ABSTRACT

COOKSON, KENDRA ANN. Seismic Performance of Steel Bridge Bent Welded Connections. (Under the direction of Dr. Mervyn Kowalsky.)

The objective of this research is to evaluate the seismic performance of steel bridge bent welded connections. Little is known about these types of systems in regards to their application in seismic regions such as Alaska. The focus of the research was with regard to use of these structures in the State of Alaska.

The research includes both an experimental and analytical portion. The experimental portion consisted of four full scale subassembly bridge bents tested under seismic loading. The current practice in Alaska, a fillet welded bridge bent, was evaluated as well as 2 additional weld configurations. The two additional weld configurations are a complete joint penetration weld with a reinforcing fillet and a simple complete joint penetration weld. All four specimens were tested under reverse cyclic loading applied by a hydraulic actuator. The analytical portion consisted of applying the concepts of direct displacement based design in order to evaluate the test results.

The results of the experiments show that that current practice in Alaska is not adequate for the level of seismic intensity throughout the state. The complete joint penetration weld with reinforcing fillet was able to achieve moderate displacement capacity with the possibility of being used in some of the lower seismic regions of the State. The simple complete joint penetration weld was only able to achieve low level of displacement capacity.
Seismic Performance of Steel Bridge Bent Welded Connections.

by
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DEDICATION

To all the people who helped keep me going,

Without your love and support this would have never been possible.

Special thanks to my parents, Pamela and John

Who always believed in me.
Kendra A. Cookson was born in San Luis Obispo, California in 1982. She received her Bachelor’s of Science in Civil Engineering at California Polytechnic State University in 2007. From there, she then went on to get her Master’s of Science at North Carolina State University in 2009.
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Introduction

1.1 Description of bridge type

Small, light frame steel bridges are used all over the state of Alaska. They are used in everything from marine wharfs to rural countryside and highway overpasses. These light frame bridges typically consist of steel pipe piles welded to a steel HP section cap as shown in Figure 1.1 through Figure 1.4. The super structure of these bridges may be constructed with a variety of different material but this will have no impact on the performance of the supporting bents.
Figure 1.1: False Pass City Dock Bridge in Alaska

Figure 1.2: 76th Ave underpass in Alaska
Figure 1.3: Bodenburg Creek Bridge in Alaska

Figure 1.4: Glacier Fork of Salmon Creek Bridge in Alaska
1.2 The Connection

The connection between the pipe pile and cap beam is typically a field fillet weld. The details of the connection will change from bridge to bridge but they all are always a field fillet welds. The following figures are of actual Alaska bridge details. Figure 1.5 shows a weld detail for Bodenburg Creek Bridge. It has a 3/8” fillet weld between the pile and a top plate, then another 3/8” field fillet weld between the top plate and cap beam. In Figure 1.6 a detail of the 76th Avenue underpass details a field fillet weld connecting the pile to the pile cap and another 3/8” field fillet connecting the pile cap to the cap beam. In Figure 1.7, the weld detail for the Lowell Creek Bridge is shown as a 3/8” field fillet weld and does not contain the pile top plate as in other connections. While all these examples show a 3/8” field fillet weld, Table 1.1 shows a database of bridges with different connection details.

Figure 1.5: Bodenburg Creek Bridge weld detail
Figure 1.6: 76th Ave underpass weld detail
Figure 1.7: Lowell Creek Bridge weld detail
1.3 Database of bridges

The following table is a representative sampling of bridges in Alaska provided by the Department of Transportation. Of each type of bridge listed, there are at least 50 to 60 bridges throughout the state. There are also numerous marine structures that exhibit similarities to this light weight steel system. Note that all welds are field fillet welds.

<table>
<thead>
<tr>
<th>Name</th>
<th>Weld Type</th>
<th>Weld Size [in]</th>
<th>Pile Diam.</th>
<th>Pile thickness [in]</th>
<th>Pile Height above ground [ft]</th>
<th># of Piles per bent</th>
<th>Cap Beam</th>
<th># of Spans</th>
<th>Span Length [ft]</th>
<th>Location</th>
<th>Figure reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>208</td>
<td>Field Fillet</td>
<td>¼</td>
<td>12”</td>
<td>unknown</td>
<td>10</td>
<td>4</td>
<td>HP14x73</td>
<td>3</td>
<td>75</td>
<td>57.618</td>
<td>-152.315</td>
</tr>
<tr>
<td>1196</td>
<td>Field Fillet</td>
<td>¼</td>
<td>12”</td>
<td>0.833</td>
<td>14</td>
<td>4</td>
<td>HP14x73</td>
<td>3</td>
<td>33</td>
<td>59.478</td>
<td>-139.608</td>
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<tr>
<td>1754</td>
<td>Field Fillet</td>
<td>⅜</td>
<td>30”</td>
<td>unknown</td>
<td>16.5</td>
<td>4</td>
<td>2W36x280</td>
<td>3</td>
<td>50</td>
<td>61.150</td>
<td>-149.700</td>
</tr>
<tr>
<td>1820</td>
<td>Field Fillet</td>
<td>⅜</td>
<td>16”</td>
<td>unknown</td>
<td>20</td>
<td>4</td>
<td>2HP10x57</td>
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<td>35</td>
<td>60.178</td>
<td>-149.365</td>
</tr>
<tr>
<td>1136</td>
<td>Field Fillet</td>
<td>⅜</td>
<td>16”</td>
<td>0.5</td>
<td>10</td>
<td>2</td>
<td>2HP14x89</td>
<td>1</td>
<td>80</td>
<td>60.105</td>
<td>-149.448</td>
</tr>
<tr>
<td>Name</td>
<td>Weld Type</td>
<td>Weld Size [in]</td>
<td>Pile Diam.</td>
<td>Pile thickness [in]</td>
<td>Pile Height above ground [ft]</td>
<td># of Piles per bent</td>
<td>Cap Beam</td>
<td># of Spans</td>
<td>Span Length [ft]</td>
<td>Location</td>
<td>Figure reference</td>
</tr>
<tr>
<td>--------</td>
<td>-----------</td>
<td>----------------</td>
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<td>--------------------</td>
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<td>----------</td>
<td>------------</td>
<td>----------------</td>
<td>----------</td>
<td>-----------------</td>
</tr>
<tr>
<td>1945</td>
<td>Field Fillet</td>
<td>$\frac{5}{16}$</td>
<td>20”</td>
<td>0.625</td>
<td>20</td>
<td>3</td>
<td>2W18x36</td>
<td>23</td>
<td>30</td>
<td>54.852</td>
<td>-163.408</td>
</tr>
<tr>
<td>1714</td>
<td>Field Fillet</td>
<td>$\frac{3}{8}$</td>
<td>12”</td>
<td>0.375</td>
<td>unknown</td>
<td>2</td>
<td>W24x84</td>
<td>1</td>
<td>74</td>
<td>61.560</td>
<td>-149.038</td>
</tr>
</tbody>
</table>
1.4 Seismic Hazard

The earth's most active seismic feature, the circum-Pacific seismic belt, brushes Alaska and the Aleutian Islands, where more earthquakes occur than in all other 49 states combined. More than 80 percent of the planet's tremors occur in the circum-Pacific belt, and about six percent of the large, shallow earthquakes are in the Alaska area, where as many as 4,000 earthquakes at various depths are detected each year. Alaska has two primary seismic zones; the first is the Aleutian Island Arc, one of the planet's most active seismic areas, which extends about 2,500 miles, from Fairbanks in central Alaska through the Kenai Peninsula to the Near Islands. It maintains a width of nearly 200 miles throughout most of the zone. The second zone begins north of Yakutat Bay in southeastern Alaska and extends southeastward to the west coast of Vancouver Island, Canada. Alaska is considered a high seismic region with some of the largest recorded events in history occurring there. On March 27, 1964, Alaska's most famous earthquake shook the state and is now known as the Great Alaska Earthquake. Registering 9.2 on the Richter Scale, this earthquake was not only the largest in state history but the largest earthquake in US history. Alaska is home to both large earthquakes and frequent ones as well. On average, a magnitude 7.0 or larger earthquake occurs every two years in Alaska ("Alaska.", USGS).
1.5 Need for research

The aforementioned small, light frame steel bridges are found throughout Alaska and the entire Pacific Northwest. Canada and Washington State both have structures similar to those found in Alaska. Despite their popularity, little has been done by way of research regarding the performance of such structures.

In British Columbia, Steunenbrug, Sesmith and Stiemer investigated the seismic behavior of steel piles to precast cap beam connections (Steunenbrug et al., 1998). The system they investigated was a steel pipe pile welded to an embedded steel plate in a precast concrete cap beam. The steel plate is anchored in the beam by means of reinforcing bars welded to the plate. They tested one full size pile segment under reverse cyclic loading. The connection between the plate and the pipe pile section was a complete joint penetration weld preformed overhead to simulate field welding. The specimen failed in the desired mode of plastic hinging in the pile. The connection exhibited strength and ductility, reaching a displacement ductility over eight.

1.6 Problem description and scope of the research

Seismic design of bridges is often based on capacity design principles where the strength hierarchy is established in a bridge to ensure that the damage is controllable, and occurs only where the designer intends (Priestley, 1996). Special attention is paid to the ductility of structural members that have been selected to develop plastic hinges; members
should be able to sustain large inelastic deformations. All other members should be designed to remain elastic while resisting the overstrength moments coming from adjacent members.

Little is known about the seismic performance of steel bridge bent structures in high seismic regions. In particular, the performance of the welded connection between the piles to cap beam is of great interest. In order for a plastic hinge to form in the piles the weld joint must remain elastic. This research aims to assess the current design practice in Alaska. If it is determined that the current design performs well, then the addition of field variables, such as misalignment of the cap beam over the piles, will be added to the tests. On the contrary, if the current practice is deemed unacceptable, designing a detail that will allow for plastic hinges to form in the piles will be the focus. The overall goal of this research is to obtain a weld detail that will allow the structure to adhere to capacity design principles such that plastic hinges form in the piles and all other regions remain in the elastic range. In line with this goal, four full-scale subassembly bridge bents were constructed and tested under reverse cyclic loading at North Carolina State University Constructed Facilities Laboratory (CFL). Analysis of the test results yielded information on the strains, curvature as well as the plastic hinge length. Using direct displacement based design, analysis was done to determine the level of seismic intensity that would cause the structure to fail.

1.7 General organization

Chapter I contains general introduction to the research. As well as background on the seismic hazard present in Alaska.
Chapter II contains a description of the specimen design, test setup and instrumentation. Information related to test fixtures, application of load as well as general overview of tests.

Chapter III presents the analysis of the test data. Information such as load displacement hysteresis, strain profiles and curvature profiles can be found.

Chapter IV contains seismic analysis of the results from tests 2 and 4. Demand and capacity analysis are performed. Design charts and examples are located in this chapter.

Chapter V contains the results and recommendations. The future work is also described in this chapter.
CHAPTER II

Experimental Program

CHAPTER 2

2.1 Introduction

The experimental program consists of specimen design, test setup, and instrumentation. An overview of each test is given, highlighting the major events of each test, force displacement hysteresis and photos.

2.2 Specimen Design

For the specimen design there were three primary points to consider, (1) actual Alaska bridges, (2) capacity design, and (3) testing lab restrictions. By looking at an inventory of bridges in Alaska and talking with members of the Alaska DOT, common aspects were determined and considered in the test unit design. Table 2.1 shows section information for seven bridges throughout the state which were considered a representative sample. From this information a round Hollow Structural Section (HSS) was chosen for the columns and a double HP section for the cap beam. The HSS section needed to be in the range of what was common and a diameter of 16” was chosen. The wall thickness was
determined using the typical AKDOT diameter to thickness ratio of 32 resulting in a thickness of $\frac{1}{2}''$ for the columns.

<table>
<thead>
<tr>
<th>Column Section</th>
<th>Column Height [ft]</th>
<th>Cap Beam section</th>
</tr>
</thead>
<tbody>
<tr>
<td>12'' CIP concrete</td>
<td>10</td>
<td>HP 14x73</td>
</tr>
<tr>
<td>12''x5/6'' CIP</td>
<td>14</td>
<td>HP 14x73</td>
</tr>
<tr>
<td>30''d concrete</td>
<td>16.5</td>
<td>2W36x280</td>
</tr>
<tr>
<td>36''</td>
<td>16</td>
<td>N/A</td>
</tr>
<tr>
<td>16''</td>
<td>20</td>
<td>2W36x280</td>
</tr>
<tr>
<td>16''x 0.5''</td>
<td>10</td>
<td>2HP14x89</td>
</tr>
<tr>
<td>20''x 0.625''</td>
<td>20</td>
<td>2W18x36</td>
</tr>
</tbody>
</table>

The height of the columns in most bridge structures ranges between 10-30 feet. A choice was made to not model an entire bridge structure, but rather from the point of inflection up, as seen in Figure 2.1. The point of inflection is approximately 10 feet above ground making our target height 10-14 feet. Typical bridges have multiple bents and varying heights. By modeling one full two column bent, the effects of axial load are included in the tests. The length between columns was not seen as critical for this reason and 11’-8” center to center was chosen. The height of the column was selected to be 11 feet from the point of inflection to the bottom of the cap beam.
In accordance with capacity design principles, the plastic hinge is to form at the top of the columns while the cap-beam remains elastic; plastic hinges may also form at the base or below grade. For a given column cross section, the following calculations were conducted: The over strength column moment was found using Equation (2.2), where $\phi^o$ is an over strength factor chosen as 1.3. The moment at the centerline of the cap beam was found using Equation (2.3) and then adjusted by Equation (2.4) to find the moment at the face of the column. From here a moment demand of 625kip-ft for a double HP section was obtained. The yield moment of one HP 14x89 is 546 kip-ft as shown in Equation (2.1). Since there are two HP sections, the total beam elastic moment capacity is 1092 kip-ft. Comparing this to the plastic moment input from the column, which from Equation (2.4) is 566 kip-ft, the over
The strength factor is $1092/566 = 1.9$. This is significantly greater than 1; therefore, the design meets capacity design requirements.

$$M_{\text{elastic cap beam}} = S_x f_y = \left(131\text{in}^3\right)(50\text{ksi}) = 546 \text{ kip-ft} \quad (2.1)$$

$$M_{\text{overstrenth capacity of column}} = Z_x f_y \phi^o = \left(112\text{in}^4\right)(50\text{ksi})(1.3) = 607 \text{ kip-ft} \quad (2.2)$$

$$M_{\text{Centerline of cap beam}} = \frac{H_{\text{Centerline}}}{H_{\text{Clear}}} M_{\text{Top of column}} = \frac{139''}{132''} (607\text{k-ft}) = 639 \text{k-ft} \quad (2.3)$$

$$M_{\text{Cap Beam at column face}} = \frac{L_{\text{Clear}}}{2} M_{\text{Centerline}} = \frac{10'-4''}{2} (639\text{kip-ft}) = 566\text{kip-ft} \quad (2.4)$$

With the sections selected, the next step involved considerations of laboratory capacity. The base supports were designed for use in another project and therefore their dimensions were known in advance. The CFL floor and wall spacing is on a 3’ grid, as shown in Figure 2.2. As a result, the test specimen must follow this hole arrangement. The centerline distance between columns was 11’-8” which was close enough to 12’ that the base supports could accommodate the 4” difference with little modification. The height of the columns had to be determined such that the actuator centerline follows to the cap beam centerline as shown in Figure 2.3. With the support height known, along with the possible
actuator locations on the strong wall, a small adjustment to the height was made resulting in a height of 11’-7 3/8” as shown in Figure 2.3.
To be able to test the specimens to high levels of ductility, the actuator stroke was checked such that a displacement ductility of 8 could achieved. Displacement ductility is defined in Equation (2.5), where the # can be any integer value; such that a ductility of 2 is twice the yield. The yield displacement, $\Delta_y$, is defined by Equation (2.6), where $\Delta'_y$ is defined by Equation (2.7) and Equation (2.8), $M_p$ is the plastic moment from Equation (2.9) and $M'_y$ is the yield moment from Equation (2.10). With a yield displacement of 1.75 inches, $\mu 8$ is equal to 18 inches, within the 20 inch actuator stroke in both the push and pull directions.

$$\mu # = # \times \Delta_y$$  \hspace{1cm} (2.5)

$$\Delta_y = \frac{M_p}{M'_y} \times \Delta'_y = \frac{470 \text{ kip-ft}}{360 \text{ kip-ft}} \times 1.31 \text{ in} = 1.75 \text{ in}$$  \hspace{1cm} (2.6)

$$\Delta'_y = \frac{\phi'_y L^2}{3} = \frac{0.000229 / \text{in} \ (131.125 \text{ in})^2}{3} = 1.31 \text{ in}$$  \hspace{1cm} (2.7)

$$\phi'_y = \frac{M'_y}{EI} = \frac{4285 \text{ kip-in}}{(29000 \text{ ksi})(685 \text{ in}^4)} = 0.000229 / \text{in}$$  \hspace{1cm} (2.8)

$$M_p = Z'_xf_y = (112 \text{ in}^3)(50 \text{ ksi})\left(\frac{1 \text{ foot}}{12 \text{ in}}\right) = 470 \text{ kip-ft}$$  \hspace{1cm} (2.9)

$$M'_y = S'_xf_y = (85.7 \text{ in}^3)(50 \text{ ksi})\left(\frac{1 \text{ foot}}{12 \text{ in}}\right) = 360 \text{ kip-ft}$$  \hspace{1cm} (2.10)

The last few details are the cap beam stiffeners and the overhang of the cap beam. The cap beam had stiffeners added above the columns to add rigidity to the joint and to help
keep the joint elastic. This is a common detail in rectangular and I-section columns. The stiffeners help engage both flanges to take the moment at the joint as well. The overhanging portion of the cap beam was added in order to meet the actuator at mid stroke to ensure a full 20 inches stroke in both the push and pull directions.
Figure 2.4: Test specimen
2.3 Test Setup

The test setup was designed to allow the application of lateral loads to the cap beam while avoiding transverse or out of plane displacement. The three major components of the test setup were the two pin supports, the out of plane frames, and the application of the lateral load.

2.4 Pin Supports

In order to have the structure displace as it would in the field, pin supports were utilized to mimic the point of inflection above the foundation. As mentioned before, these supports were designed for use in another project and exceeded the capacity needed for these tests. The supports consisted of a number of W-sections, two shoes and a steel pin. An elevation drawing of the supports can be seen in Figure 2.5. A 5 ½ inch pin is inserted through two sleeves with interior diameters of 5.502 inches. The shoes rest on rocker bearings, and are bolted to the base. The base consists of four W14x159; two on the bottom spaced 3 feet center to center with two more stacked perpendicular 2 feet 4 inches center to center, as seen in Figure 2.6. The bottom two W-sections are secured to the strong floor with four Dywidag post-tensioning bars with a diameter of 1-3/8 inches postensioned to approximately 50 kips. For detailed shop drawings of the pin supports please see Appendix 6.1.
Figure 2.5: Elevation view of pin support

Figure 2.6: Top view of pin support
2.5 Out of Plane Support

When applying the lateral load to the test unit, out of plane movement is possible. In order to minimize this movement, guide frames were used. The out of plane support consisted of four columns, two cross beams, 2 K braces and four support rollers. In Figure 2.7 a view of the cross beam and columns can be seen. The K bracing as well as the roller supports can be seen in Figure 2.8.

Figure 2.7: East west view of lateral support frames
The lateral cyclic load was applied using a 220 kip actuator with a 220 kip capacity in compression and 160 kip in tension. The actuator was placed horizontally and hung off a strong wall as seen in Figure 2.9. Chains were used to keep it horizontal between tests, but removed during loading. Due to the geometry of the test unit an adaptor piece, or shoe, was needed to connect the actuator to the cap beam, as seen in Figure 2.10. The shoe consisted of two 1 inch thick plates, labeled plate A and B, held apart by plates C and D. Plate A has hole spacing that matches up with the end of the actuator and was attached using 4, 1-½ in diameter threaded rods. Plate B has spacing to match the end plate of the cap beam and was
attached using four 1 in diameter A490 structural steel bolts. For dimensions and detail drawings see Appendix 6.2.

Figure 2.9: Actuator attached to strong wall
Directions are shown in Figure 2.11 is based upon compass coordinates, where the strong wall is north. Columns are referred to as north or south, where the north column is the northernmost column. Loading of the structure is in the direction of the actuator stroke and can be seen in Figure 2.11. In general terms the instruments used were strain gauges, string potentiometers, and a motion capturing system called Optotrack.
Figure 2.11: Elevation of test unit and loading directions
2.7.1 Strain Gauges

Strain gauges were used to measure the strain in the longitudinal direction on the columns and the cap beam. In the first test, thirty-two strain gauges were used with four of them in the transverse direction on the columns, twenty-four in the longitudinal direction and the remaining four on the cap beam. The remaining tests had twenty-six strain gauges with twenty-four strain gauges on the columns and two strain gauges on the cap beam near the column. The column strain gauges are labeled based on which column they are on (north or south), the face of the column (north, south or middle if in between the extreme fibers), and then the distance from the cap beam to the strain gauge. For instance, N-S-12 is on the north column’s south face and twelve inches down from the cap beam. The two transverse strain gauges are labeled the same way but have a T following the standard designation. The cap beam strain gauges are labeled as CB followed by the column it is near, either north or south and if it is on the top or bottom. Figure 2.12 shows the strain gauge distribution for test 1 and Figure 2.13 show the strain gauge distribution for the remaining three tests.
Figure 2.12: Test 1 strain gauge distribution
<table>
<thead>
<tr>
<th></th>
<th>North</th>
<th>CB N</th>
<th>CB S</th>
<th>South</th>
</tr>
</thead>
<tbody>
<tr>
<td>N-N-3</td>
<td>N-S-3</td>
<td>S-S-3</td>
<td>S-S-3</td>
<td></td>
</tr>
<tr>
<td>N-N-11</td>
<td>N-S-11</td>
<td>S-N-11</td>
<td>S-S-11</td>
<td></td>
</tr>
<tr>
<td>N-N-19</td>
<td>N-S-19</td>
<td>S-N-19</td>
<td>S-S-19</td>
<td></td>
</tr>
<tr>
<td>N-N-26</td>
<td>N-S-26</td>
<td>S-N-26</td>
<td>S-S-26</td>
<td></td>
</tr>
<tr>
<td>N-N-34</td>
<td>N-S-36</td>
<td>S-N-34</td>
<td>S-S-34</td>
<td></td>
</tr>
<tr>
<td>N-N-70</td>
<td>N-S-70</td>
<td>S-N-70</td>
<td>S-S-70</td>
<td></td>
</tr>
</tbody>
</table>

Figure 2.13: Tests 2 through 4 strain gauge layout
2.7.2 String potentiometers

String potentiometers ranging from 1 to 50 inches in length were used to measure the displacement in the direction of loading. They were extended using a leader and attached to the test unit with aluminum angles secured with epoxy to the surface. In tests 1 and 2, four string potentiometers were used; two were on the cap beam, one at mid-height and one at the base. Tests three and four had only one string potentiometer at the cap beam and one at the base. The string potentiometers were used to track the displacement of the cap beam and monitor any movement of the base supports.

Figure 2.14: String potentiometer locations
2.7.3 Optotrack

The Optotrack system is a motion capturing system. The system consists of a camera, markers, strobers and data acquisition. The markers are light emitting diodes (LED) that emit light that is collected by the camera. The markers are attached to a strober and the strober is connected to the data acquisition. The camera is also connected to the data acquisition. Only one LED is activated at a time and a series of markers will activate in succession, called ‘strobing’. ‘Strobing’ occurs in the same sequence each time; therefore each marker has its own marker time period when it emits light that is collected by the camera. The camera will collect the data and send it to the data acquisition. The data acquisition system recognizes the marker based upon its marker period and records the location of each marker at user defined intervals.

In all four tests, markers were placed only on the south column and the cap beam above the south column. In test 1, twenty-four markers were used and in the other three tests approximately sixty-five markers were used. The markers were hot glued to the surface of the column and cap beam, except on north and south faces of the columns. On the north and south faces, aluminum angels were attached with epoxy to the column and the markers were then hot glued to the angles. This was done because the marker must emit light in the direction of the camera in order to be recorded. A general marker layout can be seen in Figure 2.15 but the layout for each test varied and will be shown in detail in Chapter 4. The
data collected was used to calculate the cap beam displacement, pile longitudinal strain, and pile curvature.

Figure 2.15: General marker layout
Obtaining strain from Optotrack

The engineering strain over a gauge length is the change in length divided by the original length. To use the Optotrack data to perform this calculation, an initial length, $L_0$, and a time dependent length $L'$ is obtained as shown in Figure 2.16. In order to calculate the strain, the change in length is found using Equation (2.11) and the strain is found using Equation (2.12).

Figure 2.16: Graphical representation of strain calculation
\[ \Delta L = L' - L_o \]  

(2.11)

\[ \varepsilon = \frac{\Delta L}{L_o} \]  

(2.12)
Obtaining curvature from Optotrack

Average curvature in the ith cell, in Figure 2.17, in positive bending as shown in Figure 2.18, is calculated using Equations (2.13) through (2.16). \( \Delta N_i \) and \( \Delta S_i \) are the change in length on the north and south sides of the column measured from the Optotrack markers for ith cell. \( D_i \) is the horizontal distance between markers and \( G_i \) is the vertical gauge length for cell_i.

\[
\Delta N_i = L_N - G_i \quad (2.13)
\]

\[
\Delta S_i = L_S - G_i \quad (2.14)
\]

\[
\theta_i = \frac{|\Delta S_i| + |\Delta N_i|}{D_i} \quad (2.15)
\]

\[
\phi_i = \frac{\theta_i}{G_i} \quad (2.16)
\]
Figure 2.17: Cell Diagram

Figure 2.18: Positive curvature in Cell$_i$
Determining Local Buckling with Optotrack

The local buckling of the piles can be found with the Optotrack data directly. First, a plot of the pile profiles is plotted to narrow down the data range in which local buckling is occurring. In Figure 2.19 marker 2 has moved farther to the right than marker 1, indicating local buckling. Once local buckling has been identified a plot of the pile profile is made to confirm that buckling has indeed occurred.

Figure 2.19: Buckling profile
2.8 Test Procedure

Each specimen was tested quasistatically. The process consisted of pushing and pulling in load control mode at quarter yield intervals to the first yield force. Subsequent cycling was in displacement control to reach prescribed ductility levels. A typical loading history can be seen in Figure 2.20. The lateral force at first yield $F'_y$ was found through sectional analysis. The yield force was applied. Then the displacement from the Optotrack readings for the first yield displacement, $\Delta'_y$ was determined using Equation (2.17) through (2.19). From there the different ductility levels are calculated using (2.20) where $x$ is the ductility level of interest. Each ductility level is cycled through three times, where each cycle consists of a push and a pull on the specimen. The terminology used to identify the ductility levels are $\mu_#^a$ where the first number indicates the ductility level and the second number (the subscript) is the cycle number of that ductility. If the subscript is positive that indicates a push cycle and a negative number indicates a pull.

\[
\Delta_y = \frac{M_p}{M'_y} \times \Delta'_y \quad (2.17)
\]

\[
M_p = Z_x \times f_y \quad (2.18)
\]

\[
M'_y = S_x \times f_y \quad (2.19)
\]

\[
\mu x = x \times \Delta_y \quad (2.20)
\]
2.9 Test unit construction

Typical construction sequences for a real bridge are to drive the piles driven to a prescribed depth, cut the piles at the proper height, place the cap beam on the piles and finally weld the connection. Overhead field welding in adverse conditions can have an impact on the quality of the weld. Quality control and quality assurance are incorporated in the construction process to ensure a quality weld. The quality control is the responsibility of the contractor and quality assurance lies with client or owner of the structure. Tests 1 through 4 had quality control and quality assurance as a part of the test unit’s construction. Details of the quality control and quality assurance can be found in Appendix 6.3 through 6.6.
The construction of the test units needed to simulate field conditions to the extent possible. The construction sequence of each specimen followed that of a real bridge. First the piles were put in place on the supports and held vertically as seen in Figure 2.21. The cap beam was brought in overhead and placed on top of the piles as seen in Figure 2.22. Lastly an overhead weld was placed connecting the cap beam to the piles as seen in Figure 2.23, thus mimicking the actual construction of a bridge bent.

Figure 2.21: Piles in place
Figure 2.22: Cap beam being put on top of piles

Figure 2.23: Overhead welding of a test specimen
2.10 Test 1

The objective of test 1 was to assess the behavior of a bent that was designed and built using the best ‘current-practice’ of AKDOT. From the typical drawings obtained from AKDOT, it was determined that a fillet weld is the common connection between the pipe column and cap beam for these types of structures. A detail of the weld connection can be seen in Figure 2.24. The size of the fillet weld chosen below was determined by a desire to match the throat thickness with the column wall thickness. A simple check of the weld in bending predicted a failure of the weld prior to developing the full strength of the column section. While increasing the weld size to meet the bending strength of the section was an option, determining the level of performance of the current practice was the goal of test 1. In Table 2.2 a summary of the events that occurred in the test can be seen.

![Figure 2.24: Test 1 connection detail](image-url)
Table 2.2: Test 1 summary

<table>
<thead>
<tr>
<th>Ductility Cycle</th>
<th>Load [kips]</th>
<th>Displacement [inches]</th>
<th>Plan view*</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>μ₂₁</td>
<td>74</td>
<td>6.4</td>
<td><img src="image" alt="Diagram" /> South column</td>
<td>Crack appeared to originate at the bottom of weld</td>
</tr>
<tr>
<td>μ₂₋₁</td>
<td>59</td>
<td>7.1</td>
<td><img src="image" alt="Diagram" /> South column</td>
<td>Crack in heat affected zone below weld and opened up approx. ¼ “</td>
</tr>
</tbody>
</table>

*Red lines in plan view indicate location, beginning and end of crack

The yield displacement, $\Delta_y$, for this test was 2.49 inches; the equivalent yield displacement, $\Delta'_y$, using Equation (2.17) is 3.24 inches. As a result the defined ductility values were $\mu_1$ (3.24 in.), $\mu_{1.5}$ (4.86 in.), $\mu_2$ (6.48 in.), $\mu_3$ (9.72 in.), and $\mu_4$ (12.96 in.) using Equation (2.20). In this test, the load controlled cycles showed no damage to the specimen. The same observation was made for the $\mu_1$ cycles. During the $\mu_2$ cycle, an initial crack was seen in the weld throat. The crack occurred at the bottom of the weld at a load of 74k and displacement of 6.42 inches; seen with in the box A in Figure 2.25. The failure occurred in $\mu_2$; a crack on the tension face was seen at a load of 59 kips and a displacement of 7.1 inches. Upon closer inspection it was seen that the crack was in the heat affected zone just below the weld, as seen in Figure 2.26.
The force-displacement response of test 1 can be seen in Figure 2.27. It should be noted that the cycles in the push and pull direction do not appear to be to the same displacements, this is because the base moved during the test; the structural displacement is the cap beam displacement minus the base displacement. The force-displacement hysteresis shows the structural displacement. Figure 2.28 shows the base displacement during the test. The base movement yielded slightly different displacements from one cycle to the next. The base displacement was reduced in subsequent tests by removing the rocker bearings beneath the shoes of the pin supports and using an impact wrench to tighten the bolts that hold the shoes in place.

With such a low ductility level of the test unit, the current practice was determined to be inadequate to the point that Alaska DOT put a moratorium on the construction of all bridges of this type pending the outcome of the research. This has great significance due to the popularity and vast number of structures already in place. From this joint, the objective was to find a new connection design that would enhance the ductility of the steel bridge bent welded connection.
Figure 2.25: South column, southeast face.

Figure 2.26: South column failure, full length of crack.
Figure 2.27: Test1 force displacement hysteresis

Figure 2.28: Test 1 base displacement during loading
2.11 Test 2

With the performance of test 1 in mind, the focus was to design a new connection that would allow a plastic hinge to form in the columns and achieve higher levels of ductility. An overarching goal was to ensure that the connection would still be easy to build since an exotic, more complex section might render the structure type impractical. For example, the use of stiffeners, mechanical dampers, or reduced sections was determined to be difficult to implement at this point in the research program, and the emphasis was on enhanced welded connections. The alternatives considered were (1) full penetration weld, (2) full penetration weld with reinforcing fillet, and (3) partial penetration weld. With the goal of this test being to improve performance as much as possible with ‘traditional’ techniques, it was decided that the full penetration weld with a reinforcing fillet would provide the best performance. There were considerations both for and against the use of a reinforcing fillet. On the positive side, it was felt that the reinforcing fillet would provide a smooth transition from pipe to cap beam, thus minimizing and localizing stress concentrations. On the negative side, the use of the reinforcing fillet would result in more heat, resulting in a larger heat affected zone, as well as providing more opportunities for defects. A drawing of the weld detail can be seen in Figure 2.29. A summary of events during the test can be seen in Table 2.3.
Continuous weld for cyclic loading

Backer ring min. thickness 3/16"

Figure 2.29: Test 2 connection detail.

Table 2.3: Test 2 summary

<table>
<thead>
<tr>
<th>Ductility Cycle</th>
<th>Load [kips]</th>
<th>Displacement [inches]</th>
<th>Plan view*</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\mu_{3.1}$</td>
<td></td>
<td></td>
<td></td>
<td>Loaded past ductility 3 to a displacement of 16”, approx. $\mu_{5}$</td>
</tr>
<tr>
<td>$\mu_{3.1}$</td>
<td>$-91$</td>
<td>9.72</td>
<td></td>
<td>North Column</td>
</tr>
</tbody>
</table>

North Column
Because of concerns over the construction quality of the specimens, quality control and quality assurance were incorporated into the process. Randy Dempsey, an employee of NCDOT, volunteered to conduct the quality assurance. His credentials can be found in Appendix 6.3. The results for Test 2 quality control can be found in Appendix 6.4.

For this test, \( \Delta_{y}^\prime \) was 2.49 inches resulting in \( \Delta_{y} \) of 3.24 inches using Equation (2.17). As a result, the defined ductility values were \( \mu_1 \) (3.24 in.), \( \mu_{1.5} \) (4.86 in.), \( \mu_2 \) (6.48 in), \( \mu_3 \)

### Table 2.3: Test 2 summary continued

<table>
<thead>
<tr>
<th>( \mu_{41} )</th>
<th>71</th>
<th>12.96</th>
<th>Buckling on both columns (see Figure 2.33)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \mu_{42} )</td>
<td>62</td>
<td>12.96</td>
<td>Crack at the toe of the weld or a little below</td>
</tr>
<tr>
<td>( \mu_{42} )</td>
<td>62</td>
<td>12.96</td>
<td>Crack lengthen from ( \mu_{3-1} )</td>
</tr>
<tr>
<td>( \mu_{4-2} )</td>
<td>-58</td>
<td>12.96</td>
<td>Crack in buckling region of south column.(see Figure 2.34)</td>
</tr>
</tbody>
</table>

*Red lines in plan view indicate location, beginning and end of crack. Blue lines indicate extent of crack at previous ductility level.*
This specimen experienced significantly less displacement at base. The structural displacement was carefully monitored, and the resulting loading is more symmetrical than test 1. Note that there are minor differences between the displacements for a given ductility level, which are not significant from a performance perspective. The complete force deformation response can be seen in Figure 2.30 and the base displacement can be seen in Figure 2.31. Unfortunately, this specimen was subjected to an accidental ‘overload’ cycle that pushed the specimen to a ductility of 5 when it was intended to be going to ductility 3. From the force displacement response, the maximum load was reached between $\mu_3$ and $\mu_4$ on the overload cycle. Upon reversal, the peak load was closer to $\mu_2$.

This connection far out-performed the fillet connection from test one and while it did not achieve a ductility of 6 or higher, this connection was seen to have vast potential due to large drift capacity and a higher level of damping for modest levels of ductility. This connection had increased drift capacity compared to that of the connection in test 1, a fillet weld. A loss of 20% of the ultimate load carrying capacity was seen at $\mu_{3_2}$ and continued to fall until the test was stopped at $\mu_{4_3}$ at a loss of 50% of the ultimate load carry capacity. There was also significant local buckling and cracking seen at this point as well.
Figure 2.30: Test 2 force displacement hysteresis

Figure 2.31: Test 2 base displacement during loading
From the start of the test through $\mu_2$ no major events occurred. It was decided that a UT inspection during the test could give us more information about what is happening at the connection. The UT inspection was performed after the $\mu_2$ cycles and the load was removed from the specimen for the safety of the technician. As previously noted, between $\mu_2$ and $\mu_3$ the structure was accidentally loaded to $\mu_5$, after which the target load history was continued. Although the first crack was noted at $\mu_{3.1}$, it is possible that the initial damage occurred in the overload cycle to $\mu_5$. The crack was noted on the north pile near the neutral axis and can be seen in Figure 2.32. Both columns began to show significant signs of local buckling at the beginning of $\mu_{4.1}$. A photo of the north column with local buckling near the top can be seen in Figure 2.33. The south column began cracking at base of the local buckling during $\mu_{4.2}$ as seen in Figure 2.34.

![Figure 2.32: North column crack. Arrows indicate initial crack length from $\mu_{3.1}$]
Figure 2.33: Close up of local buckling on north column

Figure 2.34: South column local buckling region
2.12 Test 3

Test 2 was a significant improvement over test 1, although still not as robust as desired. For test 3, it was desired to determine if the reinforcing fillet played a complimentary role in its behavior; therefore this specimen had only the full penetration weld. A drawing of the detail can be seen in Figure 2.35. A summary of events can be seen in Table 2.4.

![Figure 2.35: Test 3 detail](image-url)
Table 2.4: Test 3 summary

<table>
<thead>
<tr>
<th>Ductility Cycle</th>
<th>Load [kips]</th>
<th>Displacement [inches]</th>
<th>Plan view</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Over load</td>
<td>-100</td>
<td>Unknown</td>
<td></td>
<td>Specimen was loaded past 50%F_y to an unknown load and displacement</td>
</tr>
<tr>
<td>( \mu_{1.5.2} )</td>
<td>-61.5</td>
<td>4.86</td>
<td>[Diagram of North column]</td>
<td>Green arrows show area where small cracks were seen</td>
</tr>
<tr>
<td>( \mu_{1.5.2} )</td>
<td>-61.5</td>
<td>4.86</td>
<td>[Diagram of North column]</td>
<td>Red line growth of crack and blue lines are old crack</td>
</tr>
</tbody>
</table>
Table 2.4: Test 3 summary continued

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>(\mu_{2.1})</td>
<td>85</td>
<td>12.96</td>
<td>North column Red lines are new crack that formed</td>
</tr>
<tr>
<td>(\mu_{2.2})</td>
<td>-70</td>
<td>12.96</td>
<td>South Column crack from (\mu_{1.5-2}) propagated through weld in location shown previously</td>
</tr>
<tr>
<td>(\mu_{2.3})</td>
<td>76</td>
<td></td>
<td>North Column crack from (\mu_{2.1}) propagated through weld in location shown previously</td>
</tr>
<tr>
<td>(\mu_{3.1})</td>
<td>-58</td>
<td>9.72</td>
<td>South column New crack on south column shown in red.</td>
</tr>
</tbody>
</table>

*Red lines in plan view indicate location, beginning and end of crack. Blue lines indicate extent of crack at previous ductility level.*
Inspection of the welding was accomplished for test 3 and the result indicated that all work was acceptable; the report can be seen in the Appendix 6.5. During test 3, a loading error occurred after 50% \( F_y \), and it was estimated that a load of -100k was applied. During the loading error, data was not recorded and therefore the hysteresis is incomplete as shown in Figure 2.36, with an estimated portion shown as a dashed line. It was estimated that a displacement of 15.74 inches was applied, resulting in a ductility of 4.86. This estimate was made by extrapolating the unloading line down and projecting a horizontal line from the last point recorded prior to the loading error. The base movement was eliminated in this test by removing the rocker bearings and by using an impact wrench to tighten the bolts connecting the shoes to the base. The yield displacement, \( \Delta_y \), for this test was assumed to be the same as the previous test due to the loading error, which made calculation of the first yield
displacement impossible. $\Delta y$ for test 2 was 2.49 inches resulting in $\Delta y$ of 3.24 inches. As a result the defined ductility values were $\mu_1$ (3.24 in.), $\mu_{1.5}$ (4.86 in.), $\mu_2$ (6.48 in), and $\mu_3$ (9.72 in.). During the overload a fracture of the weld on the north column occurred at the joint of the weld and the cap beam, as seen in Figure 2.37. The next crack that formed was during $\mu_{1.5,2}$ on the south column in the northeastern quadrant, as well as some small cracking on the south side of the south column shown in Figure 2.38. The crack in the weld of the north column grew in length and opened during $\mu_{1.5,2}$ as seen in Figure 2.39. The cracks already formed on both columns continued to grow both in length and width during $\mu_{2,1}$ and $\mu_{2.1}$. During $\mu_{2,1}$ cycle the crack on the south column propagated through the weld, as seen in Figure 2.40. During the $\mu_{2,3}$ cycle the crack crossed the weld on the north pile. The test was continued into ductility three even though the reduction in strength was clearly more than 20%. After cycle $\mu_{3,1}$ the cap beam showed distortion near both columns, as seen in Figure 2.41 and Figure 2.42. At this point the test was stopped due the extent of damage and loss of strength of the test unit. Even with the error in loading, the performance of the test structure was seen as less desirable than test 2 and deemed unacceptable for a new design.
Figure 2.37: North pile first crack along north face arrows on cap beam show extent of crack

Figure 2.38: South column crack on north east after $\mu 1.5 \mu$.

The crack is extent of the crack is shown on cap beam with marker

Figure 2.39: South pile crack through weld

Figure 2.40: North column crack propagated through weld
2.13 Test 4

With the marginal performance of test unit 3 and the loading error in test 2 (which otherwise performed reasonably well) a repeat of test 2 was selected for test 4. There were two primary questions to be answered: (1) Is the performance of test 2 repeatable, and (2) What, if any, impact did the load history have? A detail of the connection can be seen in Figure 2.29. A summary of the events in test 4 can be seen in Table 2.5. Another difference to note is that Test 4 incorporated the use of sound equipment in order to facilitate the
detection of cracks and their locations. A series of 10 microphones was employed to monitor acoustic emissions during the test. The goal was to identify the location of any potential damage, and to distinguish between emissions due to pin rotation versus potential damage induced by cracking. The microphone layout is shown in Figure 2.43. With the microphones, it was straightforward to identify emissions due to pin rotation. For example, the microphones placed at the bottom of each column, as shown in Figure 2.44, registered pin rotations. Even though all microphones registered the pin rotation emissions, those at the bottom measured the highest amplitude.

Figure 2.43: Microphone layout at top of pile

Figure 2.44: Microphone layout of pile bases
Table 2.5: Test 4 summary

<table>
<thead>
<tr>
<th>Ductility Cycle</th>
<th>Load [kips]</th>
<th>Displacement [inches]</th>
<th>Plan view</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>µ3</td>
<td>-90</td>
<td>-9.91</td>
<td></td>
<td>Local buckling of columns near weld</td>
</tr>
<tr>
<td>µ3</td>
<td>66</td>
<td>9.91</td>
<td></td>
<td>Crack formed and opened. Failure of structure.</td>
</tr>
</tbody>
</table>

*Red lines in plan view indicate location, beginning and end of crack.*

Inspection and quality control of the test unit were performed and all work was found to be acceptable. The full report can be found in the Appendix 6.6. For this test $\Delta'_y$ was measured as 2.54 inches resulting in a $\Delta_y$ of 3.30 inches. As a result, the ductility values were $\mu_1$ (3.30 in.), $\mu_1.5$ (4.95 in.), $\mu_2$ (6.60 in), and $\mu_3$ (9.91 in.). The force displacement hysteresis can be seen in Figure 2.45. No major events occurred prior to the local buckling and then crack formation. The local buckling was not as extensive as in previous tests and can be seen in Figure 2.46. The crack that formed on the tension face of the south column appeared to start at the toe of the weld in the heat affected zone. Photos of the crack can be seen in Figure 2.47 and Figure 2.48.
Figure 2.45: Test 4 force displacement hysteresis

Figure 2.46: Local buckling of south column
Figure 2.47: Crack on south column

Figure 2.48: Close up of crack after testing
2.14 Summary

In this chapter the experimental program was presented. This presentation consisted of the specimen design and construction, test procedure, instrumentation and an overview of each test. The specimen design was based on actual bridges in Alaska, capacity design principles and the restriction of the CFL lab. The final design was 2 HSS round piles 16” in diameter with a double HP14x89 cap beam whose dimensions and configuration can be seen in Figure 2.4. The construction of the test specimens was accomplished to reproduce field conditions as closely as possible. This process consisted of overhead welding and quality control monitoring throughout the construction sequence. The test procedure for these tests was reverse cyclic loading applied by means of a 220 kip actuator. The test units were pushed and pulled in load control until yield force by increments of quarter yield force. Once the unit has been subjected to the yield force, subsequent cycling was done to reach prescribed ductility levels. This process is illustrated in Figure 2.20. The instrumentation used in these test consisted of strain gauges, string potentiometers, and Optotrack. The strain gauges were used in all four tests and were used to measure the strains in the piles and cap beam. An average of 30 strain gauges was used on each test and the layout of these gauges can be seen in Figure 2.12 and Figure 2.13. String potentiometers were used to monitor the displacement of the cap beam and any movement of the base supports; their locations can be seen in Figure 2.14. The Optotrack system was used to capture the movement of the pile and
the surface changes of the piles as well. The data from the Optotrack was used to calculate strains, curvature, and local buckling.

Summaries of the four tests were presented in this chapter, highlighting the major events that occurred in each test. For each test the weld detail, summary table, force displacement hysteresis and photos were presented.

Test 1, the current practice in Alaska, was a fillet weld connection between the cap beam and pipe pile. The test unit failed at a ductility of 2 and its failure was in the weld itself. With the poor performance of the current practice, the focus of the study moved to finding an acceptable connection that would conform to design capacity principles such that the inelastic action occurs in the piles and the joint remaining elastic. In order to keep the connection simple as to make it practical and economical, an enhanced welded connection was the focus. With several alternatives considered, a complete joint penetration weld with reinforcing fillet was selected for Test 2. Test unit 2 reach an ultimate ductility of 4, giving it a higher deformation capacity than test unit 1. It is important to note that a loading error occurred during the test from $\mu_2$ to what should have been $\mu_3$. Following the error in loading significant strength degradation occurred with over 20% loss of maximum load occurring at $\mu_3$. The failure of the test unit was still in the joint, with significant cracking on both piles at or near the toe of the weld.

While the performance of specimen 2 had greater displacement capacity than test unit 1, the failure was in the pile at the heat affected zone, still within the joint and therefore unacceptable in capacity design principles. It was hypothesized by the research team that the
large amount of welding in order to achieve a full penetration weld and a ¾” reinforcing fillet, may have adversely affected the material around the weld known as the heat affected zone. In order to determine if this was the case it was decided to proceed with a test of just the complete joint penetration weld for Test 3. Test unit 3 reached an ultimate displacement ductility of 3. It is important to note that a loading error occurred prior to reaching yield load and may have had an unfavorable effect on the performance of the unit. The failure in Test 3 occurred in the weld itself with significant cracking on both piles.

Since test unit 3 did not out perform test unit 2, the reinforcing fillet was thought to have a beneficial effect on the connection. In order to determine what affect the loading error had on the results of test unit 2 and if the performance would be repeatable, a retest of the complete joint penetration weld with reinforcing fillet was selected for test 4. Test unit 4 reached an ultimate displacement ductility of 3 which was lower than test unit 2. The failure was in the same region as in test 2, at the toe of the weld.
CHAPTER III

Analysis of Test Results

CHAPTER 3

3.1 Introduction

The following is a presentation of the analysis of the results from the four tests. The analyses presented in the chapter are the force displacement hysteresis and envelopes, damping, strain profiles, local buckling, and curvature with plastic hinge results.

3.2 Force Displacement Hysteresis

The force displacement hysteresis for the first four tests are shown in the figures bellow; note that the test 3 hysteresis has an estimated line of maximum displacement during the overload cycle. An explanation of how this estimate was made can be found in Chapter 2. Following the force displacement hysteresis in Table 3.1, this summarizes the maximum loads of each test as well as the point at which 10% and 20% of the maximum strength was lost.
Figure 3.1: Test 1 hysteresis

Figure 3.2: Test 2 hysteresis

Figure 3.3: Test 3 hysteresis

Figure 3.4: Test 4 hysteresis

Table 3.1: Maximum loading information

<table>
<thead>
<tr>
<th></th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
<th>Test 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load [kips]</td>
<td>96.50</td>
<td>-102.44</td>
<td>-103.34</td>
<td>-100.44</td>
</tr>
<tr>
<td>Ductility</td>
<td>2_1</td>
<td>3_-1</td>
<td>overload</td>
<td>3_-1</td>
</tr>
<tr>
<td>Max load</td>
<td>86.85</td>
<td>N/A</td>
<td>93.01</td>
<td>90.40</td>
</tr>
<tr>
<td>90% max</td>
<td>81.95</td>
<td>3_3</td>
<td>82.67</td>
<td>-80.35</td>
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<tr>
<td>80% max</td>
<td>3_3</td>
<td>1.5_3</td>
<td>-100.44</td>
<td>3_-2</td>
</tr>
</tbody>
</table>
3.3 Force displacement envelopes.

The force-displacement response envelopes are shown below. These graphs show the peaks on the hysteresis plots from the origin to maximum displacement. Graphs are shown for the first, second, and third cycle envelopes. Clearly, test 2 had the largest drift capacity of all four tests. Comparing the three cycles it is clear that as the cycles progressed the structure showed a decrease in the load carrying ability; most notably in the high levels of ductility. Tests two and four have similar behavior, as expected, since they have the same connection detail.
Figure 3.6: Cycle 2 envelopes

Figure 3.7: Cycle 3 envelopes
3.4 Damping

The hysteretic damping values for each test are presented here. The damping values were calculated using Jacobsen’s approach (Jacobsen, 1930), where the damping is defined as the area contained within one complete cycle of the force displacement response divided by the rigid plastic response that encloses it, shown in Figure 3.8, then multiplied by $2/\pi$.

![Graphical representation of damping](image)

**Figure 3.8: Graphical representation of damping**

This calculation was performed for each ductility level and then averaged among the three cycles at each ductility level. The resulting Jacobsen’s damping values must then be scaled based on inelastic time-history analysis using Equation (3.1) to give the hysteretic damping (Montejo, 2008; Priestley et al. 2007). In order to calculate the equivalent viscous
damping for the test units it is necessary to add the elastic viscous damping to the hysteretic damping. A common value for the elastic damping of steel frames are between 2 and 5%, and 2% was selected for the calculations. The elastic damping is based on tangent stiffness and the hysteretic damping is based on secant stiffness. For this reason the elastic damping must be corrected based on nonlinear time-history analysis in order to be combined with the hysteretic damping. The correction factor $\kappa$ can be seen in Equation (3.2), where $\lambda$ is equal to -0.617 based upon the Ramberg-Osgood model to represent ductile steel structures (Priestley et al. 2007). The final step is to add the two damping value using Equation (3.3). The results are shown Figure 3.9 where it is evident that there is little difference among the tests. The design equivalent viscous damping for steel frame buildings based upon the Ramberg-Osgood model is included for reference, calculated as a percent using Equation (3.4).

\[
\xi_{\text{hyst}} = \xi_{\text{jac}} \left(0.53\mu + 0.8 \left(-\frac{\mu}{40} + 0.4\right)\right)
\]

\[
\kappa = \mu^{\lambda}
\]

\[
\xi_{\text{eq}} = \kappa \xi_{\text{el}} + \xi_{\text{hyst}}
\]

\[
\xi_{\text{eq}} = 0.05 + 0.577 \left(\frac{\mu - 1}{\mu \pi}\right)
\]
The damping values for the units tested correlate well with the design values typically used for steel structures. Tests 2 and 4 have very similar results as expected since they have the same connection type.
3.5 Local buckling

The onset of local buckling was of interest to identify. In order to accomplish this identification, pipe pile deformation profiles were created from the Optotrack data. These pile profiles depict an outline of the pile during the test at any given point in time. More information on how the local buckling was identified can be found in section on page 38.

3.5.1 Test 1

Using the locations of the LED markers on the North and South face of the South column, plots of the pile profiles are shown in Figure 3.11 and Figure 3.12. Looking at Figure 3.12 it appears that there may be local buckling occurring at the top of the column at \( \mu_2 \). Upon closer examination it was not possible to determine for sure if local buckling is actually occurring due to the loss of marker 6 during \( \mu_{1.5} \) and the 4” gauge length used in this test.
Figure 3.10: Map of LED markers for Test 1

Figure 3.11: South face pile profiles for Test 1
Figure 3.12: North face pile profiles for Test 1
3.6 Test 2

Looking at Figure 3.14 and Figure 3.15 local buckling can clearly be seen at $\mu_5$ and $\mu_3$ on the North face and $\mu_3$ on the South face although it is not as pronounced as the North face. The significant local buckling on the North face occurred from $\mu_{2.3}$ to the overload $\mu_5$ but smaller amounts of local buckling occurred well before this point.

Figure 3.13: Map of LED markers for Test 2
Figure 3.14: South face pile profiles for Test 2

Figure 3.15: North face pile profiles for Test 2
On the North face of the pile local buckling began during $\mu_{1.5_2}$ as seen in Figure 3.16. The vertical strain profile during the onset of buckling is shown in Figure 3.17, the strains are shown at the same displacements shown in Figure 3.16 and the displacement is increasing as the strains increase. You can see in this figure that there is residual tensile strain to approximately 20 inches down the pile. The effects of local buckling are seen as the strains are lower than expected between 5 and 10 inches. While the local buckling is not as pronounced there is still an effect on the strains computed. In Figure 3.18 the strain computed using the Optotrack data can be seen up to overload. The post local buckling behavior shows an increase in the tension strains in the push direction but a decrease in the compression strains in the pull direction. This is most likely due to the fact that the pile has buckled and the change in the $y$-coordinates is no longer over an undeformed length of pile. On the South face the local buckling was not seen until after the overload cycle $\mu_5$ and on to $\mu_{3.1}$ as seen in Figure 3.19. The vertical strain profile during the onset of buckling is shown in Figure 3.20, the strains are shown at the same displacements shown in Figure 3.19 and the displacement is increasing as the strains increase. You can see in this figure that there is residual tensile strain to approximately 18 inches down the pile. The effects of local buckling are seen as the strains are lower than expected between 0 and 8 inches. The strains at 8” from the top of the column show signs of local buckling in the pile as well, in Figure 3.21 there is a clear change in the strain behavior following the local buckling of the pile. The maximum strain that the South face sustained before local buckling occurred was of 9245 Microstrain during the $F_y$ cycle. The strain remains in tension for the remainder of the tests. This is again due to the
fact that the pile has deformed and the computed engineering strains calculations are no longer valid.

Figure 3.16: North face local buckling onset during $\mu_{1.52}$ for Test 2
Figure 3.17: Optotrack strain profiles at onset of buckling for North face at the same cap beam displacements seen in Figure 3.17

Figure 3.18: Optotrack strain hysteresis 2" from top of pile on North face of south column during Test 2
Figure 3.19: South face local buckling onset during $\mu_{3.4}$ for Test 2
Figure 3.20: Strain profile at onset of buckling for South face at the same cap beam displacements seen in Figure 3.19

Figure 3.21: Optotrack strain hysteresis at 8" from top of pile on South face of South column during Test 2
3.6.1 Test 3

Due to the loss of the Optotrack data for test 3 no local buckling analysis could be conducted.

Test 4

Local buckling in test 4 occurred on the North face during \( \mu_{2.1} \) but more significant local buckling occurred in \( \mu_3 \) as seen in Figure 3.23. The onset of the local buckling in the North face of the pile can be seen in Figure 3.25. It is important to note that the onset of buckling is seen to occur when the North face of the pile is in tension. This leaves questions to the validity of the method for determining buckling. The strain hysteresis in Figure 3.26 shows that the calculated Optotrack strains are in compression again in contradiction to what simple mechanics tells us that the North face should be in tension. The South face of the pile experienced local buckling during \( \mu_{3.2} \) as can be seen in Figure 3.24 and the onset can be seen in Figure 3.27. The change in the strains computed with the Optotrack data can be seen Figure 3.28 that matches up with the onset of buckling during \( \mu_{3.2} \). In Figure 3.29 the strain profiles for the onset of buckling of the South face is shown, where the first profile starts at when the pile is at zero displacement and then increasing at 1” increments until the pile is at \( \mu_{3.2} \); these chosen displacements are the same as the displacements in Figure 3.27. The maximum strain that the South face sustained before buckling occurred was 17953 Microstrain at \( \mu_{3.1} \). The strain no long behaves as it did prior to local buckling and is not
longer valid engineering strain since the pile is bulging out and the change in length is not over an undeformed region.

Figure 3.22: Map of LED marker for Test 4
Figure 3.23: North face pile profiles for Test 4

Figure 3.24: South face pile profiles for Test 4
Figure 3.25: Local buckling onset on North face during $\mu_{2,1}$ of Test 4

Figure 3.26: Optotrack strains on North face 3” from top of pile for Test 4
Figure 3.27: Local buckling onset of South face during $\mu_{3.2}$ of Test 4

Figure 3.28: Optotrack strain hysteresis 7” from top of pile for Test 4
Figure 3.29: Optotrack strain profile during onset of buckling at given cap beam displacements
3.7 Strain Profiles

Strain gauge data is presented in the form of strain profiles in this section. Strain vs. cap beam displacement hysteresis can be found in the Appendix 6.7 through 6.12. The sign convention associated with the strain gauges and the Optotrack strains is positive strain = compression and negative strains = tension. For strain gauge data only vertical profiles are presented. Optotrack data allows both vertical profiles and horizontal profiles.

3.7.1 Test 1

Strain gauge profiles

Looking at the strain profiles for the extreme fibers, either the north or south face, we see that the strains near the top of the pile, at the joint location, are the highest. In the case of the profiles not along the extreme fibers in Figure 3.32, Figure 3.33, Figure 3.38 and Figure 3.39 the maximum strains do not appear at the top of the pile as with the extreme fiber regions. The strains are also smaller on average in the middle region than in the extreme fibers. The strains at mid height of the piles, 70 inches from the top, are below the yield strain of the material in all the figures.
Figure 3.30: Test 1, north column north, face push direction

Figure 3.31: Test 1, north column, north face, pull direction
Figure 3.32: Test1, north column, middle face, push direction

Figure 3.33: Test1, north column, middle face, pull direction
Figure 3.34: Test1, north column, south face, push direction

Figure 3.35: Test1, north column, south face, pull direction
Figure 3.36: Test1, south column, north face, push direction

Figure 3.37: Test1, south column, north face, pull direction
Figure 3.38: Test1, south column, middle face, push direction

Figure 3.39: Test1, south column, middle face, pull direction
Figure 3.40: Test1, south column, south face, push direction

Figure 3.41: Test1, south column, south face, pull direction
Optotrack Stain Profiles

The Optotrack data correlates well with the strain gauge data with respect to the vertical strain profiles. In the horizontal profiles it can be seen that the neutral axis is not located in the middle of the pile as would be expected, but is relatively close. Figure 3.47 shows that at $\mu_{2.1}$ that the strain on the left hand side switches from negative to positive. The pile at this location is in compression during a pull cycle and that would indicate local buckling is a probable scenario for the change in the strain at this point in the test. This trend continues down the pile as seen in Figure 3.49, Figure 3.51 and Figure 3.53. As mentioned before it is possible that local buckling has taken place at this point in the test but the results were inconclusive.
Figure 3.42: Test1, south column, south face, push direction

Figure 3.43: Test1, south column, south face, pull direction
Figure 3.44: Test1, south column, north face, push direction

Figure 3.45: Test1, south column, north face, pull direction
Figure 3.46: Test 1, horizontal strain profile 6" from top of cap beam in push direction

Figure 3.47: Test 1, horizontal strain profile 6" from top of cap beam in pull direction
Figure 3.48: Test 1, horizontal strain profile 10" from top of cap beam in push direction

Figure 3.49: Test 1, horizontal strain profile 10" from top of cap beam in pull direction
Figure 3.50: Test 1, horizontal strain profile 14" from top of cap beam in push direction

Figure 3.51: Test 1, horizontal strain profile 14" from top of cap beam in pull direction
Figure 3.52: Test 1, horizontal strain profile 18" from top of cap beam in push direction

Figure 3.53: Test 1, horizontal strain profile 18" from top of cap beam in pull direction
3.7.1 Test 2

Strain gauge profiles

The strains at the top of the piles in most cases are the largest strains seen but in Figure 3.58 and Figure 3.59 the strain at 10” from the top have the highest strains. This could be caused by local buckling of the pile at this location. The strains at mid height of the piles, 70 inches from the top, are below the yield strain of the material. The strain gauges did not last throughout the entire duration of the test and the gauges near the top stop working as seen in Figure 3.54, Figure 3.55 and Figure 3.58 through Figure 3.61.

Figure 3.54: Test 2, north column, north face, push direction
Figure 3.55: Test 2, north column, north face, pull direction

Figure 3.56: Test 2, north column, south face, push direction
Figure 3.57: Test 2, north column, south face, push direction

Figure 3.58: Test 2, south column, north face, push direction
Figure 3.59: Test 2, south column, north face, pull direction

Figure 3.60: Test 2, south column, south face, push direction
Figure 3.61: Test 2, south column, south face, pull direction
**Optotrack strain profiles**

The vertical strain profiles using the strain gauge data correlate well with the Optotrack data. The Optotrack data shows very high strains occurring at around 8 inches from the top of the pile. This could be caused by local buckling that has occurred in the pile at this location. The strain gauges stopped working prior to the very large strain seen in Figure 3.62 through Figure 3.65 it is not possible to know if the strain calculated with the Optotrack data is accurate. One other indication that the strain is being affected by local buckling is seen in Figure 3.63 and Figure 3.64, where the strain in the top 15” of the pile abruptly goes from positive to negative or vice versa during the last two ductility levels.

The effect of local buckling on the horizontal strain profiles is shown in Figure 3.14 and Figure 3.15. The region affected by local buckling is from about 5” from the top of the pile down to approximately 20”, but Figure 3.76 shows that at µ3 the strain becomes negative for the entire cross section and the strains no longer behaves linearly. The strains in the push and pull direction are not equally effected by the local buckling of the pile. In Figure 3.70 the strains remain almost linear but in Figure 3.71 there is a distinct change in profile on the left hand side. Closer to the top of the pile the effect of local buckling in both the push and pull direction appear to be the same.
Figure 3.62: Test 2, south column, south face, push direction

Figure 3.63: Test 2, south column, south face, pull direction
Figure 3.64: Test 2, south column, north face, push direction

Figure 3.65: Test 2, south column, north face, pull direction
Figure 3.66: Test 2, horizontal profile 6" from bottom of cap beam, push direction

Figure 3.67: Test 2, horizontal profile 6" from bottom of cap beam, pull direction
Figure 3.68: Test 2, horizontal profile 10" from bottom of cap beam, push direction

Figure 3.69: Test 2, horizontal profile 10" from bottom of cap beam, pull direction
Figure 3.70: Test 2, horizontal profile 14" from bottom of cap beam, push direction

Figure 3.71: Test 2, horizontal profile 14" from bottom of cap beam, pull direction
Figure 3.72: Test 2, horizontal profile 18" from bottom of cap beam, push direction

Figure 3.73: Test 2, horizontal profile 18" from bottom of cap beam, pull direction
Figure 3.74: Test 2, horizontal profile 22" from bottom of cap beam, push direction

Figure 3.75: Test 2, horizontal profile 22" from bottom of cap beam, pull direction
Figure 3.76: Test 2, horizontal profile 26" from bottom of cap beam, push direction

Figure 3.77: Test 2, horizontal profile 26" from bottom of cap beam, pull direction
3.7.2 Test 3

Strain gauge profiles

The strain profiles in test 3 show effects of the overload in the beginning of the test. In the other tests the strains at mid height were below yield but in test 3 they are much higher. In Figure 3.80, Figure 3.81, Figure 3.84 and Figure 3.85 the strains at mid-height have passed the yield strain of the steel. The shape of the profiles differs from the previous tests. The strains are both compression and tension in both the push and pull direction were as before the strains were either compression or tension but not both. This change in the shape of the profiles is most likely due to the overload cycle that occurred prior to any of the ductility levels. This indicates that while it did not occur in the previous test there is a potential for yielding if the structure is subjected to a large displacement early on in a seismic event.
Figure 3.78: Test 3, north column, north face, push direction

Figure 3.79: Test 3, north column, north face, pull direction
Figure 3.80: Test 3, north column, south face, push direction

Figure 3.81: Test 3, north column, south face, pull direction
Figure 3.82: Test 3, south column, south face, push direction

Figure 3.83: Test 3, south column, south face, pull direction
Figure 3.84: Test 3, south column, north face, push direction

Figure 3.85: Test 3, south column, north face, pull direction
3.7.3 Test 4

Strain gauge profiles

The strains at the top of the piles in most cases are the largest strains seen but in Figure 3.86, Figure 3.88, Figure 3.59, Figure 3.91, Figure 3.92 and Figure 3.93 the strain at 10” from the top have the highest strains. This trend does not last for the duration of the test, by ductility level 2 the strains at the top are the highest and decrease as you move down the pile. In Figure 3.87 the strain at 18” are the highest but the strain at the top surpasses the strains below it by ductility level 2. This could be caused by local buckling of the pile at this location. The strains at mid height of the piles, 70 inches from the top, are below the yield strain of the material in all the figures showing that the pile remains elastic from at least mid height down as seen in all the figures above. The strain gauges didn’t last throughout the entire duration of the test and the gauges near the top stop working as seen in Figure 3.54, Figure 3.55 and Figure 3.58 through Figure 3.61.
Figure 3.86: Test 4, north column, north face, push direction

Figure 3.87: Test 4, north column, north face, pull direction
Figure 3.88: Test 4, north column, south face, push direction

Figure 3.89: Test 4, north column, south face, pull direction
Figure 3.90: Test 4, south column, north face, push direction:

Figure 3.91: Test 4, south column, north face, pull direction
Figure 3.92: Test 4, south column, south face, push direction

Figure 3.93: Test 4, south column, south face, pull direction
Optotrack profiles

The strain gauge profiles for test 4 and the Optotrack data are similar but do not correlate as well as in test 2. The vertical Optotrack strain profiles are not a smooth transition between the top of the pile and mid height but they still have an overall trend that the strains at the top are greater than the ones at mid-height. The profiles, even with the irregular shape, still show an increase in the strains as the test progresses’ through the ductility levels. The horizontal strain profiles show in Figure 3.99 and Figure 3.100 effects of local buckling at the top of the pile but not past 10” from the top of the pile. This matches what is seen in Figure 3.23and Figure 3.24 of the pile profiles for Test 4. Note that in Figure 3.101 it appears that there may be local buckling but there was no local buckling at this location just a marker that was no recorded during μ1.5 making the graph appear as though local buckling may have occurred. The amount of local buckling in this test was not as sever as in test two so the effects on the strains would be expected to be less than the previous test. The decrease in the amount of local buckling could be due to the absence of the overload cycle. The local buckling induced by the overload may have helped test unit 2 exceed the ductility of test unit 4 since that is the only major difference between the tests; test unit 3 had a max ductility of 4 vs. 3 for test unit 4.
Figure 3.94: Test 4, south column, south face, push direction

Figure 3.95: Test 4, south column, south face, pull direction
Figure 3.96: Test 4, south column, north face, push direction

Figure 3.97: Test 4, south column, north face, pull direction
Figure 3.98: Test 4, horizontal strain profile 6" from bottom of cap beam in push direction

Figure 3.99: Test 4, horizontal strain profile 6" from bottom of cap beam in pull direction
Figure 3.100: Test 4, horizontal strain profile 10" from bottom of cap beam in push direction

Figure 3.101: Test 4, horizontal strain profile 10" from bottom of cap beam in pull direction
Figure 3.102: Test 4, horizontal strain profile 14" from bottom of cap beam in push direction

Figure 3.103: Test 4, horizontal strain profile 14." from bottom of cap beam in pull direction
Figure 3.104: Test 4, horizontal strain profile 18" from bottom of cap beam in push direction

Figure 3.105: Test 4, horizontal strain profile 18" from bottom of cap beam in pull direction
Figure 3.106: Test 4, horizontal strain profile 22" from bottom of cap beam in push direction

Figure 3.107: Test 4, horizontal strain profile 22" from bottom of cap beam in pull direction
Figure 3.108: Test 4, horizontal strain profile 26" from bottom of cap beam in push direction

Figure 3.109: Test 4, horizontal strain profile 26" from bottom of cap beam in pull direction
3.8 Curvature

The curvature plots for test 1 indicate that the curvature is increasing as the test progresses. The curvature during $\mu_{1.5}$ as seen in Figure 3.111 shows a curvature...
completely different than the previous cycle’s curvature. This is again an indication of the possibility for local buckling to be occurring at this location.

**Figure 3.112: Test 2 curvature profile of south column in push direction**

The curvature for test 2 shows a slow progression of the curvature prior to ductility level 3, at which point the curvature near the top of the pile is more than 5 times larger than

139
points pervious during the test. At this point in the test local buckling has occurred causing the strains to no longer remain linear. Note that the curvature is higher at about 10” from the top where the majority of the local buckling was seen.

Figure 3.114: Test 4 curvature profile of south column in push direction

Figure 3.115: Test 4 curvature profile of south column in pull direction
The curvature plots for test 4 have the same jagged shape as the Optotrack profiles (Figure 3.94 through Figure 3.97). There are very high curvatures near the top of the pile but unlike the previous tests the curvature at the top does not make a dramatic jump but is increasing throughout the test. While it was shown that local buckling occurred on the North face during $\mu_2$ and then later on the South face during $\mu_3$ it is not clear whether the large curvature at the top of the pile is caused by the local buckling.

Using the above curvature graphs the plastic hinge length of the piles can be calculated. Considering the effects of local buckling on the Optotrack strain calculation coupled with the fact that curvature is calculated on the basis of strain, the plastic hinge length calculations were conducted at levels of ductility prior to local buckling of the pile. The total displacement of the structure consists of the yield displacement and the plastic displacement as seen in Equation (3.5). The plastic displacement is the plastic curvature multiplied times the plastic hinge length times the length of the pile from the center of the pin to the top of the cap beam shown in Equation (3.6). Using Equation (3.7) to calculate the plastic curvature, where the maximum curvature is taken off the above graphs and the yield curvature is calculated using Equation (3.8). The results for the plastic hinge length can be found in Table 3.2 which contains tests 1, 2 and 4.
\[ \Delta_{total} = \Delta_y + \Delta_p \]  \hspace{1cm} (3.5)

\[ \Delta_p = \phi_p L_p L \]  \hspace{1cm} (3.6)

\[ \phi_p = \phi_{\text{max}} - \phi_y \]  \hspace{1cm} (3.7)

\[ \phi_y = \phi_y \frac{M_p}{M_y} \]  \hspace{1cm} (3.8)

Table 3.2: Plastic hinge length [inches]

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<td>2</td>
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</tr>
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<td>19.5</td>
<td>14.4</td>
<td>16.2</td>
<td>15.4</td>
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</table>

The test 1 results of the plastic hinge length analysis yielded two numbers slightly far apart from one another. In the positive direction a plastic hinge length of 13 inches is obtained and then in the negative direction it is close to 40 inches. Table 3.3 outlines the intermediate values used to calculate these plastic hinge lengths. The curvature is considerably larger in the push direction than in the pull direction as are the displacements (due to the base displacements). In Figure 3.117 it is clear that the strains are no longer remaining linear along the cross-section and that too is contributing to the large plastic hinge.
length for $\mu - 1.5$. Due to the section no longer having linear strains across it, the plastic hinge calculations are not valid since the curvature at this point is no longer being calculated with plane sections.
Table 3.3: Test 1 Intermediate steps

<table>
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<td>0.00021</td>
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<tr>
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</tr>
<tr>
<td>$\Delta y$ [inches]</td>
<td>3.79</td>
<td>-2.56</td>
</tr>
<tr>
<td>$\varphi_{max}$ [1/inch]</td>
<td>-0.0019</td>
<td>0.00055</td>
</tr>
<tr>
<td>$\varphi_p$ [1/inch]</td>
<td>0.0003</td>
<td>-0.0015</td>
</tr>
<tr>
<td>$L_p$ [inches]</td>
<td>13.0</td>
<td>37.3</td>
</tr>
</tbody>
</table>

Figure 3.116: Test 1 strain profile for maximum curvature at $\mu_{1.5_2}$

Figure 3.117: Test 1 strain profile for maximum curvature at $\mu_{1.5_{-1}}$
The intermediate steps for test 2 can be seen in Table 3.4. The results for plastic hinge length for test 2 are all within 15% of one another, very good correlation for experimental results. The strain profiles at the location of maximum curvature that was used to calculate the plastic hinge length are shown in Figure 3.118 through Figure 3.121. The strains in these figures are linear indicating plane section are remaining plane. This behavior is probably why all the values of the plastic hinge are so close together for this test as apposed to the other tests.

Table 3.4: Test 2 intermediate steps

<table>
<thead>
<tr>
<th></th>
<th>1.5</th>
<th>-1.5</th>
<th>2</th>
<th>-2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>φ_y' [1/inch]</strong></td>
<td>-0.00027</td>
<td>0.00021</td>
<td>-0.0003</td>
<td>0.00021</td>
</tr>
<tr>
<td><strong>φ_y [1/inch]</strong></td>
<td>-0.00035</td>
<td>0.00028</td>
<td>-0.00035</td>
<td>0.00028</td>
</tr>
<tr>
<td><strong>Δ_y' [inches]</strong></td>
<td>2.65</td>
<td>-2.42</td>
<td>2.65</td>
<td>-2.42</td>
</tr>
<tr>
<td><strong>Δ_y [inches]</strong></td>
<td>3.45</td>
<td>-3.15</td>
<td>3.45</td>
<td>-3.15</td>
</tr>
<tr>
<td><strong>φ_{max} [1/inch]</strong></td>
<td>-0.0013</td>
<td>0.00123</td>
<td>-0.0024</td>
<td>0.00218</td>
</tr>
<tr>
<td><strong>φ_p [1/inch]</strong></td>
<td>-0.0009</td>
<td>0.0010</td>
<td>-0.0021</td>
<td>0.0019</td>
</tr>
<tr>
<td><strong>L_p [inches]</strong></td>
<td>12.5</td>
<td>11.8</td>
<td>11.1</td>
<td>13.0</td>
</tr>
</tbody>
</table>
Figure 3.118: Test2 strain profile for maximum curvature at $\mu_{1.5_2}$

Figure 3.119: Test2 strain profile for maximum curvature at $\mu_{1.5_3}$

Figure 3.120: Test2 strain profile for maximum curvature at $\mu_{2_3}$

Figure 3.121: Test2 strain profile for maximum curvature at $\mu_{2_1}$
The results for test 4 are not as consistent as test 2 but also not as scattered as test 1. They show some variation among the directions and these results can be explained by the nonlinear behavior of the strain across the cross-section as was the case for Test 1. Table 3.5 highlights the intermediate steps for calculating the plastic hinge length. The strains at the location of maximum curvature can be seen Figure 3.122 through Figure 3.125. In Figure 3.122 through Figure 3.125 the strain profile across the cross-section can be seen for the different ductility levels. While the amount of nonlinearity is not as severe as in Test 1 and they are not as linear as in Test 2, they are somewhere in between as are the results obtained from them.

Table 3.5: Test 4 intermediate steps

<table>
<thead>
<tr>
<th></th>
<th>Ductility</th>
<th>1.5</th>
<th>-1.5</th>
<th>2</th>
<th>-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varphi_y'$ [1/inch]</td>
<td></td>
<td>-0.00022</td>
<td>0.00016</td>
<td>-0.0002</td>
<td>0.00016</td>
</tr>
<tr>
<td>$\varphi_y$ [1/inch]</td>
<td></td>
<td>-0.00028</td>
<td>0.00021</td>
<td>-0.00028</td>
<td>0.00021</td>
</tr>
<tr>
<td>$\Delta_y'$ [inches]</td>
<td></td>
<td>2.65</td>
<td>-2.42</td>
<td>2.65</td>
<td>-2.42</td>
</tr>
<tr>
<td>$\Delta_y$ [inches]</td>
<td></td>
<td>3.45</td>
<td>-3.15</td>
<td>3.45</td>
<td>-3.15</td>
</tr>
<tr>
<td>$\varphi_{max}$ [1/inch]</td>
<td></td>
<td>-0.0009</td>
<td>0.00099</td>
<td>-0.0017</td>
<td>0.00182</td>
</tr>
<tr>
<td>$\varphi_p$ [1/inch]</td>
<td></td>
<td>-0.0006</td>
<td>0.0008</td>
<td>-0.0014</td>
<td>0.0016</td>
</tr>
<tr>
<td>$L_p$ [inches]</td>
<td></td>
<td>19.5</td>
<td>14.4</td>
<td>16.2</td>
<td>15.4</td>
</tr>
</tbody>
</table>
Figure 3.122: Test 4 strain profile for maximum curvature at $\mu_{1.5}$

Figure 3.123: Test 4 strain profile for maximum curvature at $\mu_{1.5}$

Figure 3.124: Test 4 strain profile for maximum curvature at $\mu_{2}$

Figure 3.125: Test 4 strain profile for maximum curvature at $\mu_{2}$
Looking at all the results from the tests that have valid strain profiles we see that the plastic hinge length is approximately the diameter of the pile. This correlates to what is seen in concrete structures where it is commonly between D and 2D (Priestley, 1996). The plastic hinge approach is more common type of analysis in concrete structures where plane sections remain plane and the strains remain linear up to failure. This is not the case for steel as shown in the strain profiles above. With the onset of local buckling the section no longer has linear strains making the plastic hinge method invalid. It is more common to estimate the ultimate displacement capacity not in terms of the plastic displacement and elastic or yield displacement but the ultimate displacement capacity is determined with a plastic analysis. The ultimate capacity is calculated by means of one of three simple plastic analysis; the step by step method, the statical method and the kinematic method. These methods use a bilinear elastic perfectly plastic moment curvature models in general (Bruneau, et al. 1998).

3.9 Summary

In this chapter the experimental results were presented. These results include the force displacement hysteresis and envelope that show test unit 2 had the highest displacement capacity. It was seen in the force displacement envelopes (Figure 3.5 through Figure 3.7) that as the ductility levels progresses the strength carrying capacity dropped. The damping was presented in this chapter and showed good correlation between the experimental results for test units 2 and 4 with the typical design equation (Equation (3.4)). Local buckling was
of investigated in this chapter as well. Test unit 1 had not conclusive results but test units 2 and 4 had clear points in the test where local buckling was shown to initiate.

Strain profiles were presented in this chapter. They showed the effects of the loading on the members, whether they yielded and to what extent. Both strain gauge data and Optotrack data was presented. For the Optotrack data horizontal profiles demonstrated that plane sections remained plane up to the point of local buckling. They also showed that the neutral axis of the members while not at exactly in the middle of the section it was close to middle.

Lastly curvature and plastic hinge length was presented. The curvature of the test units like the strains is only valid while plane sections remain plane, prior to the onset of local buckling. The plastic hinge length calculation, when based on valid strain profiles and curvatures, correlate well to one another. The results from the plastic hinge length calculations compare well with what is typically used in reinforced concrete.
Analysis of Demand vs. Capacity

4.1 Introduction

In this chapter capacity and demand analysis is conducted based on the results from test units 2 and 4, the CJP with reinforcing fillet well configuration. This joint was chosen based on its performance in the two tests being desirable due to the drift capacity and moderate damping of the system. One analysis will look at the capacity of the system to determine when it would not be necessary to design for lateral strength. The second analysis, given the lateral capacity of the system what is the largest seismic demand that can be sustained.

4.2 Goals

The objective of this analysis is to utilize the relationships between spectral demand (expressed as the acceleration at 1 second period) and structural response to determine the level of seismic intensity that results in exceedence of a target displacement defined by displacement ductility levels.
4.3 Minimum Spectral Demand to Require Seismic Design

In this section, the minimum spectral demand to require seismic design was investigated. The yield displacement of the structure is shown in (4.1), which assumes double bending of the pile where L is the distance of the pile from the point of fixity (POF) to the bottom of the cap beam. This calculation ignores the flexibility of the cap beam. A target displacement is then obtained from (4.2) as a function of ductility. From ASCE-7, the corner point period for Alaska was found to be either 6, 12 or 16 seconds. The corner point displacement for 5% damping is obtained from (4.3) and is then scaled according to the design damping using the relationships between damping and spectral reduction from the EuroCode (1993) as seen in Equations (4.6) and (4.7) for near and far field events. By equating (4.4) with (4.6) and (4.7) and substituting (4.1) through (4.4), Equations (4.8) and (4.9) are obtained for near and far field events. Using the damping values from Tests 2 shown in
Table 4.1 and solving for $S_{D1}$, the level of seismicity is obtained that is needed such that the bridge structures must be explicitly designed for lateral forces. This is shown graphically in Figure 4.1 through Figure 4.6. Any acceleration less than the values shown in these figures for the parameters under consideration will be sufficiently small such that the structure will not achieve the target displacement, regardless of strength provided. These values of acceleration are of course very small and indicate that in many (but not all) cases, the structures will require design for lateral forces. Accelerations greater than those shown in Figure 4.1 through Figure 4.6 will require explicit levels of lateral strength to limit the displacements to the target level.

$$\Delta_y = \frac{\phi_y L^2}{6}$$  \hspace{1cm} (4.1)

$$\phi_y = \frac{2\varepsilon_y}{D}$$  \hspace{1cm} (4.2)

$$\Delta_t = \mu \Delta_y$$  \hspace{1cm} (4.3)

$$\Delta_c = \frac{S_{D1}}{T_c^2} \frac{g}{\omega^2}$$  \hspace{1cm} (4.4)

$$\omega = \frac{2\pi}{T}$$  \hspace{1cm} (4.5)

$$\Delta_{c_{\text{Far Field}}} = \Delta_c \sqrt{\frac{7}{2 + \zeta}}$$  \hspace{1cm} Far Field  \hspace{1cm} (4.6)

$$\Delta_{c_{\text{Near Field}}} = \Delta_c \sqrt{\frac{7}{2 + \zeta}}$$  \hspace{1cm} Near Field  \hspace{1cm} (4.7)
\[
\frac{S_{Dl}}{4\pi^2} \frac{\sqrt{7}}{2 + \zeta} = \frac{\mu \varepsilon_y L^3}{3D} \quad \text{Far Field} \quad (4.8)
\]
\[
\frac{S_{Dl}}{4\pi^2} \frac{\sqrt{7}}{2 + \zeta} = \frac{\mu \varepsilon_y L^3}{3D} \quad \text{Near Field} \quad (4.9)
\]
Table 4.1: Equivalent viscous damping for Tests 2

<table>
<thead>
<tr>
<th>Ductility</th>
<th>Average damping</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.4</td>
</tr>
<tr>
<td>1.5</td>
<td>9.1</td>
</tr>
<tr>
<td>2</td>
<td>11.9</td>
</tr>
<tr>
<td>3</td>
<td>15.4</td>
</tr>
<tr>
<td>4</td>
<td>18.2</td>
</tr>
</tbody>
</table>

Figure 4.1: 6 sec Corner Point Period Far Field, double bending
Figure 4.2: 12 Sec Corner Point Period Far Field, double bending

Figure 4.3: 16 Sec Corner Point Period Far Field, double bending
Figure 4.4: 6 sec Corner Point Period near Field, double bending

Figure 4.5: 12 Sec Corner Point Period near Field, double bending
In the graphs above (Figure 4.1 through Figure 4.6) any acceleration less than the values shown for the parameters under consideration will be sufficiently small such that the structure will not achieve the target displacement, regardless of strength provided. These values of acceleration are very small and for any acceleration greater than those shown will require explicit design for lateral forces.

4.4 Minimum Spectral Demand to Develop Strength of Column Sections

Following the previous analysis, of greater interest is the level of acceleration that is needed to develop the strength of a structure with specified D/t ratios. For this analysis a
range of inertia weights from 30 kips to 360 kips were considered as well as D/t ratio of 32, both of which were based on current AKDOT practice.

The first step in the analysis was the selection of the target displacement based on Equation (4.3). The corner point displacement was then calculated using Equation (4.4). The base shear was calculated using Equation (4.10), where \( m \) is the inertia mass and \( \alpha \) is 0.5 for far field and 0.25 for near field (Priestley et al. 2007). The moment in one column is calculated using Equation (4.11). Utilizing that value of moment, the required pipe thickness is determined based on the plastic section modulus and yield stress as shown in Equation (4.12) through (4.14). The resulting D/t was compared to a D/t of 32 and the value of \( S_{D1} \) changed until D/t is equal to 32. These results are for soil type B and would need to be scaled accordingly for other soil types. The results are shown in Figure 4.18 through Figure 4.49.

From the AASHTO maps for Alaska, the value of \( S_1 \) varies from 4% to 175% of g. For any value of \( S_1 \) greater than what is shown in the figures, the level of response will exceed the target value for the chosen parameters. It can be concluded from this analysis that in some cases, the behavior of test units 2 and 4 will be acceptable in some cases (depending on the geometry of the bridge and the level of seismicity). By the same argument, it is clear that for some configurations, a higher level of ductility will be needed.
\[ V_b = \frac{4\pi^2 m \Delta_T^2}{T_c^2} \left( \frac{7}{2 + \zeta^2} \right)^{2\alpha} \]  
(4.10)

\[ M = \frac{V_b}{\text{# of columns}} \cdot \frac{L}{2} \]  
(4.11)

\[ I = \frac{\left( \frac{M}{f_y} \right)}{\text{S.F.}} \cdot \frac{D}{2} \]  
(4.12)

\[ S.F. = \frac{Z_x}{S_x} \]  
(4.13)

\[ I = \frac{\pi}{16} (D_{out}^2 - D_{in}^2) \]  
(4.14)

### 4.5 Sample Calculation of \( S_{D1} \)

![Figure 4.7: Diagram of example](image)

The following will demonstrate the analysis conducted to obtain Figure 4.18 through Figure 4.49:
2 column bent

Length, \( L \) \( = 20 \text{ ft} \)

Diameter, \( D \) \( = 1 \text{ ft} \)

Thickness, \( t \) \( = 0.375 \text{ in} \)

Mass, \( m \) \( = 932 \text{ slugs} \)

Ductility, \( \mu \) \( = 3 \)

Damping, \( \zeta \) \( = 15.6 \% \) (from
Table 4.1)

Corner point period  = 16 sec

Yield strength, \( f_y \)  = 53 ksi

Near field event

Calculating target displacement:

Using equations (4.1) and (4.2) for yield displacement:

\[
\Delta_y = \phi_y \frac{L^2}{6} \frac{2e_y}{D} \frac{L^2}{6} = \frac{(2)(0.0018)(10')^2}{6} = 0.24'
\]

Using Equation (4.3) to calculate the target displacement:

\[
\Delta_T = \mu \Delta_y = (3)(0.24') = 0.72'
\]

Calculate corner point displacement:

Guess a value of \( S_{D1} \)

\[
S_{D1} = 181 \% \ g
\]

Using equations (4.4) and (4.6):

\[
\Delta_c = \frac{S_{D1}}{T_c} \left( \frac{2\pi}{T} \right)^2 = \frac{(1.81)(32.2 \text{ ft/s}^2)}{16 \text{ sec}} = 23.61'
\]
Calculate base shear, $V_b$:

Using Equation (4.10)

$$
V_b = \frac{4\pi^2 m \Delta_c^2}{T_c^2 \Delta_r} \left( \frac{7}{2 + \zeta} \right)^{2\alpha} = \frac{4\pi^2 932 \text{slugs} (23.61)^2}{16 \text{sec}^2 0.24'(2 + 15.6)^{20.5}} = 44.3 \text{ kips}
$$
Calculate moment in one column:

Using Equation (4.11):

\[ M = \frac{V_b}{\text{#ofcolumns}} \frac{L}{2} = \frac{44.3 \text{kips}}{20'} \frac{20'}{2} = 222 \text{kip - ft} \]

Calculate moment of inertia:

Based on section properties

\[ I = \frac{\pi}{16} (D_{out}^2 - D_{in}^2) = \frac{\pi}{16} \left( (12'')^2 - (12'' - 0.375'')^2 \right) = 232 \text{in}^4 \]

Based Base on moment in one column:

In order to calculate the moment of inertia the shape factor is needed for the section based on the given properties.

\[ S.F. = \frac{Z_s}{S_y} = \frac{53.7}{41.0} = 1.3 \]

\[ I = \left( \frac{M}{f_y} \right) \frac{D}{S.F.} \frac{2}{2} = \frac{(222 \text{kip - ft})\left( \frac{12''}{1'} \right)}{53 \text{ksi} \cdot 1.3} = 232 \text{in}^4 \]

Check that both moments of inertia are equal if they are not change S_{D1}. As they are equal in this above calculation the required S_{D1}, to obtain a \( \mu \) of 3 for the chosen configuration is 181\% g.

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4.6 Design Graphs

Figure 4.8: 16 Sec Corner Point Period, Far Field, Double Bending, D/t=32, Inertia Weight 360 kips 2 piles

Figure 4.9: 16 Sec Corner Point Period, Far Field, Double Bending, D/t=32, Inertia Weight 300 kips 2 piles
Figure 4.10: 16 Sec Corner Point Period, Far Field,
Double Bending, D/t=32, Inertia Weight 240 kips 2 piles

Figure 4.11: 16 Sec Corner Point Period, Far Field,
Double Bending, D/t=32, Inertia Weight 180 kips 2 piles
Figure 4.12: 16 Sec Corner Point Period, Far Field,
Double Bending, D/t=32, Inertia Weight 120 kips 2 piles

Figure 4.13: 16 Sec Corner Point Period, Far Field,
Double Bending, D/t=32, Inertia Weight 60 kips 2 piles
Figure 4.14: 16 Sec Corner Point Period, Far Field,
Double Bending, D/t=32, Inertia Weight 30 kips 2 piles

Figure 4.15: 16 Sec Corner Point Period, Far Field,
Double Bending, D/t=32, Inertia Weight 360 kips 3 piles
Figure 4.16: 16 Sec Corner Point Period, Far Field, Double Bending, D/t=32, Inertia Weight 300 kips 3 piles

Figure 4.17: 16 Sec Corner Point Period, Far Field, Double Bending, D/t=32, Inertia Weight 240 kips 3 piles
Figure 4.18: 16 Sec Corner Point Period, Far Field,
Double Bending, D/t=32, Inertia Weight 180 kips 3 piles

Figure 4.19: 16 Sec Corner Point Period, Far Field,
Double Bending, D/t=32, Inertia Weight 120 kips 3 piles
Figure 4.20: 16 Sec Corner Point Period, Far Field, Double Bending, D/t=32, Inertia Weight 60 kips 3 piles

Figure 4.21: 16 Sec Corner Point Period, Far Field, Double Bending, D/t=32, Inertia Weight 30 kips 3 piles
Figure 4.22: 16 Sec Corner Point Period, Far Field, Double Bending, D/t=32, Inertia Weight 360 kips 4 piles

Figure 4.23: 16 Sec Corner Point Period, Far Field, Double Bending, D/t=32, Inertia Weight 300 kips 4 piles
Figure 4.24: 16 Sec Corner Point Period, Far Field, Double Bending, D/t=32, Inertia Weight 240 kips 4 piles

Figure 4.25: 16 Sec Corner Point Period, Far Field, Double Bending, D/t=32, Inertia Weight 180 kips 4 piles
Figure 4.26: 16 Second Corner Point Period, Far Field, Double Bending, D/t=32, Inertia Weight 120 kips, 4 piles

Figure 4.27: 16 Second Corner Point Period, Far Field, Double Bending, D/t=32, Inertia Weight 60 kips, 4 piles
Figure 4.28: 16 Second Corner Point Period, Far Field,
Double Bending, D/t=32, Inertia Weight 30 kips, 4 piles

Figure 4.29: 16 Sec Corner Point Period, Near Field,
Double Bending, D/t=32, Inertia Weight 360 kips 2 piles
Figure 4.30: 16 Sec Corner Point Period, Near Field, Double Bending, D/t=32, Inertia Weight 300 kips 2 piles

Figure 4.31: 16 Sec Corner Point Period, Near Field, Double Bending, D/t=32, Inertia Weight 240 kips 2 piles

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Figure 4.32: 16 Sec Corner Point Period, Near Field,
Double Bending, D/t=32, Inertia Weight 180 kips 2 piles

Figure 4.33: 16 Sec Corner Point Period, Near Field,
Double Bending, D/t=32, Inertia Weight 120 kips 2 piles
Figure 4.34: 16 Sec Corner Point Period, Near Field, Double Bending, D/t=32, Inertia Weight 60 kips 2 piles

Figure 4.35: 16 Sec Corner Point Period, Near Field, Double Bending, D/t=32, Inertia Weight 30 kips 2 piles
Figure 4.36: 16 Sec Corner Point Period, Near Field, Double Bending, D/t=32, Inertia Weight 360 kips 3 piles

Figure 4.37: 16 Sec Corner Point Period, Near Field, Double Bending, D/t=32, Inertia Weight 300 kips 3 piles
Figure 4.38: 16 Sec Corner Point Period, Near Field, Double Bending, D/t=32, Inertia Weight 240 kips 3 piles

Figure 4.39: 16 Sec Corner Point Period, Near Field, Double Bending, D/t=32, Inertia Weight 180 kips 3 piles
Figure 4.40: 16 Sec Corner Point Period, Near Field, Double Bending, D/t=32, Inertia Weight 120 kips 3 piles

Figure 4.41: 16 Sec Corner Point Period, Near Field, Double Bending, D/t=32, Inertia Weight 60 kips 3 piles
Figure 4.42: 16 Sec Corner Point Period, Near Field, Double Bending, D/t=32, Inertia Weight 30 kips 3 piles

Figure 4.43: 16 Sec Corner Point Period, Near Field, Double Bending, D/t=32, Inertia Weight 360 kips 4 piles
Figure 4.44: 16 Sec Corner Point Period, Near Field,
Double Bending, D/t=32, Inertia Weight 300 kips 4 piles

Figure 4.45: 16 Sec Corner Point Period, Near Field,
Double Bending, D/t=32, Inertia Weight 240 kips 4 piles
Figure 4.46: 16 Sec Corner Point Period, Near Field,
Double Bending, D/t=32, Inertia Weight 180 kips 4 piles

Figure 4.47: 16 Sec Corner Point Period, Near Field,
Double Bending, D/t=32, Inertia Weight 120 kips 4 piles
Figure 4.48: 16 Sec Corner Point Period, Near Field,
Double Bending, D/t=32, Inertia Weight 60 kips 4 piles

Figure 4.49: 16 Sec Corner Point Period, Near Field,
Double Bending, D/t=32, Inertia Weight 30 kips 4 piles
From the graphs above (Figure 4.8 through Figure 4.49) for any value of $S_1$ greater than what is shown, the level of response will exceed the target value for the chosen parameters (D/t ratio of 32, weights from 30 kips to 360 kips, corner point period of 16 sec, number of bents between 2 and 4, pile length either 10 or 20 feet and ductility levels between 2 and 4). It can be concluded from this analysis that in some cases, the behavior of test units 2 and 4 will be acceptable in some cases. By the same argument, it is clear that for some configurations, a higher level of ductility will be needed.

4.7 Example uses of graphs

The following will be a calculation using the previous graphs above for the following parameters:

4.7.1 Example 1

A three bent bridge of height 20 feet, diameter of 1 foot, thickness of 1/32”, an inertia weight of 240 kips with a target ductility of 4, located in soil type D and far field.
From Figure 4.17 the value of SD1 = 97% is obtained

To use this value with the seismic design maps it must first corrected for the soil type based on the modification factor, $F_v$, from Table 3.4.2.3-2 in 2006 AASHTO seismic design specifications.

$$S_1 = \frac{S_{D1}}{F_v} = \frac{97\%}{1.5} = 65\%$$

Going to the map, Figure 4.50, any areas with $S_1$ value > 65% are unacceptable.
4.7.2 Example 2

A three bent bridge of height 20 feet, diameter of 1 foot, thickness of 1/32”, an inertia weight of 240 kips with a target ductility of 2, located in soil type D and far field.

From Figure 4.17 a value of $SD_1 = 57\%$ is obtained

To use this value with the seismic design maps it must first be corrected for the soil type based on the modification factor, $F_v$, from Table 3.4.2.3-2 in 2006 AASHTO seismic design specifications.

$$S_i = \frac{S_{D1}}{F_v} = \frac{57\%}{1.5} = 38\%$$
Looking at the map below, any area with a value of $S_1$ lower than 38% will be suitable for this configuration.

![Figure 4.50: $S_1$ map of Alaska](image)

### 4.8 Example of interpolation between graphs

A four bent bridge of height 10 feet, diameter of 20 inches, thickness of 0.625 inches, an inertia weight of 200 kips with a target ductility of 2, located in soil type E and near field event.
From Figure 4.45 a value of $SD_1 = 65\%$ is obtained.

From Figure 4.46 a value of $SD_1 = 75\%$ is obtained.
Using these values and linearly interpolation for an inertia weight of 200 kips:

\[
\left( \frac{75\% - 65\%}{240 - 180} \right)(240 - 200) + 65\% = 72\%
\]

To use this value with the seismic design maps it must first corrected for the soil type based on the modification factor, \( F_v \), from Table 3.4.2.3-2 in 2006 AASHTO seismic design specifications.

\[
S_1 = \frac{S_{01}}{F_v} = \frac{72\%}{2.4} = 30\%
\]

Looking at the map in Figure 4.50, any area with a value of \( S_1 \) lower than 30\% will be suitable for this configuration.

### 4.9 Summary

In this chapter a relationship between the spectral demand and the structural response, was determine and the level of seismic intensity that results in exceedence of a target displacement defined by displacement ductility levels was found. This analysis was based on the results from test units 2 and 4, the CJP with reinforcing fillet well configuration; this joint was chosen based on its performance in the two tests being desirable due to the drift capacity and moderate damping of the system. One analysis looked at the capacity of the system to
determine when it would not be necessary to design for lateral strength. The second analysis, given the lateral capacity of the system determined the largest seismic demand that can be sustained.

The results of these analyses can be seen in the figures presented in this chapter. First the level of seismic intensity that requires the lateral design of the structure was presented. Figure 4.1 through Figure 4.6 show that any acceleration less than the values in the figures, for the parameters under consideration, will be sufficiently small such that the structure will not achieve the target displacement, regardless of strength provided. These values of acceleration are very small and indicate that in many (but not all) cases, the structures will require design for lateral forces.

The second analysis looking at the seismic demand that system can sustain given its lateral capacity. These results are show in Figure 4.8 through Figure 4.49, and for any value of $S_1$ greater than what is shown, the level of response will exceed the target value for the chosen parameters. From this analysis it can be said that in some cases, the behavior of test units 2 and 4 will be provide adequate performance. But it is also clear that for some configurations, a higher level of ductility will be needed to sustain the required level of seismic intensity. Examples were given to explain the use Figure 4.8 through Figure 4.49 when the user has something that can be pulled directly off the graph or when interpolation is need.
Conclusions and Recommendations

5.1 Summary of Tests

An experimental and analytical study was executed to investigate the seismic performance of steel bridge bent welded connections. The results obtained can be summarized as follows:

Test 1 - current practice, fillet weld: There is currently not a lot known about the seismic performance of steel bridge bents with welded connections as evident by the lack of research done on such structures. Testing the current practice in Alaska was the first test conducted in the study. The current practice in Alaska is a fillet weld connection between the cap beam and pipe pile. The test unit was tested under reverse cyclic loading and failed at a ductility of 2. The unit’s failure was in the weld itself demonstrating a flaw in design that the unit does not conform to capacity design principles, which state the damage is controllable and occurs where the designer intends it. Since the intended failure is to be in...
the pile and not in the weld, there is a serious problem with the current design being used throughout the state.

**Test 2 - complete joint penetration weld with reinforcing fillet:** With the low performance of the current practice, the focus of the study moved to finding an acceptable connection that would conform to design capacity principles such that the inelastic action occurs in the piles and the joint, including the weld, remaining elastic. In order to keep the connection simple as to make it practical and economical, an enhanced welded connection was the focus. With several alternatives considered, a complete joint penetration weld with reinforcing fillet was selected. Test unit 2 had higher deformation capacity reaching an ultimate ductility of 4. It is important to note that a loading error occurred during the test from $\mu_2.3$ to what should have been $\mu_3.1$. Preceding the error in loading, significant strength degradation occurred with over 20% loss of maximum load occurring at $\mu_3.3$. The failure of the test unit was still in joint, with significant cracking on both piles at or near the toe of the weld. Again the failure occurred in the joint and not in the pile indicating a flaw in design.

**Test 3 - Complete joint penetration weld:** While the performance of test unit 2 had greater displacement capacity than test unit 1, the failure was still in the joint and therefore unacceptable in capacity design principles. It was hypothesized by the research team that with the large amount of welding in order to achieve a full penetration weld with a ¾” reinforcing fillet, may have adversely affected the material around the weld known as the heat affected zone. In order to determine if this was the case it was decided to proceed with a test of just the complete joint penetration weld. Test unit 3 reached an ultimate displacement
ductility of 3. It is important to note that a loading error occurred prior to reaching yield load and may have had an unfavorable effect on the performance of the unit. The failure of test unit 3 occurred in the weld itself and was therefore determined to again not be in accordance with capacity design principles.

**Test 4 - Complete joint penetration weld with reinforcing fillet:** Since test unit 3 did not outperform test unit 2, the reinforcing fillet was thought to have a beneficial effect on the connection. In order to determine what affect the loading error had on the results of test unit 2 and if the performance would be repeatable, a retest of the complete joint penetration weld with reinforcing fillet was selected for test 4. Test unit 4 reached an ultimate displacement ductility of 3 which was lower than test unit 2. The failure was in the same region as in test 2, at the toe of the weld.

### 5.2 Conclusions

In regards to the current practice in Alaska, a fillet weld, the system did not conform to the capacity design principles as mentioned before but would be acceptable if very small displacements were needed. The structure was not damaged and did not have significant strength degradation until ductility 1.5 so a reliable ductility level of 1 would be reasonable for this system. While it is a low ductility level, it was the simplest and cheapest of the systems to construct. It should be mentioned again that the failure did occur in the weld and if there is any concern over the performance, an improved connection should be considered.
The complete joint penetration weld with reinforcing fillet was tested in test 2 and test 2 had the largest displacement capacity of all the test units but in regards to a reliable ductility level, it is only a two. Test 4 has the same reliable ductility level even though it had a different ultimate displacement capacity. Both test units had lost at least 20% of their strength during ductility 3 cycles. Test unit two had shown signs of a plastic hinge forming but test unit four did not have such a ductile response and had a rather sudden failure.

The complete joint penetration weld by itself had lower displacement capacity and a reliable ductility level of 1 like the fillet weld in the current practice. It is interesting to note that the unit never lost 20% of the ultimate strength but failed in fracture prior. Test unit three had the failure occur in the weld as in test unit 1.

### 5.3 Future work

The results from test units 2 and 4 show that the reinforcing fillet weld added to the complete joint penetration weld helped the stress concentration at the joint. The reduced stress concentration led to the failure staying out of the weld itself. It would be valuable to do a test with a smaller fillet weld other than ¾”. With a smaller fillet weld, there would be less welding, resulting in possible reduction of the heat affected area such that cracking would not occur there and full capacity of the section could be achieved.

Since all the failures occurred with in the joint and not in the pile as desired, relocating the plastic hinge would be an avenue worth pursuing. With the low level of performance in test unit one, the current practice of Alaska, a study to find suitable retrofit
options in order to bring these structures up to an acceptable level of performance is highly recommended.
REFERENCES


6.1 Pin support detail drawings

Figure 6.1: Pin assembly
Figure 6.2: Lower base assembly detail
Figure 6.4: Upper base assembly detail
Figure 6.5: Upper base assembly detail continued
Figure 6.6: Pin shoe pieces detail
Figure 6.7: Pin shoe detail
Figure 6.8: Angle detail
Figure 6.9: Pin
Enerpac CATG-200
degree tilt saddle
200 ton capacity

Figure 6.10: Pin sleeve detail
6.2 Actuator Shoe Detail Drawing

Figure 6.11: Actuator shoe detail
6.3 Randy Dempsey Certificates of Qualification

Figure 6.12: Certificate of test and approval of welding process and qualification of operation of welding equipment
Figure 6.13: NCDOT certificate for welding

North Carolina Department of Transportation
Division of Highways
Materials and Tests Unit

CERTIFICATE FOR WELDING
Presented To:

Randy D. Dempsey

This certifies the above named welder satisfactorily passed the North Carolina Department of Transportation 6G PIPE WELDER Qualification test as administered by the Materials and Tests Unit of the North Carolina Department of Transportation and in accordance with AWS D1.1–2006, Section 4 Structural Welding Code.

This qualifies the welder to weld fillet welds, groove welds, and pipe welds of unlimited thickness.

Limitations include:

- Vertical
- Overhead
- Horizontal
- Flat

Date Tested: 2/22/2008
Test Approved By: Steven C. Walton, Metals Engineer
Materials and Test Unit
Figure 6.14: American Welding Society certificate of welding inspector
Figure 6.15: American Welding Society certificate of welding educator
Figure 6.16: Certificate of achievement in liquid penetrant method
Figure 6.17: Certificate of achievement in ultrasonic method
Figure 6.18: Certificate of achievement in Magnetic Particle method
6.4 Test 2 Quality Control Documents

6.4.1 Test 2, Welding Procedure Specification Report

Figure 6.19: WPS for Test 2
Figure 6.20: WPS for Test 2 weld detail
## 6.4.2 Test 2, June 13, 2008 Quality Control Report

### QA Inspection Check List

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<td>Welder's Name:</td>
<td>Justin Green</td>
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<td>QA Inspector:</td>
<td>Randy Dempsey, CWI/CWE</td>
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</table>
note 1: An electrode oven was delivered to the site. The E7018HR (9 hour exposure limit rods) electrodes were delivered in a hermetically sealed container and placed in the oven immediately after breaking the seal.

note 2: The North and South pipe piles were beveled using a grinder and all mill scale and rust within 1” of the area to be welded was removed.

note 3: A 2”x 3/16” flat bar (w/ MTR) was formed and installed in each pipe pile with a CJP weld aligned to the neutral axis and welded continuous to the pipe with a 1/4” extension for the root opening + 1/16” to 1/8” fit-up tolerance.
Figure 6.22: Photo of welding backing ring
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<td>QA Inspector: CWI/CWE</td>
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<td>Follow-Up UT of Groove Weld</td>
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<td>Visual Inspection of Fillet Weld</td>
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</table>

**note 1:** The power to the electrode oven was interrupted. The electrodes from the oven were returned to the Buckner facility for re-drying and a new box of E7018HR electrodes were opened.
note 2: Due to flange tilt mill tolerance issues on the cap beam, 1/16" to 1/8" was removed (using a grinder) from the extension of the backing bar as needed to improve the joint fit-up.

Figure 6.23: photo of fit-up

Figure 6.24: photo of fit-up

note 3: Although preheat was not required due to the 70°F lab temperature, a Makita Thermocouple Heat Gun (model HG 1100) was used to raise the base metal temperature to approximately 100°F to reduce the cooling rate of the weld metal.

Figure 6.25: Makita Thermocouple Heat Gun
note 4: Interpass temperature was monitored using an EDL Pocket-Probe (model NMP) Pyrometer, which indicated temperatures from 280°F to 320°F.

Figure 6.26: ELD Pocket-Probe Pyrometer

note 5: All slag was removed with a chipping hammer. The start of some welds was contoured to a concave finish prior to covering with additional weld metal. Any anomalous material or weld discontinuity that might be detrimental to the integrity of the completed weld was removed using a wire brush or grinder.

note 6: The North Pile was welded by Justin Green and the South Pile was welded by Moises Sanchez. Both grooves were filled to the full cross section of the pipe member and found to be visually acceptable in accordance with AWS D1.1 2006 Table 6.1.
### QA Inspection Check List

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<td>Refer to the UT Inspection Report</td>
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**note 1:** The electrode oven is working correctly.
note 2: Although preheat was not required due to the 70° lab temperature, a Makita Thermocouple Heat Gun (model HG 1100) was used to raise the base metal temperature to approximately 100° F to reduce the cooling rate of the weld metal.

note 3: Interpass temperature was monitored using an EDL Pocket-Probe (model NMP) Pyrometer, which indicated temperatures from 260° F to 300° F.

note 4: All slag was removed using a chipping hammer. Any anomalous material or weld discontinuity that might be detrimental to the integrity of the completed weld was removed using a wire brush or grinder.

note 5: The leg and the throat was inspected using a 3/4" G.A.L. weld gage with a flash light as a luminous aid. The profile of the completed weld was improved using a grinder. The completed weld was found to be visually acceptable in accordance with AWS D1.1 2006 Figure 5.4 and Table 6.1.
6.4.5 Test 2, Quality Control Report Commentary

North Carolina Department of Transportation

QA Inspection Commentary

Project Description

Alaska DOT

Part Description

Owner Representative: Kendra Cookson

NCSU Constructed

Fabricator Name: Buckner Companies

Facilities Lab

Justin Green, Moises

Weld Location: Sanchez

QA Inspector: CWI/CWE

Randy Dempsey,
C1. If the cap beam was assembled with the stipulation that the bottom side needs to be flat by pushing the mill tolerance to the top, the fit-up and weld quality at the root could be improved.

C2. Purchasing the backing rings (http://www.robvon.com/html/backing.html) may prove to be a more practical and efficient method for actual production conditions.

C3. Due to the low interpass temperatures that were recorded, a WPS that stipulates 1/8" electrodes for passes 1, 2, and 3, but permits 5/32" electrodes for all subsequent passes could improve efficiency of production conditions.

C4. The approximate labor that was recorded (excluding QA and NCSU involvement) included 16 man hours for beveling the pipe and attaching the backing ring, 22 man hours for the groove weld, 2 hours for the UT (excluding travel time) with no flaws detected and 20 man hours for the 3/4" fillet weld.
### 6.4.6 Test 2, Ultrasonic Testing Report

#### Figure 6.29: Test 2 UT inspection report

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<th>Indication</th>
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<td>S1</td>
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<td>2</td>
<td>20°</td>
<td>A</td>
<td>S1</td>
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*Note: N/A for weld.*

**Surface indications (will weed)**

**Due to Gaging**

---

We, the undersigned, certify that the statements in this record are correct and that the welds were prepared and tested in accordance with the requirements of ASME Code Section IX, Structural Welding Code. 3006.1.1. Structural Welding Code.

Test Date: 12/30/06

Inspected by: [Signature]

Tested by: [Signature]
6.5 Test 3 Quality Control Documents

6.5.1 Test 3, Welding Procedure Specification Report

ANNEX N

WELDING PROCEDURE SPECIFICATION (WPS) Yes ✗
PREQUALIFIED ✗ QUALIFIED BY TESTING ___________
Or PROCEDURE QUALIFICATION RECORDS (PQR) Yes

Identification #: LPS11US
Revision: Date: By:
Authorized by: Jerry Castle Date: 8/5/08
Type: Manual [ ] Semi-Automatic [ ] Machine [ ] Automatic [ ]

POSITION
Position of groove: Horizontal/Overhead Fillet // Overhead
Vertical Position: Up [ ] Down [ ]

ELECTRICAL CHARACTERISTICS
Transfer Mode (GMAW): Short-circuiting [ ] Spray [ ] Other [ ]
Current: AC [ ] DCEN [ ] DCEP [ ] Pulsed [ ]
Tungsten Electrode (GTAW)
Size: __________________Type: __________________

BASE METAL
Material Spec.: ASTM A572 / A573 Grade 50 for HP section
Type of Grade: ___
Thickness: groove ___ fillet ___
Diameter (Pipe): 10"

FILLER METALS
AWS Specification: A 5.1
AWS Classification: E7018

SHIELDING
Flux: N/A Gas: N/A
Composition: __________________ Flow rate: __________________
Electrode-Flux (Class): __________________ Gas Cup Size: __________________

PREHEAT
Preheat temp., Min: 70 (table 3.2 note A)
Interpass Temp., Min: 70F Max: 500F

POSTWELD HEAT TREATMENT
Temp.: N/A Time: N/A

WELDING PROCEDURES

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<th>Process</th>
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<td>E7018</td>
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<td>100 – 130</td>
<td>20 – 24</td>
<td>5 to 7 in/min</td>
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Figure 6.30: Test 3 WPS
Figure 6.31: Test 3 WPS weld detail
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<td>QA Inspector: CWI/CWE</td>
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<tr>
<td>Visual Inspection of Fillet Weld</td>
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note 1: An electrode oven was delivered to the site. The E7018 (4 hour exposure limit rods) electrodes were delivered in a hermetically sealed container and placed in the oven within one hour after breaking the seal. The oven was plugged into an outlet on the inside of the lab to ensure an uninterrupted power source.

Figure 6.32: E7018 Electrodes

note 2: The North and South pipe piles were beveled to a 45° angle using a grinder and all mill scale and rust within 1" of the area to be welded was removed. The bevel angle was inspected using a mechanical protractor. One area on the North Pile was found to be less than the specified angle and was corrected prior to fit-up of the backing bar.

Figure 6.35: Mechanical Protractor  Figure 6.34: Bevel angel  Figure 6.33: South pile grinding
A 2"x 3/16" flat bar was formed and installed in each pipe pile with the CJP weld that is transverse to the length of the material aligned to the neutral axis of the pipe. The full length of the flat bar was welded continuous to the pipe with a 1/4" extension for the root opening + 1/16" fit-up tolerance. Tack welds placed in the area to be groove welded were removed by grinding.
note 4: The 50° F preheat was not necessary due to the thickness of the material and the atmospheric conditions at the work site being recorded at 98° F using an air thermometer that was placed in the shade at the same elevation and location as the material to be welded.
6.5.3 Test 3, August 11, 2008 Quality Control Report

North Carolina Department of Transportation

**QA Inspection Check List**

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**Materials & Tests Unit**

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*see note 2

*see note 3 and 4

*see note 5

acceptable, see note 6
note 1: The electrodes and electrode oven were inspected and found to be hot and undisturbed from the previous activity.

Figure 6.40: Electrode Oven

note 2: The nt fit-up was acceptable without making adjustments to the backing bar.

Figure 6.41: Oxygen/acetylene torch

note 3: Although preheat was not required due to the 70° lab temperature, an oxygen/acetylene torch was used to drive away moisture and raise the base metal temperature to approximately 125° to reduce the cooling rate of the weld metal.
note 4: Interpass temperature was monitored using 248° and 302° Nissen® Temperature Sticks. Due to one welder alternating between pipe piles, interpass temperatures did not exceed 302°.

Figure 6.42: Nissen® Temperature sticks

Figure 6.43: Temperature monitoring

note 5: All slag was removed with a chipping hammer. The start of some welds was contoured to a concave finish prior to covering with additional weld metal. Any anomalous material or weld discontinuity that could have been detrimental to the integrity of the completed weld was removed using a wire brush or grinder.

Figure 6.44: Slag removal
note 6: The North and South Pipe Piles were welded by Justin Green. Both grooves were filled to the full cross section of the pipe member and found to be visually acceptable in accordance with AWS D1.1 2006 Table 6.1.
**6.5.4 Test 3, August 12, 2008 Quality Control Report**

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**Project Description**
- **Owner Representative:** Kendra Cookson
- **Fabricator Name:** Buckner Companies
- **Welder's Name:** Justin Green
- **QA Inspector:** Randy Dempsey, CWI/CWE

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**Weld Location**
- North & South Pipe Pile

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**note 1:** The electrodes and electrode oven were inspected and found to be hot and undisturbed from
the previous activity.

note 2: Although preheat was not required due to the 70° lab temperature, an oxygen/acetylene torch was used to drive away moisture and raise the base metal temperature to approximately 125° to reduce the cooling rate of the weld metal.

note 3: Interpass temperature was monitored using a 248° and 302° Nissen® Temperature Sticks. Due to one welder alternating between pipe piles, interpass temperatures did not exceed 248°.

note 4: All slag was removed with a chipping hammer. Any anomalous material or weld discontinuity that might be detrimental to the integrity of the completed weld was removed using a wire brush or grinder.

note 5: The North and South Pipe Piles were welded by Justin Green. Both grooves were filled to the full cross section of the pipe member and after repairing several small deficiencies, found to be visually acceptable in accordance with AWS D1.1 2006 Table 6.1. A grinder was used to improve the profile of the completed weld.

Figure 6.45: Final weld on North column
6.5.5 Test 3, August 13, 2008 Quality Control Report

North Carolina Department of Transportation

QA Inspection Check List

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**Witness UT Testing of Groove Weld**

8/13/08  see UT report

**Groove Weld Repair**

8/13/08  see note 2

**Follow-Up UT of Groove Weld**

8/13/08  see UT report

**Pre-Heat & Interpass Temp. fillet weld**

**Interpass Cleaning, fillet weld**

**Visual Inspection of Fillet Weld**

---

**Note 1:** The electrodes and electrode oven were inspected and found to be hot and undisturbed from the previous activity.

**Note 2:** Although preheat was not required due to the 70° lab temperature, an oxygen/acetylene torch
was used to drive away moisture and raise the base metal temperature to approximately 125° to reduce
the cooling rate of the weld metal.

Figure 6.46: UT inspection
6.5.6 Test 3, Quality Control Report Commentary

North Carolina Department of Transportation

QA Inspection Commentary

Alaska DOT

Project Description

Part Description

Materials & Tests Unit

Bridge Bent
Test 3

Owner Representative:
Kendra Cookson

NCSU Constructed Facilities

Lab

Fabricator Name:
Buckner Companies

Welder's Name:
Justin Green

QA Inspector:
Randy Dempsey, CWI/CWE

Weld Location
North & South
Pipe Pile
Comments from Test 2

C1. If the cap beam was assembled with the stipulation that the bottom side needs to be flat by pushing the mill tolerance to the top, the fit-up and weld quality at the root could be improved.

C2. Purchasing the backing rings (http://www.robvon.com/html/backing.html) may prove to be a more practical and efficient method for actual production conditions.

C3. Due to the low interpass temperatures that were recorded, a WPS that stipulates 1/8” electrodes for passes 1, 2, and 3, but permits 5/32” electrodes for all subsequent passes could improve efficiency of production conditions.

C4. The approximate labor that was recorded (excluding QA and NCSU involvement) included 16 man hours for beveling the pipe and attaching the backing ring, 22 man hours for the groove weld, 2 hours for the UT (excluding travel time) with no flaws detected and 20 man hours for the 3/4” fillet weld.

Additional Comments from Test 3

C5. The approximate production man-hours recorded were; 10 hours for beveling the pipe and attaching the backing ring, 18 hours for applying the groove weld, 4 hours for the UT inspection and 4 hours for weld repair.

C6. Due to the deficiency found with the groove bevel, a close inspection prior to backing bar fit-up during actual production is recommended to ensure that the specification of +10°, -0° is maintained.

C7. Close QA verification of the UT Testing and follow-up UT after weld repairs have been made is recommended to ensure that the proper code and section specifications are followed.
### 6.5.7 Test 3, Ultrasonic Testing Report

![Figure 6.47: UT inspection report](image)

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We, the undersigned, certify that the statements in this record are correct and that the welds were prepared and tested in accordance with the requirements of AC of ASME Sec. IX, Structural Welding Code.

Test date: __________

Manufacture or Contractor: DL, STATE

Inspected by: __________

Authorized by: __________

Date: __________

---

Figure 6.47: UT inspection report

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6.6 Test 4 Quality Control Documentation

6.6.1 Test 4, Welding Procedure Specification Report

![Welding Procedure Specification](image)

**Figure 6.48**: WPS for test 4
Figure 6.49: Weld Detail for Test 4
### 6.6.2 Test 4, September 15, 2008 Quality Control Report

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#### Materials & Tests Unit

**Owner Representative:** Kendra Cookson  
**Fabricator Name:** Buckner Companies  
**Welder’s Name:** Justin Green  
**QA Inspector:** Randy Dempsey, CWI/CWE  
**Project Location:** Facilities Lab  
**NCSU Constructed**  
**Bridge Bent**  
**Test 4**  
**Weld Location**  
**North & South**  
**Pipe Pile**  

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**note 1:** The electrode oven and electrodes (E7018, 4 hour exposure limit rods) from the previous test have remained on site and will be used for today’s operations. According to NCSU sources, the oven’s power source has been uninterrupted.

**note 2:** The North and South pipe piles were beveled to a 45° angle using a grinder and all mill scale and rust within 1” of the area to be welded was removed. The bevel angle was inspected using a tri-square.
note 3: A 2"x 3/16" flat bar was pre-formed to an approximate diameter and installed in each pipe pile with the CJP weld that is transverse to the length of the material aligned to the neutral axis of the pipe. The full length of the flat bar was welded continuous to the pipe with a 1/4" extension for the root opening + 1/16" fit-up tolerance. Tack welds placed in the area to be groove welded were removed by grinding.

Figure 6.51: Pre-formed flat bar

note 4: The 50°F preheat was not necessary due to the thickness of the material and the atmospheric conditions at the work site being 73°F for a low and 85°F for a high, according to weather.com.
# 6.6.3 Test 4, September 17, 2008 Quality Control Report

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**note 1:** The electrodes and electrode oven were inspected and found to be hot and undisturbed from the previous activity.

**note 2:** The joint fit-up was acceptable after making adjustments to the backing bar by grinding excess material to close the gap.

**note 3:** Although preheat was not required due to the 70° lab temperature, an oxygen/acetylene torch was used to drive away moisture and raise the base metal temperature to approximately 125° to
reduce the cooling rate of the weld metal.

**note 4:** Interpass temperature was monitored using 248° and 302° Nissen® Temperature Sticks. Interpass temperatures did not exceed 302°.

**note 5:** All slag was removed with a chipping hammer. The start of some welds was contoured to a concave finish prior to covering with additional weld metal. Any anomalous material or weld discontinuity that could have been detrimental to the integrity of the completed weld was removed using a wire brush or grinder.
## 6.6.4 Test 4, September 18, 2008 Quality Control Report

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**Owner Representative:** Kendra Cookson

**Project Location:** Facilities Lab

**Fabricator Name:** Buckner Companies

**Welder’s Name:** Justin Green, Chris

**QA Inspector:** CWI/CWE

**Date**

| 9/18/08 |

**Comments**

- see note 1

**Consumable Storage/Control**

- see note 1

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<td>Visual Inspection of Groove Weld</td>
<td></td>
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<tr>
<td>Witness UT Testing of Groove Weld</td>
<td>9/18/08</td>
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<tr>
<td>Groove Weld Repair</td>
<td>9/18/08</td>
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<tr>
<td>Follow-Up UT of Groove Weld</td>
<td>9/18/08</td>
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<tr>
<td>Pre-Heat &amp; Interpass Temp. fillet weld</td>
<td>9/18/08</td>
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<tr>
<td>Interpass Cleaning, fillet weld</td>
<td>9/18/08</td>
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<tr>
<td>Visual Inspection of Fillet Weld</td>
<td>9/18/08</td>
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</tbody>
</table>

**note 1:** The electrode oven is working correctly.

**note 2:** Although preheat was not required due to the 70° lab temperature, an oxy/acetylene torch was used to raise the base metal temperature to approximately 100° F to reduce the cooling rate of
the weld metal.

**note 4:** Interpass temperature was monitored using 248° and 302° Nissen® Temperature Sticks. Interpass temperatures did not exceed 302°.

**note 4:** All slag was removed using a chipping hammer. Any anomalous material or weld discontinuity that might be detrimental to the integrity of the completed weld was removed using a wire brush or grinder.

**note 5:** The leg and the throat was inspected using a 3/4” G.A.L. weld gage with a flash light as a luminous aid. The profile of the completed weld was improved using a grinder. The completed weld was found to be visually acceptable in accordance with AWS D1.1 2006 Figure 5.4 and Table 6.1.
6.6.5 Test 4, Ultrasonic Testing Report

![Figure 6.52: UT inspection report](image)

<table>
<thead>
<tr>
<th>Line Number</th>
<th>Piece Number</th>
<th>Transducer Angle</th>
<th>From Face</th>
<th>Log&quot;</th>
<th>Indication Level</th>
<th>Reference Level</th>
<th>Attenuation Factor</th>
<th>Indication Rating</th>
<th>Angular Distance (Sound Path)</th>
<th>Depth from &quot;A&quot; Surface</th>
<th>From X</th>
<th>From Y</th>
<th>Discontinuity Elevation</th>
<th>Remarks</th>
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</thead>
<tbody>
<tr>
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<td>N</td>
<td>70&quot;</td>
<td>A 3/8</td>
<td>1&quot;</td>
<td>1/8&quot;</td>
<td></td>
<td></td>
<td></td>
<td>Reject</td>
<td>1/8&quot;</td>
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<td></td>
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</tr>
<tr>
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<td>A 3/8</td>
<td>1&quot;</td>
<td>1/8&quot;</td>
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<tr>
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<td>1&quot;</td>
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<td>Reject</td>
<td>1/8&quot;</td>
<td></td>
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<td></td>
<td>Add</td>
</tr>
</tbody>
</table>

We, the undersigned, certify that the statements in this record are correct and that the welds were prepared and tested in accordance with the requirements of AC of AM 21.1, 2005 Structural Welding Code.

Test Date: 3/18/28
Manufacturer or Contractor: JUCOAK STEEL

Inspected by: 
Authorized by: 
Date: 

Figure 6.52: UT inspection report
6.7 Test 1: Strain gauge hysteresis

Figure 6.53: Strain gauge N-N-70 hysteresis

Figure 6.54: Strain gauge N-M-70 hysteresis

Figure 6.55: Strain gauge S-N-70 hysteresis

Figure 6.56: Strain gauge S-M-70 hysteresis

Figure 6.57: Strain gauge S-S-70 hysteresis

Figure 6.58: Strain gauge S-N-20 hysteresis
Figure 6.59: Strain gauge S-N-12 hysteresis

Figure 6.60: Strain gauge S-N-4 hysteresis

Figure 6.61: Strain gauge S-M-20 hysteresis

Figure 6.62: Strain gauge S-M-12 hysteresis

Figure 6.63: Strain gauge S-M-4 hysteresis

Figure 6.64: Strain gauge S-S-20 hysteresis
Figure 6.65: Strain gauge S-S-12 hysteresis

Figure 6.66: Strain gauge S-S-4 hysteresis

Figure 6.67: Strain gauge S-S-4T hysteresis

Figure 6.68: Strain gauge S-N-4T hysteresis

Figure 6.69: Strain gauge N-S-20 hysteresis

Figure 6.70: Strain gauge N-S-12 hysteresis
Figure 6.71: Strain gauge N-S-4 hysteresis

Figure 6.72: Strain gauge N-S-4T hysteresis

Figure 6.73: Strain gauge N-N-20 hysteresis

Figure 6.74: Strain gauge N-N-12 hysteresis

Figure 6.75: Strain gauge N-N-4T hysteresis

Figure 6.76: Strain gauge N-N-4 hysteresis
Figure 6.77: Strain gauge N hysteresis

Figure 6.78: Strain gauge N-M-12 hysteresis

Figure 6.79: Strain gauge N-M-20 hysteresis

Figure 6.80: Strain gauge 30 hysteresis

Figure 6.81: Strain gauge CB N-bottom
6.8 Test 1: Optotrack
Hysteresis

Figure 6.82: Strain hysteresis location S-S-6

Figure 6.83: Strain hysteresis location S-S-10

Figure 6.84: Strain hysteresis location S-S-14

Figure 6.85: Strain hysteresis location S-S-18

Figure 6.86: Strain hysteresis location S-1-6

Figure 6.87: Strain hysteresis location S-1-10
Figure 6.88: Strain hysteresis location S-1-14

Figure 6.89: Strain hysteresis location S-1-18

Figure 6.90: Strain hysteresis location S-2-6

Figure 6.91: Strain hysteresis location S-2-10

Figure 6.92: Strain hysteresis location S-2-14

Figure 6.93: Strain hysteresis location S-2-18
Figure 6.94: Strain hysteresis location S-3-6

Figure 6.95: Strain hysteresis location S-3-10

Figure 6.96: Strain hysteresis location S-3-14

Figure 6.97: Strain Hysteresis location S-3-18

Figure 6.98: Strain hysteresis location S-N-6

Figure 6.99: Strain hysteresis location S-N-10
Figure 6.100: Strain hysteresis location S-N-14

Figure 6.101: Strain hysteresis location S-N-18
6.9 Test2: Strain gauge hysteresis

Figure 6.102: Strain gauge N-S-11 hysteresis

Figure 6.105: Strain gauge S-N-3 hysteresis

Figure 6.103: Strain gauge N-S-19 hysteresis

Figure 6.106: Strain gauge S-N-11 hysteresis

Figure 6.104: Strain gauge N-S-27 hysteresis

Figure 6.107: Strain gauge S-N-19 hysteresis
Figure 6.108: Strain gauge S-N-27 hysteresis

Figure 6.109: Strain gauge S-N-35 hysteresis

Figure 6.110: Strain gauge S-N-70 hysteresis

Figure 6.111: Strain gauge S-S-3 hysteresis

Figure 6.112: Strain gauge S-S-19 hysteresis

Figure 6.113: Strain gauge S-S-27 hysteresis
Figure 6.114: Strain gauge S-S-35 hysteresis

Figure 6.115: Strain gauge S-S-70 hysteresis

Figure 6.116: Strain gauge N-N-3 hysteresis

Figure 6.117: Strain gauge N-N-11 hysteresis

Figure 6.118: Strain gauge N-N-19 hysteresis

Figure 6.119: Strain gauge N-N-27 hysteresis
Figure 6.120: Strain gauge N-N-35 hysteresis

Figure 6.121: Strain gauge N-N-70 hysteresis

Figure 6.122: Strain gauge CB-N

Figure 6.123: Strain gauge CB -S hysteresis
6.10 Test 2: Optotrack strains

Figure 6.124: Strain hysteresis at location S-S-6

Figure 6.125: Strain hysteresis at location S-S-8

Figure 6.126: Strain hysteresis at location S-S-10

Figure 6.127: Strain hysteresis at location S-S-12

Figure 6.128: Strain hysteresis at location S-S-14

Figure 6.129: Strain hysteresis at location S-S-16
Figure 6.130: Strain hysteresis at location S-S-18

Figure 6.133: Strain hysteresis at location S-S-26

Figure 6.131: Strain hysteresis at location S-S-20

Figure 6.134: Strain hysteresis at location S-S-28

Figure 6.132: Strain hysteresis at location S-S-22

Figure 6.135 Strain hysteresis at location S-S-30
Figure 6.136: Strain hysteresis at location S-1-6

Figure 6.137: Stain hysteresis at location S-1-10

Figure 6.138: Strain hysteresis at location S-1-18

Figure 6.139: Strain hysteresis at location S-1-22

Figure 6.140: Strain hysteresis at location S-1-26

Figure 6.141: Strain hysteresis at location S-2-6
Figure 6.142: Strain hysteresis at location S-2-10

Figure 6.143: Strain hysteresis at location S-2-14

Figure 6.144: Strain hysteresis at location S-2-18

Figure 6.145: Strain hysteresis at location S-2-23

Figure 6.146: Strain hysteresis at location S-2-27

Figure 6.147: Strain hysteresis at location S-3-6
Figure 6.148: Strain hysteresis at location S-3-10

Figure 6.149: Strain hysteresis at location S-3-14

Figure 6.150: Strain hysteresis at location S-3-18

Figure 6.151: Strain hysteresis at location S-3-22

Figure 6.152: Strain hysteresis at location S-3-27

Figure 6.153: Strain hysteresis at location S-N-2
Figure 6.154: Strain hysteresis at location S-N-4

Figure 6.155: Strain hysteresis at location S-N-6

Figure 6.156: Strain hysteresis at location S-N-8

Figure 6.157: Strain hysteresis at location S-N-11

Figure 6.158: Strain hysteresis at location S-N-14

Figure 6.159: Strain hysteresis at location S-N-16
Figure 6.160: Strain hysteresis at location S-N-18

Figure 6.161: Strain hysteresis at location S-N-20

Figure 6.162: Strain hysteresis at location S-N-24

Figure 6.163: Strain hysteresis at location S-N-26

Figure 6.164: Stain hysteresis at location S-N-30

Figure 6.165: Stain hysteresis at location S-N-31
Figure 6.166: Stain hysteresis at location S-N-34
6.11 Test 4: Strain gauge hysteresis

Figure 6.167: Stain gauge N-N-35 hysteresis

Figure 6.168: Strain gauge N-N-19 hysteresis

Figure 6.169: Strain gauge N-N-11 hysteresis

Figure 6.170: Strain gauge N-N-3 hysteresis

Figure 6.171: Stain gauge N-S-35 hysteresis

Figure 6.172: Strain gauge N-S-27 hysteresis
Figure 6.173: Strain gauge N-S-3 hysteresis

Figure 6.174: Strain gauge N-S-70 hysteresis

Figure 6.175: Strain gauge N-S-70 hysteresis

Figure 6.176: Strain gauge S-N-11 hysteresis

Figure 6.177: Strain gauge S-N-27 hysteresis

Figure 6.178: Strain gauge S-S-3 hysteresis
Figure 6.179: Strain gauge S-S-11 hysteresis

Figure 6.180: Strain gauge S-S-27 hysteresis

Figure 6.181: Strain gauge S-S-35 hysteresis

Figure 6.182: Strain gauge S-S-70 hysteresis
6.12 Test 4: Optotrack hysteresis

Figure 6.183: Strain hysteresis at location S-S-6

Figure 6.184: Strain hysteresis at location S-S-8

Figure 6.185: Strain hysteresis at location S-S-10

Figure 6.186: Strain hysteresis at location S-S-12

Figure 6.187: Strain hysteresis at location S-S-14

Figure 6.188: Strain hysteresis at location S-S-16
Figure 6.189: Strain hysteresis at location S-S-18

Figure 6.190: Strain hysteresis at location S-S-20

Figure 6.191: Strain hysteresis at location S-S-22

Figure 6.192: Strain hysteresis at location S-S-23

Figure 6.193: Strain hysteresis at location S-S-26

Figure 6.194: Strain hysteresis at location S-S-28
Figure 6.195: Strain hysteresis at location S-S-30

Figure 6.196: Strain hysteresis at location S-S-32

Figure 6.197: Strain hysteresis at location S-S-34

Figure 6.198: Strain hysteresis at location S-S-35

Figure 6.199: Strain hysteresis at location S-1-4

Figure 6.200: Strain hysteresis at location S-1-8
Figure 6.201: Strain hysteresis at location S-1-12

Figure 6.202: Strain hysteresis at location S-1-16

Figure 6.203: Strain hysteresis at location S-1-20

Figure 6.204: Strain hysteresis at location S-1-28

Figure 6.205: Strain hysteresis at location S-1-32

Figure 6.206: Strain hysteresis at location S-2-4
Figure 6.207: Strain hysteresis at location S-2-12

Figure 6.208: Strain hysteresis at location S-2-16

Figure 6.209: Strain hysteresis at location S-2-20

Figure 6.210: Strain hysteresis at location S-2-24

Figure 6.211: Strain hysteresis at location S-3-4

Figure 6.212: Strain hysteresis at location S-3-8
Figure 6.213: Strain hysteresis at location S-3-12

Figure 6.214: Strain hysteresis at location S-3-16

Figure 6.215: Strain hysteresis at location S-3-20

Figure 6.216: Strain hysteresis at location S-3-24

Figure 6.217: Strain hysteresis at location S-3-28

Figure 6.218: Strain hysteresis at location S-3-32
Figure 6.219: Strain hysteresis at location S-N-1

Figure 6.220: Strain hysteresis at location S-N-3

Figure 6.221: Strain hysteresis at location S-N-5

Figure 6.222: Strain hysteresis at location S-N-7

Figure 6.223: Strain hysteresis at location S-N-9

Figure 6.224: Strain hysteresis at location S-N-11
Figure 6.225: Strain hysteresis at location S-N-13

Figure 6.226: Strain hysteresis at location S-N-15

Figure 6.227: Strain hysteresis at location S-N-17

Figure 6.228: Strain hysteresis at location S-N-19

Figure 6.229: Strain hysteresis at location S-N-21

Figure 6.230: Strain hysteresis at location S-N-23
Figure 6.231: Strain hysteresis at location S-N-25

Figure 6.232: Strain hysteresis at location S-N-27

Figure 6.233: Strain hysteresis at location S-N-29

Figure 6.234: Strain hysteresis at location S-N-31

Figure 6.235: Strain hysteresis at location S-N-3