426. During this same cycle two additional North reinforcing bars buckled, N2 and N4, as shown in the right photo of Figure 3.426 and the left photo of Figure 3.427. Rupture of bar S3 lead to a 24% loss in strength measured at \(\mu_6^{-3} = -6.88\)" relative to the peak load in the pull direction of loading. During the push cycle to \(\mu_6^{+4} = 6.89\)"), previously buckled extreme fiber North reinforcement bar N3 ruptured as shown in the right photo of Figure 3.427. Rupture of bar N3 lead to a 29% loss in strength measured at \(\mu_6^{+4} = 6.89\)" relative to the peak load in the push direction of loading. Normally a fourth cycle at displacement ductility six would not appear within the load history, but it was apparent that it would not take additional displacement to rupture the North reinforcement. After reaching \(\mu_6^{+5} = 6.90\)"), the test was concluded. Photos of the specimen after removal of all of the instrumentation appear in Figure 3.428.
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Figure 3.417 T19 – (Left) North at $F_y'$, (Mid) Front at $-F_y'$, and (Right) South at $-F_y'$

Figure 3.418 T19 – (Left) North at $\mu_1^{+3}$, (Mid) Front at $\mu_1^{-3}$, and (Right) South at $\mu_1^{-3}$
Figure 3.419 T19 – (Left) North at $\mu_{1.5}^+$, (Mid) Front at $\mu_{1.5}^-$, and (Right) South at $\mu_{1.5}^-$.

Figure 3.420 T19 – (Left) North at $\mu_2^+$, (Mid) Front at $\mu_2^-$, and (Right) South at $\mu_2^-$. 
Figure 3.421  T19 – (Left) North Crushing at ($\mu_3^{-2}$) and (Right) South Side at ($\mu_3^{+3}$)

Figure 3.422  T19 – (Left) Front Crack Distribution at ($\mu_3^{-3}$) and (Right) Front at ($\mu_4^{-3}$)
Figure 3.423  T19 – Buckling of the South Bar S3 after Reversal from $(\mu_5^{-2})$

Figure 3.424  T19 – Buckling of the North Bar N3 after Reversal from $(\mu_5^{+3})$
Figure 3.425 T19 – Deformation in (Left) Bar S3 at $(\mu_6^{+1})$ and (Right) Bar N3 at $(\mu_6^{-1})$

Figure 3.426 T19 – (Left) Rupture of Previously Buckled Bar S3 during $(\mu_6^{-3})$ and (Right) Buckling of Bar N2 at $(\mu_6^{-3})$
Figure 3.427 T19 – (Left) Buckling of Bar N4 at ($\mu_6^{-3}$) and (Right) Rupture of Previously Buckled Bar N3 during ($\mu_6^{+4}$)

Figure 3.428 T19 – After the Conclusion of the Test (Left) North and (Right) South
Test 19 Aspect Ratio of 5.33 and 10% Axial Load – Strain Data Analysis

North Reinforcement

The vertical strain profile for North extreme fiber bar N3 placed into tension during push cycles appears in the right half of Figure 3.429. This figure shows both extreme fiber bars on the same graph to illustrate the effects of tension shift on strain profiles. As the hinge rotates about inclined flexural shear cracks, compressive strains are concentrated at the base and tensile strains are fanned out to a greater height following the crack distribution. Near the footing, cracks remain effectively horizontal, but above this base section the flexural shear cracks are inclined as shown in Figure 3.422.

The compressive vertical strain profile for North extreme fiber bar N3 during pull cycles appears in the left half of Figure 3.430. From displacement ductility four and onwards, two adjacent gage lengths on bar N3, located 5.17” and 7.15” above the footing, were combined into a single gage length centered 6.17” above the footing. The common LED for the two original gage lengths was partially debonded affecting its spatial readings. Similarly, the common LED for gage lengths centered 11.18” and 13.13” above the footing was not allowed to move freely by a piece of steel tie wire. These two gage lengths were also combined into a single gage length for cycles after displacement ductility four. These are the only two locations that were affected by this phenomenon.

A peak tensile strain of 0.0366, located 1.38” above the footing, was measured for North extreme fiber bar N3 during $(\mu_5^{\pm 1} = 5.72")$. The relationship between tensile strain and displacement for this gage length appears in Figure 3.433. Each line represents a single push cycle which began with the column at zero displacement and ended at the peak during a continuous push cycle. The solid line contains data during the push cycle loading up to the peak displacement, and the dashed line represents the subsequent reversal of load. This is the first gage length in which moment curvature prediction for the relationship between tensile strain and displacement using the PCK (2007) Lp Hinge Method matches the test results. Even though this gage length sustained the highest tensile strains, bar buckling occurred 8”
above the footing where multiple spiral layers yielded. The lowest spiral layer, which crossed the peak tensile gage length, remained elastic due to the additional confinement and restraint provided by the footing.

The largest compressive strain of -0.0243, located 9.16” above the footing, was measured during \( (\mu_5^{-2} = -5.71") \). The peak compressive strain of -0.0243 in bar N3 exceeds the Mander ultimate concrete compressive strain of -0.0205. The relationship between compressive strain and displacement for bar N3, gage length centered 9.16” above the footing, appears in Figure 3.435. Here the measured compressive strains are significantly larger than the moment curvature prediction with the PCK (2007) Lp Hinge Method. As a comparison, compressive strains measured 6.17” above the footing are shown in Figure 3.434. For both gage lengths, the measured compressive strains match the prediction up until displacement ductility two, but at larger displacements the measured strains are greater than the moment curvature prediction.

The strains in the lowest six transverse steel layers restraining North extreme fiber bar N3 are plotted in Figure 3.432. The individual data points are from strain gages attached to each spiral layer at a specific height above the footing. The vertical grey dashed line represents a fictitious spiral yield strain of \( \frac{fy}{Es} \), see Figure 3.409, that approximates the point at which permanent deformation in the spiral layer begins. The strain gage on the spiral layer 3.18” above the footing stopped functioning after displacement ductility two. This is of little consequence, since bar buckling occurred 8” above the footing over spiral layers which went inelastic during displacement ductility three. After reversal from \( (\mu_5^{+3} = 5.74") \), the tensile strain for these spiral layers rapidly increased as they accommodated the outward deformation of the buckled extreme fiber bar.

The strain hysteresis over the buckled region of bar N3, gage length located 6.17” above the footing, appears in Figure 3.438. Visible Buckling occurred after reversal from \( (\mu_5^{+3} = 5.74") \), here the deformation shown in Figure 3.424 does not have a large influence on the measured strains. The transverse steel strain gage hysteresis for the layer over the outward
buckled region of bar N3 appears in Figure 3.439. Outward deformation during visible buckling leads to an increase in the spiral strain, causing the gage to go beyond its measurable range. A similar increase in the measured spiral strains occurred during ($\mu_5^{-2} = -5.71''$), but this was not accompanied by visible bar buckling.

**South Reinforcement**

The vertical strain profile for South extreme fiber bar S3 placed into tension during pull cycles appears in the right half of Figure 3.430. A peak tensile strain of 0.0322 was measured 3.63” above the footing during ($\mu_5^{-1} = -5.72''$). The relationship between tensile strain and displacement for this gage length is shown in Figure 3.436. The same comments on the accuracy of the moment curvature prediction for the North reinforcement bar N3 also apply to bar S3.

Vertical strain profiles for bar S3 in compression during push cycles appear in the left half of Figure 3.429. A peak compression strain of -0.0224 was measured 3.63” above the footing during ($\mu_5^{+1} = 5.72''$). The relationship between compressive strain and displacement for this gage length appears in Figure 3.437. The measured strains match the moment curvature prediction with the PCK (2007) LP Hinge Method well through ductility two, but exceed the prediction at higher ductility levels. The peak compressive strain of -0.0224 measured in bar S3 is larger than the Mander ultimate concrete compressive strain of -0.0205.

The strains in the lowest six transverse steel layers restraining South extreme fiber bar S3 are plotted in Figure 3.431. Compressive demands during displacement ductility three led to two layers of transverse steel entering the inelastic range. Prior to buckling, the strain in the two inelastic spiral layers increased and a third layer entered the inelastic range during ($\mu_5^{+1} = 5.72''$). The strain hysteresis for the outward buckled region of extreme fiber bar S3, gage length located 5.59” above the footing, appears in Figure 3.440. The strain gage hysteresis for the spiral layer overlaying the outward buckled region of bar S3 appears in Figure 3.441. Visible buckling of bar S3 occurred after reversal from ($\mu_5^{-2} = -5.71''$), but
the data suggests that significant outward deformation occurred after reversal from \((\mu^2 = -5.72")\). During this reversal outward deformation over bar S3 occurred as shown by the increased tensile strains measured in Figure 3.440. This coincides with a spike in the measured tensile strains in the spiral restraint, in Figure 3.441, which caused the strain gage to go off scale preventing further measurement.

![Figure 3.429](image)

**Figure 3.429** T19 – Extreme Fiber Vertical Strain Profiles during Push Cycles
Chapter 3: Experimental Observations

Figure 3.430  T19 – Extreme Fiber Vertical Strain Profiles during Pull Cycles

Figure 3.431  T19 – Spiral Strains on the South Side during Push Cycles
Figure 3.432 T19 – Spiral Strains on the North Side during Pull Cycles

Figure 3.433 T19 – Tensile Strain-Displacement for Bar N3 during Push Cycles
Figure 3.434 T19 – Compressive Strain-Displacement for Bar N3 during Pull Cycles

Figure 3.435 T19 – Compressive Strain-Displacement for Bar N3 during Pull Cycles
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Figure 3.436 T19 – Tensile Strain-Displacement for Bar S3 during Pull Cycles

Figure 3.437 T19 – Compressive Strain-Displacement for Bar S3 during Push Cycles
Chapter 3: Experimental Observations

Figure 3.438 T19 – Bar N3 Strain Hysteresis to Buckling (Gage Length 6.17” Above)

Figure 3.439 T19 – Spiral Strain Hysteresis for the Layer over North Buckled Region
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Figure 3.440  T19 – Bar S3 Strain Hysteresis to Buckling (Gage Length 5.59” Above)

Figure 3.441  T19 – Spiral Strain Hysteresis for the Layer over South Buckled Region
Test 19 – Curvature and Strain Penetration Data

The cross section curvature for each horizontal section above the footing is determined by connecting the strain measurements from all eight instrumented bars with a least squared error line. The curvature is then extracted from the slope of the least squared error line, see Figure 3.442 and Figure 3.443. The cross section curvature profiles in these figures are shown for the horizontal section closest to the footing-column interface. For these sections, it appears that the plane sections hypothesis is appropriate. Vertical curvature profiles are plotted for push and pull cycles as shown in Figure 3.444 and Figure 3.445 respectively. These figures show that plastic curvatures have a linear distribution at higher displacement ductility levels. The extent of plastic curvatures above the footing can be calculated by determining where the linear plastic curvature regression intersects the triangular yield curvature distribution, shown as a grey dashed line. Circular data points in Figure 3.451 plot the measured spread of plasticity as a function of base section curvature ductility.

The target marker on each bar placed closest the footing-column interface can be used to create slip hysteresis and horizontal slip profiles attributable to strain penetration. The slip hysteresis for extreme fiber bars N3 and S3 appear in Figure 3.448 and Figure 3.449 respectively. If the tensile and compressive slip of all of the instrumented bars is plotted along the cross section depth, the base rotation attributable to strain penetration may be calculated. The slip profiles for push and pull cycles appear in Figure 3.446 and Figure 3.447 respectively. The rotation of the base section can be extracted from the slope of the least squared error line connecting all six measured bar slips. Combining the curvatures over the instrumented region (5ft above the footing), bar slip profiles, and an elastic curvature assumption above the instrumented region, the top column displacement can be calculated. This top column displacement calculated from the Optotrak system is compared to the top column displacement measured with a string potentiometer at the center of loading in Figure 3.450. The Optotrak integrated displacements match well in the push direction of loading, but they are uniformly over predicting the pull displacements by a small margin.
Figure 3.442 T19 – Lowest Horizontal Section Strain Profiles during Push Cycles

Figure 3.443 T19 – Lowest Horizontal Section Strain Profiles during Pull Cycles
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Figure 3.444 T19 – Push Cycle Curvature Profiles with Plastic Regression

Figure 3.445 T19 – Pull Cycle Curvature Profiles with Plastic Regression
Figure 3.446 T19 – Base Rotation due to Strain Penetration during Push Cycles

Figure 3.447 T19 – Base Rotation due to Strain Penetration during Pull Cycles
Figure 3.448 T19 – Base Section Slip Hysteresis for North Extreme Fiber Bar N3

Figure 3.449 T19 – Base Section Slip Hysteresis for South Extreme Fiber Bar S3
Figure 3.450  T19 – Comparison of Measured and Optotrak Integrated Displacements

Figure 3.451  T19 – Measured Spread of Plasticity (Circular Data Points)
3.3.2 Test 20 – Aspect Ratio of 5.33 and 5% Axial Load

Table 3.35 Observations for Test 20 – Aspect Ratio of 5.33 and 5% Axial Load

VALUES OF INTEREST:

Concrete Compressive Strength: \( f'_c = 6467 \text{ psi} \)
Axial Load: \( P = 73.4 \text{ kips} \) \( (P/(f'_c A_g) = 5\%) \)
Column Length and Aspect Ratio: \( 8ft \) \( (L/D = 5.33) \)
Analytical First Yield Force: \( F'_y = 18.58 \text{ kips} \)
Experimental First Yield Displacement: \( \Delta'_y = 0.86" \)
Analytical Nominal Moment Capacity: \( M_n = 205.63 \text{ kip} \ast ft \)
Equivalent Yield Displacement: \( \Delta_y = 1.18" \)
Maximum Lateral Force: \( 27.3 \text{ kips} \)

DAMAGE OBSERVATIONS:

First Cracking North: \( 3/4Fy' = 0.53" \)
First Cracking South: \( -3/4Fy' = -0.56" \)
Cover Concrete Crushing North: \( \mu_3^{-2} = -3.56" \)
Cover Concrete Crushing South: \( \mu_3^{+1} = 3.55" \)
Transverse Steel Yield North: At \(-4.04" \) during pull to \( \mu_4^{-1} = -4.72" \)
Transverse Steel Yield South: At \(3.54" \) during push to \( \mu_3^{+1} = 3.55" \)
Longitudinal Bar Buckling North: *Deformation after Reversal from \( \mu_6^{+4} = 7.10" \)
Longitudinal Bar Buckling South: Reversal from \( \mu_5^{-2} = -5.93" \)
Longitudinal Bar Fracture South: At \(-5.43" \) during pull to \( \mu_6^{-3} = -7.01" \)

*\( \mu_5^{-2} = -5.93" \) represents the second pull cycle of displacement ductility five
Table 3.36 Strain Data Summary for Test 20 – Aspect Ratio of 5.33 and 5% Axial Load

MATERIAL STRAINS:

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<tr>
<th>Component</th>
<th>Strain Value</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North</td>
<td>( \varepsilon_s = 0.0065 ) (compression)</td>
<td>( \mu_{\Delta 2} = -2.36&quot; )</td>
</tr>
<tr>
<td>Cover Concrete Crushing South</td>
<td>( \varepsilon_s = 0.0046 ) (compression)</td>
<td>( \mu_{\Delta 2} = 2.36&quot; )</td>
</tr>
<tr>
<td>Transverse Steel Yield North</td>
<td>( \varepsilon_s = 0.0114 ) (compression)</td>
<td></td>
</tr>
<tr>
<td>Transverse Steel Yield South</td>
<td>( \varepsilon_s = 0.0109 ) (compression)</td>
<td></td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North</td>
<td>( \varepsilon_s = 0.046 ) (peak tension prior to ( bb ))</td>
<td></td>
</tr>
<tr>
<td>* (Measurable Deformation in N3)</td>
<td>( \varepsilon_s = 0.016 ) (peak comp. prior to ( bb ))</td>
<td></td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South</td>
<td>( \varepsilon_s = 0.037 ) (peak tension prior to ( bb ))</td>
<td>( \varepsilon_s = 0.016 ) (peak comp. prior to ( bb ))</td>
</tr>
</tbody>
</table>

Mander (1988) Ultimate Concrete Compression Strain, \( \varepsilon_{cu} = 0.0202 \)

Figure 3.452 T20 – Cross Section Bar Designation and Target Marker Application
Figure 3.453  T20 – Symmetric Three Cycle Set Load History

Figure 3.454  T20 – Lateral Force vs. Top Column Displacement Response
Chapter 3: Experimental Observations

Figure 3.455  T20 – Compressive Axial Load from One Jack (Total = 2*Value)

Figure 3.456  T19 and T20 Hysteretic Comparison with Different Axial Loads
Test 20 Aspect Ratio of 5.33 and 5% Axial Load – Experimental Observations

The transverse steel detailing, longitudinal reinforcement content, and material properties remain constant for Tests 19-24. The 18” diameter columns contain 10 #6 (A706) bars for longitudinal reinforcement \((A_{st}/A_g = 1.7\%)\) and a #3 A706 spiral at 2” on center \((4A_{sp}/(D’s) = 1.3\%)\). The specific specimen chosen for Test 20 has an 8ft cantilever length \((L/D = 5.33)\), and was subjected to \((P/(f’cA_g) = 5\%)\) axial load. The symmetric three-cycle-set laboratory load history is used to evaluate the seismic performance of structural components. The load history begins with elastic cycles to the following increments of the analytically predicted first yield force: \(\frac{1}{4} F_y’, \frac{1}{2} F_y’, \frac{3}{4} F_y’, \text{ and } F_y’\). The first yield force for the tested material and geometric properties was determined using moment curvature analysis (Test 20: Cumbia Fy’ = 18.58 kips with f’c = 6467 psi). The first yield displacement was obtained as an average for the experimental first yield push and pull cycles \((\Delta y’ = 0.86”)\).

Vertical strain profiles for both push and pull cycles up to the first yield force appear in Figure 3.457 with a dashed line representing the yield strain of the longitudinal reinforcement. The equivalent yield displacement, used to determine the displacement ductility levels \((\mu_{\Delta 1} = 1 * \Delta y)\), is then calculated as \(\Delta y = \Delta y’(M_n/M_y’) = 1.18”\) for Test 20. The symmetric three-cycle-set load history resumes with three balanced cycles at each of the following ductility levels: 1, 1.5, 2, 3, 4, 5, 6, 7, 8, etc. The full symmetric three-cycle-set load history appears in Figure 3.453 and the resulting lateral force vs. top column displacement hysteresis is shown in Figure 3.454. The monotonic moment curvature prediction does not include P-\(\Delta\) effects. A comparison of the measured hysteretic response for Tests 19 and 20 is shown in Figure 3.456. The strength of Test 19 was higher due to the larger axial load, but the deformation capacity for the two specimens remained similar.
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Figure 3.457 T20 – Strain Profiles before Yield (Left) Bar N3 and (Right) Bar S3

The test began with cycles in $\frac{1}{4} F_y$ (first yield force) increments in each direction of loading until the first yield force was reached. The first cracks on the North and South sides of the specimen were observed during $(\frac{3}{4} F_y = 13.94 \text{ kips})$ and $(-\frac{3}{4} F_y = -13.83 \text{ kips})$ respectively. The first cracks appeared at the location of the spirals, therefore they were difficult to locate and measure. Crack widths measured on the outside surface of the concrete core have little meaning when describing a serviceability limit state which applies to columns with cover concrete. The crack distribution on all sides of the specimen at first yield, $(F_y = 18.61 \text{ kips})$ and $(-F_y = -18.51 \text{ kips})$, appears in Figure 3.458. Similarly, the crack progression at displacement ductility 1, 1.5, and 2 appear in Figure 3.459, Figure 3.460, and Figure 3.461 respectively. During these cycles the cracks became more numerous and increased in inclination on the shear faces of the specimen. A small amount of core concrete crushed on the South side of the specimen during $(\mu_3^+ = 3.55")$, as shown in the left photo of Figure 3.462. A similar observation was made, see the right photo of Figure 3.462, on the North side of the specimen during $(\mu_3^- = -3.56")$. The crushing on each side of the specimen during ductility three was not severe, and it appeared that only a thin layer of concrete flaked off between spiral layers. Photos of the crack distributions during displacement ductility three and four appear in Figure 3.463 and Figure 3.464 respectively.
The South extreme fiber bar S3 visibly buckled after reversal from \( (\mu_5^{-2} = -5.93\,\)"\), as shown in Figure 3.465. Photos of the crack distribution during displacement ductility five appear in Figure 3.466. Permanent deformation in spiral layers restraining the outward buckled region of bar S3 allows for further cycle to cycle degradation of the core concrete, see the left photo of Figure 3.468. An additional South reinforcing bar S4 buckled after reversal from \( (\mu_6^{-2} = -7.10\,\)"\), as shown in the right photo of Figure 3.468. Previously buckled South extreme fiber bar S3 ruptured during the pull cycle to \( (\mu_6^{-3} = -7.12\,\)"\), as shown in the right photo of Figure 3.469. Rupture of bar S3 lead to a 23% loss in strength measured at \( (\mu_6^{-3} = -7.12\,\)"\) relative to the peak load in the pull direction of loading. A fourth cycle at displacement ductility six was conducted. Although North bar buckling was not observed in the test, subsequent analysis of the strain data suggest that significant measurable deformation occurred in Bar N3 upon reversal from \( (\mu_6^{+4} = 7.10\,\)"\). Photos of the specimen after removal of the instrumentation appear in Figure 3.470.

Figure 3.458 T19 – (Left) North at Fy', (Mid) Front at -Fy', and (Right) South at -Fy'
Figure 3.459  T20 – (Left) North at $\mu_{1}^{+3}$, (Mid) Front at $\mu_{1}^{-3}$, and (Right) South at $\mu_{1}^{-3}$

Figure 3.460  T20 – (Left) North at $\mu_{1.5}^{+3}$, (Mid) Front at $\mu_{1.5}^{-3}$, and (Right) South at $\mu_{1.5}^{-3}$
Figure 3.461  T20 – (Left) North at $\mu_2^+$, (Mid) Front at $\mu_2^-$, and (Right) South at $\mu_2^-$

Figure 3.462  T20 – (Left) Cover Concrete Crushing on the South Side at $\mu_3^+$ and (Right) Crushing on the North Side during $\mu_3^-$
Figure 3.463  T20 – (Left) North at $\mu^+_3$, (Mid) Front at $\mu^-_3$, and (Right) South at $\mu^-_3$

Figure 3.464  T20 – (Left) North at $\mu^+_4$, (Mid) Front at $\mu^-_4$, and (Right) South at $\mu^-_4$
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Figure 3.465  T20 – Buckling of Bar S3 after Reversal from $\mu_5^{-2} = -5.93''$)

Figure 3.466  T20 – (Left) North at $\mu_5^{+3}$, (Mid) Front at $\mu_5^{-3}$, and (Right) South at $\mu_5^{-3}$
Figure 3.467  T20 – (Left) North Side at $\mu_6^{+2}$ and (Right) Deformation in Bar S3 at $\mu_6^{+2}$

Figure 3.468  T20 – (Left) Deformation in Spiral Layers over Buckled Bar S3 during $\mu_6^{-2}$ and (Right) Buckling of Bar S4 after Reversal from $\mu_6^{-2}$
Figure 3.469 T20 – (Left) Lateral Displacement at ($\mu_6^{-3} = 7.01"$) and (Right) Fracture of Previously Buckled Bar S3 during $\mu_6^{-3}$

Figure 3.470 T20 – After Testing (Left) North Side and (Right) South Side
Test 20 Aspect Ratio of 5.33 and 5% Axial Load – Strain Data Analysis

North Reinforcement

Vertical strain profiles for north extreme fiber bar N3 placed into tension during push cycles appear in the right half of Figure 3.471. This figure shows both extreme fiber bars on the same graph to illustrate the effects of tension shift on strain profiles. Compressive vertical strain profiles for North extreme fiber bar N3 during pull cycles appear in the left half of Figure 3.472. As the hinge rotates about inclined flexural shear cracks, compressive strains are concentrated at the base and tensile strains are fanned out to a greater height following the crack distribution.

A peak tensile strain of 0.0458, located 1.38” above the footing, was measured for North extreme fiber bar N3 during ($\mu_6^a = 7.10$”). The largest tensile strain in bar N3 measured during cycles at displacement ductility five was 0.0372 for the gage length 7.06” above the footing. The relationship between tensile strain and displacement for the gage length 3.17” above the footing appears in Figure 3.475. Each line represents a single push cycle which began with the column at zero displacement and ended at the peak during a continuous push cycle. The solid line contains data during the push cycle loading up to the peak displacement, and the dashed line represents the subsequent reversal of load. This gage length follows trends observed in the previous specimens. The moment curvature prediction with the PCK (2007) Lp Hinge Method matches well through displacement ductility three, but at larger displacements it begins to over predict the measured strains at an increasing rate.

The largest compressive strain of -0.0198, located 3.17” above the footing, was measured during ($\mu_6^{-1} = -7.09”$). The peak compressive strain of -0.0198 in bar N3 is lower the Mander ultimate concrete compressive strain of -0.0202. The relationship between compressive strain and displacement for bar N3 appears in Figure 3.476, for the gage length centered 3.17” above the footing. Here the measured compressive strains are significantly larger than the moment curvature prediction with the PCK (2007) Lp Hinge Method.
The strains in the lowest six transverse steel layers restraining North extreme fiber bar N3 are plotted in Figure 3.474. The individual data points are from strain gages attached to each spiral layer. The vertical grey dashed line represents a spiral yield strain of \( \frac{f_y}{E_s} \), see Figure 3.409, which approximates the point at which permanent deformation in the spiral layer begins. Two adjacent spiral layers, 5.22” and 7.28” above the footing, entered the inelastic range during displacement ductility four. While the spiral strains over these layers marginally increased during displacement ductility five, it appears that the largest increases in spiral demand occurred over the layers 1.31” and 3.31” above the footing during displacement ductility six. The spiral strains in the lowest two transverse steel layers sharply increased during (\( \mu_6^{+4} = 7.10" \)), which suggests that measurable outward deformation of Bar N3 is present. The strain hysteresis for bar N3, gage length located 3.17” above the footing, appears in Figure 3.479. While visible buckling was not observed during the load history, measurable deformation occurred after reversal from (\( \mu_6^{+4} = 7.10" \)). The transverse steel strain gage hysteresis for the layer in the region of Bar N3 outward deformation appears in Figure 3.480. Larger cycle to cycle increases in the transverse steel demand were observed during ductility six as the apparent measurable deformation increased.

\textit{South Reinforcement}

Vertical strain profiles for south extreme fiber bar S3 placed into tension during pull cycles appear in the right half of Figure 3.472. A peak tensile strain of 0.0367 was measured 5.29” above the footing during (\( \mu_{s}^{-1} = -5.89" \)). The relationship between tensile strain and displacement for this gage length is shown in Figure 3.477. The same comments on the accuracy of the moment curvature prediction for the North reinforcement bar N3 also apply to bar S3. Vertical strain profiles for bar S3 in compression during push cycles appear in the left half of Figure 3.471. A peak compression strain of -0.0155 was measured 1.45” above the footing during (\( \mu_{s}^{+2} = 5.92" \)). The relationship between compressive strain and displacement for the gage length 3.30” above the footing appears in Figure 3.478. The measured strains match the moment curvature prediction with the PCK (2007) Lp Hinge Method well through ductility 1.5, but at higher ductility levels the measured compressive
strains are larger than the moment curvature prediction. The peak compressive strain of -0.0155 measured in bar S3 is larger than the Mander ultimate concrete compressive strain of -0.0202.

The strains in the lowest six transverse steel layers restraining south extreme fiber bar S3 are plotted in Figure 3.473. The strain gage attached to the spiral layer 7.31” above the footing stopped functioning after displacement ductility two. A single spiral layer, located 3.31” above the footing, entered the inelastic range during displacement ductility three. The measured strain in the spiral layer 3.31” above the footing increased during $(\mu^2 = 5.92”)$ relative to the values measured during the previous push cycle. The South extreme fiber bar S3 visibly buckled after reversal from $(\mu^2 = -5.93”)$, as shown in Figure 3.465. The outward buckled region occurred over the previously inelastic spiral layer 3.31” above the footing. The strain hysteresis for the outward buckled region of bar S3 appears in Figure 3.481. The strain hysteresis for the spiral layer overlaying the outward buckled region of bar S3 is shown in Figure 3.482. Cycle to cycle increases in the measured spiral strains were observed during displacement ductility five, and a sharp increase occurred after reversal from $(\mu^2 = -5.93”)$ when visible bar buckling was observed.
Figure 3.471 T20 – Extreme Fiber Vertical Strain Profiles during Push Cycles

Figure 3.472 T20 – Extreme Fiber Vertical Strain Profiles during Pull Cycles
Figure 3.473  T20 – Spiral Strains on the South Side during Push Cycles

Figure 3.474  T20 – Spiral Strains on the North Side during Pull Cycles
Figure 3.475  T20 – Tensile Strain-Displacement for Bar N3 during Push Cycles

Figure 3.476  T20 – Compressive Strain-Displacement for Bar N3 during Pull Cycles
Figure 3.477  T20 – Tensile Strain-Displacement for Bar S3 during Pull Cycles

Figure 3.478  T20 – Compressive Strain-Displacement for Bar S3 during Push Cycles
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Figure 3.479  T20 – Extreme Fiber Bar N3 Strain Hysteresis (3.17” above the Footing)

Figure 3.480  T20 – Spiral Strain Hysteresis 1.31” Above the Footing on North Side
Figure 3.481  T20 – Extreme Fiber Bar S3 Strain Hysteresis (5.29” above the Footing)

Figure 3.482  T20 – Spiral Strain Hysteresis 3.31” Above the Footing on South Side
Test 20 – Curvature and Strain Penetration Data

The cross section curvature for each horizontal section above the footing is determined by connecting the strain measurements from all eight instrumented bars with a least squared error line. The curvature is then extracted from the slope of the least squared error line, see Figure 3.483 and Figure 3.484. The cross section curvature profiles in these figures are shown for the second horizontal section above the footing-column interface. This is because the target markers in the lowest gage lengths for bars near the middle of the column were not visible to the Optotrack position monitor. For these sections, it appears that the plane sections hypothesis is appropriate. Vertical curvature profiles are plotted for push and pull cycles in Figure 3.485 and Figure 3.486 respectively. These figures show that plastic curvatures have a linear distribution at higher displacement ductility levels. The extent of plasticity is determined by the intersection of the linear plastic curvature regression and the triangular yield curvature profile, shown as a grey dashed line. The dashed lines for each curvature distribution represent a least squared error linear fit to the plastic portion of the measured curvatures. The measured spread of plasticity as a function of base section curvature ductility is shown in Figure 3.493.

The target marker on each bar placed closest the footing-column interface can be used to create slip hysteresis and horizontal slip profiles attributable to strain penetration. The slip hysteresis for extreme fiber bars N3 and S3 appear in Figure 3.487 and Figure 3.488 respectively. If the tensile and compressive slip of all of the instrumented bars is plotted along the cross section depth, the base rotation attributable to strain penetration may be calculated. The slip profiles for push and pull cycles appear in Figure 3.489 and Figure 3.490 respectively. The rotation of the base section can be extracted from the slope of the least squared error line connecting all six measured bar slips.

Combining the curvatures over the instrumented region (5ft above the footing), bar slip profiles, and an elastic curvature assumption above the instrumented region, the top column displacement can be calculated by integrating the curvature distributions and extrapolating the fixed-end rotations to the center of loading. The top column displacements calculated
from the Optotrak system are compared to displacements measured with a string potentiometer at the center of loading in Figure 3.491. The Optotrak integrated displacements match well throughout the entire range of response. A closer look at the deformation components from the Optotrak integrated displacements appear in Figure 3.492.

![Figure 3.483 T20 – Sample Cross Section Strain Profiles during Push Cycles](image-url)
Figure 3.484  T20 – Sample Cross Section Strain Profiles during Pull Cycles

Figure 3.485  T20 – Push Cycle Curvature Profiles with Plastic Regression
Figure 3.486  T20 – Pull Cycle Curvature Profiles with Plastic Regression

Figure 3.487  T20 – Strain Penetration Bond Slip Hysteresis for Bar N3
Figure 3.488 T20 – Strain Penetration Bond Slip Hysteresis for Bar S3

Figure 3.489 T20 – Base Rotation due to Strain Penetration during Push Cycles
Figure 3.490  T20 – Base Rotation due to Strain Penetration during Pull Cycles

Figure 3.491  T20 – Comparison of Measured and Optotrak Integrated Displacements
Figure 3.492 T20 – Components of the Optotrak Integrated Deformation

Figure 3.493 T20 – Measured Spread of Plasticity (Circular Data Points)
### 3.3.3 Test 21 – Aspect Ratio of 7.33 and 5% Axial Load

Table 3.37 Observations for Test 21 – Aspect Ratio of 7.33 and 5% Axial Load

**VALUES OF INTEREST:**

- Concrete Compressive Strength: \( f'_c = 6390 \text{ psi} \)
- Axial Load: \( P = 72.5 \text{ kips} \) \( (P/(f'_c A_g) = 5\%) \)
- Column Length and Aspect Ratio: \( 11\text{ ft} \) \( (L/D = 7.33) \)
- Analytical First Yield Force: \( F'_y = 13.50 \text{ kips} \)
- Experimental First Yield Displacement: \( \Delta'_y = 1.43" \)
- Analytical Nominal Moment Capacity: \( M_n = 205.5 \text{ kip ft} \)
- Equivalent Yield Displacement: \( \Delta_y = 1.98" \)
- Maximum Lateral Force: 19.34 kips

**DAMAGE OBSERVATIONS:**

- First Cracking North: \( 1/2Fy' = 0.40" \)
- First Cracking South: \( -1/2Fy' = -0.45" \)
- Cover Concrete Crushing North: \( \mu_3^{-1} = -5.94" \)
- Cover Concrete Crushing South: \( \mu_2^{+3} = 3.97" \)
- Transverse Steel Yield North: At \(-6.50"\) during pull to \( \mu_4^{-1} = -7.91" \)
- Transverse Steel Yield South: At \(5.67"\) during push to \( \mu_3^{+1} = 5.94" \)
- Longitudinal Bar Buckling North: Reversal from \( \mu_6^{+1} = 11.86" \)
- Longitudinal Bar Buckling South: Reversal from \( \mu_5^{-2} = -9.88" \)
- Longitudinal Bar Fracture North: At \(7.59"\) during push to \( \mu_6^{+4} = 11.88" \)
- Longitudinal Bar Fracture South: At \(-2.81"\) during pull to \( \mu_6^{-4} = -11.86" \)

*\( \mu_5^{-2} = -9.88" \) represents the second pull cycle of displacement ductility five
### Table 3.38 Strain Data Summary for Test 21 – Aspect Ratio of 7.33 and 5% Axial Load

**MATERIAL STRAINS:**

- **Cover Concrete Crushing North:** $\varepsilon_s = 0.0046$ (compression) \(\mu_{A2} = -3.95\)
- **Cover Concrete Crushing South:** $\varepsilon_s = 0.0048$ (compression)
- **Transverse Steel Yield North:** $\varepsilon_s = 0.0146$ (compression)
- **Transverse Steel Yield South:** $\varepsilon_s = 0.0102$ (compression)
- **Longitudinal Bar Buckling North:** $\varepsilon_s = 0.051$ (peak tension prior to $bb$)
  
  $\varepsilon_s = 0.024$ (peak comp. prior to $bb$)
- **Longitudinal Bar Buckling South:** $\varepsilon_s = 0.036$ (peak tension prior to $bb$)
  
  $\varepsilon_s = 0.034$ (peak comp. prior to $bb$)

Mander (1988) Ultimate Concrete Compression Strain, $\varepsilon_{cu} = 0.0203$

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**Figure 3.494 T21 – Cross Section Bar Designation and Target Marker Application**
Figure 3.495  T21 – Symmetric Three Cycle Set Load History

Figure 3.496  T21 – Lateral Force vs. Top Column Displacement Response
Figure 3.497  T21 – Compressive Axial Load from One Jack (Total = 2*Value)

Test 21 Aspect Ratio of 7.33 and 5% Axial Load – Experimental Observations

Specimens 19-24 focus on the effects of axial load and aspect ratio on column behavior. The 18” diameter columns contain 10 #6 (A706) bars for longitudinal reinforcement ($A_{st}/A_g = 1.7\%$) and a #3 A706 spiral at 2” on center ($4A_{sp}/(D') = 1.3\%$). The specific specimen chosen for Test 21 has an 11ft cantilever length ($L/D = 7.33$), and was subjected to ($P/(f'_c A_g) = 5\%$) axial load. The symmetric three-cycle-set laboratory load history is used to evaluate the seismic performance of structural components. The load history begins with elastic cycles to the following increments of the analytically predicted first yield force: $\frac{1}{4} F_y'$, $\frac{1}{2} F_y'$, $\frac{3}{4} F_y'$, and $F_y'$. The first yield force for the tested material and geometric properties was determined using moment curvature analysis (Test 21: Cumbia $F_y' = 13.50$ kips with $f_c = 6390$ psi). The first yield displacement was obtained as an average for the experimental first yield push and pull cycles ($\Delta'_y = 1.43''$). Vertical strain profiles for both
push and pull cycles up to the first yield force appear in Figure 3.498 with a dashed line representing the yield strain of the longitudinal reinforcement. The equivalent yield displacement, used to determine the displacement ductility levels ($\mu_{\Delta_1} = 1 * \Delta_y$), is then calculated as $\Delta_y = \Delta'_{fy}(M_n/M_{fy}') = 1.98"$ for Test 21. The symmetric three-cycle-set load history resumes with three balanced cycles at each of the following ductility levels: 1, 1.5, 2, 3, 4, 5, 6, 7, 8, etc. The full symmetric three-cycle-set load history appears in Figure 3.495 and the resulting lateral force vs. top column displacement hysteresis is shown in Figure 3.496. The monotonic moment curvature prediction does not include P-$\Delta$ effects.

![Figure 3.498 T21 – Strain Profiles before Yield (Left) Bar N3 and (Right) Bar S3](image)

The test began with cycles in $\frac{1}{4} F_y'$ (first yield force) increments in each direction of loading until the first yield force was reached. The first cracks on the North and South sides of the specimen were observed during $(1/2Fy' = 6.42 \text{ kips})$ and $(-1/2Fy' = -6.75 \text{ kips})$ respectively. The first crack occurred at the footing column interface. Crack widths measured on the outside surface of the concrete core have little meaning when describing a serviceability limit state which applies to columns with cover concrete. Cracking in the column occurred during $(3/4Fy' = 9.95 \text{ kips})$ and $(-3/4Fy' =$
−10.10 kips). The crack distribution on all sides of the specimen at first yield, \((Fy' = 13.28 \text{ kips})\) and \((-Fy' = -13.50 \text{ kips})\), appears in Figure 3.499.

Similarly, the crack progression at displacement ductility 1, 1.5, and 2 appear in Figure 3.500, Figure 3.501, and Figure 3.502 respectively. During these cycles the cracks became more numerous and increased in inclination on the shear faces of the specimen. A small amount of core concrete crushed on the South side of the specimen during \((\mu_2^{+3} = 3.97")\), as shown in the left photo of Figure 3.503. A similar observation was made, see the right photo of Figure 3.503, on the North side of the specimen during \((\mu_3^{-1} = -5.94")\). The crushing on each side of the specimen during ductility three was not severe, and it appeared that only a thin layer of concrete flaked off between spiral layers. Photos of the crack distributions during displacement ductility three and four appear in Figure 3.504 and Figure 3.505 respectively.

The South extreme fiber bar S3 visibly buckled after reversal from \((\mu_5^{-2} = -9.88")\), as shown in Figure 3.506. Photos of the crack distribution during displacement ductility five appear in Figure 3.507. The North extreme fiber bar N3 visibly buckled after reversal from \((\mu_6^{+4} = 11.86")\), as shown in the right photo of Figure 3.508. Additional deformation in the previously buckled bar S3 appears in the left photo of Figure 3.508 and Figure 3.509. An additional North reinforcing bar buckled after reversal from \((\mu_6^{+3} = 11.87")\), as shown in Figure 3.509. The previously buckled North extreme fiber bar N3 ruptured during \((\mu_6^{+4} = 11.88")\), see Figure 3.510. Similarly, previously buckled bar S3 ruptured during \((\mu_6^{-4} = -11.86")\). A photo of the south ruptured bar and lateral displacement at \((\mu_6^{+4} = -11.86")\) appears in Figure 3.510 and Figure 3.511.
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Figure 3.499 T21 – (Left) North at $F_y'$, (Mid) Front at -$F_y'$, and (Right) South at -$F_y'$

Figure 3.500 T21 – (Left) North at $\mu_1^{+3}$, (Mid) Front at $\mu_1^{-3}$, and (Right) South at $\mu_1^{-3}$
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Figure 3.501  T21 – (Left) North at $\mu_{1.5}^+3$, (Mid) Front at $\mu_{1.5}^-$3, and (Right) South at $\mu_{1.5}^-3$

Figure 3.502  T21 – (Left) North at $\mu_{2.5}^+3$, (Mid) Front at $\mu_{2.5}^-3$, and (Right) South at $\mu_{2.5}^-3$
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Figure 3.503 T21 – (Left) South Crushing at $\mu_2^{+3}$ and (Right) North Crushing at $\mu_3^{-1}$

Figure 3.504 T21 – (Left) North at $\mu_3^{+3}$, (Mid) Front at $\mu_3^{-3}$, and (Right) South at $\mu_3^{-3}$
Figure 3.505 T21 – (Left) North at $\mu_4^+3$, (Mid) Front at $\mu_4^-3$, and (Right) South at $\mu_4^-3$

Figure 3.506 T21 – Buckling of South Bar S3 after Reversal from ($\mu_5^2 = -9.88''$)
Figure 3.507  T21 – (Left) North Side at $\mu_5^{+3}$ and (Right) South Side at $\mu_5^{-3}$

Figure 3.508  T21 – (Left) Deformation in Bar S3 at $\mu_6^{+1}$ and (Right) Buckling of Bar N3 after reversal from $\mu_6^{-1}$, Note the Two Outward Deformed Regions
Figure 3.509 T21 – (Left) Deformation in Bar S3 at $\mu_6^{+3}$ and (Right) Buckling of Bar N4 during $\mu_6^{-3}$ and Additional Deformation in Bar N3

Figure 3.510 T21 – Fracture of (Left) Bar N3 at $\mu_6^{+4}$ and (Right) Bar S3 during at $\mu_6^{-4}$
Chapter 3: Experimental Observations

Test 21 Aspect Ratio of 7.33 and 5% Axial Load – Strain Data Analysis

North Reinforcement

Vertical strain profiles for north extreme fiber bar N3 placed into tension during push cycles appear in the right half of Figure 3.512. This figure shows both extreme fiber bars on the same graph to illustrate the effects of tension shift on strain profiles. Each reinforcing bar contains thirty separate \( \approx 2'' \) gage lengths which appear as a single data point at its center linked to adjacent gage lengths with straight lines. Compressive vertical strain profiles for North extreme fiber bar N3 during pull cycles appear in the left half of Figure 3.513. As the hinge rotates about inclined flexural shear cracks, compressive strains are concentrated at the base and tensile strains are fanned out to a greater height following the crack distribution. Near the footing cracks remain effectively horizontal, but above this base section the flexural shear cracks are inclined on the shear faces of the column.
A peak tensile strain of 0.051, located 1.49” above the footing, was measured for North extreme fiber bar N3 during \( \mu_{6}^{+1} = 11.86” \). The largest tensile strain in bar N3 measured during cycles at displacement ductility five was 0.0437 for the gage length 1.49” above the footing. The relationship between tensile strain and displacement for this gage length appears in Figure 3.516. Each line represents a single push cycle which began with the column at zero displacement and ended at the peak during a continuous push cycle. The solid line contains data during the push cycle loading up to the peak displacement, and the dashed line represents the subsequent reversal of load. This gage length matches the moment curvature prediction well when the PCK (2007) Lp Hinge Method is used.

The largest compressive strain of -0.0237, located 3.37” above the footing, was measured during \( \mu_{5}^{-3} = -9.89” \). This value exceeds the Mander ultimate concrete compressive strain of -0.0203. The relationship between compressive strain and displacement for bar N3 appears in Figure 3.517, for the gage length centered 3.37” above the footing. The measured compressive strains are significantly larger than the moment curvature prediction with the PCK (2007) Lp Hinge Method.

The strains in the lowest six transverse steel layers restraining North extreme fiber bar N3 are plotted in Figure 3.515. The individual data points are from strain gages attached to each spiral layer. The data points are connected with lines only to show trends for the particular displacement level. The vertical grey dashed line represents a spiral yield strain of \( \text{fy/Es} \), see Figure 3.409, which approximates the point at which permanent deformation in the spiral layer begins. The spiral layer located 7.22” above the footing went into the inelastic range during at -6.49” during the pull cycle to \( \mu_{4}^{-1} = -7.91” \). The lowest two spiral layers, 1.22” and 3.28” above the footing, went inelastic during \( \mu_{5}^{-1} = -9.93” \). Cycle to cycle increases in the spiral strains were observed over these gage lengths during displacement ductility five.

The strain hysteresis for bar N3, gage length located 7.27” above the footing, appears in Figure 3.521. This gage length coincides with the upper buckled region of bar N3 shown in
Figure 3.508. During later cycles of the load history the buckled deformation increased significantly over the lower buckled region. The transverse steel strain hysteresis for the spiral layer overlaying the upper buckled region appears in Figure 3.522. The strain hysteresis for the lower buckled region of bar N3 appears in Figure 3.523 for the gage length located 3.37” above the footing. The corresponding spiral strain hysteresis for the lowest spiral layer is shown in Figure 3.524. Over both buckled regions, the spiral strains spiked after reversal from ($\mu_5^{+1} = 11.86$”) when visible bar buckling was observed.

South Reinforcement

Vertical strain profiles for south extreme fiber bar S3 placed into tension during pull cycles appear in the right half of Figure 3.513. A peak tensile strain of 0.0360 was measured 7.05” above the footing during ($\mu_5^{-2} = -9.88$”). The relationship between tensile strain and displacement for this gage length is shown in Figure 3.518. The moment curvature analysis with the PCK (2007) Lp Hinge Method over predicts the measured tensile strains at an increasing rate at higher ductility levels. Vertical strain profiles for bar S3 in compression during push cycles appear in the left half of Figure 3.512. A peak compression strain of -0.0335 was measured 1.23” above the footing during ($\mu_5^{+1} = 9.88$”). This gage length was obstructed by debris during ($\mu_5^{+2} = 9.89$”). A strain of -0.0235 was measured over the second largest compressive gage length, located 5.02” above the footing, during ($\mu_5^{+2} = 9.89$”). The relationships between compressive strain and displacement for these gage lengths appear in Figure 3.519 and Figure 3.520. Again, the measured compressive strains are significantly larger than the moment curvature prediction with the PCK (2007) Lp Hinge Method. The peak compressive strains for these two gage lengths are also larger than the Mander ultimate concrete compressive strain value of -0.0203.

The strains in the lowest six transverse steel layers restraining south extreme fiber bar S3 are plotted in Figure 3.514. A single spiral layer located 5.16” above the footing entered the inelastic range at 5.67” during the push cycle to ($\mu_3^{+1} = 5.94$”). The measured spiral strains in the layers located 3.09” and 5.16” above the footing increased significantly during
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\((\mu^+_5 = 9.89")\) compared to the valued measured at \((\mu^+_5 = 9.88")\). The South extreme fiber bar S3 visibly buckled after reversal from \((\mu^-_5 = -9.88")\), as shown in Figure 3.506. The strain hysteresis for the outward buckled region of bar S3, gage length located 3.02” above the footing, appears in Figure 3.525. A transverse steel strain hysteresis for the spiral layer overlaying the outward buckled region is shown in Figure 3.526. The data suggests that measurable deformation was present after reversal from \((\mu^-_5 = -9.93")\), even though visible bar buckling was not observed until the following cycle. The measurable deformation decreased the apparent compressive strains measured during the push to \((\mu^+_5 = 9.89")\), and lead to an increase in the spiral restraint strain. Visible bar buckling occurred after reversal from \((\mu^-_5 = -9.88")\), leading to an elongation of the gage length over the outward buckled region and a spike in the transverse steel strain.

![Figure 3.512 T21 – Extreme Fiber Vertical Strain Profiles during Push Cycles](image-url)
Figure 3.513  T21 – Extreme Fiber Vertical Strain Profiles during Pull Cycles

Figure 3.514  T21 – Spiral Strains on the South Side during Push Cycles
Figure 3.515  T21 – Spiral Strains on the North Side during Pull Cycles

Figure 3.516  T21 – Tensile Strain-Displacement for Bar N3 during Push Cycles
Figure 3.517 T21 – Compressive Strain-Displacement for Bar N3 during Pull Cycles

Figure 3.518 T21 – Tensile Strain-Displacement for Bar S3 during Pull Cycles
Figure 3.519 T21 – Compressive Strain-Displacement for Bar S3 during Push Cycles

Figure 3.520 T21 – Compressive Strain-Displacement for Bar S3 during Push Cycles
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Figure 3.521 T21 – Extreme Fiber Bar N3 Strain Hysteresis to Buckling (7.27” Above)

Figure 3.522 T21 – Spiral Strain Hysteresis over North Buckled Region (7.22” Above)
Figure 3.523 T21 – Extreme Fiber Bar N3 Strain Hysteresis to Buckling (3.37” Above)

Figure 3.524 T21 – Spiral Strain Hysteresis over North Buckled Region (1.22” Above)
Figure 3.525 T21 – Extreme Fiber Bar S3 Strain Hysteresis to Buckling (3.02” Above)

Figure 3.526 T21 – Spiral Strain Hysteresis over South Buckled Region (3.09” Above)
Test 21 Aspect Ratio of 7.33 and 5% Axial Load – Curvature and Strain Penetration

The cross section curvature for each horizontal section above the footing is determined by connecting the strain measurements from all eight instrumented bars with a least squared error line. The curvature is then extracted from the slope of the least squared error line, see Figure 3.527 and Figure 3.528. The cross section curvature profiles in these figures are shown for the first horizontal section above the footing-column interface. This is because the target markers in the lowest gage lengths for bars near the middle of the column were not visible to the Optotrak position monitor. For these sections, it appears that the plane sections hypothesis is appropriate. Vertical curvature profiles are plotted for push and pull cycles in Figure 3.529 and Figure 3.530 respectively. These figures show that plastic curvatures have a linear distribution at higher displacement ductility levels. The extent of plastic curvatures above the footing can be calculated by determining where the linear plastic curvature distribution intersects the triangular yield curvature profile, shown as a grey dashed line. The dashed lines for each curvature distribution represent a least squared error linear fit to the plastic portion of the measured curvatures. The data points used to create the least squared error lines appear as circle data markers. The extent of plastic curvature vs. base curvature ductility is shown graphically in Figure 3.536.

The target marker on each bar placed closest the footing-column interface can be used to create slip hysteresis and horizontal slip profiles attributable to strain penetration. The slip hysteresis for bars N3 and S2 appear in Figure 3.531 and Figure 3.532 respectively. Bar S2 is shown since the lowest LED in extreme fiber bar S3 was obstructed by debris. If the tensile and compressive slip of all of the instrumented bars is plotted along the cross section depth, the base rotation attributable to strain penetration may be calculated. The slip profiles for push and pull cycles appear in Figure 3.533 and Figure 3.534 respectively. The rotation of the base section can be extracted from the slope of the least squared error line connecting all six measured bar slips.

Combining the curvatures over the instrumented region (5ft above the footing), bar slip profiles, and an elastic curvature assumption above the instrumented region, the top column
displacement can be calculated by integrating the curvature distributions and extrapolating the fixed-end rotations to the center of loading. The top column displacements calculated from the Optotrak system are compared to displacements measured with a string potentiometer at the center of loading in Figure 3.535. The Optotrak integrated displacements match well throughout the entire range of response, implying that the shear displacement component is small.

Figure 3.527  T21 – Sample Cross Section Strain Profiles during Push Cycles
Figure 3.528  T21 – Sample Cross Section Strain Profile during Pull Cycles

Figure 3.529  T21 – Curvature Profiles during Push Cycles with Plastic Regression
Figure 3.530  T21 – Curvature Profiles during Pull Cycles with Plastic Regression

Figure 3.531  T21 – Bond Slip Hysteresis for Bar N3 due to Strain Penetration
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Figure 3.532 T21 – Bond Slip Hysteresis for Bar S4 due to Strain Penetration

Figure 3.533 T21 – Base Rotation due to Strain Penetration during Push Cycles
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Figure 3.534  T21 – Base Rotation due to Strain Penetration during Pull Cycles

Figure 3.535  T21 – Comparison of Measured and Optotrak Integrated Displacements
Figure 3.536 T21 – Measured Spread of Plasticity (Circular Data Points)
### 3.3.4 Test 22 – Aspect Ratio of 7.33 and 10% Axial Load

**Table 3.39 Observations for Test 22 – Aspect Ratio of 7.33 and 10% Axial Load**

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<th>VALUES OF INTEREST:</th>
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<tr>
<td>Concrete Compressive Strength: ( f'_c = 6530 \text{ psi} )</td>
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<tr>
<td>Axial Load: ( P = 148 \text{ kips} ) ( P/(f'_c A_g) = 10% )</td>
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<tr>
<td>Column Length and Aspect Ratio: ( 11 \text{ ft} ) ( L/D = 7.33 )</td>
</tr>
<tr>
<td>Analytical First Yield Force: ( F'_y = 16.1 \text{ kips} )</td>
</tr>
<tr>
<td>Experimental First Yield Displacement: ( \Delta'_y = 1.59&quot; )</td>
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<tr>
<td>Analytical Nominal Moment Capacity: ( M_n = 233.3 \text{ kip} \ast \text{ ft} )</td>
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<tr>
<td>Equivalent Yield Displacement: ( \Delta_y = 2.09&quot; )</td>
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<td>Maximum Lateral Force: ( 21.9 \text{ kips} )</td>
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<tr>
<td>First Cracking North: ( 3/4Fy' = 0.90&quot; )</td>
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<tr>
<td>First Cracking South: ( -1/2Fy' = -0.52&quot; )</td>
</tr>
<tr>
<td>Cover Concrete Crushing North: ( \mu_3^{-1} = -6.27&quot; )</td>
</tr>
<tr>
<td>Cover Concrete Crushing South: ( \mu_3^{+1} = 6.26&quot; )</td>
</tr>
<tr>
<td>Transverse Steel Yield North: At (-6.07&quot;) during pull to ( \mu_3^{-1} = -6.27&quot; )</td>
</tr>
<tr>
<td>Transverse Steel Yield South: At (6.17&quot;) during push to ( \mu_3^{+1} = 6.26&quot; )</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North: Reversal from ( \mu_5^{+1} = 10.45&quot; )</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South: Reversal from ( \mu_6^{-1} = -12.53&quot; )</td>
</tr>
<tr>
<td>Longitudinal Bar Fracture North: At (4.81&quot;) during push to ( \mu_6^{+2} = 12.56&quot; )</td>
</tr>
<tr>
<td>Longitudinal Bar Fracture South: Test Concluded without Fracture of South Bars</td>
</tr>
</tbody>
</table>

\( \mu_6^{-1} = -12.53" \) represents the first pull cycle of displacement ductility six
Table 3.40 Strain Data Summary for Test 22 – Aspect Ratio of 7.33 and 10% Axial

**MATERIAL STRAINS:**

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Strain Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North</td>
<td>$\varepsilon_s = 0.0063 \text{ (compression)}$ $\mu_{\Delta 2} = -4.17''$</td>
</tr>
<tr>
<td>Cover Concrete Crushing South</td>
<td>$\varepsilon_s = 0.0085 \text{ (compression)}$ $\mu_{\Delta 2} = 4.17''$</td>
</tr>
<tr>
<td>Transverse Steel Yield North</td>
<td>$\varepsilon_s = 0.0103 \text{ (compression)}$</td>
</tr>
<tr>
<td>Transverse Steel Yield South</td>
<td>$\varepsilon_s = 0.0124 \text{ (compression)}$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North</td>
<td>$\varepsilon_s = 0.041 \text{ (peak tension prior to \text{bb})}$ $\varepsilon_s = 0.016 \text{ (peak comp. prior to \text{bb})}$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South</td>
<td>$\varepsilon_s = 0.053 \text{ (peak tension prior to \text{bb})}$ $\varepsilon_s = 0.035 \text{ (peak comp. prior to \text{bb})}$</td>
</tr>
</tbody>
</table>

Mander (1988) Ultimate Concrete Compression Strain, $\varepsilon_{cu} = 0.0201$

**Figure 3.537 T22 – Cross Section Bar Designation and Target Marker Application**
Figure 3.538  T22 – Symmetric Three Cycle Set Load History

Figure 3.539  T22 – Lateral Force vs. Displacement Response
Chapter 3: Experimental Observations

Figure 3.540  T22 – Compressive Axial Load from One Jack (Total = 2*Value)

Figure 3.541  T20 and T21 Hysteretic Comparison with Different Axial Load Levels
Test 22 Aspect Ratio of 7.33 and 10% Axial Load – Experimental Observations

The symmetric three-cycle-set laboratory load history is typically used to evaluate the seismic performance of structural components. The load history begins with elastic cycles to the following increments of the analytically predicted first yield force: $\frac{1}{4} F_y'$, $\frac{1}{2} F_y'$, $\frac{3}{4} F_y'$, and $F_y'$. The first yield force for the tested material and geometric properties was determined using moment-curvature analysis (Test 22: Cumbia $F_y' = 16.1$ kips with $f'_c = 6530$ psi). The first yield displacement was obtained as an average for the experimental first yield push and pull cycles ($\Delta'y = 1.59''$). Vertical strain profiles for both push and pull cycles up to the first yield force appear in Figure 3.542 with a dashed line representing the yield strain of the longitudinal reinforcement. The equivalent yield displacement, used to determine the displacement ductility levels ($\mu \Delta_1 = 1 * \Delta_y$), is then calculated as $\Delta_y = \Delta'_y (M_n/M'_y) = 2.09''$ for Test 22. The symmetric three-cycle-set load history resumes with three balanced cycles at each of the following displacement ductility levels: 1, 1.5, 2, 3, 4, 5, 6, 7, 8, etc. The full symmetric three-cycle-set load history appears in Figure 3.538, and the resulting lateral force versus top column displacement hysteresis is shown in Figure 3.539. The monotonic moment-curvature prediction does not include P-Δ effects.

![Figure 3.542](image_url)
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The test began with cycles in \( \frac{1}{4} F_y' \) (first yield force) increments in each direction of loading until the first yield force was reached. The first cracks on the North and South sides of the specimen were observed during \((3/4F_y' = 0.90")\) and \((-1/2F_y' = -0.52")\) respectively. The crack distribution on all sides of the specimen at first yield, \((F_y' = 1.48")\) and \((-F_y' = -1.69")\), appears in Figure 3.543. Similarly, the crack progression at displacement ductility 1, 1.5, and 2 appear in Figure 3.544, Figure 3.545, and Figure 3.546 respectively. During these cycles the cracks became more numerous and increased in inclination on the shear faces of the specimen. A small amount of core concrete crushed on the South side of the specimen during \(\mu_3^{+1} = 6.26"\), as shown in the left photo of Figure 3.547. A similar observation was made on the North side of the specimen during \(\mu_3^{-1} = -6.27"\), see the right photo of Figure 3.547. The crushing on each side of the specimen during ductility three was not severe, and it appeared that only a thin layer of concrete flaked off between spiral layers. Photos of the crack distributions during displacement ductility three and four appear in Figure 3.548 and Figure 3.549 respectively.

The North extreme fiber bar N3 visibly buckled under compressive stress during the reversal from \(\mu_5^{+1} = 10.45"\), as shown in Figure 3.550. At this time the buckled deformation was small. Additional outward deformation was observed in buckled bar N3 at \(\mu_5^{-2} = -10.45"\), see the left photo of Figure 3.551. The South reinforcing bars remained intact during ductility five, and additional crushing of the core concrete was observed up to 20" above the footing (right photo of Figure 3.551). A photo of the specimen at \(\mu_6^{+1} = 12.54"\) and additional deformation in previously buckled bar N3 during \(\mu_6^{-1} = -12.53"\) appears in Figure 3.552. The previously buckled North extreme fiber bar N3 fractured at 4.81” during the push cycle to \(\mu_6^{+2} = 10.45"\), Figure 3.553. Visible buckling of the South extreme fiber bar was observed 17” above the footing at \(\mu_6^{+2} = 10.45"\), as shown in the right photo of Figure 3.553. Buckling of adjacent North reinforcing bars N2 and N4 was observed during the final pull cycle to \(\mu_6^{-2} = -12.53"\). Photos of the specimen after the test are shown in Figure 3.554.
Figure 3.543 T22 – (Left) North at $F_y'$, (Mid) Front at $-F_y'$, and (Right) South at $-F_y'$

Figure 3.544 T22 – (Left) North at $\mu_1^{+3}$, (Mid) Front at $\mu_1^{-3}$, and (Right) South at $\mu_1^{-3}$
Figure 3.545 T22 – (Left) North at $\mu_{1.5}^{+3}$, (Mid) Front at $\mu_{1.5}^{-3}$, and (Right) South at $\mu_{1.5}^{-3}$

Figure 3.546 T22 – (Left) North at $\mu_{2}^{+3}$, (Mid) Front at $\mu_{2}^{-3}$, and (Right) South at $\mu_{2}^{-3}$
Figure 3.547 T22 – (Left) Concrete Crushing on the South Side at $\mu_3^{+1}$ and (Right) Concrete Crushing on the North Side at $\mu_3^{-1}$

Figure 3.548 T22 – (Left) North at $\mu_3^{+3}$, (Mid) Front at $\mu_3^{-3}$, and (Right) South at $\mu_3^{-3}$
Figure 3.549 T22 – (Left) North at $\mu_4^+3$, (Mid) Front at $\mu_4^-3$, and (Right) South at $\mu_4^-3$

Figure 3.550 T22 – Buckling of Bar N3 after Reversal from $(\mu_5^+1 = 10.45$"
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Figure 3.551 T22 – (Left) Deformation in Buckled Bar N3 at ($\mu_5^{-2} = -10.45''$) and (Right) South Side of the Specimen at ($\mu_5^{+3} = 10.46''$)

Figure 3.552 T22 – (Left) Specimen at ($\mu_6^{+1} = 12.54''$) and (Right) Additional Deformation in Buckled Bar N3 at ($\mu_6^{-1} = -12.53''$)
Figure 3.553 T22 – (Left) Fracture of Previously Buckled Bar N3 during $(\mu_6^{+2} = 12.56")$ and (Right) Buckling of Extreme Fiber Bar S3 at $(\mu_6^{+2} = 12.56")$

Figure 3.554 T22 – After the Test (Left) South Side and (Right) North Side
Test 22 Aspect Ratio of 7.33 and 10% Axial Load – Strain Data Analysis

North Reinforcement

Vertical strain profiles for the North extreme fiber bar N3 placed into tension during push cycles appear in the right half of Figure 3.555. This figure shows both extreme fiber bars on the same graph to illustrate the effects of tension shift on strain profiles. Compressive vertical strain profiles for North extreme fiber bar N3 during pull cycles appear in the left half of Figure 3.556.

A peak tensile strain of 0.041, located 6.73” above the footing, was measured for North extreme fiber bar N3 during \( (\mu_5^{+1} = 10.45") \). The largest tensile strain in bar N3 measured during cycles at displacement ductility four was 0.033 for the gage length 4.61” above the footing. The relationship between tensile strain and displacement for the gage length 6.73” above the footing appears in Figure 3.559. Each line represents a single push cycle which began with the column at zero displacement and ended at the peak of a continuous push cycle. The solid line contains data during the push cycle loading up to the peak displacement, and the dashed line represents the subsequent reversal of load. The monotonic moment-curvature prediction, with the PCK (2007) Lp Hinge Method, matches well until displacement ductility two, when it begins to over predict the measured tensile strains.

The largest compressive strain of -0.020, located 2.54” above the footing, was measured during \( (\mu_5^{-1} = -10.43") \). It is important to note that visible bar buckling occurred during this cycle, see Figure 3.550, but the outward deformation was small enough to barely affect the strains measured over the lowest gage lengths. This value is equal to the Mander ultimate concrete compressive strain of -0.020. The relationship between compressive strain and displacement for bar N3 appears in Figure 3.560, for the gage length centered 2.54” above the footing. The monotonic moment-curvature prediction with the PCK (2007) Lp Hinge Method matches well until ductility two, when the measured strains begin to exceed the prediction at an increasing rate. The effect of the compressive demand on the North side of the specimen on eight spiral layers near the footing-column interface is shown in Figure
3.558. Two layers of transverse steel entered the inelastic range during displacement ductility three, which coincides with the region where the moment-curvature analysis begins to underpredict the measured response. By displacement ductility four, four layers of transverse steel had entered the inelastic range. At \( \mu_5^{-1} = -10.43'' \), the peak spiral strain measured 0.022 for the layer 2.72'' above the footing at the location of slight visible bar buckling.

North extreme fiber bar N3 visibly buckled during the reversal from \( \mu_5^{+1} = 10.45'' \), as depicted in Figure 3.550. The previous peak compressive strain and tensile strain in the transverse steel measured over this region were -0.016 and 0.009 respectively. The strain hysteresis for the outward buckled region of bar N3 appear in Figure 3.564 and Figure 3.565 for gage lengths located 2.54'' and 4.61'' above the footing respectively. The gage length normally increases in the outward buckled region during the compressive cycle since the target markers are applied to the convex side of the longitudinal bar undergoing outward deformation. The effect of outward bar buckling on the transverse steel restraint is shown in Figure 3.566 for spiral layer 2.72'' above the footing. Outward deformation of the longitudinal bar caused the spiral strain to increase during the reversal from \( \mu_5^{+1} = 10.45'' \) when visible bar buckling was observed. Transverse steel in this region yielded during displacement ductility three, see Figure 3.558. Cycle to cycle increases in the transverse steel strain during displacement ductility three and four were observed. Presumably, this explains the cycle to cycle change in hysteretic loop shape for the longitudinal steel after spiral yielding. This implies that some level of measurable deformation took place before visible bar buckling.

South Reinforcement

Vertical strain profiles for the South extreme fiber bar S3 placed into tension during pull cycles appear in the right half of Figure 3.556. A peak tensile strain of 0.053, located 2.88'' above the footing, was measured in the South extreme fiber bar S3 during \( \mu_6^{-1} = -12.53'' \). The relationship between tensile strain and displacement for this gage length appears in
Figure 3.561. The moment-curvature prediction matches well until displacement ductility three, when the PCK (2007) Lp Hinge Method begins to over predict the measured tensile strains.

Compressive vertical strain profiles for the South extreme fiber bar S3 during push cycles appear in the left half of Figure 3.555. The region under high compressive demands extends higher up the column than previous tests. The visible effects of the compressive demand can be seen in the right photo of Figure 3.551. In this photo it is clear that crushing of the core concrete occurred over the regions of the column where the largest compressive demand was measured. A peak compressive strain of -0.035 was measured 16.74” above the footing on bar S3 during \(\mu_{6+1} = 12.54\)”. This exceeds the Mander ultimate concrete compressive strain of -0.020. A second peak compressive strain of -0.025 was measured 6.81” above the footing during the same cycle. The relationship between compressive strain and displacement on bar S3 for gage lengths 6.81” and 16.74” above the footing appear in Figure 3.562 and Figure 3.563 respectively. The relationship between compressive strain and displacement for the gage length 6.81” above the footing begins to exceed the moment-curvature prediction with the PCK (2007) Lp Hinge Method after displacement ductility two. Similarly, measured strains 16.74” above the footing are significantly under predicted after displacement ductility three. It is apparent that measurable deformation occurred over the gage length 16.74” above the footing, which aligns with the location of later visible bar buckling.

The effects of compressive demand on spiral strains in transverse steel layers overlaying bar S3 are shown in Figure 3.557. A single layer of transverse steel yielded during displacement ductility three, while three other layers had strains just below yield. The regions of heightened compressive demand shown in the left half of Figure 3.555 also produced the largest transverse steel strains. A peak transverse steel strain of 0.0124 was measured in the spiral layer 14.81” above the footing during \(\mu_{6+1} = 12.54\)”. It is apparent that there was a cycle to cycle increase in the transverse steel strains measured 14.81” above
the footing during displacement ductility five, indicating some level of measurable deformation.

A peak tensile strain of 0.053, located 2.88” above the footing, was measured in the South extreme fiber bar S3 during \( \mu_6^{-1} = -12.53” \). Visible bar buckling occurred during the following reversal, but not at the location of previous peak tensile strains. Instead, visible bar buckling occurred over the region of heightened compressive demand and large inelastic spiral strains approximately 14-17” above the footing, as shown in the right photo of Figure 3.553. The strain hysteresis for the outward buckled region of bar S3 appears in Figure 3.567 for the gage length 14.72” above the footing. The effect of outward visible bar buckling of spiral restraint strain hysteresis is shown in Figure 3.569 for the layer located 14.81” above the footing. Vertical strain profiles in the right half of Figure 3.556 and the strain hysteresis in Figure 3.567 both indicate that tensile demand over the region 14-17” above the footing is significantly lower than regions of the column closer to the footing-column interface. The strain hysteresis for the peak tensile gage length, located 6.81” above the footing (Figure 3.568), appears to be unaffected by visible bar buckling higher up the column. Even though bar buckling did not happen over the peak tensile gage length, it is important to note that tensile strains sustained during \( \mu_6^{-1} = -12.53” \) were required to initiate visible bar buckling during \( \mu_6^{+2} = 12.56” \). The measured spiral strains did not spike on the South side of the specimen until the initial compressive cycle of \( \mu_6^{+1} = 12.54” \). This implies that the previous compressive demand and multiple layers of inelastic transverse steel lowered the magnitude of the peak tensile strains required to initiate bar buckling during the following reversal of load.
Figure 3.555  T22 – Extreme Fiber Bar Vertical Strain Profiles during Push Cycles

Figure 3.556  T22 – Extreme Fiber Bar Vertical Strain Profiles during Pull Cycles
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Figure 3.557  T22 – Spiral Strains on the South Side during Push Cycles

Figure 3.558  T22 – Spiral Strains on the North Side during Pull Cycles
Chapter 3: Experimental Observations

Figure 3.559  T22 – Tensile Strain-Displacement for Bar N3 during Push Cycles

Figure 3.560  T22 – Compressive Strain-Displacement for Bar N3 during Pull Cycles
Figure 3.561 T22 – Tensile Strain-Displacement for Bar S3 during Pull Cycles

Figure 3.562 T22 – Compressive Strain-Displacement for Bar S3 during Push Cycles
Chapter 3: Experimental Observations

Figure 3.563 T22 – Compressive Strain-Displacement for Bar S3 during Push Cycles

Figure 3.564 T22 – Strain Hysteresis for Bar N3 to Buckling (2.54” above the Footing)
Figure 3.565  T22 – Strain Hysteresis for Bar N3 to Buckling (4.61” above the Footing)

Figure 3.566  T22 – Spiral Strain Hysteresis over North Bucked Region (2.72” Above)
Figure 3.567  T22 – Strain Hysteresis for Bar S3 to Buckling (14.72” above the Footing)

Figure 3.568  T22 – Strain Hysteresis for Bar S3 to Buckling (6.81” above the Footing)
Test 22 Aspect Ratio of 7.33 and 10% Axial Load – Curvature and Strain Penetration

The cross section curvature for each horizontal section above the footing is determined by connecting the strain measurements from all eight instrumented bars with a least squared error line. The curvature is then extracted from the slope of the least squared error line, see Figure 3.570 and Figure 3.571. The cross section curvature profiles in these figures are shown for the first horizontal section above the footing-column interface which had measurements for all six instrumented bars. This is because the target markers in the lowest gage lengths for bars near the middle of the column were not visible to the Optotrak position monitor. For these sections, it appears that the plane sections hypothesis is appropriate. Vertical curvature profiles are plotted for push and pull cycles in Figure 3.572 and Figure 3.573 respectively. These figures show that plastic curvatures have a linear distribution at higher displacement ductility levels. The extent of plastic curvatures above the footing can be calculated by determining where the linear plastic curvature distribution intersects the
triangular yield curvature profile, shown as a grey dashed line. The measured spread of plasticity is depicted by circular data points in Figure 3.579 as a function of base section curvature ductility.

The target marker on each bar placed closest the footing-column interface can be used to create bond slip hysteresis and horizontal bond slip profiles attributable to strain penetration. The bond slip hysteresis for bars N3 and S3 appear in Figure 3.574 and Figure 3.575 respectively. If the tensile and compressive bond slips of the instrumented bars are plotted along the cross section depth, the base rotation attributable to strain penetration may be calculated. The slip profiles for push and pull cycles appear in Figure 3.576 and Figure 3.577 respectively. The rotation of the base section can be extracted from the slope of the least squared error line connecting all of the measured bar slips.

Combining the curvatures over the instrumented region (5ft above the footing), bar slip profiles, and an elastic curvature assumption above the instrumented region, the top column displacement can be calculated by integrating the curvature distributions and extrapolating the fixed-end rotations to the center of loading. The top column displacements calculated from the Optotrak system are compared to displacements measured with a string potentiometer at the center of loading in Figure 3.578. The Optotrak integrated displacements match well throughout the entire range of response, implying that the shear displacement component is small.
Figure 3.570  T22 – Cross Section Curvatures during Push Cycles (6.78” above Footing)

Figure 3.571  T22 – Cross Section Curvatures during Pull Cycles (4.76” above Footing)
Figure 3.572  T22 – Push Cycle Curvature Profiles with Plastic Regression

Figure 3.573  T22 – Pull Cycle Curvature Profiles with Plastic Regression
Figure 3.574  T22 – Bar N3 Bond Slip Hysteresis due to Strain Penetration

Figure 3.575  T22 – Bar S3 Bond Slip Hysteresis due to Strain Penetration
Figure 3.576  T22 – Bond Slip Profiles during Push Cycles due to Strain Penetration

Figure 3.577  T22 – Bond Slip Profiles during Pull Cycles due to Strain Penetration
Figure 3.578  T22 – Comparison of Measured and Optotrak Integrated Displacements

Figure 3.579  T22 – Measured Spread of Plasticity (Circular Data Points)
### 3.3.5 Test 23 – Aspect Ratio of 8.67 and 5% Axial Load

**Table 3.41 Observations for Test 23 – Aspect Ratio of 8.67 and 5% Axial Load**

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<thead>
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<td>Concrete Compressive Strength:</td>
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<tr>
<td>Axial Load:</td>
<td>$P = 75 \text{ kips} \ (P/(f'_c A_g) = 5%)$</td>
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<table>
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<td>First Cracking North:</td>
<td>$1/2Fy' = 0.63&quot;$</td>
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<tr>
<td>First Cracking South:</td>
<td>$-1/2Fy' = -0.62&quot;$</td>
</tr>
<tr>
<td>Cover Concrete Crushing North:</td>
<td>$\mu_3^{-1} = -8.31&quot;$</td>
</tr>
<tr>
<td>Cover Concrete Crushing South:</td>
<td>$\mu_3^{+1} = 8.31&quot;$</td>
</tr>
<tr>
<td>Transverse Steel Yield North:</td>
<td>At $-9.08&quot;$ during pull to $\mu_4^{-1} = -11.12&quot;$</td>
</tr>
<tr>
<td>Transverse Steel Yield South:</td>
<td>At $10.83&quot;$ during push to $\mu_4^{+1} = 11.09&quot;$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North:</td>
<td>Reversal from $\mu_6^{+1} = 16.65&quot;$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South:</td>
<td>Reversal from $\mu_6^{-1} = -16.65&quot;$</td>
</tr>
<tr>
<td>Longitudinal Bar Fracture North:</td>
<td>At $5.50&quot;$ during push to $\mu_7^{+2} = 19.43&quot;$</td>
</tr>
<tr>
<td>Longitudinal Bar Fracture South:</td>
<td>Test Concluded without Fracture of South Bars</td>
</tr>
</tbody>
</table>

*$\mu_6^{-1} = -12.53"$ represents the first pull cycle of displacement ductility six*
Table 3.42 Strain Data Summary for Test 23 – Aspect Ratio of 8.67 and 5% Axial Load

MATERIAL STRAINS:

Cover Concrete Crushing North: \( \epsilon_s = 0.0052 \) (compression) \( \mu_{\Delta 2} = -5.56" \)

Cover Concrete Crushing South: \( \epsilon_s = 0.0062 \) (compression) \( \mu_{\Delta 2} = 5.54" \)

Transverse Steel Yield North: \( \epsilon_s = 0.0136 \) (compression)

Transverse Steel Yield South: \( \epsilon_s = 0.0151 \) (compression)

Longitudinal Bar Buckling North: \( \epsilon_s = 0.051 \) (peak tension prior to bb)

Longitudinal Bar Buckling South: \( \epsilon_s = 0.048 \) (peak tension prior to bb)

Mander (1988) Ultimate Concrete Compression Strain, \( \epsilon_{cu} = 0.0199 \)

Figure 3.580 T23 – Cross Section Bar Designation and Target Marker Application
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Figure 3.581 T23 – Symmetric Three Cycle Set Load History

Figure 3.582 T23 – Lateral Force vs. Displacement Response
Figure 3.583  T23 – Compressive Axial Load from One Jack (Total = 2*Value)

Figure 3.584  T23 – Test Setup for the Largest Aspect Ratio Specimens
Test 23 Aspect Ratio of 8.67 and 5% Axial Load – Experimental Observations

The symmetric three-cycle-set laboratory load history is typically used to evaluate the seismic performance of structural components. The load history begins with elastic cycles to the following increments of the analytically predicted first yield force: $\frac{1}{4} F_y'$, $\frac{1}{2} F_y'$, $\frac{3}{4} F_y'$, and $F_y'$. The first yield force for the tested material and geometric properties was determined using moment-curvature analysis (Test 23: Cumbia $F_y' = 11.55$ kips with $f'c = 6606$ psi). The first yield displacement was obtained as an average for the experimental first yield push and pull cycles ($\Delta y' = 2.01"$). Vertical strain profiles for both push and pull cycles up to the first yield force appear in Figure 3.585 with a dashed line representing the yield strain of the longitudinal reinforcement. The equivalent yield displacement, used to determine the displacement ductility levels ($\mu_{\Delta 1} = 1 \times \Delta y$), is then calculated as $\Delta y = \Delta y' (M_n/M_{y'}) = 2.78"$ for Test 23. The symmetric three-cycle-set load history resumes with three balanced cycles at each of the following displacement ductility levels: 1, 1.5, 2, 3, 4, 5, 6, 7, 8, etc. The full symmetric three-cycle-set load history appears in Figure 3.581, and the resulting lateral force versus top column displacement hysteresis is shown in Figure 3.582.

![Figure 3.585 T23 – Strain Profiles before Yield (Left) Bar N3 and (Right) Bar S3](image-url)
The test began with cycles in \( \frac{1}{4} F_y' \) (first yield force) increments in each direction of loading until the first yield force was reached. The first cracks on the North and South sides of the specimen were observed during \((1/2F_y' = 0.63")\) and \((-1/2F_y' = -0.62")\) respectively. The crack distribution on all sides of the specimen at first yield, \((F_y' = 2.00")\) and \((-F_y' = -2.02")\), appears in Figure 3.586. The crack progression at displacement ductility 1, 1.5, and 2 appear in Figure 3.587, Figure 3.588, and Figure 3.589 respectively. During these cycles the cracks became more numerous and increased in inclination on the shear faces of the specimen. A small amount of core concrete crushed on the South side of the specimen during \((\mu_3^+ = 8.31")\), Figure 3.590. A similar observation was made on the North side of the specimen during \((\mu_3^- = -8.31")\), Figure 3.590. The crushing on each side of the specimen during ductility three was not severe, and it appeared that only a thin layer of concrete flaked off between spiral layers. Photos of the crack distributions during displacement ductility 3, 4, and 5 appear in Figure 3.591, Figure 3.592, and Figure 3.593 respectively.

The North extreme fiber bar N3 visibly buckled after reversal from \((\mu_6^+ = 16.65")\), as shown in Figure 3.594. The South extreme fiber bar S3 showed signs of measurable deformation after reversal from \((\mu_6^- = -16.65")\), see Figure 3.595. A definitive visible buckling observation could not be made at this time because the outward deformation was small. Figure 3.596 depicts additional deformation in previously buckled bar N3 at \((\mu_6^2 = -16.66")\) and bar S3 at \((\mu_6^3 = 16.67")\). It is clear that visible buckling occurred over the same region (6.5" above the footing) where measurable deformation was observed in bar S3. A second outward buckled region was observed 10.5" above the footing on buckled bar S3. The buckled deformation in bars S3 and N3 became worse during \((\mu_7^+ = 19.43")\) and \((\mu_7^- = -19.44")\) respectively, see Figure 3.597. Adjacent bars N2 and N4 buckled during \((\mu_7^1 = -19.44")\). Previously buckled bar N3 ruptured at 5.50" during the push cycle to \(\mu_7^2\), Figure 3.598. The test was concluded after bar fracture occurred, and photos of the specimen after removal of the instrumentation appear in Figure 3.599.
Figure 3.586  T23 – (Left) North at $F_y'$, (Mid) Front at -$F_y'$, and (Right) South at -$F_y'$

Figure 3.587  T23 – (Left) North at $\mu_1^{+3}$, (Mid) Front at $\mu_1^{-3}$, and (Right) South at $\mu_1^{-3}$
Figure 3.588  T23 – (Left) North at $\mu_{1.5}^+3$, (Mid) Front at $\mu_{1.5}^-3$, and (Right) South at $\mu_{1.5}^-3$

Figure 3.589  T23 – (Left) North at $\mu_2^+3$, (Mid) Front at $\mu_2^-3$, and (Right) South at $\mu_2^-3$
Figure 3.590 T23 – (Left) Concrete Crushing on the South Side at $\mu_3^{+1}$ and (Right) Concrete Crushing on the North Side at $\mu_3^{-1}$

Figure 3.591 T23 – (Left) North at $\mu_3^{+3}$, (Mid) Front at $\mu_3^{-3}$, and (Right) South at $\mu_3^{-3}$
Figure 3.592 T23 – (Left) North at $\mu_4^+ 3$, (Mid) Front at $\mu_4^- 3$, and (Right) South at $\mu_4^- 3$

Figure 3.593 T23 – (Left) North at $\mu_5^+ 3$, (Mid) Front at $\mu_5^- 3$, and (Right) South at $\mu_5^- 3$
Figure 3.594 T23 – Buckling of Bar N3 after Reversal from ($\mu_6^{+1} = 16.65''$)

Figure 3.595 T23 – Measurable Deformation of Bar S3 during ($\mu_6^{+2} = 16.65''$)
Figure 3.596 T23 – (Left) Additional Deformation in Buckled Bar N3 at \( \mu_{6}^{-2} = -16.66'' \) and (Right) Additional Deformation in Buckled Bar S3 at \( \mu_{6}^{+3} = 16.67'' \)

Figure 3.597 T23 – (Left) Additional Deformation in Buckled Bar S3 at \( \mu_{7}^{+1} = 19.43'' \) and (Right) Additional Deformation in Buckled Bar N3 at \( \mu_{7}^{-1} = -19.44'' \)
Figure 3.598 T23 – Fracture of Previously Buckled Bar N3 at 5.50" during $\mu_7^{\pm2}$

Figure 3.599 T23 – After the Test (Left) South Side and (Right) North Side
Test 23 Aspect Ratio of 8.67 and 5% Axial Load – Strain Data Analysis

North Reinforcement

Vertical strain profiles for the North extreme fiber bar N3 placed into tension during push cycles appear in the right half of Figure 3.600. This figure shows both extreme fiber bars on the same graph to illustrate the effects of tension shift on strain profiles. Compressive vertical strain profiles for North extreme fiber bar N3 during pull cycles appear in the left half of Figure 3.601. A peak tensile strain of 0.0514, located 3.95” above the footing, was measured for North extreme fiber bar N3 during (μ₆⁺¹ = 16.65”). The bar visibly buckled under compressive stress during the following reversal of load. The relationship between tensile strain and displacement for this gage length appears in Figure 3.604. The solid line contains data during the push cycle loading up to the peak displacement, and the dashed line represents the subsequent reversal of load. The monotonic moment-curvature prediction with the PCK (2007) Lp Hinge Method matches well until displacement ductility three, when it begins to over predict the measured tensile strains.

A peak compressive strain of -0.0224 was measured 7.82” above the footing during (μ₅⁻² = −13.90”). This value exceeds the Mander ultimate concrete compressive strain of 0.0199. The relationship between compressive strain and displacement for this gage length appears in Figure 3.605. The measured compressive strains begin to exceed the moment-curvature prediction with the PCK (2007) Lp Hinge Method after displacement ductility two. Spiral strains measured in the first eight layers about the footing on the North side of the specimen appear in Figure 3.603 for compressive pull cycles. The spiral layer 5.69” above the footing entered the inelastic range at -9.08” during the pull cycle to (μ₄⁻¹ = −11.12”). A peak spiral strain of 0.0030 was measured over this spiral layer before visible bar buckling.

Strain hystereses for gage lengths 1.94” and 3.95” above the footing on bar N3 appear in Figure 3.608 and Figure 3.609. This region aligns with the outward buckling of bar N3 shown in Figure 3.594. Transverse steel strain hysteresis for spiral layers 1.75” and 3.75” above the footing are shown in Figure 3.610 and Figure 3.611. Bar N3 visibly buckled after
the reversal from \( \mu_6^+ = 16.65'' \). The outward bar buckling lead to a spike in the transverse steel restraint strains which were previously in the elastic range. It is important to note that a small amount of measurable deformation occurred during compressive cycles of displacement ductility five. Cycle-to-cycle increases in compressive demand in Figure 3.608 during displacement ductility five align with similar increases in spiral demands.

*South Reinforcement*

Vertical strain profiles for the South extreme fiber bar S3 placed into tension during pull cycles appear in the right half of Figure 3.601. A peak tensile strain of 0.0479, located 2.10” above the footing, was measured during \( \mu_6^- = -16.65'' \). The following reversal to \( \mu_6^+ = 16.68'' \) produced measurable deformation in the region of bar S3 which later visibly buckled. A tensile strain of 0.0506 was measured over the same gage length at \( \mu_6^- = -16.66'' \). The relationship between tensile strain and displacement for the gage length 2.10” appears in Figure 3.606. The moment-curvature prediction matches well until displacement ductility three, when the PCK (2007) Lp Hinge Method begins to over predict the measured tensile strains.

Compressive vertical strain profiles for the South extreme fiber bar S3 during push cycles appear in the left half of Figure 3.600. The effect of this compressive demand on spiral strains is shown in Figure 3.602. A peak compressive strain of -0.0305 was measured 5.93” above the footing at \( \mu_6^+ = 16.65'' \). At this displacement a peak spiral strain of 0.0072 was measured 5.93” above the footing. As previously mentioned, measurable deformation occurred during \( \mu_6^+ = 16.65'' \). The compressive strain 5.93” above the footing on bar S3 increased to -0.0406 during this cycle, and the spiral strains increased to 0.0083. The relationship between compressive strain and displacement for the gage length 5.93” above the footing appears in Figure 3.607. The measured compressive strains exceed the moment-curvature prediction with the PCK (2007) Lp Hinge Method after displacement ductility 1.5. Spiral yielding on the South side of the specimen occurred at 10.83” above the footing during the push cycle to \( \mu_4^+ = 11.09'' \).
The strain hysteresis for the outward buckled region of bar S3 appears in Figure 3.612 for the gage length 4.01” above the footing. A picture of the outward buckled region of bar S3 appears in the right photo of Figure 3.596. This strain hysteresis clearly shows that measurable deformation occurred after reversal from \( \mu_0^{-1} = -16.65" \), at the same location where the bar visibly buckled during \( \mu_0^{+3} = 16.67" \). A strain hysteresis for the gage length 5.93” above the footing appears in Figure 3.613. For this gage length, the measurable deformation resulted in additional compressive demand. The transverse steel strain hysteresis for spiral layers 5.78” and 7.75” above the footing appear in Figure 3.614 and Figure 3.615 respectively. The transverse steel layer 5.78” aligns with the lower buckled region of bar S3, while the spiral layer 7.75” overlaid the upper buckled region. The measurable deformation during \( \mu_0^{+2} = 16.68" \) lead to a spike in the measured spiral strains 5.78” above the footing, which caused the strain gage to become debonded. This same cycle only lead to a small increase in the spiral strains measured 7.75” above the footing. For this spiral layer, the large spike in transverse steel strain occurred during \( \mu_0^{+3} = 16.67" \) when visible buckling was observed.
Figure 3.600  T23 – Extreme Fiber Bar Vertical Strain Profiles during Push Cycles

Figure 3.601  T23 – Extreme Fiber Bar Vertical Strain Profiles during Pull Cycles
Chapter 3: Experimental Observations

Figure 3.602 T23 – South Spiral Strains during Compressive Push Cycles

Figure 3.603 T23 – North Spiral Strains during Compressive Pull Cycles
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Figure 3.604 T23 – Tensile Strain-Displacement for Bar N3 during Push Cycles

Figure 3.605 T23 – Compressive Strain-Displacement for Bar N3 during Pull Cycles
Chapter 3: Experimental Observations

Figure 3.606 T23 – Tensile Strain-Displacement for Bar S3 during Pull Cycles

Figure 3.607 T23 – Compressive Strain-Displacement for Bar S3 during Push Cycles
Chapter 3: Experimental Observations

Figure 3.608  T23 – Strain Hysteresis for Bar N3 to Buckling (1.94” Above the Footing)

Figure 3.609  T23 – Strain Hysteresis for Bar N3 to Buckling (3.95” Above the Footing)
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Figure 3.610 T23 – Spiral Strain Hysteresis over North Buckled Region (1.75” Above)

Figure 3.611 T23 – Spiral Strain Hysteresis over North Buckled Region (3.75” Above)
Figure 3.612  T23 – Strain Hysteresis for Bar S3 to Buckling (4.01” Above the Footing)

Figure 3.613  T23 – Strain Hysteresis for Bar S3 to Buckling (5.93” Above the Footing)
Chapter 3: Experimental Observations

Figure 3.614 T23 – Spiral Strain Hysteresis over South Buckled Region (5.78” Above)

Figure 3.615 T23 – Spiral Strain Hysteresis over South Buckled Region (7.75” Above)
Test 23 Aspect Ratio of 8.67 and 5% Axial Load – Curvature and Strain Penetration

The cross section curvature for each horizontal section above the footing is determined by connecting the strain measurements from all eight instrumented bars with a least squared error line. The curvature is then extracted from the slope of the least squared error line, see Figure 3.616 and Figure 3.617. The cross section curvature profiles in these figures are shown for the first horizontal section above the footing-column interface. For these sections, it appears that the plane sections hypothesis remains appropriate. Vertical curvature profiles are plotted for push and pull cycles in Figure 3.618 and Figure 3.619 respectively. These figures show that plastic curvatures have a linear distribution at higher displacement ductility levels. The extent of plastic curvatures above the footing can be calculated by determining where the linear plastic curvature distribution intersects the triangular yield curvature profile, shown as a grey dashed line. The measured spread of plasticity as a function of base section curvature ductility appears in Figure 3.625.

The target marker on each bar placed closest to the footing-column interface can be used to monitor strain penetration behavior. The bond slip hysteresis for bars N3 and S3 appear in Figure 3.620 and Figure 3.621 respectively. If the tensile and compressive bond slips of the instrumented bars are plotted along the cross section depth, the base rotation attributable to strain penetration may be calculated. The slip profiles for push and pull cycles appear in Figure 3.622 and Figure 3.623 respectively. The rotation of the base section can be extracted from the slope of the least squared error line connecting all of the measured bar slips.

Combining the curvatures over the instrumented region (5ft above the footing), bond slip profiles, and an elastic curvature assumption above the instrumented region, the top column displacement can be calculated by integrating the curvature distributions and extrapolating the fixed-end rotations to the center of loading. The top column displacements calculated from the Optotrak system are compared to displacements measured with a string potentiometer at the center of loading in Figure 3.624. The Optotrak integrated displacements slightly exceed the measured response in both directions of loading.
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**Figure 3.616** T23 – Cross Section Curvatures during Push Cycles (2.02” above Footing)

**Figure 3.617** T23 – Cross Section Curvatures during Pull Cycles (2.02” above Footing)
Figure 3.618  T23 – Push Cycle Curvature Profiles with Plastic Regression Lines

Figure 3.619  T23 – Pull Cycle Curvature Profiles with Plastic Regression Lines
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Figure 3.620  T23 – Bar N3 Bond Slip Hysteresis due to Strain Penetration

Figure 3.621  T23 – Bar S3 Bond Slip Hysteresis due to Strain Penetration
Figure 3.622 T23 – Bond Slip Profiles during Push Cycles due to Strain Penetration

Figure 3.623 T23 – Bond Slip Profiles during Pull Cycles due to Strain Penetration
Figure 3.624  T23 – Comparison of Measured and Optotrak Integrated Displacements

Figure 3.625  T23 – Measured Spread of Plasticity (Circular Data Points)
### 3.3.6 Test 24 – Aspect Ratio of 8.67 and 10% Axial Load

**Table 3.43 Observations for Test 24 – Aspect Ratio of 8.67 and 10% Axial Load**

<table>
<thead>
<tr>
<th>VALUES OF INTEREST:</th>
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<tr>
<td>Concrete Compressive Strength:</td>
<td>( f'_c = 6473 \text{ psi} )</td>
</tr>
<tr>
<td>Axial Load:</td>
<td>( P = 147 \text{ kips} \ (P/(f'_c A_g) = 10%) )</td>
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<tr>
<td>Column Length and Aspect Ratio:</td>
<td>( 13 \text{ ft} \ (L/D = 8.67) )</td>
</tr>
<tr>
<td>Analytical First Yield Force:</td>
<td>( F'_y = 13.58 \text{ kips} )</td>
</tr>
<tr>
<td>Experimental First Yield Displacement:</td>
<td>( \Delta'_y = 2.16&quot; )</td>
</tr>
<tr>
<td>Analytical Nominal Moment Capacity:</td>
<td>( M_n = 233.7 \text{ kip ft} )</td>
</tr>
<tr>
<td>Equivalent Yield Displacement:</td>
<td>( \Delta_y = 2.86&quot; )</td>
</tr>
<tr>
<td>Maximum Lateral Force:</td>
<td>( 17.92 \text{ kips} )</td>
</tr>
</tbody>
</table>

**DAMAGE OBSERVATIONS:**

- First Cracking North: \( 1/2Fy' = 0.63" \)
- First Cracking South: \( -1/2Fy' = -0.69" \)
- Cover Concrete Crushing North: \( \mu^2_2 = -5.72" \)
- Cover Concrete Crushing South: \( \mu^+3_1 = 8.58" \)
- Transverse Steel Yield North: At \(-6.60" \) during pull to \( \mu^{-1}_3 = -8.58" \)
- Transverse Steel Yield South: At \(7.34" \) during push to \( \mu^{+1}_3 = 8.58" \)
- Longitudinal Bar Buckling North: Reversal from \( \mu^{+1}_5 = 14.29" \)
- Longitudinal Bar Buckling South: Reversal from \( \mu^{-1}_5 = -14.31" \)
- Longitudinal Bar Fracture North: At \(10.49" \) during push to \( \mu^{+4}_5 = 14.33" \)
- Longitudinal Bar Fracture South: At \(-7.71" \) during pull to \( \mu^{-6}_5 = -14.33" \)

\( \mu^{\pm 1}_5 = -14.31" \) represents the first pull cycle of displacement ductility five
Table 3.44 Strain Data Summary for Test 24 – Aspect Ratio 8.67 and 10% Axial Load

**MATERIAL STRAINS:**

<table>
<thead>
<tr>
<th>Component</th>
<th>Strain</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North</td>
<td>( \varepsilon_s = 0.0085 ) (compression)</td>
<td></td>
</tr>
<tr>
<td>Cover Concrete Crushing South</td>
<td>( \varepsilon_s = 0.0083 ) (compression) *( \mu_{\Delta_2} = 5.73)&quot;</td>
<td></td>
</tr>
<tr>
<td>Transverse Steel Yield North</td>
<td>( \varepsilon_s = 0.0155 ) (compression)</td>
<td></td>
</tr>
<tr>
<td>Transverse Steel Yield South</td>
<td>( \varepsilon_s = 0.0131 ) (compression)</td>
<td></td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North</td>
<td>( \varepsilon_s = 0.037 ) (peak tension prior to ( bb ))</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( \varepsilon_s = 0.028 ) (peak comp. prior to ( bb ))</td>
<td></td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South</td>
<td>( \varepsilon_s = 0.045 ) (peak tension prior to ( bb ))</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( \varepsilon_s = 0.020 ) (peak comp. prior to ( bb ))</td>
<td></td>
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<tr>
<td>Mander (1988) Ultimate Concrete Compression Strain, ( \varepsilon_{cu} = 0.0202 )</td>
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Figure 3.626 T24 – Cross Section Bar Designation and Target Marker Application
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Figure 3.627  T24 – Symmetric Three Cycle Set Load History

Figure 3.628  T24 – Lateral Force vs. Displacement Response
Figure 3.629  T24 – Compressive Axial Load from One Jack (Total = 2*Value)

Figure 3.630  Hysteretic Comparison of T23 and T24 with Different Axial Load Levels
Test 24 Aspect Ratio of 8.67 and 10% Axial Load – Experimental Observations

The symmetric three-cycle-set laboratory load history is typically used to evaluate the seismic performance of structural components. The load history begins with elastic cycles to the following increments of the analytically predicted first yield force: \( \frac{1}{4} \) \( F_y' \), \( \frac{1}{2} \) \( F_y' \), \( \frac{3}{4} \) \( F_y' \), and \( F_y' \). The first yield force for the tested material and geometric properties was determined using moment-curvature analysis (Test 24: Cumbia \( F_y' = 13.58 \text{ kips} \) with \( f'_c = 6473 \text{ psi} \)). The first yield displacement was obtained as an average for the experimental first yield push and pull cycles \( (\Delta_{\gamma} = 2.16\text{"}) \). Vertical strain profiles for both push and pull cycles up to the first yield force appear in Figure 3.631 with a dashed line representing the yield strain of the longitudinal reinforcement. The equivalent yield displacement, used to determine the displacement ductility levels \( (\mu_{\Delta 1} = 1 \ast \Delta_{\gamma}) \), is then calculated as \( \Delta_{\gamma} = \Delta_{\gamma}' \left( \frac{M_y}{M_y'} \right) = 2.86\text{"} \) for Test 24. The symmetric three-cycle-set load history resumes with three balanced cycles at each of the following displacement ductility levels: 1, 1.5, 2, 3, 4, 5, 6, 7, 8, etc. The full symmetric three-cycle-set load history appears in Figure 3.627, and the resulting lateral force versus top column displacement hysteresis is shown in Figure 3.628.

![Figure 3.631 T24 – Strain Profiles before Yield, (Left) North and (Right) South](image-url)
The test began with cycles in \( \frac{1}{4} F_{y}' \) (first yield force) increments in each direction of loading until the first yield force was reached. The first cracks on the North and South sides of the specimen were observed during \( (1/2F_{y}' = 0.63") \) and \( (-1/2F_{y}' = -0.69") \) respectively. The crack distribution on all sides of the specimen at first yield, \( (F_{y}' = 2.10") \) and \( (-F_{y}' = -2.23") \), appears in Figure 3.632. The crack progression at displacement ductility 1, 1.5, and 2 appear in Figure 3.633, Figure 3.634, and Figure 3.637 respectively. During these cycles the cracks became more numerous and increased in inclination on the shear faces of the specimen.

The first signs of visible flaking of the core concrete, which precedes crushing, occurred on the South side at \( (\mu_2^+ = 5.71") \) and the North side at \( (\mu_2^- = -5.71") \) as shown in Figure 3.635. A small amount of core concrete crushed on the North side of the specimen during the following pull cycle to \( (\mu_2^- = -5.72") \), as shown in the left photo of Figure 3.636. A similar observation was made on the South side of the specimen during \( (\mu_3^+ = 8.58") \). A photo of crushing on the South side of the specimen appears in the right portion of Figure 3.636, a better quality picture is not available. The crushing on each side of the specimen during ductility three was not severe, and it appeared that only a thin layer of concrete flaked off between spiral layers. The crack progression during displacement ductility three and four are shown in Figure 3.638 and Figure 3.639 respectively.

Progressively larger spiral strains were observed on the North side of the specimen during displacement ductility four. In previous tests, this occurred when the extreme fiber longitudinal bar measurably deformed before visible bar buckling. A photo of the North extreme fiber bar N3 at \( (\mu_4^- = -11.44") \), in left side of Figure 3.640, shows that visible buckling had not yet occurred over the regions of large inelastic spiral strains. Instead, visible bar buckling of bar N3 occurred under compressive stress during the reversal from \( (\mu_5^+ = 14.29") \), as shown in the right photo of Figure 3.640. Similarly, the South extreme fiber bar S3 buckled during the reversal from \( (\mu_5^- = -14.32") \), as shown in the left photo of Figure 3.641. The first observation of buckling on each side of the specimen occurred when the outward deformation of the bar was still small.
Additional deformation in the previously buckled bar N3 was observed during the subsequent pull cycle to \( \mu_5^{−2} = −14.33" \) in Figure 3.642. Similarly, additional deformation in buckled bar S3 was observed during \( \mu_5^{+3} = 14.33" \), see the right photo of Figure 3.641. The outward deformation in buckled bar N3 continued to increase during \( \mu_5^{−3} = −14.33" \), and the bar fractured in tension during the subsequent push cycle to \( \mu_5^{+4} = 14.33" \), as shown in Figure 3.643. Fracture of bar N3 lead to a 24% loss in strength measured at \( \mu_5^{+4} = 14.33" \) relative to the peak lateral force measured in the push direction of loading. Adjacent North reinforcing bar N4 buckled during \( \mu_5^{+4} = −14.32" \), see the left photo of Figure 3.644.

The deformation in previously buckled bar S3 became even more severe during \( \mu_5^{+6} = 14.34" \) as shown in the right photo of Figure 3.644. Bar S3 fractured in tension during the subsequent reversal to \( \mu_5^{−6} = −14.33" \), see Figure 3.645. Fracture of bar S3 lead to a 39% loss in strength measured at \( \mu_5^{−6} = −14.33" \) relative to the peak lateral force measured in the pull direction of loading. At this time the test was concluded with two buckled bars and one fractured bar on the North side of the specimen and one buckled and fractured bar on the South side of the specimen. Photos of all sides of the specimen after removal of the instrumentation appear in Figure 3.646.
Figure 3.632  T24 – (Left) North at $F_y'$, (Mid) Front at $-F_y'$, and (Right) South at $-F_y'$

Figure 3.633  T24 – (Left) North at $\mu_1^+$, (Mid) Front at $\mu_1^-$, and (Right) South at $\mu_1^-$
Figure 3.634 T24 – (Left) North at $\mu_{1.5}^+3$, (Mid) Front at $\mu_{1.5}^-3$, and (Right) South at $\mu_{1.5}^-3$

Figure 3.635 T24 – Visible Flaking of Cover Concrete which Precedes Crushing (Left) South Side during $\mu_2^+1$ and (Right) North Side during $\mu_2^-1$
Figure 3.636 T24 – Concrete Crushing on (Left) the North Side during $\mu_{2}^{-2}$ and (Right) the South Side during $\mu_{3}^{+1}$

Figure 3.637 T24 – (Left) North at $\mu_{2}^{+3}$, (Mid) Front at $\mu_{2}^{-3}$, and (Right) South at $\mu_{2}^{-3}$
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Figure 3.638 T24 – (Left) North at $\mu_3^+3$, (Mid) Front at $\mu_3^-3$, and (Right) South at $\mu_3^-3$

Figure 3.639 T24 – (Left) North at $\mu_4^+3$, (Mid) Front at $\mu_4^-3$, and (Right) South at $\mu_4^-3$
Figure 3.640 T24 – (Left) North Side during $\mu_4^{-3}$ without Visible Bar Buckling and (Right) Visible Bar Buckling of N3 during $\mu_5^{-1}$

Figure 3.641 T24 – (Left) Visible Bar Buckling of S3 during $\mu_5^{+2}$ and (Right) Additional Deformation in Buckled Bar S3 during $\mu_5^{+3}$
Figure 3.642 T24 – Additional Deformation in Previously Buckled Bar N3 during $\mu_5^{-2}$

Figure 3.643 T24 – (Left) Additional Deformation in Buckled Bar N3 during $\mu_5^{-3}$ and (Right) Fracture of Previously Buckled Bar N3 during $\mu_5^{+4}$
Figure 3.644 T24 – (Left) Buckling of Adjacent Bar N4 during $\mu_5^{-4}$ and (Right) Additional Deformation in Bar S3 during $\mu_5^{+6}$

Figure 3.645 T24 – Fracture of Previously Buckled Bar S3 during $\mu_5^{-6}$
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Test 24 Aspect Ratio of 8.67 and 10% Axial Load – Strain Data Analysis

North Reinforcement

Vertical strain profiles for the North extreme fiber bar N3 placed into tension during push cycles appear in the right half of Figure 3.647. This figure shows both extreme fiber bars on the same graph to illustrate the effects of tension shift on strain profiles. Compressive vertical strain profiles for North extreme fiber bar N3 during pull cycles appear in the left half of Figure 3.648. A peak tensile strain of 0.0365, located 11.13” above the footing, was measured for North extreme fiber bar N3 during $(\mu_5^+ = 14.29”)$. A tensile strain of 0.0359 was measured 4.98” above the footing during the same cycle. Extreme fiber bar N3 visibly buckled under compressive stress during the following reversal of load to $(\mu_5^- = -14.32”)$. The relationship between tensile strain and displacement for the gage length 4.98” above the footing appears in Figure 3.651. The moment-curvature prediction
with the PCK (2007) Lp Hinge Method begins to overestimate the measured tensile strains after displacement ductility three at an increasing rate. Each line represents a single push cycle which began with the column at zero displacement and ended at the peak of a continuous push cycle. The solid line contains data during the push cycle loading up to the peak displacement, and the dashed line represents the subsequent reversal of load.

A peak compressive strain of -0.0283 was measured 2.97” above the footing during $(\mu_4^3 = -11.44")$. This value exceeds the Mander ultimate concrete compressive strain of 0.0202. A second region of large compressive demand produced a strain of -0.0219 for the gage length 7.07” above the footing. The relationship between compressive strain and displacement for the gage length 2.97” above the footing appears in Figure 3.652. The measured compressive strains begin to exceed the moment-curvature prediction with the PCK (2007) Lp Hinge Method after displacement ductility 1.5. Each cycle of displacement ductility four was shown to highlight measurable deformation of bar N3.

The effects of compression on eight spiral layers on the North side of the specimen appear in Figure 3.650. The first spiral layer on the North side of the specimen entered the inelastic range at $-6.60"$ during pull cycle to $(\mu_3^{-1} = -8.58")$. Measureable deformation during displacement ductility four lead to an increase in the spiral strains measured 5.41” above the footing from 0.0074, 0.0124, to 0.0189 during consecutive cycles of displacement ductility four. This measurable deformation aligns with the region of outward visible bar buckling shown in Figure 3.642. Gage lengths above and below the outward deformed region of bar N3 would have apparent larger compressive strains since the target markers are located on the concave side of the buckled bar. Alternatively, the gage lengths located on the convex side of the outward buckled bar increase under compressive demand.

Strain hysteresis for extreme fiber bar N3 up to visible bar buckling are shown in Figure 3.655, Figure 3.656, and Figure 3.657 for gage lengths 2.97”, 4.98” and 7.07” above the footing respectively. The gage length 4.98” represents the outward buckled region of bar N3, and aligns with the largest spiral strains measured 5.41” above the footing. The gage lengths 2.97” and 7.07” above the footing appear above and below the outward buckled region.
These gage lengths had significant measurable deformation during successive pull cycles of displacement ductility four. Transverse steel strain hysteresis for spiral layers 5.41” and 7.34” above the footing appear in Figure 3.658 and Figure 3.659 respectively. It is clear that the measurable deformation lead to significant increases in the spiral strains measured during displacement ductility four. The strain gages on each spiral layer debonded, preventing further measurement, during the pull cycle to \( (\mu_5^{-1} = -14.32") \) when outward visible bar buckling was observed for bar N3.

**South Reinforcement**

Vertical strain profiles for the South extreme fiber bar S3 placed into tension during pull cycles appear in the right half of Figure 3.648. A peak tensile strain of 0.0454, located 5.19” above the footing, was measured during \( (\mu_5^{-1} = -14.32") \). The South extreme fiber bar S3 buckled under compressive stress during the following reversal to \( (\mu_5^{+2} = 14.33") \). The relationship between tensile strain and displacement for the gage length 5.19” above the footing appears in Figure 3.653. For this gage length, the moment-curvature prediction with the PCK (2007) Lp Hinge Method only slightly overestimates the measured tensile strains after displacement ductility three.

Compressive vertical strain profiles for bar S3 appear in the left half of Figure 3.647. A peak compressive strain of -0.02044 was measured 1.30” above the footing during \( (\mu_5^{+1} = 14.29") \). Regions of compressive demand 5.19” and 9.29” above the footing produced compressive strains of -0.0176 and -0.0187 respectively. The relationship between compressive strain and displacement for gage length 9.29” above the footing appears in Figure 3.654. The measured compressive strains are underestimated by moment-curvature analysis with the PCK (2007) Lp Hinge Method after displacement ductility 1.5. Measured spiral strains the lowest eight South spiral layers appear in Figure 3.649. A peak spiral strain of 0.0100 was measured 3.06” above the footing during \( (\mu_5^{+1} = 14.29") \). Over this same gage length, spiral strains increased from 0.0042, 0.0049, to 0.0057 during successive
compressive cycles of displacement ductility four. Outward visible bar buckling was observed over the gage length 3.14” above the footing as shown in Figure 3.641.

Longitudinal steel strain hysteresis for gage lengths 3.14” and 5.19” above the footing on bar S3 appear in Figure 3.660 and Figure 3.661 respectively. The gage length 3.14” above the footing aligns with the outward buckled region of bar S3, in Figure 3.641, and the largest spiral strains measured 3.06” above the footing. Transverse steel strain hysteresis for spiral layers 3.06” and 5.15” above the footing appear in Figure 3.662 and Figure 3.663 respectively. Visible buckling of bar S3 altered the strain and displacement relationship for the gage length 3.14” above the footing and caused a spike in measured spiral strains 3.06” and 5.15” above the footing. The spiral stain hystereses indicate a small cycle-to-cycle deformation during successive compressive cycles of displacement ductility four.

![T24 – Extreme Fiber Bar Vertical Strain Profiles during Push Cycles](image-url)
Figure 3.648  T24 – Extreme Fiber Bar Vertical Strain Profiles during Pull Cycles

Figure 3.649  T24 – Spiral Strains on the South Side during Push Cycles
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Figure 3.650  T24 – Spiral Strains on the North Side during Pull Cycles

Figure 3.651  T24 – Tensile Strain-Displacement for Bar N3 during Push Cycles
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Figure 3.652 T24 – Compressive Strain-Displacement for Bar N3 during Pull Cycles

Figure 3.653 T24 – Tensile Strain-Displacement for Bar S3 during Pull Cycles
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Figure 3.654 T24 – Compressive Strain-Displacement for Bar S3 during Push Cycles

Figure 3.655 T24 – Strain Hysteresis for Bar N3 to Buckling (2.97” Above the Footing)
Figure 3.656  T24 – Strain Hysteresis for Bar N3 to Buckling (4.98” Above the Footing)

Figure 3.657  T24 – Strain Hysteresis for Bar N3 to Buckling (7.07” Above the Footing)
Figure 3.658  T24 – Spiral Strain Hysteresis over North Buckled Region (5.41” Above)

Figure 3.659  T24 – Spiral Strain Hysteresis over North Buckled Region (7.34” Above)
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Figure 3.660  T24 – Strain Hysteresis for Bar S3 to Buckling (3.14” Above the Footing)

Figure 3.661  T24 – Strain Hysteresis for Bar S3 to Buckling (5.19” Above the Footing)
Figure 3.662  T24 – Spiral Strain Hysteresis over South Buckled Region (3.06” Above)

Figure 3.663  T24 – Spiral Strain Hysteresis over South Buckled Region (5.16” Above)
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**Test 24 Aspect Ratio of 8.67 and 10% Axial Load – Curvature and Strain Penetration**

The cross section curvature for each horizontal section above the footing is determined by connecting the strain measurements from all eight instrumented bars with a least squared error line. The curvature is then extracted from the slope of the least squared error line, see Figure 3.664 and Figure 3.665. The cross section curvature profiles in these figures are shown for the third horizontal section above the footing-column interface. For these sections, it appears that the plane sections hypothesis remains appropriate. Vertical curvature profiles are plotted for push and pull cycles in Figure 3.666 and Figure 3.667 respectively. These figures show that plastic curvatures have a linear distribution at higher displacement ductility levels. The extent of plastic curvatures above the footing can be calculated by determining where the linear plastic curvature distribution intersects the triangular yield curvature profile, shown as a grey dashed line. The dashed lines for each curvature distribution represent a least squared error linear fit to the plastic portion of the measured curvatures. The measured spread of plasticity as a function of abase section curvature ductility appears on Figure 3.673.

The target marker on each bar placed closest the footing-column interface can be used to create bond slip hysteresis and horizontal bond slip profiles attributable to strain penetration of reinforcement into the footing. The bond slip hysteresis for bars N4 and S3 appear in Figure 3.668 and Figure 3.669 respectively. The lowest target marker on extreme fiber bar N3 was obstructed by wires during the test preventing bond slip and strain measurements for the lowest gage length. The first spiral layer was located close to the footing-column interface, therefore there was not enough room to place LEDs on many of the other reinforcing bars. If the lowest LEDs on these bars were used they would incorrectly include a large portion of the flexural strains measured between the footing and the next LED above the spiral layer. For this reason, the bond slip profiles for the base section are only measured in terms of bars S3 and N4 in Figure 3.670 and Figure 3.671. The rotation of the base section can is extracted from the slope of the least squared error line for the measured data.
Combining the curvatures over the instrumented region (5ft above the footing), bond slip profiles, and an elastic curvature assumption above the instrumented region, the top column displacement can be calculated by integrating the curvature distributions and extrapolating the fixed-end rotations to the center of loading. The top column displacements calculated from the Optotrak system are compared to displacements measured with a string potentiometer at the center of loading in Figure 3.672. The Optotrak integrated displacements exceed the measured response in the push direction of loading. The calculations have been verified, and it is likely that the strain penetration rotations calculated using only two reinforcing bars contribute to this error. Additionally, any errors in curvatures or base rotations are amplified by the long moment arm to the center of loading for this column with a high aspect ratio.

Figure 3.664 T24 – Cross Section Curvatures during Push Cycles (4.97” Above)
Figure 3.665  T24 – Cross Section Curvatures during Pull Cycles (4.97” Above)

Figure 3.666  T24 – Push Cycle Curvature Profiles with Plastic Regression
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Figure 3.667  T24 – Pull Cycle Curvature Profiles with Plastic Regression

Figure 3.668  T24 – Bar N4 Bond Slip Hysteresis due to Strain Penetration
Figure 3.669  T24 – Bar S3 Bond Slip Hysteresis due to Strain Penetration

Figure 3.670  T24 – Base Rotation due to Strain Penetration during Push Cycles
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Figure 3.671 T24 – Base Rotation due to Strain Penetration during Pull Cycles

Figure 3.672 T24 – Comparison of Measured and Optotrak Integrated Displacements
Figure 3.673  T24 – Measured Spread of Plasticity (Circular Data Points)
3.4 Steel Content and Axial Load Variable Tests 25-30

The effects of longitudinal steel content and higher levels of axial load ratio on column performance were the main variables for Tests 25-30. The test matrix for the eight columns is shown in Table 3.45, and the material properties of the reinforcement appear in Table 3.46. Similar 18” and 24” column configurations were used so that the results could be compared to previous experiments with either different axial load or longitudinal steel content. The shear span for all six cantilever columns was 8ft (244cm). Stress-strain curves for the longitudinal and transverse steel appear in Figure 3.674 and Figure 3.675. The test series used the full cover concrete blockout method with target markers applied to both longitudinal and transverse steel, Figure 3.677.

The 18” (457mm) diameter bridge columns, Figure 3.676, contained either 10 #6 ($A_{st}/A_g = 1.7\%$) or 10 #8 ($A_{st}/A_g = 3.1\%$) A706 bars for longitudinal reinforcement and a #3 (9.5mm) A706 spiral at 2” spacing ($4A_{sp}/D's = 1.3\%$). The 24” (610mm) diameter bridge columns, Figure 3.676, contained either 16 #6 ($A_{st}/A_g = 1.6\%$) or 16 #7 ($A_{st}/A_g = 2.1\%$) A706 bars for longitudinal reinforcement and a #3 (9.5mm) A706 spiral at 2” spacing ($4A_{sp}/D's = 1\%$). Previous 18” diameter specimens were subjected to 5% and 10% axial load. Two specimens, with nominally identical geometry and reinforcement, are subjected to 15% and 20% axial load. Previous 24” diameter specimens utilized approximately 5% axial load, this same test configuration is subjected to 10% axial load here. In addition, the combination of higher steel content and different levels of axial load is investigated.
### Table 3.45 Column Summary for Steel Content and Axial Load Variable Tests 25-30

<table>
<thead>
<tr>
<th>Test</th>
<th>Load History</th>
<th>D (in)</th>
<th>L/D</th>
<th>Long. Steel ($\rho_l$)</th>
<th>Spiral Detailing ($\rho_s$)</th>
<th>$f'_c$ (psi)</th>
<th>$P/f'_c*Ag$</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>Three Cycle Set</td>
<td>24</td>
<td>4</td>
<td>16 #7 bars (2.1%)</td>
<td>#3 at 2” (1%)</td>
<td>6289</td>
<td>5%</td>
</tr>
<tr>
<td>26</td>
<td>Three Cycle Set</td>
<td>24</td>
<td>4</td>
<td>16 #7 bars (2.1%)</td>
<td>#3 at 2” (1%)</td>
<td>5890</td>
<td>10%</td>
</tr>
<tr>
<td>27</td>
<td>Three Cycle Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>6149</td>
<td>10%</td>
</tr>
<tr>
<td>28</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2” (1.3%)</td>
<td>6239</td>
<td>15%</td>
</tr>
<tr>
<td>29</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2” (1.3%)</td>
<td>5912</td>
<td>20%</td>
</tr>
<tr>
<td>30</td>
<td>Three Cycle Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #8 bars (3.1%)</td>
<td>#3 at 2” (1.3%)</td>
<td>6050</td>
<td>15%</td>
</tr>
</tbody>
</table>

### Table 3.46 Reinforcement Material Property Summary for Columns 25-30

<table>
<thead>
<tr>
<th>Longitudinal Reinforcement</th>
<th>$\varepsilon_y$</th>
<th>$f_y$ (ksi)</th>
<th>$\varepsilon_h$</th>
<th>$f_h$ (ksi)</th>
<th>$\varepsilon_u$</th>
<th>$f_u$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tests 25-30 (#6 Bar)</td>
<td>0.00237</td>
<td>68.7</td>
<td>0.01363</td>
<td>68.8</td>
<td>0.11781</td>
<td>93.7</td>
</tr>
<tr>
<td>Tests 25-30 (#7 Bar)</td>
<td>0.00240</td>
<td>69.7</td>
<td>0.01261</td>
<td>69.7</td>
<td>0.11440</td>
<td>95.5</td>
</tr>
<tr>
<td>Tests 25-30 (#8 Bar)</td>
<td>0.00243</td>
<td>70.5</td>
<td>0.01095</td>
<td>70.5</td>
<td>0.10929</td>
<td>97.7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Transverse Steel</th>
<th>$\varepsilon_y$ (0.2% offset)</th>
<th>$f_y$ (ksi)</th>
<th>$\varepsilon_u$</th>
<th>$f_u$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tests 25-30 (#3 Spiral)</td>
<td>0.00428</td>
<td>63.9</td>
<td>0.11313</td>
<td>95.2</td>
</tr>
</tbody>
</table>
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Figure 3.674 Test 25-30 – Longitudinal Steel Tensile Test Results

Figure 3.675 Test 25-30 – Transverse Steel Tensile Test Results
Figure 3.676 Tests 25-30 Cross Section and Bar Designation for Both Diameters

Figure 3.677 Complete Cover Concrete Blockout with Direct Application of Target Markers to Longitudinal and Transverse Steel
3.4.1 Test 25 – 24” Dia. Column with 2.1% Long. Steel and 5% Axial Load

Table 3.47 Observations for Test 25 – 24” Dia. With 2.1% Steel and 5% Axial Load

VALUES OF INTEREST:

Concrete Compressive Strength: \( f'_c = 6287 \) psi
Axial Load: \( P = 131 \) kips \( (P/(f'_cA_g) = 5\%) \)
Longitudinal Steel Content: \( 16 \# 7 \) Bars \( (A_{st}/A_g = 2.1\%) \)
Analytical First Yield Force: \( F'_y = 52.9 \) kips
Experimental First Yield Displacement: \( \Delta'_y = 0.74" \)
Analytical Nominal Moment Capacity: \( M_n = 584.2 \) kip * ft
Equivalent Yield Displacement: \( \Delta_y = 1.02" \)
Maximum Lateral Force: \( 81.1 \) kips

DAMAGE OBSERVATIONS:

First Cracking North: \( 1/2Fy' = 0.21" \)
First Cracking South: \( -1/2Fy' = -0.26" \)
Cover Concrete Crushing North: \( \mu_{15}^{-2} = -1.52" \)
Cover Concrete Crushing South: \( \mu_{15}^{+3} = 1.53" \)
Transverse Steel Yield North: At \(-3.06"\) during pull to \( \mu_{3}^{-3} = -3.06" \)
Transverse Steel Yield South: At \(2.44"\) during push to \( \mu_{3}^{+1} = 3.08" \)
Longitudinal Bar Buckling North: Reversal from \( \mu_{5}^{+2} = 6.14" \)
Longitudinal Bar Buckling South: Reversal from \( \mu_{5}^{-1} = -5.12" \)
Longitudinal Bar Fracture North: At \(7.00"\) during push to \( \mu_{7}^{+2} = 7.17" \)
Longitudinal Bar Fracture South: At \(0.50"\) during pull to \( \mu_{7}^{-1} = -7.17" \)

\( \mu_{5}^{-1} = -5.12" \) represents the first pull cycle of displacement ductility five
### Table 3.48 Strain Data Summary for Test 25 – 24” Dia. With 2.1% Steel and 5% Axial

<table>
<thead>
<tr>
<th>MATERIAL STRAINS:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North:</td>
<td>$\varepsilon_S = 0.0036 \quad (compression)$</td>
</tr>
<tr>
<td>Cover Concrete Crushing South:</td>
<td>$\varepsilon_S = 0.004 \quad (compression)$</td>
</tr>
<tr>
<td>Transverse Steel Yield North:</td>
<td>$\varepsilon_S = 0.0091 \quad (compression)$</td>
</tr>
<tr>
<td>Transverse Steel Yield South:</td>
<td>$\varepsilon_S = 0.0125 \quad (compression)$</td>
</tr>
</tbody>
</table>
| Longitudinal Bar Buckling North:         | $\varepsilon_S = 0.042 \quad (peak \ tension \ prior \ to \ bb)$  
                                         | $\varepsilon_S = 0.016 \quad (peak \ comp. \ prior \ to \ bb)$ |
| Longitudinal Bar Buckling South:         | $\varepsilon_S = 0.035 \quad (peak \ tension \ prior \ to \ bb)$  
                                         | $\varepsilon_S = 0.019 \quad (peak \ comp. \ prior \ to \ bb)$ |
| Mander (1988) Ultimate Concrete Compression Strain, $\varepsilon_{cu} = 0.016$ |

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**Figure 3.678** T25 – Cross Section Bar Designation
Chapter 3: Experimental Observations

Figure 3.679 T25 – Target Marker Application and Optotrak Output

Figure 3.680 T25 – Symmetric Three Cycle Set Load History
Figure 3.681 T25 – Lateral Force vs. Top Column Displacement Response

**Test 25 – 24” Dia. with 2.1% Steel and 5% Axial Load – Experimental Observations**

Specimens 25-30 focus on the effects of longitudinal steel content, longitudinal bar diameter, and higher levels of axial load on column behavior. The 24” diameter column contains 16 #7 (A706) bars for longitudinal reinforcement ($A_{st}/A_g = 2.1\%$) and a #3 A706 spiral at 2” on center ($4A_{sp}/(D's) = 1\%$). The specimen had an 8ft cantilever length ($L/D = 4$), and was subjected to ($P/(f_c' A_g) = 5\%$) axial load. The symmetric three-cycle-set load history is commonly used to evaluate the seismic performance of structural components. The load history begins with elastic cycles to the following increments of the analytically predicted first yield force: $\frac{1}{4} F_y'$, $\frac{1}{2} F_y'$, $\frac{3}{4} F_y'$, and $F_y'$. The experimental first yield displacement is then determined by taking the average of the recorded displacements during the first yield push and pulls cycles. The equivalent yield displacement, used to determine the displacement ductility levels ($\mu_{\Delta 1} = 1 \ast \Delta y$), is then calculated as $\Delta_y = \Delta_y' (M_n/M_y')$. 
The symmetric three-cycle-set load history resumes with three balanced cycles at each of the following displacement ductility levels: 1, 1.5, 2, 3, 4, 5, etc. The imposed displacement history and resulting hysteretic response are shown in Figure 3.680 and Figure 3.681.

![Figure 3.682 T25 – Strain Profiles before Yield, (Left) North and (Right) South](image)

The test began with cycles in $\frac{1}{4} F_y'$ (first yield force) increments in each direction of loading until the first yield force was reached. The first cracks on the North and South sides of the specimen were observed during $(1/2 F_y' = 0.21\text{"})$ and $(-1/2 F_y' = -0.26\text{"})$ respectively, Figure 3.683. The crack distribution on all sides of the specimen at first yield, $(F_y' = 0.71\text{"})$ and $(-F_y' = -0.77\text{"})$, appears in Figure 3.684. Tensile vertical strain profiles for north and south reinforcing bars during elastic push and pull cycles appear in Figure 3.682. The average experimental first yield displacement was used to calculate the equivalent yield displacement, $\Delta_y = \Delta_y' \left( M_n / M_y' \right) = 1.02\text{"}$, which defined the reversal amplitudes for the remainder of the test.

The crack progression at displacement ductility 1, 1.5, 2, 3, 4, and 5 appear in Figure 3.685, Figure 3.687, Figure 3.688, Figure 3.689, Figure 3.690, and Figure 3.693 respectively. During these cycles the cracks became more numerous and increased in inclination on the shear faces of the specimen. The first signs of concrete crushing on the North side of the
specimen occurred just above the footing during \( \mu_{1.5} = 2 = -1.52\)”, Figure 3.686. To a smaller extent, crushing on the South side of the specimen was observed during \( \mu_{1.5} = 3 = 1.53\)”, Figure 3.686. As compressive demands increased during displacement ductility 1.5 to 5, crushing gradually increased on each side of the specimen without influencing the measured lateral forces. This compressive demand combined with local longitudinal bar restraint demands led to spiral yielding on each side of the specimen during displacement ductility three, Figure 3.689. Although these spiral strains increased during successive cycles of displacement ductility four and during the first cycle at ductility five, the reinforcing bars remained visually straight, Figure 3.691

Figure 3.683  T25 – (Left) North 1\textsuperscript{st} Cracking during \( (1/2Fy' = 0.21\)”), (Right) South 1\textsuperscript{st} Cracking during \( (-1/2Fy' = -0.26\)”)
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Figure 3.684  T25 – (Left) North Crack Distribution at ($F_y' = 0.71"$), (Middle and Right) Crack Distributions on the Front and Right Sides at ($-F_y' = -0.77"$)

Figure 3.685  T25 – (Left) North Crack Distribution at ($\mu_1^{+3} = 1.02"$), (Middle and Right) Crack Distributions on the Front and Right Sides at ($\mu_1^{-3} = -1.02"$)
Figure 3.686  T25 – (Left) North 1st Crushing during ($\mu_{1\frac{1}{5}}^{-2} = -1.52''$), (Right) South 1st Crushing during ($\mu_{1\frac{1}{5}}^{+3} = 1.53''$)

Figure 3.687  T25 – (Left) North Crack Distribution at ($\mu_{1\frac{1}{5}}^{+3} = 1.53''$), (Middle and Right) Crack Distributions on the Front and Right Sides at ($\mu_{1\frac{1}{5}}^{-3} = -1.53''$)
Figure 3.688  T25 – (Left) North Crack Distribution at ($\mu_2^{+3} = 2.04''$), (Middle and Right) Crack Distributions on the Front and Right Sides at ($\mu_2^{-3} = -2.04''$)

Figure 3.689  T25 – (Left) North Crack Distribution at ($\mu_3^{+3} = 3.06''$), (Middle and Right) Crack Distributions on the Front and Right Sides at ($\mu_3^{-3} = -3.06''$)
Figure 3.690  T25 – (Left) North Crack Distribution at ($\mu_4^3 = 4.08''$), (Middle and Right) Crack Distributions on the Front and Right Sides at ($\mu_4^{-3} = -4.09''$)

Figure 3.691  T25 – (Left) No Buckling of N3 during ($\mu_5^1 = 5.11''$), (Right) No Buckling of S3 during ($\mu_5^{-1} = -5.12''$)
The south extreme fiber bar buckled during the subsequent reversal from tensile strains sustained during ($\mu_5^{-1} = -5.12"$), as shown in the left photo of Figure 3.692. The outward buckled region occurred over the second gage length above the footing which encompassed the second spiral layer. The buckled deformation increased during ($\mu_5^{+3} = 5.10"$). Two adjacent south reinforcing bars, S2 and S4, buckled during ($\mu_6^{+2} = 6.14"$), Figure 3.694. Although buckling on the north side of the specimen was delayed by a displacement ductility level, three extreme fiber bars visibly buckled during after reversal from tensile strains sustained during ($\mu_6^{+2} = 6.14"$), Figure 3.694. Two additional north bars, N1 and N5, buckled during ($\mu_6^{-3} = -6.15"$), Figure 3.695, which produced a pronounced outward deformation of the spiral overlaying the five buckled bars on the north side of the specimen. South bar S1 buckled during ($\mu_7^{+1} = 7.16"$), Figure 3.695. The extreme fiber south bar S3 ruptured at 0.50" during the pull cycle to ($\mu_7^{-1} = -7.17"$), Figure 3.696. The north extreme fiber bars N2 and N3 ruptured at 7.00" during the push cycle to ($\mu_7^{+2} = 7.17"$), Figure 3.696. The test was concluded with fractured bars on each side of the specimen, photos of the specimen after removal of the instrumentation appear in Figure 3.697 and Figure 3.698.

Figure 3.692  T25 – (Left) Buckling of Bar S3 during ($\mu_5^{+2} = 5.10"$), (Right) Increased Buckled Deformation at ($\mu_5^{+3} = 5.10"$)
Figure 3.693  T25 – North, Front, and South Sides of the Specimen at ($\mu_{5}^{3} = -5.11\"$)

Figure 3.694  T25 – (Left) Buckling of Adjacent South Bars S2 and S4 during ($\mu_{6}^{+2} = 6.14\"$), (Right) Buckling of North Bars N2, N3, and N4 during ($\mu_{6}^{-2} = -6.17\"$)
Figure 3.695  T25 – (Left) Buckling of N1 and N5 during ($\mu_6^{-3} = -6.15\"$), (Right) Buckling of S1 during ($\mu_7^{+1} = 7.16\"$)

Figure 3.696  T25 – (Left) Fracture of Bar S3 during ($\mu_7^{-1} = -7.17\"$), (Right) Fracture of Bars N3 and N4 during ($\mu_7^{+2} = 7.17\"$)
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Figure 3.697 T25 – (Left) North Side of the Specimen after Test, (Right) Front Side

Figure 3.698 T25 – (Left) South Side of the Specimen after Test, (Right) Back Side
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Test 25 – 24” Dia. with 2.1% Steel and 5% Axial Load – Strain Data Analysis

North Reinforcement

Vertical strain profiles for the north extreme fiber bar N3, which is placed into tension during push cycles, appear in the right half of Figure 3.699. This figure shows both extreme fiber bars on the same graph to illustrate the effects of tension shift. Compression strains are concentrated near the footing-column interface while tension strains are spread higher above the footing following the inclined flexural-shear crack distribution. Compressive vertical strain profiles for north extreme fiber bar N3 during pull cycles appear in the left half of Figure 3.700. A peak tensile strain of 0.0422 was measured 7.44” above the footing on bar N3 during ($\mu_6^{+2} = 6.14”$), before the bar visibly buckled during the subsequent reversal of load. A compressive strain of -0.0091 was measured 1.63” above the footing on bar N3 during ($\mu_3^{-3} = -3.06”$), when the spiral in the confinement region yielded.

Measured spiral strains in six layers which overlaid the north extreme fiber bar appear in Figure 3.703 for pull cycles which placed the north side in compression. Successive cycles during displacement ductility four and five produced larger inelastic demands on the spiral reinforcement. For the second spiral layer above the footing, inelastic strains decreased the lateral restraint stiffness, which led to measurable outward deformation of the north extreme fiber bar before visible buckling. The measureable deformation formed a convex outward deformed region on the outside surface of the longitudinal bar, and an inward concave region just above the outward deformation. Optotrak gage lengths in the convex outward deformed region would show increased tensile strains during compression cycles which should have resulted in larger levels of compression, Figure 3.708. Similarly, gage lengths on the concave region would show some degree of increased compression due to the deformed geometry, Figure 3.709. As a comparison, the gage length just above the concave and concave regions remained straight and produced stable hysteretic response, Figure 3.710. It is important to note that all three gage lengths on Bar N3 showed rapid increase in the apparent deformation when visible buckling was observed during ($\mu_6^{-2} = -6.17”$), Figure
3.693. Although the measured compression strains in bar N3 may have been influenced by bar deformation, a compression strain of -0.0161 was measured 5.45” above the footing during ($\mu_5^{-3} = -5.11$”). The peak compression strain of -0.0269, measured during ($\mu_6^{-1} = -6.14$”), was likely influence by bar deformation, which is why it has been excluded from Figure 3.705.

Tensile strain in the second spiral layer above the footing, which overlaid the outward deformed region of bar N3, spiked during visible bar buckling, Figure 3.711. The figure contains spiral data obtained from a strain gage and an Optotrak gage length, Figure 3.679. The Optotrak strains were calculated from arc-lengths which utilized the measured 3D distance chord lengths between two adjacent LEDs and the known outside diameter of the spiral reinforcement. It is important to note that arc-length calculations become inaccurate once severe yielding in the spiral leads to the reinforcement straightening out to the left and right of the localized yielding directly over the bar. The arc-strains are still presented because the strain gage debonded, preventing further measurement to the point of visible bar buckling. The distribution of arc-strains measured around the circumference of the second spiral layer above the footing appears in Figure 3.716 and Figure 3.717. The north region is under compression during pull cycles in Figure 3.717. The middle of the section corresponds to zero along the circumference, and negative values wrap around the north side of the specimen. Specifically, measured-arc strains which overlay the three north extreme fiber bars N2, N3, and N4 are shown with vertical dashed lines. The spiral yielding is more evenly distributed along the north circumference, when compared to localized spiral yielding observed on the south side of the specimen in Figure 3.716. Also, yielding along multiple spiral layers above the footing on the north side of the specimen, Figure 3.703, is more evenly distributed than localized yielding on the south side observed in Figure 3.702. These two observations support the fact that bar buckling occurred one displacement ductility level later on the north side when compared to the south side. Furthermore, when the north side did buckle, three bars buckled simultaneously due to the distributed spiral yielding.
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The relationship between tensile strain and displacement for the peak tensile gage lengths on bar N3, 7.44” above the footing, appears in Figure 3.704. The gage length centered 3.54” above the footing had slightly lower strain magnitudes, but significantly higher unloading strains. This gage length corresponded to the outward deformed region when the bar buckled, Figure 3.694. The monotonic moment curvature prediction with the PCK (2007) Lp Hinge Method overestimates the measured tension strains at an increasing rate at higher levels of ductility. The relationship between compressive strain and displacement for the gage length 5.45” above the footing on bar N3 appears in Figure 3.705. The measured compressive strains slightly exceed the prediction with the PCK (2007) Lp Hinge Method after transverse steel yielding occurred in the north confinement region during $(\mu_3^- = -3.06")$.

**South Reinforcement**

Vertical strain profiles for the south extreme fiber bar S3, which is placed into tension during pull cycles, appear in the right half of Figure 3.700. Compressive vertical strain profiles for south extreme fiber bar S3 during push cycles appear in the left half of Figure 3.699. A peak tension strain of 0.0353 was measured 7.36” above the footing on bar S3 during $(\mu_5^- = -5.12")$, before visible bar buckling occurred during the subsequent reversal of load. The tension strains measured in lower gage lengths on bar S3 were smaller, although adjacent bars S2 and S4 had large tensile strains near the footing-column interface, Figure 3.701. A compressive strain of -0.0125 was measured 1.58” above the footing on bar S3 during $(\mu_3^+ = 3.08")$, when the spiral in the confinement region yielded. Measured spiral strains in six layers which overlaid the south extreme fiber bar appear in Figure 3.702 for push cycles which placed that side in compression. Successive cycles during displacement ductility four produced large inelastic demands on the second layer of spiral reinforcement. The measured strains obtained from the Optotrak system and a strain gage overlaying the second spiral appear in Figure 3.715. The spiral strains spiked when the bar visibly buckled during $(\mu_5^+ = 5.10")$.
Since Optotrak LEDs are placed on the outside surface of the bar, measurable deformation can be monitored in the concave and convex regions of the deformed shape. The outward deformed region (convex) developed in the gage length 3.47” above the footing on bar S3, Figure 3.712. Above the convex region, a concave region developed which increased compression strains 5.44” above the footing, Figure 3.713. The region 7.36” above the footing on bar S3 appears to be unaffected by the measurable deformation which occurred below, Figure 3.714. The concave and convex deformed regions of bar S3 show a sharp deviation when visible bar buckling was observed during \( \mu_5^{+2} = 5.10” \). Spiral strains measured around the circumference of the second spiral layer above the footing depict large localized inelasticity at the location of the extreme fiber bars S3 and S4 during push cycles, Figure 3.716. The magnitude and localized nature of the spiral strains, both around the circumference (Figure 3.716) and vertically above the footing (Figure 3.702), contributed to bar buckling one displacement ductility level earlier than the north side of the specimen.

The relationship between tension strain and displacement for the gage length 7.36” above the footing on bar S3 appear in Figure 3.706. Similar observations to those commented on for north reinforcement apply here as well, the moment curvature analysis overestimated tension strains. The relationship between compressive strain and displacement for the gage length 5.44” above the footing appears in Figure 3.707. The measured compressive strains begin to exceed the moment curvature prediction with the PCK (2007) Lp Hinge Method during ductility three, when the transverse steel entered the inelastic range. A peak compression strain of -0.0190 was measured 5.44” above the footing during \( \mu_5^{+1} = 5.11” \). This gage length was on the concave side of the measurable deformation, so it is difficult to say how much the value may be influenced.
Figure 3.699  T25 – Extreme Fiber Bar Vertical Strain Profiles during Push Cycles

Figure 3.700  T25 – Extreme Fiber Bar Vertical Strain Profiles during Pull Cycles
Figure 3.701  T25 – Adjacent South Bar S4 (above) and Bar S2 (not shown) had Large Tension Strains near the Footing

Figure 3.702  T25 – Spiral Strains on the South Side during Push Cycles
Figure 3.703 T25 – Spiral Strains on the North Side during Pull Cycles

Figure 3.704 T25 – Tension Strain-Displacement for Bar N3 during Push Cycles
Figure 3.705 T25 – Compressive Strain-Displacement for Bar N3 during Pull Cycles

Figure 3.706 T25 – Tensile Strain-Displacement for Bar S3 during Pull Cycles
Figure 3.707  T25 – Compressive Strain-Displacement for Bar S3 during Push Cycles

Figure 3.708  T25 – Bar N3 Strain Hysteresis to Buckling (3.53” above the Footing)
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Figure 3.709  T25 – Bar N3 Strain Hysteresis to Buckling (5.45” above the Footing)

Figure 3.710  T25 – Bar N3 Strain Hysteresis to Buckling (7.44” above the Footing)
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Figure 3.711  T25 – Spiral Strain Hysteresis over North Buckled Region (3.69” Above)

Figure 3.712  T25 – Bar S3 Strain Hysteresis to Buckling (3.47” above the Footing)
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Figure 3.713  T25 – Bar S3 Strain Hysteresis to Buckling (5.44” above the Footing)

Visible Buckling

Concave Region above Outward Deformation Leads to Increased Compression

Figure 3.714  T25 – Bar S3 Strain Hysteresis to Buckling (7.36” above the Footing)

Visible Buckling (Less Apparent)
Figure 3.715  T25 – Spiral Strain Hysteresis over South Buckled Region (3.47” Above)

Figure 3.716  T25 – Spiral Strain Distribution for the 2nd Spiral above the Footing during Push Cycles (Positive Location = South)
Figure 3.717  T25 – Spiral Strain Distribution for the 2$^{nd}$ Spiral above the Footing during Pull Cycles (Negative Location = North)

Test 25 – Curvature and Strain Penetration Data Analysis

Cross section strain profiles for the third horizontal section above the footing appear in Figure 3.718 and Figure 3.719 for push and pull cycles respectively. This was the first horizontal section in which LEDs for bars instrumented on the shear face of the column were visible to the camera. The third section, 5.36” above the footing, had the largest measured curvatures during pull cycles and the second largest curvatures during push cycles of any horizontal cut through the instrumented region. It appears that reinforcing bars on the shear face of the column have larger tensile strains than those predicted under the plane sections hypothesis. This will continue to be monitored in future tests to investigate the repeatability of such observations. The cross section curvature is calculated by the slope of the least squared line connecting strains measured in twelve reinforcing bars at various locations in the column, Figure 3.679. If the curvatures for many horizontal cross sections are analyzed,
curvature profiles for the plastic hinge region can be constructed; Figure 3.720 and Figure 3.721 for push and pull cycles respectively. Measured curvatures during displacement ductility one closely match the elastic curvature profile, which linearly decreases from yield curvature at the footing-column interface to zero at the center of the applied lateral load.

Plastic curvatures were found to follow a linear distribution. Linear least squared error plastic curvature lines were fit to the plastic portion of the measured curvature profiles. The extrapolation of this linear curvature line with the footing-column interface was taken as the base section curvature, since LEDs are incapable of measuring strains in this region. As the base section curvature ductility increased, the height at which the linear plastic curvature distribution intersected the elastic curvature profile also increased. The height of this intersection is termed the extent of plasticity. The measured spread of plasticity as a function of base section curvature ductility appears as circular data points in Figure 3.725.

Curvature profiles describe the elastic and plastic flexural displacements of the column, but do not address fixed-end rotations which result from strain penetration of longitudinal reinforcement into the footing. The measured vertical displacements of Optotrak LEDs placed closest to the footing column interface can be used to quantify this fixed-end rotation, Figure 3.722 and Figure 3.723. The fixed-end rotation is taken as the slope of the least squared error line connecting the strain penetration bond slip of reinforcement occurs over the partially bonded region over which the bar is being developed in tension or compression. The strain penetration displacement is obtained by multiplying this rotation by the cantilever height of the column. If an elastic curvature profile assumption is made for curvatures higher than those measured with instrumentation, then the entire curvature profile may be integrated to obtain the total column flexural displacement. This column flexural displacement was added to the strain penetration displacement, and compared to the experimentally measured displacements in Figure 3.724.
Figure 3.718  T25 – Push Cycle Cross Section Curvature Profiles, 5.36” Above Footing

Figure 3.719  T25 – Pull Cycle Cross Section Curvature Profiles, 5.36” Above Footing
Figure 3.720  T25 – Push Cycle Curvature Distribution with Plastic Regression

Figure 3.721  T25 – Pull Cycle Curvature Distribution with Plastic Regression
Figure 3.722  T25 – Push Cycle Base Rotations due to Strain Penetration

Figure 3.723  T25 – Pull Cycle Base Rotations due to Strain Penetration
Figure 3.724  T25 – Comparison of Measured and Optotrak Integrated Displacements

Figure 3.725  T25 – Measured Spread of Plasticity (Circular Data Points)
### 3.4.2 Test 26 – 24” Dia. Column with 2.1% Long. Steel and 10% Axial Load

#### Table 3.49 Observations for Test 26 – 24” Dia. With 2.1% Steel and 10% Axial Load

<table>
<thead>
<tr>
<th>VALUES OF INTEREST:</th>
<th></th>
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<tbody>
<tr>
<td>Concrete Compressive Strength: $f'_c = 5890 \text{ psi}$</td>
<td></td>
</tr>
<tr>
<td>Axial Load: $P = 244.7 \text{ kips } (P/(f'_cA_g) = 10%)$</td>
<td></td>
</tr>
<tr>
<td>Longitudinal Steel Content: $16 #7 Bars \ (A_{st}/A_g = 2.1%)$</td>
<td></td>
</tr>
<tr>
<td>Analytical First Yield Force: $F'_y = 59.84 \text{ kips}$</td>
<td></td>
</tr>
<tr>
<td>Experimental First Yield Displacement: $\Delta'_y = 0.75''$</td>
<td></td>
</tr>
<tr>
<td>Analytical Nominal Moment Capacity: $M_n = 636.83 \text{ kip \cdot ft}$</td>
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<tr>
<td>Equivalent Yield Displacement: $\Delta_y = 0.99''$</td>
<td></td>
</tr>
<tr>
<td>Maximum Lateral Force: $88.8 \text{ kips}$</td>
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<table>
<thead>
<tr>
<th>DAMAGE OBSERVATIONS:</th>
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<tbody>
<tr>
<td>First Cracking North: $1/2Fy' = 0.24''$</td>
<td></td>
</tr>
<tr>
<td>First Cracking South: $-1/2Fy' = -0.25''$</td>
<td></td>
</tr>
<tr>
<td>Cover Concrete Crushing North: $\mu_{15}^{-2} = -1.49''$</td>
<td></td>
</tr>
<tr>
<td>Cover Concrete Crushing South: $\mu_{15}^{+2} = 1.50''$</td>
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<tr>
<td>Transverse Steel Yield North: At $-2.68''$ during pull to $\mu_3^{-1} = -2.99''$</td>
<td></td>
</tr>
<tr>
<td>Transverse Steel Yield South: At $2.43''$ during push to $\mu_3^{+1} = 2.97''$</td>
<td></td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North: Reversal from $\mu_3^{+2} = 4.98''$</td>
<td></td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South: Reversal from $\mu_4^{-3} = -3.98''$</td>
<td></td>
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<tr>
<td>Longitudinal Bar Fracture North: At $5.23''$ during push to $\mu_6^{+4} = 5.97''$</td>
<td></td>
</tr>
<tr>
<td>Longitudinal Bar Fracture South: At $-5.90''$ during pull to $\mu_6^{-2} = -5.98''$</td>
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$^*\mu_5^{+2} = 4.98''$ represents the second push cycle of displacement ductility five
Table 3.50 Strain Data Summary for Test 26 – 24” Dia. with 2.1% Steel and 10% Axial

<table>
<thead>
<tr>
<th>MATERIAL STRAINS:</th>
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</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North:</td>
<td>$\varepsilon_s = 0.0045 \text{ (compression)}$</td>
</tr>
<tr>
<td>Cover Concrete Crushing South:</td>
<td>$\varepsilon_s = 0.0046 \text{ (compression)}$</td>
</tr>
<tr>
<td>Transverse Steel Yield North:</td>
<td>$\varepsilon_s = 0.0089 \text{ (compression)}$</td>
</tr>
<tr>
<td>Transverse Steel Yield South:</td>
<td>$\varepsilon_s = 0.0121 \text{ (compression)}$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North:</td>
<td>$\varepsilon_s = 0.032 \text{ (peak tension prior to ( bb ))}$</td>
</tr>
<tr>
<td></td>
<td>$\varepsilon_s = 0.016 \text{ (peak comp. prior to ( bb ))}$</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South:</td>
<td>$\varepsilon_s = 0.024 \text{ (peak tension prior to ( bb ))}$</td>
</tr>
<tr>
<td></td>
<td>$\varepsilon_s = 0.027 \text{ (peak comp. prior to ( bb ))}$</td>
</tr>
<tr>
<td>Mander (1988) Ultimate Concrete Compression Strain, $\varepsilon_{cu} = 0.0167$</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.726 T26 – Cross Section Bar Designation
Figure 3.727 T26 – Target Marker Application and Optotrak Spatial Output

Figure 3.728 T26 – Compressive Axial Load from One Jack (Total = 2*Value)
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Figure 3.729  T26 – Symmetric Three Cycle Set Load History

Figure 3.730  T26 – Lateral Force vs. Displacement Response
Test 26 – 24” Dia. with 2.1% Steel and 10% Axial Load – Experimental Observations

Specimens 25-30 focus on the effects of longitudinal steel content, longitudinal bar diameter, and higher levels of axial load on column behavior. This section summarizes experimental observations and data analysis for column Test 26. The 24” diameter column contains 16 #7 (A706) bars for longitudinal reinforcement \(\frac{A_{st}}{A_g} = 2.1\%\) and a #3 A706 spiral at 2” on center \(\frac{4A_{sp}}{(D’s)} = 1\%\). The specimen had an 8ft cantilever length \(L/D = 4\), and was subjected to \(\frac{P}{(f’c’A_g)} = 10\%\) axial load. The symmetric three-cycle-set load history is commonly used to evaluate the seismic performance of structural components. The load history begins with elastic cycles to the following increments of the analytically predicted first yield force: \(\frac{1}{4} F_y\), \(\frac{1}{2} F_y\), \(\frac{3}{4} F_y\), and \(F_y\). The experimental first yield displacement is then determined by taking the average of the recorded displacements during the first yield push and pulls cycles. The equivalent yield displacement, used to determine
the displacement ductility levels ($\mu_{\Delta_1} = 1 \times \Delta_y$), is then calculated as $\Delta_y = \Delta'_y (M_n/M'_y)$. The symmetric three-cycle-set load history resumes with three balanced cycles at each of the following displacement ductility levels: 1, 1.5, 2, 3, 4, 5, etc. The imposed displacement history and resulting hysteretic response appears in Figure 3.729 and Figure 3.730.

The test began with cycles in $\frac{1}{4} F_y$ (first yield force) increments in each direction of loading until the first yield force was reached. A single crack formed at the footing-column interface on the North and South sides of the specimen during $(1/2 F_y' = 0.24")$ and $(-1/2 F_y' = -0.25")$ respectively. More distributed cracking formed above the base section during $(3/4 F_y' = 0.46")$ and $(-3/4 F_y' = -0.48")$, Figure 3.733. The crack distribution on all sides of the specimen at first yield, $(F_y' = 0.74")$ and $(-F_y' = -0.76")$, appears in Figure 3.734. Tensile vertical strain profiles for north and south reinforcing bars during elastic push and pull cycles appear in Figure 3.732. The average experimental first yield displacement was used to calculate the equivalent yield displacement, $\Delta_y = \Delta'_y (M_n/M'_y) = 0.99"$, which defined the reversal amplitudes for reminder of the test.
The crack progression at displacement ductility 1, 1.5, 2, 3, and 4 appear in Figure 3.735, Figure 3.737, Figure 3.739, Figure 3.740, and Figure 3.742 respectively. During these cycles the cracks became more numerous and increased in inclination on the shear faces of the specimen. Small amounts concrete flaking was observed on the south and north sides of the specimen during \((\mu_{1.5}^+ = 1.49\)\) and \((\mu_{1.5}^- = -1.49\)\), Figure 3.736. This visible flaking resulted in small amounts of concrete crushing during \((\mu_{1.5}^+ = 1.50\)\) and \((\mu_{1.5}^- = -1.49\)\). Crushing on the south and north sides of the specimen increased at \((\mu_{1.99}^+ = 1.99\)\) and \((\mu_{1.99}^- = -1.99\)\), Figure 3.738. As compressive demands increased during displacement ductility 1.5 to 4, crushing gradually increased on each side of the specimen without influencing the measured lateral forces. Compressive demand during \((\mu_{3}^+ = 2.97\)\) and \((\mu_{3}^- = -2.99\)\) lead to spiral yielding in confinement regions. Spiral strains on each side of the specimen increased during each successive cycle of ductility four, but the extreme fiber reinforcement remained visibly straight, Figure 3.741.

The south extreme fiber bar visibly buckled during \((\mu_{5}^+ = 4.98\)\), as shown in the left photo of Figure 3.743. The outward buckled region occurred over the second gage length above the footing which encompassed the second spiral layer. The buckled deformation increased during \((\mu_{5}^+ = 4.98\)\). The north extreme fiber bar visibly buckled during the reversal from tension strains sustained during \((\mu_{5}^+ = 4.98\)\), Figure 3.744. Two adjacent south reinforcing bars, S2 and S4, buckled during \((\mu_{5}^+ = 4.97\)\), Figure 3.745. Two adjacent north reinforcing bars, N2 and N4, buckled during \((\mu_{6}^- = -5.97\)\), Figure 3.746. Severe buckling of three bars on the south side of the specimen during \((\mu_{6}^+ = 5.97\)\) led to significant crushing of core concrete, Figure 3.747. Previously buckled south extreme fiber bar S3 ruptured during \((\mu_{6}^- = -5.98\)\), Figure 3.747, resulting in the first significant loss in the strength. South reinforcing bars S1 and S5 buckled during \((\mu_{6}^+ = 5.97\)\), Figure 3.748. Two additional south bars, S2 and S4, ruptured during \((\mu_{6}^- = -5.99\)\), Figure 3.748. Previously buckled north reinforcing bars N3 and N4 ruptured during \((\mu_{6}^+ = 5.97\)\), Figure 3.749. The test was concluded with fractured bars on each side of the specimen, photos appear in Figure 3.749 and Figure 3.750.
Figure 3.733 T26 – (Left) North Distributed Cracking during ($3/4Fy' = 0.46''$), (Right) South Distributed Cracking during ($-3/4Fy' = -0.48''$)

Figure 3.734 T26 – (Left) North Crack Distribution at ($Fy' = 0.74''$), (Middle and Right) Crack Distributions on the Front and Right Sides at ($-Fy' = -0.76''$)
Figure 3.735  T26 – (Left) North Crack Distribution at \( \mu^{+3}_{1} = 1.00'' \), (Middle and Right) Crack Distributions on the Front and Right Sides at \( \mu^{-3}_{1} = -1.01'' \)

Figure 3.736  T26 – (Left) South Concrete Flaking during \( \mu^{+1}_{1.5} = 1.49'' \), (Right) North Concrete Flaking during \( \mu^{-1}_{1.5} = -1.49'' \)
Figure 3.737 T26 – (Left) North Crack Distribution at ($\mu_{1.5}^{+3} = 1.50''$), (Middle and Right) Crack Distributions on the Front and Right Sides at ($\mu_{1.5}^{-3} = -1.50''$)

Figure 3.738 T26 – (Left) South Crushing during ($\mu_{2}^{-1} = 1.99''$), (Right) North Crushing during ($\mu_{2}^{-1} = -1.99''$)
Figure 3.739  T26 – (Left) North Crack Distribution at \((\mu_2^3 = 1.99\)"), (Middle and Right) Crack Distributions on the Front and Right Sides at \((\mu_2^{-3} = -1.99\)"").

Figure 3.740  T26 – (Left) North Crack Distribution at \((\mu_3^3 = 2.98\)"), (Middle and Right) Crack Distributions on the Front and Right Sides at \((\mu_3^{-3} = -2.98\)"").
Figure 3.741 T26 – (Left) Bar S3 Remained Visibly Straight during \( \mu_4^{+3} = 3.98" \), (Right) Crack Distribution on the Back Side

Figure 3.742 T26 – (Left) North Crack Distribution at \( \mu_4^{+3} = 3.98" \), (Middle and Right) Crack Distributions on the Front and Right Sides at \( \mu_4^{-3} = -3.98" \)
Figure 3.743 T26 – (Left) Slight Visible Buckling of Bar S3 during \( (\mu^+_5 = 4.98\text{"}) \),
(Right) Increased Deformation in Buckled Bar S3 during \( (\mu^+_{5} = 4.98\text{"}) \)

Figure 3.744 T26 – Slight Visible Buckling of Bar N3 during \( (\mu^-_{5} = -4.99\text{"}) \)
Chapter 3: Experimental Observations

Figure 3.745  T26 – Buckling of Adjacent South Bars S2 and S4 during ($\mu_{5}^{+3} = 4.97''$)

Figure 3.746  T26 – Buckling of North Bars N4 and N2 during ($\mu_{6}^{-1} = -5.97''$)
Figure 3.747 T26 – (Left) South Side of the Specimen during \( \mu_6^{+2} = 5.97'' \), (Right) Fracture of Previously Buckled Bar S3 during \( \mu_6^{-2} = 5.98'' \)

Figure 3.748 T26 – (Left) Buckling of South Bars S1 and S5 during \( \mu_6^{+3} = 5.97'' \), (Right) Fracture of Previously Buckled South Bars S2 and S4 during \( \mu_6^{-3} = -5.99'' \)
Figure 3.749  T26 – (Left) Fracture of Previously Buckled Bars N3 and N4 during \( (\mu_6^{+4} = 5.97") \), Front (Middle) and Back (Right) After the Test

Figure 3.750  T26 – After the Test (Left) South Side and (Right) North Side
Test 26 – 24” Dia. with 2.1% Steel and 10% Axial Load – Strain Data Analysis

North Reinforcement

Vertical strain profiles for the north extreme fiber bar N3, which is placed into tension during push cycles, appear in the right half of Figure 3.751. This figure shows both extreme fiber bars on the same graph to illustrate the effects of tension shift. Compression strains are concentrated near the footing-column interface while tension strains are spread higher above the footing following the inclined flexural-shear crack distribution. Compressive vertical strain profiles for north extreme fiber bar N3 during pull cycles appear in the left half of Figure 3.752. A peak tensile strain of 0.0319 was measured 6.00” above the footing on bar N3 during ($\mu_5^{+1} = 4.98^\circ$), before the bar visibly buckled during ($\mu_5^{-2} = -4.97^\circ$). The relationship between strain and displacement for this gage length appears in Figure 3.756. Moment-curvature analysis with the PCK (2007) Lp Hinge Method begins to over predict the measured tensile strains at an increasing rate beyond displacement ductility two. A compressive strain of −0.0089 was measured 2.24” above the footing on bar N3 during ($\mu_3^{-1} = -2.99^\circ$), when the first spiral in the confinement region yielded. Measured spiral strains in six layers which overlaid the north extreme fiber bar appear in Figure 3.755. Spiral tensions trains increased during each successive pull cycle of displacement ductility four. The relationship between compressive strain and displacement, for the gage length 6.00” above the footing on bar N3, appears in Figure 3.757. The measured compressive strains match the moment-curvature prediction with the PCK (2007) Lp Hinge Method through displacement ductility four. A peak compressive strain of -0.0164 was measured 6.00” above the footing on bar N3 during ($\mu_5^{-1} = -4.98^\circ$).

Strain hysteresis for gage lengths 4.16” and 5.45” above the footing on bar N3 appear in Figure 3.760 and Figure 3.761. Both remained stable until the point of visible bar buckling during ($\mu_5^{-2} = -4.97^\circ$). Tensile strain in the second spiral layer above the footing, which overlaid the outward deformed region of bar N3, spiked during visible bar buckling, Figure 3.762. The figure contains spiral data obtained from a strain gage and an Optotrak gage.
length, Figure 3.727. The Optotrak strains were calculated using arc-lengths obtained from measured 3D distance chord lengths and the known outside diameter of the spiral. It is important to note that arc-length calculations become inaccurate once severe yielding in the spiral leads to the reinforcement straightening out to the left and right of the localized yielding directly over the bar where the strain gage is located. The distribution of arc-strains measured around the circumference of the first spiral layer above the footing appears in Figure 3.769. This first spiral layer only encompassed the north region of the column which was under compression during pull cycles. The middle of the section corresponds to zero along the circumference, and negative values wrap around the north side of the specimen. Specifically, measured-arc strains which overlay the three north extreme fiber bars N2, N3, and N4 are shown with vertical dashed lines. The distribution of measured spiral strains for the second and third layers above the footing on the north side appear in Figure 3.771 and Figure 3.773. Inelastic spiral strains were concentrated in the first two spiral layers above the footing in the region between bars N2, N3, and N4.

**South Reinforcement**

Vertical strain profiles for the south extreme fiber bar S3, which is placed into tension during pull cycles, appear in the right half of Figure 3.752. Compressive vertical strain profiles for south extreme fiber bar S3 during push cycles appear in the left half of Figure 3.751. The compressive vertical strain profiles for bars S2, S3, and S4 had a similar shape for the first three gage lengths above the footing, Figure 3.753. Their measured compressive strains appear to be influenced by measurable deformation which occurred after yielding of the spiral reinforcement. A diagram which shows the location and effect of measurable deformation appears in Figure 3.763. The first and third gage lengths above the footing had increased compression while the gage length had additional tension during pull cycles. This behavior is observable in the measured strain hysteresis 1.96” (Figure 3.764), 3.98” (Figure 3.765), and 5.89” (Figure 3.766) above the footing. As a comparison, the gage length 9.96” above the footing on bar S3, Figure 3.767, appears to be unaffected by measurable deformation.
A peak tension strain of 0.0318 was measured 3.98” above the footing on bar S3 during $(\mu_5^{-1} = -4.98”)$. Visible buckling was observed during the previous push cycle, but it is expected that the bar straightened out and produced reliable strains at $(\mu_5^{-1} = -4.98”)$. The peak tension strain before bar buckling of 0.0244 was measured 9.96” above the footing on bar S3 during $(\mu_4^{-1} = -3.98”)$. The relationship between tension strain and displacement for the gage length 9.96” above the footing appears in Figure 3.758. Moment-curvature analysis with the PCK (2007) Lp Hinge Method begins to over predict the measured tension strains during displacement ductility three. A compressive strain of -0.0121 was measured 1.96” above the footing during $(\mu_3^{-1} = -2.99”)$ when first spiral layer in the south confinement region yielded. The peak compression strain of -0.0273, measured 5.88” above the footing on bar S3, is unreliable due to measured deformation. The relationship between compressive strain and displacement for this gage length appears in Figure 3.759. The measurable deformation led to compressive strains which significantly exceed the moment-curvature prediction with the PCK (2007) Lp Hinge Method.

Measured strains in six spiral layers which overlaid the south extreme fiber bar appear in Figure 3.754 for push cycles. Cycles during displacement ductility four produced successively larger inelastic demands on the bottom three layers of spiral reinforcement. Strain gage and Optotrak strain hysteresis for the spiral layer 4.03” above the footing appear in Figure 3.768. The two measurement methods match well until the strain gage debonded during the third push cycle of displacement ductility four. The spiral strains spiked during $(\mu_5^{+1} = 4.98”)$ when visible bar buckling was observed, Figure 3.743. Measured arc-strains around the circumference of spirals on the south side of the specimen during push cycles appear in Figure 3.770 and Figure 3.772. Inelastic spiral strains were localized over bars S2, S3, and S4.
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Figure 3.751 T26 – Extreme Fiber Bar Vertical Strain Profiles during Push Cycles

Figure 3.752 T26 – Extreme Fiber Bar Vertical Strain Profiles during Pull Cycles
Figure 3.753  T26 – Similar Compressive Strain Profiles Observed in Adjacent South Bars S4 (Left) and S2 (Right)

Figure 3.754  T26 – Spiral Strains on the South Side during Push Cycles
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Figure 3.755  T26 – Spiral Strains on the North Side during Pull Cycles

Figure 3.756  T26 – Tensile Strain-Displacement for Bar N3 during Push Cycles
Figure 3.757  T26 – Compressive Strain-Displacement for Bar N3 during Pull Cycles

Figure 3.758  T26 – Tensile Strain-Displacement for Bar S3 during Pull Cycles
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Figure 3.759 T26 – Compressive Strain-Displacement for Bar S3 during Push Cycles

Figure 3.760 T26 – Bar N3 Strain Hysteresis to Buckling (4.16” Above Footing)
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Figure 3.761  T26 – Bar N3 Strain Hysteresis to Buckling (5.45” Above Footing)

Figure 3.762  T26 – Spiral Strain Hysteresis over North Buckled Region (3.97” Above)
Figure 3.763  T26 – Locations of Measurable Deformation before Visible Buckling

Figure 3.764  T26 – Bar S3 Strain Hysteresis to Buckling (1.96” Above Footing)
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Figure 3.765  T26 – Bar S3 Strain Hysteresis to Buckling (3.98” Above Footing)

Figure 3.766  T26 – Bar S3 Strain Hysteresis to Buckling (5.89” Above Footing)
Figure 3.767  T26 – Bar S3 Strain Hysteresis to Buckling (9.96” Above Footing)

Figure 3.768  T26 – Spiral Strain Hysteresis over South Buckled Region (4.03” Above)
Figure 3.769  T26 – Spiral Strain Distribution for the 1\textsuperscript{st} Spiral above the Footing during Pull Cycles (Negative Location = North)

Figure 3.770  T26 – (2\textsuperscript{nd} Spiral Layer North and 1\textsuperscript{st} Spiral Layer South) above the Footing during Push Cycles (Positive = South)
Figure 3.771 T26 – (2nd Spiral Layer North and 1st Spiral Layer South) above the Footing during Pull Cycles (Negative = North)

Figure 3.772 T26 – (3rd Spiral Layer North and 2nd Spiral Layer South) above the Footing during Push Cycles (Positive = South)
Figure 3.773  T26 – (3\textsuperscript{rd} Spiral Layer North and 2\textsuperscript{nd} Spiral Layer South) above the Footing during Pull Cycles (Negative = North)

Test 26 – Curvature and Strain Penetration Data

Cross section strain profiles for the fourth horizontal section above the footing appear in Figure 3.774 and Figure 3.775 for push and pull cycles respectively. The plane section hypothesis fits the measured strain data well for this section since it lies above the region which was influence by measurable deformation on the south side of the specimen. The curvature is calculated as the slope of the least squared line connecting strains measured in twelve instrumented reinforcing bars in the cross section, Figure 3.727. If the curvatures for many horizontal cross sections are analyzed, curvature profiles for the plastic hinge region can be constructed; Figure 3.776 and Figure 3.777 for push and pull cycles respectively. Measured curvatures during displacement ductility one closely match the elastic curvature profile, which linearly decreases from yield curvature at the footing-column interface to zero at the center of the applied lateral load.
Plastic curvatures were found to follow a linear distribution. Linear least squared error plastic curvature lines were fit to the plastic portion of the measured curvature profiles. The extrapolation of this linear curvature line with the footing-column interface was taken as the base section curvature, since LEDs are incapable of measuring strains in this region. As the base section curvature ductility increased, the height at which the linear plastic curvature distribution intersected the elastic curvature profile also increased. This measured spread of plasticity as a function of base section curvature ductility is plotted in Figure 3.781 with circular data points.

Curvature profiles describe the elastic and plastic flexural displacements of the column, but do not address fixed-end rotations which result from development of longitudinal reinforcement into the footing. The measured vertical displacements of Optotrak LEDs placed closest to the footing column interface can be used to quantify this fixed-end rotation, Figure 3.778 and Figure 3.779. The fixed-end rotation is taken as the slope of the least squared error line fit to the bond slip profile. The strain penetration displacement is obtained by multiplying this rotation by the cantilever height of the column. If an elastic curvature profile assumption is made for curvatures higher than those measured with instrumentation, then the entire curvature profile may be integrated to obtain the total column flexural displacement. This column flexural displacement was added to the strain penetration displacement, and compared to the experimentally measured displacements in Figure 3.780. The Optotrak integrated displacement matches well with those obtained from a string potentiometer placed at the center of the lateral load, which indicates that shear deformation are small.
Figure 3.774 T26 – Push Cycle Cross Section Strain Profiles, 7.97” Above Footing

Figure 3.775 T26 – Pull Cycle Cross Section Strain Profiles, 7.97” Above Footing
Figure 3.776 T26 – Push Cycle Curvature Profiles with Plastic Regression

Figure 3.777 T26 – Pull Cycle Curvature Profiles with Plastic Regression
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Figure 3.778 T26 – Fixed-End Rotation due to Strain Penetration during Push Cycles

Figure 3.779 T26 – Fixed-End Rotation due to Strain Penetration during Pull Cycles
Figure 3.780  T26 – Comparison of Measured and Optotrak Integrated Displacements

Figure 3.781  T26 – Measured Spread of Plasticity (Circular Data Points)
### 3.4.3 Test 27 – 24” Dia. Column with 1.6% Long. Steel and 10% Axial Load

**Table 3.51 Observations for Test 27 – 24” Dia. With 1.6% Steel and 10% Axial Load**

<table>
<thead>
<tr>
<th>VALUES OF INTEREST:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Compressive Strength:</td>
</tr>
<tr>
<td>$f'_c = 6149 \text{ psi}$</td>
</tr>
<tr>
<td>Axial Load:</td>
</tr>
<tr>
<td>$P = 255.5 \text{ kips } (P/(f'_c A_g) = 10%)$</td>
</tr>
<tr>
<td>Longitudinal Steel Content:</td>
</tr>
<tr>
<td>16 #6 Bars $(A_{st}/A_g = 1.6%)$</td>
</tr>
<tr>
<td>Analytical First Yield Force:</td>
</tr>
<tr>
<td>$F'_y = 50.53 \text{ kips}$</td>
</tr>
<tr>
<td>Experimental First Yield Displacement:</td>
</tr>
<tr>
<td>$\Delta'_y = 0.70&quot;$</td>
</tr>
<tr>
<td>Analytical Nominal Moment Capacity:</td>
</tr>
<tr>
<td>$M_n = 531.72 \text{ kip } \ast \text{ ft}$</td>
</tr>
<tr>
<td>Equivalent Yield Displacement:</td>
</tr>
<tr>
<td>$\Delta_y = 0.92&quot;$</td>
</tr>
<tr>
<td>Maximum Lateral Force:</td>
</tr>
<tr>
<td>$70.19 \text{ kips}$</td>
</tr>
</tbody>
</table>

**DAMAGE OBSERVATIONS:**

- First Cracking North: $1/2 Fy' = 0.20"$
- First Cracking South: $-1/2 Fy' = -0.21"$
- Cover Concrete Crushing North: $\mu_{1,5}^{-1} = -1.38"$
- Cover Concrete Crushing South: $\mu_{1,5}^{+2} = 1.38"$
- Transverse Steel Yield North: At $-2.77"$ during pull to $\mu_3^{-1} = -2.76"$
- Transverse Steel Yield South: At $2.76"$ during push to $\mu_3^{+1} = 2.76"$
- Longitudinal Bar Buckling North: Reversal from $\mu_5^{+1} = 4.60"$
- Longitudinal Bar Buckling South: Reversal from $\mu_4^{-3} = -3.67"$
- Longitudinal Bar Fracture North: At $1.37"$ during push to $\mu_6^{+3} = 5.53"$
- Longitudinal Bar Fracture South: At $-4.94"$ during pull to $\mu_6^{-2} = -5.53"$

*$\mu_5^{+1} = 4.60"$ represents the first push cycle of displacement ductility five*
Table 3.52 Strain Data Summary for Test 27 – 24” Dia. with 1.6% Steel and 10% Axial

<table>
<thead>
<tr>
<th>MATERIAL STRAINS:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North:</td>
<td>( \varepsilon_s = 0.0036 ) (compression)</td>
</tr>
<tr>
<td>Cover Concrete Crushing South:</td>
<td>( \varepsilon_s = 0.0038 ) (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield North:</td>
<td>( \varepsilon_s = 0.0168 ) (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield South:</td>
<td>( \varepsilon_s = 0.0124 ) (compression)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North:</td>
<td>( \varepsilon_s = 0.036 ) (peak tension prior to ( bb )) ( \varepsilon_s = 0.032 ) (peak comp. prior to ( bb ))</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South:</td>
<td>( \varepsilon_s = 0.024 ) (peak tension prior to ( bb )) ( \varepsilon_s = 0.023 ) (peak comp. prior to ( bb ))</td>
</tr>
<tr>
<td>Mander (1988) Ultimate Concrete Compression Strain, ( \varepsilon_{cu} = 0.0163 )</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.782 T27 – Test Setup and Cross Section Bar Designation
Figure 3.783 T27 – Target Marker Application and Optotrak Spatial Output

![Figure 3.783 T27 – Target Marker Application and Optotrak Spatial Output](image)

Figure 3.784 T27 – Compressive Axial Load from One Jack (Total = 2*Value)

![Figure 3.784 T27 – Compressive Axial Load from One Jack (Total = 2*Value)](image)
Figure 3.785  T27 – Symmetric Three Cycle Set Load History

Figure 3.786  T27 – Lateral Force vs. Displacement Hysteretic Response
Test 27 – 24” Dia. with 1.6% Steel and 10% Axial Load – Experimental Observations

Specimens 25-30 focus on the effects of longitudinal steel content, longitudinal bar diameter, and higher levels of axial load on column behavior. This report summarizes experimental observations and data analysis for column Test 27. The 24” diameter column contains 16 #6 (A706) bars for longitudinal reinforcement ($A_{st}/A_g = 1.6\%$) and a #3 A706 spiral at 2” on center ($4A_{sp}/(D’s) = 1\%$). The specimen had an 8ft cantilever length ($L/D = 4$), and was subjected to $(P/(f_c’A_g) = 10\%$) axial load. The symmetric three-cycle-set load history is commonly used to evaluate the seismic performance of structural components. The load history begins with elastic cycles to the following increments of the analytically predicted first yield force: $\frac{1}{4} F_y’$, $\frac{1}{2} F_y’$, $\frac{3}{4} F_y’$, and $F_y’$. The experimental first yield displacement is then determined by taking the average of the recorded displacements during the first yield push and pulls cycles. The equivalent yield displacement, used to determine
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the displacement ductility levels \( (\mu_{\Delta_1} = 1 \ast \Delta_y) \), is then calculated as \( \Delta_y = \Delta'_y (M_n / M'_y) \). The symmetric three-cycle-set load history resumes with three balanced cycles at each of the following displacement ductility levels: 1, 1.5, 2, 3, 4, 5, etc. The imposed displacement history and resulting hysteretic response appear in Figure 3.785 and Figure 3.786.

![Figure 3.788 T27 – Strain Profiles before Yield, (Left) North and (Right) South](image)

The test began with cycles in \( \frac{1}{4} F_y' \) (first yield force) increments in each direction of loading until the first yield force was reached. The first cracks on the north and south sides of the specimen formed during \( (1/2Fy' = 0.20") \) and \( (-1/2Fy' = -0.21") \) respectively, Figure 3.789. The crack distribution on all sides of the specimen at first yield, \( (Fy' = 0.68") \) and \( (-Fy' = -0.72") \), appears in Figure 3.790. Tensile vertical strain profiles for north and south reinforcing bars during elastic push and pull cycles appear in Figure 3.788. The average experimental first yield displacement was used to calculate the equivalent yield displacement, \( \Delta_y = \Delta'_y (M_n / M'_y) = 0.92" \), which defined the reversal amplitudes for reminder of the test.

The crack progression at displacement ductility 1, 1.5, 2, 3, and 4 appear in Figure 3.791, Figure 3.793, Figure 3.795, Figure 3.797, and Figure 3.799 respectively. During
these cycles the cracks became more numerous and increased in inclination on the shear faces of the specimen. Small amounts concrete flaking was observed on the south and north sides of the specimen during \((\mu_{1.5} = 1.38\)") and \((\mu_{1.5} = -1.38\)"), Figure 3.792. In previous tests, this flaking leads to crushing during subsequent cycles, but crushing was not observed until displacement ductility two. Crushing on the south and north sides of the specimen occurred during \((\mu_{2} = 1.84\)") and \((\mu_{2} = -1.83\)"), Figure 3.794. As compressive demands increased during displacement ductility 1.5 to 4, crushing gradually increased on each side of the specimen. Compressive demand during \((\mu_{3} = 2.76\)") and \((\mu_{3} = -2.76\)"") lead to spiral yielding in confinement regions, Figure 3.796. Spiral strains on each side of the specimen increased during each successive cycle of ductility four, but the extreme fiber reinforcement remained visibly straight, Figure 3.798.

The south extreme fiber bar visibly buckled during \((\mu_{5} = 4.60\)"), as shown in the left photo of Figure 3.800. The outward buckled region occurred over the second gage length above the footing which encompassed the second spiral layer. The buckled deformation increased during \((\mu_{5} = 4.61\)"), Figure 3.800. The north extreme fiber bar visibly buckled during the reversal from tension strains sustained during \((\mu_{5} = 4.60\)"), Figure 3.801. An adjacent north reinforcing bar N4 buckled during \((\mu_{5} = -4.59\)"), Figure 3.802. Previously buckled north reinforcement placed into tension during \((\mu_{5} = 4.60\)""") straightened out, showing large amounts of permanent deformation in spirals overlaying the outward buckled region, Figure 3.802. An adjacent north bar N2 buckled during \((\mu_{5} = 4.60\)"), Figure 3.803. Two additional south reinforcing bars, S2 and S4, buckled during \((\mu_{6} = 5.52\)"), Figure 3.803 and Figure 3.804. Significant core concrete crushing behind three buckled north reinforcing occurred during \((\mu_{6} = -5.54\)"), Figure 3.804. The previously buckled extreme fiber south bar S3 ruptured during \((\mu_{6} = -5.53\)"), Figure 3.805, leading to a significant loss in strength. Two previously buckled north bars N3 and N4 ruptured before straightening out in tension during the third push cycle of ductility six, Figure 3.805. Photos of the specimen after the instrumentation was removed appear in Figure 3.806 and Figure 3.807.
Figure 3.789 T27 – (Left) Cracking on the North Side during \(1/2Fy' = 0.20''\), (Right) South Cracking \((-1/2Fy' = -0.21'')\)

Figure 3.790 T27 – (Left) North Crack Distribution at \((Fy' = 0.68'')\), (Middle and Right) Crack Distributions on the Front and Right Sides at \((-Fy' = -0.72'')\)
Figure 3.791 T27 – (Left) North Crack Distribution at \( (\mu_{1}^{+3} = 0.92") \), (Middle and Right) Crack Distributions on the Front and Right Sides at \( (\mu_{1}^{-3} = -0.92") \)

Figure 3.792 T27 – (Left) South Concrete Flaking during \( (\mu_{1.5}^{+2} = 1.38") \), (Right) North Concrete Flaking during \( (\mu_{1.5}^{-1} = -1.38") \)
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Figure 3.793 T27 – (Left) North Crack Distribution at \((\mu_{1.5}^{+3} = 1.38")\), (Middle and Right) Crack Distributions on the Front and Right Sides at \((\mu_{1.5}^{-3} = -1.38")\)

Figure 3.794 T27 – (Left) South Crushing during \((\mu_{2}^{+1} = 1.84")\), (Right) North Crushing during \((\mu_{2}^{-1} = -1.83")\)
Figure 3.795 T27 – (Left) North Crack Distribution at ($\mu_2^+ = 1.84''$), (Middle and Right) Crack Distributions on the Front and Right Sides at ($\mu_2^- = -1.84''$)

Figure 3.796 T27 – (Left) South Spiral Yield at ($\mu_3^+ = 2.76''$), (Right) North Spiral Yield at ($\mu_3^- = -2.76''$)
Figure 3.797 T27 – (Left) North Crack Distribution at \( \mu_3^+ = 2.76" \), (Middle and Right) Crack Distributions on the Front and Right Sides at \( \mu_3^- = -2.76" \)

Figure 3.798 T27 – (Left) Bar S3 Remained Visibly Straight during \( \mu_4^+ = 3.66" \), (Right) Bar N3 Remained Visibly Straight during \( \mu_4^- = -3.67" \)
Figure 3.799 T27 – (Left) North Crack Distribution at ($\mu_4^{+3} = 3.66''$), (Middle and Right) Crack Distributions on the Front and Right Sides at ($\mu_4^{-3} = -3.67''$)

Figure 3.800 T27 – (Left) Very Slight Visible Buckling of Bar S3 during ($\mu_5^{+1} = 4.60''$), (Right) Increased Deformation in Buckled Bar S3 during ($\mu_5^{+2} = 4.61''$)
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Figure 3.801 T27 – Visible Buckling of Bar N3 during ($\mu_5^{-1} = -4.60''$)

Figure 3.802 T27 – (Left) Buckling of Adjacent Bar N4 ($\mu_5^{-2} = -4.59''$), (Right) North Spiral Deformation at ($\mu_5^{+3} = 4.60''$)
Figure 3.803  T27 – (Left) Buckling of Adjacent Bar N2 during \( \mu_5^{-3} = -4.60'' \), (Right) Buckling of Adjacent Bar S4 during \( \mu_6^{+1} = 5.52'' \)

Figure 3.804  T27 – (Left) Buckling of Adjacent Bar S2 during \( \mu_6^{+1} = 5.52'' \), (Right) North Side at \( \mu_6^{-1} = -5.54'' \)
Figure 3.805 T27 – (Left) Fracture of Previously Buckled Bar S3 during ($\mu_6^{-2} = -5.53\)”, (Right) Fracture of Previously Buckled South Bars N3 and N4 during ($\mu_6^{+3}$)

Figure 3.806 T27 – After the Test (Left) South Side and (Right) North Side
Vertical strain profiles for the north extreme fiber bar N3, which is placed into tension during push cycles, appear in the right half of Figure 3.808. This figure shows both extreme fiber bars on the same graph to illustrate the effects of tension shift. Compression strains are concentrated near the footing-column interface while tension strains are spread higher above the footing following the inclined flexural-shear crack distribution. Compressive vertical strain profiles for north extreme fiber bar N3 during pull cycles appear in the left half of Figure 3.809. A peak tensile strain of 0.0361 was measured 7.56” above the footing on bar N3 during ($\mu_5^{+1} = 4.60^\circ$), before the bar visibly buckled during ($\mu_5^{-1} = -4.60^\circ$). The relationship between tension strain and displacement for this gage length appears in Figure 3.812. Moment-curvature analysis with the PCK (2007) Lp Hinge Method begins to over
predict the measured tensile strains at an increasing rate beyond displacement ductility two. A compressive strain of $-0.0168$ was measured 1.70" above the footing on bar N3 during ($\mu_3^{-1} = -2.76"$), when the first spiral in the confinement region yielded.

Measured spiral strains in six layers which overlaid the north extreme fiber bar appear in Figure 3.811. Spiral tension strains increased during each successive pull cycle of displacement ductility four. The relationship between compressive strain and displacement for the bar N3 gage length 5.57" above the footing appears in Figure 3.814. The measured compressive strains match the moment-curvature prediction with the PCK (2007) Lp Hinge Method through ($\mu_4^{-1} = -3.67"$), but begin to deviate during the second and third pull cycles. A diagram depicting the influence of measurable outward deformation on three adjacent gage lengths is shown in Figure 3.822. Inspection of compressive strain profiles for bar N3 point out that the first three gage lengths above the footing were influenced by measurable deformation before visible bar buckling, Figure 3.809. A peak compressive strain of $-0.0322$ was measured 1.70" above the footing on bar N3 during ($\mu_4^{-3} = -3.67"$). It is likely that this compressive strain is influenced by measurable deformation, but it is unclear why the deformation was observed before yielding of the transverse steel, Figure 3.813.

Strain hysteresis for gage lengths 3.59", 5.57" and 7.56" above the footing on bar N3 appear in Figure 3.818, Figure 3.819, and Figure 3.820. The hysteresis remained stable through ($\mu_4^{-1} = -3.67"$), when the peak spiral tension strain measured by a strain gage reached 0.0048 for the layer 3.78" above the footing, Figure 3.821. The hysteresis contains spiral data from a strain gage and an Optotrak gage length, Figure 3.783. The Optotrak strains were calculated using arc-lengths obtained from measured 3D distance chord lengths and the known outside diameter of the spiral. It is important to note that arc-length calculations become inaccurate once severe yielding in the spiral leads to the reinforcement straightening out to the left and right of the localized yielding directly over the bar where the strain gage is located. The bar N3 strain hystereses and the overlaying spiral strain hysteresis show a major deviation during ($\mu_5^{-1} = -4.60"$) when visible bar buckling was observed,
Figure 3.801. The distribution of arc-strains measured around the circumference of the second and third spiral layers above the footing during compressive pull cycles appear in Figure 3.829 and Figure 3.831 respectively. The north side of the specimen is on the left side of the graph with negative location values, specific locations of bars N2, N3, and N4 are highlighted with vertical dashed lines. The largest spiral tension strains were measured directly over reinforcing bars, and inelastic spiral strains are concentrated in the compressive zone with elastic strains near the center of the section.

South Reinforcement

Vertical strain profiles for the south extreme fiber bar S3, which is placed into tension during pull cycles, appear in the right half of Figure 3.809. Compressive vertical strain profiles for south extreme fiber bar S3 during push cycles appear in the left half of Figure 3.808. The measured compressive strains for bar S3 appear to be influenced by measurable deformation which occurred after yielding of the transverse steel. A diagram which shows the location and effect of measurable deformation in Bar S3 appears in Figure 3.822. The first and third gage lengths above the footing had increased compression while the second gage length had additional tension during pull cycles. This behavior is observable in the measured strain hysteresis 1.77” (Figure 3.823), 3.60” (Figure 3.824), and 5.48” (Figure 3.825) above the footing. As a comparison, the gage length 7.40” above the footing on bar S3, Figure 3.826, appears to be unaffected by measurable deformation.

The peak tension strain before bar buckling of 0.0243 was measured 3.60” above the footing on bar S3 during ($\mu_{4}^{-3} = -3.67\)"). The relationship between tension strain and displacement for this gage lengths 3.60” and 11.23” above the footing appear in Figure 3.815 and Figure 3.816. Moment-curvature analysis with the PCK (2007) Lp Hinge Method begins to over predict the measured tension strains during displacement ductility two. A compressive strain of -0.0124 was measured 1.77” above the footing during ($\mu_{3}^{1} = -2.76\)"), when the first spiral layer in the south confinement region yielded. The peak compression strain of -0.0228, measured 1.77” above the footing on bar S3, may be unreliable due to deformation. The relationship between compressive strain and displacement for this gage
length appears in Figure 3.817. The measurable deformation led to compressive strains which significantly exceed the moment-curvature prediction, with the PCK (2007) Lp Hinge Method, in this gage length as well as the one 5.48” above the footing.

Measured strains in six spiral layers which overlaid the south extreme fiber bar appear in Figure 3.810 for push cycles. Cycles during displacement ductility four produced successively larger inelastic demands on the bottom three layers of spiral reinforcement. Strain gage and Optotrak strain hysteresis for the spiral layer 3.63” above the footing appear in Figure 3.827. The two measurement methods match well until ($\mu_4^+ = 3.68”$), when the effects of measurable deformation became apparent and presumably the arc-strains no longer represent the geometry of the spiral over bar S3. The spiral strains spiked during ($\mu_5^+ = 4.60”$) when visible bar buckling was observed, Figure 3.800, and increased more significantly as the buckled deformation grew during ($\mu_5^{+2} = 4.61”$). Measured arc-strains around the circumference of the second and third spiral layers on the south side of the specimen during push cycles appear in Figure 3.828 and Figure 3.830. The largest inelastic spiral strains were localized over bars S2, S3, and S4 which are shown on the right side of the figures with vertical dashed lines.
Figure 3.808  T27 – Extreme Fiber Bar Vertical Strain Profiles during Push Cycles

Figure 3.809  T27 – Extreme Fiber Bar Vertical Strain Profiles during Pull Cycles
Figure 3.810  T27 – Spiral Strains on the South Side during Push Cycles

Figure 3.811  T27 – Spiral Strains on the North Side during Pull Cycles
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Figure 3.812  T27 – Tensile Strain-Displacement for Bar N3 during Push Cycles

Figure 3.813  T27 – Compressive Strain-Displacement for Bar N3 during Pull Cycles
Figure 3.814 T27 – Compressive Strain-Displacement for Bar N3 during Pull Cycles

Figure 3.815 T27 – Tensile Strain-Displacement for Bar S3 during Pull Cycles
Figure 3.816 T27 – Tensile Strain-Displacement for Bar S3 during Pull Cycles

Figure 3.817 T27 – Compressive Strain-Displacement for Bar S3 during Push Cycles
Figure 3.818  T27 – Bar N3 Strain Hysteresis to Buckling (3.59” Above the Footing)

Figure 3.819  T27 – Bar N3 Strain Hysteresis to Buckling (5.57” Above the Footing)
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Figure 3.820  T27 – Bar N3 Strain Hysteresis to Buckling (7.56” Above the Footing)

Figure 3.821  T27 – Spiral Strain Hysteresis over North Buckled Region (3.78” Above)
Figure 3.822 T27 – Location of Measurable Deformation and Bar S3 Buckled Shape

Figure 3.823 T27 – Bar S3 Strain Hysteresis to Buckling (1.77” Above the Footing)
Figure 3.824  T27 – Bar S3 Strain Hysteresis to Buckling (3.60” Above the Footing)

Figure 3.825  T27 – Bar S3 Strain Hysteresis to Buckling (5.48” Above the Footing)
Figure 3.826 T27 – Bar S3 Strain Hysteresis to Buckling (7.40” Above the Footing)

Figure 3.827 T27 – Spiral Strain Hysteresis over South Buckled Region (3.63” Above)
Figure 3.828  T27 – Spiral Strain Distribution for the 2nd Spiral above the Footing during Push Cycles (Positive = South)

Figure 3.829  T27 – Spiral Strain Distribution for the 2nd Spiral above the Footing during Pull Cycles (Negative = North)
Figure 3.830  T27 – Spiral Strain Distribution for the 3rd Spiral above the Footing during Push Cycles (Positive = South)

Figure 3.831  T27 – Spiral Strain Distribution for the 3rd Spiral above the Footing during Pull Cycles (Negative = North)
Cross section strain profiles for the second horizontal section above the footing appear in Figure 3.832 and Figure 3.833 for push and pull cycles respectively. This is the first horizontal section above the footing with instrumented gage lengths on bars S0 and N0. The curvature is calculated as the slope of the least squared line connecting strains measured in twelve instrumented reinforcing bars. If the curvatures for many horizontal cross sections are analyzed, curvature profiles for the plastic hinge region can be constructed; Figure 3.834 and Figure 3.835 for push and pull cycles respectively. Plastic curvatures were found to follow a linear distribution. Linear least squared error plastic curvature lines were fit to the plastic portion of the measured curvature profiles. The extrapolation of this linear curvature line with the footing-column interface was taken as the base section curvature. As the base section curvature ductility increased, the height at which the linear plastic curvature distribution intersected the elastic curvature profile also increased. Circular data points in Figure 3.839 plot the measured spread of plasticity as a function of base section curvature ductility.

Curvature profiles describe the elastic and plastic flexural displacements of the column, but do not address fixed-end rotations which result from development of longitudinal reinforcement into the footing. The measured vertical displacements of Optotrak LEDs placed closest to the footing column interface can be used to quantify this fixed-end rotation, Figure 3.836 and Figure 3.837. The fixed-end rotation is taken as the slope of the least squared error line fit to the bond slip profile. If an elastic curvature profile assumption is made for curvatures higher than those measured with instrumentation, then the entire curvature profile may be integrated to obtain the total column flexural displacement. This column flexural displacement was added to the strain penetration displacement, and compared to the experimentally measured displacements in Figure 3.838. The Optotrak integrated displacement matches well with those obtained from a string potentiometer placed at the center of the lateral load, which indicates that shear deformation are small.
Figure 3.832  T27 – Strain Profiles during Push Cycles for Section 3.58” Above Footing

Figure 3.833  T27 – Strain Profiles during Pull Cycles for Section 3.58” Above Footing
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Figure 3.834 T27 – Push Cycle Curvature Profiles with Linear Plastic Regression

Figure 3.835 T27 – Pull Cycle Curvature Profiles with Linear Plastic Regression
Figure 3.836  T27 – Base Rotation due to Strain Penetration during Push Cycles

Figure 3.837  T27 – Base Rotation due to Strain Penetration during Pull Cycles
Figure 3.838  T27 – Comparison of Measured and Optotrak Integrated Displacements

Figure 3.839  T27 – Measured Spread of Plasticity (Circular Data Points)
3.4.4 Test 28 – 18” Dia. Column with 1.7% Long. Steel and 15% Axial Load

Table 3.53 Observations for Test 28 – 18” Dia. With 1.7% Steel and 15% Axial Load

<table>
<thead>
<tr>
<th>VALUES OF INTEREST:</th>
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<tbody>
<tr>
<td>Concrete Compressive Strength: ( f_c' = 6239 \text{ psi} )</td>
</tr>
<tr>
<td>Axial Load: ( P = 212.4 \text{ kips} ) ( (P/(f_c'A_g) = 15%) )</td>
</tr>
<tr>
<td>Longitudinal Steel Content: ( 10 #6 \text{ Bars} ) ( (A_{st}/A_g = 1.7%) )</td>
</tr>
<tr>
<td>Analytical First Yield Force: ( F_y' = 25.17 \text{ kips} )</td>
</tr>
<tr>
<td>Experimental First Yield Displacement: ( \Delta_y' = 1.05&quot; )</td>
</tr>
<tr>
<td>Analytical Nominal Moment Capacity: ( M_n = 255.23 \text{ kip} \times \text{ft} )</td>
</tr>
<tr>
<td>Equivalent Yield Displacement: ( \Delta_y = 1.34&quot; )</td>
</tr>
<tr>
<td>Maximum Lateral Force: ( 31.94 \text{ kips} )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>DAMAGE OBSERVATIONS:</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Cracking North: ( 3/4Fy' = 0.60&quot; )</td>
</tr>
<tr>
<td>First Cracking South: ( -1/2Fy' = -0.33&quot; )</td>
</tr>
<tr>
<td>Cover Concrete Crushing North: ( \mu_{15}^{-3} = -2.00&quot; )</td>
</tr>
<tr>
<td>Cover Concrete Crushing South: ( \mu_{15}^{+3} = 2.00&quot; )</td>
</tr>
<tr>
<td>Transverse Steel Yield North: ( \text{At } -3.72&quot; \text{ during pull to } \mu_{3}^{-1} = -4.00&quot; )</td>
</tr>
<tr>
<td>Transverse Steel Yield South: ( \text{At } 3.20&quot; \text{ during push to } \mu_{3}^{+1} = 4.00&quot; )</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North: ( \text{Reversal from } \mu_{5}^{+2} = 6.68&quot; )</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South: ( \text{Reversal from } \mu_{4}^{-3} = -5.34&quot; )</td>
</tr>
<tr>
<td>Longitudinal Bar Fracture North: ( \text{At } 6.92&quot; \text{ during push to } \mu_{6}^{+2} = 8.00&quot; )</td>
</tr>
<tr>
<td>Longitudinal Bar Fracture South: ( \text{At } -2.94&quot; \text{ during pull to } \mu_{6}^{-2} = -8.01&quot; )</td>
</tr>
</tbody>
</table>

\*\( \mu_{5}^{+2} = 6.68" \) represents the second push cycle of displacement ductility five
Table 3.54 Strain Data Summary for Test 28 – 18” Dia. with 1.7% Steel and 15% Axial

<table>
<thead>
<tr>
<th>MATERIAL STRAINS:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North: $\varepsilon_s = 0.0051$ (compression)</td>
</tr>
<tr>
<td>Cover Concrete Crushing South: $\varepsilon_s = 0.0055$ (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield North: $\varepsilon_s = 0.0123$ (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield South: $\varepsilon_s = 0.0143$ (compression)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North: $\varepsilon_s = 0.036$ (peak tension prior to $bb$)</td>
</tr>
<tr>
<td>$\varepsilon_s = 0.034$ (peak comp. prior to $bb$)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South: $\varepsilon_s = 0.030$ (peak tension prior to $bb$)</td>
</tr>
<tr>
<td>$\varepsilon_s = 0.024$ (peak comp. prior to $bb$)</td>
</tr>
<tr>
<td>Mander (1988) Ultimate Concrete Compression Strain, $\varepsilon_{cu} = 0.0194$</td>
</tr>
</tbody>
</table>

Figure 3.840 T28 – Test Setup and Cross Section Bar Designation
Figure 3.841 T28 – Target Marker Application and Optotrak Spatial Output

Figure 3.842 T28 – Compressive Axial Load from One Jack (Total = 2*Value)
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Figure 3.843 T28 – Symmetric Three Cycle Set Load History

Figure 3.844 T28 – Lateral Force vs. Top Column Displacement Response
Test 28 – 18” Dia. with 1.7% Steel and 15% Axial Load – Experimental Observations

Specimens 25-30 focus on the effects of longitudinal steel content, longitudinal bar diameter, and higher levels of axial load on column behavior. The 18” diameter column chosen for Test 28 contains 10 #6 (A706) bars for longitudinal reinforcement ($A_{st}/A_g = 1.7\%$) and a #3 A706 spiral at 2” on center ($4A_{sp}/(D's) = 1.3\%$). The specimen had an 8ft cantilever length ($L/D = 5.33$), and was subjected to $P/(f'_cA_g) = 15\%$ axial load. The experiments utilized a quasi-static displacement controlled loading procedure. The symmetric three-cycle-set load history is commonly used to evaluate the seismic performance of structural components. The load history begins with elastic cycles to the following increments of the analytically predicted first yield force: $\frac{1}{4} F'_y$, $\frac{1}{2} F'_y$, $\frac{3}{4} F'_y$, and $F'_y$. The experimental first yield displacement is then determined by taking the average of the recorded displacements during the first yield push and pulls cycles. The equivalent yield
displacement, used to determine the displacement ductility levels ($\mu_{\Delta 1} = 1 \ast \Delta y$), is then calculated as $\Delta y = \Delta'_y \left( M_n / M'_y \right)$. The symmetric three-cycle-set load history resumes with three balanced cycles at each of the following displacement ductility levels: 1, 1.5, 2, 3, 4, 5, etc. The lateral displacement history and resulting hysteretic response for Test 28 appear in Figure 3.843 and Figure 3.844. Previous Tests 19 and 20 contained similar geometry and detailing, but had 10% and 5% axial load. A hysteretic comparison of the Tests 19, 20 and 28 is shown in Figure 3.845.

![Figure 3.846 T28 – Strain Profiles before Yield, (Left) North and (Right) South](image)

The test began with cycles in $\frac{1}{4} Fy'$ (first yield force) increments in each direction of loading until the first yield force was reached. The first cracks on the north and south sides of the specimen formed during $(3/4Fy' = 0.60")$ and $(-1/2Fy' = -0.33")$ respectively, Figure 3.847. The crack distribution on all sides of the specimen at first yield, $(Fy' = 1.04")$ and $(-Fy' = -1.07")$, appears in Figure 3.848. Tensile vertical strain profiles for north and south reinforcing bars during elastic push and pull cycles appear in Figure 3.846. The average experimental first yield displacement was used to calculate the equivalent yield displacement, $\Delta y = \Delta'_y \left( M_n / M'_y \right) = 1.05"$, which defined the reversal amplitudes for reminder of the test.
The crack progression at displacement ductility 1, 1.5, 2, 3, and 4 appear in Figure 3.849, Figure 3.851, Figure 3.853, Figure 3.855, and Figure 3.857 respectively. During these cycles the cracks became more numerous and increased in inclination on the shear faces of the specimen. Small amounts concrete flaking was observed on the south and north sides of the specimen during \( \mu_1^+ = 2.00'' \) and \( \mu_1^- = -2.00'' \), Figure 3.850. In previous tests, this flaking leads to crushing during subsequent cycles, but crushing was not observed until displacement ductility two. Crushing on the south and north sides of the specimen increased at \( \mu_2^+ = 2.66'' \) and \( \mu_2^- = -2.65'' \), Figure 3.852. As compressive demands increased during displacement ductility 1.5 to 4, crushing gradually increased on each side of the specimen. Compressive demand during \( \mu_3^+ = 4.00'' \) and \( \mu_3^- = -4.00'' \) led to spiral yielding in confinement regions, Figure 3.854. Spiral strains on each side of the specimen increased during each successive cycle of ductility four, but the extreme fiber reinforcement remained visibly straight, Figure 3.856.

The south extreme fiber bar S3 visibly buckled during \( \mu_5^+ = 6.68'' \), as shown in the left photo of Figure 3.858. The outward buckled region occurred over the second and third gage lengths above the footing. The buckled deformation of bar S3 increased during \( \mu_5^+ = 6.68'' \), Figure 3.859. The north extreme fiber bar visibly buckled after reversal from \( \mu_5^+ = 6.68'' \), Figure 3.860. The buckled deformation of bar N3 increased over the fourth gage length above the footing during \( \mu_3^- = -6.69'' \), Figure 3.860. The previously buckled south extreme fiber bar ruptured during \( \mu_6^- = -8.01'' \), Figure 3.861. Bar S3 ruptured at the same location where the outward deformation was the largest during \( \mu_6^+ = 8.00'' \). An additional north reinforcing bar, N2, buckled during \( \mu_6^- = -8.01'' \), Figure 3.862. The previously buckled north extreme fiber bar N3 ruptured during \( \mu_6^+ = 8.00'' \), Figure 3.863. Two additional south reinforcing bars, S2 and S4, buckled during \( \mu_6^+ = 8.00'' \), Figure 3.863. At this time the test was concluded with ruptured reinforcement on each side of the specimen which led to significant losses in strength in each direction of loading. Photos of the specimen after the instrumentation was removed appear in Figure 3.864 and Figure 3.865.
Figure 3.847  T28 – (Left) Cracking on the South Side during \((-1/2Fy' = -0.33")\),
(Right) North Cracking \((3/4Fy' = 0.60")\)

Figure 3.848  T28 – (Left) North Crack Distribution at \((Fy' = 1.04")\), (Middle and
Right) Crack Distributions on the Front and Right Sides at \((-Fy' = -1.07")\)
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Figure 3.849  T28 – (Left) North Crack Distribution at ($\mu_{1}^{+3} = 1.34"$), (Middle and Right) Crack Distributions on the Front and Right Sides at ($\mu_{1}^{-3} = -1.34"$)

Figure 3.850  T28 – (Left) South Concrete Flaking during ($\mu_{1.5}^{+3} = 2.00"$), (Right) North Concrete Flaking during ($\mu_{1.5}^{-3} = -2.00"$)
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Figure 3.851 T28 – (Left) North Crack Distribution at \((\mu^{+3}_{1.5} = 2.00")\), (Middle and Right) Crack Distributions on the Front and Right Sides at \((\mu^{-3}_{1.5} = -2.00")\)

Figure 3.852 T28 – (Left) South Crushing during \((\mu^{+1}_{2} = 2.66")\), (Right) North Crushing during \((\mu^{-1}_{2} = -2.65")\)
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Figure 3.853 T28 – (Left) North Crack Distribution at ($\mu_{2}^{+3} = 2.67\prime\prime$), (Middle and Right) Crack Distributions on the Front and Right Sides at ($\mu_{2}^{-3} = -2.66\prime\prime$)

Figure 3.854 T28 – (Left) South Spiral Yield at ($\mu_{3}^{+1} = 4.00\prime\prime$), (Right) North Spiral Yield at ($\mu_{3}^{-1} = -4.00\prime\prime$)
Figure 3.855  T28 – (Left) North Crack Distribution at \( \mu_3^+ = 4.00'' \), (Middle and Right) Crack Distributions on the Front and Right Sides at \( \mu_3^- = -4.00'' \)

Figure 3.856  T28 – (Left) Bar S3 Remained Visibly Straight during \( \mu_4^+ = 5.34'' \), (Right) Bar N3 Remained Visibly Straight during \( \mu_4^- = -5.34'' \)
Figure 3.857  T28 – (Left) North Crack Distribution at $\mu_4^+ = 5.34^\prime$), (Middle and Right) Crack Distributions on the Front and Right Sides at $\mu_4^- = -5.34^\prime$)

Figure 3.858  T28 – (Left) Visible Buckling of Bar S3 during $\mu_5^+ = 6.68^\prime$), (Right) Bar N3 Remained Visibly Straight during $\mu_5^- = -6.68^\prime$)
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Figure 3.859 T28 – (Left) Lateral Deformation and (Right) Increased Deformation in Bar S3 during ($\mu_{5}^{+2} = 6.68"$)

Figure 3.860 T28 – (Left) Visible Buckling of Bar N3 during ($\mu_{5}^{-2} = -6.69"$), (Right) Increased Deformation in Bar N3 during ($\mu_{5}^{-3} = -6.69"$)
Figure 3.861 T28 – (Left) Increased Deformation in Bar S3 ($\mu_0^{+1} = 8.00"$), (Right) Fracture of Bar S3 during ($\mu_0^{-1} = -8.01"$)

Figure 3.862 T28 – (Left) Increased Deformation in Bar N3 during ($\mu_0^{-1} = -8.01"$), (Right) Buckling of Adjacent Bar N2 during ($\mu_0^{+1} = -8.01"$)
Figure 3.863  T28 – (Left) Fracture of Bar N3 during $\mu_0^{+2} = 8.00^\prime$, (Right) Buckling of Adjacent Bars S2 and S4 during $\mu_0^{+2} = 8.00^\prime$.

Figure 3.864  T28 – After the Test (Left) South Side and (Right) North Side
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Figure 3.865  T28 – After the Test (Left) Front Side and (Right) Back Side

Test 28 – 18” Dia. with 1.7% Steel and 15% Axial Load – Strain Data Analysis

North Reinforcement

Vertical strain profiles for the north extreme fiber bar N3, which is placed into tension during push cycles, appear in the right half of Figure 3.866. This figure shows both extreme fiber bars on the same graph. Compressive vertical strain profiles for north extreme fiber bar N3 during pull cycles appear in the left half of Figure 3.867. A peak tensile strain of 0.0362 was measured 3.99” above the footing on bar N3 during (μ₅² = 6.68”), before bar buckling was observed during (μ₅⁻² = −6.69”). The relationship between tension strain and displacement for the gage lengths 2.04” and 5.92” above the footing appear in Figure 3.870 and Figure 3.871. Moment-curvature analysis with the PCK (2007) Lp Hinge Method begins to over predict the measured tensile strains at an increasing rate beyond displacement ductility two for both gage lengths. A compressive strain of −0.012 was measured 3.99” and 9.83” above the footing on bar N3 during (μ₅⁻¹ = −4.00”), when the first spiral in the confinement region yielded. The relationship between compressive strain and displacement
for the bar N3 gage length 9.83” above the footing appears in Figure 3.872. The measured compressive strains exceed the moment-curvature prediction with the PCK (2007) Lp Hinge Method beyond displacement ductility two. Successive pull cycles during displacement ductility four produced larger compressive strains. The gage length 9.83” above the footing was located just above the outward buckled deformation, Figure 3.860, suggesting that the measured strains may have been influenced by deformation before visible buckling.

Measured spiral strains in six layers which overlaid the north extreme fiber bar appear in Figure 3.869. Spiral tension strains in the third and fourth layers above the footing increased during each successive pull cycle of displacement ductility four. A spiral strain hysteresis for the layer 7.97” above the footing which overlaid the outward deformed region of bar N3 appears in Figure 3.878. The hysteresis contains spiral data from a strain gage and an Optotrak gage length, Figure 3.841. The Optotrak strains were calculated using arc-lengths obtained from measured 3D distance chord lengths and the known outside diameter of the spiral. It is important to note that arc-length calculations become inaccurate once severe yielding in the spiral leads to the reinforcement straightening out to the left and right of the localized yielding directly over the bar where the strain gage is located. Each consecutive pull cycle of displacement ductility four led to larger inelastic spiral strains, which indicates that measurable outward deformation occurred before visible bar buckling. The sharpest increase in measured spiral strains occurred during \((\mu_5^2 = -6.69")\), when visible bar buckling was observed and the strain gage debonded preventing further measurement. The distribution of arc-strains measured around the circumference of the third and fourth spiral layers above the footing during pull cycles appear in Figure 3.887 and Figure 3.888 respectively. The north side of the specimen is on the left side of the graph with negative location values and specific locations of bars N2, N3, and N4 are highlighted with vertical dashed lines.

Strain hysteresis for gage lengths 5.92”, 7.87” and 9.83” above the footing on bar N3 appear in Figure 3.875, Figure 3.876, and Figure 3.877. The gage lengths 5.92” and 7.87” above the footing overlaid the region which outwardly deformed during \((\mu_5^2 = -6.69")\),
Figure 3.860. The diagram in Figure 3.879 depicts the effect of buckled deformation on strains measured by LEDs attached to the outside surface of reinforcing bars. Gage lengths in the outward deformed region show increased tension during compressive cycles and gage lengths just above show additional compression. The hysteresis 7.87” above the footing remained stable until \( \mu_5^{-1} = -6.69’’ \) when measured compressive strains deviated from previous trends. By contrast, measurable deformation in the gage length 9.83” above the footing, Figure 3.877, started during displacement ductility three, increased during successive cycles of ductility four, and spiked as the bar buckled during \( \mu_5^{-2} = -6.69’’ \). The compressive strain values for this gage length may be influenced by measurable deformation, and may not represent the actual level of compression since similar spikes were not observed in adjacent bars N2 and N4.

**South Reinforcement**

Vertical strain profiles for the south extreme fiber bar S3, which is placed into tension during pull cycles, appear in the right half of Figure 3.867. Compressive vertical strain profiles for south extreme fiber bar S3 during push cycles appear in the left half of Figure 3.866. The peak tension strain of 0.0358 was measured 9.50” above the footing on bar S3 during \( \mu_5^{-1} = -6.68’’ \). It is important to note that visible bar buckling was observed during \( \mu_5^{+1} = 6.68’’ \), Figure 3.858. The buckled deformation was small, and the bar was expected to straighten out during \( \mu_5^{-1} = -6.68’’ \). The peak tension strain prior to visible bar buckling of 0.0299 was measured 5.76” above the footing during \( \mu_5^{+1} = -5.34’’ \). The relationship between tension strain and displacement for the gage length 7.65” above the footing appears in Figure 3.873. Moment-curvature analysis with the PCK (2007) Lp Hinge Method begins to over predict the measured tension strains at an increasing rate beyond displacement ductility two. A compressive strain of -0.0143 was measured 1.93” above the footing when the first confinement steel yielded on the south side of the specimen during \( \mu_3^{+1} = 4.00’’ \). A peak compressive strain of -0.0243 was measured over this same gage length during \( \mu_4^{+3} = 5.34’’ \). Moment-curvature analysis with the PCK (2007) Lp Hinge
Method under predicts the measured compressive strains in the gage length 1.93” above the footing, Figure 3.874.

Measured strains in six spiral layers which overlaid the south extreme fiber bar appear in Figure 3.868 for push cycles. Cycles during displacement ductility four produced successively larger inelastic demands on the second and third spiral layers. Strain gage and Optottrak strain hysteresis for the spiral layer 3.89” and 5.84” above the footing appear in Figure 3.883 and Figure 3.884 respectively. The two measurement methods match well until \((\mu_4^+ = 5.35”)\), when the effects of measurable deformation became apparent and presumably the arc-strains no longer represent the geometry of the spiral over bar S3. The spiral strains increased during \((\mu_5^+ = 4.60”)\) when visible bar buckling was observed, Figure 3.858, and increased more significantly as the buckled deformation grew during \((\mu_5^+ = 6.68”)\). Measured arc-strains around the circumference of the second and third spiral layers on the south side of the specimen during push cycles appear in Figure 3.885 and Figure 3.886. The largest inelastic spiral strains were localized over bars S2, S3, and S4 which are shown on the right side of the figures with vertical dashed lines.

The influence of measurable deformation before bar buckling becomes easier to understand when the specific gage lengths in question are located on buckled bar, Figure 3.879. The second and third gage lengths above the footing overlay the outward deformed region. The strain hysteresis for the gage length 5.76” above the footing, Figure 3.881, shows a significant deviation during \((\mu_5^+ = 6.68”)\), but not during \((\mu_5^+ = 6.68”)\) when visible bar buckling was first observed. The gage lengths 1.93” (Figure 3.880) and 7.65” (Figure 3.882) above the footing show additional compression during push cycles for the regions just above and below the outward deformation observed during bar buckling. The largest spiral strains are not measured in layers overlaying the peak compressive gage lengths, but rather the second and third spiral layers which encompassed the outward deformed region, Figure 3.868.
Figure 3.866  T28 – Extreme Fiber Bar Vertical Strain Profiles during Push Cycles

Figure 3.867  T28 – Extreme Fiber Bar Vertical Strain Profiles during Pull Cycles
Figure 3.868  T28 – Spiral Strains on the South Side during Push Cycles

Figure 3.869  T28 – Spiral Strains on the North Side during Pull Cycles
Chapter 3: Experimental Observations

Figure 3.870  T28 – Tensile Strain-Displacement for Bar N3 during Push Cycles

Figure 3.871  T28 – Tensile Strain-Displacement for Bar N3 during Push Cycles
Figure 3.872 T28 – Compressive Strain-Displacement for Bar N3 during Pull Cycles

Figure 3.873 T28 – Tensile Strain-Displacement for Bar S3 during Pull Cycles
Figure 3.874  T28 – Compressive Strain-Displacement for Bar S3 during Push Cycles

Figure 3.875  T28 – Strain Hysteresis for Bar N3 to Buckling (5.92” Above the Footing)
Visible Buckling of Bar N3

Measured Outward Deformation Leads to Additional Tension

Figure 3.876  T28 – Strain Hysteresis for Bar N3 to Buckling (7.87” Above the Footing)

Visible Buckling of Bar N3

Measured Inward Deformation Leads to Additional Compression

Figure 3.877  T28 – Strain Hysteresis for Bar N3 to Buckling (9.83” Above the Footing)
Figure 3.878  T28 – Spiral Strain Hysteresis over North Buckled Region (7.97” Above)

Figure 3.879  T28 – Measureable Deformation Matches Buckled Shape of Bar S3
Chapter 3: Experimental Observations

Figure 3.880  T28 – Strain Hysteresis for Bar S3 to Buckling (1.93” Above the Footing)

Figure 3.881  T28 – Strain Hysteresis for Bar S3 to Buckling (5.76” Above the Footing)
Figure 3.882  T28 – Strain Hysteresis for Bar S3 to Buckling (7.65” Above the Footing)

Figure 3.883  T28 – Spiral Strain Hysteresis over South Buckled Region (3.89” Above)
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**Figure 3.884** T28 – Spiral Strain Hysteresis over South Buckled Region (5.84” Above)

![Graph showing strain hysteresis over a south buckled region with visible buckling.](image)

**Figure 3.885** T28 – Spiral Strain Distribution for the 2\textsuperscript{nd} Spiral above the Footing during Push Cycles (Positive = South)

![Graph showing strain distribution along the spiral circumference during push cycles.](image)
Chapter 3: Experimental Observations

Figure 3.886 T28 – Spiral Strain Distribution for the 3rd Spiral above the Footing during Push Cycles (Positive = South)

Figure 3.887 T28 – Spiral Strain Distribution for the 3rd Spiral above the Footing during Pull Cycles (Negative = North)
Test 28 – Curvature and Strain Penetration Data

Cross section strain profiles for the third horizontal section above the footing appear in Figure 3.889 and Figure 3.890 for push and pull cycles respectively. The curvature is calculated as the slope of the least squared line connecting strains measured in eight instrumented reinforcing bars. If the curvatures for many horizontal cross sections are analyzed, curvature profiles for the plastic hinge region can be constructed; Figure 3.891 and Figure 3.892 for push and pull cycles respectively. Linear least squared error regression lines were fit to the plastic portion of the measured curvature profiles. The extrapolation of this linear curvature line with the footing-column interface was taken as the base section curvature. As the base section curvature ductility increased, the height at which the linear plastic curvature distribution intersected the elastic curvature profile also increased. Circular data points in Figure 3.896 plot the measured spread of plasticity as a function of base section curvature ductility.
Curvature profiles describe the elastic and plastic flexural displacements of the column, but do not address fixed-end rotations which result from development of longitudinal reinforcement into the footing. The measured vertical displacements of Optotrak LEDs placed closest to the footing column interface can be used to quantify this fixed-end rotation, Figure 3.893 and Figure 3.894. The fixed-end rotation is taken as the slope of the least squared error line fit to the bond slip profile. The strain penetration displacement is obtained by multiplying this rotation by the cantilever height of the column. If an elastic curvature profile assumption is made for curvatures higher than those measured with instrumentation, then the entire curvature profile may be integrated to obtain the total column flexural displacement. This column flexural displacement was added to the strain penetration displacement, and compared to the experimentally measured displacements in Figure 3.895. The Optotrak integrated displacement matches well with those measured using a string potentiometer at the center of loading, which indicates that shear deformation are small.

Figure 3.889  T28 – Push Cycle Strain Profiles for Section 5.86” Above the Footing
Chapter 3: Experimental Observations

Figure 3.890  T28 – Pull Cycle Strain Profiles for Section 5.86” Above the Footing

Figure 3.891  T28 – Push Cycle Curvature Profiles with Linear Plastic Regression
Chapter 3: Experimental Observations

Figure 3.892  T28 – Pull Cycle Curvature Profiles with Linear Plastic Regression

Figure 3.893  T28 – Base Rotation due to Strain Penetration during Push Cycles
Figure 3.894  T28 – Base Rotation due to Strain Penetration during Pull Cycles

Figure 3.895  T28 – Comparison of Measured and Optotrak Integrated Displacements
Figure 3.896  T28 – Measured Spread of Plasticity (Circular Data Points)
3.4.5 Test 29 – 18” Dia. Column with 1.7% Long. Steel and 20% Axial Load

Table 3.55 Observations for Test 29 – 18” Dia. With 1.7% Steel and 20% Axial Load

VALUES OF INTEREST:

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<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Compressive Strength: $f'_c$</td>
<td>5911 psi</td>
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<tr>
<td>Axial Load: $P = 268.4$ kips ($P/(f'_c A_g) = 20%$)</td>
<td></td>
</tr>
<tr>
<td>Longitudinal Steel Content: 10 #6 Bars ($A_{st}/A_g = 1.7%$)</td>
<td></td>
</tr>
<tr>
<td>Analytical First Yield Force: $F'_y$</td>
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</tr>
<tr>
<td>Experimental First Yield Displacement: $\Delta'_y$</td>
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<td>Analytical Nominal Moment Capacity: $M_n$</td>
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<td>Equivalent Yield Displacement: $\Delta_y$</td>
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<td>Maximum Lateral Force:</td>
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DAMAGE OBSERVATIONS:

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<tr>
<th>Observation</th>
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<tr>
<td>First Cracking North:</td>
<td>3/4$Fy' = 0.60&quot;</td>
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<tr>
<td>First Cracking South:</td>
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<td>Cover Concrete Crushing North: $\mu_{1.5}^{-1}$</td>
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<td>Cover Concrete Crushing South: $\mu_{1.5}^{+1}$</td>
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<tr>
<td>Longitudinal Bar Buckling North: Reversal from $\mu_6^{+1} = 8.06&quot;$</td>
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<tr>
<td>Longitudinal Bar Buckling South: Reversal from $\mu_5^{-1} = -6.72&quot;$</td>
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<td>Longitudinal Bar Fracture North: Test Concluded Before North Bar Fractured</td>
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<tr>
<td>Longitudinal Bar Fracture South: At $-1.62&quot;$ during pull to $\mu_6^{-1} = -8.06&quot;$</td>
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*$\mu_5^{-1} = -6.72"$ represents the first pull cycle of displacement ductility five
Table 3.56 Strain Data Summary for Test 29 – 18” Dia. with 1.7% Steel and 20% Axial

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<td>Cover Concrete Crushing North:</td>
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<td>Cover Concrete Crushing South:</td>
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<td>Longitudinal Bar Buckling North:</td>
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<td>$\varepsilon_s = 0.044 \text{ (peak comp. prior to } \text{bb)}$</td>
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<tr>
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<tr>
<td></td>
<td>$\varepsilon_s = 0.032 \text{ (peak comp. prior to } \text{bb)}$</td>
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</table>

Mander (1988) Ultimate Concrete Compression Strain, $\varepsilon_{cu} = 0.020$

Figure 3.897 T28 – Test Setup and Cross Section Bar Designation
Figure 3.898 T28 – Target Marker Application and Optotrak Spatial Output

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Figure 3.899 T28 – Compressive Axial Load from One Jack (Total = 2*Value)
Chapter 3: Experimental Observations

Figure 3.900  T29 – Symmetric Three Cycle Set Load History

Figure 3.901  T29 – Lateral Force vs. Top Column Displacement Response
Figure 3.902 T29 – Comparison for T28 and T29 with Different Axial Load Levels

Test 29 – 18” Dia. with 1.7% Steel and 20% Axial Load – Experimental Observations

Specimens 25-30 focus on the effects of longitudinal steel content, longitudinal bar diameter, and higher levels of axial load on column behavior. This report summarizes experimental observations and data analysis for column Test 29. The 18” diameter column contains 10 #6 (A706) bars for longitudinal reinforcement ($A_{st}/A_g = 1.7\%$) and a #3 A706 spiral at 2” on center ($4A_{sp}/(D') = 1.3\%$). The specimen had an 8ft cantilever length ($L/D = 5.33$), and was subjected to ($P/(f'_c A_g) = 20\%$) axial load. The experiments utilized a quasi-static displacement controlled loading procedure. The symmetric three-cycle-set load history is commonly used to evaluate the seismic performance of structural components. The load history begins with elastic cycles to the following increments of the analytically predicted first yield force: $\frac{1}{4} F'_y$, $\frac{1}{2} F'_y$, $\frac{3}{4} F'_y$, and $F'_y$. The experimental first yield displacement is then determined by taking the average of the recorded displacements during
the first yield push and pulls cycles. The equivalent yield displacement, used to determine the displacement ductility levels \((\mu_{\Delta 1} = 1 \times \Delta_y)\), is then calculated as \(\Delta_y = \Delta'_y \left(\frac{M_n}{M_y'}\right)\). The symmetric three-cycle-set load history resumes with three balanced cycles at each of the following displacement ductility levels: 1, 1.5, 2, 3, 4, 5, etc. The displacement history and resulting hysteretic response for Test 29 appears in Figure 3.900 and Figure 3.901. Previous Tests 20, 19, and 28 contained similar geometry and detailing, but had 5, 10, and 15% axial load. A hysteretic comparison of Tests 28 and 29 appears in Figure 3.902.

![Figure 3.903 T29 – Strain Profiles before Yield, (Left) North and (Right) South](image)

The test began with cycles in \(\frac{1}{4} Fy'\) (first yield force) increments in each direction of loading until the first yield force was reached. The first cracks on the north and south sides of the specimen formed during \((3/4Fy' = 0.60")\) and \((-3/4Fy' = -0.62")\) respectively, Figure 3.904. The crack distribution on all sides of the specimen at first yield, \((Fy' = 1.07")\) and \((-Fy' = -1.10")\), appears in Figure 3.905. Tensile vertical strain profiles for north and south reinforcing bars during elastic push and pull cycles appear in Figure 3.903. The average experimental first yield displacement was used to calculate the equivalent yield displacement, \(\Delta_y = \Delta'_y \left(\frac{M_n}{M_y'}\right) = 1.08"\), which defined the reversal amplitudes for reminder of the test.
Chapter 3: Experimental Observations

The crack progression at displacement ductility 1, 1.5, 2, 3, and 4 appear in Figure 3.906, Figure 3.908, Figure 3.910, Figure 3.912, and Figure 3.914 respectively. During these cycles the cracks became more numerous and increased in inclination on the shear faces of the specimen. Small amounts concrete crushing was observed on the south and north sides of the specimen during \((\mu_1^{+1} = 2.01\text{''})\) and \((\mu_1^{-1} = -2.02\text{''})\), Figure 3.907. This crushing increased during the next two cycles of displacement ductility 1.5, Figure 3.908. As compressive demands increased during displacement ductility 1.5 to 4, crushing gradually increased on each side of the specimen. Compressive demand during \((\mu_3^{+1} = 4.03\text{''})\) and \((\mu_3^{-1} = -4.03\text{''})\) led to spiral yielding in confinement regions, Figure 3.911. Spiral strains on each side of the specimen increased during each successive cycle of ductility four, but the extreme fiber reinforcement remained visibly straight, Figure 3.913.

Even though spiral strains in confinement regions increased, the south extreme fiber bar remained visibly straight during \((\mu_5^{+1} = 6.72\text{''})\) and similarly the north bar during \((\mu_5^{-1} = -6.72\text{''})\), Figure 3.915. The south extreme fiber bar visibly buckled during \((\mu_5^{+2} = 6.72\text{''})\), Figure 3.916. The outward buckled region occurred over the third and fourth gage lengths above the footing. Adjacent south reinforcing bars S2 and S4 buckled during \((\mu_6^{+1} = 8.06\text{''})\) and the buckled deformation in bar S3 significantly increased, Figure 3.917. The south bar fractured during \((\mu_6^{-1} = -8.06\text{''})\), Figure 3.918, resulting in the first significant loss in strength. Visible buckling of north extreme fiber bar N3 and adjacent bar N2 was observed during \((\mu_6^{-1} = -8.06\text{''})\), Figure 3.918. At this point the test was concluded without fracturing north reinforcement, since it was clear that multiple cycles would be needed with a south extreme fiber region which already had significant damage. Photos of the specimen after the instrumentation was removed appear in Figure 3.919 and Figure 3.920.
Figure 3.904  T29 – (Left) Cracking on the North Side during \((3/4 F y' = 0.60")\),
(Right) South Cracking \((-3/4 F y' = -0.62")\)

Figure 3.905  T29 – (Left) North Crack Distribution at \((F y' = 1.07")\), (Middle and Right) Crack Distributions on the Front and Right Sides at \((-F y' = -1.10")\)
Figure 3.906  T29 – (Left) North Crack Distribution at ($\mu_{1}^{+3} = 1.35^\prime$), (Middle and Right) Crack Distributions on the Front and Right Sides at ($\mu_{1}^{-3} = -1.35^\prime$)

Figure 3.907  T29 – (Left) South Concrete Crushing during ($\mu_{15}^{+1} = 2.01^\prime$), (Right) North Crushing during ($\mu_{15}^{-1} = -2.02^\prime$)
Figure 3.908 T29 – (Left) North Crack Distribution at ($\mu_{1.5}^{-3} = 2.01''$), (Middle and Right) Crack Distributions on the Front and Right Sides at ($\mu_{1.5}^{+3} = -2.01''$)

Figure 3.909 T29 – Extent of Crushing during Displacement Ductility 1.5, (Left) South and (Right) North
Figure 3.910 T29 – (Left) North Crack Distribution at \( \mu_2^+ = 2.69'' \), (Middle and Right) Crack Distributions on the Front and Right Sides at \( \mu_2^- = -2.69'' \)

Figure 3.911 T29 – Spiral Layers which Yielded during Displacement Ductility Three, (Left) South and (Right) North
Figure 3.912 T29 – (Left) North Crack Distribution at \((\mu_3^+ = 4.02\text{"})\), (Middle and Right) Crack Distributions on the Front and Right Sides at \((\mu_3^- = -4.03\text{"})\)

Figure 3.913 T29 – (Left) Bar S3 Remained Visibly Straight during \((\mu_4^+ = 5.37\text{"})\), (Middle) Bar N3 Remained Visibly Straight during \((\mu_4^- = -5.37\text{"})\), (Right) Back of the Specimen at \((\mu_4^- = -5.37\text{"})\)
Figure 3.914 T29 – (Left) North Crack Distribution at ($\mu_{4}^{+3} = 5.37\"$), (Middle and Right) Crack Distributions on the Front and Right Sides at ($\mu_{4}^{-3} = -5.37\")

Figure 3.915 T29 – (Left) Bar S3 Remained Visibly Straight during ($\mu_{5}^{+1} = 6.72\"$), (Middle) Lateral Deformation at ($\mu_{5}^{-1} = -6.72\"$), (Right) Bar N3 Remained Visibly Straight during ($\mu_{5}^{-1} = -6.72\"$)
Figure 3.916  T29 – (Left and Middle) Visible Buckling of Bar S3 during \( \mu_5^{+2} = 6.72'' \), (Right) Bar N3 Remained Visibly Straight during \( \mu_5^{+3} = 6.72'' \)

Figure 3.917  T29 – (Left) Buckling of Adjacent Bars S2 and S4 during \( \mu_6^{+1} = 8.06'' \), (Middle and Right) Increase in the Buckled Deformation of Bar S3 during \( \mu_6^{+1} = 8.06'' \)
Figure 3.918  T29 – (Left) Fracture of Bar S3 during \( \mu_6^{-1} = -8.06'' \), (Middle and Right) Buckling of Bars N2 and N3 during \( \mu_6^{-1} = -8.06'' \)

Figure 3.919  T29 – After the Test (Left) North Side and (Right) South Side
Vertical strain profiles for the north extreme fiber bar N3, which is placed into tension during push cycles, appear in the right half of Figure 3.921. This figure shows both extreme fiber bars on the same graph. Compressive vertical strain profiles for north extreme fiber bar N3 during pull cycles appear in the left half of Figure 3.922. Strain profiles for additional cycles of displacement ductility five appear in Figure 3.923 and Figure 3.924, since bar buckling was delayed by one ductility level on the north side of the specimen. A peak tensile strain of 0.0549 was measured 6.27” above the footing on bar N3 during ($\mu_0^+ = 8.06"$), before bar buckling was observed during ($\mu_0^- = -8.06"$). The relationship between tension strain and displacement for this gage length appears in Figure 3.927. The measured tension strains for this gage length are predicted well by monotonic moment-curvature analysis with the PCK (2007) Lp Hinge Method. A compressive strain of $-0.0142$ was measured 4.33”
above the footing on bar N3 during \((\mu_3^{-1} = -4.03\,\)\), when the first spiral in the confinement region yielded. The relationship between compressive strain and displacement for this gage length appears in Figure 3.928. The measured compressive strains exceed the moment-curvature prediction beyond displacement ductility two.

Measured spiral strains in six layers which overlaid the north extreme fiber bar appear in Figure 3.926. Spiral tension strains in the second and third layers above the footing increased during each successive pull cycle of displacement ductility four and five. A spiral strain hysteresis for the layer 6.41” above the footing which overlaid the outward deformed region of bar N3 appears in Figure 3.934. The hysteresis contains spiral data from a strain gage and an Optotrak gage length, Figure 3.898. The Optotrak strains were calculated using arc-lengths obtained from measured 3D distance chord lengths and the known outside diameter of the spiral. It is important to note that arc-length calculations become inaccurate once severe yielding in the spiral leads to the reinforcement straightening out to the left and right of the localized yielding directly over the bar where the strain gage is located. Each consecutive pull cycle of displacement ductility five led to larger inelastic spiral strains, which indicates that measurable outward deformation occurred before visible bar buckling. The strain gage debonded during \((\mu_5^{-1} = -6.72\,\)\), preventing further measurement. The sharpest increase in measured spiral strains occurred during \((\mu_6^{-1} = -8.06\,\)\), when visible bar buckling was observed. The distribution of arc-strains measured around the circumference of the second, fourth, and fifth spiral layers above the footing during pull cycles appear in Figure 3.940, Figure 3.941, and Figure 3.942. The north side of the specimen is on the left side of the graph with negative location values and specific locations of bars N2, N3, and N4 are highlighted with vertical dashed lines.

Strain hysteresis for gage lengths 4.33”, 6.27”, and 8.22” above the footing on bar N3 appear in Figure 3.931, Figure 3.932, and Figure 3.933. The effect of measurable deformation on recorded strains can be visualized when comparing the location of the gage lengths with the deformed regions of the buckled extreme fiber bar, Figure 3.935. The gage length 6.27” above the footing overlaid the outward deformed region of bar N3, Figure
3.918. This gage length experienced additional tension during successive pull cycles of displacement ductility five as a result of measurable outward deformation. This deformation rapidly increased during $(\mu_6^{-1} = -8.06\text{''})$, when visible buckling was observed. The gage lengths 4.33” and 8.22” lie just below and above the outward deformed region. These gage lengths experience additional compression due to measurable deformation, as shown in Figure 3.931. Larger compressive strains were measured during each successive cycle of displacement ductility four and five, but the exact strain magnitudes may be influenced by the measurable deformation. The effect of this deformation is less apparent in the hysteresis 8.22” above the footing, Figure 3.933.

**South Reinforcement**

Vertical strain profiles for the south extreme fiber bar S3, which is placed into tension during pull cycles, appear in the right half of Figure 3.922. Compressive vertical strain profiles for south extreme fiber bar S3 during push cycles appear in the left half of Figure 3.921. A peak tension strain of 0.0357 was measured 6.30” above the footing on bar S3 at $(\mu_5^{-1} = -6.72\text{''})$, before the bar visibly buckled during $(\mu_5^{+2} = 6.72\text{''})$. A tension strain of 0.0342 was measured 12.05” above the footing during $(\mu_5^{-1} = -6.72\text{’’})$. The relationship between tension strain and displacement for the gage length 6.30” above the footing appears in Figure 3.929. Moment-curvature analysis with the PCK (2007) Lp Hinge Method begins to over predict measured tension strains at an increasing rate beyond displacement ductility two. A compressive strain of -0.0156 was measured 2.03” above the footing when the first confinement steel yielded on the south side of the specimen during $(\mu_3^{+1} = 4.03\text{’’})$. A peak compressive strain of -0.0316 was measured 12.05” above the footing during $(\mu_5^{+1} = 6.72\text{’’})$. The relationship between compressive strain and displacement for the gage length 8.22” above the footing appears in Figure 3.930. The measured compressive strains exceed the moment-curvature prediction with the PCK (2007) Lp Hinge Method beyond displacement ductility two.
Measured strains in six spiral layers which overlaid the south extreme fiber bar appear in Figure 3.925 for push cycles. Cycles during displacement ductility four produced successively larger inelastic demands on the third and fourth spiral layers. Strain gage and Optotrak strain hysteresis for the spiral layer 6.28” above the footing appears in Figure 3.939. The two measurement methods begin the deviate during displacement ductility three and four, when the effects of measurable deformation became apparent and presumably the arc-strains no longer represent the geometry of the spiral over bar S3. The spiral strains increased during \((\mu_5^{+2} = 6.72")\) when visible bar buckling was observed. Measured arc-strains around the circumference of the fourth and fifth spiral layers on the south side of the specimen during push cycles appear in Figure 3.943 and Figure 3.944.

The influence of measurable deformation before bar buckling becomes easier to understand when the specific gage lengths in question are located on buckled bar, Figure 3.935. Bar S3 strain hysteresis for gage lengths 4.34”, 6.30” and 8.22” above the footing appear in Figure 3.936, Figure 3.937, and Figure 3.938. The gage length 6.30” above the footing overlaid the outward buckled region, Figure 3.916. The hysteresis remained stable until visible bar buckling occurred during \((\mu_5^{+2} = 6.72")\). The gage lengths 4.34” and 8.22” above the footing were just below and above the outward buckled region. Significantly larger compressive strains were measured 8.22” above the footing, Figure 3.938, after confinement steel yielded during \((\mu_3^{+1} = 4.03")\).
Figure 3.921 T29 – Extreme Fiber Bar Vertical Strain Profiles during Push Cycles

Figure 3.922 T29 – Extreme Fiber Bar Vertical Strain Profiles during Pull Cycles
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Figure 3.923  T29 – Extreme Fiber Bar N3 Tension Strain Profiles for Push Cycles

Figure 3.924  T29 – Extreme Fiber Bar N3 Compression Strain Profiles for Pull Cycles
Figure 3.925  T29 – Spiral Strain on the South Side of the Specimen during Push Cycles

Figure 3.926  T29 – Spiral Strain on the North Side of the Specimen during Pull Cycles
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Figure 3.927 T29 – Tensile Strain-Displacement for Bar N3 during Push Cycles

Figure 3.928 T29 – Compressive Strain-Displacement for Bar N3 during Pull Cycles
Figure 3.929 T29 – Tensile Strain-Displacement for Bar S3 during Pull Cycles

Figure 3.930 T29 – Compressive Strain-Displacement for Bar S3 during Push Cycles
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Figure 3.931  T29 – Bar N3 Strain Hysteresis to Buckling (4.33” Above the Footing)

Figure 3.932  T29 – Bar N3 Strain Hysteresis to Buckling (6.27” Above the Footing)
Figure 3.933  T29 – Bar N3 Strain Hysteresis to Buckling (8.22” Above the Footing)

Figure 3.934  T29 – Spiral Strain Hysteresis over North Buckled Region (6.41” Above)
Figure 3.935  T29 – Buckled Shape of Bar N3 (Left) and Bar S3 (Middle)

Figure 3.936  T29 – Bar S3 Strain Hysteresis to Buckling (4.34” Above the Footing)
Visible Buckling in the Outward Deformed Region Leads to Additional Tension

Figure 3.937 T29 – Bar S3 Strain Hysteresis to Buckling (6.30” Above the Footing)

Visible Buckling

Measured Inward Deformation over Successive Cycles Leads to Additional Compression

Figure 3.938 T29 – Bar S3 Strain Hysteresis to Buckling (8.22” Above the Footing)
Figure 3.939  T29 – Spiral Strains over the North Buckled Region (6.28” Above)

Figure 3.940  T29 – Spiral Strain Distribution for the 2nd Spiral above the Footing during Pull Cycles (Negative = North)
Figure 3.941  T29 – Spiral Strain Distribution for the 4th Spiral above the Footing during Pull Cycles (Negative = North)

Figure 3.942  T29 – Spiral Strain Distribution for the 5th Spiral above the Footing during Pull Cycles (Negative = North)
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Figure 3.943 T29 – Spiral Strain Distribution for the 4th Spiral above the Footing during Push Cycles (Positive = South)

Figure 3.944 T29 – Spiral Strain Distribution for the 5th Spiral above the Footing during Push Cycles (Positive = South)
Test 29 – Curvature and Strain Penetration Data

Cross section strain profiles for the second horizontal section above the footing appear in Figure 3.945 and Figure 3.946 for push and pull cycles respectively. Individual bar strains appear to deviate from the planes section hypothesis for this section. The curvature is calculated as the slope of the least squared line connecting strains measured in eight instrumented reinforcing bars, Figure 3.897. If the curvatures for many horizontal cross sections are analyzed, curvature profiles for the plastic hinge region can be constructed; Figure 3.947 and Figure 3.948 for push and pull cycles respectively. Measured curvatures during displacement ductility one closely match the elastic curvature profile, which linearly decreases from yield curvature at the footing-column interface to zero at the center of the applied lateral load. Linear least squared error plastic curvature lines were fit to the plastic portion of the measured curvature profiles. The extrapolation of this linear curvature line with the footing-column interface was taken as the base section curvature. As the base section curvature ductility increased, the height at which the linear plastic curvature distribution intersected the elastic curvature profile also increased. Circular data points in Figure 3.952 plot the measured spread of plasticity as a function of base section curvature ductility.

Curvature profiles describe the elastic and plastic flexural displacements of the column, but do not address fixed-end rotations which result from development of longitudinal reinforcement into the footing. The measured vertical displacements of Optotrak LEDs placed closest to the footing column interface can be used to quantify this fixed-end rotation, Figure 3.949 and Figure 3.950. The fixed-end rotation is taken as the slope of the least squared error line fit to the bond slip profile. If an elastic curvature profile assumption is made for curvatures higher than those measured with instrumentation, then the entire curvature profile may be integrated to obtain the total column flexural displacement. This column flexural displacement was added to the strain penetration displacement, and compared to the experimentally measured displacements in Figure 3.951.
Figure 3.945 T29 – Push Cycle Strain Profiles for Section 4.31” Above the Footing

Figure 3.946 T29 – Pull Cycle Strain Profiles for Section 4.31” Above the Footing
Figure 3.947 T29 – Push Cycle Curvature Profiles with Linear Plastic Regression

Figure 3.948 T29 – Pull Cycle Curvature Profiles with Linear Plastic Regression
Figure 3.949  T29 – Base Rotation due to Strain Penetration during Push Cycles

Figure 3.950  T29 – Base Rotation due to Strain Penetration during Pull Cycles
Figure 3.951  T29 – Comparison of Measured and Optotrak Integrated Displacements

Figure 3.952  T29 – Measured Spread of Plasticity (Circular Data Points)
3.4.6 Test 30 – 18” Dia. Column with 3.1% Long. Steel and 15% Axial Load

Table 3.57 Observations for Test 30 – 18” Dia. With 3.1% Steel and 15% Axial Load

VALUES OF INTEREST:

- Concrete Compressive Strength: $f'_c = 6050 \text{ psi}$
- Axial Load: $P = 206 \text{ kips}$ ($P/(f'_c A_g) = 15\%$)
- Longitudinal Steel Content: 10 #8 Bars ($A_{st}/A_g = 3.1\%$)
- Analytical First Yield Force: $F'_y = 34.61 \text{ kips}$
- Experimental First Yield Displacement: $\Delta'_y = 1.15"$
- Analytical Nominal Moment Capacity: $M_n = 356.35 \text{ kip * ft}$
- Equivalent Yield Displacement: $\Delta_y = 1.48"$
- Maximum Lateral Force: 47.48 kips

DAMAGE OBSERVATIONS:

- First Cracking North: $3/4Fy' = 0.69"$
- First Cracking South: $-3/4Fy' = -0.76"$
- Cover Concrete Crushing North: $\mu_{1.5}^{-1} = -2.21"$
- Cover Concrete Crushing South: $\mu_{1.5}^{+1} = 2.21"$
- Transverse Steel Yield North: At $-2.74"$ during pull to $\mu_2^{-1} = -2.95"$
- Transverse Steel Yield South: At 2.74" during push to $\mu_2^{+1} = 2.95"$
- Longitudinal Bar Buckling North: Reversal from $\mu_5^{+1} = 7.39"$
- Longitudinal Bar Buckling South: Reversal from $\mu_5^{-1} = -7.39"$
- Longitudinal Bar Fracture North: At 8.75" during push to $\mu_6^{+1} = 8.88"$
- Longitudinal Bar Fracture South: At $-3.27"$ during the final pull cycle of the test

$*\mu_5^{-1} = -7.39"$ represents the first pull cycle of displacement ductility five
Table 3.58 Strain Summary for Test 30 – 18” Dia. With 3.1% Steel and 15% Axial

<table>
<thead>
<tr>
<th>MATERIAL STRAINS:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover Concrete Crushing North: $\varepsilon_s = 0.0052$ (compression)</td>
</tr>
<tr>
<td>Cover Concrete Crushing South: $\varepsilon_s = 0.0059$ (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield North: $\varepsilon_s = 0.0095$ (compression)</td>
</tr>
<tr>
<td>Transverse Steel Yield South: $\varepsilon_s = 0.0094$ (compression)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling North: $\varepsilon_s = 0.036$ (peak tension prior to $bb$) $\varepsilon_s = 0.022$ (peak comp. prior to $bb$)</td>
</tr>
<tr>
<td>Longitudinal Bar Buckling South: $\varepsilon_s = 0.033$ (peak tension prior to $bb$) $\varepsilon_s = 0.026$ (peak comp. prior to $bb$)</td>
</tr>
</tbody>
</table>

Mander (1988) Ultimate Concrete Compression Strain, $\varepsilon_{cu} = 0.0197$

Figure 3.953 T30 – Test Setup and Cross Section Bar Designation
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Figure 3.954 T30 – Target Marker Application and Optotrak Spatial Output

Figure 3.955 T30 – T28 – Compressive Axial Load from One Jack (Total = 2*Value)
Figure 3.956  T30 – Symmetric Three Cycle Set Load History

Figure 3.957  T30 – Lateral Force vs. Top Column Displacement Response
Figure 3.958 Hysteretic Comparison for T28 and T30 with Different Steel Content

Test 30 – 18” Dia. with 3.1% Steel and 15% Axial Load – Experimental Observations

Specimens 25-30 focus on the effects of longitudinal steel content, longitudinal bar diameter, and higher levels of axial load on column behavior. This section summarizes experimental observations and data analysis for Test 30. The 18” diameter column contains 10 #8 (A706) bars for longitudinal reinforcement ($A_{st}/A_g = 3.1\%$) and a #3 A706 spiral at 2” on center ($4A_{sp}/(D's) = 1.3\%$). The specimen had an 8ft cantilever length ($L/D = 5.33$), and was subjected to $(P/(f'_cA_g) = 15\%)$ axial load. Previous Test 28 was nominally identical, except it was reinforced with 10 #6 longitudinal bars (1.7%). The symmetric three-cycle-set load history is commonly used to evaluate the seismic performance of structural components. The load history begins with elastic cycles to the following increments of the analytically predicted first yield force: $\frac{1}{4} F'_y$, $\frac{1}{2} F'_y$, $\frac{3}{4} F'_y$, and $F'_y$. The experimental first yield displacement is then determined by taking the average of the
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recorded displacements during the first yield push and pulls cycles. The equivalent yield displacement, used to determine the displacement ductility levels \( \mu_1 = 1 \cdot \Delta_y \), is then calculated as \( \Delta_y = \Delta'_y \left( M_n / M'_y \right) \). The symmetric three-cycle-set load history resumes with three balanced cycles at each of the following displacement ductility levels: 1, 1.5, 2, 3, 4, 5, etc.

![Figure 3.959 T30 – Strain Profiles before Yield, (Left) North and (Right) South](image)

The test began with cycles in \( \frac{1}{4} F_y' \) (first yield force) increments in each direction of loading until the first yield force was reached. The first cracks on the north and south sides of the specimen formed during \( (3/4Fy' = 0.69") \) and \( (-3/4Fy' = -0.76") \) respectively, Figure 3.960. The crack distribution on all sides of the specimen at first yield, \( (Fy' = 1.10") \) and \( (-Fy' = -1.20") \), appears in Figure 3.961. Tensile vertical strain profiles for north and south reinforcing bars during elastic push and pull cycles appear in Figure 3.959. The average experimental first yield displacement was used to calculate the equivalent yield displacement, \( \Delta_y = \Delta'_y \left( M_n / M'_y \right) = 1.15" \), which defined the reversal amplitudes for reminder of the test.
The crack progression at displacement ductility 1, 1.5, 2, and 3 appear in Figure 3.962, Figure 3.964, Figure 3.966, and Figure 3.967 respectively. During these cycles the cracks became more numerous and increased in inclination on the shear faces of the specimen. Small amounts concrete crushing was observed on the south and north sides of the specimen during \((\mu_{1.5}^{+1} = 2.21")\) and \((\mu_{1.5}^{-1} = -2.21")\), Figure 3.963. As compressive demands increased during displacement ductility 1.5 to 4, crushing gradually increased on each side of the specimen. Compressive demand during \((\mu_{2}^{+1} = 2.95")\) and \((\mu_{2}^{-1} = -2.95")\) led to spiral yielding in confinement regions, Figure 3.965. Spiral strains on each side of the specimen increased during successive cycles of ductility four, but the extreme fiber reinforcement remained visibly straight, Figure 3.968.

Even though spiral strains in confinement regions increased, the south extreme fiber bar remained visibly straight during \((\mu_{5}^{+1} = 7.39")\), Figure 3.969. The north extreme fiber bar visibly buckled during \((\mu_{5}^{-1} = -7.39")\), Figure 3.969. Visible buckling of the south extreme fiber bar was observed during the subsequent reversal to \((\mu_{5}^{+2} = 7.38")\), Figure 3.970. The buckled deformation in the north (Figure 3.970) and south (Figure 3.971) extreme fiber bars increased during \((\mu_{5}^{-2} = -7.40")\) and \((\mu_{5}^{+3} = 7.39")\) respectively. The extent of spiral deformation over the south buckled bar during \((\mu_{5}^{-3} = -7.39")\) is shown in Figure 3.971. The previously buckled north extreme fiber bar fractured during \((\mu_{6}^{+1} = 8.88")\), Figure 3.972, resulting in the first significant loss in strength. Two adjacent north reinforcing bars, N2 and N4, buckled during \((\mu_{6}^{-1} = -8.88")\), Figure 3.972. Two adjacent south reinforcing bars, S2 and S4, buckled during \((\mu_{6}^{+2} = 8.87")\), Figure 3.973. The south extreme fiber bar fractured at -3.27” during \(\mu_{6}^{-2}\), Figure 3.973. At this point the test was concluded with fractured reinforcement on each side of the specimen and severe strength loss. Photos of the specimen after the instrumentation was removed appear in Figure 3.974 and Figure 3.975. Crushed concrete behind buckled bars in extreme fiber regions was removed, highlighting the effect confinement loss behind deformed transverse steel.
Figure 3.960  T30 – (Left) Cracking on the North Side during \((3/4Fy' = 0.69")\), (Right) South Cracking \((-3/4Fy' = -0.76")\)

Figure 3.961  T30 – (Left) North Crack Distribution at \((Fy' = 1.10")\), (Middle and Right) Crack Distributions on the Front and Right Sides at \((-Fy' = -1.20")\)
Figure 3.962 T30 – (Left) North Crack Distribution at ($\mu_{1}^{+3} = 1.48\"$), (Middle and Right) Crack Distributions on the Front and Right Sides at ($\mu_{1}^{-3} = −1.48\"$)

Figure 3.963 T30 – (Left) South Concrete Crushing during ($\mu_{1.5}^{+1} = 2.21\"$), (Right) North Crushing during ($\mu_{1.5}^{-1} = −2.21\"$)
Figure 3.964  T30 – (Left) North Crack Distribution at \( \mu_{1.5}^+ = 2.01" \), (Middle and Right) Crack Distributions on the Front and Right Sides at \( \mu_{1.5}^- = -2.01" \)

Figure 3.965  T30 – Spiral Layers which Yielded during Displacement Ductility Two, (Left) South and (Right) North
Figure 3.966 T30 – (Left) North Crack Distribution at \( \mu_2^+ = 2.95'' \), (Middle and Right) Crack Distributions on the Front and Right Sides at \( \mu_2^- = -2.95'' \)

Figure 3.967 T30 – (Left) North Crack Distribution at \( \mu_3^+ = 4.43'' \), (Middle and Right) Crack Distributions on the Front and Right Sides at \( \mu_3^- = -4.43'' \)
Figure 3.968  T30 – (Left) Bar S3 Remained Visibly Straight during ($\mu_{4}^{+3} = 5.92''$) and 
(Right) Bar N3 Remained Visibly Straight during ($\mu_{4}^{-3} = -5.91''$)

Figure 3.969  T30 – (Left) Bar S3 Remained Visibly Straight during ($\mu_{5}^{+1} = 7.39''$), 
(Middle and Right) Visible Buckling of Bar N3 during ($\mu_{5}^{-1} = -7.39''$)
Figure 3.970 T30 – (Left and Middle) Visible Buckling of Bar S3 during \((\mu_5^{+2} = 7.38")\), (Right) Increased Deformation in Bar N3 during \((\mu_5^{-2} = -7.40")\)

Figure 3.971 T30 – (Left) Increased Deformation in Bar S3 during \((\mu_5^{+3} = 7.39")\), (Middle) Increased Deformation in Bar N3 during \((\mu_5^{-3} = -7.39")\), (Right) South Spiral Deformation during \((\mu_5^{-3} = -7.39")\)
Figure 3.972 T30 – (Left) Increased Deformation in Bar S3 during \(\mu_6^{+1} = 8.88\), (Middle) Fracture of Bar N3 during \(\mu_6^{+1} = 8.88^"\), Buckling of Adjacent Bars N2 and N4 during \(\mu_6^{-1} = -8.88^"\)

Figure 3.973 T30 – (Left) Buckling of Adjacent Bars S2 and S4 during \(\mu_6^{+2} = 8.87\), (Middle) South Spiral Deformation during \(\mu_6^{+2} = 8.87\), (Right) Fracture of Bar S3 during \(\mu_6^{-2}\)
Figure 3.974  T30 – After the Test (Left) North and (Right) South Side of the Specimen

Figure 3.975  T30 – (Left to Right) North, Front, South, and Back Sides after the Test
Test 30 – 18” Dia. with 3.1% Steel and 15% Axial Load – Strain Data Analysis

North Reinforcement

Vertical strain profiles for the north extreme fiber bar N3, which is placed into tension during push cycles, appear in the right half of Figure 3.976. This figure shows both extreme fiber bars on the same graph. Compressive vertical strain profiles for north extreme fiber bar N3 during pull cycles appear in the left half of Figure 3.977. Tension strain profiles for adjacent north reinforcing bars N2 and N4 appear in Figure 3.979. By comparison, these bars do not show as large of a decrease in tension strain measured over the first gage length above the footing. A peak tensile strain of 0.0362 was measured 4.07” above the footing on bar N3 during \( \mu_5^{+1} = 7.39" \), before bar buckling was observed during \( \mu_5^{-1} = -7.39" \). The relationship between tension strain and displacement for this gage length appears in Figure 3.981. The measured tension strains for this gage length are predicted well by monotonic moment-curvature analysis with the PCK (2007) Lp Hinge Method. Prior to displacement ductility five, the peak tensile gage length was centered 6.08” above the footing. Tension strains measured in this gage length, Figure 3.982, slightly exceed the prediction. A compressive strain of \(-0.0095\) was measured 2.09” above the footing on bar N3 during \( \mu_2^{-1} = -2.95" \), when confinement steel yielded on the north side of the specimen. A compressive strain of \(-0.0210\) was measured 2.09” above the footing on bar N3 during \( \mu_4^{-1} = -5.92" \). It appears that measurable deformation occurred in the North bar during \( \mu_4^{+2} \) and \( \mu_4^{+3} \), resulting in smaller compressive strains in the gage length 4.07” above the footing and larger strains measured 6.08” above the footing. A peak compressive strain prior to buckling of -0.0222 was measured 6.08” above the footing during \( \mu_4^{-3} = -5.91" \). The relationship between compressive strain and displacement for gage length 2.09” above the footing on bar N3 appears in Figure 3.983. The measured compressive strains exceed the moment-curvature prediction with the PCK (2007) Lp Hinge Method.

Measured spiral strains in six layers which overlaid the north extreme fiber bar appear in Figure 3.980. The first spiral layer was located at the footing-column interface, and was not
instrumented with a strain gage or LEDs. Spiral tension strains in the second and third layers above the footing increased during each successive pull cycle of displacement ductility four. A spiral strain hysteresis for the layer 4.19” above the footing, which overlaid the outward deformed region of bar N3, appears in Figure 3.989. The hysteresis contains spiral data from a strain gage and an Optotrak gage length, Figure 3.954. The Optotrak strains were calculated using arc-lengths obtained from measured 3D distance chord lengths and the known outside diameter of the spiral. It is important to note that arc-length calculations become inaccurate once severe yielding in the spiral leads to the reinforcement straightening out to the left and right of the localized yielding directly over the bar where the strain gage is located. Each consecutive pull cycle of displacement ductility four led to larger inelastic spiral strains, which indicates that measurable outward deformation occurred before visible bar buckling. The sharpest increase in measured spiral strains occurred during \( (\mu_5^{-1} = -7.39") \), when visible bar buckling was observed. The distribution of arc-strains measured around the circumference of the second and third spiral layers above the footing during pull cycles appears in Figure 3.995 and Figure 3.997. The north side of the specimen is on the left side of the graph with negative location values and specific locations of bars N2, N3, and N4 are highlighted with vertical dashed lines.

Strain hysteresis for gage lengths 4.07” and 6.08” above the footing on bar N3 appear in Figure 3.987 and Figure 3.988. The effect of measurable deformation on recorded strains can be visualized when comparing the location of the gage lengths with the deformed regions of the buckled extreme fiber bar, Figure 3.990. The gage length 4.07” above the footing overlaid the outward deformed region of bar N3, Figure 3.969. This gage length experienced additional tension during successive pull cycles of displacement ductility four as a result of measurable outward deformation. This deformation rapidly increased during \( (\mu_5^{-1} = -7.39") \), when visible buckling was observed. The gage length 6.08” was located just above the outward deformed region of bar N3. This gage length experience additional compression due to measurable deformation, as shown in Figure 3.988. Larger compressive strains were measured during each successive cycle of displacement ductility four.
South Reinforcement

Vertical strain profiles for the south extreme fiber bar S3, which is placed into tension during pull cycles, appear in the right half of Figure 3.977. Tension strain profiles for adjacent south reinforcing bars S2 and S4 appear in Figure 3.978. Compressive vertical strain profiles for south extreme fiber bar S3 during push cycles appear in the left half of Figure 3.976. A peak tension strain of 0.0329 was measured 8.18” above the footing on bar S3 at \( \mu_5^{-1} = -7.39" \), before the bar visibly buckled during \( \mu_5^{+2} = 7.38" \). Similar peak tension strains of 0.0315 and 0.0327 were measured 4.16” and 12.14” above the footing during \( \mu_5^{+1} = -7.39" \). The relationship between tension strain and displacement for the gage length 12.14” above the footing on bar S3 appears in Figure 3.984. The measured tension strains match the moment-curvature prediction well when the PCK (2007) Lp Hinge Method is used. A compressive strain of -0.0094 was measured 2.25” above the footing when the first confinement steel layer yielded on the south side of the specimen during \( \mu_2^{+1} = 2.25" \). A peak compressive strain of -0.0263 was measured 2.25” above the footing during \( \mu_5^{+1} = 7.39" \). During the same cycle, a compression strain of -0.0260 was measured 6.20” above the footing. The relationship between compressive strain and displacement for the gage lengths 2.25” and 6.20” above the footing appears in Figure 3.985 and Figure 3.986. The measured compressive strains exceed the moment-curvature prediction with the PCK (2007) Lp Hinge Method beyond displacement ductility 1.5 for the gage length 2.25” above the footing.

Measured strains in six spiral layers which overlaid the south extreme fiber bar appear in Figure 3.980 for push cycles. Cycles during displacement ductility four produced successively larger inelastic demands on the second and third spiral layers. Strain gage and Optotrak strain hysteresis for the spiral layer 6.28” above the footing appears in Figure 3.993. The two measurement methods begin the deviate during displacement ductility three and four. The spiral strains increased significantly during \( \mu_5^{+2} = 7.38" \) when visible bar buckling was observed. Measured arc-strains around the circumference of the second and third spiral layers on the south side of the specimen during push cycles appear in Figure
Spiral strain hystereses for stain gages placed at mid-section (Diameter/2) appear in Figure 3.998 and Figure 3.999. The spiral layer 2.16” above the footing, Figure 3.998, was not crossed by inclined flexural-shear cracks. The spiral layer 6.22” above the footing was crossed by only mildly inclined flexural-shear cracks. The three spiral layers instrumented on the shear face of the column did not yield before bar buckling.

The influence of measurable deformation before bar buckling becomes easier to understand when the specific gage lengths in question are located on buckled bar, Figure 3.990. Bar S3 strain hysteresis for gage lengths 4.16” and 6.20” above the footing on bar S3 appear in Figure 3.991 and Figure 3.992. The gage length 4.16” above the footing, Figure 3.991, overlaid the outward buckled region. Smaller compressive strain magnitudes were measured during \( \mu_4^{+2} \), \( \mu_4^{+3} \), and \( \mu_5^{+1} \) due to small amounts of outward deformation prior to visible bar buckling during \( \mu_5^{+2} = 7.38'' \). The gage length 6.20” above the footing was located just above the outward buckled region. Successively larger compressive strains were measured during \( \mu_4^{+2} \), \( \mu_4^{+3} \), and \( \mu_5^{+1} \), Figure 3.992.
Chapter 3: Experimental Observations

Figure 3.976 T30 – Extreme Fiber Bar Vertical Strain Profiles during Push Cycles

Figure 3.977 T30 – Extreme Fiber Bar Vertical Strain Profiles during Pull Cycles
Chapter 3: Experimental Observations

Figure 3.978 T30 – (Left) Bar S4 and (Right) Bar S2 Tensile Strain Profiles

Figure 3.979 T30 – (Left) Bar N4 and (Right) Bar N2 Tensile Strain Profiles

Figure 3.980 T30 – Spiral Strains for Compressive Cycles (Left) South, (Right) North
Figure 3.981 T30 – Tensile Stain-Displacement for Bar N3 during Push Cycles

Figure 3.982 T30 – Tensile Stain-Displacement for Bar N3 during Push Cycles
Figure 3.983 T30 – Compressive Strain-Displacement for Bar N3 during Pull Cycles

Figure 3.984 T30 – Tensile Strain-Displacement for Bar S3 during Pull Cycles
Figure 3.985  T30 – Compressive Strain-Displacement for Bar S3 during Push Cycles

Figure 3.986  T30 – Compressive Strain-Displacement for Bar S3 during Push Cycles
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Figure 3.987 T30 – Bar N3 Strain Hysteresis to Buckling (4.07” Above the Footing)

Figure 3.988 T30 – Bar N3 Strain Hysteresis to Buckling (6.08” Above the Footing)
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Figure 3.989  T30 – Spiral Strains over the North Buckled Region (4.19” Above)

Figure 3.990  T30 – Buckled Shape of Bar N3 (Left) and Bar S3 (Middle)
Visible Buckling of Bar S3

Measured Outward Deformation Leads to Additional Tension

Figure 3.991 T30 – Bar S3 Strain Hysteresis to Buckling (4.16” Above the Footing)

Visible Buckling of Bar S3

Measured Deformation Leads to Additional Compression

Figure 3.992 T30 – Bar S3 Strain Hysteresis to Buckling (6.20” Above the Footing)
Figure 3.993 T30 – Spiral Strains over the South Buckled Region (6.28” Above)

Figure 3.994 T30 – Spiral Strain Distribution for the 2nd Spiral above the Footing during Push Cycles (Positive= South)
Figure 3.995  T30 – Spiral Strain Distribution for the 2nd Spiral above the Footing during Pull Cycles (Negative = North)

Figure 3.996  T30 – Spiral Strain Distribution for the 3rd Spiral above the Footing during Push Cycles (Positive= South)
Figure 3.997  T30 – Spiral Strain Distribution for the 3rd Spiral above the Footing during Pull Cycles (Negative = North)

Figure 3.998  T30 – Second Spiral Layer Strain Hysteresis at Midsection (2.16” Above)
Test 30 – Curvature and Strain Penetration Data

Cross section strain profiles for the second horizontal section above the footing appear in Figure 3.1000 and Figure 3.1001 for push and pull cycles respectively. This is the first horizontal section above the footing which included strain measurements for bars S1 and N1. Individual bar strains appear to deviate from the planes section hypothesis for this section, but the overall trend is captured well. The curvature is calculated as the slope of the least squared line connecting strains measured in eight instrumented reinforcing bars. If the curvatures for many horizontal cross sections are analyzed, curvature profiles for the plastic hinge region can be constructed; Figure 3.1002 and Figure 3.1003 for push and pull cycles respectively. Measured curvatures during displacement ductility one closely match the elastic curvature profile, which linearly decreases from yield curvature at the footing-column interface to zero at the center of the applied lateral load. Plastic curvatures were found to follow a linear distribution. Linear least squared error plastic curvature lines were fit to the
plastic portion of the measured curvature profiles. The extrapolation of this linear curvature line with the footing-column interface was taken as the base section curvature. As the base section curvature ductility increased, the height at which the linear plastic curvature distribution intersected the elastic curvature profile also increased. Circular data points in Figure 3.1007 plot the measured spread of plasticity as a function of base section curvature ductility.

Curvature profiles describe the elastic and plastic flexural displacements of the column, but do not address fixed-end rotations which result from development of longitudinal reinforcement into the footing. The measured vertical displacements of Optotrack LEDs placed closest to the footing column interface can be used to quantify this fixed-end rotation, Figure 3.1004 and Figure 3.1005. The fixed-end rotation is taken as the slope of the least squared error line fit to the bond slip profile. The strain penetration displacement is obtained by multiplying this rotation by the cantilever height of the column. If an elastic curvature profile assumption is made for curvatures higher than those measured with instrumentation, then the entire curvature profile may be integrated to obtain the total column flexural displacement. This column flexural displacement was added to the strain penetration displacement, and compared to the experimentally measured displacements in Figure 3.1006. The Optotrack integrated displacements slightly exceed those obtained from a string potentiometer placed at the center of the lateral load.
Figure 3.1000  T30 – Push Cycle Strain Profiles for the Section 4.11” Above the Footing

Figure 3.1001  T30 – Pull Cycle Strain Profiles for the Section 4.11” Above the Footing
Chapter 3: Experimental Observations

Figure 3.1002  T30 – Push Cycle Curvature Profiles with Plastic Regression Lines

Figure 3.1003  T30 – Pull Cycle Curvature Profiles with Plastic Regression Lines
Figure 3.1004 T30 – Base Rotation due to Strain Penetration during Push Cycles

Figure 3.1005 T30 – Base Rotation due to Strain Penetration during Pull Cycles
Figure 3.1006  T30 – Comparison of Measured and Optotrak Integrated Displacements

Figure 3.1007  T30 – Measured Spread of Plasticity (Circular Data Points)
Chapter 4: The Effect of Load History on Column Performance

In this section, the importance of displacement history and its effects on performance limit states, the relationship between strain and displacement, and the spread of plasticity in reinforced concrete structures is explored. An experimental study was carried out to assess the performance of thirty circular, well-confined, bridge columns with varying lateral displacement history, transverse reinforcement detailing, axial load, aspect ratio, and longitudinal steel content. Eight of these columns, with similar geometry and detailing, were subjected to various unidirectional displacement histories including standardized laboratory reversed cyclic loading and recreations of the displacement responses obtained from non-linear time history analysis of multiple earthquakes with distinct characteristics. Longitudinal reinforcing bars were instrumented to obtain strain hysteresis, vertical strain profiles, cross section curvatures, curvature distributions, and fixed-end rotations attributable to strain penetration. Results have shown that the limit state of reinforcement bar buckling was influenced by load history, but the relationship between strain and displacement along the envelope curve was not. The main impact of load history on bar buckling is its influence on accumulated strains within the longitudinal reinforcement and transverse steel.

4.1 Introduction

The goal of performance based seismic engineering is to design structures to achieve a predictable level of performance under a specific earthquake hazard within definable levels of reliability, as defined by the Structural Engineering Association of California, (SEAOC 1999). To satisfy the aims of performance based design, levels of damage which interrupt the serviceability of the structure or require more invasive repair techniques must be related to engineering criteria. For reinforced concrete flexural members such as bridge columns, concrete compressive and steel tensile strain limits are very good indicators of damage.
Closely spaced transverse steel hoops or spirals provide adequate confinement and shear resistance to produce a flexural mode of failure for columns without detailing deficiencies. The displacement capacity of these columns is limited by buckling and subsequent fracture of longitudinal reinforcement or rupture of confinement steel. An understanding of the spread of plasticity in reinforced concrete structures is required to determine the deformation at damage limit states for design.

A summary of the performance strain limits from (Kowalsky 2000) appear in Table 4.1. Serviceability limit states such as concrete cover crushing or residual crack widths exceeding 1mm may occur during smaller, more frequent earthquakes, (Priestley et. al. 1996). While the serviceability limit states do not pose a safety concern, the hinge regions must be repaired to prevent corrosion of internal reinforcing steel. At higher ductility demands produced by larger less frequent earthquakes, reinforcing bar buckling may lead to permanent elongation in the transverse steel, which diminishes its effectiveness in confining the concrete core. Bar buckling and significant damage to the core concrete represent the damage control limit states, which lead to significant repair costs. Furthermore, rupture of previously buckled bars during subsequent cycles of loading leads to strength loss. The life safety or collapse prevention limit state is characterized by fracture of previously buckled bars or rupture of confinement steel under displacements exceeding those required to initiate bar buckling.

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Concrete Compressive Strain Limit</th>
<th>Steel Tensile Strain Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Serviceability</td>
<td>0.004</td>
<td>0.015</td>
</tr>
<tr>
<td></td>
<td>Cover Concrete Crushing</td>
<td>Residual Crack Widths Exceed 1mm</td>
</tr>
<tr>
<td>Damage Control</td>
<td>$\approx 0.018$, Mander et al. (1988) $\varepsilon_{cu}$</td>
<td>0.060</td>
</tr>
<tr>
<td></td>
<td>Limit of Economical Concrete Repair</td>
<td>Tension Based Bar Buckling</td>
</tr>
</tbody>
</table>
Previous experimental studies on circular bridge columns with constant axial loads have shown that reinforcing bar buckling is influenced by unidirectional lateral displacement history: (Moyer and Kowalsky 2003), (Kunnath et al. 1997), and (Freytag 2006). Tests by Moyer and Kowalsky (2003) utilized fabricated load histories to investigate the influence of previous tensile strains on buckling of longitudinal reinforcement under compressive stress. Their results suggest that reinforcement buckling occurs after reversal from a peak tensile strain, while the bar is still under net elongation but compressive stress. After reversal from the peak displacement, the cracks on the tensile side begin to close, and before the column reaches zero displacement the reinforcement enters a state of compressive stress but net elongation. It is during this time, while the cracks are still open, that the reinforcement is the sole source of compression zone stability and the bars are prone to buckling. According to Kunnath et al. (1997), random displacement cycles provide a better means for understanding the effects of cumulative damage and assessing the performance of structures subjected to low-cycle fatigue. Freytag (2006) utilized instrumentation to detect lateral displacements of buckled bars in column tests with fabricated lateral displacement histories before the event could be visibly observed. The tests conducted by Freytag (2006) supported the tension based bar buckling mechanism, and suggested that bar buckling may be gradual phenomenon occurring over multiple cycles. Analytical studies by Syntzirma et. al. (2010) concluded that when flexural members are controlled by bar buckling, the deformation capacity cannot be defined uniquely since it is a function of the applied cyclic deformation history.

Tests by Wong et. al. (1993) focused on the effect of bidirectional lateral displacement history for squat circular columns with various levels of constant axial load. Their results indicated that bidirectional load path led to additional degradation in strength and stiffness, and a reduction in the deformation capacity when compared to nominally identical columns subjected to unidirectional load histories. Additionally, they found that the shape of the bidirectional load path did not significantly affect the displacement capacity of the columns. Experiments conducted by Bousias et al. (1995) investigated the influence of bidirectional load path and variable axial load history. They observed a significant coupling between the three loading directions, and found that the magnitude and history of the axial load
influenced the axial expansion and shortening of the column. Experiments by Esmaeily and Xiao (2005) demonstrated importance of considering axial load history when predicting the response of bridge columns. They concluded, “At a certain displacement and axial load level, the flexural capacity is significantly different depending on the history of axial loading path, from the flexural capacity at the same displacement and the same level but constant axial load.”

### 4.1.1 Test Setup

While the progression of damage in flexural bridge columns has been thoroughly investigated in the past, to the author’s knowledge, none of the previous studies had the ability to measure large strains at the level of the longitudinal reinforcement up to bar buckling. The goal of the experimental program is to investigate the impact of load history and design variables on the relationship between strain and displacement, performance strain limits, and the spread of plasticity. In total, the thirty specimens of the research program focus on the effects of load history, axial load, aspect ratio, transverse reinforcement, and longitudinal steel content. All of the variables found to be statistically significant towards describing bar buckling in an experimental dataset by Berry and Eberhard (2005) appear in the test matrix. This section focuses on eight specimens which had unidirectional lateral displacement history as the primary variable.

The specimen was designed to represent a single degree of freedom bridge column subjected to lateral and axial load, Figure 4.1. The test specimen consists of a footing, column, and loading cap. The footing is a capacity protected member which secures the specimen to the lab strong floor using post tensioned bars. A 220kip (980kN) hydraulic actuator, with a 40” (1016mm) stroke capacity, applies lateral load to the loading cap of the specimen. A spreader beam, two hydraulic jacks, and a load cell are placed above the loading cap to apply a constant axial compressive load through post tensioning bars which run beneath the lab strong floor. A self-regulating axial load system was utilized with a third hydraulic jack in a force controlled uniaxial testing machine to regulate the pressure, and thus
the load, of two jacks on top of the specimen to maintain a constant axial load throughout testing. This technique of axial load application does not replicate true P-Δ effects induced by a vertical gravity load. Instead, the axial force follows the direction of the post tensioning bar which is hinged at the floor and centered above the column. Examples of test setups which recreate P-Δ effects may be found in (Dutta et al. 1999) and (Esmaeily and Xiao 2002).

The load history variable specimens had nominally identical geometry and longitudinal steel content, and were subjected to different quasi-static unidirectional lateral displacement histories. The 24” (610mm) diameter bridge columns, Figure 4.2, contained 16 #6 (19mm) A706 bars for longitudinal reinforcement ($A_{st}/A_g = 1.6\%$) and a #3 (9.5mm) A706 spiral at either 2” (51mm) ($4A_{sp}/(D's) = 1\%$) or 1.5” (38mm) (1.3%) on center. The shear span for the cantilever columns was 8ft (244cm), and they had a moment to shear ratio of ($M/VD = 4$). The specimens were subjected to a constant axial load of 170kips (756kN), ($P/(f'cA_g) \approx 5\%$) depending on the concrete compressive strength. The test matrix for the eight columns is shown in Table 4.2, and the material properties of the longitudinal and transverse reinforcement appear in Table 4.3.
Figure 4.1 Test Setup for Load History Tests (Two Optotrak Position Monitors)

Figure 4.2 Column Cross Section for Load History Tests
Chapter 4: The Effect of Load History on Column Performance

Figure 4.3 (Left) Optotrak Spatial Coordinate Output and (Right) Comparison of Optotrak Strains to Traditional Measurements

Table 4.2 Test Matrix for Load History Variable Columns

<table>
<thead>
<tr>
<th>Test</th>
<th>Load History</th>
<th>Spiral Detailing ($\rho_s$)</th>
<th>$f_c$ (ksi, MPa)</th>
<th>$P/f_c \cdot Ag$</th>
</tr>
</thead>
<tbody>
<tr>
<td>T9</td>
<td>Three-Cycle</td>
<td>#3 at 2&quot; (51mm) (1%)</td>
<td>6.81, 46.97</td>
<td>5.5%</td>
</tr>
<tr>
<td>T8</td>
<td>Chile 2010</td>
<td>#3 at 2&quot; (51mm) (1%)</td>
<td>6.99, 48.18</td>
<td>5.4%</td>
</tr>
<tr>
<td>T8b</td>
<td>Three-Cycle</td>
<td>#3 at 2&quot; (51mm) (1%)</td>
<td>6.99, 48.18</td>
<td>5.4%</td>
</tr>
<tr>
<td>T10</td>
<td>Chichi 1999</td>
<td>#3 at 2&quot; (51mm) (1%)</td>
<td>5.26, 36.29</td>
<td>7.1%</td>
</tr>
<tr>
<td>T10b</td>
<td>Three-Cycle</td>
<td>#3 at 2&quot; (51mm) (1%)</td>
<td>5.26, 36.29</td>
<td>7.1%</td>
</tr>
<tr>
<td>T11</td>
<td>Kobe 1995</td>
<td>#3 at 2&quot; (51mm) (1%)</td>
<td>6.18, 41.85</td>
<td>6.2%</td>
</tr>
<tr>
<td>T12</td>
<td>Japan 2011</td>
<td>#3 at 2&quot; (51mm) (1%)</td>
<td>6.10, 42.06</td>
<td>6.2%</td>
</tr>
<tr>
<td>T16</td>
<td>Three-Cycle</td>
<td>#3 at 1.5&quot; (38mm) (1.3%)</td>
<td>6.71, 46.27</td>
<td>5.6%</td>
</tr>
<tr>
<td>T17</td>
<td>Llolleo 1985</td>
<td>#3 at 1.5&quot; (38mm) (1.3%)</td>
<td>7.59, 52.33</td>
<td>5.0%</td>
</tr>
<tr>
<td>T17b</td>
<td>Three-Cycle</td>
<td>#3 at 1.5&quot; (38mm) (1.3%)</td>
<td>7.59, 52.33</td>
<td>5.0%</td>
</tr>
<tr>
<td>T18</td>
<td>Darfield NZ 2010</td>
<td>#3 at 1.5&quot; (38mm) (1.3%)</td>
<td>7.80, 53.83</td>
<td>4.8%</td>
</tr>
<tr>
<td>T18b</td>
<td>Three-Cycle</td>
<td>#3 at 1.5&quot; (38mm) (1.3%)</td>
<td>7.80, 53.83</td>
<td>4.8%</td>
</tr>
</tbody>
</table>
Table 4.3 Reinforcement Material Properties

<table>
<thead>
<tr>
<th>Longitudinal Reinforcement</th>
<th>( \varepsilon_y )</th>
<th>( f_y ) (ksi)</th>
<th>( \varepsilon_h ) (hardening)</th>
<th>( f_h ) (ksi)</th>
<th>( \varepsilon_u ) (max stress)</th>
<th>( f_u ) (ksi)</th>
<th>Transverse Steel</th>
<th>( f_y ) (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T8 – T12</td>
<td>0.00235</td>
<td>68.1</td>
<td>0.0131</td>
<td>68.2</td>
<td>0.1189</td>
<td>92.8</td>
<td>T8 – T12</td>
<td>74.1</td>
</tr>
<tr>
<td>T16 – T18</td>
<td>0.00235</td>
<td>68.1</td>
<td>0.0146</td>
<td>68.2</td>
<td>0.1331</td>
<td>94.8</td>
<td>T16 – T18</td>
<td>64.6</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Longitudinal Reinforcement</th>
<th>( \varepsilon_y )</th>
<th>( f_y ) (MPa)</th>
<th>( \varepsilon_h ) (hardening)</th>
<th>( f_h ) (MPa)</th>
<th>( \varepsilon_u ) (max stress)</th>
<th>( f_u ) (MPa)</th>
<th>Transverse Steel</th>
<th>( f_y ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T8 – T12</td>
<td>0.00235</td>
<td>470</td>
<td>0.0131</td>
<td>470</td>
<td>0.1189</td>
<td>640</td>
<td>T8 – T12</td>
<td>511</td>
</tr>
<tr>
<td>T16 – T18</td>
<td>0.00235</td>
<td>470</td>
<td>0.0146</td>
<td>470</td>
<td>0.1331</td>
<td>654</td>
<td>T16 – T18</td>
<td>445</td>
</tr>
</tbody>
</table>

4.1.2 Instrumentation

Numerous experimental studies in the past, for example (Moyer and Kowalsky 2003) and (Hines et. al. 2004), utilized an array of linear potentiometers to measure cross section curvatures in the column hinge regions. These potentiometers were attached to the ends of imbedded threaded rods to measure the average strain in the tensile and compressive extreme fiber regions of the column. Since the potentiometers are placed at the ends of the rods, their measurements are affected by rotations of the rods themselves.

The experimental program presented in this paper utilized an innovative technique of applying a commercially available instrumentation system to measure large strains at the level of the reinforcement with multiple Optotrak Certus HD 3D position sensors produced by Northern Digital Inc. The Optotrak position monitoring system can read the location of target markers placed on the specimen in three dimensional space during a test, Figure 4.3. By calculating the change in three dimensional distances between target markers, strains can be determined with respect to the original unloaded gage lengths. The 3D accuracy of the Optotrak Certus system reported by Northern Digital Inc. is 0.1mm with a resolution of 0.01mm. Target markers were applied directly to six longitudinal bars in the extreme fiber regions of the column to obtain strain hysteresis, vertical strain profiles, cross section curvatures, curvature distributions, and fixed-end rotations attributable to strain penetration. To accomplish this, vertical strips of cover concrete overlaying the extreme fiber bars were
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blocked out over the instrumented region as shown in Figure 4.2. Strain gages were applied to layers of transverse steel overlaying the longitudinal reinforcement to observe the interaction between compressive demand, transverse steel strain, and buckling restraint. An illustration of the accuracy of the Optotrak system compared to traditional measurement techniques appears in Figure 4.3. A tensile test on a reinforcing bar was conducted with a 51mm Optotrak gage length, a 51mm extensometer gage length, and a centrally placed strain gage. Closer inspection demonstrates that the Optotrak strains oscillate around the measurements predicted by the conventional instrumentation, but the general trend is captured throughout the entire tensile test. Electrical resistance strain gages fail to remain attached at large inelastic strain levels, which are of interest to this study.

The top column displacement was obtained through a string potentiometer placed at the center of the lateral load. The lateral load and stroke of the 220kip (980kN) hydraulic actuator were measured through an integrated load cell and linear variable differential transformer (LVDT). An axial load cell monitored the contribution of one hydraulic jack to the total axial load of the column, the total axial load double the recorded value.

4.1.3 Loading Protocol

The specimens were subjected to various unidirectional top-column displacement histories including standardized laboratory reversed cyclic loading, and recreations of the displacement responses obtained from non-linear time history analysis of multiple earthquakes with distinct characteristics. The top-column displacement histories used in the tests are shown in Figure 4.4 through Figure 4.9. The experiments utilized a quasi-static displacement controlled loading procedure. The symmetric three-cycle-set load history is commonly used to evaluate the seismic performance of structural components. The load history begins with elastic cycles to the following increments of the analytically predicted first yield force: \( \frac{1}{4} F'_y \), \( \frac{1}{2} F'_y \), \( \frac{3}{4} F'_y \), and \( F'_y \). The experimental first yield displacement is then determined by taking the average of the recorded displacements during the first yield push and pulls cycles. The equivalent yield displacement, used to determine the displacement
ductility levels \( \mu_{\Delta_1} = (1 * \Delta_y) \), is then calculated as \( \Delta_y = \Delta'_y \left( \frac{M_n}{M'_y} \right) \). The symmetric three-cycle-set load history resumes with three balanced cycles at each of the following displacement ductility levels: 1, 1.5, 2, 3, 4, 6, 8, 10, 12, etc.

For earthquake time-history tests, the analytical top column displacement history is determined using non-linear time history analysis (NLTHA). The original acceleration input of the earthquake record is multiplied by a constant scale factor to produce a peak displacement response suitable for the experimental test. This is necessary because the amplitude of peak response is an important variable when comparing the performance of the columns subjected to different load histories. The goal of the experimental load history is not to re-produce the exact displacement response which the specific acceleration record may have created, but rather to compare the performance of columns subjected to specific characteristics in the displacement histories obtained from NLTHA. Specific earthquake top-column displacement response characteristics were chosen including: the number and amplitude of cycles prior to the peak, degree of symmetry, and peak displacement in each direction of loading. The symmetric three-cycle-set experiments (T9 and T16) were conducted prior to earthquake tests to establish the displacement ductility levels. The scaling factors of the acceleration input used in NLTHA of the earthquake load histories were determined based on the displacement capacities of T9 and T16, which had bar buckling during displacement ductility eight. Four earthquake records were scaled approximately displacement ductility nine while two records were scaled to ductility ten. The strains at the first yield displacement of each earthquake test were verified to confirm that the ductility levels from T9 and T16 remained appropriate. Specimens which had un-buckled reinforcement during the earthquake load histories were subjected to a symmetric three-cycle-set displacement history to evaluate the columns post-earthquake performance (T8b, T10b, T17b, T18b).
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Figure 4.4 Control Specimen Load Histories (Left) T9 and (Right) T16

Figure 4.5 Test 8 Chile 2010 Load History and Post EQ Three Cycle Set

Figure 4.6 Test 10 Chichi Load History and Post EQ Three Cycle Set
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Figure 4.7 (Left) T11 Kobe 1995 Load History and (Right) T12 Japan 2011 LH

Figure 4.8 Test 17 Llolleo 1985 Load History and Post EQ Three Cycle Set

Figure 4.9 Test 18 Darfield NZ Load History and Post EQ Three Cycle Set
4.2 Experimental Results

4.2.1 Damage Observations

The deformation capacity of all of the cyclically loaded specimens was limited by longitudinal bar buckling and subsequent rupture during later cycles of the load history. The following sequence of damage was observed: cracking, longitudinal reinforcement yield, cover concrete crushing, yielding of transverse steel, bar buckling, and then reinforcement rupture. Rupture of transverse steel was never observed. The first significant loss of strength occurred when previously buckled reinforcement ruptured in tension.

For the load history variable tests, cover concrete crushing occurred at an average compressive strain of 0.0045, when the compressive demand exceeded the unconfined concrete compressive strength. The average compressive strain measured at spiral yield in the confinement region was 0.015. As a reference, the Mander et. al. (1988) ultimate concrete compressive strain of specimens T8-T12 is equal to 0.0185 and 0.0183 for tests T16-T18. On average, spiral yielding in the confinement region occurred at approximately 80% of the Mander et. al. (1988) ultimate concrete compressive strain. Noticeable influences of displacement history on cover concrete crushing on spiral yielding were not discernible based on the test results.

4.2.2 Test 11 – Response to the Kobe 1995 Earthquake

Sample results from test unit LH4 are presented to explain the influence of load history on accumulated strains in the longitudinal and transverse steel. The acceleration input of the Kobe 1995 earthquake record was multiplied by 1.13 to produce an analytical top column response equivalent to displacement ductility 9.9, as shown in Figure 4.7. A 24” (610mm) diameter bridge column with a constant axial load equivalent to \( P/(f_c' A_g) = 6.2\% \), confined with a #3 (9.5mm) spiral at 2” (51mm) on center, was subjected to a quasi-static loading procedure which recreated the analytical Kobe displacement history obtained from NLTHA. The test began with a small pull cycle followed by a near monotonic push to the
peak displacement ductility of 9.9. The North longitudinal reinforcement, shown in Figure 4.11, is placed into tension during push cycles while the South side is subjected to compression, refer to Figure 4.12. The push cycle to displacement ductility 9.9 resulted in a peak tensile strain of 0.059 in the North extreme fiber bar, a peak compressive strain of -0.037 in the South extreme fiber bar, and two layers of inelastic transverse steel in the compression zone (Figure 4.12). The particular longitudinal reinforcement gage lengths depicted in Figure 4.11 and Figure 4.12 do not align with the peak tensile and compressive strains, but rather with the location which later outwardly buckled. This region is presented because it provides a clear representation of when buckling occurred, since strains measured by target markers on the convex side of a buckled bar increase under compressive demand. The peak tensile strain of 0.059 was not sufficient to buckle the North bar during the subsequent reversal to displacement ductility -6.1.

At displacement ductility -6.1, tensile strains in the South bar reached 0.033, compressive strains in the North bar measured -0.0119, and the transverse steel on the North side of the specimen remained elastic (Figure 4.11). This peak tensile strain, combined with multiple layers of inelastic transverse steel, was sufficient to buckle the South extreme fiber bar after reversal of loading. At this time, the measured strains in the South bar no longer represent engineering strains due to the outward buckled deformation between target markers. As the South bar buckles outwards, the strain in the transverse steel begins to rapidly increase as shown in Figure 4.12. A peak tensile strain of 0.053 was measured in the North extreme fiber bar at the end of the push cycle to displacement ductility 9.3. Again, note that this peak tensile strain occurred over an adjacent gage length to that shown in Figure 4.11 which depicts outward buckled region. It was during the following reversal from displacement ductility 9.3 that the North extreme fiber bar visually buckled under compressive stress. The outward buckled deformation in the North bar caused measured strains in the transverse steel to sharply increase as shown in Figure 4.11. Prior to buckling of the North extreme fiber bar, the transverse steel remained elastic.
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Figure 4.10 (Left) Kobe 1995 EQ Load History and (Right) Hysteretic Response

Figure 4.11 (Left) North Bar Strain History (Right) Overlying Spiral Strain History

Figure 4.12 (Left) South Bar Strain History (Right) Overlying Spiral Strain History
4.2.3 The Effect of Load History on Reinforcement Bar Buckling

The main impact of load history on column behavior is its effect on accumulated strains within the longitudinal and transverse reinforcement. Large concrete compressive demand results in inelastic strains in the transverse steel, which reduces its effectiveness in restraining the longitudinal bars from buckling. Load histories with compressive demand sufficient to produce inelastic transverse steel may require lower values of peak tensile strain to initiate bar buckling after reversal of load. The symmetric three-cycle-set load history is more severe than the load histories produced by actual earthquakes, when evaluated to the same peak displacement, due to the balanced repeated cycles at each ductility level. Multiple cycles at the same amplitude allow each side of the specimen to be subjected to the peak compressive and tensile cycles, creating the worst situation for bar buckling a given peak displacement.

The relationship between strain and displacement during the largest push and pull cycles of each load history variable test appear in Figure 4.14 and Figure 4.15. Not all of the peak excursions produced bar buckling after reversal from the peak tensile strains shown, for instance the Kobe 1995 record (LH4) did not buckle the North bar until the second peak as discussed in the previous section. An extreme fiber bar buckling summary is shown in Table 4.4 and Table 4.5. The 24” (610mm) diameter bridge columns contained a #3 (9.5mm) spiral at either 2” (51mm) or 1.5” (38mm) on center. A specimen with each transverse steel detailing was subjected to a symmetric three-cycle-set load history (T9 and T16) which produced bar buckling during cycles at displacement ductility eight. For the specimen subjected to T9, fracture of previously buckled reinforcement occurred during repeated cycles at displacement levels equal to those necessary to initially produce bar buckling. The following four earthquake load histories: Chichi 1999 (T10), Chile 2010 (T8), Llolleo 1985 (T17), and Darfield 2010 (T18) were scaled to produce peak response displacement ductility of 8.8, 8.7, 9.0, and 9.0 respectively. The longitudinal steel placed into tension during the peak push cycles of these four load histories did not buckle during the earthquake record.
For the case of the Darfield 2010 load history (T18), the peak push displacement led to inelastic layers of transverse steel, and the following pull cycle to displacement ductility -7.3 had sufficient tensile demand to buckle a single bar during the subsequent reversal of load. The four specimens were then subjected to symmetric three-cycle-set load histories (T10b, T8b, T17b, and T18b) to determine the post-earthquake performance of the columns. Reinforcement buckling occurred during cycles at displacement ductility six of the cyclic load histories after the Chichi 1999 (T10b) and Llolleo 1985 (T17b) records. This indicates that the earthquake load histories led to a reduction in the displacement capacity at bar buckling of two ductility levels during tests T10b and T17b when compared to initially undamaged specimens T9 and T16. This is a potential issue for post-earthquake inspection because there was no visible indication of the reduced ductility capacity after T10 and T17.

Bar buckling occurred during ductility eight of the cyclic load history conducted after the Chile 2010 earthquake (T8b). The cyclic load history after the Darfield 2010 record (T18b) ruptured the previously buckled bar during ductility six. This shows that increases in displacement demands beyond those which were initially required buckle a reinforcing bar (displacement ductility -7.3 with inelastic spiral restraint) are not required to rupture the previously buckled bar. A dependable level of displacement at which fracture of a previously buckled bar would occur cannot be reliably developed because it is dependent on the severity of the buckled deformation. Since bar buckling permanently deforms the spiral restraint, even smaller cycles after the initiation of buckling degrade the core concrete and increase the buckled deformation. Uniaxial bar tests by Restrepo-Posada et. al. (1994) showed that micro-cracks develop at the locations of ribs on the compression side of a severely buckled bar, Figure 4.13. Once these micro-cracks develop, the future tensile capacity of the bar is compromised.

To produce buckling after reversal from the peak displacement response of earthquake load histories, the Kobe 1995 (T11) and Japan 2011 (T12) records were scaled to displacement ductility 9.9 and 10 respectively. As discussed previously, bar buckling did not
occur until reversal from the second peak cycle to displacement ductility 9.3 of the Kobe record (LH4).

The relationship between strain and displacement along the envelope curve of cyclic response does not appear to be affected by seismic load history. Since the curves shown in Figure 4.14 and Figure 4.15 are from the peak cycles of their respective load histories, only portions of the curve lie upon the envelope of cyclic response. At low ductility levels the measured tensile strains during the Kobe 1995 (T11) and Darfield 2010 (T12) peak cycles are larger than the other records because these were near monotonic push cycles to the peak displacement while the crack distribution was still forming.

Figure 4.13 Micro-Cracks Present on a Buckled Reinforcing Bar, Photo Obtained from Restrepo-Posada et. al. (1994)
Figure 4.14 North Bar (Left) Peak Push Cycle Tensile Strain-Displacement and (Right) Peak Pull Cycle Compressive Strain-Displacement

Table 4.4 Bar Buckling Summary for the North Extreme Fiber Bar

<table>
<thead>
<tr>
<th>Test</th>
<th>Disp. (mm) before Buckling North Bar</th>
<th>Disp. Ductility before Buckling North Bar</th>
<th>Peak Tensile Strain of North Bar</th>
<th>Peak Strain Compressive</th>
<th>Peak Spiral Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>T9</td>
<td>171</td>
<td>8 (1st Cycle)</td>
<td>0.053</td>
<td>-0.018</td>
<td>0.0027</td>
</tr>
<tr>
<td>T8</td>
<td>No Buckling (184)</td>
<td>No Buckling, 8.7</td>
<td>No Buckling (0.051)</td>
<td>-0.013</td>
<td>0.0021</td>
</tr>
<tr>
<td>T8b</td>
<td>169</td>
<td>8 (1st Cycle)</td>
<td>0.043</td>
<td>-0.028</td>
<td>0.0255</td>
</tr>
<tr>
<td>T10</td>
<td>188</td>
<td>No Buckling, 8.9</td>
<td>No Buckling (0.052)</td>
<td>-0.003</td>
<td>0.0009</td>
</tr>
<tr>
<td>T10</td>
<td>169</td>
<td>No Buckling, 8</td>
<td>No Buckling (0.048)</td>
<td>-0.016</td>
<td>0.0115</td>
</tr>
<tr>
<td>T11</td>
<td>Initial 210, Buckled after 2nd Peak (197)</td>
<td>Initial 9.9, Buckled after 2nd Peak 9.3</td>
<td>Initial 0.059, Buckled after 2nd Peak (0.053)</td>
<td>-0.012</td>
<td>0.0017</td>
</tr>
<tr>
<td>T12</td>
<td>209</td>
<td>10</td>
<td>0.058</td>
<td>-0.021</td>
<td>Debonded after 0.008</td>
</tr>
<tr>
<td>T16</td>
<td>169</td>
<td>8 (3rd Cycle)</td>
<td>0.056</td>
<td>-0.019</td>
<td>0.0096</td>
</tr>
<tr>
<td>T17</td>
<td>No Buckling (190)</td>
<td>No Buckling, 9</td>
<td>No Buckling (0.055)</td>
<td>-0.023</td>
<td>0.0091</td>
</tr>
<tr>
<td>T17b</td>
<td>127</td>
<td>6 (2nd Cycle)</td>
<td>0.035</td>
<td>-0.039</td>
<td>Debonded after 0.011</td>
</tr>
<tr>
<td>T18</td>
<td>No Buckling (190)</td>
<td>No Buckling, 9</td>
<td>No Buckling (0.062)</td>
<td>-0.021</td>
<td>0.0031</td>
</tr>
<tr>
<td>T18b</td>
<td>No Buckling (127)</td>
<td>No Buckling, 6 (2nd Cycle)</td>
<td>No Buckling (0.036)</td>
<td>-0.023</td>
<td>0.0039</td>
</tr>
</tbody>
</table>
Figure 4.15 South Bar (Left) Peak Pull Cycle Tensile Strain-Displacement and (Right) Peak Push Cycle Compressive Strain-Displacement

<table>
<thead>
<tr>
<th>Test</th>
<th>Disp. (mm) before Buckling South Bar</th>
<th>Disp. Ductility before Buckling South Bar</th>
<th>Peak Tensile Strain of South Bar</th>
<th>Peak Strain Compressive</th>
<th>Peak Spiral Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>T9</td>
<td>-170</td>
<td>-8 (1st Cycle)</td>
<td>0.051</td>
<td>-0.015</td>
<td>0.0034</td>
</tr>
<tr>
<td>T8</td>
<td>No Buckling (-112)</td>
<td>No Buckling, -5.3</td>
<td>No Buckling (0.032)</td>
<td>-0.022</td>
<td>0.0055</td>
</tr>
<tr>
<td>T8b</td>
<td>-169</td>
<td>-8 (1st Cycle)</td>
<td>0.048</td>
<td>-0.032</td>
<td>Off Scale &gt;0.036</td>
</tr>
<tr>
<td>T10</td>
<td>No Buckling (-54)</td>
<td>No Buckling, -2.5</td>
<td>No Buckling (0.016)</td>
<td>-0.032</td>
<td>0.0131</td>
</tr>
<tr>
<td>T10b</td>
<td>-127</td>
<td>-6 (1st Cycle)</td>
<td>0.038</td>
<td>-0.045</td>
<td>Off Scale &gt;0.036</td>
</tr>
<tr>
<td>T11</td>
<td>-129</td>
<td>-6.1</td>
<td>0.033</td>
<td>-0.037</td>
<td>0.0159</td>
</tr>
<tr>
<td>T12</td>
<td>Initial -166, Buckled after Peak to 107</td>
<td>Initial -7.9, Buckled after Peak to -5.1</td>
<td>Initial 0.044, Buckled after Peak to 0.028</td>
<td>-0.032</td>
<td>0.0120</td>
</tr>
<tr>
<td>T16</td>
<td>-170</td>
<td>-8 (1st Cycle)</td>
<td>0.052</td>
<td>-0.030</td>
<td>0.0095</td>
</tr>
<tr>
<td>T17</td>
<td>No Buckling (-128)</td>
<td>No Buckling (-6)</td>
<td>No Buckling (0.039)</td>
<td>-0.039</td>
<td>0.0077</td>
</tr>
<tr>
<td>T17b</td>
<td>-127</td>
<td>-6 (Second)</td>
<td>0.036</td>
<td>-0.043</td>
<td>Debonded after 0.019</td>
</tr>
<tr>
<td>T18</td>
<td>-154</td>
<td>-7.3</td>
<td>0.047</td>
<td>-0.048</td>
<td>Off Scale &gt;0.016</td>
</tr>
</tbody>
</table>


4.3 Spread of Plasticity

4.3.1 Test 16 – Deformation Components Three Cycle Set Load History with #3 Spiral at 1.5” (38mm)

The Optotrak position monitoring system allows for a closer examination of column flexure and strain penetration deformation components. To describe the process and capabilities of the instrumentation technique, sample test results related to the spread of plasticity in a symmetric three-cycle-set load history (T16) are presented. The displacement history along with data points which mark cycles where cover crushing, spiral yield, visible bar buckling, and bar fracture occurred in appear in Figure 4.16. The measured compressive and tensile strains for South and North extreme fiber bars during push cycles appear in Figure 4.16. This figure shows strain profiles for extreme fiber bars on each side of the specimen to illustrate the effects of tension shift. Figure 4.17 shows that cracks near the footing remain effectively horizontal, but above this section the flexural shear crack distribution is inclined. The cracks form a naturally inclined cut in a free body diagram representation of the bridge column. Hines et. al. (2004) developed a procedure for evaluating the effects of tension shift on the spread of plasticity by quantifying stress components acting along an inclined flexural shear crack. Due to the effects of tension shift, compressive strains are concentrated near the footing-column interface and tensile strains are fanned out to a greater height following the crack distribution. This mechanism is observable in Figure 4.16.

The instrumentation technique allows for monitoring of individual strain hysteresis, such as Figure 4.17 which depicts the outward buckled region of the North extreme fiber bar. Stable hysteretic loops were observed until the second pull cycle of displacement ductility eight where measurable deformation occurred at the location consistent with outward visible buckling during the following pull cycle. This measurable deformation was not discernible by eye, and therefore poses an issue for to post-earthquake inspection.
Since the deformations at performance limit states are required for design, a closer look at the relationship between strain and displacement is warranted. Monotonic moment-curvature analysis is typically utilized in design because it provides an accurate prediction for the envelope curve of the lateral force versus deformation response for flexure-dominated columns with constant axial load, as shown in Figure 4.21. Experimental tests by Esmaeily and Xiao (2005) show that alterations to the cyclic response prediction must be made to account for variable axial loads. The monotonic moment-curvature analysis was conducted in a script called CUMBIA (Montejo and Kowalsky 2007). The program utilizes the following material models for the confined and unconfined concrete and reinforcing steel: (Mander et al. 1988) and (King et al. 1986). Top column displacements are obtained using the plastic hinge method and shear displacement models presented in Priestley et. al. (2007).

A comparison of the measured and predicted relationships between strain and displacement during push cycles appears in Figure 4.18. The solid lines represent individual push cycles which begin with the column at zero displacement and end at the peak of the respective displacement ductility level. The dashed lines represent the subsequent reversal of loading. Moment-curvature analysis over predicts the measured tensile strains and under predicts compressive strains at an increasing rate at higher ductility levels. Due to the effects of tension shift, the tensile strains are fanned out to a greater height above the footing-column interface, which may decrease the peak tensile strain at the base of the column (near the footing). Compression strains are concentrated at the base of the column as the hinge region rotates about inclined flexural shear cracks. These compressive strains are further localized at the location of inelastic layers of transverse steel. Syntzirma et. al. (2010) proposed that the lumped rotation at the support caused by strain penetration of longitudinal steel into the footing may lead to a local increase in the axial strain in the compressive zone.

The measured compression strains in the South reinforcing bar (Figure 4.16) are larger at the location where several layers of transverse steel entered the inelastic range, as shown in Figure 4.19. The spiral layer closest to the footing-column interface remained elastic due to the additional confinement provided by the footing. Two layers of transverse steel entered
the inelastic range during displacement ductility six, but bar buckling did not occur until reversal from tensile strains sustained during the first pull cycle of ductility eight. The outward buckled region of the South extreme fiber bar occurred over the previously inelastic transverse steel layers as shown in Figure 4.19.

The measured strains of six reinforcing bars are plotted along the cross section to obtain curvatures in Figure 4.20. The curvature was taken as the slope of the least squared error line. Curvature profiles obtained from thirty-two horizontal cross sections at different heights above the footing appear in Figure 4.20. Procedures developed by Hines et al. (2004) were followed to extract numerical information related to the shape of the linear plastic curvature profiles. The dashed lines for each curvature distribution represent a least squared error linear fit to the plastic portion of the measured curvatures. Following recommendations from Hines et al. (2004), the extent of plastic curvatures above the footing is calculated by determining where the linear plastic curvature distribution intersects the elastic curvature profile, shown as a grey dashed line. The elastic curvature profile forms a triangular shape along the length of the column with a value of zero at the top and the yield curvature at the base. The total base section curvature is determined by the intersection of the plastic least squared error line and the x-axis at the footing-column interface.

The target marker on each bar placed closest to the footing-column interface can be used to measure the effects of strain penetration. Development of fully anchored column longitudinal bars into the footing leads to bond slips along the partially anchored region of the bars near the footing-column interface, as described by Zhao and Sritharan (2007). They additionally note that this bond slip is not a pull-out of the entire bar embedment length resulting from poor bond between the concrete and reinforcing bar. If the measured bond slips of the target markers are plotted along the cross section, the fixed-end rotation attributable to strain penetration may be calculated as the slope of a least squared error line, Figure 4.21.

The hysteretic response in Figure 4.21 was obtained from a string potentiometer which measured deflections at the center of the applied lateral load. This total deformation is the
sum of the column flexure, column shear, and strain penetration components. The flexural displacement may be determined by integrating the measured curvature distribution and adding the strain penetration deformation component. The curvatures above the instrumented region are assumed to follow the triangular yield curvature profile. The integrated displacements from the Optotrak system are compared to the measured displacements in Figure 4.21. The good agreement suggests that the shear deformation component is small relative to the total deformation.

Figure 4.16  (a) T16 Load History and (b) Vertical Strain Profiles

Figure 4.17  (a) Inclined Flexural Shear Cracks and (b) North Strain Hysteresis
Chapter 4: The Effect of Load History on Column Performance

Figure 4.18 (a) North Tensile and (b) South Bar Compressive Strain-Displacement

Figure 4.19 (a) South Spiral Strains and (b) South Buckled Bar

Figure 4.20 (a) Cross Section Strain Profiles and (b) Vertical Curvature Profiles
Chapter 4: The Effect of Load History on Column Performance

Figure 4.21 (a) Strain Penetration Rotation and (b) Optotrak Integrated Displacements

Figure 4.22 (a) Measured Spread of Plasticity and (b) Curvature Profiles for T17

Figure 4.23 (a) Measured Rotations and (b) Equivalent Strain Penetration Lengths
4.3.2 Measured Spread of Plasticity

Plastic curvature profiles have a linear distribution which intersects the yield curvature profile at a height above the footing termed the extent of plasticity. This process is shown visually in the curvature profiles for the specimen subjected to the Llolleo Chile 1985 (T17) displacement history in Figure 4.22. For this test, the data points come from the first occurrence of the displacement ductility level along the envelope curve of cyclic response. The measured extent of plasticity vs. base curvature ductility is shown in Figure 4.22. The spread of plasticity for column tests with varying geometry and predictive equations for the extent of plasticity appear in Hines et. al. (2004).

In design, limit state curvatures are converted to target displacements using an equivalent curvature distribution. While there are many versions of the plastic hinge method, they all operate by integrating a simplified curvature distribution with the moment area method. The moment-curvature analysis presented in this section utilizes the plastic hinge method presented in Priestley et. al. (2007). In this method, the elastic and plastic curvature distributions are separated into simplified shapes to facilitate design. The elastic flexural displacement is determined using a triangular curvature distribution. The plastic flexural displacement is obtained using a uniform curvature distribution with a constant height called the plastic hinge length. The width of the rectangle is equal to the plastic curvature at the base section. To account for the effects of strain penetration, the curvature distribution extends into the footing by a depth termed the strain penetration length.

The use of a constant plastic hinge length does not account for the spread of plasticity observed in the physical tests. The constant plastic hinge length is not physical parameter; it is a numerical convenience to obtain the top column displacement. Improvements to the plastic hinge method for member deformation are necessary to produce accurate limit state target displacements at levels of response other than the ultimate condition. A further complication is noticed upon inspection of the tensile and compressive strain predictions in Figure 4.18. Accurate tensile strain predictions would require a plastic hinge length which
expands at higher ductility levels. The larger deformations at a given curvature would increase the accuracy of the prediction. The opposite is true for compression, where a shrinking plastic hinge length is needed to match measured compressive strain and displacement relationship. This is because moment-curvature analysis cannot capture the localization of compressive strains at the base of the column observed in test results with inelastic confinement steel.

The measured base rotation attributable to strain penetration is plotted against the base curvature ductility in Figure 4.21. Equivalent strain penetration lengths are determined by dividing the fixed-end rotations by the base curvatures in Figure 4.23. The top column displacement due to strain penetration is equal to the base curvature multiplied by the equivalent strain penetration length multiplied by the column clear height. A constant equivalent strain penetration length appears suitable for the range of curvature ductility presented in Figure 4.23. The effect of other variables on the spread of plasticity is studied in later sections of this report.

4.4 Conclusions

In this paper, the influence of unidirectional lateral displacement history on performance limit states, the relationship between strain and displacement, and the spread of plasticity in reinforced concrete bridge columns was explored. Results have shown that reinforcement bar buckling was influenced by load history, but the relationship between strain and displacement along the envelope curve of cyclic response was not. The symmetric three-cycle-set load history was found to be more severe than the displacement history produced by real earthquakes, when evaluated to the same peak displacement, due to the high number of inelastic reversals of loading of increasing magnitude.

Load histories with compressive demand sufficient to produce inelastic transverse steel may require lower values of peak tensile strain to initiate bar buckling after reversal of load. Every buckled longitudinal bar, with the exception for one, occurred over previously inelastic layers of transverse steel restraint. Additional research is required in order to relate
compressive demands to anticipated strains in the confinement steel of flexural tests. When bar buckling occurs over multiple spiral layers, the anticipated level of restraint provided by the transverse steel should include the effects of confinement.

In two experiments, fracture of previously buckled reinforcement occurred at levels of displacement equal to or lower than those required to initially produce bar buckling. The authors believe that a dependable level of displacement at which fracture of a previously buckled bar would occur cannot be reliably developed because it is dependent on the severity of the buckled deformation.

A technique of applying a commercially available position monitoring system (Optotrak Certus HD produced by Northern Digital Inc.) was developed which allows for monitoring longitudinal steel strains until bar buckling and subsequent fracture. The use of a constant plastic hinge length to calculate the displacements at varying levels of response does not account for the measured spread of plasticity in reinforced concrete bridge columns.
Chapter 5: Impact of Steel Content, Aspect Ratio, and Axial Load Ratio on Column Performance

This section discusses a research program supported by the Alaska Department of Transportation and Alaska University Transportation Center aimed at defining accurate limit state displacements which relate to specific levels of damage in reinforced concrete bridge columns subjected to seismic hazards. The experimental portion of the study aims to assess the performance of thirty large scale circular bridge columns. A key feature of the experiments is the high fidelity strain data obtained through the use of an optical 3D position measurement system. In this section, this data is utilized to explore the impact of design variables on key performance limit states. These design variables include: (1) lateral displacement history, (2) axial load, (3) longitudinal steel content, (4) aspect ratio, and (5) transverse steel detailing. The impact of lateral displacement history was the focus of Chapter 4, so this section instead focuses only on specimens subjected to symmetric three-cycle-set load histories.

The following sequence of damage was observed in all of the cyclically loaded tests: (1) concrete cracking, (2) longitudinal steel yield, (3) cover concrete crushing, (4) confinement steel yielding, (5) longitudinal bar buckling, and (6) fracture of previously buckled reinforcement. The deformation capacity of all of the specimens was limited by bar buckling and subsequent fracture. Spiral fracture was never observed, since longitudinal steel rupture occurred first, resulting in large levels of strength loss. The instrumentation system allowed for monitoring of longitudinal steel and transverse steel strains in the plastic hinge region. In this section, the impact of design variables on measured strains prior to the following limit states are explored: (1) cover concrete crushing, (3) confinement steel yielding and (3) longitudinal bar buckling. In Chapter 8, strain limit design expressions are developed based on the information provided in this section. In Chapter 7, an equivalent curvature
distribution which is consistent with strain-based displacement predictions is presented. In performance based design, an understanding of the damping-ductility relationship is needed to assess the correct value of damping at the design limit state. For columns subjected to cyclic loading, the equivalent viscous damping is calculated and compared to current design expressions which are based on specific hysteretic rules. In general, the equivalent viscous damping is equal to the viscous plus the hysteretic damping, both corrected and combined in a manner which is consistent with a design procedure based on secant stiffness to the design limit state.

5.1 Test Setup and Instrumentation

The goal of the experimental program is to investigate the impact of load history and other design variables on the relationship between strain and displacement, performance strain limits, and the spread of plasticity. The main variables for the thirty tests include: (1) lateral displacement history, (2) axial load, (3) longitudinal steel content, (4) aspect ratio, and (5) transverse steel detailing. The specimen was designed to represent a single degree of freedom bridge column subjected to lateral and axial load, Figure 5.1. The test specimen consists of a footing, column, and loading cap. The footing is a capacity protected member which secures the specimen to the lab strong floor using post tensioned bars. A 200kip hydraulic actuator, with a 40in stroke capacity, applies lateral load to the loading cap of the specimen. A spreader beam, two hydraulic jacks, and a load cell are placed above the loading cap to apply a constant axial compressive load. The top column displacement was obtained through a string potentiometer placed at the center of the lateral load.

The experimental program utilized multiple Optotrak Certus HD 3D position sensors developed by Northern Digital Inc. to monitor material strains. The position sensors track the locations of the target markers in 3D space, returning X-Y-Z spatial coordinates with an accuracy of 0.1mm with a resolution of 0.01mm. Two different cross sections were utilized in the study, an 18” and a 24” diameter configuration shown in Figure 5.2. The 24” configuration had 16 A706 longitudinal bars of either #6 (0.75 in) or #7 (0.875 in) diameter
and a #3 (0.375 in) or #4 (0.5 in) A706 spiral at variable spacing. For both specimens the cover depth to the outside of the spiral was ½”, which led to an outside spiral diameter of either 23” or 17”.

Two different instrumentation techniques were utilized for the 24” specimens. The first method, shown in the middle and left photos of Figure 5.1, utilized vertical cover concrete blockout strips over extreme fiber reinforcement which were installed during construction. This technique had two Optotrak position monitors, one facing each extreme fiber region. The blockout reached the outside surface of the longitudinal steel, where target markers were directly applied to the reinforcement. Care was taken during construction to insure that the spiral reinforcement was always in direct contact with the longitudinal reinforcement, therefore the blockout did not interfere with core concrete confinement or longitudinal bar restraint. The second instrumentation method, shown in Figure 5.3, had a full cover concrete blockout in the plastic hinge region. This technique utilized three Optotrak position monitors, two facing the extreme fiber regions, and one facing a shear face. This allowed for instrumentation of additional longitudinal bars and transverse steel within the cross section. The Optotrak spatial coordinate output from this technique is shown in Figure 5.3.

The 18” column configuration utilized 10 A706 longitudinal bars of either #6 (0.75 in) or #8 (1 in) diameter and a #3 (0.375 in) A706 spiral at 2” on center. All of the 18” diameter specimens utilized the complete cover blockout instrumentation method, shown in the right photo of Figure 5.1. While all of the 24” specimens had an 8ft cantilever length (L/D = 4), the following aspect ratios were evaluated for 18” specimens: 8ft, 11ft, and 13ft for (L/D = 5.33, 7.33, and 8.67). The specimens were constructed in groups of six specimens, where each test series evaluated the impact of specific design variables. The impacts of these variables are discussed in individual sections of this chapter. An overview of the longitudinal and transverse reinforcement properties for each test series is shown in Table 5.1. Note that Specimens 1-6 and Test 7 have been excluded, since they utilized a separate instrumentation technique whose measured strains ultimately should not be compared to the improved techniques previously mentioned.
5.2 Symmetric Three-Cycle-Set Loading Protocol

The symmetric three-cycle-set load history is commonly used to evaluate the seismic performance of structural components. An example of this load history utilized in column test twenty-five (T25) is shown in Figure 5.4. The load history begins with elastic cycles to the following increments of the analytically predicted first yield force: $\frac{1}{4} F'_y$, $\frac{1}{2} F'_y$, $\frac{3}{4} F'_y$, and $F'_y$. The experimental first yield displacement is then determined by taking the average of the recorded displacements during the first yield push and pulls cycles. The equivalent yield displacement, used to determine the displacement ductility levels ($\mu_{\Delta 1} = 1 \ast \Delta_y$), is then calculated as $\Delta_y = \Delta'_y(M_n/M'_y)$.

The symmetric three cycle set load history resumes with three balanced cycles at each of the following displacement ductility levels: 1, 1.5, 2, 3, 4. The traditional technique utilizes an increase of two displacement ductility levels for each series of three cycles past displacement ductility four. This load history was used for column T9, and T13 through T16. It became apparent that the increase from displacement ductility 4-6-8 was too large when the influence of individual variables was desired. For columns T19 through T30, a single displacement ductility increase was used for each set of cycles beyond displacement ductility four.
Chapter 5: Impact of Steel Content, Aspect Ratio, and Axial Load Ratio

Figure 5.1 (Left and Middle) 24” Diameter Columns with 8ft Cantilever Lengths, (Right) 18” Diameter Columns Either 8ft, 11ft, or 13ft Cantilever Lengths

Figure 5.2 Tests 25-30 Cross Sections and Bar Designation for Both Diameters
Chapter 5: Impact of Steel Content, Aspect Ratio, and Axial Load Ratio

Figure 5.3 Target Marker Application and Optotrak Spatial Output

Table 5.1 Longitudinal and Transverse Reinforcement Properties

<table>
<thead>
<tr>
<th>Longitudinal Reinforcement</th>
<th>Bar Size, Bar Diameter (in)</th>
<th>$e_y$ (ksi)</th>
<th>$f_y$ (ksi)</th>
<th>$e_{h}$ (hardening) (ksi)</th>
<th>$f_{h}$ (ksi)</th>
<th>$e_u$ (max stress) (ksi)</th>
<th>$f_u$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T8 – T12</td>
<td>#6, (0.75 in)</td>
<td>0.00235</td>
<td>68.1</td>
<td>0.0131</td>
<td>68.2</td>
<td>0.1189</td>
<td>92.8</td>
</tr>
<tr>
<td>T13 – T18</td>
<td>#6, (0.75 in)</td>
<td>0.00235</td>
<td>68.1</td>
<td>0.0146</td>
<td>68.2</td>
<td>0.1331</td>
<td>94.8</td>
</tr>
<tr>
<td>T19 – T24</td>
<td>#6, (0.75 in)</td>
<td>0.00250</td>
<td>68.1</td>
<td>0.0153</td>
<td>68.1</td>
<td>0.1208</td>
<td>92.4</td>
</tr>
<tr>
<td>T25 – T26</td>
<td>#7, (0.875 in)</td>
<td>0.00240</td>
<td>69.7</td>
<td>0.0126</td>
<td>69.7</td>
<td>0.1144</td>
<td>95.5</td>
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<td>T27 – T29</td>
<td>#6, (0.75 in)</td>
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<td>0.0136</td>
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<td>0.1178</td>
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<td>T30</td>
<td>#8, (1 in)</td>
<td>0.00243</td>
<td>70.5</td>
<td>0.0110</td>
<td>70.5</td>
<td>0.1093</td>
<td>97.7</td>
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<table>
<thead>
<tr>
<th>Transverse Reinforcement</th>
<th>Bar Size, Bar Diameter (in)</th>
<th>$f_y$ (ksi)</th>
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</thead>
<tbody>
<tr>
<td>T8 – T12</td>
<td>#3, (0.375 in)</td>
<td>74.1</td>
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<tr>
<td>T13</td>
<td>#4, (0.5 in)</td>
<td>69.9</td>
</tr>
<tr>
<td>T14 – T18</td>
<td>#3, (0.375 in)</td>
<td>64.6</td>
</tr>
<tr>
<td>T19 – T24</td>
<td>#3, (0.375 in)</td>
<td>65.6</td>
</tr>
<tr>
<td>T25 – T30</td>
<td>#3, (0.375 in)</td>
<td>63.9</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Series</th>
<th>Optotrak Target Marker Instrumentation Technique</th>
</tr>
</thead>
<tbody>
<tr>
<td>T8 – T12</td>
<td>Vertical Blockout Strips</td>
</tr>
<tr>
<td>T13 – T18</td>
<td>Vertical Blockout Strips</td>
</tr>
<tr>
<td>T19 – T24</td>
<td>Complete Cover Blockout</td>
</tr>
<tr>
<td>T25 – T30</td>
<td>Complete Cover Blockout</td>
</tr>
<tr>
<td>T1 – T6, T7</td>
<td>Post Extensions, Tests Excluded</td>
</tr>
</tbody>
</table>
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5.3 Gradual Bar Buckling Mechanism with Inelastic Transverse Steel Restraint

The deformation capacity of all of the cyclically loaded specimens was limited by longitudinal bar buckling and subsequent rupture of reinforcement during later cycles of loading. For many of these tests, measurable deformation could be observed in the recorded longitudinal and spiral strain hysteresis prior to the visible bar buckling observation. This deformation occurred once the transverse steel restraining the longitudinal bar went inelastic under prior compressive demands. This process is best demonstrated through analysis of strains collected from for Test 25. The 24” diameter column contains 16 #7 (A706) bars for longitudinal reinforcement \( \frac{A_{sl}}{A_g} = 2.1\% \) and a #3 A706 spiral at 2” on center \( \frac{4A_{sp}}{(D's)} = 1\% \). The specimen had an 8ft cantilever length \( \frac{L}{D} = 4 \), concrete strength \( f'_c = 6.29 \text{ ksi} \), and \( \frac{P}{(f'_{c}A_g)} = 5\% \) axial load. The imposed symmetric three-cycle-set displacement history and resulting hysteretic response appear in Figure 5.4 and Figure 5.5.

![Figure 5.4 Test 25 – Symmetric Three-Cycle-Set Load History](image-url)
5.3.1 North Reinforcement

The north reinforcement is exposed to tension during push cycles and compression during pull cycles (negative displacements). Measured spiral strains in five layers which overlaid the north extreme fiber bar appear in Figure 5.11 for pull cycles which placed the north side in compression. A compressive strain of -0.0091 was measured 1.63" above the footing on bar N3 during ($\mu_3^{-3} = -3.06"$), when the second spiral layer above the footing yielded. In the cycle naming system, $\mu_3^{-3}$ represents the third pull cycle of displacement ductility three. Successive cycles during displacement ductility four and five produced larger inelastic demands on the spiral reinforcement. For the second spiral layer above the footing, inelastic strains decreased the lateral restraint stiffness, which led to measurable outward deformation of the north extreme fiber bar before visible buckling. The measurable deformation formed a convex outward deformed region on the outside surface of the
longitudinal bar, and an inward concave region just above the outward deformation. The locations for this deformation are easier to inspect on the later buckled shape of bar N3 shown in the left photo of Figure 5.6. Optotrak gage lengths in the convex outward deformed region would show increased tensile strains during compression cycles which should have resulted in larger levels of compression, left half of Figure 5.8. Similarly, gage lengths on the concave region would show some degree of increased compression due to the deformed geometry, right half of Figure 5.8.

As a comparison, the gage length just above the convex and concave regions remained straight and produced stable hysteretic response, left half of Figure 5.10. It is important to note that all three gage lengths on Bar N3 showed rapid increase in the apparent deformation when visible buckling was observed during \( \mu_6^{-2} = -6.17" \). Although the measured compression strains in bar N3 may have been influenced by bar deformation, a compression strain of -0.0161 was measured 5.45" above the footing during \( \mu_5^{-3} = -5.11" \). The peak compression strain of -0.0269, measured during \( \mu_6^{-1} = -6.14" \), was likely influenced by bar deformation. A peak tensile strain of 0.0422 was measured 7.44" above the footing on bar N3 during \( \mu_6^{+2} = 6.14" \), before the bar visibly buckled during the subsequent reversal of load. A strong argument can be made that the bar actually buckled during \( \mu_6^{-1} = -6.14" \), which is why the measured compression strain during this cycle is disregarded.

Tensile strain in the second spiral layer above the footing, which overlaid the outward deformed region of bar N3, spiked during visible bar buckling, left half of Figure 5.7. The figure contains spiral data obtained from a strain gage and an Optotrak gage length, Figure 5.6. The Optotrak strains were calculated from arc-lengths which utilized the measured 3D distance chord lengths between two adjacent LEDs and the known outside diameter of the spiral reinforcement. It is important to note that arc-length calculations become inaccurate once severe yielding in the spiral leads to the reinforcement straightening out to the left and right of the localized yielding directly over the longitudinal bar. The arc-strains are still presented because the strain gage debonded, preventing further measurement prior to visible bar buckling.
The distribution of arc-strains measured around the circumference of the second spiral layer above the footing appears in Figure 5.12. The north region is under compression during pull cycles in the right half of Figure 5.12. The middle of the section corresponds to zero along the circumference, and negative values wrap around the north side of the specimen. Specifically, measured-arc strains which overlay the three north extreme fiber bars N2, N3, and N4 are shown with vertical dashed lines. The spiral yielding is more evenly distributed along the north circumference, when compared to localized spiral yielding observed on the south side of the specimen. Also, yielding along multiple spiral layers above the footing on the north side of the specimen is more evenly distributed than localized yielding on the south side observed in Figure 5.11. These two observations support the fact that bar buckling occurred one displacement ductility level later on the north side in comparison to the south. Furthermore, when the north side did buckle, three bars buckled simultaneously due to the distributed spiral yielding.

5.3.2 South Reinforcement

A peak tension strain of 0.0353 was measured 7.36” above the footing on bar S3 during \((\mu_5^{\pm} = -5.12”)\), before visible bar buckling occurred during the subsequent reversal of load. The tension strains measured in lower gage lengths on bar S3 were smaller, although adjacent bars S2 and S4 had large tensile strains near the footing-column interface. A compressive strain of -0.0125 was measured 1.58” above the footing on bar S3 during \((\mu_3^{\pm} = 3.08”)\), when the first spiral in the confinement region yielded. Measured spiral strains in five layers which overlaid the south extreme fiber bar appear in Figure 5.11 for compressive push cycles. Successive cycles during displacement ductility four produced large inelastic demands on the second layer of spiral reinforcement. The measured strains obtained from the Optotrak system and a strain gage for second spiral overlaying bar S3 appear in Figure 5.7. The spiral strains spiked when the bar visibly buckled during \((\mu_5^{\pm} = 5.10”)\).
Since Optotrak LEDs are placed on the outside surface of the bar, measurable deformation can be monitored in the concave and convex regions of the deformed shape, Figure 5.6. Note again that Figure 5.6 is taken after visible buckling. The outward deformed region (convex) developed in the gage length 3.47” above the footing on bar S3, Figure 5.9. Above the convex region, a concave region developed which increased the measured compression strains 5.44” above the footing, Figure 5.9. The region 7.36” above the footing on bar S3 appears to be unaffected by the measurable deformation which occurred below, Figure 5.10. The concave and convex deformed regions of bar S3 show a sharp deviation when visible bar buckling was observed during \( \mu_5^{+2} = 5.10” \). Spiral strains measured around the circumference of the second spiral layer above the footing depict large localized inelasticity at the location of the extreme fiber bars S3 and S4 during push cycles, Figure 5.12. The magnitude and localized nature of the spiral strains, both around the circumference (Figure 5.12) and vertically above the footing (Figure 5.11), contributed to bar buckling one displacement ductility level earlier than the north side of the specimen.

![Figure 5.6](image.png)  (Left) Three Buckled North Bars and (Right) Single Buckled South Bar
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Figure 5.7 Spiral Strain Hysteresis for (Left) North and (Right) South Buckled Region

Figure 5.8 North Bar Strains for (Left) Outward Deformed Region and (Right) Inward

Figure 5.9 South Bar Strains for (Left) Outward Deformed Region and (Right) Inward
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Figure 5.10 Strain Hysteresis above Deformation, (Left) North and (Right) South Bar

Figure 5.11 Spiral Strains Over Extreme Fiber Bars, (Left) North and (Right) South

Figure 5.12 Strains around Spiral Circumference, (Left) Push and (Right) Pull Cycles
5.4 Transverse Steel Detailing Variable Experiments

The effect of transverse steel detailing on restraint of longitudinal bars was the main variable for the experiments in Table 5.2. The 24” (610mm) diameter bridge columns contained 16 #6 (19mm) A706 bars for longitudinal reinforcement \( \left( \frac{A_{st}}{A_g} = 1.6\% \right) \) and either a #3 (9.5mm) or #4 (12.7mm) A706 spiral at variable spacing. The material properties for T9 differ from T13-T16 since the specimen came from a different test series, Table 5.1. The shear span for the cantilever columns was 8ft (244cm), and they had a moment to shear ratio of \( \frac{M}{VD} = 4 \). The specimens were subjected to a constant axial load of 170kips (756kN), \( \left( \frac{P}{f'_c A_g} \approx 5\% \right) \) depending on the concrete compressive strength. The following transverse volumetric steel ratios were investigated: \( \left( 4A_{sp}/(D's) \right) = 0.5\% \) (6d01 spacing), 0.7%, 1%, and two separate detailing arrangements for 1.3%. Both the volumetric ratio and spacing of the transverse steel are important when describing confinement and bar buckling restraint. Two columns were tested with 1.3% transverse steel, one with a #3 spiral at 1.5” spacing and another with a #4 spiral at 2.75” spacing. For comparison, a specimen was tested with a #3 spiral at 2.75” spacing.

An engineer has the most control over the size and spacing of transverse steel to improve buckling resistance. A summary of key displacement and strain values at damage observations in the tests appears in Table 5.3 and Table 5.4. The displacement and measured compression strain at the end of the cycle where cover concrete crushing was first observed appear in the second column. Similarly, the displacement and measured compression strain at the end of the cycle when spiral yielding was experimentally measured appear in the third column. It is important to note that the displacement at the end of the cycle was used because there are two contributing factors to spiral yielding, demands due to dilation of the core concrete under compression and demands related to the restraint of longitudinal reinforcement. The spiral yielding observation came from strain gages applied to spiral layers directly over the extreme fiber reinforcement. In some instances, spiral yielding is attributed more to the longitudinal bar restraint, since it occurred at low levels of
displacement during the reversal from the peak tensile strain. The peak displacement appears in the table since the restraint demand is linked to the prior tensile displacement, which has the same magnitude as the compressive cycle where spiral yielding was observed. That being said, most spiral yielding observations were accompanied by large levels of compression strain near the compressive peak of a given cycle. The third column shows the peak displacement and tensile strain which occurred just prior to bar buckling during the subsequent reversal of load. The previous peak compressive strain and displacement which occurred before the tensile cycle which induced bar buckling is shown in the fourth column. Compressive demand is linked to the degree of inelasticity and thus stiffness of the spiral layers restraining the longitudinal bar.

Influence of transverse steel detailing on the peak tensile strain and drift prior to bar buckling after reversal of load is shown graphically in Figure 5.13. The transverse steel variable tests from Table 5.2 are shown with red data points while all experiments T8 – T30 are shown with blue data points. The influence of other variables is decreased when inspecting relationships for just the experiments in Table 5.2. Specimens T14 \( (4A_{sp}/(D's)) = 0.5\% \) and T15 \( (0.7\%) \) had bar buckling during displacement ductility six. In comparison, T9 \( (1\%) \), T13 \( (1.3\%) \), and T16 \( (1.3\%) \) all had bar buckling occur during repeated cycles at displacement ductility eight. The peak tension strains measured prior to bar buckling are largely a function of the displacement amplitude at which bar buckling was observed. It became apparent that the increase from displacement ductility 4-6-8 was too large when the influence of individual variables was desired. For columns T19 through T30, a single displacement ductility increase was used for each set of cycles beyond displacement ductility four.

The degree of inelasticity in the spiral reinforcement overlaying the extreme fiber bar is direction related to its ability to restrain that bar from buckling. The measured peak spiral strains and measured longitudinal bar compressive strains for the peak compressive cycle prior to bar buckling appear in the right half of Figure 5.14. These measured peak compressive strains are quite large, and may be influenced by (1) localization of compression
over inelastic spiral layers and (2) the effects of measurable deformation prior to visible buckling which locally increase the perceived strain. The relationship between measured peak spiral strain and peak longitudinal steel tensile strains prior to bar buckling is shown in the left half of Figure 5.14. Larger inelastic spiral strains diminished its ability to restrain the reinforcement, which resulted in smaller peak tensile strains measured prior to bar buckling after reversal of load. There is still considerable scatter, since the distribution of spiral strains around the circumference of the column and over multiple layers influences bar buckling as discussed in the previous section for T25.

The impact of transverse steel content on measured compressive strains at the peak of the cycle when cover concrete crushing occurred appears in the left half of Figure 5.15. Among all of the variables investigated, volumetric steel ratio was the most impactful variable when considering cover concrete crushing. Larger amounts of confinement steel resulted in higher compressive strains at cover concrete crushing. This relationship did not carry over when considering the impact of volumetric steel ratio on the measured compressive strain at the peak of the cycle where spiral yielding was observed, right half of Figure 5.15. Higher amounts of confinement steel did not result in larger compressive strains measured at initial spiral yield. The majority of the columns had either 1% or 1.3% volumetric steel ratio, with only two specimens having 0.5% and 0.7%.
## Table 5.2 Transverse Steel Detailing Variable Experiments

<table>
<thead>
<tr>
<th>Test</th>
<th>Load History</th>
<th>D (in)</th>
<th>L/D</th>
<th>Long. Steel (ρ_l)</th>
<th>Spiral Detailing (ρ_s)</th>
<th>f'c (ksi)</th>
<th>P/f'c*Ag</th>
</tr>
</thead>
<tbody>
<tr>
<td>T9</td>
<td>3-Cycle-Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>6.81</td>
<td>5.5%</td>
</tr>
<tr>
<td>T13</td>
<td>3-Cycle-Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#4 at 2.75” (1.3%)</td>
<td>6.10</td>
<td>6.2%</td>
</tr>
<tr>
<td>T14</td>
<td>3-Cycle-Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 4” (0.5%)</td>
<td>6.64</td>
<td>5.7%</td>
</tr>
<tr>
<td>T15</td>
<td>3-Cycle-Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2.75” (0.7%)</td>
<td>7.23</td>
<td>5.2%</td>
</tr>
<tr>
<td>T16</td>
<td>3-Cycle-Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 1.5” (1.3%)</td>
<td>6.71</td>
<td>5.6%</td>
</tr>
</tbody>
</table>

## Table 5.3 Transverse Steel Variable Experiments, Limit State Displacements

<table>
<thead>
<tr>
<th>Test</th>
<th>Δ (μₜ₅) at Cover Crushing</th>
<th>Δ (μₜ₅) at Spiral Yielding</th>
<th>Δ (μₜ₅) at Peak Tension Prior to Bar Buckling</th>
<th>Δ (μₜ₅) at Peak Comp. Prior to Bar Buckling</th>
</tr>
</thead>
<tbody>
<tr>
<td>Side</td>
<td>North</td>
<td>South</td>
<td>North</td>
<td>South</td>
</tr>
<tr>
<td>T9</td>
<td>1.67” (2)</td>
<td>1.69” (2)</td>
<td>5.05” (6)</td>
<td>6.71” (8)</td>
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<tr>
<td>T13</td>
<td>1.61” (2)</td>
<td>1.60” (2)</td>
<td>4.85” (6)</td>
<td>6.46” (8)</td>
</tr>
<tr>
<td>T14</td>
<td>1.19” (1.5)</td>
<td>1.20” (1.5)</td>
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<td>4.80” (6)</td>
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<tr>
<td>T15</td>
<td>1.25” (1.5)</td>
<td>1.68” (2)</td>
<td>5.00” (6)</td>
<td>3.33” (4)</td>
</tr>
<tr>
<td>T16</td>
<td>1.65” (2)</td>
<td>1.66” (2)</td>
<td>4.98” (6)</td>
<td>4.98” (6)</td>
</tr>
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</table>

## Table 5.4 Transverse Steel Variable Experiments, Limit State Strains

<table>
<thead>
<tr>
<th>Test</th>
<th>εₛ at Cover Crushing</th>
<th>εₛ at Spiral Yielding</th>
<th>εₛ at Peak Tension Prior to Buckling</th>
<th>εₛ at Peak Comp. Prior to Buckling</th>
</tr>
</thead>
<tbody>
<tr>
<td>Side</td>
<td>North</td>
<td>South</td>
<td>North</td>
<td>South</td>
</tr>
<tr>
<td>T9</td>
<td>-0.0041</td>
<td>-0.0032</td>
<td>-0.0139</td>
<td>-0.0163</td>
</tr>
<tr>
<td>T13</td>
<td>-0.0046</td>
<td>-0.0036</td>
<td>-0.0166</td>
<td>-0.0162</td>
</tr>
<tr>
<td>T14</td>
<td>-0.0029</td>
<td>-0.0030</td>
<td>N/A</td>
<td>-0.0152</td>
</tr>
<tr>
<td>T15</td>
<td>-0.0027</td>
<td>-0.0041</td>
<td>-0.0199</td>
<td>-0.0125</td>
</tr>
<tr>
<td>T16</td>
<td>-0.0048</td>
<td>-0.0038</td>
<td>-0.0120</td>
<td>-0.0152</td>
</tr>
</tbody>
</table>
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Figure 5.13 Trans. Steel and Peak Tension Strain or Disp. Prior to Bar Buckling

Figure 5.14 Peak Spiral Strains and Prior Tensile/Comp. Strain before Bar Buckling

Figure 5.15 Trans. Steel and Comp. Strain at Cover Crushing and Spiral Yielding
5.5 Aspect Ratio Variable Experiments

The effects of aspect ratio and axial load ratio on column performance were the main variables for Tests 19-24 in Table 5.5. The 18” (457mm) diameter bridge columns contained 10 #6 (19mm) A706 bars for longitudinal reinforcement \( (A_{st}/A_g = 1.7\%) \) and a #3 (9.5mm) A706 spiral at 2” spacing \( (4A_{sp}/D's = 1.3\%) \). The shear span for the cantilever columns was either 8ft (244cm), 11ft (335cm), or 13ft (396cm) and they had a moment to shear ratio of \( (M/VD = 5.33, 7.33, \text{ or } 8.67) \). For each aspect ratio, one specimen was subjected to \( (P/(f'_cA_g) = 5\%) \) and the other was subjected to 10% axial load. A photo of the test setup for the column with the largest aspect ratio appears in Figure 5.1. The test series had a full cover concrete blockout with target markers applied to both longitudinal and transverse steel.

In design, strain-based limit state displacements are evaluated using an equivalent curvature distribution such as the Plastic Hinge Method from Priestley, Calvi, and Kowalsky (2007). The moment gradient component of the plastic hinge length is dependent on the column length. Aspect ratio also influences shear in the column, which impacts the additional spread in plasticity due to tension shift. Aspect ratio is not expected to influence bar buckling behavior, but the tests are included to evaluate its effect on the spread of plasticity. The measured spread of plasticity is discussed in greater detail in Chapters 6 and 7.

The recorded displacements and strains at key damage observations appear in Table 5.6 and Table 5.7 for the aspect ratio variable experiments. The influence of aspect ratio on the recorded peak tensile strain and it associated lateral drift prior to bar buckling upon reversal of load is shown graphically in Figure 5.16. The trends imply that the peak tensile strain measured prior to bar buckling was not influenced by column aspect ratio. The lateral drift measured prior to bar buckling in the following load reversal is however strongly influenced by column aspect ratio.
Table 5.5 Aspect Ratio Variable Experiments

<table>
<thead>
<tr>
<th>Test</th>
<th>Load History</th>
<th>D (in)</th>
<th>L/D</th>
<th>Long. Steel (ρ_l)</th>
<th>Spiral Detailing (ρ_s)</th>
<th>f’c (ksi)</th>
<th>P/f’c*Ag</th>
</tr>
</thead>
<tbody>
<tr>
<td>T20</td>
<td>3-Cycle-Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2” (1.3%)</td>
<td>6.47</td>
<td>5%</td>
</tr>
<tr>
<td>T19</td>
<td>3-Cycle-Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2” (1.3%)</td>
<td>6.33</td>
<td>10%</td>
</tr>
<tr>
<td>T21</td>
<td>3-Cycle-Set</td>
<td>18</td>
<td>7.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2” (1.3%)</td>
<td>6.39</td>
<td>5%</td>
</tr>
<tr>
<td>T22</td>
<td>3-Cycle-Set</td>
<td>18</td>
<td>7.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2” (1.3%)</td>
<td>6.53</td>
<td>10%</td>
</tr>
<tr>
<td>T23</td>
<td>3-Cycle-Set</td>
<td>18</td>
<td>8.67</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2” (1.3%)</td>
<td>6.61</td>
<td>5%</td>
</tr>
<tr>
<td>T24</td>
<td>3-Cycle-Set</td>
<td>18</td>
<td>8.67</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2” (1.3%)</td>
<td>6.47</td>
<td>10%</td>
</tr>
</tbody>
</table>

Table 5.6 Aspect Ratio Variable Experiments, Limit State Displacements

<table>
<thead>
<tr>
<th>Test</th>
<th>Δ (μ_s) at Cover Crushing</th>
<th>Δ (μ_s) at Spiral Yielding</th>
<th>Δ (μ_s) at Peak Tension Prior to Bar Buckling</th>
<th>Δ (μ_s) at Peak Comp. Prior to Bar Buckling</th>
</tr>
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<td>Side</td>
<td>North</td>
<td>South</td>
<td>North</td>
<td>South</td>
</tr>
<tr>
<td>T20</td>
<td>2.36” (2)</td>
<td>2.36” (2)</td>
<td>4.72” (4)</td>
<td>3.35” (3)</td>
</tr>
<tr>
<td>T19</td>
<td>2.29” (2)</td>
<td>2.29” (2)</td>
<td>3.42” (3)</td>
<td>3.43” (3)</td>
</tr>
<tr>
<td>T21</td>
<td>3.95” (2)</td>
<td>3.97” (2)</td>
<td>7.91” (4)</td>
<td>5.94” (3)</td>
</tr>
<tr>
<td>T22</td>
<td>4.17” (2)</td>
<td>4.17” (2)</td>
<td>6.27” (3)</td>
<td>6.26” (3)</td>
</tr>
<tr>
<td>T23</td>
<td>5.56” (2)</td>
<td>5.54” (2)</td>
<td>11.1” (4)</td>
<td>11.1” (4)</td>
</tr>
<tr>
<td>T24</td>
<td>5.72” (2)</td>
<td>5.73” (2)</td>
<td>8.58” (3)</td>
<td>8.58” (3)</td>
</tr>
</tbody>
</table>

Table 5.7 Aspect Ratio Variable Experiments, Limit State Strains

<table>
<thead>
<tr>
<th>Test</th>
<th>εs at Cover Crushing</th>
<th>εs at Spiral Yielding</th>
<th>εs at Peak Tension Prior to Bar Buckling</th>
<th>εs at Peak Comp. Prior to Bar Buckling</th>
</tr>
</thead>
<tbody>
<tr>
<td>Side</td>
<td>North</td>
<td>South</td>
<td>North</td>
<td>South</td>
</tr>
<tr>
<td>T20</td>
<td>-0.0065*</td>
<td>-0.0046*</td>
<td>-0.0114</td>
<td>-0.0109</td>
</tr>
<tr>
<td>T19</td>
<td>-0.0060*</td>
<td>-0.0065*</td>
<td>-0.0103</td>
<td>-0.0119</td>
</tr>
<tr>
<td>T21</td>
<td>-0.0046*</td>
<td>-0.0048</td>
<td>-0.0146</td>
<td>-0.0102</td>
</tr>
<tr>
<td>T22</td>
<td>-0.0063*</td>
<td>-0.0085*</td>
<td>-0.0103</td>
<td>-0.0124</td>
</tr>
<tr>
<td>T23</td>
<td>-0.0052*</td>
<td>-0.0062*</td>
<td>-0.0136</td>
<td>-0.0151</td>
</tr>
<tr>
<td>T24</td>
<td>-0.0085</td>
<td>-0.0083*</td>
<td>-0.0155</td>
<td>-0.0131</td>
</tr>
</tbody>
</table>
Chapter 5: Impact of Steel Content, Aspect Ratio, and Axial Load Ratio

5.6 Longitudinal Steel Content Variable Experiments

The effects of longitudinal steel content and higher levels of axial load on column performance were the main variables for Tests 25-30. The test matrix for the six columns and T9, which serves as basis of comparison for 24” diameter specimens, is shown in Table 5.8 and the material properties of the reinforcement appear in Table 5.1. Columns with similar 18” and 24” column configurations were used so that the results could be compared to previous experiments with either different axial load or longitudinal steel content. The shear span for the longitudinal steel content variable columns was 8ft (244cm). Tests 25-30 had the full cover concrete blockout, while Test 9 had vertical blockout strips over extreme fiber reinforcement, Figure 5.1.

The 18” (457mm) diameter bridge columns, Figure 5.2, contained either 10 #6 ($A_{st}/A_g = 1.7\%$) or 10 #8 ($A_{st}/A_g = 3.1\%$) A706 bars for longitudinal reinforcement and a #3 (9.5mm) A706 spiral at 2” spacing ($4A_{sp}/D's = 1.3\%$). Both 18” diameter columns, T28 and T30, were subjected to ($P/(f_c'A_g) = 15\%$) axial load. The 24” (610mm) diameter bridge columns contained either 16 #6 ($A_{st}/A_g = 1.6\%$) or 16 #7 ($A_{st}/A_g = 2.1\%$) A706 bars for longitudinal reinforcement and a #3 (9.5mm) A706 spiral at 2” spacing.
(4A_{sp}/D's = 1%). A pair of 24" diameter columns with each reinforcing ratio were subjected to similar levels of axial load, T9 and T25 with \((P/(f'_c A_g) \approx 5\%\)) and T26 and T27 with 10%. It is important to note that T9 came from a prior test series with different material properties. The recorded displacements and strains at key limit states for the longitudinal steel content variable tests are shown in Table 5.9 and Table 5.10. As shown in Figure 5.17 and Figure 5.18, longitudinal steel content was found to not significantly impact bar buckling behavior, but higher levels of steel content did result in confinement steel yielding at lower compressive strain levels.

### Table 5.8 Longitudinal Steel Content Variable Experiments

<table>
<thead>
<tr>
<th>Test</th>
<th>Load History</th>
<th>D (in)</th>
<th>L/D</th>
<th>Long. Steel ((\rho_l))</th>
<th>Spiral Detailing ((\rho_s))</th>
<th>(f'_c) (ksi)</th>
<th>(P/f'_c Ag)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T9</td>
<td>3-Cycle-Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>6.81</td>
<td>5.5%</td>
</tr>
<tr>
<td>T25</td>
<td>3-Cycle-Set</td>
<td>24</td>
<td>4</td>
<td>16 #7 bars (2.1%)</td>
<td>#3 at 2” (1%)</td>
<td>6.29</td>
<td>5%</td>
</tr>
<tr>
<td>T27</td>
<td>3-Cycle-Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>6.15</td>
<td>10%</td>
</tr>
<tr>
<td>T26</td>
<td>3-Cycle-Set</td>
<td>24</td>
<td>4</td>
<td>16 #7 bars (2.1%)</td>
<td>#3 at 2” (1%)</td>
<td>5.89</td>
<td>10%</td>
</tr>
<tr>
<td>T28</td>
<td>3-Cycle-Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2” (1.3%)</td>
<td>6.24</td>
<td>15%</td>
</tr>
<tr>
<td>T30</td>
<td>3-Cycle-Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #8 bars (3.1%)</td>
<td>#3 at 2” (1.3%)</td>
<td>6.05</td>
<td>15%</td>
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</table>

### Table 5.9 Longitudinal Steel Variable Experiments, Limit State Displacements

<table>
<thead>
<tr>
<th>Test</th>
<th>(\Delta (\mu_A)) at Cover Crushing</th>
<th>(\Delta (\mu_A)) at Spiral Yielding</th>
<th>(\Delta (\mu_A)) at Peak Tension Prior to Bar Buckling</th>
<th>(\Delta (\mu_A)) at Peak Comp. Prior to Bar Buckling</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>North</td>
<td>South</td>
<td>North</td>
<td>South</td>
</tr>
<tr>
<td>T9</td>
<td>1.67”</td>
<td>1.69”</td>
<td>5.05”</td>
<td>6.71”</td>
</tr>
<tr>
<td>T25</td>
<td>1.52”</td>
<td>1.53”</td>
<td>3.06”</td>
<td>3.08”</td>
</tr>
<tr>
<td>T27</td>
<td>1.38”</td>
<td>1.38”</td>
<td>2.76”</td>
<td>2.76”</td>
</tr>
<tr>
<td>T26</td>
<td>1.49”</td>
<td>1.50”</td>
<td>2.99”</td>
<td>2.97”</td>
</tr>
<tr>
<td>T28</td>
<td>2.00”</td>
<td>2.00”</td>
<td>4.00”</td>
<td>4.00”</td>
</tr>
<tr>
<td>T30</td>
<td>2.21”</td>
<td>2.21”</td>
<td>2.95”</td>
<td>2.95”</td>
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</tbody>
</table>
Chapter 5: Impact of Steel Content, Aspect Ratio, and Axial Load Ratio

Table 5.10 Longitudinal Steel Variable Experiments, Limit State Strains

<table>
<thead>
<tr>
<th>Test</th>
<th>$\varepsilon$ at Cover Crushing</th>
<th>$\varepsilon$ at Spiral Yielding</th>
<th>$\varepsilon$ at Peak Tension Prior to Bar Buckling</th>
<th>$\varepsilon$ at Peak Comp. Prior to Bar Buckling</th>
</tr>
</thead>
<tbody>
<tr>
<td>Side</td>
<td>North</td>
<td>South</td>
<td>North</td>
<td>South</td>
</tr>
<tr>
<td>T9</td>
<td>-0.0041</td>
<td>-0.0032</td>
<td>-0.0139</td>
<td>-0.0163</td>
</tr>
<tr>
<td>T25</td>
<td>-0.0036</td>
<td>-0.0040</td>
<td>-0.0091</td>
<td>-0.0125</td>
</tr>
<tr>
<td>T27</td>
<td>-0.0036</td>
<td>-0.0038</td>
<td>-0.0168</td>
<td>-0.0124</td>
</tr>
<tr>
<td>T26</td>
<td>-0.0045</td>
<td>-0.0046</td>
<td>-0.0089</td>
<td>-0.0121</td>
</tr>
<tr>
<td>T28</td>
<td>-0.0051</td>
<td>-0.0055</td>
<td>-0.0123</td>
<td>-0.0143</td>
</tr>
<tr>
<td>T30</td>
<td>-0.0052</td>
<td>-0.0059</td>
<td>-0.0095</td>
<td>-0.0094</td>
</tr>
</tbody>
</table>

Figure 5.17 Long. Steel Ratio and Peak Tension Strain or Disp. Prior to Bar Buckling

Figure 5.18 Long. Steel Ratio and Peak Comp. Strain or Drift Prior to Spiral Yielding
5.7 Axial Load Ratio Variable Experiments

Axial load influences the distribution of forces within the cross section. Columns with higher levels of axial load are expected to have a reduced deformation capacity but higher lateral forces. Axial load was maintained as a variable over multiple test series, so care should be taken when comparing test results from Table 5.11. The 18” (457mm) diameter bridge columns contained 10 #6 ($A_{st}/A_g = 1.7\%$) A706 bars for longitudinal reinforcement and a #3 (9.5mm) A706 spiral at 2” spacing ($4A_{sp}/D's = 1.3\%$). Four of these columns with the same aspect ratio, ($M/VD = 5.33$), were subjected to ($P/(f'c'A_g) = 5, 10, 15, and 20\%$) axial load. In these tests, the increase in axial load did not have a significant impact on the displacement ductility or peak tensile strain measured prior to bar buckling. However, as axial load increased, the measured compressive strains proceeding bar buckling also increased. Aspect ratio variable Tests 21-24, evaluated 18” diameter specimens subjected to 5 and 10% axial load with aspect ratios 7.33 and 8.67. For the two columns with an aspect ratio of 7.33, the increase in axial load did not lead to a reduction in the displacement measured prior to bar buckling. In the two columns with an aspect ratio of 8.67, bar buckling occurred one displacement ductility level earlier for the specimen subjected to 10% axial load.

Two 24” (610mm) diameter bridge columns containing 16 #7 ($A_{st}/A_g = 2.1\%$) A706 bars for longitudinal reinforcement and a #3 (9.5mm) A706 spiral at 2” spacing ($4A_{sp}/D's = 1\%$) were tested, one with 5% and the other with 10% axial load. The specimen subjected to the higher axial load level suffered bar buckling at lower levels of displacement ductility and with smaller previous peak tensile strains. Two 24” (610mm) diameter bridge columns containing 16 #6 ($A_{st}/A_g = 1.6\%$) A706 bars for longitudinal reinforcement and a #3 (9.5mm) A706 spiral at 2” spacing ($4A_{sp}/D's = 1\%$) were tested, one with 5.5% and the other with 10% axial load. Comparison between these tests requires consideration that they came from different test series, with different material properties for the longitudinal and transverse steel. Test 9, with 5.5% axial load, had transverse steel with a
yield stress of 74.1 ksi. Test 27, with 10% axial load, had transverse steel with a yield stress of 63.9 ksi. Test 9 had bar buckling on each side of the specimen during displacement ductility eight. In Test 27, bar buckling occurring during the first push and pull cycles of displacement ductility five, with significantly larger values of previous peak compressive strains measured in the longitudinal steel when compared to Test 9. Since axial load did not have his large of an impact when considering columns within the same test series, this difference in performance is largely attributed to the increased strength in the transverse steel utilized in T9. This is further evident when comparing the displacement ductility levels at which initial spiral yielding in the confinement region was observed in the two tests.

A graphical summary of the influence of axial load ratio on bar buckling and initial spiral yielding behavior is shown in Figure 5.19 and Figure 5.20 respectively. For both limit states, axial load does not have as impactful of a change on the behavior as expected. It is important to note that comparatively fewer columns were tested with higher levels of axial load though.

### Table 5.11 Axial Load Ratio Variable Experiments

<table>
<thead>
<tr>
<th>Test</th>
<th>Load History</th>
<th>D (in)</th>
<th>L/D</th>
<th>Long. Steel ($\rho_l$)</th>
<th>Spiral Detailing ($\rho_s$)</th>
<th>$f'_c$ (ksi)</th>
<th>$P/f'_c*Ag$</th>
</tr>
</thead>
<tbody>
<tr>
<td>T9</td>
<td>3-Cycle-Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>6.81</td>
<td>5.5%</td>
</tr>
<tr>
<td>T27</td>
<td>3-Cycle-Set</td>
<td>24</td>
<td>4</td>
<td>16 #6 bars (1.6%)</td>
<td>#3 at 2” (1%)</td>
<td>6.15</td>
<td>10%</td>
</tr>
<tr>
<td>T25</td>
<td>3-Cycle-Set</td>
<td>24</td>
<td>4</td>
<td>16 #7 bars (2.1%)</td>
<td>#3 at 2” (1%)</td>
<td>6.29</td>
<td>5%</td>
</tr>
<tr>
<td>T26</td>
<td>3-Cycle-Set</td>
<td>24</td>
<td>4</td>
<td>16 #7 bars (2.1%)</td>
<td>#3 at 2” (1%)</td>
<td>5.89</td>
<td>10%</td>
</tr>
<tr>
<td>T20</td>
<td>3-Cycle-Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2” (1.3%)</td>
<td>6.47</td>
<td>5%</td>
</tr>
<tr>
<td>T19</td>
<td>3-Cycle-Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2” (1.3%)</td>
<td>6.33</td>
<td>10%</td>
</tr>
<tr>
<td>T28</td>
<td>3-Cycle-Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2” (1.3%)</td>
<td>6.24</td>
<td>15%</td>
</tr>
<tr>
<td>T29</td>
<td>3-Cycle-Set</td>
<td>18</td>
<td>5.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2” (1.3%)</td>
<td>5.91</td>
<td>20%</td>
</tr>
<tr>
<td>T21</td>
<td>3-Cycle-Set</td>
<td>18</td>
<td>7.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2” (1.3%)</td>
<td>6.39</td>
<td>5%</td>
</tr>
<tr>
<td>T22</td>
<td>3-Cycle-Set</td>
<td>18</td>
<td>7.33</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2” (1.3%)</td>
<td>6.53</td>
<td>10%</td>
</tr>
<tr>
<td>T23</td>
<td>3-Cycle-Set</td>
<td>18</td>
<td>8.67</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2” (1.3%)</td>
<td>6.61</td>
<td>5%</td>
</tr>
<tr>
<td>T24</td>
<td>3-Cycle-Set</td>
<td>18</td>
<td>8.67</td>
<td>10 #6 bars (1.7%)</td>
<td>#3 at 2” (1.3%)</td>
<td>6.47</td>
<td>10%</td>
</tr>
</tbody>
</table>
### Table 5.12 Axial Load Variable Experiments, Limit State Displacements

<table>
<thead>
<tr>
<th>Test</th>
<th>$\Delta (\mu_s)$ at Cover Crushing</th>
<th>$\Delta (\mu_s)$ at Spiral Yielding</th>
<th>$\Delta (\mu_s)$ at Peak Tension Prior to Bar Buckling</th>
<th>$\Delta (\mu_s)$ at Peak Comp. Prior to Bar Buckling</th>
</tr>
</thead>
<tbody>
<tr>
<td>Side</td>
<td>North</td>
<td>South</td>
<td>North</td>
<td>South</td>
</tr>
<tr>
<td>T9</td>
<td>1.67&quot; (2)</td>
<td>1.69&quot; (2)</td>
<td>5.05&quot; (6)</td>
<td>6.71&quot; (8)</td>
</tr>
<tr>
<td>T27</td>
<td>1.38&quot; (1.5)</td>
<td>1.38&quot; (1.5)</td>
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<td>2.76&quot; (3)</td>
</tr>
<tr>
<td>T25</td>
<td>1.52&quot; (1.5)</td>
<td>1.53&quot; (1.5)</td>
<td>3.06&quot; (3)</td>
<td>3.08&quot; (3)</td>
</tr>
<tr>
<td>T26</td>
<td>1.49&quot; (1.5)</td>
<td>1.50&quot; (1.5)</td>
<td>2.99&quot; (3)</td>
<td>2.97&quot; (3)</td>
</tr>
<tr>
<td>T20</td>
<td>2.36&quot; (2)</td>
<td>2.36&quot; (2)</td>
<td>4.72&quot; (4)</td>
<td>3.35&quot; (3)</td>
</tr>
<tr>
<td>T19</td>
<td>2.29&quot; (2)</td>
<td>2.29&quot; (2)</td>
<td>3.42&quot; (3)</td>
<td>3.43&quot; (3)</td>
</tr>
<tr>
<td>T28</td>
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<td>2.00&quot; (1.5)</td>
<td>4.00&quot; (3)</td>
<td>4.00&quot; (3)</td>
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<tr>
<td>T29</td>
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<td>4.03&quot; (3)</td>
<td>2.69&quot; (2)</td>
</tr>
<tr>
<td>T21</td>
<td>3.95&quot; (2)</td>
<td>3.97&quot; (2)</td>
<td>7.91&quot; (4)</td>
<td>5.94&quot; (3)</td>
</tr>
<tr>
<td>T22</td>
<td>4.17&quot; (2)</td>
<td>4.17&quot; (2)</td>
<td>6.27&quot; (3)</td>
<td>6.26&quot; (3)</td>
</tr>
<tr>
<td>T23</td>
<td>5.56&quot; (2)</td>
<td>5.54&quot; (2)</td>
<td>11.1&quot; (4)</td>
<td>11.1&quot; (4)</td>
</tr>
<tr>
<td>T24</td>
<td>5.72&quot; (2)</td>
<td>5.73&quot; (2)</td>
<td>8.58&quot; (3)</td>
<td>8.58&quot; (3)</td>
</tr>
</tbody>
</table>

### Table 5.13 Axial Load Variable Experiments, Limit State Strains

<table>
<thead>
<tr>
<th>Test</th>
<th>$\varepsilon_s$ at Cover Crushing</th>
<th>$\varepsilon_s$ at Spiral Yielding</th>
<th>$\varepsilon_s$ at Peak Tension Prior to Bar Buckling</th>
<th>$\varepsilon_s$ at Peak Comp. Prior to Bar Buckling</th>
</tr>
</thead>
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<tr>
<td>Side</td>
<td>North</td>
<td>South</td>
<td>North</td>
<td>South</td>
</tr>
<tr>
<td>T9</td>
<td>-0.0041</td>
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</tr>
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<td>-0.0046</td>
<td>-0.0089</td>
<td>-0.0121</td>
</tr>
<tr>
<td>T20</td>
<td>-0.0065*</td>
<td>-0.0046*</td>
<td>-0.0114</td>
<td>-0.0109</td>
</tr>
<tr>
<td>T19</td>
<td>-0.0060*</td>
<td>-0.0065*</td>
<td>-0.0103</td>
<td>-0.0119</td>
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<tr>
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<td>-0.0055</td>
<td>-0.0123</td>
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<td>-0.0102</td>
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<td>T22</td>
<td>-0.0063*</td>
<td>-0.0085*</td>
<td>-0.0103</td>
<td>-0.0124</td>
</tr>
<tr>
<td>T23</td>
<td>-0.0052*</td>
<td>-0.0062*</td>
<td>-0.0136</td>
<td>-0.0151</td>
</tr>
<tr>
<td>T24</td>
<td>-0.0085</td>
<td>-0.0083*</td>
<td>-0.0155</td>
<td>-0.0131</td>
</tr>
</tbody>
</table>
5.8 Equivalent Viscous Damping

In direct displacement based design, the concept of equivalent viscous damping is used to reduce the elastic response spectra to a level consistent with the inelastic response at the design limit state. The equivalent viscous damping is the combination of viscous and hysteretic damping components. The hysteretic damping for a complete cycle at a given level of inelastic response is calculated using the Jacobsen’s (1960) area-based approach, Eqn 5.1. In Eqn 5.1, $A_h$ is the area enclosed by a complete cycle of force-displacement
response at given displacement amplitude, and $A_{box}$ is the area of rectangular bounding box for the same hysteretic cycle. This process is shown graphically in Figure 5.21 for the third cycle of displacement ductility one and six for column Test 9. Jacobsen’s (1960) approach is related to the secant stiffness to maximum response, which is consistent with the direct-displacement based design procedure which characterizes a structure based on secant stiffness and damping at the peak response.

Dwairi and Kowalsky (2007) studied the accuracy of the area-based hysteretic damping component using non-linear time history analysis and typical hysteretic loop shapes. This was accomplished by varying the damping value until the displacement for the equivalent substitute structure matched the response obtained from time-history analysis with a non-linear hysteretic rule. Dwairi and Kowalsky (2007) produced displacement ductility dependent correction factors for the hysteretic damping of many common loop shapes, and an expression was developed based on these results to correct the damping of other loop shapes, Eqn 5.2. The uncorrected and corrected Jacobsen’s (1960) hysteretic damping applied to Tests 8-30 appears in Figure 5.22. The 18” and 24” diameter specimens were separated since it is apparent that the results are different between the two datasets.

The equivalent viscous damping is the combination of the viscous damping and the corrected hysteretic damping. The viscous damping is connected to either the initial or tangent stiffness while the hysteretic damping is related to the secant stiffness. Analytical predictions to shake table experiments using non-linear time history analysis have shown that tangent stiffness proportional damping offers a better prediction to the peak response displacement. Grant et al. (2005) offered separate correction factors to translate either initial or tangent stiffness viscous damping to secant stiffness proportional damping. These correction factors are specific to a given hysteretic loop shape. The Thin Takeda (TT) tangent stiffness proportional viscous damping correction factor presented in Eqn 5.3 is appropriate for bridge columns since it is based on the Thin Takeda Hysteretic rule and tangent stiffness proportional viscous damping. The result of this correction factor applied to 5% viscous damping appears in Figure 5.23.
The final equivalent viscous damping ratio is the combination of the corrected viscous and corrected hysteretic damping, Eqn 5.4. These values were computed at the peak of each cycle for Tests 8-30 which utilized a symmetric three-cycle-set load history. Design expressions from Priestley, Calvi, and Kowalsky (2007) for the equivalent viscous damping with 5% elastic tangent stiffness damping appear in Eqn 5.5 and Eqn 5.6 for two typical hysteretic rules used for reinforced concrete structures. The Thin Takeda (TT) hysteretic rule is commonly used for columns with axial load while the Fat Takeda (TF) is used for beams. The computed equivalent viscous damping for Tests 8-30, lies between the two design expressions, and is apparently more linear in shape. Application of the area-based hysteretic damping approach includes additional damping at displacement ductility one, Figure 5.21.

\[
\xi_{\text{hyst}}^\text{JAC} = \frac{A_h}{2\pi A_{\text{box}}} \quad \text{Jacobsen’s (1960) Hysteretic Damping} \quad \text{Eqn 5.1}
\]

\[
EVD_{\mu\Delta}^{\text{Ratio}} = (0.53\mu_\Delta + 0.8)(\xi_{\text{hyst}}^\text{JAC})^{-\left(\frac{\mu_\Delta}{40}+0.4\right)} \quad \text{Correction Factor for Area-Based Hysteretic Damping} \quad \text{Eqn 5.2}
\]

\[
\kappa = \mu_\Delta^\lambda \quad \text{where,} \quad \lambda = -0.378 \quad \text{*Thin Takeda (TT) Tangent Stiffness, Viscous Damping Correction Factor} \quad \text{Eqn 5.3}
\]

\[
\xi_{\text{EVD}} = \kappa(\xi_{el}) + EVD_{\mu\Delta}^{\text{Ratio}}(\xi_{\text{hyst}}^\text{JAC}) \quad \text{Equivalent Viscous Damping} \quad \text{Eqn 5.4}
\]

\[
\xi_{\text{EVD}} = 0.05 + 0.444 \left(\frac{\mu_\Delta - 1}{\mu_\Delta \pi}\right) \quad \text{Equivalent Viscous Damping Expression for Thin Takeda Hysteretic Rule (Columns)} \quad \text{Eqn 5.5}
\]

\[
\xi_{\text{EVD}} = 0.05 + 0.565 \left(\frac{\mu_\Delta - 1}{\mu_\Delta \pi}\right) \quad \text{Equivalent Viscous Damping Expression for Fat Takeda Hysteretic Rule (Beams)} \quad \text{Eqn 5.6}
\]
Chapter 5: Impact of Steel Content, Aspect Ratio, and Axial Load Ratio

Figure 5.21 Jacobsen’s Area-Based Hysteretic Damping for Ductility 1 and 6 of Test 9

Figure 5.22 (Left) Uncorrected and (Right) Corrected Jacobsen Hysteretic Damping

Figure 5.23 (Left) Corrected Viscous Damping, (Right) Equivalent Viscous Damping
5.9 Conclusions

An experimental study was conducted to assess the impact of design variables on the seismic performance of circular well-confined bridge columns. A key feature of the experiments is the high fidelity strain data obtained through the use of an optical 3D position measurement system. The instrumentation system allowed for monitoring of longitudinal steel and transverse steel strains in the plastic hinge region. For many of these tests, measurable deformation could be observed in the recorded longitudinal and spiral strain hysteresis prior to the visible bar buckling observation. This deformation occurred once the transverse steel restraining the longitudinal bar went inelastic under prior compressive demands. The distribution of spiral strains measured both around the circumference of the column and over multiple layers was found to impact the observed bar buckling behavior. Localized spiral demands led to increased levels of measured compressive strain and early buckling of longitudinal reinforcement.

In this section, the impact of design variables on measured strains prior to the following limit states was explored: (1) cover concrete crushing, (3) confinement steel yielding and (3) longitudinal bar buckling. Inspection of the compressive strains measured at the peak of the cycle where cover crushing was observed suggests that the behavior is dependent on the amount of confinement steel in the bridge columns. Specimens with larger transverse volumetric steel ratios had higher measured compressive strains at cover concrete crushing. Initial yielding of confinement steel under compressive demands was found to be influenced most significantly by longitudinal steel content. Higher longitudinal reinforcing content increases both confinement and restraint demands in the transverse reinforcement. For specimens with similar levels of volumetric steel ratio, the spiral yield strength was found to significantly influence compressive behavior.

The deformation capacity of all of the cyclically loaded specimens was limited by bar buckling and subsequent fracture during later cycles of loading. Specimens with higher amounts of confinement steel had larger peak tensile strains measured in the longitudinal
reinforcement and higher drifts recorded prior to bar buckling after reversal of load. In general, specimens with higher axial load ratio had reduced peak tensile strains and drifts measured prior to bar buckling. The magnitude of the peak tensile strain measured prior to bar buckling was significantly influenced by the previous compressive demand and measured peak spiral strain in the layers restraining the bar from buckling. Column aspect ratio was not found to influence the strains measured prior to bar buckling, but it did have a significant influence on the lateral drift. Columns with higher aspect ratios had larger drifts measured prior to bar buckling.

The goal of the research program is to define accurate limit state displacements which relate to specific levels of damage in reinforced concrete bridge columns. In Chapter 8, strain limit design expressions are developed based on the information provided in this section. In Chapter 7, an equivalent curvature distribution which is consistent with strain-based displacement predictions is presented. In performance based design, an understanding of the damping-ductility relationship is needed to assess the correct value of damping at the design limit state. For the cyclically loaded experiments of this study, the equivalent viscous damping was calculated and compared to current design expressions which are based on specific hysteretic rules. In general, the equivalent viscous damping is equal to the viscous plus the hysteretic damping, both corrected and combined in a manner which is consistent with a design procedure based on secant stiffness to the design limit state.
Chapter 6: Bridge Column Response Prediction Techniques

6.1 Background and Motivation

This section discusses a research program supported by the Alaska Department of Transportation and Alaska University Transportation Center aimed at defining accurate limit state displacements which relate to specific levels of damage in reinforced concrete bridge columns subjected to seismic hazards. The experimental portion of the study aims to assess the performance of thirty large scale circular bridge columns. A key feature of the experiments is the high fidelity strain data obtained through the use of an optical 3D position measurement system. In this section, this data is utilized to explore column deformation components, the relationship between material strain and displacement, and the accuracy of two common response prediction techniques utilized in design: (1) monotonic moment-curvature analysis paired with an equivalent curvature distribution and (2) cyclic fiber analysis paired with an element representation of the beam or column.

6.1.1 Experimental Program

The goal of the experimental program is to investigate the impact of load history and other design variables on the relationship between strain and displacement, performance strain limits, and the spread of plasticity. The main variables for the thirty tests include: (1) lateral displacement history, (2) axial load, (3) longitudinal steel content, (4) aspect ratio, and (5) transverse steel detailing. The specimen was designed to represent a single degree of freedom bridge column subjected to lateral and axial load, Figure 6.2. The test specimen consists of a footing, column, and loading cap. The footing is a capacity protected member which secures the specimen to the lab strong floor using post tensioned bars. A 200kip hydraulic actuator, with a 40in stroke capacity, applies lateral load to the loading cap of the specimen. A spreader beam, two hydraulic jacks, and a load cell are placed above the
loading cap to apply a constant axial compressive load. The top column displacement was obtained through a string potentiometer placed at the center of the lateral load.

**Instrumentation**

While the progression of damage in flexural bridge columns has been thoroughly investigated in the past, to the authors’ knowledge, none of the previous studies measured strains at the level of the reinforcement throughout the entire range of response. Traditional instrumentation methods utilized linear potentiometers placed on the ends of threaded rods embedded in the core concrete to calculate changes in displacement outside of the cover concrete. A diagram of this instrumentation system from Hose et al. (1997) appeared in Hines et al. (2003), Figure 6.1. This method does not measure material strains at the locations of interests, and its measurements are influenced small rotations of the rods themselves which result due to the curvature gradient over the gage length. The experimental program discussed in this paper utilized multiple Optotrak Certus HD 3D position sensors developed by Northern Digital Inc. to monitor material strains. The position sensors track the locations of the target markers in 3D space, returning X-Y-Z spatial coordinates with an accuracy of 0.1mm with a resolution of 0.01mm.

A technique of applying target markers to longitudinal and transverse reinforcement, Figure 6.2, was utilized in the plastic hinge region. Strains were computed by dividing the change in three dimensional distance between two adjacent target markers by the original unloaded gage length. An illustration of the accuracy of the Optotrak system compared to traditional measurement techniques appears in Figure 6.2. The tensile test on a reinforcing bar contained the following instrumentation: (1) 2” Optotrak gage length, (2) 2” MTS Extensometer, and (3) centrally located electrical resistance strain gage.
Loading Protocol

The specimens were subjected to various unidirectional top-column displacement histories including standardized laboratory reversed cyclic loading and recreations of the displacement responses obtained from non-linear time history analysis of multiple earthquakes with distinct characteristics. The experiments utilized a quasi-static
displacement controlled loading procedure. The symmetric three-cycle-set load history is commonly used to evaluate the seismic performance of structural components. The load history begins with elastic cycles to the following increments of the analytically predicted first yield force: \( \frac{1}{4} F'_{y}, \frac{1}{2} F'_{y}, \frac{3}{4} F'_{y}, \) and \( F'_{y} \). The experimental first yield displacement is then determined by taking the average of the recorded displacements during the first yield push and pulls cycles. The equivalent yield displacement, used to determine the displacement ductility levels \( (\mu \Delta_{1} = 1 \times \Delta_{y}) \), is then calculated as \( \Delta_{y} = \Delta'_{y}(M_{n}/M'_{y}) \). The symmetric three-cycle-set load history resumes with three balanced cycles at each of the following displacement ductility levels: 1, 1.5, 2, 3, 4, 5, etc.

### 6.2 Measured Deformation Components

An understanding of the components of deformation and the spread of plasticity in reinforced concrete bridge columns is necessary to determine the relationship between material strain limits and lateral displacements, which are required for design. In the following section, the non-linear behavior of RC bridge columns is explored through presentation of sample results for Test #9. The 24” diameter bridge column contained 16 #6 A706 bars for longitudinal reinforcement \( (A_{st}/A_{g} = 1.6\%) \) and a #3 A706 spiral at 2” pitch \( (4A_{sp}/(D's) = 1\%) \). The column was subjected to symmetric three-cycle-set load history, Figure 6.3, and a constant compressive axial load of 170 kips \( (P/(\bar{f}_{c}A_{g}) = 5.5\%) \). The cantilever specimen had an aspect ratio \( (L/D = 4) \). The following sequence of damage was observed in all of the cyclically loaded columns: (1) concrete cracking, (2) cover concrete crushing, (3) spiral yielding in confinement regions, (4) longitudinal bar buckling, and (5) fracture of previously buckled longitudinal reinforcement. The lateral force versus deformation response for Test 9 appears in Figure 6.3. Longitudinal bar buckling occurred on each side of the specimen during cycles at displacement ductility eight, while bar fracture occurred during the first push cycle to displacement ductility ten. Reinforcement strains obtained by Optotak target markers are no longer reliable once the buckled deformation develops over multiple spiral layers, Figure 6.4.
Figure 6.3 (Left) Symmetric Three-Cycle-Set Top Column Displacement History from Test #9, (Right) Lateral Force vs. Top Column Displacement Response

Extreme fiber vertical strain profiles, Figure 6.4, depict strains measured in North and South reinforcing bars in the plastic hinge region, which forms just above the footing-column interface in the region of maximum moment. Compressive strains are concentrated near the footing, while tension strains are fanned out to a greater height following the inclined flexural shear crack distribution. This phenomenon is known as tension shift, and it leads to tension strains above the base section which are larger than those which would develop based on the plane sections hypothesis and the moment at that height alone, Hines et al. (2004). It is important to note that the crack distribution in Figure 6.4 is from a separate experiment (Test #16), but this photo is selected for its clarity. The tension shift effect leads to a fanned compression strut pattern which emanates from the compressive toe region of the column. In this region, yielding of the transverse steel can lead to a localization of compressive demand. This occurred over the second gage length above the footing for the North extreme fiber bar in Figure 6.4. The relationship between compressive strain and displacement for this gage length and the measured strain in six spiral layers overlaying the North bar are shown in Figure 6.5. Spiral yielding occurred during the first pull cycle of displacement ductility six, and each successive cycle at ductility six produced larger compressive strains in the North bar and larger tensile strains in the second spiral layer above the footing.
Cross section strain profiles for the first horizontal cross section above the footing appear in Figure 6.6. The cross section curvature is calculated by the slope of the least squared line connecting strains measured in six reinforcing bars in extreme fiber regions of the column. If the curvatures for many horizontal cross sections are analyzed, curvature profiles for the plastic hinge region can be constructed, Figure 6.7. Measured curvatures during displacement ductility one closely match the elastic curvature profile, which linearly decreases from yield curvature at the footing-column interface to zero at the center of the
applied lateral load. Plastic curvatures were found to follow a linear distribution, which agrees with observations made by Hines et. al. (2004). Procedures developed by Hines et. al. (2004) were followed to extract important information about the curvature profiles. Linear least squared error lines were fit to the plastic portion of the curvature profiles to highlight their linearity. The base curvature is calculated as the intersection of the linear plastic curvature profile with the footing-column interface. The initial plastic curvature profiles are heavily influenced by individual crack locations, so the linear representation does not fit as well. As the base curvature increases, the height at which the linear plastic curvature profile intersects the elastic curvature profile also increases. This spread of plasticity can be attributed to two sources: (1) increases in moment which influence the moment gradient and (2) the effects of tension shift which spread tension strains higher above the footing.

Curvature profiles in Figure 6.7 describe the elastic and plastic flexural displacements of the column, but do not address fixed-end rotations which result from strain penetration of longitudinal reinforcement into the footing. Development of fully anchored column longitudinal bars into the footing leads to bond slip along the partially anchored region of the bars near the footing-column interface, as described by Zhao and Sritharan (2007). The vertical displacement of target markers placed closest to the footing-column interface can be used to monitor the pull out and push in of the reinforcing bar over the partially bonded region. If the measured bond slips of six reinforcing bars are plotted along the cross section, the fixed-end rotation attributable to strain penetration may be calculated as the slope of a least squared error line, Figure 6.6. The strain penetration displacement is obtained by multiplying this rotation by the cantilever height of the column. If an elastic curvature profile assumption is made for curvatures higher than those measured in Figure 6.7, then the entire curvature profile may be integrated to obtain the total column flexural displacement. This column flexural displacement was added to the strain penetration displacement, and compared to the experimentally measured displacements in the right half of Figure 6.3. The Optotrak integrated displacement matches well with those obtained from a string potentiometer placed at the center of the lateral load, which indicates that the shear deformation component is small.
6.3 Response Prediction Methods

In order to successfully design for a particular performance strain limit, methods of relating strain to lateral displacement must accurately describe the components of deformation and the spread of plasticity in reinforced concrete bridge columns. There are two main techniques which are currently utilized in design to accomplish this task: (1) monotonic moment-curvature analysis paired with an equivalent curvature distribution and
(2) cyclic fiber analysis paired with an element representation of the beam or column. In the following section, the predictive capabilities of these two techniques are examined.

6.3.1 Sectional Response Prediction

The first, and most basic, functionality of these two methods is relating strains to curvatures and moments through monotonic moment-curvature analysis or cyclic section analysis with fiber discretization. A script developed by Montejo and Kowalsky (2007) called Cumbia was selected to perform monotonic M-ϕ. In the program, the cover concrete is assumed to follow the Mander (1988) unconfined concrete model, while the core concrete follows the Mander (1988) confined concrete model. The fiber-based cyclic M-ϕ analysis was conducted in OpenSees – Open System for Earthquake Engineering Simulation, Version 2.4.2. The confined and unconfined concrete fiber parameters for the Concrete02 model were selected to emulate the associated Mander (1988) confined an unconfined stress-strain curves. The cyclic ReinforcingSteel model was selected for the longitudinal steel fibers. In both analysis techniques, the Mander model input was based on the geometry, transverse steel detailing, tested spiral yield strength, and tested concrete compressive cylinder strength. Uniaxial tensile tests on reinforcing bars were used to calibrate the monotonic and cyclic material models utilized in the respective techniques.

The cross section curvature history for the first horizontal section above the footing, calculated in the same manner as Figure 6.6, is plotted against the base moment in right half of Figure 6.7. The backbone curve of the measured cyclic M-ϕ response (black) is reasonably predicted by the monotonic section analysis in Cumbia (blue). Similarly, the measured cyclic M-ϕ response is adequately predicted by the cyclic fiber-based section analysis in OpenSees (red). Both analysis methods perform well in predicting the strain history for the South bar, Figure 6.8.
6.3.2 Member Response Prediction

Now that each analysis technique has shown adequate performance in predicting sectional response, the next level of analysis involves predicting member response. The monotonic $M-\phi$ analysis in Cumbia is translated into member response using the plastic hinge method presented in Priestley, Calvi, and Kowalsky (2007). In this method, Figure 6.9, the elastic and plastic curvature distributions are separated into equivalent simplified shapes to facilitate design. The elastic flexural displacement is determined using a triangular yield curvature distribution. The plastic flexural displacement is obtained using a uniform curvature distribution with a constant height termed the plastic hinge length. The width of the rectangle is equal to the plastic curvature at the base section. To account for the effects of strain penetration, the curvature distribution extends into the footing by a depth termed the strain penetration length.
The cyclic displacement history from the experiment was recreated using a combination of two elements in OpenSees: (1) a beam with hinges element to model the column flexural deformations and (2) a zero length strain penetration element to model the fixed end rotations due to strain penetration. The beam with hinges element, developed by Scott and Fenves (2006), is a force-based beam column element with a plastic hinge integration method based on modified Gauss–Radau quadrature. The zero length strain penetration element, developed by Zhao and Sritharan (2007), models the fixed-end rotations attributable to strain penetration of longitudinal reinforcement into the footing. The reinforcement fibers in the zero length element are replaced with Bond SP01 bar stress-slip uniaxial material which accounts for the bond slip of reinforcement at the footing-column interface. The recorded strains in the South reinforcing bar and measured bond slip at the footing-column interface were used to calibrate the Bond SP01 stress-slip model. The measured strain history, left of Figure 6.11, was converted into a stress-strain history using the calibrated ReinforcingSteel

\[ \Delta = \Delta'_y \frac{M}{M'_y} + \Delta_p + \Delta_{shear} \]

\[ L_{sp} = 0.022f_y \delta_{bl} \text{ (MPa)} \]

\[ L_{eff} = L_c + L_{sp} \]

\[ \Delta'_y = \frac{\phi'_y L_{eff}^2}{3} \]

\[ L_p = kL_c + L_{sp} \geq 2L_{sp} \]

\[ k = 0.2 \left( \frac{f_u}{f_y} - 1 \right) \leq 0.08 \]

\[ \phi_p = \phi - \phi'_y \frac{M}{M'_y} \]

\[ \Delta_p = \phi_p L_p (L_c + L_{sp} - 0.5L_p) \]

Figure 6.9 Plastic Hinge Method from Priestley, Calvi, and Kowalsky (2007)
material model in OpenSees. The model derived stresses are paired with the measured strain penetration bond slips in Figure 6.10. This stress-slip history for the South reinforcing bar was used to calibrate parameters in the Bond SP01 uniaxial material model in OpenSees.

The measured lateral force versus top column displacement response from Test 9 appears in the right half of Figure 6.8 with response predictions using the modeling techniques previously described. The backbone curve of the hysteretic response is reasonably predicted by Cumbia, which utilized the plastic hinge method to translate curvatures to displacements. The measured compressive strains in Figure 6.11 match well with the Cumbia prediction, but the tensile strains are over predicted at higher displacement ductility levels. A plastic hinge length of 1.2*Lp, where Lp was calculated using the plastic hinge method presented in Priestley, Calvi, and Kowalsky (2007), was selected in the beam with hinges element in OpenSees to match the recorded strain data presented in Figure 6.11. A plastic hinge length equal to 1.0*Lp led to larger strains than those recorded, therefore a blind prediction would not have offered such good agreement. Furthermore the peak compressive gage length, left half of Figure 6.5, had measured compressive strains which exceed both predictive methods in regions with inelastic confinement steel.

![Figure 6.10](image)

Figure 6.10 (Left) Measured Strain Penetration Bond Slip Hysteresis at Footing-Column Interface, (Right) Method of Calibrating the Stress-Slip Model
6.3.3 Motivation for a New Equivalent Curvature Distribution

The analysis techniques produced accurate sectional response predictions, but they both over predicted the relationship between tension strain and displacement with the plastic hinge length recommended in Priestley, Calvi, and Kowalsky (2007). It is clear that improvements can be made to each analysis technique if either the equivalent curvature distribution or the integration scheme for the respective methods better reflect the measured spread in plasticity. Plastic curvatures were found to follow a linear distribution which intersects the elastic curvature profile at a height termed the extent of plasticity, Figure 6.7. This measured extent of plasticity is plotted versus base section curvature ductility in Figure 6.12 for Tests 8-30 in the research program. Furthermore, the additional column deformation attributable to strain penetration of reinforcement into the footing is described by measured fixed-end rotations in Figure 6.13. A new equivalent curvature distribution is formulated in the next chapter to improve material strain-displacement predictions.
Figure 6.12 Measured Spread of Plasticity in Circular Bridge Column Tests 8-30

Figure 6.13 Measured Fixed-End Rotation Attributable to Strain Penetration
Chapter 7: Modified Plastic Hinge Method

7.1 Goals for the Modified Plastic Hinge Method

Accurate predictions of column deformation at key performance limit states are necessary to design bridge structures for specific levels of performance under defined levels of seismic hazard. In design, limit state curvatures are converted to target displacements using an equivalent curvature distribution. An experimental study was carried out to assess the performance of thirty circular, well-confined, bridge columns with varying lateral displacement history, transverse reinforcement detailing, axial load, aspect ratio, and longitudinal steel content. A key feature of the experiments is the high fidelity strain data obtained through the use of an optical 3D position measurement system. The process through which this instrumentation system was used to quantify components of column deformation was explained in Chapter 5. Specifically, column curvature distributions and fixed-end rotations attributable to strain penetration of reinforcement into the footing were quantified. In Chapter 5, the measured tensile and compressive strain-displacement relationships were compared to the current plastic hinge method recommended in Priestley, Calvi, and Kowalsky (2007), Figure 7.1. This method was found to over predict the tensile strain-displacement relationship and under-predict the compressive strains.

In the following section, the measured curvature and strain penetration data is used to formulate a new equivalent curvature distribution to improve the accuracy of strain-displacement predictions. There are several key aspects of the proposed Modified Plastic Hinge Model, Figure 7.2, which differentiate it from the current method: (1) strain penetration and column flexure are decoupled, (2) plastic curvatures are assumed to follow a linear distribution, and (3) separate plastic hinge lengths are recommended for tensile and compressive strain-displacement predictions.
Chapter 7: Modified Plastic Hinge Method

\[ \Delta = \Delta'_y \frac{M}{M'_y} + \Delta_p + \Delta_{shear} \]

\[ L_{sp} = 0.022f_yd_{bl} \text{ (MPa)} \]

\[ L_{eff} = L_c + L_{sp} \]

\[ \Delta'_y = \frac{\phi' y L_{eff}^2}{3} \]

\[ L_p = kL_c + L_{sp} \geq 2L_{sp} \]

\[ k = 0.2 \left( \frac{f_u}{f_y} - 1 \right) \leq 0.08 \]

\[ \phi_p = \phi - \phi'_y \frac{M}{M'_y} \]

\[ \Delta_p = \phi_p L_p (L_c + L_{sp} - 0.5L_p) \]

Figure 7.1 Plastic Hinge Method from Priestley, Calvi, and Kowalsky (2007)

\[ \Delta = \Delta'_y \frac{M}{M'_y} + \Delta_p + \Delta_{sp} + \Delta_{shear} \]

\[ L_{sp} = \text{new expression} \]

\[ L_{pr} = \text{new expression} \]

\[ \Delta'_y = \frac{\phi'_y L_c^2}{3} \]

\[ \phi_p = \phi - \phi'_y \frac{M}{M'_y} \]

\[ \Delta_p = \left( \frac{1}{2} \phi_p L_{pr} \right) \left( L_c - \frac{1}{3} L_{pr} \right) \]

\[ \Delta_{sp} = \phi L_{sp} L_c \]

Figure 7.2 Goal for the Modified Plastic Hinge Method
7.2 Deformation due to Strain Penetration of Reinforcement into Adjoining Members

Development of fully anchored column longitudinal bars into the footing leads to bond slips along the partially anchored region of the bars near the footing-column interface, as described by Zhao and Sritharan (2007). They additionally note that this bond slip is not a pull-out of the entire bar embedment length resulting from poor bond between the concrete and reinforcing bar. The measured strain penetration bond slip hysteresis for an extreme fiber bar in Test 9 appears in Figure 7.3. Measured bond slips in multiple bars were used to quantify the fixed-end rotation due to strain penetration of reinforcement into the footing, Figure 7.3. In the proposed Modified Plastic Hinge Method, the displacement due to strain penetration is separated from column flexural displacements. An equivalent curvature block is placed at the footing-column or column-cap interface which describes the rotation due to strain penetration or reinforcement into the adjoining member, Figure 7.2. The area of this strain penetration block, $\theta_{sp} = L_{sp}\phi_{base}$ in Eqn 7.1, represents the fixed-end rotation. The top column displacement due to strain penetration is obtained by multiplying the fixed-end rotation by the column length, Eqn 7.2. Equivalent strain penetration lengths were calculated for each column test, Figure 7.6, by dividing the measured fixed-end rotation, Figure 7.5, by the base-section curvature, Eqn 7.3. These equivalent strain penetration lengths were found to remain constant over the range of ductility experienced in individual tests.

\[
\theta_{sp} = L_{sp}\phi_{base} \quad \text{Fixed-End Rotation due to Strain Penetration} \quad \text{Eqn 7.1}
\]

\[
\Delta_{sp} = \theta_{sp}L = L_{sp}\phi_{base}L \quad \text{Top Column Deformation due to Strain Penetration} \quad \text{Eqn 7.2}
\]

\[
L_{sp}^{meas} = \frac{\theta_{sp}^{meas}}{\phi_{base}^{meas}} \quad \text{Eqn 7.3}
\]
The proposed form of the equivalent strain penetration length ($L_{sp}$) equation takes the form Eqn 7.4 with parameters X which account for the effect of individual variables. Parameters and coefficients were selected by minimizing the sum of the squared error between the equation result and the average measured equivalent strain penetration length from each test, since the measured values were found not to vary as a function of ductility.

$$L_{sp} = X \frac{f_{ye} d_{bl}}{\sqrt{f'_{cef}}} \text{ Eqn 7.4}$$

After experimentation with a constant for X, it was found the accuracy of the equation could be improved by including the following parameters Eqn 7.5. The variable U changes depending on the units of stress input into the equation, with the final result taking the same units used for the longitudinal bar diameter.
\[
L_{sp} = U \left( 1 - \frac{P}{f_{ce} A_g} - \frac{L_c}{16D} \right) \frac{f_{ye} d_{bl}}{\sqrt{f_{ce}' f}}
\]

Eqn 7.5

\[U = 0.4 \text{ for stress input as ksi, } U = 0.152 \text{ for stress input as MPa}\]

\[\frac{P}{f_{ce} A_g} \] column axial load ratio expressed as a decimal rather than a percent

\[\frac{L_c}{D} \] cantilever aspect ratio, equivalent to \( \frac{M}{VD} \)

\(f_{ye} \text{ and } d_{bl} \) are the yield stress and bar diameter of the longitudinal reinforcement

\(f_{ce}' \) expected concrete strength of the adjoining member

An overview of the accuracy of the proposed \(L_{sp}\) equation is shown in Figure 7.7. The sensitivity of the proposed equation to individual test variables appears in Figure 7.8. Variables without a significant trend indicate that their influence on strain penetration behavior is appropriately described by Eqn 7.5. Alternatively, the accuracy of the proposed equation can be compared to individual observations at various levels of ductility, rather than the average value from each test. The numerical results of this comparison appear below for both the equivalent strain penetration length, and the resulting column displacement attributable to strain penetration of reinforcement into the adjoining member. This comparison is shown graphically in Figure 7.9 as a function of base section curvature ductility.

A cumulative probability distribution for the ratio of measured to predicted strain penetration displacement for each data observation appears in Figure 3.519. Each observation of measured to predicted strain penetration displacement is given a probability of 1/n, where n is the total number of observations. These individual observations are sorted in ascending order. The first observation, with the lowest ratio of measured/equation value has a probability of 1/n, while the second has a probability of 2/n, until the final observation has
a probability of 1. A near vertical line at a measured/predicted ratio of one would denote an accurate prediction with low variability.

$$\text{mean} \left( \frac{L_{sp}^{meas}}{L_{sp}^{eqn}} \right) = 0.99, COV = 0.152 \quad \text{Statistics for Equivalent Strain Penetration Length}$$

$$\text{mean} \left( \frac{\Delta_{sp}^{meas} = \theta_{sp}^{meas} L_c}{\Delta_{sp}^{eqn} = L_{sp}^{eqn} \phi_{base} L_c} \right) = 0.99, COV = 0.146 \quad \text{Statistics for Deformation Attributable to Strain Penetration}$$

- Test 8-18 [L/D=4] (ALR=5%) [24" Dia] (1.6% Long. Steel)
- Test 19 [L/D=5.33] (ALR=10%) [18" Dia] (1.7% Long. Steel)
- Test 20 [L/D=5.33] (ALR=5%) [18" Dia] (1.7% Long. Steel)
- Test 21 [L/D=7.33] (ALR=5%) [18" Dia] (1.7% Long. Steel)
- Test 22 [L/D=7.33] (ALR=10%) [18" Dia] (1.7% Long. Steel)
- Test 23 [L/D=8.67] (ALR=5%) [18" Dia] (1.7% Long. Steel)
- Test 24 [L/D=8.67] (ALR=10%) [18" Dia] (1.7% Long. Steel)
- Test 25 [L/D=4] (ALR=5%) [24" Dia] (2.1% Long. Steel)
- Test 26 [L/D=4] (ALR=10%) [24" Dia] (2.1% Long. Steel)
- Test 27 [L/D=4] (ALR=10%) [24" Dia] (1.6% Long. Steel)
- Test 28 [L/D=5.33] (ALR=15%) [18" Dia] (1.7% Long. Steel)
- Test 29 [L/D=5.33] (ALR=20%) [18" Dia] (1.7% Long. Steel)
- Test 30 [L/D=5.33] (ALR=15%) [18" Dia] (3.1% Long. Steel)

Figure 7.4 Additional Information about Each Experiment
Figure 7.5 Strain Penetration Rotation Measured in Experiments

Figure 7.6 Equivalent Strain Penetration Lengths for Individual Experiments
Figure 7.7 Average Measured Strain Penetration Lengths and Result of Eqn 7.5
Figure 7.8 Sensitivity of Strain Penetration Length Equation to Individual Variables
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Figure 7.9 Strain Penetration Length from Each Observation Compared to Eqn 7.5

Figure 7.10 CDF for Strain Penetration Disp. / Value with Eqn 7.5 and Eqn 7.2
7.3 Tensile and Compressive Plastic Hinge Lengths

Plastic curvatures were found to follow a linear distribution as shown in the push cycle curvature profiles for Test 13, Figure 7.11. This section aims to answer the following question, would a plastic hinge length expression based on the measured spread of plasticity offer a better prediction for the relationship between strain and displacement? Measured curvatures at displacement ductility one closely match the assumed elastic curvature profile which has a value of \( \phi' \left( \frac{M_n}{M_n'} \right) = \phi_y \) at the column base and zero at the center of the applied lateral load. As curvature ductility increases, the height at which the linear plastic curvature distribution intersects the elastic curvature distribution also increases. This spread in plasticity is due to the effects of moment gradient and tension shift.

Vertical strain profiles for Test 13, Figure 7.12, depict strains measured in the extreme fiber reinforcing bars during push cycles. Compressive strains are concentrated near the footing, while tension strains are fanned out to a greater height following the inclined flexural shear crack distribution. This is due in large part to the effects of tension shift, which leads to tension strains above the base section which exceed those that would develop based on the plane sections hypothesis and the moment at that height alone. Tension shift leads to a fanned compression strut pattern which emanates from the compressive toe region of the column, where the local compressive demand is increased beyond that which would be predicted based on the plane sections hypothesis. Since tension strain are spread further above the base section, the magnitude of the peak tensile strain near the footing may be reduced. Observations of peak tensile and peak compressive strain-displacement relationships support this theory, as will be discussed in later sections of this report.
Figure 7.11 Curvature Profiles with Linear Plastic Curvature Regressions

Figure 7.12 (Left) Extreme Fiber Bar Vertical Strain Profiles and (Right) Crack Profile
The influence of moment gradient on the spread of plasticity can be evaluated by superimposing the moment-curvature relation for the cross-section upon the moment profile of the column at specific levels of response, Figure 7.13 for Test 13. The deformation at the center of the applied lateral load can be evaluated using a layered integration technique and the Moment-Area method, Eqn 7.6. The plastic displacement from curvature integration, Eqn 7.9, is obtained by subtracting the elastic displacement from the total integrated displacement. The elastic post yield displacement takes the form of Eqn 7.7 for a column in single bending, and Eqn 7.8 for a column in double bending.

\[ \Delta_{T_i} = \left[ \int_{x=0}^{x=L} \phi(x) \, dx \right] \bar{x} \]

First moment of curvature diagram, where \( \bar{x} \) is its centroid. Evaluated using layered approach and the Moment-Area method.  

\[ \Delta_e = \phi_y \left( \frac{M}{M_y'} \right) \frac{L^2}{3} \]  
(Single Bending) Elastic Displacement after Frist Yield  

Eqn 7.7

\[ \Delta_e = \phi_y' \left( \frac{M}{M_y'} \right) \frac{L^2}{6} \]  
(Double Bending) Elastic Displacement after Frist Yield  

Eqn 7.8

\[ \Delta_{pl} = \Delta_T - \Delta_e \]

Plastic Displacement from Curvature Integration  

Eqn 7.9
Equivalent curvature distributions are utilized in design to translate a known moment-curvature relation into a member force-deformation response. The current iteration of the plastic hinge method from Priestley, Calvi, and Kowalsky (2007) appears in Figure 7.1. In this approach, abbreviated as PCK (2007) Lp, an equivalent rectangular distribution of constant curvature is used to compute the plastic flexural displacement of the column. A portion of this rectangular hinge length \( (L_p = kL + L_{sp}) \) is attributed to moment gradient \( (kL) \), while the \( (L_{sp}) \) component describes the influence of strain penetration. Since a separate strain penetration model is recommended in this study, consider only on the moment gradient component \( (kL) \) in the following discussion. The value of \( k \) can be solved for by setting \( \Delta_{pl} \) from Eqn 7.9 equal to the \( \Delta_p \) evaluated using an assumed rectangular plastic curvature distribution Eqn 7.11. Alternatively, a parameter \( k^* \) can be solved for by setting \( \Delta_{pl} \) from Eqn 7.9 equal to the \( \Delta_p \) from a triangular plastic curvature distribution Eqn 7.13. For this discussion, the height of a rectangular plastic curvature distribution is termed \( L_p \), while the height of a triangular plastic curvature distribution is termed \( L_{pr} \), Figure 7.14.
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Notation: \( L_c = L/2 \) for Double Bending and \( L_c = L \) for Single Bending.

\[
\phi_p = \phi_{base} - \phi'_y \left( M / M'_y \right) \quad \text{Plastic Curvature at the Base Section} \quad \text{Eqn 7.10}
\]

\[
\Delta_p = \phi_p (kL)[L - kL/2] \quad \text{(Single Bending) Plastic Displacement from a Rectangular Hinge Length (Lp)} \quad \text{Eqn 7.11}
\]

\[
\Delta_p = \phi_p (kL_c)[L - kL_c] \quad \text{(Double Bending) Plastic Displacement from a Rectangular Hinge Length (Lp)} \quad \text{Eqn 7.12}
\]

\[
\Delta_p = \phi_p (k^*L/2)[L - k^*L/3] \quad \text{(Single Bending) Plastic Displacement from a Triangular Hinge Length (Lpr)} \quad \text{Eqn 7.13}
\]

\[
\Delta_p = \phi_p (k^*L_c/2)[L - 2k^*L_c/3] \quad \text{(Double Bending) Plastic Displacement from a Triangular Hinge Length (Lpr)} \quad \text{Eqn 7.14}
\]

An equivalent triangular plastic curvature distribution is proposed since it reflects the shape of experimentally measured curvature profiles, Figure 7.11. Linear least squared error lines were fit to the plastic portion of the measured curvature profiles to quantify their shape following recommendations proposed by Hines, Restrepo, and Seible (2004). The base-section curvature is computed as the intersection of the linear plastic curvature distribution and the footing-column interface. The extent of plasticity is evaluated as the intersection of the linear plastic curvature distribution and the elastic curvature profile. This extent of plasticity is plotted as a function of base-section curvature ductility in Figure 7.19 for individual experiments.
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Figure 7.14 (Single Bending) Parameters $k$ and $k^*$ Describe the Moment Gradient Component of the Plastic Hinge Length

$$\Delta_p = \phi_p (kL)[L - kL/2]$$

Figure 7.15 Parameter $k$ and $k^*$ Solution for Test 13 Using Rectangular and Triangular Plastic Curvature Distributions
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The result of the solution process for $k$ and $k^*$ appears in Figure 7.15 as a function of curvature ductility. The equation for $k$ presented in Priestley, Calvi, and Kowalsky (2007), in Figure 7.1, compares well with the maximum value of $k$ computed for Test 13. The shape of the $k^*$ distribution in Figure 7.15 is similar to that of $k$, except the values are scaled by a factor of two. To further evaluate the relationship between $k$ and $k^*$, Eqn 7.11 and Eqn 7.13 were set equal to each other and $k^*$ was evaluated over a range of $k$, Figure 7.17. This process was performed for single bending, and separately for double bending using Eqn 7.12 and Eqn 7.14 described graphically in Figure 7.16. For both single and double bending, the analysis implies that $k^* \cong 2k$ with sufficient accuracy.

In Figure 7.18, the height of the equivalent triangular plastic curvature profile, $L_{pr} = k^* L$, is compared with measured extent of plasticity obtained from the intersection of the measured linear plastic curvature profile and the elastic curvature distribution at various levels of curvature ductility, Figure 7.11. The extent of plasticity obtained from integration of the analytical curvature distribution only accounts for the influence of moment gradient, which explains why it under predicts the measured values.
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Figure 7.16 (Double Bending) Parameters $k$ and $k^*$ for Moment Gradient Component

Figure 7.17 Relationship between $k$ and $k^*$ for Plastic Hinge Distributions
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Figure 7.18 Comparison of Numerically Integrated Lpr with Measured Lpr for Test 13

Figure 7.19 Extent of Plasticity Normalized to Column Length for Each Experiment
The difference between the measured and moment-curvature integrated extent of plasticity in Figure 7.18 is largely attributed to the influence of tension shift. The additional spread of plasticity due to tension shift is related to the distance between the tensile and compressive force resultants, \(jd\), and the angle of the inclined flexural shear cracks. The logarithmic best fit to the measured extent of plasticity for Test 9, Figure 7.22, can be used to evaluate the accuracy of strain-displacement predictions with an equivalent curvature distribution which reflects measured spread in plasticity. The plastic hinge method presented in Priestley, Calvi, and Kowalsky (2007) is coded into the monotonic moment-curvature analysis script CUMBIA. The response predicted by the logarithmic best fit to the measured spread in plasticity for Test 9, Figure 7.22, is computed using Eqn 7.15 through Eqn 7.26 for the column in single bending. Equations for the plastic displacement of a column in double bending using either a rectangular or triangular plastic curvature distribution are derived in Figure 7.20 and Figure 7.21 respectively. For the purpose of the strain-displacement comparison using the logarithmic best fit to the measured spread of plasticity in Test 9, Figure 7.22, the same shear displacement model built into CUMBIA was utilized for both methods. The logarithmic fit to the measured spread of plasticity provided a more accurate tensile strain-displacement prediction, but decreased the accuracy of the compressive strain-displacement prediction.

\[
\Delta_e = \phi_{base} L^2 / 3 \quad \text{(Single) Elastic Flexural Displacement before First Yield} \quad \text{Eqn 7.15}
\]

\[
\Delta_e = \phi_{base} L^2 / 6 \quad \text{(Double) Elastic Flexural Disp. before First Yield} \quad \text{Eqn 7.16}
\]

\[
\Delta_e = \phi'_{y} \left( M / M_{y} \right) L^2 / 3 \quad \text{(Single) Elastic Flexural Disp. after First Yield} \quad \text{Eqn 7.17}
\]

\[
\Delta_e = \phi'_{y} \left( M / M_{y} \right) L^2 / 6 \quad \text{(Double) Elastic Flexural Disp. after First Yield} \quad \text{Eqn 7.18}
\]
\[ \phi_p = \phi_{base} - \phi'_y \left( \frac{M}{M'_y} \right) \] Plastic Curvature at the Base Section  

Eqn 7.19

\[ \Delta_p = \phi_p \left( L_{pr}/2 \right) \left[ L - L_{pr}/3 \right] \] (Single) Plastic Disp. for Triangular Plastic Curvature Distribution  

Eqn 7.20

\[ \Delta_p = \phi_p \left( L_{pr}/2 \right) \left[ L - 2 L_{pr}/3 \right] \] (Double) Plastic Disp. for Triangular Plastic Curvature Distribution  

Eqn 7.21

\[ \Delta_p = \phi_p L_p \left[ L - L_p/2 \right] \] (Single) Plastic Disp. for Rectangular Plastic Curvature Distribution  

Eqn 7.22

\[ \Delta_p = \phi_p L_p \left[ L - L_p \right] \] (Double) Plastic Disp. for Rectangular Plastic Curvature Distribution  

Eqn 7.23

\[ L_{sp} = U \left( 1 - \frac{P}{f'_{ce} A_g} - \frac{L_c}{16D} \right) \frac{f_{ye} d_{bi}}{\sqrt{f'_{ce} f_{ce}}} \]  

\[ U = 0.4 \text{ for ksi and 0.152 for MPa units} \]  

Eqn 7.24

\[ \Delta_{sp} = L_{sp} \phi_{base} L \] Displacement due to Strain Penetration  

Eqn 7.25

\[ \Delta_T = (\Delta_e + \Delta_{sp} + \Delta_p + \Delta_{shear}) \] Total Top Column Displacement  

Eqn 7.26

Separate plastic hinge lengths for tension and compressive strain-displacement predictions are needed to evaluate accurate strain limit based target displacements. Tension strains are influenced by the total spread in plasticity, while compressive strains are more closely related to only the moment gradient component, Figure 7.23. The proposed tension hinge length, \( L_{pr_t} \) Eqn 7.28, was calibrated to match the upper bound of the measured spread of plasticity in each test. The proposed compressive hinge length, \( L_{pr_c} \) Eqn 7.29, only
contains a term related to the moment gradient effect. Both expressions utilize the observation that $k^* \approx 2k$ from Figure 7.17. This geometric relationship between triangular and rectangular plastic curvature distributions can be used to translate the proposed triangular-based $L_{pr}$ expressions to rectangular-based $L_p$ equations (Eqn 7.30 and Eqn 7.31). Even though the form of the proposed compressive hinge length resembles $L_p$ from Priestley, Calvi, and Kowalsky (2007), Figure 7.1, the predicted strain-displacement relationships differ due to the fact that strain penetration is now decoupled from column flexural displacements.

\[ k = 0.2 \left( \frac{f_u}{f_y} - 1 \right) \leq 0.08 \quad \text{Same Definition of } k \text{ as Priestley, Calvi, and Kowalsky (2007)} \quad \text{Eqn 7.27} \]

$L_{pr_t} = 2kL_c + 0.75D \quad \text{Tension Hinge Length Based on Triangular Distribution} \quad \text{Eqn 7.28}$

$L_{pr_c} = 2kL_c \quad \text{Compression Hinge Length Based on Triangular Distribution} \quad \text{Eqn 7.29}$

$L_{p_t} = L_{pr_t}/2 = kL_c + 0.375D \quad \text{Tension Hinge Length Based on Rect. Dist.} \quad \text{Eqn 7.30}$

$L_{p_c} = L_{pr_c}/2 = kL_c \quad \text{Compression Hinge Length Based on Rectangular Dist.} \quad \text{Eqn 7.31}$
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Moment Area Method for the Plastic Displacement with Rectangular Hinge Length

\[ \Delta_p = \left( \phi_p L_p \right) \left( L - \frac{L_p}{2} \right) - \left( \phi_p L_p \right) \left( \frac{L_p}{2} \right) \]

\[ \Delta_p = \left( \phi_p L_p \right) (L - L_p) \]

Figure 7.20 (Double Bending) Plastic Displacement with a Rectangular Hinge Length

Moment Area Method for the Plastic Displacement with Triangular Hinge Length

\[ \Delta_p = \left( \frac{1}{2} \phi_p L_{pr} \right) \left( L - \frac{L_{pr}}{3} \right) - \left( \frac{1}{2} \phi_p L_{pr} \right) \left( \frac{L_{pr}}{3} \right) \]

\[ \Delta_p = \left( \frac{1}{2} \phi_p L_{pr} \right) \left( L - \frac{2L_{pr}}{3} \right) \]

Figure 7.21 (Double Bending) Plastic Displacement with a Triangular Hinge Length
Figure 7.22 (Test 9) Response Comparison with Logarithmic Best Fit to Measured Spread of Plasticity (Best Fit Lpr)
Since the $Lpr_t$ equation was calibrated based on the upperbound extent of plasticity of each experiment, these regions need to be isolated in order to compare the accuracy of the proposed equation. For example, data points from the three highest curvature ductility values in Test 9, Figure 7.24, are included in the formulation. A comparison of the measured upperbound extent of plasticity and the result of the tensile plastic hinge length expression, $Lpr_t$ Eqn 7.28, appears in Figure 7.25. A sensitivity analysis of the proposed $Lpr_t$ equation to individual test variables is shown in Figure 7.26. Of the variables investigated, the $Lpr_t$ expression underpredicts the spread of plasticity for tests Tests 23 and 24 with the highest aspect ratio. However, this was not found to influence the accuracy of tensile strain-displacement predictions for these tests, Figure 7.54 and Figure 7.55.

Figure 7.23 Tensile and Compressive Triangular Plastic Hinge Lengths
A strain-displacement comparison for Test 9 appears in Figure 7.29 for the Priestley, Calvi, and Kowalsky (2007) $L_p$ method integrated into CUMBIA and the proposed Modified $L_{pr}$ method. Both the tensile and the compressive strain-displacement predictions are improved with proposed method. The degree of improvement for the compressive strain-displacement relationship becomes more apparent when evaluating the dataset as a whole. A test by test comparison of the strain-displacement relationship prediction appears in Figure 7.40 through Figure 7.61. There are still many instances where the measured compressive strains significantly exceed the prediction in regions of the column with inelastic transverse steel. The strain-displacement prediction for column Test 9 using the triangular distribution of curvature from $L_{pr_t}$ is compared to the rectangular-based $L_{p_t}$ in Figure 7.30 and Figure 7.31. The slight difference in the two methods is attributed to the factor of two approximation used as a conversion between rectangular and triangular equivalent curvature distributions, Figure 7.17.

The purpose of an equivalent curvature distribution is to translate a known moment-curvature relationship into the backbone curve of a member force-deformation response. The question remains, which hinge length, tension or compression, offers a better member response prediction. For column Test 9, Figure 7.27 and Figure 7.28 plot the force-deformation response predicted with $L_p$ from Priestley, Calvi, and Kowalsky (2007) integrated into CUMBIA, and the response from either $L_{pr_c}$ or $L_{pr_t}$. Of the two proposed methods, the compressive hinge length provides a more accurate force-displacement prediction since the analysis was terminated at 140% of the Mander ultimate concrete compression strain for the confined core, which can only be assessed with $L_{pr_c}$. The elastic range of response closely resembles that of the current method.

$$\text{mean} \left( \frac{L_{pr}^{meas}}{L_{pr}^{eqn}} \right) = 0.98, \text{COV} = 0.069$$

Statistics for Tensile Triangular Plastic Hinge Length
Figure 7.24  Isolated Upper Bound Lpr used in Sensitivity Analysis for Eqn 7.28

Figure 7.25  Comparison of Tensile Lpr Equation and Measured Extent of Plasticity
Figure 7.26 Sensitivity of Tensile Lpr Equation to Individual Variables
Figure 7.27  Hysteretic Response for Test 9 with Compressive Lpr and PCK (2007) Lp

Figure 7.28  Hysteretic Response for Test 9 with Tensile Lpr and PCK (2007) Lp
Figure 7.29 Spread of Plasticity and Strain-Displacement Relationship for Test 9 with Modified Lpr Hinge Method
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Figure 7.30 Prediction using Triangular and Rectangular Tensile Hinge Lengths

Figure 7.31 Prediction using Triangular and Rectangular Tensile Hinge Lengths
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7.4 Tensile Strain-Displacement Predictions using the Modified Plastic Hinge Method

Tensile strain-displacement predictions using Modified Plastic Hinge Model are based on the upper bound measured extent of plasticity of bridge column tests. A comparison of the accuracy of the Modified vs. the current Plastic Hinge Method from Priestley, Calvi, and Kowalsky (2007), Figure 7.1, is made utilizing the measured peak tensile strains prior to bar buckling and their associated top-column displacements. As shown in the statistics below, both equivalent curvature distributions yield conservative predictions for the tensile strain-displacement relationship, however, the modified plastic hinge method shows significant improvement. The tensile triangular plastic hinge length, Eqn 7.28, was formulated using the upper bound measured spread of plasticity. Near the footing, tensile strains only gradually reduce with increases in height. The linear shape of the plastic curvature profile near the footing is attributed to larger compressive strains near the footing-column interface. The distribution of tensile strains is influenced by the tension shift effect, which may decrease the amplitude of tensile strains at the footing-column interface and instead increase tensile strain magnitudes above the base section following the inclined flexural-shear crack distribution.

*Mean, Coefficient of Variation, and Root Mean Squared Error*

\[
\text{mean} \left( \frac{\Delta_{\text{measured}}^{bb} \text{ at } \varepsilon_{\text{measured}}^{bb} \text{ with } \Delta_{\text{Cumbia}}^{bb} \text{ at } \varepsilon_{\text{measured}}^{bb} \text{ with } Lp \text{ EQN}}{\Delta_{\text{Cumbia}}^{bb} \text{ at } \varepsilon_{\text{measured}}^{bb} \text{ with } Lp \text{ EQN (PCK 2007)}} \right) = 1.12, \text{COV} = 0.061, \text{RMSE} = 0.121
\]

\[
\text{mean} \left( \frac{\Delta_{\text{measured}}^{bb} \text{ at } \varepsilon_{\text{measured}}^{bb} \text{ with } \Delta_{\text{Cumbia}}^{bb} \text{ at } \varepsilon_{\text{measured}}^{bb} \text{ with } Lp \text{ EQN (PCK 2007)}}{\Delta_{\text{Cumbia}}^{bb} \text{ at } \varepsilon_{\text{measured}}^{bb} \text{ with } Lp \text{ EQN (PCK 2007)}} \right) = 1.27, \text{COV} = 0.116, \text{RMSE} = 0.222
\]
Alternatively, this tensile predictive capacity of the hinge models can be compared using cumulative probability distributions, Figure 7.32. Each observation of measured/predicted bar buckling displacement is given a probability of $1/n$, where $n$ is the total number of observations. These individual observations are sorted in ascending order. The first observation, with the lowest ratio of measured/equation value has a probability of $1/n$, while the second has a probability of $2/n$, until the final observation has a probability of $1$. A near vertical line at a measured/predicted ratio of one would denote an accurate prediction with low variability. Both hinge methods produce cumulative probability distributions to the right of one with conservative predictions for the bar buckling displacement which were lower than those experienced in the tests. The cumulative probability distributions and the comparison of the Root Mean Squared Error for the two equivalent curvature distributions show that the Modified Lpr hinge method is more accurate for tensile strain-displacement predictions. A sensitivity analysis for the Tensile Lpr bar buckling strain-displacement for individual test variables appears in Figure 7.33.

Figure 7.32 Comparison of Plastic Hinge Method Tensile Predictive Capabilities
Figure 7.33  Sensitivity of Tensile Lpr Bar Buckling Strain-Displacement Predictions to Individual Variables
7.5 Compressive Strain-Displacement Predictions using the Modified Plastic Hinge Method

Compressive strain-displacement predictions using Modified Plastic Hinge Model are based on only the moment gradient component of the spread of plasticity ($Lpr_c = 2kL$). Comparisons of the accuracy of the Modified vs. the current Plastic Hinge Method from Priestley, Calvi, and Kowalsky (2007), Figure 7.1, were made utilizing the measured compressive strain and displacement at cover concrete crushing and confinement (spiral) steel yielding observations. Note that these measured compression strains are from instruments applied to the extreme fiber longitudinal bar, which is an approximation to the cover and core concrete strain. Both equivalent curvature distributions yield unconservative predictions for the displacement at cover crushing and spiral yielding, but the Modified Plastic Hinge Method shows improvement.

The cumulative probability distribution describing the accuracy of both hinge models in predicting the column displacement at the measured cover crushing strain appears in Figure 7.34. A sensitivity analysis of the impact of individual variables on the accuracy of the cover crushing strain-displacement prediction is shown in Figure 7.35. The same comparison was repeated in Figure 7.36 and Figure 7.37 for the displacement prediction at the measured compression strain that coincided with the observation of spiral yielding. More accurate compressive strain-displacement predictions were made at the lower cover crushing strain when compared to that of spiral yielding, which may be influenced by more localized compressive behavior. After spiral yielding, significantly larger compressive strains were measured in many tests, as can be seen in the peak compressive strain-displacement relationships of individual tests results, Figure 7.40 through Figure 7.61. The Modified Plastic Hinge Method improved the accuracy of compressive strain-displacement predictions, but it can be argued that the effects of tension shift should reduce the compressive hinge length below that of just the moment gradient component, due to the localization of compressive demands at the pivot point for the fanned diagonal compressive strut pattern.
\* Mean, Coefficient of Variation, and Root Mean Squared Error

\[
\text{mean} \left( \frac{\Delta_{\text{Measured at } \varepsilon_{\text{crushing}}^{\text{crush}}} \Delta_{\text{Cumia at } \varepsilon_{\text{crushing}}^{\text{crush}}} \text{ with } Lp EQN} {\text{EQN (PCK 2007)}} \right) = 0.93, COV = 0.105, RMSE = 0.145
\]

\[
\text{mean} \left( \frac{\Delta_{\text{Measured at } \varepsilon_{\text{crushing}}^{\text{crush}}} \Delta_{\text{Cumia at } \varepsilon_{\text{crushing}}^{\text{crush}}} \text{ with } Lp EQN (PCK 2007)} {\text{EQN (PCK 2007)}} \right) = 0.80, COV = 0.152, RMSE = 0.340
\]

Figure 7.34 Comparison of Plastic Hinge Method Compressive Predictive Capabilities at Cover Crushing Observations
Figure 7.35  Sensitivity of Compressive Lpr Cover Crushing Strain-Displacement Predictions to Individual Variables
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*Mean, Coefficient of Variation, and Root Mean Squared Error*

\[
\text{mean} \left( \frac{\Delta_{\text{Measured}} \text{ at } \varepsilon_{\text{Measured}}^{\text{spiral yield}}}{\Delta_{\text{Cumbia}} \text{ at } \varepsilon_{\text{Measured}}^{\text{spiral yield}} \text{ with Lpr EQN}} \right) = 0.88, COV = 0.192, RMSE = 0.266
\]

\[
\text{mean} \left( \frac{\Delta_{\text{Measured}} \text{ at } \varepsilon_{\text{Measured}}^{\text{spiral yield}}}{\Delta_{\text{Cumbia}} \text{ at } \varepsilon_{\text{Measured}}^{\text{spiral yield}} \text{ with Lp EQN (PCK 2007)}} \right) = 0.71, COV = 0.221, RMSE = 0.529
\]

Figure 7.36  Comparison of Plastic Hinge Method Compressive Predictive Capabilities at Spiral Yielding Observations
Figure 7.37  Sensitivity of Compressive Lpr Spiral Yielding Strain-Displacement Predictions to Individual Variables
7.6 Elastic Force-Deformation Predictions using the Modified Plastic Hinge Method

The equivalent curvature distributions prior to first yield of longitudinal reinforcement for the Priestley, Calvi, and Kowalsky (2007) and the Modified Plastic Hinge Methods closely resemble one another, Figure 7.1 and Figure 7.2. Separate equivalent curvature distributions are used for elastic column flexure and base rotations attributable to strain penetration in the Modified Plastic Hinge Method. These two deformation components are combined into a single equivalent curvature distribution in the PCK (2007) method through the use of an effective column length which extends into adjoining members by a depth equal to the strain penetration length.

A comparison of the accuracy of the Modified vs. the current Plastic Hinge Method from Priestley, Calvi, and Kowalsky (2007), Figure 7.1, is made utilizing the measured top column displacement at the analytically predicted first yield force for longitudinal reinforcement. Both methods yield conservative predictions for the column displacement at the analytical first yield force. The cumulative probability distributions for both equivalent curvature profiles appear in Figure 7.38, and a sensitivity analysis for results of the Modified Plastic Hinge Method is shown in Figure 7.39.
*Mean, Coefficient of Variation, and Root Mean Squared Error*

\[
\text{mean} \left( \frac{\Delta_{\text{Measured at } F'_y}}{\Delta_{\text{Cumbia at } F'_y \text{ with } Lpr EQN}} \right) = 1.09, \text{COV} = 0.058, \text{RMSE} = 0.404
\]

\[
\text{mean} \left( \frac{\Delta_{\text{Measured at } F'_y}}{\Delta_{\text{Cumbia at } F'_y \text{ with } Lp EQN (PCK 2007)}} \right) = 1.07, \text{COV} = 0.059, \text{RMSE} = 0.332
\]

**Figure 7.38** Comparison of Plastic Hinge Method Predictive Capabilities at Analytical First Yield Force
Figure 7.39 Sensitivity of Modified Hinge Method Displacement at Analytical First Yield Force Predictions to Individual Variables
7.7 Conclusion

In design, concrete compressive and steel tensile strain limits are related to column deformations through the use of an equivalent curvature distribution. An experimental study was carried out to assess the performance of thirty circular, well-confined, bridge columns with varying lateral displacement history, transverse reinforcement detailing, axial load, aspect ratio, and longitudinal steel content. Material strains, cross section curvatures, and fixed-end rotations due to strain penetration of reinforcement into the adjoining member were quantified through the use of a 3D position monitoring system. This data was used to formulate a new equivalent curvature distribution aimed at improving tensile and compressive strain-displacement predictions.

The key aspects of the proposed Modified Plastic Hinge Model which differentiate it from the current method recommended in Priestley, Calvi, and Kowalsky (2007) include: (1) a decoupling of column flexure and strain penetration deformation components, (2) a linear plastic curvature distribution which emulates the measured curvature profiles, and (3) separate plastic hinge lengths for tensile and compressive strain-displacement predictions. In the experiments, the measured extent of plasticity was found to increase due to the combined effects of moment gradient and tension shift. The proposed tension hinge length, \( L_{pr_t} \) Eqn 7.28, was calibrated to match the upper bound of the measured spread of plasticity in each test. The proposed compressive hinge length, \( L_{pr_c} \) Eqn 7.29, only contains a term related to the moment gradient effect. Expressions for the elastic and plastic column flexural displacement for both single and double bending were derived. Expressions which describe the additional column deformation due to strain penetration of reinforcement into the adjoining member were formulated based on the measured fixed-end rotations. When compared to the current plastic hinge method recommended in Priestley, Calvi, and Kowalsky (2007), the proposed Modified Plastic Hinge method improved the tensile and compressive-strain displacement predictions while maintaining similar levels of accuracy for elastic displacements.
Figure 7.40 Spread of Plasticity and Strain-Displacement Relationship for Earthquake Load History Test 8
Figure 7.41 Spread of Plasticity and Strain-Displacement Relationship for Earthquake Load History Test 10
Figure 7.42 Spread of Plasticity and Strain-Displacement Relationship for Earthquake Load History Test 11
Figure 7.43 Spread of Plasticity and Strain-Displacement Relationship for Earthquake Load History Test 12
Figure 7.44 Spread of Plasticity and Strain-Displacement Relationship for Test 13
Figure 7.45 Spread of Plasticity and Strain-Displacement Relationship for Test 14
Figure 7.46 Spread of Plasticity and Strain-Displacement Relationship for Test 15
Figure 7.47 Spread of Plasticity and Strain-Displacement Relationship for Test 16
Chapter 7: Modified Plastic Hinge Method

Figure 7.48 Spread of Plasticity and Strain-Displacement Relationship for Earthquake Load History Test 17
Figure 7.49 Spread of Plasticity and Strain-Displacement Relationship for Earthquake Load History Test 18
Figure 7.50 Spread of Plasticity and Strain-Displacement Relationship for Test 19
Figure 7.51 Spread of Plasticity and Strain-Displacement Relationship for Test 20
Figure 7.52 Spread of Plasticity and Strain-Displacement Relationship for Test 21
Figure 7.53 Spread of Plasticity and Strain-Displacement Relationship for Test 22
Figure 7.54 Spread of Plasticity and Strain-Displacement Relationship for Test 23
Figure 7.55 Spread of Plasticity and Strain-Displacement Relationship for Test 24
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Figure 7.56 Spread of Plasticity and Strain-Displacement Relationship for Test 25
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Figure 7.57 Spread of Plasticity and Strain-Displacement Relationship for Test 26
Figure 7.58 Spread of Plasticity and Strain-Displacement Relationship for Test 27
Figure 7.59 Spread of Plasticity and Strain-Displacement Relationship for Test 28
Figure 7.60 Spread of Plasticity and Strain-Displacement Relationship for Test 29
Figure 7.61 Spread of Plasticity and Strain-Displacement Relationship for Test 30
Chapter 8: Performance Strain Limits for Circular Bridge Columns

8.1 Background

This section discusses a research program supported by the Alaska Department of Transportation and Alaska University Transportation Center aimed at defining accurate limit state displacements which relate to specific levels of damage in reinforced concrete bridge columns subjected to seismic hazards. Bridge columns are designed as ductile elements which form plastic hinges to dissipate energy in a seismic event. To satisfy the aims of performance based design, levels of damage which interrupt the serviceability of the structure or require more invasive repair techniques must be related to engineering criteria. For reinforced concrete flexural members such as bridge columns, concrete compressive and steel tensile strain limits are very good indicators of damage.

Serviceability limit states such as concrete cover crushing or residual crack widths exceeding 1mm may occur during smaller, more frequent earthquakes, Priestley et al. (1996). While the serviceability limit states do not pose a safety concern, the hinge regions must be repaired to prevent corrosion of internal reinforcing steel. At higher ductility demands produced by larger less frequent earthquakes, reinforcing bar buckling may lead to permanent elongation in the transverse steel, which diminishes its effectiveness in confining the concrete core. Bar buckling and significant damage to the core concrete represent the damage control limit states, which when exceeded lead to significant repair costs, Priestley et al. (1996). Furthermore, rupture of previously buckled bars during subsequent cycles of loading leads to rapid strength loss. The life safety or collapse prevention limit state is characterized by fracture of previously buckled bars or confinement steel.

A summary of the current performance strain limit recommendations from (Kowalsky 2000) appear in Table 8.1. The first occurrence of these limit states in a cyclic column test
by Goodnight et al. is shown in Figure 8.1. The symmetric three-cycle-set load history and resulting force versus deformation response are shown with labels representing the first occurrence of cover crushing, bar buckling, and bar fracture limit states. Note that the damage control concrete compressive strain limit of 0.018 is shown as a typical value of the Mander et al (1988) ultimate concrete compressive strain for a volumetric steel ratio around 1%. If the exact detailing is known based on confinement or shear demands, then the appropriate ultimate concrete compressive strain value should be used instead.

While the progression of damage in flexural bridge columns has been thoroughly investigated in the past, to the authors’ knowledge, none of the previous studies measured strains at the level of the reinforcement throughout the entire range of response. Traditional instrumentation methods utilized linear potentiometers placed on the ends of threaded rods embedded in the core concrete to calculate changes in displacement outside of the concrete cover. A diagram of this instrumentation system from tests by Hose et al. (1997) appeared in Hines et al. (2003), Figure 8.5. This method does not measure material strains at the locations of interests, and its measurements are influenced small rotations of the rods themselves which result due to the curvature gradient over the gage length.

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Concrete Compressive Strain Limit</th>
<th>Steel Tensile Strain Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Serviceability</td>
<td>0.004</td>
<td>0.015</td>
</tr>
<tr>
<td></td>
<td>Cover Concrete Crushing</td>
<td>Residual Crack Widths Exceed 1mm</td>
</tr>
<tr>
<td>Damage Control</td>
<td>( \approx 0.018, \text{Mander et al. (1988)} ) ( \varepsilon_{cu} )</td>
<td>0.060</td>
</tr>
<tr>
<td></td>
<td>Limit of Economical Concrete Repair</td>
<td>Tension Based Bar Buckling</td>
</tr>
</tbody>
</table>
Figure 8.1 Displacement History, Hysteretic Response, and Performance Limit States

Figure 8.2 Curvature Rod and Linear Potentiometer Instrumentation from Tests by (Hose et al. 1997), Figure appears in (Hines et al. 2003)
8.2 Experimental Program

The goal of the experimental program is to investigate the impact of load history and other design variables on the relationship between strain and displacement, performance strain limits, and the spread of plasticity. In this section, the details of the experimental program are briefly described before presenting a series of predictive limit state expressions which were formulated based on the test results. The main variables for the thirty circular bridge column tests included: (1) lateral displacement history, (2) axial load, (3) longitudinal steel content, (4) aspect ratio, and (5) transverse steel detailing.

The specimen was designed to represent a single degree of freedom bridge column subjected to lateral and axial load, Figure 8.3. The test specimen consists of a footing, column, and loading cap. The footing is a capacity protected member which secures the specimen to the lab strong floor using post tensioned bars. A 200kip hydraulic actuator, with a 40in stroke capacity, applies lateral load to the loading cap of the specimen. A spreader beam, two hydraulic jacks, and a load cell are placed above the loading cap to apply a constant axial compressive load. The top column displacement was obtained through a string potentiometer placed at the center of the lateral load.

A key feature of the experiments was the high fidelity strain data obtained through the use of an optical 3D position measurement system. The experimental program utilized multiple Optotrak Certus HD 3D position sensors developed by Northern Digital Inc. to monitor material strains. The position sensors track the locations of the target markers in 3D space, returning X-Y-Z spatial coordinates with an accuracy of 0.1mm with a resolution of 0.01mm. A technique of applying target markers to longitudinal and transverse reinforcement, Figure 8.6, was utilized in the plastic hinge region. Strains are computed by dividing the change in three dimensional distance between two adjacent target markers by the original unloaded gage length.

An overview of geometry, reinforcement, material properties, and bar buckling observations for column Tests 8-30 appears in Table 8.2. An observational summary for
each test appears in Chapter 3. The impact of lateral displacement history is the subject of Chapter 4, and the influence of other design variables is summarized in Chapter 5. Two different cross sections were utilized in the study, an 18” and a 24” diameter configuration as shown in Figure 8.4. The 24” configuration had 16 A706 longitudinal bars of either #6 (0.75 in) or #7 (0.875 in) diameter and a #3 (0.375 in) or #4 (0.5 in) A706 spiral at variable spacing. For both specimens the cover depth to the outside of the spiral was ½”, which led to an outside spiral diameter of either 23” or 17”. Two techniques of blocking out the cover concrete were employed to attach the Optotrak target markers to the outside surface of the reinforcing steel. Cover concrete was either blocked out in longitudinal strips over extreme fiber bars or around the entire circumference of the column plastic hinge region, Figure 8.3. The results of Tests 1-6 were excluded in the formulation of design recommendations because they utilized a steel post extension instrumentation technique which suffered from the same limitations as the curvature rod method.

Figure 8.3 (Left) Test Setup, (Middle) Optotrak Target Marker Application Method, (Right) Optotrak Strain Comparison to Traditional Techniques
8.2.1 Loading Protocol

The specimens were subjected to various unidirectional top-column displacement histories including standardized laboratory reversed cyclic loading and recreations of the displacement responses obtained from non-linear time history analysis of multiple earthquakes with distinct characteristics. The experiments utilized a quasi-static displacement controlled loading procedure. The symmetric three-cycle-set load history is commonly used to evaluate the seismic performance of structural components. The load history begins with elastic cycles to the following increments of the analytically predicted first yield force: $\frac{1}{4} F'_y$, $\frac{1}{2} F'_y$, $\frac{3}{4} F'_y$, and $F'_y$. The experimental first yield displacement is then determined by taking the average of the recorded displacements during the first yield push and pulls cycles. The equivalent yield displacement, used to determine the displacement ductility levels ($\mu_{\Delta 1} = 1 \ast \Delta_y$), is then calculated as $\Delta_y = \Delta'_y(M_n/M'_y)$. The symmetric three-cycle-set load history resumes with three balanced cycles at each of the following displacement ductility levels: 1, 1.5, 2, 3, 4, 5, etc.
8.3 Observed Damage Sequence

The following sequence of damage was observed in all of the cyclically loaded experiments: (1) concrete cracking, (2) longitudinal steel yielding, (3) cover concrete crushing, (4) confinement steel yielding, (5) longitudinal bar buckling, and (6) fracture of previously buckled reinforcement. The first significant loss in strength occurred when previously buckled reinforcement fractured. Fracture of confinement steel was never observed. The impact of individual variables on the displacement and material strains at key performance limit states was the main focus of Chapters 4 and 5.

In this section, the measured strain data is used to refine strain limit recommendations from Kowalsky (2000) in Figure 8.1. Particular attention is paid to the limit state of longitudinal bar buckling, since it limited the deformation capacity of all of the cyclically loaded specimens. Empirical expression are developed to predict the compressive strain at cover crushing, the compressive strain at spiral yielding, and the peak tensile strain prior to visible buckling after reversal of loading. Limit state displacements are evaluated using the Modified Lpr Plastic Hinge Method and compared to those observed experimentally. The formulation for this equivalent curvature distribution appears in Chapter 7.

Analytical studies by Berry (2006) utilized a database of experimentally tested bridge columns with defined material, geometric, and reinforcing properties, with reported bar buckling observations. The dataset, in Table 8.3, is a subset of the Column Structural Performance Database <www.ce.washington.edu/~peera1>. Berry (2006) created a predictive drift-based bar buckling expression for the dataset. As a means of comparison, the strain-based bar buckling expression and displacement formulated based on the Goodnight et al. dataset is compared to observed buckling displacements for columns in the Berry (2006) dataset. A second empirical drift-based expression was developed to predict bar buckling in the combined dataset.

Feng (2013) proposed a series of equations to describe bar buckling behavior observed in finite element analysis of reinforcing bars. The influence of the following behaviors were
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included in the analysis: (1) dilation of core concrete under compression, (2) restraint provided by individual spiral layers which can go inelastic, and (3) development of the longitudinal bar into the adjoining member. The analysis resulted in a multi-linear regression model which forms a boundary for tensile versus compressive strain relationship which would initiate bar buckling after reversal of load.

The results for each of these bar buckling prediction methods are compared to the combined Berry (2006) and Goodnight et al. datasets, and limitations of each method are explored. At the end of this discussion, recommendations for each of the performance limit states are provided. In some cases the current values are either verified or shown to be conservative, while in others new expressions are recommended.
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Table 8.2 Goodnight et al. Bridge Column Dataset

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Table 8.3 Berry et al. (2006) Dataset

* Additional information for each test can be found in the Column Structural Performance
Database <www.ce.washington.edu/~peera1>.


8.4 Equation to Predict Peak Tension Strain Prior to Bar Buckling Upon Reversal of Load

Based on the Goodnight et al. dataset, Table 1.1, which contained measured strain data for reinforcing bars, an empirical equation was devised to predict the peak tensile strain prior to bar buckling upon reversal of load, Eqn 8.1. Due to the empirical nature of the equation, limits on its applicability to columns outside the dataset must be employed. These limits will be addressed when comparing the accuracy of bar buckling predictions for the Berry et al. dataset, Table 8.3. A comparison of the accuracy of Eqn 8.1 evaluated against the measured peak tensile strains prior to bar buckling from the Goodnight et al. dataset appears in Figure 8.5. The accuracy of the equations can alternatively be visualized in the format of a cumulative probably distribution, Figure 8.6. The x-axis of Figure 8.6, (Strain at Bar Buckling / Equation Strain), can be interpreted as a ratio of demand to capacity. Each observation of bar buckling is given a probability of 1/n where n is to the total number of observations. These individual observations are sorted in ascending order of measured/equation strain. The first observation, with the lowest ratio of measured/equation value has a probability of 1/n, while the second has a probability of 2/n, until the final observation has a probability of 1. The normal cumulative distribution function is evaluated with the mean and standard deviation of the (Strain at Bar Buckling / Equation Strain) dataset. If the normal distribution is selected, and the Goodnight et al. dataset is chosen to be representative of bridge columns in general, Figure 8.6 would imply that there is a 40% probability of bar buckling if the demand equals the predicted capacity. Comments on applicability of Eqn 8.1 to columns outside the dataset will be reserved for later sections of this report when predictions are made for the Berry et al. (2006) dataset in Table 8.3.
\[ \varepsilon_{sb} = 0.03 + 700 \rho_s \frac{f_{yh}}{E_s} - 0.1 \frac{P}{f'_c A_g} \]  
Peak Tension Strain Prior to Bar Buckling  \hspace{1em} \text{Eqn 8.1}

\[ \rho_s = \frac{4A_{sp}}{D's} \]  
Transverse Volumetric Steel Ratio, influences confinement and bar restraint.

\[ \frac{f_{yh}}{E_s} \]  
Inelastic Transverse steel is less effective at restraining longitudinal bars.

\[ \frac{P}{f'_c A_g} \]  
Axial Load Ratio expressed as a decimal.

\[ \text{mean} \left( \frac{\varepsilon_{sb}^{\text{Measured}}}{\varepsilon_{sb}^{\text{Eqn 8.1}}} \right) = 1.05 \text{ and COV} = 0.199 \]

**Figure 8.5** Graph of Measured Peak Tensile Strains and Result of Eqn 8.1
Figure 8.6 Cumulative Probability Distribution for Peak Tensile Strain Prior to Bar Buckling Eqn 8.1 (x-axis represents demand / calculation)

Figure 8.7 Sensitivity of Eqn 8.1 to Individual Variables in the Goodnight et al. Dataset
Figure 8.8 Sensitivity of Eqn 8.1 to Individual Variables in the Goodnight et al. Dataset
8.5 Column Deformation at Peak Tensile Strain Prior to Bar Buckling

Predictions for the lateral column deformation at the peak tension strain prior to bar buckling can be made by employing monotonic section analysis and an equivalent curvature distribution. A modified plastic hinge method based on the measured spread of plasticity in the Goodnight et al. dataset is briefly described below for a cantilever column in single bending, Eqn 8.2 through Eqn 8.11 and Figure 8.9.

\[\Delta_T = (\Delta_e + \Delta_sp + \Delta_p + \Delta_{shear})\] Total Top Column Displacement Eqn 8.2

\[\Delta_e = \phi_{base}L^2/3\] (Single) Elastic Flexural Displacement before First Yield Eqn 8.3

\[\Delta_e = \phi'_y(M/M'_y)L^2/3\] (Single) Elastic Flexural Displacement after First Yield Eqn 8.4

\[L_{sp} = U \left(1 - \frac{P}{f_{ce}A_g} - \frac{L_c}{16D}\right)f_{ye}d_{bl} \sqrt{\frac{f_{ce}}{f_{ce_f}}}\] \[U = 0.4\] for ksi and 0.152 for MPa units Eqn 8.5

\[\Delta_{sp} = (\phi_{base}L_{sp})L\] Strain Penetration Displacement Eqn 8.6

\[\phi_p = \phi_{base} - \phi'_y(M/M'_y)\] Plastic Curvature at the Base Section Eqn 8.7

\[\Delta_p = \phi_p(L_{pr}/2)[L - L_{pr}/3]\] (Single) Plastic Displacement for Triangular Plastic Curvature Distribution Eqn 8.8

\[k = 0.2 \left(\frac{f_u}{f_y} - 1\right) \leq 0.08\] Same Definition of k as Priestley, Calvi, and Kowalsky (2007) Eqn 8.9
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\[ L_{pr_t} = 2kL_c + 0.75D \]  
(Single Bending, \( L_c = L \)) Tension Hinge Length  
Based on Triangular Distribution  

Eqn 8.10

\[ L_{pr_c} = 2kL_c \]  
(Single Bending, \( L_c = L \)) Compression Hinge Length  
Based on Triangular Distribution  

Eqn 8.11

Figure 8.9 Modified Plastic Hinge Method for a Column in Single Bending
Predictions for the peak tensile strain before bar buckling, Eqn 8.1 are translated to top column displacements using monotonic section analysis in a script named Cumbia and Eqn 8.2 through Eqn 8.11. Specifically, the tensile triangular plastic hinge length, $L_{pt}$ from Eqn 8.10, is employed to translate a tensile strain to a top column displacement. The result of this analysis appears in Figure 8.10 and Figure 8.11. Introduction of the Modified Plastic Hinge Method adds conservatism, with fewer specimens experiencing bar buckling at the Eqn 8.1 strain. As discussed previously, this additional conservatism is less than would be induced if the plastic hinge method from Priestly, Calvi, and Kowalsky (2007) were used instead.

$$\text{mean}\left( \frac{\Delta_{\text{measured}}^{bb} \text{ at } \varepsilon_s^{bb}\text{measured}}{\Delta_{\text{Cumbia}} \text{ at } \varepsilon_s^{bb}\text{Eqn 8.1 with Modified Lpr Method}} \right) = 1.17 \text{ and } \text{COV} = 0.157$$

Figure 8.10 Graph of Measured Peak Tensile Drift and Result of $\Delta_{\text{Cumbia}} \text{ at } \varepsilon_s^{bb}\text{Eqn 8.1}$
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Figure 8.11 Cumulative Probability Distribution for $\text{Drift}_{bb}$ at $\varepsilon_{\text{Eqn}}^{bb}$ (x-axis represents demand / calculation)

8.6 Berry (2006) Statistical Drift-Based Bar Buckling Model for Circular Bridge Columns

Analytical studies by Berry (2006) utilized a database of experimentally tested bridge columns with defined material, geometric, and reinforcing properties, with reported bar buckling observations. The dataset, in Table 8.3, is a subset of the Column Structural Performance Database <www.ce.washington.edu/~peera1>. The dataset in Table 8.3 is more specific to bridge columns than the generalized circular column dataset presented in Berry and Eberhard (2005). Berry (2006) devised a statistical drift-based bar buckling model, Eqn 8.12, to fit the bridge column dataset.

It is important to note that low cycle fatigue load history tests by Kunnath et al. (1997) specimens A4, A5, and A6 were excluded from the formulation of Eqn 8.12. These load
histories are more severe than traditional symmetric three-cycle-set deformation histories which include a gradual ramp up of aptitude with three cycles at a given level of displacement. The low cycle fatigue load histories consisted of repeated cycling at a given displacement amplitude until bar buckling was observed. Alternatively, the peak excursion style load histories of Moyer and Kowalsky (2001) specimens No. 2, 3, and 4 were included in the dataset. These load histories were devised to isolate specific influences of peak tensile strain on bar buckling and are, in general, less severe than a symmetric three-cycle-set load history employed in Moyer and Kowalsky (2001) Specimen No. 1. Future evaluation of the strain based bar buckling prediction in this report includes the entire dataset of Table 8.3, regardless of load history employed.

\[
\frac{\Delta_{bb}^{\text{calculated}}}{L} \times (\%) = 3.25 \left(1 + 150 \frac{\rho_{eff} d_{bl}}{D}\right) \left(1 - \frac{P}{f'_c A_g}\right) \left(1 + \frac{L}{10D}\right)
\]


\[
\rho_{eff} = \frac{\rho_s f_{yh}}{f'_c}
\]

Parameter termed the effective confinement ratio.

mean \left(\frac{\Delta_{bb}^{\text{measured}}}{\Delta_{bb}^{\text{calc}}}\right) = 1.01 \text{ and } COV = 24.7

For the Berry (2006) Circular Bridge Column Dataset
8.7 Berry (2006) Bar Buckling Model Applied to the Goodnight et al. Dataset

Experiments within the Goodnight et al. dataset fall within the range of applicability of the Berry (2006) drift-based bar buckling model, Eqn 8.12. The accuracy of Eqn 8.12 in predicting bar buckling observations in the Goodnight et al. dataset is shown in Figure 8.13 and Figure 8.14.

\[
\text{mean} \left( \frac{\Delta_{\text{measured}}}{\Delta_{\text{calc}}} \right) = 0.92 \text{ and } COV = 0.171
\]

Berry (2006) Eqn 8.12 applied to Goodnight et al. dataset
Figure 8.13 Graph of Measured Peak Tensile Drift and Result of Berry (2006) Eqn 8.12

Figure 8.14 CDF for Berry (2006) Eqn 8.12 Applied to Goodnight et al. Dataset
8.8 Evaluation of Strain Based Bar Buckling Predictions for the Berry (2006) Dataset

Strain based predictions for bar buckling of individual tests within the Berry (2006) dataset, Table 8.3, were evaluated. Reported material, geometric, and reinforcement properties of each experiment were used to run a moment-curvature analysis. The peak tensile strain prior to bar buckling from Eqn 8.1 and its associated deformation from the Modified Plastic Hinge Method were compared to reported bar buckling observations. The measured to predicted bar buckling displacement predictions using the Eqn 8.1 strain and Modified Plastic Hinge Method for experiments in the Berry (2006) dataset are shown in Figure 8.15 and Figure 8.17. Measured to predicted bar buckling ratios for many of the Berry (2006) dataset experiments follow trends in the Goodnight et al. dataset, but some reported buckling observations occur at significantly larger deformations than the strain-based approach from Eqn 8.1 would predict.

Specific variables within the Berry (2006) dataset can account for some of the disparity in bar buckling predictions, Figure 8.15. The severity of the imposed lateral displacement history can influence bar buckling. Low cycle fatigue experiments by Kunnath et al. (1997), specimens A4, A5, and A6, have smaller displacements at bar buckling than the prediction. Alternatively, the peak excursion load histories of Moyer and Kowalsky (2001), specimens No. 2, 3, and 4, have larger displacements at bar buckling. Earthquake load history tests by Kunnath et al (1997) specimens A7, A8, A9, A10, and A12 along with Goodnight et al. specimens 8,10,11,12,17, and 18 are included in their respective datasets.

Scaled specimens by Lehman et al. (1998) and Calderone et al. (2000) utilized smooth spirals with a yield stress which exceeded the upper bound yield stress for the A706 steel designation (78ksi). A comparison of the stress-strain response of spiral reinforcement from Lehman et al. (1998) and Goodnight et al. (material from Tests 25-30) appears in Figure 8.16. The stability of spiral layers confining the core concrete and restraining longitudinal bars from buckling is influenced by this change in behavior. In general, experiments from
Lehman et al. (1998) and Calderone et al. (2000) had bar buckling observations at displacements which exceed the strain based prediction.

The influence of individual variables on the accuracy of strain based bar buckling displacements for the combined Goodnight and Berry (2006) datasets appears in Figure 8.18. Column aspect ratio and the ratio of spiral spacing to longitudinal bar diameter appear to be the only variables which show a trend regarding the accuracy of the strain-based approach.

\[
\text{mean}\left(\frac{\Delta_{\text{Measured}}^{bb} \text{ for the Combined Dataset}}{\Delta_{\text{Cumibia at } \varepsilon_{EQN1}^{bb} \text{ with Modified Lpr Method}}}\right) = 1.21 \text{ and } COV = 0.220
\]

Figure 8.15 Cumulative Probability Distribution for Strain Based Bar Buckling Applied to Berry (2006) Dataset with Influence of Specific Test Variables
Figure 8.16 (Left) Spiral Stress-Strain Response Reported in Lehman et al. (1998) and Similar Characteristics in Spirals from Calderone et al. (2000); (Right) Sample Spiral Tensile Test Result for Specimens 25-30 of Goodnight et al.

Figure 8.17 CDF for Strain Based Bar Buckling Eqn 8.1 for the Combined Dataset
Figure 8.18 Sensitivity of Strain Based Bar Buckling Eqn 8.1 to Individual Variables in the Combined Dataset
8.9 Drift Based Approach Considering Combined Berry (2006) and Goodnight et al. Datasets

Since material strains are not available for tests in the Berry (2006) dataset, an equation with parameters designed to fit both datasets can only be created on the basis of drift. The strain based approach in Eqn 8.1 utilizes the same variables as the drift based Berry (2006) Eqn 8.12, with the exception of L/D which is only needed to evaluate the drift. Also, $\rho_s f_{yh}/E_s$ was found to more adequately describe the effect of transverse steel on bar buckling in the Goodnight et al. dataset when compared to $\rho_s f_{yh}/f'_c$ employed by Berry (2006). Both were considered in the formulation of the drift-based equation for the combined dataset, and $\rho_s f_{yh}/E_s$ provided a more accurate result. The proposed Eqn 8.13 can be used to evaluate the peak tensile displacement before bar buckling is expected to occur upon reversal of loading in a cyclic deformation history. It is important to note that lateral displacement history influences bar buckling, and that this equation is formed without consideration of the varying load histories utilized in each experiment. The influence of individual variables on the accuracy of Eqn 8.13 applied to the combined Goodnight and Berry (2006) datasets appears in Figure 8.20. A graphical comparison of the accuracy of the three predictive bar buckling methods appears in Figure 8.19 in the form of a cumulative probability distribution.

$$\frac{\Delta_{bb}^{\text{calculated}}}{L} (%) = 0.9 - 3.13 \frac{P}{f'_c A_g} + 142000 \rho_s \frac{f_{yh}}{E_s} + 0.45 \frac{L}{D}$$ \text{ for Combined Dataset} \hspace{1cm} \text{Eqn 8.13}

$$\text{mean} \left( \frac{\Delta_{bb}^{\text{measured}}}{\Delta_{bb}^{\text{calculated}}} \right) = 1.09 \text{ and } COV = 0.241 \text{ for Combined Berry (2006) and Goodnight et al. Dataset}$$
Figure 8.19 Cumulative Probability Distribution for Drift-Based Equation for Combined Dataset Compared to Berry (2006) and Strain-Based Approaches
Figure 8.20  Sensitivity of Eqn 8.13 to Individual Variables in the Combined Dataset
8.10 Feng (2013) Bar Buckling Strain Limit Expressions from Finite Element Analysis

Feng (2013) proposed a series of equations to describe bar buckling behavior observed in finite element analysis. The analysis model, Figure 8.21, considered an extreme fiber longitudinal bar with realistic boundary conditions. The influence of the following behaviors were included in the analysis: (1) dilation of core concrete under compression, (2) restraint provided by individual spiral layers which can go inelastic, and (3) development of the longitudinal bar into the adjoining member. Specific strain histories were applied to the longitudinal bar to evaluate the influence of peak tension strain and prior compressive strains on bar buckling behavior. This behavior, once quantified, was used to create the multi-linear regression Eqn 8.14 through Eqn 8.18 to predict the peak tensile strain prior to bar buckling upon reversal of load. In the expression, $\varepsilon_t$ and $\varepsilon_c$ are the tensile and compressive longitudinal bar strains (both taken as positive), $d_{bl}$ and $d_h$ are the longitudinal and transverse steel bar diameters, and $s$ is the centerline spacing of transverse steel.

Figure 8.21 Feng (2013) Finite Element Model Geometry for Critical Region of Extreme Fiber Bar with Realistic Boundary Conditions
An evaluation of the Feng (2013) method applied to Test 9 from Goodnight et al. appears in Figure 8.22. The bar buckling prediction is defined as the intersection of the multi-linear regression and the tensile-compressive bar strain relationship from moment-curvature analysis. The tensile and compressive longitudinal bar strains are evaluated at each level of curvature in the section analysis. The tension and compression strain couple at the intersection point represents a compression cycle followed by a tension cycle to the same level of displacement, which would induce bar buckling upon subsequent reversal of load. The Priestley, Calvi, and Kowalsky (2007) hinge method is used to translate the curvature at the intersection point strain to member deformation, Figure 8.23. If the displacement amplitudes or the strain history is known, the model can be used to evaluate the influence of previous load history on bar buckling. However, this was not done for bar buckling predictions in this study, since the load history is not known in the design of new structures.

\[
\varepsilon_t = \frac{-15 \left( \varepsilon_c - \frac{0.0205}{\sqrt{s_{bl}}} \right)}{\left( \frac{d_{bl}}{d_h} - 1 \right)^2} \quad \text{Eqn 8.14}
\]

\[
\varepsilon_t \geq -1.7 \frac{s}{d_{bl}} \frac{d_h}{d_{bl}} \varepsilon_c + 0.045 \frac{s}{d_{bl}} \quad \text{Eqn 8.15}
\]

\[
\varepsilon_t \leq 0.09, \quad \text{if } \frac{s}{d_{bl}} < 3 \quad \text{Eqn 8.16}
\]

\[
\varepsilon_t \leq 0.06, \quad \text{if } \frac{s}{d_{bl}} > 4 \quad \text{Eqn 8.17}
\]

\[
\varepsilon_t \leq 0.09 - 0.03 \left( \frac{s}{d_{bl}} - 3 \right), \quad \text{if } 3 < \frac{s}{d_{bl}} < 4 \quad \text{Eqn 8.18}
\]
Figure 8.22 Feng (2013) Method Applied to Test 9 from Goodnight et al.

Figure 8.23 Bar Buckling at Intersection Point using PCK (2007) Hinge Length
The predicted peak tension strains which result when the Feng (2013) method is applied to Tests 8-30 of the Goodnight et al. dataset appear in Figure 8.24. For now, consider only the data points which evaluate the peak tension strain according to the tension-compression strain relationship evaluated at the same curvature and displacement, which is consistent with the PCK (2007) plastic hinge method. The peak tension strains predicted by this method exceed those measured in the tests. The Goodnight et al. (2014) modified hinge method, Figure 8.9 and Eqn 8.2 through Eqn 8.11, utilizes separate tensile and compressive hinge lengths to account for the influence of tension shift and moment gradient on the distribution of plastic curvature in columns. An adjustment to the tensile-compressive strain relationship in Figure 8.22 are necessary to account for the use of separate tensile and compressive hinge lengths. For Test 9, the extreme fiber bar tensile and compressive strain-displacement relationships are evaluated using the modified hinge method in Figure 8.25. A linear regression is used to compute the tensile and compressive strains at the same level of displacement, Figure 8.26. In comparison, this approach produces lower peak tension strains prior to bar buckling, which reflects the difference in the tensile-strain displacement relationships for the two hinge methods. To reinforce this point, the tensile strain-displacement relationship for Test 9 appears in Figure 8.27 with a monotonic prediction using the two plastic hinge methods.

The multi-linear regression approach from Feng (2013), combined with the strain-displacement relationships from the modified plastic hinge method, produced predicted peak tension strains prior to bar buckling which more closely resemble those measured in the Goodnight et al. dataset, Figure 8.24. This approach produces peak tensile strains which resemble those predicted from the statistical strain-based bar buckling approach from Eqn 8.1. The cumulative probability distribution for the ratio of the measured peak tension strain prior to bar buckling to the predicted value appears in Figure 8.28. This figure illustrates the non-conservatism in applying the Feng (2013) method with the PCK (2007) hinge method to predict peak tension strains measured in the Goodnight et al. dataset. The accuracy of these three methods in predicting the peak tension strain prior to bar buckling in the Goodnight et al. dataset is shown below. Ideally, a predictive equation would have a mean value of one
and minimize the coefficient of variation, resulting in a near vertical line at one in the cumulative probability distribution. This would also minimize the root mean squared error, which can be used to compare the accuracy of the three bar buckling strain predictions.

*Mean, Coefficient of Variation, and Root Mean Squared Error

\[
\text{mean} \left( \frac{\varepsilon_{S_{\text{Measured}}}^{bb}}{\varepsilon_{S_{\text{Eqn 8.1}}}^{bb}} \right) = 1.05, \text{COV} = 0.199, \text{RMSE} = 0.188
\]

\[
\text{mean} \left( \frac{\varepsilon_{S_{\text{Measured}}}^{bb}}{\varepsilon_{S_{\text{Feng (2013) with PCK (2007) Lp}}}^{bb}} \right) = 0.81, \text{COV} = 0.224, \text{RMSE} = 0.387
\]

\[
\text{mean} \left( \frac{\varepsilon_{S_{\text{Measured}}}^{bb}}{\varepsilon_{S_{\text{Feng (2013) with Modified Lpr Method}}}^{bb}} \right) = 1.11, \text{COV} = 0.216, \text{RMSE} = 0.190
\]

In the following section, the computed deformation at the predicted peak tensile strains are compared to the measured displacement prior to bar buckling in the Goodnight et al. dataset. The measured peak tensile drift sustained before bar buckling for columns in the Goodnight et al. dataset is compared to the predicted bar buckling drift using the following methods: [1] strain-based Eqn 8.1 with the modified plastic hinge method, [2] drift-based Eqn 8.13 with coefficients based on the combined dataset, [3] Berry (2006) drift-based Eqn 8.12, [4] Feng (2013) multi-linear regression with the PCK (2007) hinge method, and [5] Feng (2013) method with strain-displacement relationships from the modified hinge method. A test by test comparison of the result of these four methods and the measured peak tensile drift prior to bar buckling is shown in Figure 8.29. The cumulative probability distribution for the (Measured / Predicted) peak tension drift for the four methods appears in Figure 8.30. These figures as well as the summary statistics listed below indicate that the drift-based Eqn 8.13 and the Feng (2013) approach with the PCK (2007) Lp hinge method produce the most accurate results.
As discussed previously, both plastic hinge methods induce conservatism when translating the measured peak tension strains to predicted displacements. The modified plastic hinge method reduces this conservatism significantly. The unconservative peak tensile strain predictions from the Feng (2013) method with the strain-displacement relationship from moment-curvature analysis appear to be balanced by the conservatism in the strain-displacement relationships from the PCK (2007) hinge method. Larger tension strains are predicted than those measured in the test, and the strain-displacement relationship predicts that these values would occur at lower levels of deformation than would occur with the measured tensile strain-displacement relationship.

By comparison, the methods which produced more accurate peak tensile strain predictions suffer from the induced conservatism when translating these tensile strains to lateral displacements, even when using the modified plastic hinge method which more closely resembles the measured strain-displacement relationship. As shown in Figure 8.24,
the strain-based Eqn 8.1 model combined with the modified hinge method and the Feng (2013) approach with the strain-displacement relationships from the modified hinge method produced the most accurate peak tensile strain predictions. These methods, however, produce conservative peak tensile displacement predictions due to smaller level of, but still present, conservatism from the modified plastic hinge method. The drift-based Eqn 8.13 does not need a plastic hinge method, since it is a direct calculation rather than a translation of a strain to displacement.

Figure 8.24 Comparison of Measured Peak Tensile Strains Prior to Bar Buckling and Result of Feng (2013) Method
Figure 8.25  Tensile and Compressive Stain Relationship at Same Displacement, Consistent with Separate Goodnight et al. Lpr Hinge Lengths

Figure 8.26  Bar Buckling at Intersection Point using Goodnight et al. (2014) Hinge Length Expressions
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Figure 8.27 Tensile Strain-Displacement for Test 9 with Moment-Curvature Prediction

Figure 8.28 Cumulative Probability Distribution for the (Measured / Predicted) Peak Tension Strains Prior to Bar Buckling in the Goodnight et al. Dataset
Figure 8.29 Comparison of Measured Peak Tensile Displacement Prior to Bar Buckling and Result of Feng (2013) Method

Figure 8.30 Cumulative Probability Distribution for the (Measured / Predicted) Peak Tension Displacement Prior to Bar Buckling in the Goodnight et al. Dataset
8.11 Bar Buckling Predictions for the Combined Berry (2006) and Goodnight et al. Dataset

In the following section, the accuracy of predicting the peak tensile displacement prior to bar buckling after reversal of load for columns in the combined Berry (2006) and Goodnight et al. datasets is explored. The Berry (2006) dataset contained 36 modernly detailed bridge columns which had reported bar buckling observations in literature, Table 8.3. The Goodnight et al. dataset included 23 columns with 44 extreme fiber bar buckling observations, Table 8.2. The 23 specimens are from Tests 8-30 which were constructed at NCSU and utilized the Optotrak instrumentation method with direct application of target markers to the surface of longitudinal bars. For this combined dataset, bar buckling predictions were made with the following techniques: (1) peak tension strain from Eqn 8.1 translated to a displacement using the modified hinge method, (2) Berry (2006) drift-based bar buckling Eqn 8.12, (3) drift-based Eqn 8.13 with coefficients fit to the combined dataset, (4) Feng (2013) multi-linear regression with the PK (2007) hinge method, and (5) Feng (2013) approach with strain-displacement relationships from the modified hinge method. A comparison of the accuracy of these methods appears in Figure 8.31, and in the statistics below.

* Mean, Coefficient of Variation, and Root Mean Squared Error

(1) mean \left( \frac{\Delta_{\text{Measred in Combined Dataset}}^{bb}}{\Delta_{\text{Cumbia at } \varepsilon_{\text{Eqn 8.1 with Modified Lpr}}^{bb}}} \right) = 1.21, COV = 0.220, RMSE = 0.218

(2) mean \left( \frac{(\Delta/L)^{bb}_{\text{Measred in Combined Dataset}}}{(\Delta/L)^{bb}_{\text{Berry (2006) Eqn 8.12}}} \right) = 0.93, COV = 0.228, RMSE = 0.286

(3) mean \left( \frac{(\Delta/L)^{bb}_{\text{Measred in Combined Dataset}}}{(\Delta/L)^{bb}_{\text{Combined Dataset Eqn 8.13}}} \right) = 1.09, COV = 0.241, RMSE = 0.200
(4) \( \text{mean} \left( \frac{\Delta_{\text{Measured in Combined Dataset}}^{bb}}{\Delta_{\text{Cumibia at } \varepsilon_{Feng}^{bb} (2013) \text{ with } PCK (2007) \text{ Lp}}} \right) = 1.12, COV = 0.234, RMSE = 0.202 \)

(5) \( \text{mean} \left( \frac{\Delta_{\text{Measured in Combined Dataset}}^{bb}}{\Delta_{\text{Cumibia at } \varepsilon_{Feng}^{bb} (2013) \text{ with Modified Lpr}}} \right) = 1.28, COV = 0.266, RMSE = 0.247 \)

Figure 8.31 Cumulative Probability Distribution for the (Measured / Predicted) Peak Tension Displacement Prior to Bar Buckling in the Combined Berry (2006) and Goodnight et al. Datasets

Cheok and Stone (1989) tested two full-scale, circular, spirally reinforced columns with constant axial load and applied quasi-static cyclic lateral displacement history. The performance of the two full-scale columns was later compared to 1/6 scale model tests. The first column, NIST Full Scale Flexure in Figure 8.32, had a 5ft diameter, 30ft cantilever length, longitudinal steel content of 2%, volumetric steel ratio of 0.6%, and a constant axial load of 6.9%. The second column, NIST Full Scale Shear in Figure 8.33, had a 5ft diameter, 15ft cantilever length, longitudinal steel content of 2%, volumetric steel ratio of 1.5%, and a constant axial load of 7.1%.

For the NIST Full Scale Flexure column, bar buckling was observed after reversal from 538mm. As summarized in the table, the predictive techniques produced the following result: (1) 511.4mm, (2) 431.7mm, (3) 501.8mm, (4) 598.1mm, (5) 488.3mm, and (6) 499.7mm.

![Figure 8.32 Bar Buckling Predictions Applied to NIST, Full Scale Flexure Specimen from Cheok and Stone (1989), Note: The Force-Deformation Predictions Include P-Delta Effects](image)
For the NIST Full Scale Shear column, bar buckling was observed after reversal from 285mm. As summarized in the table, the predictive techniques produced the following result: (1) 266.7mm, (2) 209.6mm, (3) 304.5mm, (4) 324.3mm, (5) 350.2mm, and (6) 332.2mm.

![Figure 8.33 Bar Buckling Predictions Applied to NIST, Full Scale Shear Specimen from Cheok and Stone (1989), Note: The Force-Deformation Predictions Include P-Delta Effects](image)

<table>
<thead>
<tr>
<th>Cheek and Stone (1989) NIST, Full Scale Shear</th>
<th>Strain-Displacement Input Values</th>
<th>Reported Damage Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter: 1520.0 mm</td>
<td>$f_{yh}$ 435 MPa</td>
<td>Conc. Crushing: N/A mm</td>
</tr>
<tr>
<td>Cover to Longitudinal Bars: 69.8 mm</td>
<td>$f_y$ 475 MPa</td>
<td>Sig. Spalling: 142 mm</td>
</tr>
<tr>
<td>Number of Longitudinal Bars: 25</td>
<td>$f_u$ 665 MPa</td>
<td>Bar Buckling (bb): 285 mm</td>
</tr>
<tr>
<td>Diameter of Longitudinal Bars: 43.0 mm</td>
<td>$f'c$ 34.3 MPa</td>
<td>Bar Fracture: 356 mm</td>
</tr>
<tr>
<td>Diameter of Transverse Steel: 19.1 mm</td>
<td>$d_{bl}$ 43 mm</td>
<td>Spiral Fracture: 356 mm</td>
</tr>
<tr>
<td>Spacing of Transverse Steel: 54.0 mm</td>
<td>$p_l$ 0.02</td>
<td></td>
</tr>
<tr>
<td>Axial Load: 4450.0 kN</td>
<td>$p_s$ 0.015</td>
<td></td>
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<tr>
<td>Concrete Compressive Strength: 34.30 MPa</td>
<td>$P/\left(F_{c}^{*} A_g\right)$ 0.071</td>
<td></td>
</tr>
<tr>
<td>Long Steel Yielding Stress: 475.00 MPa</td>
<td>Length 4570 mm</td>
<td></td>
</tr>
<tr>
<td>Long Steel Max. Stress: 665.00 MPa</td>
<td>Dia 1520 mm</td>
<td></td>
</tr>
<tr>
<td>Transverse Steel Yielding Stress: 435.00 MPa</td>
<td>$L_{sp}$ 392.85 mm</td>
<td></td>
</tr>
<tr>
<td>Member Length: 4570.0 mm</td>
<td>$k$ 0.08 PCK eqn</td>
<td></td>
</tr>
<tr>
<td>Single Bending, Uniaxial</td>
<td>$L_{pr,comp}$ 731.2 mm</td>
<td></td>
</tr>
<tr>
<td>PCK (2007) Plastic Hinge Length: 899 mm</td>
<td>$L_{pr,tension}$ 1871.2 mm</td>
<td></td>
</tr>
<tr>
<td>PCK (2007) Strain Penetration Length: 449 mm</td>
<td>$\phi' = 2.698 \times 10^{-6} \text{1/mm}$</td>
<td></td>
</tr>
<tr>
<td>Longitudinal Steel Ratio: 0.020</td>
<td>$M_{y} = 8906.96 \text{kN\cdot m}$</td>
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</tr>
<tr>
<td>Transverse Steel Ratio: 0.015</td>
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<td></td>
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<tr>
<td>Axial Load Ratio: 0.071</td>
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</tbody>
</table>

**Figure 8.33 Bar Buckling Predictions Applied to NIST, Full Scale Shear Specimen from Cheok and Stone (1989), Note: The Force-Deformation Predictions Include P-Delta Effects**
8.13 Compressive Strain at Cover Concrete Crushing

For Tests 8-30, the measured compressive strains in the extreme fiber bars at the peak of the cycles where cover concrete crushing was observed were used to develop two empirical expressions. The first, Eqn 8.19, is a single value expression developed based on minimizing the sum of squared error between the prediction and the measured strain. The observed cover crushing behavior was found to be influenced by the amount of confinement steel provided in the column, as shown in Figure 8.34. Columns with additional confinement steel had larger measured compressive strains at the peak of the cycle where cover crushing was observed. This relationship was used to formulate a second empirical expression to predict the compressive strain at cover concrete crushing, Eqn 8.20. The results of the two equations are compared with the measured compressive strains at the peak of the cycle were cover concrete crushing was observed in Figure 8.35.

Tests within the Goodnight et al. dataset may not be the best gage for assessing limit states related to the cover concrete due to the blockouts installed during construction for instrumentation with the Optotrak system. For columns with the full cover blockout the first sign of concrete flaking in the compression zone was taken as the cover concrete crushing observation. The current serviceability concrete compressive strain of 0.004 is still recommended for the design of new structures, since inevitably the measured compressive strain at the peak of a cycle exceeds the raw value which initiated the cover crushing.

\[ \varepsilon_{\text{crushing}}^{\text{cover}} = 0.00475 \]  
\[ \varepsilon_{\text{crushing}}^{\text{cover}} = 197\rho_s \frac{f_y}{E_s} \]

Compression Strain at Cover Crushing which Minimizes the Sum of Squared Error in Prediction  
Compression Strain at Cover Crushing in Terms of Transverse Volumetric Steel Ratio  

Eqn 8.19  
Eqn 8.20
The current damage control concrete compressive strain limit is defined by the Mander et al. (1988) ultimate concrete compression strain. This expression was developed based on an energy balance between the core concrete dilation and the confinement provided by the transverse steel for a column subjected to uniform compression. Comparatively little is known about the relationship between compressive demand and confinement in inelastic
flexural members such as bridge columns which have a strain gradient and a fanned compressive strut pattern emanating from the compressive toe region of the column. Experimental results within the Goodnight et al dataset imply that after initial yielding of the confinement steel, localization of compressive demand can occur over several spiral layers. Measured compressive strains in this region have exceeded the Mander et al. (1988) ultimate concrete compressive strain without resulting in fracture of confinement steel. For many of these tests, measurable deformation could be observed in the recorded longitudinal and spiral strain hysteresis prior to the visible bar buckling observation. The distribution of spiral strains measured both around the circumference of the column and over multiple layers was found to impact the observed bar buckling behavior. Localized spiral demands led to increased levels of measured compressive strain and early buckling of longitudinal reinforcement.

Since the stiffness of the transverse reinforcement restraining the bar from buckling is linked to its degree of inelasticity, perhaps an intermediate limit state of initial spiral yield in the confinement region is needed. Initial spiral yield is termed as an intermediate limit state because it marks the point where localization of compressive demand begins. This localization led to measurable deformation prior to visible bar buckling. An understanding of the variables which influence the initial yielding behavior is helpful. Higher levels longitudinal steel content resulted in lower compressive strains measured at the peak of the cycle where transverse steel in the confinement region initially yielded, Figure 8.36. The additional restraint demands required to maintain stability of the longitudinal reinforcement reduces the available component left over for core concrete confinement. The spiral yielding observation occurred at higher values of measured compressive strain for experiments with larger transverse steel yield stress, Figure 8.36. The measured spiral yielding behavior was expected to be influenced the same variables which impact confinement. Nether transverse volumetric steel ratio or the magnitude of the computed Mander (1988) ultimate concrete compressive strain influenced the measured spiral yielding behavior is Tests 8-30, Figure 8.37. On average, the initial spiral yielding observation was observed at 75% of the computed Mander (1988) ultimate concrete compressive strain.
Two empirical equations were made to predict the compressive strain at spiral yielding, Eqn 8.21 and Eqn 8.22. The first, Eqn 8.21, is a single valued expression formulated based on minimizing the sum of squared error between the prediction and measured result. The second expression, Eqn 8.22, was created in the same manner, but includes the influence of steel content and spiral yield stress which were found to influence the measured behavior. A comparison of the results for these two equations is shown graphically in Figure 8.38. Alternatively, the accuracy of Eqn 8.22 in predicting the measured compressive strains at the peak of the cycle where spiral yielding was observed can be viewed using a cumulative probability distribution in Figure 8.39. The mean and coefficient of variation for this strain comparison appears below. The compressive strains at spiral yield from Eqn 8.22 were translated to lateral displacements using the compressive strain-displacement relationship from the Modified Lpr Plastic Hinge Method. The cumulative probability distribution in Figure 8.40 cam be used to gage the accuracy and conservatism of displacements evaluated at the Eqn 8.22 spiral yield strain. A measured to predicted displacement ratio lower than one implies that the mean value for the computed spiral yield displacements is conservative.

\[
\varepsilon_{y\text{ielding}}^{\text{spiral}} = 0.0124 \quad \text{Compression Strain at Spiral Yielding} \quad \text{Eqn 8.21}
\]

\[
\varepsilon_{y\text{ielding}}^{\text{spiral}} = 0.009 - 0.3 \frac{A_s}{A_g} + 3.9 \frac{f_y}{E_s} \quad \text{Compression Strain at Spiral Yielding} \quad \text{Eqn 8.22}
\]

\[
\text{mean} \left( \frac{\varepsilon_{\text{Measured}}^{\text{spiral yield in Goodnight et al. Dataset}}}{\varepsilon_{\text{Eqn 8.22}}^{\text{spiral yield}}} \right) = 1.06, COV = 0.167
\]

\[
\text{mean} \left( \frac{\Delta_{\text{Measured}}^{\text{spiral yield in Goodnight et al. Dataset}}}{\Delta_{\text{Cumbia at \varepsilon_{\text{Eqn 8.22}}^{\text{spiral yield}}} \text{ with Modified Lpr}}} \right) = 0.94, COV = 0.186
\]
Chapter 8: Performance Strain Limits for Circular Bridge Columns

Figure 8.36 Impact of Long. Steel and Spiral Strength on Comp. Strain at Spiral Yield

Figure 8.37 Impact of Mander $\varepsilon_{cu}$ and Trans. Steel on Comp. Strain at Spiral Yield

Figure 8.38 Result of Compression Strain at Spiral Yield Eqn 8.21 and Eqn 8.22
Figure 8.39  CDF for Comp. Strain at Spiral Yield Eqn 8.22 for Goodnight Dataset

Figure 8.40  CDF for Modified Lpr Drift at Eqn 8.22 Strain for Goodnight Dataset
8.15 Residual Crack Widths

The current serviceability steel tensile strain limit of $\varepsilon_s = 0.015$ represents the tensile strain at the peak of a given cycle that is expected to result in 1mm residual crack widths measured at zero lateral force. The residual crack width of 1 mm represents the limit at which epoxy injection may be needed to prevent corrosion of internal reinforcing steel. Residual crack widths were not directly measured in experiments from the Goodnight et al. dataset. A process through which the measured peak crack widths of a given cycle were used to approximate the residual crack widths using the measure extreme fiber reinforcement strains is described below. It is important to note that this process was only utilized in Tests 8-18, which had the vertical cover concrete blockout strips over extreme fiber reinforcement. Tests 19-30 had a full cover concrete blockout, which prevented accurate measurements of crack widths.

The measured peak tensile strains and their associated measured crack widths at cycle peaks prior to the cover concrete crushing observation in Tests 8-18 are shown in Figure 8.41. The residual tension strain was taken as the strain measured in the extreme fiber bar at zero displacement during the reversal from a cycle peak. The relationship between measured peak and residual tension strains appears in the right half of Figure 8.41. If the ratio of the peak tension strain to the residual tension strain is assumed to be the same as the ratio between the peak crack width and the residual crack width, then the residual crack widths can be computed. The resulting relationship between peak tension strain and computed residual crack widths appear in the left half of Figure 8.42. Using this method, the computed residual crack widths never exceed the threshold value of 1mm which was taken as the steel tensile serviceability limit state. Further measurements of crack widths were prevented by cover concrete crushing on each side of the specimen before the computed residual crack widths reached 1 mm. A residual crack width of 0.5 mm would be expected to occur after reversal from a peak tensile strain of 0.015, based on the linear relationship presented in Figure 8.42.
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Figure 8.41 Peak Tension Strains and (Left) Peak Crack Width or (Right) Residual Crack Width Measured at Zero Displacement during the Following Reversal

Figure 8.42 (Left) Peak Tensile Strains and Calculated Residual Crack Widths, (Right) Comparison of Peak Tensile and Crack Width to Residual Tensile and Crack Width
8.16 Conclusion

To satisfy the aims of performance based design, levels of damage which interrupt the serviceability of the structure or require more invasive repair techniques must be related to engineering criteria. For reinforced concrete flexural members such as bridge columns, concrete compressive and steel tensile strain limits are very good indicators of damage. An experimental study was carried out to assess the performance of thirty circular, well-confined, bridge columns with varying lateral displacement history, transverse reinforcement detailing, axial load, aspect ratio, and longitudinal steel content. A key feature of the experiments is the high fidelity strain data obtained through the use of an optical 3D position measurement system. Previous performance strain limit recommendations from Kowalsky (2000) have been revisited in light of the data collected in the experiments.

Serviceability limit states represent the point at which repair is necessary, interrupting the serviceability of the structure, but not posing a safety concern. The current serviceability concrete compressive strain limit of $\varepsilon_c = 0.004$ was found to be conservative, but is ultimately recommended since the measured compressive strains at cover concrete crushing come from the peak of the cycle where cover crushing was observed. Within the Goodnight et al. dataset, larger measure compressive strain at cover crushing were observed in columns with higher levels of confinement steel. The current serviceability steel tensile strain limit of $\varepsilon_s = 0.015$ represents the tensile strain at the peak of a given cycle that is expected to result in 1 mm residual crack widths measured at zero lateral force. The residual crack width of 1 mm represents the limit at which epoxy injection may be needed to prevent corrosion of internal reinforcing steel. Although residual crack widths were not directly measured in experiments from the Goodnight et al. dataset, approximations for the residual crack widths were made based on the measured peak crack widths and recorded bar strains. For columns in the Goodnight et al. dataset, the calculated residual crack widths never exceeded 1 mm before cover concrete crushing occurred, preventing further crack width measurements. Ultimately no further recommendations on the serviceability steel tensile strain limit are
Experimental results within the Goodnight et al. dataset suggest that after initial yielding of the confinement steel, localization of compressive demand can occur over several spiral layers. Measured compressive strains in this region have exceeded the Mander et al. (1988) ultimate concrete compressive strain without resulting in fracture of confinement steel. For many of these tests, measurable deformation could be observed in the recorded longitudinal and spiral strain hysteresis prior to the visible bar buckling observation. Currently there is not an intermediate strain limit between serviceability and damage control which is related to a change in compressive behavior that results due to confinement steel entering the inelastic range. To gain a better understanding of the behavior, trends were analyzed in measured compressive strains at the initial spiral yield observation in the Goodnight et al. dataset. Specimens with higher levels of longitudinal steel content had smaller measured compressive strains at the spiral yielding observation since a larger component of its capacity was utilized for bar restraint. An empirical expression for intermediate concrete compressive strain limit related to initial yielding of confinement steel was developed, Eqn 8.22. This expression provided a mean measured/predicted compression steel at spiral yield of 1.06 with a coefficient of variation of 0.167. When converted to lateral displacements using the compressive strain-displacement relationship for the Modified Lpr Hinge Method, the resulting mean measured/predicted displacement was 0.94 with a coefficient of variation of 0.186.

The damage control limit state represents the limit of economical repair, where past this point repair may be uneconomical or unfeasible. The current damage control concrete compressive strain limit is defined by the Mander et al. (1988) ultimate concrete compression strain. This expression was developed based on an energy balance between the core concrete dilation and the confinement provided by the transverse steel for a column subjected to uniform compression. In the Goodnight et al. dataset, severe yielding of the transverse steel resulted in localized compressive demand. In this region, measured compressive strains have
exceeded the Mander et al. (1988) ultimate concrete compressive strain without resulting in confinement steel fracture as the energy balance approach would imply. The current damage control concrete compressive strain limit is still recommended because it is related to significant levels of damage in the core concrete which would coincide with observed measurable deformation and potential buckling of reinforcing bars.

The current damage control steel tensile strain limit of $\varepsilon_s = 0.06$ is related to the peak tensile strain that is expected to initiate bar buckling in longitudinal reinforcement upon reversal of load. Measured peak tensile strains prior to bar buckling in the Goodnight et al. dataset suggest that this value is too large. An empirical expression for the peak tensile strain prior to bar buckling, Eqn 8.1, was formulated based on measured trends in the dataset. Additional confinement steel increased the measured tensile strains prior to bar buckling while higher axial load ratio reduced the peak tensile strain in the dataset. For columns in the Goodnight et al. dataset, Eqn 8.1 produced a mean measured/predicted peak tensile strain of 1.05 with a coefficient of variation of 0.199. When the result of Eqn 8.1 is translated to a top column displacement using the tensile-strain displacement relationship from the Modified Lpr Hinge Method produced a mean measured/predicted displacement of 1.17 with a coefficient of variation of 0.157.

Berry (2006) utilized a subset of the PEER Column Performance Dataset to make an empirical drift-based expression to predict the peak drift prior to bar buckling, Eqn 8.12. The bridge column dataset utilized by Berry (2006) appears in Table 8.3. When the Berry (2006) Eqn 8.12 is applied to the Goodnight et al. dataset, the resulting mean measured/predicted displacement at bar buckling is 0.92 with a coefficient of variation of 0.171.

Feng (2013) proposed a series of equations to describe bar buckling behavior observed in finite element analysis. The influence of the following behaviors were included in the analysis: (1) dilation of core concrete under compression, (2) restraint provided by individual spiral layers which can go inelastic, and (3) development of the longitudinal bar into the adjoining member. The analysis resulted in a multi-linear regression model, Eqn 8.14 through Eqn 8.18, which forms a boundary for tensile versus compressive strain relationship.
Bar buckling strains predicted using this method were unconservative. The method resulted in a mean measured/predicted peak tensile strain proceeding bar buckling of 0.81 with a coefficient of variation of 0.224. This unconservative peak tensile strain when combined with the conservative tensile-strain displacement relationship from the Priestley, Calvi, and Kowalsky (2007) plastic hinge method resulted in accurate peak displacements proceeding bar buckling.

Finally, all of the predictive bar buckling methods previously described where employed to predict bar buckling in bridge columns from the Berry (2006) dataset. It became apparent that a drift-based empirical expression, similar to the original expression derived by Berry (2006), could be developed utilizing the combined dataset including the experiments from Goodnight et al. The resulting empirical drift-based bar buckling expression, Eqn 8.13, produced a mean measured/predicted drift prior to bar buckling in the combined dataset of 1.09 and coefficient of variation of 0.241. This was the most accurate predictive method for bath buckling in the combined dataset, but this is since the same dataset was used to find the empirical parameters. The strain-based bar buckling expression, Eqn 8.1, combined with the tensile strain-displacement relationships from the Modified Lpr Hinge Method produced a mean measured/predicted bar buckling displacement of 1.21 with a coefficient of variation of 0.220. Finally the Feng (2013) multi-linear regression method utilizing strain-displacement relationships from the PCK (2007) Lp Hinge Method produced a mean measured/predicted displacement proceeding bar buckling of 1.12 and a coefficient of variation of 0.234. Of the methods studied, these three bar buckling methods produced the most accurate results. While the drift-based approach is the easiest to employ in design, it is also limited to specific column configurations. The strain-based approaches do not suffer from this limitation, but require appropriate equivalent curvature distributions for the respective methods.
Chapter 9: Weldability of A706 Reinforcing Steel

An experimental study was carried out to assess the performance of thirty circular, well-confined, bridge columns with varying lateral displacement history, transverse reinforcement detailing, axial load, aspect ratio, and longitudinal steel content. A key feature of the experiments is the high fidelity strain data obtained through the use of an optical 3D position measurement system. Over the course of the study three different techniques were used to monitor material strains with the Optotrak system: (1) a single positon monitor with steel post extensions, (2) two position monitors with vertical cover concrete blockouts, and (3) three position monitors with a complete cover blockout.

9.1 Test 7 and Weldability of A706 Reinforcing Steel

The single position monitor method was used for the first seven specimens, which were ultimately excluded from design recommendations in this report. This instrumentation technique, shown in Figure 9.3, had target markers applied to the ends of tack welded steel posts. Vertical cover concrete blockout strips over six extreme fiber bars were created by blocking out the cover concrete with insulation foam during casting. The measured strains from the target markers at the ends of the post extensions suffer from the same issues of the traditional curvature rod method. Strains are not measured at the location of interest; therefore the recorded values are influenced by rotations of the rods themselves. This was however not the biggest problem with this instrumentation method, since it was found that the ASTM A706 longitudinal steel utilized in Tests 7-12 had a reduced strain capacity under the influence of the surface tack welds. Ultimately, the single position monitor technique was abandoned, and Tests 8-30 utilized multiple position monitors with direct application of target markers to the reinforcing steel.
Section 9.1.1 Test 7 Experimental Observations and Comparison to Test 9

In the following section, experimental observations for nominally identical Tests 7 and 9 are compared. The 24” diameter bridge columns contained 16 #6 A706 bars for longitudinal reinforcement ($A_{st}/A_g = 1.6\%$) and a #3 A706 spiral at 2” pitch ($4A_{sp}/(D's) = 1\%$). Both columns were subjected to symmetric three-cycle-set load histories, Figure 9.2, and a constant compressive axial load of 170 kips ($P/(f_c' A_g) = 5.5\%$). The cantilever specimens had an aspect ratio ($L/D = 4$). The only difference is that column Test 7 utilized a single Optotrak position monitor and tack welded steel post extensions, while Test 8 had multiple position monitors and direct application of target markers to the reinforcing steel. A comparison of the resulting force versus deformation response for the two tests appears in Figure 9.1. In Test 7 rupture of longitudinal reinforcement occurred prior to longitudinal bar buckling. In the other cyclically loaded experiments, bar buckling always occurred prior to rupture. Three south reinforcing bars fractured during pull cycles of displacement ductility six, and two north reinforcing bars fractured during the first push cycle of displacement ductility eight.

The first yield force for the tested material and geometric properties was determined using moment curvature analysis ($F_y' = 46.85\text{ kips with } f_c' = 6545\text{ psi}$). The initial part of the symmetric three cycle set load history contains reversals of loading at $\frac{1}{4} F_y'$, $\frac{1}{2} F_y'$, $\frac{3}{4} F_y'$, and $F_y'$. After the specimen has reached the first yield force in each direction, the first yield displacement is obtained as an average ($\Delta_y' = 0.62''$). The equivalent yield displacement, used to determine the displacement ductility levels ($\mu_{\Delta 1} = 1 * \Delta_y$), is then calculated as $\Delta_y = \Delta_y' (M_n/M_y') = 0.83''$. The symmetric three-cycle-set load history then resumes with three complete cycles of loading at each displacement ductility level shown in Figure 9.2.

Concrete cracking occurred during push and pull cycles of $\frac{3}{4} F_y'$ on the north and south sides of the specimen respectively. The first signs of cover concrete crushing were observed at ($\mu_{1.5}^{+3} = 1.26''$) on the south side of the specimen, Figure 9.4. Cover concrete on the north
side of the specimen did not begin to crush until \( \mu_2^{-1} = -1.66'' \). Additional cracks formed and the extent of cover concrete crushing increased while the test proceeded without incident through displacement ductility four, Figure 9.4. During the first pull cycle of displacement ductility six \( \mu_6^{-1} = -4.97'' \), south reinforcing bar S2 fractured, resulting in a significant loss in strength. This observation was unexpected, since fracture never occurred prior to visible buckling in any of the cyclically loaded experiments and the strain demand should not have exceeded the strain at maximum stress for the plain reinforcing bars. Also, Bar S2 is adjacent to the extreme fiber bar and should experience smaller strain demands. Ultimately, this early fracture is attributed to embrittlement of the A706 reinforcing steel through the heat effects of welding the steel post extensions utilized in the instrumentation system. This was also unexpected, since the reinforcing steel in the first six specimens was unaffected by the surface tack welds.

The load history was continued to evaluate the degradation behavior of the specimen, and to see if a similar observation would occur on the north side of the specimen. The extreme fiber south reinforcing Bar S3 buckled during the third push cycle of displacement ductility six \( \mu_6^{+3} = 4.99'' \). Since Bar S2 fractured prior to this observation, the peak tensile strain has little relevance due to the distortion of equilibrium in the cross section. Bar S3 was the first and only bar to buckle during the symmetric three-cycle-set load history which highlights the change in failure mechanism due to the weld process lowering the strain capacity of the steel.

During the third pull cycle of ductility six \( \mu_6^{-3} = -4.98'' \), Bars S3 and S4 ruptured, resulting in a significant loss in strength. Three cycles at displacement ductility six were concluded without buckling or rupture of north reinforcement. North Bars N2 and N3 ruptured during the first push cycle of ductility eight \( \mu_8^{+1} = 6.65'' \). The fractures occurred roughly ten inches above the footing at the location of the largest crack throughout the test. With large losses in strength on both sides of the column the test was concluded.
Figure 9.1 Tig-Welded Steel Posts Reduced Deformation Capacity in One Column

Figure 9.2 Symmetric Three-Cycle-Set Load History for Tests 7 and 9
Figure 9.3  Single Optotrak Method Utilizing Post Extensions (Ultimately Abandoned)

Figure 9.4  (Left) Crushing at $\mu_{1.5}^{+3}$ on South Side and (Right) Crack Distribution at $\mu_{4}^{-3}$
9.1.2 A706 Reinforcement Properties and Weldability for Tests 1-6 and 7-12

Additional material tests were carried out on A706 longitudinal reinforcement from Tests 1-6 and 7-12 to determine why one batch had reduced strain at maximum stress, but the other was not influenced by the surface tack welds utilized in the instrumentation technique. A summary of material properties for seven plain reinforcing bars and four bars with tack welded posts at 2” spacing appears in Table 9.1 for Tests 7-12. For each tension test, the following data appears in the table: (1) stress and strain at yield; (2) stress and strain at the beginning of strain hardening (end of yield plateau); and (3) stress and strain at maximum stress (ultimate condition). The average strain at maximum stress for the plain bars was 0.1331, while the ultimate strain was reduced for the tack welded bars to 0.0938. The effects of welding on this batch of steel are evident in the stress-strain response for the fourth bar test, Figure 9.6, which had a strain at maximum stress of only 0.0731. The material tests confirm the observations of early fracture of longitudinal reinforcement in Test 7. To highlight the fact that welding was not an issue for reinforcing bars in Tests 1-6, additional
material tests on plain reinforcing bars and steel with both 2” and 4” tack welded post spacings were tested. The strain at maximum stress for the three groups of bar tests remained comparable, without a significant reduction in the presence of welds, Table 9.2.

To further investigate the impact of welding on the longitudinal reinforcement, the metallurgical content of both batches were compared to allowable limits in the ASTM A706 standard. All of the welded post specimens utilized the same tungsten inert gas (TIG) welder technique, filler material, and maximum input amperage that controls the level of heat applied when the pedal is fully compressed. The duration of applied heat should not have changed significantly between testing series. Relevant sections of the ASTM standard for A706 reinforcing bars appear in Figure 9.7. Section 1.5 of the ASTM standard states that the chemical composition and carbon equivalent of A706 steel are limited to enhance the weldability of the steel. The standard recommends structural welding procedures that appear in AWS D1.4/D1.4M, the structural welding code for reinforcing steel. The tested material properties and chemical composition from heat analysis obtained from the mill test certification for all of the A706 steel was within the allowable range forth both batches of steel, Figure 9.7.

9.1.3 Conclusion

The failure mechanism for one column experiment was shifted from fracture of previously buckled bars to early brittle fracture of reinforcement for a batch of steel with surface tack welded posts utilized for instrumentation. The stress and strain at initial yielding and strain hardening were not influenced by the surface tack welds. The maximum stress in the presence of the tack welds was not influenced, but the strain capacity was reduced in the presence of welds. In large bridge columns, individual circular butt welded hoops are commonly utilized instead of spiral reinforcement. This test highlights the need to quality control of welded A706 reinforcement. The strain at maximum stress, and not the value of the stress itself, is the critical parameter in defining the influence of welding on the reinforcement behavior.
Table 9.1 Reinforcement Material Properties for Tests 7-12, Influenced by Welding

<table>
<thead>
<tr>
<th>Test Series</th>
<th>Bar Number</th>
<th>$\varepsilon_y$</th>
<th>$f_y$ (ksi)</th>
<th>$\varepsilon_{h}$ (hardening)</th>
<th>$f_h$ (ksi)</th>
<th>$\varepsilon_{u}$ (max stress)</th>
<th>$f_u$ (ksi)</th>
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</thead>
<tbody>
<tr>
<td>Plain Bar</td>
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<td>67.0</td>
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<th>$\varepsilon_{h}$ (hardening)</th>
<th>$f_h$ (ksi)</th>
<th>$\varepsilon_{u}$ (max stress)</th>
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Figure 9.6 Longitudinal Steel from Tests 7-12 with Reduced Strain at Maximum Stress
## Table 9.2 Reinforcement Material Properties for Tests 1-6, Not Influenced By Welds

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<tr>
<th>Test Series</th>
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<th>$\varepsilon_h$ (hardening)</th>
<th>$f_h$ (ksi)</th>
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<td>89.7</td>
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### Test Series Averages

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<th>$f_y$ (ksi)</th>
<th>$\varepsilon_h$ (hardening)</th>
<th>$f_h$ (ksi)</th>
<th>$\varepsilon_u$ (max stress)</th>
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<tr>
<td>Plain Bar Averages</td>
<td>0.00220</td>
<td>63.7</td>
<td>0.0143</td>
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<td>Welded Posts at 4”</td>
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<td>0.0131</td>
<td>63.2</td>
<td>0.1201</td>
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</tr>
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<td>0.0143</td>
<td>65.5</td>
<td>0.1105</td>
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</table>

---

1.5 Welding—This specification limits chemical composition (6.2) and carbon equivalent (6.4) to enhance the weldability of the material. When steel is to be welded, a welding procedure suitable for the chemical composition and intended use or service should be used. The use of the latest edition of AWS D1.4/D1.4M is recommended. This document describes the proper selection of the filler metals, preheat/interpass temperatures, as well as, performance and procedure qualification requirements.

6.2 The chemical composition as shown by heat analysis shall be limited by the following:

\[
\begin{align*}
\text{Element} & & \text{max., %} \\
\text{Carbon} & & 0.30 \\
\text{Manganese} & & 1.50 \\
\text{Phosphorus} & & 0.035 \\
\text{Sulfur} & & 0.045 \\
\text{Silicon} & & 0.50
\end{align*}
\]

6.4 The heat analysis shall be such as to provide a carbon equivalent (C.E.) not exceeding 0.55% as calculated by the following formula:

\[
\text{C.E.} = \%\text{C} + \frac{\%\text{Mn}}{6} + \frac{\%\text{Cu}}{40} + \frac{\%\text{Ni}}{20} + \frac{\%\text{Cr}}{10} - \frac{\%\text{Mo}}{50} - \frac{\%\text{V}}{10}
\]

Figure 9.7 ASTM A706 Specification for Reinforcing Steel
Table 9.3  Mill Specification Reports for Longitudinal Steel from Tests 1-6 and 7-12

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<thead>
<tr>
<th>Tests</th>
<th>C</th>
<th>Mn</th>
<th>P</th>
<th>S</th>
<th>Si</th>
<th>Cu</th>
<th>Ni</th>
<th>Cr</th>
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<td>0.21</td>
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<table>
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<tr>
<th>Tests</th>
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<th>V</th>
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<td>0.001</td>
<td>0.001</td>
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</tr>
<tr>
<td>7-12</td>
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<td>N/A</td>
<td>0.010</td>
<td>N/A</td>
<td>N/A</td>
<td>0.490</td>
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</table>
Chapter 10: Summary of Column Tests 1-6

The initial six columns for the Load History research program were constructed by a local contractor. Although the need for accurate detailing was expressed and tolerances were specified, the resulting specimens had discrepancies in transverse steel spacings in the plastic hinge region. This influenced the restraint of longitudinal reinforcing bars and significantly impacted the performance of these specimens. Furthermore, these tests utilized the single position monitor and welded steel post extension instrumentation technique discussed in Chapter 9. The strain capacity reinforcing steel was not influenced by the surface tack welded posts, but the technique produced less reliable strains when compared to the use of multiple position monitors and direct application of target markers to the longitudinal reinforcement. The strains measured at the ends of the welded post extensions are influenced by rotations of the posts themselves which arise due to the curvature gradient over the gage length. For this reason, as well as the detailing errors mentioned, the occurrence of limit states in Tests 1-6 were not included in the formulation of design recommendations. A brief summary of each test is included in this section to highlight the observed behavior.

10.1 Test Setup and Instrumentation for Specimens 1-6

The 24” diameter bridge columns contained 16 #6 A706 bars for longitudinal reinforcement ($A_{st}/A_g = 1.6\%$) and a #3 A706 spiral at 2” pitch ($4A_{sp}/D's = 1\%$). The columns were subjected to a constant compressive axial load of 170 kips ($P/(f_c' A_g) \approx 5\%$). The cantilever specimens had an aspect ratio ($L/D = 4$). The main variable for the first six experiments was the lateral displacement history employed in the test. Due to the detailing errors, the spacing of transverse steel in regions of the column critical to bar buckling was also variable. In the following section, the load history for each experiment and a brief summary of the observed damage is summarized for Tests 1-6. The cross section and instrumentation system utilized in Tests 1-6 appears in Figure 10.1 and Figure 10.2. Tests 1 and 3-6 had vertical cover concrete blockout strips and tack welded steel post extensions.
Chapter 10: Summary of Column Tests 1-6

For comparison, Test 2 was conducted with a full cover concrete to verify that the blockout strips did not influence the performance of the column after initial crushing. A summary of reinforcement material properties for both longitudinal and transverse steel appears in Table 10.1. The material properties for the longitudinal reinforcement were not influenced by surface tack welds. The tested material properties for the column concrete in Tests 1-6 appear in Table 10.1. Since the columns were constructed by a contractor it is unknown why there is such disparity in the column concrete strength and associated axial load ratios under the constant applied compressive load of 170 kips.

Table 10.1 Material Properties for Tests 1-6

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<th>Longitudinal Steel</th>
<th>$\varepsilon_y$</th>
<th>$f_y$ (ksi)</th>
<th>$\varepsilon_h$ (hardening)</th>
<th>$f_h$ (ksi)</th>
<th>$\varepsilon_u$ (max stress)</th>
<th>$f_u$ (ksi)</th>
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<tbody>
<tr>
<td>Plain Bar Averages</td>
<td>0.00220</td>
<td>63.7</td>
<td>0.0143</td>
<td>63.7</td>
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<td>Welded Posts at 4”</td>
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<th>4</th>
<th>5</th>
<th>6</th>
</tr>
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<td>5.06</td>
<td>7.50</td>
<td>7.93</td>
<td>8.40</td>
<td>10.18</td>
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<tr>
<td>Axial ($P/f'_cA_g$ in %)</td>
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<td>7.4</td>
<td>5.0</td>
<td>4.7</td>
<td>4.5</td>
<td>3.69</td>
</tr>
</tbody>
</table>
Figure 10.1 Single Optotrak Method Utilizing Post Extensions (Ultimately Abandoned)

Figure 10.2 (Left) Cross Section with Blockouts and (Right) Test 2 with Full Cover
10.2 Test 1: Pushover Load History

To evaluate the tensile bar buckling mechanism, a pushover load history was utilized for Test 1. In a cyclic load history bar buckling is expected to occur after reversal from a peak tensile strain while the bar is under net elongation, but compressive stress. During the subsequent reversal while the cracks are still closing, the longitudinal reinforcement is the sole source of compression zone stability. Furthermore, the tangent modulus for the reinforcing bar is reduced during reversals from larger values of tensile strain, which influences buckling behavior. For these reasons, bar buckling was not expected to occur under direct compression alone for the well confined bridge column. As the load increased, the test was paused for observation at each of the labeled observation points below in the resulting hysteretic response, Figure 10.3. The small drops in force resulted from relaxation at observation displacements. The specimen was briefly unloaded and reloaded after displacement ductility eight.

The recorded displacement at the analytical first yield force of \( F'_y = 43.84 \text{ kips} \) was \( \Delta'_y = 0.48" \). The equivalent yield displacement, used to determine the displacement ductility levels \( \mu_\Delta = 1 \times \Delta_y \), is then calculated as \( \Delta_y = \Delta'_y \left( M_n / M'_y \right) = 0.65" \). Concrete cracking was first observed at \( \frac{3}{4} F_y \). Displacement ductility two marked the onset of cover concrete crushing, while the cracks on the tension side measured 1.25 mm in width. At the base of the column, \( \frac{1}{4} " \) cracks widths were measured at ductility twelve \( \mu_{\Delta 12} = 7.74" \). There was no loss in strength through displacement ductility eighteen \( \mu_{\Delta 18} = 11.57" \). At 12.5”, two longitudinal bars on the tension side ruptured causing an 18% loss in strength. The bars ruptured at the location of the largest crack on the tension side, Figure 10.5. The displacement was increased to ductility twenty-two \( \mu_{\Delta 22} = 14.20" \) without further strength loss or rupture of reinforcement. This concluded the monotonic portion of the test. Upon reversal of loading all of the bars previously exposed to tension buckled and a residual displacement over twelve inches was observed.
Figure 10.3 Test 1 – Hysteretic Response for Pushover Load History

Figure 10.4 Test 1 – (Left) Compressive and (Right) Tensile Sides at $\mu_{\Delta 18} = 11.57$"
10.3 Test 2: Three-Cycle-Set with Full Cover Concrete

The second test utilized a column with full cover concrete to determine the effect, if any, of longitudinal cover blockouts and welded posts used in other specimens to determine steel strains in the inelastic range. The specimen was subjected to a symmetric three-cycle-set load history, which is commonly used to evaluate the seismic performance of structural components. Since the symmetric three-cycle-set load history is considered as more severe than the demands produced by real earthquakes, the force displacement response can be used for comparison to other tests.

The first yield force for the tested material and geometric properties was determined using moment curvature analysis \( F_y' = 42.69 \text{kips for } f_c' = 5.06 \text{ksi} \). The initial portion of the symmetric three cycle set load history contains reversals of loading at \( \frac{1}{4} F_y' \), \( \frac{1}{2} F_y' \), \( \frac{3}{4} F_y' \), and \( F_y' \). After the specimen has reached the first yield force in each direction, the first...
yield displacement is obtained as an average \( \Delta'_y = 0.51" \). The equivalent yield displacement, used to determine the displacement ductility levels \( (\mu_{\Delta 1} = 1 \cdot \Delta_y) \), is then calculated as \( \Delta_y = \Delta'_y \left( \frac{M_n}{M'_y} \right) = 0.83" \). The symmetric three-cycle-set load history then resumes with three complete cycles of loading at each displacement ductility level shown in Figure 9.2. The resulting hysteretic response for Test 2 appears in Figure 10.7. A hysteretic comparison of the monotonic and cyclic response from Tests 1 and 2 is shown in Figure 10.8.

The first cracks on the north and south sides of the specimen were observed during \( \frac{3}{4} \) Fy’ push and pull cycles respectively. Cover concrete crushing was first observed on the south side of the specimen, Figure 10.9, during the first push cycle of displacement ductility three \( (\mu_{3}^{+1} = 1.72") \). The north extreme fiber bar buckled after reversal from \( (\mu_{8}^{+1} = 5.27") \), Figure 10.10. Buckled occurred over adjacent \( \approx 2.5" \) transverse steel spacings, which is larger than the specified 2” spacing in construction of the specimens. Adjacent north bars N2 and N4 buckled during ductility eight, resulting in the buckled deformation shown in Figure 10.10. The previously buckled north extreme fiber bar N3 ruptured during \( (\mu_{10}^{+1} = 6.60") \), resulting in the first notable loss in strength.

The spacing of transverse steel is critical to the location and deformation at the initiation of bar buckling. The south side of the specimen, with an average 2” spacing of transverse steel, had bar buckling delayed until \( (\mu_{10}^{+2} = 6.59") \). During \( (\mu_{10}^{+2} = 6.59") \), two additional north reinforcing bars N2 and N4 ruptured. During \( (\mu_{10}^{+3} = 6.56") \), an additional north bar fractured and an adjacent south bar buckled. During \( (\mu_{10}^{+3} = -6.77") \), three previously buckled bars on the south side of the specimen ruptured resulting in a significant loss in strength. A full cycle was completed at \( \mu_{\Delta 12} \) to evaluate further strength degradation even though significant strength loss had already occurred in both directions of loading.
Figure 10.6 Test 2 – Symmetric Three-Cycle-Set Load History

Figure 10.7 Test 2 – Hysteretic Response
Figure 10.8 Hysteretic Comparison of Monotonic Test 1 and Cyclic Test 2

Figure 10.9 Test 2 – (Left) Initial Crushing on the South Side during \( \mu_3^{+1} = 1.72'' \) and (Right) Crack Distribution at \( \mu_4^{-3} = -2.68'' \)
Figure 10.10 Test 2 – (Left) Buckling of Bar N3 during $(\mu_8^{-1} = -5.39\,\text{"})$ and (Right) Deformation in Multiple Buckled North Bars at $(\mu_8^{-3} = -5.36\,\text{"})$

Figure 10.11 Test 2 – (Left) Fracture of Previously Buckled Bar N3 during $(\mu_{10}^{+1} = 6.60\,\text{"})$ and (Right) Buckling of South Extreme Fiber Bar S3 during $(\mu_{10}^{+2} = 6.59\,\text{"})$
10.4 Test 3: Three-Cycle-Set with Cover Blockouts

The third specimen was nominally identical to the second, except vertical cover concrete blockout strips were installed over extreme fiber reinforcement during construction for the instrumentation system. The first yield force for the tested material and geometric properties was determined using moment curvature analysis \( F'_y = 44.20 \text{ kips for } f'_c = 7.50 \text{ ksi} \). The initial portion of the symmetric three cycle set load history contains reversals of loading at \( \frac{1}{4} F'_y \), \( \frac{1}{2} F'_y \), \( \frac{3}{4} F'_y \), and \( F'_y \). After the specimen has reached the first yield force in each direction, the first yield displacement is obtained as an average \( (\Delta'_y = 0.47") \). The equivalent yield displacement, used to determine the displacement ductility levels \( (\mu_{\Delta 1} = 1 * \Delta_y) \), is then calculated as \( \Delta_y = \Delta'_y \left( M_n / M'_y \right) = 0.67" \). The symmetric three-cycle-set load history then resumes with three complete cycles of loading at each displacement ductility level shown in Figure 10.12. The resulting hysteretic response for Test 3 and a comparison of the response of Test 2 appears in Figure 10.13 and Figure 10.14 respectively.

The first cracks on the south side of the specimen were observed during the \( -\frac{1}{2} F'_y \) pull cycle. Cover concrete crushing was observed on the north side of the specimen during \( (\mu_{3}^{-1} = -2.05") \), Figure 10.15. The crack distribution on the front of the specimen during \( (\mu_{3}^{-3} = -2.02") \) is also shown in Figure 10.15. A bar on the north side of the specimen visibly buckled during \( (\mu_{6}^{-1} = -4.07") \), Figure 10.16, at the location of the largest transverse steel spacing of 3” above the column base. Since the specified transverse steel spacing was only 2”, the detailing error led to a reduction in the deformation capacity of the column. Three additional north reinforcing bars buckled during \( (\mu_{8}^{-1} = -5.45") \), before any south reinforcement showed signs of buckling. Two previously buckled north reinforcing bars fractured during \( (\mu_{8}^{+3} = 5.37") \), Figure 10.17, resulting in the first significant loss in strength. A third north bar fractured during \( (\mu_{10}^{+1} = 6.67") \), followed by a fourth during \( (\mu_{10}^{+2} = 6.65") \).
Compare this to the south side of the specimen, Figure 10.16, where the critical transverse steel spacing above the footing was only 1.5” on center. Visible buckling of two south reinforcing bars was delayed until \( \mu_{10}^2 = 6.65" \), when cross section equilibrium had already been influenced by fractured north reinforcing bars. One of these south buckled bars ruptured during \( \mu_{10}^2 = -6.70" \), followed by the second during \( \mu_{10}^3 = -6.70" \). This test exemplifies the need for consistent transverse steel detailing, and shows ultimately why the results from Tests 1-6 were not included in the formulation of design recommendations for this research program.

Figure 10.12 Test 3 – Symmetric Three-Cycle-Set Load History
Figure 10.13 Test 3 – Hysteretic Response

Figure 10.14 Hysteretic Comparison of Tests 2 and 3
Figure 10.15 Test 3 – (Left) North Concrete Crushing during \( \mu_{3}^{-1} = -2.05'' \) and (Right) Crack Distribution on the Front of the Specimen during \( \mu_{3}^{-3} = -2.02'' \)

Figure 10.16 Test 3 – (Left) Buckling of North Reinforcement during \( \mu_{6}^{-1} = -4.07'' \) and (Right) Buckling of South Reinforcement during \( \mu_{10}^{+2} = 6.65'' \)
10.5 Test 4: 1940 El Centro Earthquake Load History

The goal of the physical tests in the research program is to investigate the impact of load history on the relationship between strain and displacement and the performance strain limits. Tests one through three focused on monotonic and symmetric three-cycle-set load histories. The displacement input for Tests 4-6 is determined using acceleration time histories from recorded earthquakes and non-linear time history analysis. Test four was subjected to an analytical load history obtained from the north-south component of the 1940 El Centro earthquake. The top column displacement history in Figure 10.18 was determined using Ruaumoko inelastic time history analysis with a viscous damping ratio of 5.7% based on tangent stiffness and a thin Takeda hysteretic rule. A scale of three times the original acceleration values in the El Centro time history was chosen to produce buckling on both sides of the specimen. The scale was chosen based on the amplitude of the resulting peak displacements in the load history for the push and pull directions of loading. The maximum spacing of the transverse steel in the plastic hinge region on the North and South sides of the
column was 3" and 2" respectively. The load history was oriented such that the south side would experience tension under displacement ductility ten, corresponding to the deformation which induced bar buckling for a similar transverse steel spacing in prior experiments. In the cycle naming system, \((\mu_{1.0}^{1.00 \text{ sec}} = -6.71")\) represents a pull cycle to displacement ductility 10 which occurred 3.00 seconds into the analytical 1940 El Centro load history. The time axis in Figure 10.18 is a reference to the peak cycles in the analytical load history, in the experiment the load was applied in a quasi-static manner.

The fourth test began with the application of 170 kips of axial load, which is an axial load ratio of 5% for the tested cylinder strength \((f'_c = 7.93 ksi)\). During the fourth half cycle of load the specimen was held at the analytical first yield force of 44.02 kips to determine the first yield displacement for ductility calculations \((\Delta'_y = 0.50")\). The push cycle then resumed until the target displacement of \((\mu_{1.0}^{1.00 \text{ sec}} = 0.67")\) was reached. The crack distribution on the north side of the specimen during this cycle, and the south side during the subsequent cycle to \((\mu_{1.0}^{1.32 \text{ sec}} = -0.66")\) is shown in Figure 10.22. Concrete crushing was observed on the south side of the specimen during the push cycle to \((\mu_{5.0}^{1.67 \text{ sec}} = 3.96")\), Figure 10.23. Crushing on the north side of the specimen was observed during \((\mu_{-6.4}^{2.03 \text{ sec}} = -4.40")\).

Longitudinal steel buckling was observed during the reversal from \((\mu_{5.8}^{2.44 \text{ sec}} = 3.91")\), during the pull cycle to \((\mu_{-10.0}^{3.00 \text{ sec}} = -6.71")\), when the extreme fiber north Bar N3 visibly deformed as shown in Figure 10.24. On the South side of the column, bar buckling was observed after reversal from \((\mu_{-4.3}^{3.86 \text{ sec}} = -2.95")\), while on the way to \((\mu_{5.7}^{4.45 \text{ sec}} = 3.81")\) as shown in Figure 10.24. Two additional bars on the north side of the specimen buckled during the pull cycle to \((\mu_{-2.6}^{4.78 \text{ sec}} = -1.77")\). An additional south reinforcing bar buckled during the push cycle to \((\mu_{-0.2}^{4.97 \text{ sec}} = -0.11")\). The previously buckled south reinforcing bar ruptured during the pull cycle to \((\mu_{-2.6}^{8.36 \text{ sec}} = -1.80")\). The remainder of the load history consisted of small inelastic cycles which did not produce further notable damage.

The scale chosen for the 1940 El Centro acceleration time history produced buckling on each side of the specimen, and a single fractured reinforcing bar. The reinforcing bars did
not immediately buckle upon reversal from the first inelastic peak matching the predicted buckling displacements for the given spiral spacing from previous tests, which indicates that the displacement amplitudes in the load history were properly scaled to produce bar buckling.

Figure 10.18 Test 4 – Analytical Lateral Displacement History for 1940 El Centro EQ
Figure 10.19 Test 4 – Hysteretic Response for the 1940 El Centro EQ

Figure 10.20 Test 4 – Comparison of Analytical and Experimental Hysteretic Response
Figure 10.21 Comparison of Cyclic Test 3 and 1940 El Centro EQ Load History

Figure 10.22 Test 4 – (Left) Crack distribution at $\left(\mu_{1.0}^{1.00\text{sec}} = 0.67"\right)$ and (Right) Crack Distribution at $\left(\mu_{1.0}^{-1.32\text{sec}} = -0.66"\right)$
Figure 10.23 Test 4 – (Left) Crushing on the South during \( \mu_{59}^{1.67 \text{sec}} = 3.96" \) and (Right) Crushing on the North during \( \mu_{64}^{2.03 \text{sec}} = -4.40" \)

Figure 10.24 Test 4 – (Left) Buckling of North Bar N3 at \( \mu_{10.0}^{3.00 \text{sec}} = -6.71" \) and (Right) Buckling of South Bar S3 and S4 at \( \mu_{57}^{4.45 \text{sec}} = 3.81" \)
10.6 Test 5: 1978 Tabas Earthquake Load History

The first three tests focused on monotonic and symmetric three-cycle-set load histories. The displacement input for Tests 4-6 is determined using acceleration time histories from recorded earthquakes and non-linear time history analysis. Test five was subjected to an analytical load history obtained from the 1978 Tabas earthquake in Iran. The top column displacement history in Figure 10.25 was determined using Ruaumoko inelastic time history analysis with a viscous damping ratio of 5.7% based on tangent stiffness and a thin Takeda hysteretic rule. The scale factor for the acceleration values in the Tabas load history was chosen based on displacements that caused buckling in previous tests with similar transverse steel spacing in the plastic hinge. The resulting hysteretic response for the Tabas load history is shown in Figure 10.26.

The fifth test began with the application of 170 kips of axial load, which is an axial load ratio of 5% for the tested cylinder strength \( f'_c = 8.4 \text{ ksi} \). During the first half cycle of load the specimen was held at the analytical first yield force of 44 kips to determine the first yield displacement for ductility calculations \( \Delta'_y = 0.55'' \). To allow for direct comparison to previous tests, the ductility levels for Test 5 were based on a first yield displacement of 0.50" and corresponding ductility one displacement of 0.67". Previous tests results with a maximum transverse steel spacing of 3" (north) and 2" (south) in the plastic hinge region have shown that buckling occurs upon reversal from \( \mu_{\Delta_6} = 4.04'' \) and \( \mu_{\Delta_{10}} = 6.72'' \) accordingly. In Figure 10.25, the peak displacements from the analytical load history are highlighted, \( \mu_{7.7}^{11.38 \text{ sec}} = 5.20'' \) or \( \mu_{7.5}^{11.98 \text{ sec}} = 5.07'' \) to buckle the North reinforcement upon reversal and \( \mu_{10.0}^{14.26 \text{ sec}} = -6.68'' \) for the South. First crushing of cover concrete on the north and south sides of the specimen were observed at \( \mu_{2.4}^{5.08 \text{ sec}} = 1.59'' \) and \( \mu_{2.1}^{5.4 \text{ sec}} = -1.57'' \) respectfully, Figure 10.27. In the physical test, bar buckling was not observed during the Tabas load history. Photos of the north and south sides of the specimen at the conclusion of the Tabas load history appear in Figure 10.28.
The post-earthquake capacity of the column was investigated by subjecting the same specimen to a symmetric three-cycle-set load history identical to Tests 2 and 3, Figure 10.29. The resulting hysteretic response appears in for the cyclic aftershock load history appears in Figure 10.30 and Figure 10.31. The cyclic aftershock hysteretic response is compared to that of an initially undamaged column from Test 3 in Figure 10.32.

The first sign of reinforcement buckling occurred during the first pull cycle of ductility six \( (\mu_6^{-1} = -4.07") \), when the North extreme fiber bar began to visible deform in two separate locations as shown in Figure 10.33. During \( (\mu_8^{-1} = -5.46") \), Bar S1 ruptured in tension without previous signs of buckling and Bar N4 buckled at the same location as the upper buckled region Bar N3. The rupture of Bar S1 was not expected, since it is not an extreme fiber experiencing the largest demands. The early fracture is attributed to the effects of welding on the strain capacity of the base material, but it is unclear why this was only observed in a single experiment and not at the location of the largest demands. The influence of welding on the steel material properties from reinforcing bar tensile tests is the topic of the previous chapter of this report.

On the way to \( (\mu_8^{+2} = 5.35") \), Bar S3 became the first bar to buckle on the South side of the column and previously buckled bar N3 ruptured in tension. Other notable events occurred as follows: Bars S2 and S4 buckled during \( \mu_8^{+3} \); Bar S4 ruptured during \( \mu_{10}^{-1} \); Bar N4 ruptured during \( \mu_{10}^{+2} \); and Bar N2 ruptured during \( \mu_{10}^{+3} \). Envelope curves connecting the peaks of the third cycle of each ductility level in Tests 2, 3, and 5b appear in Figure 10.34. This figure illustrates the degree of stiffness degradation in the cyclic aftershock load history caused by the prior Tabas earthquake record. The degradation behavior at the ultimate level appears to be similar between the three experiments.
Figure 10.25 Test 5 – Analytical 1978 Tabas Earthquake Load History

Figure 10.26 Test 5 – Hysteretic Response for 1978 Tabas EQ Load History
Figure 10.27 Test 5 – (Left) North Concrete Crushing at ($\mu_{2.4}^{5.08\text{ sec}} = 1.59"$) and (Right) South Concrete Crushing at ($\mu_{2.1}^{5.4\text{ sec}} = -1.57"$)

Figure 10.28 Test 5 – (Left) North and (Right) South Sides after Tabas EQ LH
Figure 10.29 Test 5b – Symmetric Three-Cycle-Set Aftershock Load History

Figure 10.30 Test 5b – Hysteretic Response for Cyclic Aftershock Load History
Figure 10.31 Tests 5 and 5b – Comparison of Tabas EQ LH and Cyclic Aftershock

Figure 10.32 Tests 3 and 5b – Hysteretic Comparison for Initially Undamaged Symmetric Three-Cycle-Set and Cyclic Aftershock Load Histories
Figure 10.33 Test 5b – (Left) Buckling of Bar N3 during \( \mu_6^{-1} = -4.07'' \) and (Right) Three South Buckled Bars at \( \mu_8^{+3} = 8.35'' \)

Figure 10.34 Test 5b – (Left) Third Cycle Envelope Curves for Tests 2, 3, and 5b (Right) Fracture of Bar S1 during \( \mu_8^{-1} = -5.46'' \)
10.7 Test 6: 1978 Tabas Earthquake Load History

The first three tests focused on monotonic and symmetric three-cycle-set load histories. The displacement input for Tests 4-6 is determined using acceleration time histories from recorded earthquakes and non-linear time history analysis. Test five was subjected to an analytical load history obtained from the 1978 Tabas earthquake in Iran. The top column displacement history in Figure 10.35 was determined using Ruaumoko inelastic time history analysis with a viscous damping ratio of 5.7% based on tangent stiffness and a thin Takeda hysteretic rule. The Tabas load history utilized in Test 6 was an exact inverse of the displacement history from Test 5. The resulting hysteretic response for the Tabas load history is shown in Figure 10.36. A hysteretic comparison of the inverted response from Test 5 and the measured response from Test 6 appears in Figure 10.37.

The sixth test began with the application of 170 kips of axial load, which is an axial load ratio of 3.7% for the tested cylinder strength (\( f'_c = 10.2 \text{ ksi} \)). During the first half cycle of load the specimen was held at the analytical first yield force of 44 kips to determine the first yield displacement for ductility calculations (\( \Delta'_y = 0.53'' \)). To allow for direct comparison to previous tests, the ductility levels for Test 6 were based on a first yield displacement of 0.50" and corresponding ductility one displacement of 0.67". Cover concrete crushing was observed on the north side of the specimen during (\( \mu_{-2.4}^{5.08 \text{ sec}} = -1.59'' \)), Figure 10.38. A photo depicting crushing on the south side of the specimen at (\( \mu_{4.6}^{10.32 \text{ sec}} = 3.11'' \)) appears in Figure 10.38.

The extent of crushing on the north and south sides of the column at (\( \mu_{-7.8}^{11.38 \text{ sec}} = -5.24'' \)) and (\( \mu_{10.0}^{14.26 \text{ sec}} = 6.70'' \)) respectively appear in Figure 10.39. The north extreme fiber bars N2 and N3 during the reversal from the peak displacement of (\( \mu_{10.0}^{14.26 \text{ sec}} = 6.70'' \)) as shown in Figure 10.40. The photo of the buckled deformation was taken at (\( \mu_{-1.5}^{15.86 \text{ sec}} = -0.98'' \)). An additional north reinforcing bar buckled, and the deformation in the two previously buckled bars increased during (\( \mu_{-0.1}^{20.90 \text{ sec}} = -0.06'' \)), Figure 10.40.
Figure 10.35 Test 6 – 1978 Tabas Earthquake Load History (Inverse of LH from T5)

Figure 10.36 Test 6 – Hysteretic Response for Tabas Load History
Figure 10.37 Comparison of Test 6 Response and Inverse of Test 5 Response

Figure 10.38 Test 6 – (Left) North Crushing observed during \( \mu_{-2.4}^{5.08 \text{ sec}} = -1.59'' \) and (Right) Extent of South Crushing at \( \mu_{4.6}^{10.32 \text{ sec}} = 3.11'' \)
Figure 10.39 Test 6 – (Left) Extent of North Crushing during ($\mu_{11.38 \, sec} = -5.24''$) and (Right) Extent of South Crushing during ($\mu_{14.26 \, sec} = 6.70''$)

Figure 10.40 Test 6 – (Left) Initial Buckling of Bars N2 and N3 at ($\mu_{15.86 \, sec} = -0.98''$) during Reversal from ($\mu_{10.0} = 6.70''$) and (Right) Buckled Deformation in North Bars during ($\mu_{20.90 \, sec} = -0.06''$)
Chapter 11: Design Recommendations for Limit State Displacements

11.1 Performance Strain Limits

To satisfy the aims of performance based design, levels of damage which interrupt the serviceability of the structure or require more invasive repair techniques must be related to engineering criteria. Cover concrete crushing and residual crack widths exceeding 1mm represent serviceability limit states, which when exceeded require repair. Longitudinal bar buckling and significant damage to the core concrete represent damage control limit states which represent the limit of economical repair. The ultimate limit state is characterized by fracture of previously buckled reinforcement or rupture of confinement steel. The first occurrence of these limit states in a column test by Goodnight et al. is shown in Figure 11.1.

Figure 11.1 Displacement History, Hysteretic Response, and Performance Limit States
11.1.1 Serviceability Limit States

When exceeded, serviceability limit states represent the point at which repair is necessary, interrupting the serviceability of the structure, but not posing a safety concern. The serviceability limit states are characterized by crushing of cover concrete or residual crack widths which exceed 1mm, both should be repaired to prevent corrosion of internal reinforcing steel.

**Cover Concrete Crushing:**

\[ \varepsilon_c = 0.004 \]

Concrete compression strain related to crushing of the cover concrete. Evaluated at the extreme compression fiber. Eqn 11.1

**Residual Crack Widths (1 mm):**

\[ \varepsilon_s = 0.015 \]

Steel tensile strain limit related to residual crack widths which exceed 1 mm. Evaluated at the location of extreme longitudinal reinforcing bar. Eqn 11.2

11.1.2 Intermediate Compressive Limit State

Currently there is not an intermediate strain limit between serviceability and damage control which is related to a change in compressive behavior that results due to confinement steel yielding. Experimental results suggest that localization of compressive demand can occur in regions with inelastic transverse steel. This localization can lead to compression strains which exceed predictions utilizing moment-curvature analysis and an equivalent curvature distribution. Furthermore, inelastic transverse steel restraint resulted in measurable outward deformation of longitudinal reinforcement prior to visible bar buckling observations. As a limit state, initial yielding of confinement steel prompts a change in repair strategy from epoxy injection of cracks and patching of cover concrete to the need for additional transverse stiffness via either a steel jacket or FRP wraps in the plastic hinge region.
Chapter 11: Design Recommendations for Limit State Displacements

Initial Yielding of Confinement Steel:

\[ \varepsilon_c = 0.009 - 0.3 \frac{A_{st}}{A_g} + 3.9 \frac{f_{yh}}{E_s} \]

Concrete compression strain at initial yield of confinement steel. Evaluated at the centerline of the transverse steel, i.e. the concrete core.

Eqn 11.3

11.1.3 Damage Control Limit States

The damage control limit state represents the limit of economical repair, where past this point repair may be uneconomical or unfeasible. The current damage control concrete compressive strain limit is defined by the Mander et al. (1988) ultimate concrete compression strain. While the intermediate compressive limit state was related to initial yielding of confinement steel, the damage control limit state is a reasonable approximation to compressive strain levels which influence the ability of the transverse steel to restrain longitudinal bars from buckling.

Bar buckling was observed to occur after reversal from a peak tensile strain while the bar is under net elongation, but compressive stress. Although prior compression is important to describing the restraint provided by transverse steel, expressions developed based on the peak tension strain or drift measured before bar buckling upon reversal of load were found to produce the most accurate predictions. Furthermore, higher levels of tensile strain reduce the tangent modulus of the reinforcing during the subsequent stress reversal. Sufficient confinement steel should be provided such that the Mander (1988) Ultimate Concrete Compressive Strain exceeds the compressive strain at the bar buckling displacement. For new design, the strain-based Eqn 11.5 or the drift-based Eqn 11.6 can be used to evaluate the peak displacement at which bar buckling is expected to occur after reversal during a cyclic load history. The parameters in these expressions are known or may be reasonably approximated at the onset of design, and later confirmed after finalizing the transverse steel detailing.
**Ultimate Concrete Compressive Strain:**

Concrete compression strain related to limit of economical repair of core concrete. Evaluated at the centerline of the transverse steel. For a typical $\rho_s = \frac{4A_{sp}}{D's} = 1\%$, the computed Mander (1988) $\varepsilon_{cu} \approx 0.018$.

Eqn 11.4

**Strain-Based Bar Buckling:**

Peak Tension Strain Prior to Bar Buckling. Evaluated at the location of extreme longitudinal reinforcing bar.

$$\varepsilon_s = 0.03 + 700\rho_s \frac{f_{yh}}{E_s} - 0.1 \frac{P}{f'_c A_g}$$

Eqn 11.5

$\rho_s = \frac{4A_{sp}}{D's}$ Transverse Volumetric Steel Ratio, influences confinement and bar restraint.

$f_{yh}/E_s$ Inelastic Transverse steel is less effective at restraining longitudinal bars.

$P/f'_c A_g$ Axial Load Ratio expressed as a decimal rather than a percent.

**Drift-Based Bar Buckling:**

In the following expression, $L$ is the length from the column base to the point of contraflexure and $D$ is the diameter of the cross section. The result of the expression is the drift as a percent in which bar buckling would be expected to occur during the subsequent reversal in a cyclic load history.

$$\frac{\Delta}{L} (\%) = 0.9 - 3.13 \frac{P}{f'_c A_g} + 142000\rho_s \frac{f_{yh}}{E_s} + 0.45 \frac{L}{D}$$

Eqn 11.6

Drift at Bar Buckling
**Feng (2013) Bar Buckling Model:**

Feng (2013) proposed a series of equations to describe bar buckling behavior observed in finite element analysis. The resulting bar buckling model consists of a series of three equations (Eqn 11.7 through Eqn 11.11) which form a border between tension and compression strain couples, which when exceeded produce bar buckling. In the expression, \( \varepsilon_t \) and \( \varepsilon_c \) are the tensile and compressive longitudinal bar strains (both taken as positive), \( d_{bl} \) and \( d_h \) are the longitudinal and transverse steel bar diameters, and \( s \) is the centerline spacing of transverse steel.

The bar buckling prediction is defined as the intersection of the multi-linear regression and the tensile-compressive bar strain relationship from moment-curvature analysis, Figure 11.2. The tension and compression strain couple at the intersection point represents a compression cycle followed by a tension cycle to the same level of displacement, which would induce bar buckling upon subsequent reversal of load. The Priestley, Calvi, and Kowalsky (2007) plastic hinge method is used to translate the intersection point strain to curvatures and finally member deformation. When compared to the Goodnight et al. dataset, the predicted tensile strains at a bar buckling are unconservative, but when combined with the conservative tensile strain-displacement relationship of the PCK (2007) plastic hinge method, the resulting bar buckling displacement is improved.

\[
\varepsilon_t = \frac{-15 \left( \varepsilon_c - 0.0205 \right)}{\left( \frac{d_{bl}}{d_h} - 1 \right)^2} \tag{Eqn 11.7}
\]

\[
\varepsilon_t \geq -1.7 \frac{s}{d_{bl}} \sqrt{\frac{d_h}{d_{bl}}} \varepsilon_c + 0.045 \frac{s}{d_{bl}} \tag{Eqn 11.8}
\]
\[ \varepsilon_t \leq 0.09, \quad \text{if } \frac{s}{d_{bl}} < 3 \quad \text{Eqn 11.9} \]

\[ \varepsilon_t \leq 0.06, \quad \text{if } \frac{s}{d_{bl}} > 4 \quad \text{Eqn 11.10} \]

\[ \varepsilon_t \leq 0.09 - 0.03 \left( \frac{s}{d_{bl}} - 3 \right), \quad \text{if } 3 < \frac{s}{d_{bl}} < 4 \quad \text{Eqn 11.11} \]

Figure 11.2 (Left) Feng (2013) Method with Tension/Compression Bar Strains from Moment-Curvature Analysis and (Right) Bar Buckling at Intersection Point using PCK (2007) Hinge Length
11.2 Modified Plastic Hinge Method

The Modified Plastic Hinge Method was developed to improve the accuracy of strain-displacement predictions necessary for successful implementation of strain-based limit states. Equivalent curvature distributions for the Modified Plastic Hinge Method appear in Figure 11.3 for a fixed-fixed column in double bending and a fixed-free column in single bending. The key aspects of the proposed Modified Plastic Hinge Model which differentiate it from the current method recommended in Priestley, Calvi, and Kowalsky (2007) include: (1) a decoupling of column flexure and strain penetration deformation components, (2) a linear plastic curvature distribution which emulates the measured curvature profiles, and (3) separate plastic hinge lengths for tensile and compressive strain-displacement predictions.

In the experiments, the measured extent of plasticity was found to increase due to the combined effects of moment gradient and tension shift. The proposed tension hinge length, $L_{pr_t}$ Eqn 11.14, was calibrated to match the upper bound of the measured spread of plasticity in each test. The proposed compressive hinge length, $L_{pr_c}$ Eqn 11.15, only contains a term related to the moment gradient effect. Expressions for the elastic and plastic column flexural displacement for both single and double bending were derived. Expressions which describe the additional column deformation due to strain penetration of reinforcement into the adjoining member were derived based on the measured fixed-end rotations. Part of this was the formulation of a new equivalent strain penetration length $L_{sp}$ Eqn 11.12.

Elastic displacements are computed when the base section curvature is either at or below the first yield curvature, $\phi'_y$. The elastic displacement of a column in single bending is calculated using Eqn 11.16 through Eqn 11.18. The elastic displacement of a column in double bending is computed using Eqn 11.19 through Eqn 11.21. The elastic displacement is the addition of elastic column flexural, strain penetration, and shear deformations. Shear displacements were negligible for columns in the Goodnight et al. dataset, therefore no further guidance is provided.
Inelastic displacements are computed when the base section curvature exceeds the first yield curvature, $\phi'_y$. To account for additional elastic flexibility of the column, the first yield displacement is multiplied by the ratio of the current base section moment to the moment at first yield of longitudinal reinforcement, $M/M'_y$. The plastic curvature at the base section is obtained by subtracting the elastic curvature from the base section curvature, $\phi_p = \phi_{base} - \phi'_y (M/M'_y)$. For translation of a tensile strain limit to a lateral displacement, the tensile triangular plastic hinge length should be used, $L_{pr_t}$ Eqn 11.14. If instead, a compressive strain limit is translated to a later displacement, the compressive triangular plastic hinge length should be employed, $L_{pr_c}$ Eqn 11.15. Expressions needed to compute the inelastic displacement of a column in single bending are shown in Eqn 11.22 through Eqn 11.26. Expressions needed to compute the inelastic displacement of a column in double bending appear in Eqn 11.27 through Eqn 11.31. The inelastic flexural displacement is the sum of the elastic column flexural, plastic column flexural, strain penetration, and shear deformations.

**List of Selected Terminology:**

$L$ = Length of the Column  

$D$ = Column Diameter  

$L_c$ = Length to the Point of Contraflexure, ($L_c = L/2$ for a Column in Double Bending)  

$k$ = Moment Gradient Coefficient of Plastic Hinge Length Expression  

$M'_y$ and $\phi'_y$ = Moment and Curvature at First Yield of Longitudinal Reinforcement  

$M$ = Column Base-Section Moment  

$\phi_p = \phi_{base} - \phi'_y M/M'_y$ = Plastic Curvature at the base Section  

$\Delta_e$ and $\Delta_p$ = Elastic and Plastic Column Flexural Displacement
\( \Delta_{\text{shear}} \) = Column Shear Displacement

\( L_{pr_t} \) = Tension Hinge Length Based on Triangular Distribution

\( L_{pr_c} \) = Compression Hinge Length Based on Triangular Distribution

\( \Delta_{sp} \) = Column Displacement Attributable to Strain Penetration of Reinforcement

\( L_{sp} \) = Equivalent Strain Penetration Length

\( f_{ye} \) and \( f_{ue} \) = Yield and Ultimate Stress of Longitudinal Steel with Expected Properties

\( f_{ce}' \) = Unconfined Column Concrete Compressive Strength with Expected Properties

\( f_{ce}'_f \) = Concrete Compressive Strength of the Adjoining Member

\( d_{bl} \) = Diameter of the Longitudinal Reinforcement and Column Axial Load

\( P \) = Column Compressive Axial Load
Figure 11.3 Equivalent Curvature Profiles for the Modified Plastic Hinge Method, (Left) Column in Double Bending and (Right) Column in Single Bending
11.2.1 Strain Penetration Length and Tension/Comp. Plastic Hinge Lengths

\[ L_{sp} = U \left( 1 - \frac{P}{f'_{ce}A_g} - \frac{L_c}{16D} \right) \frac{f_{ye}d_{bi}}{\sqrt{f'_{ce}f}} \]

\[ U = 0.4 \text{ for ksi and 0.152 for MPa units} \]

Eqn 11.12

\[ k = 0.2 \left( \frac{f_u}{f_y} - 1 \right) \leq 0.08 \]

Same Definition of k as Priestley, Calvi, and Kowalsky (2007)

Eqn 11.13

\[ L_{pr_t} = 2kL_c + 0.75D \]

Tension Hinge Length Based on Triangular Dist.

Eqn 11.14

\[ L_{pr_c} = 2kL_c \]

Compression Hinge Length Based on Triangular Distribution

Eqn 11.15
11.2.2 Elastic Displacements for a Column in Single Bending

\[ \Delta_e = \phi_{\text{base}} L^2 / 3 \]  
(Single) Elastic Flexural Displacement before First Yield  
Eqn 11.16

\[ \Delta_{sp} = L_{sp} \phi_{\text{base}} L \]  
Displacement due to Strain Penetration  
Eqn 11.17

\[ \Delta_T = (\Delta_e + \Delta_{sp} + \Delta_{shear}) \]  
Total Top Column Displacement  
Eqn 11.18

11.2.3 Elastic Displacements for a Column in Double Bending

\[ \Delta_e = \phi_{\text{base}} L^2 / 6 \]  
(Double) Elastic Flexural Disp. before First Yield  
Eqn 11.19

\[ \Delta_{sp} = L_{sp} \phi_{\text{base}} L \]  
Displacement due to Strain Penetration  
Eqn 11.20

\[ \Delta_T = (\Delta_e + \Delta_{sp} + \Delta_{shear}) \]  
Total Top Column Displacement  
Eqn 11.21
11.2.4 Inelastic Displacements for a Column in Single Bending

\[ \Delta_e = \phi'_y \left( \frac{M}{M'_y} \right) \frac{L^2}{3} \]  
(Single) Elastic Flexural Disp. after First Yield  
Eqn 11.22

\[ \phi_p = \phi_{base} - \phi'_y \left( \frac{M}{M'_y} \right) \]  
Plastic Curvature at the Base Section  
Eqn 11.23

\[ \Delta_p = \phi_p \left( \frac{L_{pr}}{2} \right) \left[ L - \frac{L_{pr}}{3} \right] \]  
(Single) Plastic Disp. for Triangular Plastic Curvature Distribution  
Eqn 11.24

\[ \Delta_{sp} = L_{sp} \phi_{base} L \]  
Displacement due to Strain Penetration  
Eqn 11.25

\[ \Delta_T = \left( \Delta_e + \Delta_{sp} + \Delta_p + \Delta_{shear} \right) \]  
Total Top Column Displacement  
Eqn 11.26

11.2.5 Inelastic Displacements for a Column in Double Bending

\[ \Delta_e = \phi'_y \left( \frac{M}{M'_y} \right) \frac{L^2}{6} \]  
(Double) Elastic Flexural Disp. after First Yield  
Eqn 11.27

\[ \phi_p = \phi_{base} - \phi'_y \left( \frac{M}{M'_y} \right) \]  
Plastic Curvature at the Base Section  
Eqn 11.28

\[ \Delta_p = \phi_p \left( \frac{L_{pr}}{2} \right) \left[ L - 2 \frac{L_{pr}}{3} \right] \]  
(Double) Plastic Disp. for Triangular Plastic Curvature Distribution  
Eqn 11.29

\[ \Delta_{sp} = L_{sp} \phi_{base} L \]  
Displacement due to Strain Penetration  
Eqn 11.30

\[ \Delta_T = \left( \Delta_e + \Delta_{sp} + \Delta_p + \Delta_{shear} \right) \]  
Total Top Column Displacement  
Eqn 11.31
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12.1 Problem Statement

Seismic bridge design practice utilizes the simplifying assumption of unidirectional response, which results in consideration of orthogonal directions of loading on an individual basis. Typically, for bridges, the two directions are transverse and longitudinal to the direction of traffic (for a straight bridge). Such a division is usually employed for assessing demand and capacity, and would be appropriate if the two directions of loading were uncoupled from each other.

Prior research at NCSU investigated the impact of loading history on the unidirectional response of RC bridge columns, leading towards recommendations on strain limits and plastic hinge lengths which consider the impacts of real seismic loading. The next step in this progression is to consider the impact of multi-directional loading path on these recommendations. It is possible that multi-directional loading, even for circular columns, could lead to adjustments to unidirectional strain limits proposed for design, as well as the manner in which those strain limits are converted to design displacements via the plastic hinge method for member deformations.

12.2 Background

Bridge columns are designed as ductile elements which form plastic hinges to dissipate energy in a seismic event. The goal of performance based seismic engineering is to design structures to achieve a predictable level of performance under a specific earthquake hazard within definable levels of reliability, as defined by the Structural Engineering Association of California (SEAOC 1999). To satisfy the aims of performance based design, levels of damage which interrupt the serviceability of the structure or require more invasive repair
techniques must be related to engineering criteria. For reinforced concrete flexural members such as bridge columns, concrete compressive and steel tensile strain limits are good indicators of damage.

Serviceability limit states such as concrete cover crushing or residual crack widths exceeding 1mm may occur during smaller, more frequent earthquakes (Priestley et al. (1996)). While the serviceability limit states do not pose a safety concern, the hinge regions must be repaired to prevent corrosion of internal reinforcing steel. At higher ductility demands produced by larger less frequent earthquakes, reinforcing bar buckling may lead to permanent elongation in the transverse steel, which diminishes its effectiveness in confining the concrete core. Bar buckling and significant damage to the core concrete represent the damage control limit states, which when exceeded lead to significant repair costs (Priestley et al. (1996)). Furthermore, rupture of previously buckled bars during subsequent cycles of loading leads to rapid strength loss. The life safety or collapse prevention limit state is characterized by fracture of previously buckled bars. A summary of damage observations from a reinforced concrete bridge column tested at NCSU by Goodnight et al. (2014) appears in Figure 12.1.

Performance based seismic engineering requires accurate limit state-based engineering demand parameters, such as material strains, and an accurate method of relating these quantities to a capacity measure in design, such as member deformation. In the following section, advancements in strain limit-based displacement predictions are summarized for the current AKDOT sponsored project (Goodnight et al. (2014)). The experimental portion of the research program is complete and design recommendations are being finalized for the AKDOT. Finally, the importance of considering bi-directional displacement history is evaluated based on specific observations reported in literature.
While the progression of damage in flexural bridge columns has been investigated in the past for both uniaxial and biaxial deformation demands, traditional methods of deriving material strains from measured curvatures do not assess strains at the locations of interest, namely the longitudinal reinforcement and core concrete. These methods utilize an array of linear potentiometers placed on the ends of threaded rods embedded in the core concrete to calculate changes in displacement outside of the cover concrete. A new instrumentation technique was devised to overcome this limitation. Goodnight et al. (2014) investigated the impact of unidirectional-lateral displacement history and design variables on the material strain limits and the relationship between strain and displacement for circular reinforced
Concrete columns. To date, all thirty experiments have been completed and design recommendations are being finalized for AKDOT.

The specimen was designed to represent a single degree of freedom bridge column subjected to lateral and axial load, Figure 12.2. The test specimen consists of a footing, column, and loading cap. The footing is a capacity protected member which secures the specimen to the lab strong floor using post tensioned bars. A 200kip hydraulic actuator, with a 40in stroke capacity, applies lateral load to the loading cap of the specimen. A spreader beam, two hydraulic jacks, and a load cell are placed above the loading cap to apply a constant axial compressive load. The top column displacement was obtained through a string potentiometer placed at the center of the lateral load. The experimental program utilized multiple Optotrak Certus HD 3D position monitors developed by Northern Digital Inc. The position monitors track the locations of the target markers in 3D space, returning X-Y-Z spatial coordinates with an accuracy of 0.1mm and with a resolution of 0.01mm.

A technique of applying target markers to longitudinal and transverse reinforcement, Figure 12.2, was utilized in the plastic hinge region. Strains are computed by dividing the change in three dimensional distance between two adjacent target markers by the original unloaded gage length. Both transverse and longitudinal steel was instrumented to measure the interaction between core concrete confinement and longitudinal bar restraint demands in spiral layers leading up to visible bar buckling. Closely spaced transverse steel restrains the dilation of core concrete under compressive demands, improving its strength and deformation capacity. These spiral layers locally support the longitudinal bar, reducing its unbraced length, which delays the onset of bar buckling. Large compressive cycles can yield the transverse steel, reducing its effectiveness as a boundary condition restraining the longitudinal bar from buckling. Longitudinal bars are prone to buckling during reversals from peak tensile strains, while the cracks are still open, and they are the sole source of compression zone stability. This behavior is observable in the measured longitudinal and overlaying spiral strain hysteresis, Figure 12.3, in the region where outward bar buckling was observed just above the footing-column interface. Some small level of outward deformation
prior to visible bar buckling is observable in the longitudinal and transverse steel hysteresis during the first two pull cycles to -6.6”. Upon visible bar buckling, a significant deviation in both hystereses is noted, confirming the observation.

Figure 12.2 Optotrak Instrumentation Technique Applied to Columns in Goodnight et al. (2014). Target Markers Monitor Longitudinal and Transverse Reinforcement Strains in the Column Hinge Regions.

Figure 12.3 Goodnight et al. (2014) Test 16. [Left] Longitudinal Steel Strain Hysteresis and [Right] Spiral Strain Hysteresis for the Layer over the Outward Buckled Region of the Longitudinal Bar.
The instrumentation technique proved pivotal in improving the accuracy of strain-limit based displacement predictions. The data was used to calculate material strains, curvature distributions, and fixed end rotations due to strain penetration of reinforcement into the footing. The decoupling of column flexural displacements from fixed-end rotations due to strain penetration, along with an understanding of the spread of plasticity, allowed for creation of a new equivalent curvature distribution. In design, there are two main techniques to assess the member displacement at a given material strain level: (1) monotonic moment-curvature analysis paired with an equivalent curvature distribution and (2) cyclic fiber analysis paired with an element representation of the beam or column. In the following section, the predictive capabilities of these two methods are evaluated along with the design recommendations from Goodnight et al. (2014).

For this comparison, the monotonic section analysis from a script developed at NCSU called Cumbia (2007) is translated into member response using the plastic hinge method presented in Priestley, Calvi, and Kowalsky (2007) and the modified NCSU hinge method proposed by Goodnight et al. (2014), Figure 12.5. In both methods, the elastic and plastic curvature distributions are separated into equivalent simplified shapes to facilitate design. The PCK (2007) hinge method utilizes a constant plastic hinge length which includes the influence of strain penetration. Independent measurement of deformation components allowed for a decoupling of strain penetration and column flexure in the NCSU hinge method. Furthermore, the shape and size of the plastic hinge length are now related to physical quantities which reflect the measured extent of plastic curvatures, Figure 12.4. The total spread of plasticity in RC bridge columns is due to the combined effects of moment gradient and tension shift. Tension shift concentrates compressive strains near the footing-column interface while fanning tension strains above the footing following the inclined flexural-shear crack distribution. Moment gradient spreads plasticity in accordance with the increase in moment higher above the footing when the base-section moment exceeds the nominal value. Tension strains were found to be influenced by the combined effect of moment gradient and tension shift, while compressive strains are more closely related to only the moment gradient component. Separate expressions for tensile and compressive plastic
hinge lengths were devised to improve both the tensile and compressive strain-displacement predictions of the equivalent curvature distribution. In Figure 12.6, the strain-displacement predictions for the two hinge methods are compared to the measured response on an extreme fiber bar instrumented in a cyclic column test from Goodnight et al. (2014). The NCSU hinge method shows improvement, but both methods fail to account for the localization of compressive strains once severe transverse steel yielding has occurred.

Monotonic section analysis and an equivalent curvature distribution allow for an accurate prediction of the backbone curve of cyclic response, but if the actual cyclic response is required, then cyclic fiber analysis paired with an element representation of the beam or column is needed. The cyclic displacement history from the experiment was recreated using a combination of two elements in OpenSees: (1) a beam with hinges element to model the column flexural deformations and (2) a zero length strain penetration element to model the fixed end rotations due to strain penetration. The beam with hinges element, developed by Scott and Fenves (2006), is a force-based beam-column element with a plastic hinge integration method. The zero length strain penetration element, developed by Zhao and Sritharan (2007), models the fixed-end rotations attributable to strain penetration of longitudinal reinforcement into the footing. Response predictions for the two analysis methods appear in Figure 12.7. The predicted tensile strains from OpenSees exceeded the measured response with the plastic hinge length from PCK (2007). The hinge length in OpenSees was changed to 1.2*Lp from PCK (2007) to match the measured tensile strain displacement relationship in Figure 12.7. Manually changing the plastic hinge length to match tensile response leads to an under prediction of the compressive strain-displacement relationship for the peak compressive gage length, leading to different hinge lengths which are proposed based on the NCSU equivalent curvature distribution.
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Figure 12.4 Goodnight et al. (2014) Test 9. [Left] Measured Fixed-End Rotation due to Strain Penetration of Longitudinal Reinforcement into the Footing and [Right] Curvature Distribution with Linear Dashed Plastic Curvature Regression Lines

Figure 12.5 [Left] Proposed NCSU Plastic Hinge Method from Goodnight et al. (2014) and [Right] PCK (2007) Method
Figure 12.6 Goodnight et al. (2014) Test 9. Strain-Displacement Relationships and Moment Curvature Predictions with PCK and NCSU Plastic Hinge Methods

Figure 12.7 Goodnight et al. (2014) Test 9. Member Response Prediction with Cyclic Fiber Model and Monotonic Section Analysis
12.3 Brief Load Path Literature Review

In a seismic event, bridge columns undergo demands in the longitudinal and transverse directions simultaneously, which produce a two-dimensional displacement path. Past experimental studies on biaxial bending with either constant or variable axial load provide insight into their effect on member behavior.


Sixteen circular shear-dominated columns were tested with different biaxial displacement histories, volumetric steel ratios, and levels of applied constant axial load. The purpose of the research was to determine the influence of biaxial loading on the “concrete component” of shear resistance. Short circular columns were tested with a 400mm diameter, an aspect ratio of 2, and a steel content of 3.2%. Axial loads of 0, 19 or 39% were utilized. Transverse volumetric steel ranged between 0.39 and 2.46%. Four displacement patterns were considered, Type u, b, s, and r in Figure 12.8. The uniaxial ‘u’ pattern had five cycles at a given amplitude before increasing to a larger displacement ductility level. Biaxial ‘b’ and ‘s’ patterns had two cycles at each amplitude before ramping up the displacement. Due to the bidirectional nature of the load history this resulted in four complete reversals at each displacement level. The multi-directional ‘r’ pattern was used to simulate a displacement path originating from NLTHA of a column under earthquake excitation.

Wong et al. (1993) concluded that biaxial response reduced the deformation capacity by one ductility level when compared to a nominally identical column subjected to uniaxial response. Furthermore, Wong et al. (1993) conclude that there was not a clear difference in results of the two b- and s-type laboratory biaxial displacement patterns. The following observation is of significance to bar buckling, “For columns reinforced with similar spiral steel content, the commencement of spiral yielding was consistently observed at lower ductilities when more severe displacement orbits were imposed.” On the topic of load path
effects, Wong et al. (1993) note, “The difference in the response of columns with identical properties subjected to simple biaxial b-type displacement patterns or to more sophisticated s-type patterns was small enough to be disregarded in design. Moreover, the performance of the unit tested with the realistic earthquake simulating random biaxial displacement pattern was found to be better than its companion unit under b-type displacement history. These suggest that if biaxial seismic effects are to be studied further, test using biaxial b-type (orthogonal) displacement paths should be sufficient.”

![Diagram of displacement patterns](image)

**Figure 12.8** Wong et al. (1993), Lateral Displacement Histories


Two nominally identical circular columns were tested, one subjected to uniaxial and the other biaxial loading. The specimens were designed for flexural-shear failure. The 350mm diameter circular specimens had 2.5% longitudinal steel, 0.5% transverse volumetric steel, a constant 20% axial load, and an aspect ratio of 4.3. A clover leaf style lateral displacement history utilized in the biaxial test, Figure 12.9. Two complete cycles at a given displacement
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Amplitude were conducted for the bidirectional tests. Strain gages were applied in each quadrant for the first three spiral layers. The location of these strain gages directly overlaid the extreme fiber reinforcing bars subjected to the peak excursions. The reported transverse steel strain hysteresis for bidirectional and unidirectional loading appears in Figure 12.9. Although the shape of the measured response is similar, the biaxial loading resulted in an additional accumulation of hoop strain during repeated cycles at displacement amplitude of 27mm. This additional hoop strain can reduce its ability to restrain longitudinal bars from buckling as well as influence confinement effects. The reported observations indicate that this spike in measured hoop strains coincided with visible bar buckling. In comparison, bar buckling was reported one displacement amplitude larger in the uniaxial test. Osorio et al. (2012) note, “Results show that biaxial shear loading affects the shear mechanisms, producing larger transversal strains for the same load intensity and lower crack angles.” It is still important to note that the displacement amplitudes are small, and the level of confinement/restraint is low in the shear dominated columns.

Figure 12.9 Osorio et al. (2012), [Left] Biaxial Displacement History and [Right] Measured Hoop Strains for Uniaxial and Biaxial Displacement Patterns
12.3.3 Kazuhiro Tsuno and Robert Park (2004). “Experimental Study of Reinforced Concrete Bridge Piers Subjected to Bi-Directional Quasi-Static Loading”

The study considered four rectangular bridge columns subjected to different bidirectional load paths, and one repeat load history for a fifth column with a lower concrete strength. The specimens were 550mm square with an aspect ratio of 4.1, a steel content of 1.2%, a volumetric steel ratio of 1%, and a constant axial load equivalent to 4.4%. The first specimen, S1, was subjected to a symmetric uniaxial two-cycle-set load history which served as a baseline for comparison to biaxial tests. Specimen S2 was subjected to the exact opposite of the load history used for S1, with the high ductility cycles occurring first followed by a gradual decrease in ductility. Specimen S3 utilized a bi-directional orthogonal symmetric two cycle set load history, Figure 12.10. Four total cycles were conducted at each ductility level, two in the E-W direction followed by two in the N-S direction. Specimen S4 utilized a bi-directional s-shape load history, Figure 12.11. Two complete orbital paths (1-16) were completed at each ductility level. The fifth column had reduced concrete strength and was subjected to the same uniaxial load history as S1.

Tsuno and Park (2004) reported that reinforcement buckled at $\mu_{\Delta 8}$ and ruptured at $\mu_{\Delta 12}$ during the uniaxial two-cycle-set load history of S1. Spiral fracture was reported during $\mu_{\Delta 10}$. In specimen S2, bar buckling was reported during the reversal from the first cycle of $\mu_{\Delta 12}$ before the specimen reached zero displacement. Again, this was an inverse of the two-cycle-set load history which began with high ductility reversals. For bi-directional load history of specimen S3, bar buckling was reported during $\mu_{\Delta 6}$ and bar fracture during $\mu_{\Delta 8}$. For bi-directional load history of specimen S4, bar buckling was reported during $\mu_{\Delta 6}$ and bar fracture during $\mu_{\Delta 8}$. The fifth specimen, with weaker concrete and a uniaxial two-cycle-set load history had reported bar buckling during $\mu_{\Delta 6}$ and bar fracture during $\mu_{\Delta 10}$.

Tsuno and Park (2004) utilized an array of linear potentiometers to monitor curvature distributions in the main orthogonal directions of the load histories. Cross section curvature
profiles obtained from the potentiometers were used to calculate an equivalent rectangular plastic hinge length \( L_p \). \( L_p \) is the length over which plastic curvatures are assumed to remain constant, and in their formulation it includes the strain penetration component. A sample curvature profile for specimen S1, and the resulting \( L_p \) values for all of the experiments is shown in Figure 12.12. Tsuno and Park (2004) provide a summary of observations related to the influence of bi-directional loading on curvature profiles and computed plastic hinge lengths, which are repeated below.

“1. The plastic hinge zone length \( L_p \) tends to be stable at around the theoretical values after some cyclic loadings and is not affected by bi-directional loading. The plastic hinge zone length is shorter than the theoretical values until the displacement ductility factor \( \mu_\Delta \) reaches around 4. The concrete strength of a column might affect the plastic hinge zone length \( L_p \). No significant difference in the \( L_p-\mu_\Delta \) relationship was observed between tests S1-S4, which suggests that \( L_p \) is not affected by bi-directional loading after some cycles of loading.

2. If an extremely large displacement, such as \( \mu_{\Delta 12} \), for the specimens used in this research is applied to a column at the early stage of cyclic loading, it may lead to the buckling of main-bars and confinement failure with only small energy dissipation. However, as long as the displacement amplitude in the cyclic loading starts at a small level and increases step-by-step, like the standard loading pattern suggested by Park, the energy dissipation capacity of a column until the ultimate state is the same for both uni-directional and bi-directional loading.

3. The maximum displacement of a column when it reaches the ultimate state in a bi-directional cyclic loading, is smaller than that of the same column subjected to the standard uni-directional loading pattern suggested by Park.”
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Figure 12.10 Tsuno and Park (2004). Bi-Directional Load History for Specimen S3

Figure 12.11 Tsuno and Park (2004). Bi-Directional Load History for Specimen S4

Figure 12.12 Tsuno and Park (2004) Calculated Lp Values from Curvature Profiles

The research focused on the effect of three dimensional load path, including a mixture of displacement and force controlled lateral input as well as varying axial load for square columns. The columns were 250mm square and an aspect ratio of six. The cross section had 8, 16mm bars uniformly distributed around the perimeter and a double 8mm diameter hoop arrangement with 70mm spacing. The load histories utilized in the experiments are shown in Figure 12.13. Load histories S0, S1, S2, S5, S6, S7, and S8 utilized constant axial force and varying imposed lateral displacement history. Specimens S3 and S4 had a mixture of force control and displacement controlled loading histories in the orthogonal directions and a constant axial force. Test units S9, S10, and S11 had varying lateral and axial loading history. The load histories were devised to evaluate specific characteristics unique to bidirectional loading.

Bousias et al. (1995) note the following regarding the influence of load path, “The strong coupling between the two transverse directions produced an apparent reduction of strength and stiffness in each of the two transverse directions considered separately, but also increased the hysteretic energy dissipation. This increase is manifested by the larger width of the hysteresis loops in a transverse direction in the presence of a nonzero force or deflection in the orthogonal direction, as compared with the cases of cyclic uniaxial bending. Moreover, biaxial force paths are rotated with respect to the biaxial deflection paths in the sense in which these are traced, so that the vector resultant of transverse displacements always lags behind the vector resultant of transverse forces.”
Figure 12.13 [Left] Load Histories Utilized in Bousias et al. (1995) and [Right] Selected Test S7 from Bousias et al. (1995) which Demonstrates Lag of Force Resultant behind Disp. Resultant
12.4 Study Objectives

The objective of the research described in this research is to determine the impact of a 2-dimensional loading path on the definition of displacement-based performance limit states and the relationship between strain and displacement (i.e., plastic hinges). The specific issues with regard to load path are the impact of multi-directional loading on: (1) Accumulation of strain in reinforcing steel; (2) Uni-directional design (which is the normal practice); and (3) Crack formation and the plastic hinge length method for member deformations.

12.5 Research Plan

12.5.1 Task One: Detailed Literature Review

As previously noted, the research team has been studying the issue of loading history and its impact on the relationship between strain and displacement and strain limits themselves. As part of that work, a more accurate model for plastic hinge lengths in concrete bridge columns has been developed, along with a simple method for accurately predicting the force-displacement response of a column considering column flexure and strain penetration. This was possible due to the fidelity of the data that was obtained through the use of a 3D non-contact position measurement system, as well as by detailed fiber modeling of RC columns. As part of that work, a detailed literature review was conducted on the impact of load history (which will not be repeated here). In addition, pilot analytical studies were undertaken to assess the impact of loading path on the recommendations developed as part of that research. The literature review will focus on past studies on load path and its possible impacts on performance-based design. This will include an examination of different loading protocols, as well as real bi-directional EQ load histories.
12.5.2 Task Two: Load Path Analysis

Accurate fiber models capable of predicting force deformation response and local strain information were developed as part of the load history research previously conducted at NCSU. Examples of this are shown in Figure 12.14 for strain and force versus deformation response. As part of this task, the load path analysis conducted during the load history research project will be extended.

![Figure 12.14 Analytical and Experimental Comparison for (Left) Strain Hysteresis and (Right) Force-Displacement Curve](image)

12.5.3 Task Three: Experimental Studies on Columns

The experimental work will involve a series of 12 circular bridge columns, each subjected to bi-directional loading and constant axial force. The dimensions of the cross section, length of the member, and reinforcement detailing mirror previously tested specimens from Goodnight et al. (2014). The uniaxial response of similarly detailed columns serves as the basis of comparison for the proposed experiments. Other variables that will be considered include longitudinal steel ratio, transverse steel detailing, and axial load ratio. The proposed test setup is shown in Figure 12.15. The lateral load will be applied by two
hydraulic actuators, which form a 45-45-90 triangle in the X-Y plane with initially concentric lines of action. The input displacement control and post-processing of the actuator loads is defined based on the initial and deformed geometry of the setup. Through geometry, each actuator load is resolved into the X and Y components, which in turn is used to compute the resultant force on the column. A constant vertical load is maintained by a single hydraulic jack placed above the specimen which post tensions a 1 ¾” Dywidag bar located within a 3” PVC duct at the center of the cross-section. A 10”x10”x8” pocket at the bottom of the footing-column joint provides a reaction for the axial load bar.

The bi-directional loading paths will be prescribed in some cases and earthquake time history based in others, Figure 12.16. The Type-B and Type-S load paths serve as a two-dimensional extension of the symmetric-three-cycle set load history employed in Goodnight et al. (2014), Figure 12.1. The Type-B load path, Figure 12.16, consists of a reversal in the y-direction 1-4 followed by a reversal in the x-direction 5-8. The Type-S load path follows the double figure eight path 1-16 in Figure 12.16. The Type-S load history still contains defined reversals to evaluate the damage induced by the prior cycle while providing some out of plane deformation demands. The load paths induced by earthquakes are more random in nature in both path and sequence of displacement amplitudes. The load path for earthquake records will come from time history analysis of the bridge column under orthogonal directions of seismic input. The shape of the load path will be simplified into straight line approximations along the displacement orbit, so that it can be used in the lab as a displacement controlled loading procedure. Regardless of which load history is used, the test will begin in the same format. A single Type-B reversal will be completed in the following fractions of the analytical first yield force: ¼, ½, ¾, and $F_y'$. The recorded first yield displacements in each of the principle directions, N-S-E-W, will be averaged to find the experimental first yield displacement. The experimental first yield displacement is multiplied by the ratio of the nominal to the first yield moment to determine the equivalent yield displacement, $\mu_{\Delta_1} = \Delta_y' (M_{ni}/M_{y})$. The Type-B and Type-S load paths then resume with two complete reversals at each of the following displacement amplitudes: $\mu_{\Delta_1}, \mu_{\Delta_{1.5}}, \mu_{\Delta_2}, \mu_{\Delta_3}, \mu_{\Delta_4}, \mu_{\Delta_5}, \mu_{\Delta_6}, \mu_{\Delta_7},$ etc. until failure is observed.
The proposed test matrix for the 12 bi-directional load path columns is shown in Table 12.1. The geometry and reinforcement was selected to mirror that of previous uniaxial tests by Goodnight et al. (2014), shown in Table 12.2. That way, a separate uniaxial load history experiment may not be necessary to compare to the results of the biaxial tests. Although it is unlikely, if the material properties of the reinforcement vary significantly from prior experiments, Tests 5, 6, and 9 may be replaced by uniaxial load histories. Two longitudinal steel ratios will be evaluated in the test matrix, 1.6% and 2.1%. Similarly, two transverse volumetric steel ratios are included, 0.7% and 1%. The majority of the tests will have a constant level of applied axial load equivalent to $P/(f'_c A_g) \approx 9\%$, depending on the concrete compressive strength. The detailing was chosen to be representative of the well-detailed bridge columns with a flexural failure mode dictated by fracture of previously buckled longitudinal bars. The ranges of variables in the test matrix were selected to measure their influence of the observed failure mode. The specimens will be constructed at the NCSU lab and will utilize the same three-dimensional position monitoring system employed during Goodnight et al. (2014), Figure 12.2.
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Figure 12.15 Sample Test Setup and Positioning on the Lab Strong Wall and Floor

Figure 12.16 (Left) Prescribed Load History and (Right) Load History from Japan Earthquake 2011
Chapter 12: Future Research on the Effects of Seismic Load Path

Table 12.1 Proposed Test Configurations

<table>
<thead>
<tr>
<th>Test</th>
<th>Load Path</th>
<th>Long. Steel</th>
<th>Trans. Steel</th>
<th>Axial</th>
<th>Cross Section Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Type-B</td>
<td>16 #7 (2.1%)</td>
<td>#3@2&quot; (1%)</td>
<td>9%</td>
<td>24in Dia. and 8ft Length</td>
</tr>
<tr>
<td>2</td>
<td>Type-S</td>
<td>16 #7 (2.1%)</td>
<td>#3@2&quot; (1%)</td>
<td>9%</td>
<td>0.5in Cover to Outside of Spiral</td>
</tr>
<tr>
<td>3</td>
<td>Type-B</td>
<td>16 #7 (2.1%)</td>
<td>#3@2.75&quot; (.7%)</td>
<td>9%</td>
<td>3in Dia. PVC Duct at Center for</td>
</tr>
<tr>
<td>4</td>
<td>Type-S</td>
<td>16 #7 (2.1%)</td>
<td>#3@2.75&quot; (.7%)</td>
<td>9%</td>
<td>1 ¾” Dia. Dywidag Axial Bar</td>
</tr>
<tr>
<td>5</td>
<td>Type-S</td>
<td>16 #7 (2.1%)</td>
<td>#3@2&quot; (1%)</td>
<td>4%</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Type-S</td>
<td>16 #7 (2.1%)</td>
<td>#3@2.75&quot; (.7%)</td>
<td>4%</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Type-B</td>
<td>16 #6 (1.6%)</td>
<td>#3@2&quot; (1%)</td>
<td>9%</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Type-S</td>
<td>16 #6 (1.6%)</td>
<td>#3@2&quot; (1%)</td>
<td>9%</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>ACI374/EQ1</td>
<td>16 #6 (1.6%)</td>
<td>#3@2&quot; (1%)</td>
<td>9%</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>EQ2</td>
<td>16 #6 (1.6%)</td>
<td>#3@2&quot; (1%)</td>
<td>9%</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>EQ3</td>
<td>16 #6 (1.6%)</td>
<td>#3@2&quot; (1%)</td>
<td>9%</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>EQ4</td>
<td>16 #6 (1.6%)</td>
<td>#3@2&quot; (1%)</td>
<td>9%</td>
<td></td>
</tr>
</tbody>
</table>

Table 12.2 Uniaxial Experiments from Goodnight et al. (2014) Serve as Comparison

<table>
<thead>
<tr>
<th>Test</th>
<th>Load Path</th>
<th>Long. Steel</th>
<th>Trans. Steel</th>
<th>Axial</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>3-Cycle-Set</td>
<td>16 #6 (1.6%)</td>
<td>#3@2&quot; (1%)</td>
<td>5.4%</td>
</tr>
<tr>
<td>8</td>
<td>Chile (2010)</td>
<td>16 #6 (1.6%)</td>
<td>#3@2&quot; (1%)</td>
<td>5.4%</td>
</tr>
<tr>
<td>10</td>
<td>Chichi (1999)</td>
<td>16 #6 (1.6%)</td>
<td>#3@2&quot; (1%)</td>
<td>7.1%</td>
</tr>
<tr>
<td>11</td>
<td>Kobe (1995)</td>
<td>16 #6 (1.6%)</td>
<td>#3@2&quot; (1%)</td>
<td>6.2%</td>
</tr>
<tr>
<td>12</td>
<td>Japan (2011)</td>
<td>16 #6 (1.6%)</td>
<td>#3@2&quot; (1%)</td>
<td>6.2%</td>
</tr>
<tr>
<td>13</td>
<td>3-Cycle-Set</td>
<td>16#6 (1.6%)</td>
<td>#4@2.75&quot; (1.3%)</td>
<td>6.2%</td>
</tr>
<tr>
<td>14</td>
<td>3-Cycle-Set</td>
<td>16#6 (1.6%)</td>
<td>#3@4&quot; (0.5%)</td>
<td>5.7%</td>
</tr>
<tr>
<td>15</td>
<td>3-Cycle-Set</td>
<td>16#6 (1.6%)</td>
<td>#3@2.75&quot; (0.7%)</td>
<td>5.2%</td>
</tr>
<tr>
<td>16</td>
<td>3-Cycle-Set</td>
<td>16#6 (1.6%)</td>
<td>#3@1.5&quot; (1.3%)</td>
<td>5.6%</td>
</tr>
<tr>
<td>17</td>
<td>Chile (1985)</td>
<td>16#6 (1.6%)</td>
<td>#3@1.5&quot; (1.3%)</td>
<td>5%</td>
</tr>
<tr>
<td>18</td>
<td>Darfield (2010)</td>
<td>16#6 (1.6%)</td>
<td>#3@1.5&quot; (1.3%)</td>
<td>4.8%</td>
</tr>
<tr>
<td>25</td>
<td>3-Cycle-Set</td>
<td>16 #7 (2.1%)</td>
<td>#3@2&quot; (1%)</td>
<td>5%</td>
</tr>
<tr>
<td>26</td>
<td>3-Cycle-Set</td>
<td>16 #7 (2.1%)</td>
<td>#3@2&quot; (1%)</td>
<td>10%</td>
</tr>
<tr>
<td>27</td>
<td>3-Cycle-Set</td>
<td>16 #6 (1.6%)</td>
<td>#3@2&quot; (1%)</td>
<td>10%</td>
</tr>
</tbody>
</table>
12.5.4 Task Four: Analysis of Data and Model Calibration

The experimental data will serve multiple purposes. First, it will provide information that may be directly used as design recommendations (for example, strain limits). It also will provide data for model calibration that is then used for the studies described below (such as for plastic hinge length and location).

12.5.5 Task Five: Recommendations

The previous load history research project has developed recommendations for strain limits at key performance limit states as well as more accurate models for correlating strains to displacement that are consistent with the measured data. Based on the work of the load path project, it is the hope that the recommendations from the previous study (Goodnight et al., 2014; Feng et al. 2014a, b, c) will either be confirmed, or modified to consider load path effects. In addition, this work will provide recommendations on how best to establish uni-directional target displacements that consider the impacts of 2D load path. These recommendations, which will be based on 2D nonlinear time history analysis, will likely take the form of amplification of the uni-directional target displacement to account for multi-directional loading.
REFERENCES


Henry, L. and Mahin (1999). Study of Buckling of Longitudinal Bars in Reinforced Concrete Bridge Columns. Report to the California Dept. of Transportation


References


