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Seismic testing of existing full-scale pile-to-deck connections: precast prestressed and steel piles

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Seismic Testing of Existing Full-Scale Pile-to-deck Connections: 
Precast Prestressed and Steel Piles

A Thesis submitted in partial satisfaction of the requirements 
for the degree of Master of Science

in

Structural Engineering

by

Jared Keith Bell

Committee in charge:

Professor José I. Restrepo, Chair
Professor Ahmed Elgamal
Professor P. Benson Shing

2008
The Thesis of Jared Keith Bell is approved and it is acceptable in quality and form for publication on microfilm:

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Chair

University of California, San Diego

2008
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LIST OF SYMBOLS

\[ \Delta: \] Deck top displacement.
\[ \delta_c: \] Deformation component from rotation at the connection.
\[ \delta_p: \] Deformation component from pile flexibility.
\[ \varepsilon_c: \] Extreme concrete fiber compression strain.
\[ \varepsilon_{sme}: \] Strain at peak stress of confining reinforcement.
\[ \varepsilon_{smd}: \] Strain at peak stress of dowel reinforcement.
\[ \theta: \] Pile Rotation.
\[ \rho_k: \] Effective volume ratio of confining steel.
\[ \varphi: \] Section curvature.
\[ d_b: \] Dowel bar diameter.
\[ EI: \] Pile bending stiffness.
\[ f'_{cc}: \] Strength of confined concrete.
\[ f_{su}: \] Ultimate stress of the longitudinal steel.
\[ f_y: \] Yield stress of the longitudinal steel.
\[ f_{yh}: \] Yield stress of the confining steel.
\[ h: \] Pile height from collar pin to connection interface.
\[ l_p: \] Plastic hinge length.
\[ l_e: \] Effective pile unrestrained length.
\[ V: \] Lateral force on the pile.
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ABSTRACT OF THE THESIS

Seismic Testing of Existing Full-Scale Pile-to-deck Connections:
Precast Prestressed and Steel Piles

by

Jared Keith Bell

Master of Science in Structural Engineering

University of California, San Diego, 2008

Professor José I. Restrepo, Chair

This report details the seismic behavior of full-scale pile-to-deck connections
existing at the Port of Los Angeles.

The fist specimen was a prestressed precast 24” octagonal pile connected to a
deck representing a tributary section of the wharf superstructure. The second
specimen was a steel HP pile connected to a similar deck.

The first specimen was designed to replicate a redundant lateral system
existing at Berth 145 in the Port of Los Angeles. This redundant system’s seismic
behavior affects the details of the retrofit process, and it was hoped that demonstration
of sufficient ductility would limit modification to the existing system. The load protocol for this test is based on the Port of Los Angeles’ strain based limit states, Operational Level Earthquake (OLE) and Contingency Level Earthquake (CLE).

The second test was designed to replicate a steel pile and concrete deck gravity system, part of a landside retrofit to allow for the support of a crane rail at another berth, Berth 226, at the Port of Los Angeles. With a lateral system already present, these steel piles were only required to maintain their gravity load capacity after a seismic event.

Both test units achieved deformation capacities of at least 3% drift ratio and maintained their gravity load capacity throughout.
1 Introduction

Two tests were carried out in specimens representing specific existing connection conditions and details in wharves constructed before seismic code with explicit provisions to achieve ductility were implemented at the Port of Los Angeles. The first specimen represented connections located at Berth 145 (Figure 1-1a) which consisted of a 24” octagonal precast prestressed pile with eight #9 dowels grouted into the pile extending into a reinforced concrete deck.

In this structure (Berth 145), the crane rail required a new line of columns (line G) and an extension of the superstructure deck to be placed closely behind the row of columns (Line F) whose connection were tested in this study. This new system is designed to replace the existing row of 24” octagonal piles (line F).

The shortest pile provides the least flexibility and is therefore critical in our analysis of the entire wharf movement. As the wharf deck moves as a unit, all piles will experience similar drift causing the shortest to experience the highest drift ratio. The retrofit’s purpose is to allow for crane rail placement (line G) and includes new primary seismic columns to replace the test specimen type column (line F). However, removing the connection in line F is an expensive and time consuming procedure.

The goal of this test was to evaluate the seismic behavior of the existing pile-to-deck connection, accounting for the presence of a restraining retrofit and to verify if the specimen (connection at line F), subjected to quasi-static cyclic loading, has an adequate ductile behavior and has sufficient displacement capacity to resist the design displacement of the rest of the wharf. More specifically, the test aims to determine
whether or not the pile-to-deck connection can function after plastic deformation occurs consistent with an event corresponding to an intensity with a probability of exceedance of 50% in 50 years which is named by POLA seismic code as Operational Level Earthquake (OLE).

If the connection satisfies the expectations, the retrofitting program that is already being implemented in the Port of Los Angeles, Berth 145 can be reduced by allowing the existing connection to remain as they are now.

The second connection test represents existing conditions at Berth 226 (Figure 1-1b). The specimen is a steel HP14x117 section pile, with eight #11 dowels welded to the flanges of the pile, extending into a reinforced concrete deck. This steel pile is part of a landside retrofit to allow for the support of a crane rail with a tributary vertical load of 300 kips per steel pile. The aim of the test was to verify if the connection is able to withstand the design displacement of the rest of the wharf during an OLE seismic event which is reach for a displacement of 2”.

The lateral-force-resisting system for the Berth 145 is comprised of 24” octagonal concrete piles. The Port of Los Angeles has defined two limit states for these seismic concrete piles based on strain. The lesser is the Operational Level Earthquake (OLE) performance state that limits the maximum displacement of the superstructure to 2”. This implies that at this limit state the wharf must remain operational after an event of such intensity. Also, at the OLE limit state, the damage on the structure should be visually observable and accessible. However, due to the special conditions in this case, accessibility alone is acceptable.
Given that the concrete piles at the landside row meet their Operational Level Earthquake (OLE) limit at 2” of displacement, the primary goal of the test was to evaluate if the steel piles and the connection were able to carry 300 kips of vertical load after displacing 2”. The secondary goal was to evaluate the seismic behavior of the new line of steel piles used to retrofit Berths 226 – 232 and other similar Berths at the Port of Los Angeles.

Figure 1-1: Wharf Transversal Section. Berth 226 (bottom)

The Port of Los Angeles Code for Seismic Design, Upgrade, and Repair of Container Wharves, 5/2004 defines two levels of design earthquake motions. For Operating Level Earthquakes (OLE) the performance criterion requires that forces and deformations not result in significant structural damage. More specifically, all damage
must be visually observable and accessible and operations must not be interrupted. For Contingency Level Earthquakes (CLE) the performance criterion allows forces and deformations to result in controlled inelastic structural behavior. Again, damage must be visually observable and accessible. However, temporary loss of operating time is allowed. In both cases, life safety and prevention of collapse are paramount.

The performance limits for OLE and CLE are defined by the strains on the steel and concrete at the connection and in the prestressed pile. In the case of OLE, for solid octagonal piles, at the pile head, the extreme concrete fiber in compression must not exceed .5% and the extreme tensile strain in the dowels must not exceed 1%. The in-ground portion of the pile has the same compression strain limit, but the incremental prestressing strain in the pile strands must not be more than .5%. To clarify, the dowels are bars grouted into the pile that extend into the deck reinforcing and are cast into the deck, while the prestressing strands exist solely in the pile.

In the case of CLE serviceability limit states, for solid octagonal piles, at the pile head, the extreme concrete fiber compression strain is governed by the equation:

$$\varepsilon_c = 0.004 + (1.4 \rho_s f_{yh} \varepsilon_{smd}) / f'_{cc} \quad \text{while} \quad 0.005 < \varepsilon_c < 0.020$$

The extreme tensile strain in the dowels must not exceed 5% or .6 $\varepsilon_{smd}$, which ever occurs first. For the in-ground portion of the pile, the compression strain is governed by the same equation, but instead is bounded by .005 and .008. The tensile strain must not exceed a total strain of 1.5% in the prestressing strands. $\rho_s$ is the effective volume ratio of confining steel, $f_{yh}$ is the yield stress of the confining steel, $\varepsilon_{smd}$ is the strain at peak stress of confining reinforcement, $\varepsilon_{smd}$ is the strain at peak stress of dowel
reinforcement, and $f'_{cc}$ is the strength of confined concrete approximated by $1.5f'_{c}$.

For further details refer directly to POLA Seismic Code (2004).
2 Literature Review

With respect to the precast prestressed column-deck connection, two relevant wharf pile-deck connection tests have been published. Sritharan and Priestley (1998) tested a new concept using headed bars for pile-to-deck connections that hoped to improve constructability while maintaining sufficient ductility. This moment resisting connection consisted of bulbed dowel bars extending from a pile into a 3’ deep deck with overlapping #9 headed bond bars also with bulbed ends (Figure 2-1). The deck contained simple top and bottom mat reinforcing. The specimen was constructed full-scale and tested in the inverted position as shown in Figure 1-2. The test results showed that sufficient displacement ductility was achieved (Figure 1-3).

![Figure 2-1: Proposed Pile-to-deck Connection](image)
Roeder et al. (2002) tested eight, two-thirds scale pile-deck connections. The main variables of the testing program were the reinforcing details of the piles. The
sizes of the dowel bars changed throughout the testing program, and they were bent either inward or outward in the deck depending on the specimen. The size and spacing of the reinforcing spiral also changed in different specimens, and two of the specimens did not consist of prestressed piles, whereas the others all did. The test specimens were built upright and then they were flipped over and tested in an inverted position. Figure 1-4 illustrates the test set-up. The deck of the test units was seated on shims, and the gap between the test floor and the deck was hydrostoned. The deck was then prestressed to the strong floor using high-strength threaded rods. Under this condition, the deck and joint flexibilities may not be properly modeled.

In this test program the piles were pushed and pulled with a horizontal hydraulic actuator. Axial load was applied to some test specimens using two high-strength threaded rods. It is not clear in the report if the axial force was actively controlled and maintained constant during the test. The piles were restrained transversely using a guiding mechanism. Table 1-1 details the specimens used in the Roeder testing program. The specified concrete strength in this test program was $f'_c=6,000$ psi, and actual strengths ranged between 6,800 and 9,880 psi.
Figure 1-4: Roeder et al. Test Set Up
Table 1-1: Roeder et al. Summary of Test Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Connection Type</th>
<th>Special Conditions</th>
<th>Goals of Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Extended Outward Bent Dowel (Fig. 2.1b)</td>
<td>No axial load.</td>
<td>Comparison with Specimen 3 shows the behavior of extended pile connections as compared to precast pile connections.</td>
</tr>
<tr>
<td>2</td>
<td>Extended Outward Bent Dowel with spiral reinforcement</td>
<td>No axial load. Spirals around dowels in deck.</td>
<td>Comparison with Specimen 1 shows effect of spiral reinforcement in connection zone.</td>
</tr>
<tr>
<td>3</td>
<td>Outward Bent Dowel (Fig. 2.1a)</td>
<td>No axial load.</td>
<td>Baseline connection for most comparisons.</td>
</tr>
<tr>
<td>4</td>
<td>Outward Bent Dowel (Fig. 2.1a)</td>
<td>With axial load.</td>
<td>Comparison with Specimen 3 shows the effect of axial load on connection performance.</td>
</tr>
<tr>
<td>5</td>
<td>Inward Bent Dowel (Fig. 2.2a)</td>
<td>With axial load.</td>
<td>Comparison with Specimen 4 shows the difference between outward and inward bent details.</td>
</tr>
<tr>
<td>6</td>
<td>T-Headed Dowel Bar (Fig. 2.1c)</td>
<td>With axial load.</td>
<td>Comparisons with Specimens 4 and 5 show the difference between bent dowel and T-Headed dowel details.</td>
</tr>
<tr>
<td>7</td>
<td>Bond Bar (Fig. 2.1c)</td>
<td>With axial load.</td>
<td>Comparisons with Specimens 4 and 5 show the difference between bent dowel and Bond Bar details.</td>
</tr>
<tr>
<td>8</td>
<td>Outward Bent Dowel but lighter deck reinforcement</td>
<td>With axial load.</td>
<td>Comparisons with Specimens 3 and 4 show the effect of the substantial shear reinforcement in the deck.</td>
</tr>
</tbody>
</table>
The main conclusions drawn by Roeder et al. (2002) indicated that while significant deterioration in the resistance was noted in these connection tests and the various connection details exhibited significant differences in behavior, all test specimens were able to tolerate large inelastic deformations while maintaining the basic integrity and compressive load capacity of the connection.

The limitations of the experimental results were evaluated by Roeder et al. and the following recommendations were made:

- Effects of increased axial load require further investigation, particularly for connections with precast concrete piles.
- Full scale test using large pile diameters and shorter piles are recommended as a decrease of ductility and deformation capacity can be expected. Tests are needed to quantify these effects.

Regarding to the steel pile-deck connection, no reference was found in the literature about test carried out on this type of connections.
3 Experimental Program

This chapter describes the geometry, reinforcement detailing, instrumentation and set up of the full size pile-to-deck connections that were tested.

3.1 Specimen A.1 – Prestressed octagonal concrete pile

3.1.1 Test Set Up

The specimen was built and tested upright and supported by a steel collar as shown in Figure 3-1. The collar was fixed from rotation using wooden blocks until the vertical and horizontal actuators were placed and pressurized. A horizontal hydraulic actuator was used for applying the lateral force to the system whilst the vertical actuator shown in the figure is used in order to keep the deck in a horizontal position. This set up is intended to represent the actual boundary conditions existing on the structure.

In determining the boundary conditions for the test of specimen A.1 it was important to accurately reproduce the point of inflection in the prestressed precast octagonal concrete pile. While the test specimen is just under 8 feet in length, the actual pile is on the order of 5 times longer. Due to soil-structure interaction, the pile bends at several locations with a point of inflection near the surface at approximately 8 feet below the deck soffit. To capture the behavior of the point of inflection it is necessary to ensure the prestressing strands are developed as they would be in the continuous pile. For this reason, the pile is extended almost 3 feet beneath the ideal pin boundary condition created with the steel collar.
A large moment frame of wide flange sections was used to prevent out of plane sway of the specimen. The specimen is a representation of a slice of a significantly long berth and therefore in the real berth, this slice would be restrained from moving out of plane. The large moment frames was used to correctly simulate these boundary conditions. The specimen is allowed to move vertically when the structure displaced horizontally, but remains planar due to slaved actuator behavior which will be discussed in more detail in the following Test Control section.

![Figure 3-1: General Dimensions of Specimen A.1](image)

3.1.2 Connection Detailing

Complex reinforcing patterns differ slightly along the length of any berth so a representative pattern was chosen to most accurately simulate the behavior of the
system. Further, additional transverse reinforcing was added to simulate the rigidity of the overlying concrete deck retrofit that is currently in place (Figure 3-2 and Figure 3-3).

Berth 145 has been retrofitted to allow the addition of a crane rail. This retrofit contains redundant seismic columns and an overlying deck that makes the specimen secondary for seismic behavior of the wharf. The seismic behavior of the test pile-to-deck connection is controlled by 8 #9 bars that are grouted 5 feet into the pile and extend 2'9” into the deck with 90 degree hooks bent outwards. The prestressed concrete pile is embedded 2 inches into the deck. The piles are spaced 10 feet in the longitudinal direction and 16’8” in the transverse direction. The expected dominating behavior is a plastic hinge forming at the top of pile and opening and closing of the pile-to-deck interface.
Figure 3-2: Pile Geometry and Reinforcement for Specimen A.1

Figure 3-3: Plan View of Reinforcement for Specimen A.1
3.1.3 External and Internal Instrumentation

The two main types of data collected were lateral displacement of the deck and curvature of the pile immediately below the soffit. Several linear potentiometers were placed to ensure that fixed boundary conditions remained fixed. The vertical potentiometers on the actual pile measured curvature while the rest were present to monitor slippage and boundary conditions. There were four potentiometers connected to the four corners of the deck to monitor that the deck itself stayed level. The vertical actuator was programmed to react to changes in displacement to those four potentiometers (Figure 3-4). Strain gauges were also placed on two of the extreme dowels to compare the actual strain in the bars to the strains defined by POLA’s Seismic Code for OLE and CLE events (Figure 3-5).

Figure 3-4: Potentiometer Locations
The strain gauges were placed along two extreme dowels in the pile. The dowel bars were ground smooth in the areas where the gauges were to be applied and covered with m-coat and mastic tape (Figure 3-6).
3.1.4 Loading Protocol and Test Control

The procedure for testing both specimens followed both Krier (2006) and Sritharan and Priestley (1998) in which the initial test loading was force-controlled until yield. Once yielding was reached, the test is switched to displacement-control (Figure 3-7).

The lateral load was applied with a single horizontal actuator programmed to follow the load protocol. Two vertical actuators were slaved using software written by Dr. Chris Latham. The software used input from 4 string potentiometers at the four corners of the deck. The slaved actuators responded to differences in the four corners and kept the deck level throughout the test.

Figure 3-7: Loading Protocol
3.2 Specimen A.2 – Steel pile

3.2.1 Test Set Up

Several berths with flat slab were constructed between 1981 and 1989, including Berths 226-229. Of these similar berths, some have been retrofitted to allow for the support of crane rails to meet increasing wharf activity. A typical steel pile tested to verify that after 2” of lateral displacement the vertical load capacity is not lost. The test set up is similar to the one used for the specimen A.1. In this case however, an additional vertical load, representing the crane load, was applied to the top of the connection by using a pinned steel beam and thread rods prestressed with hydraulic jacks as shown in Figure 3-8. The short distance from the soffit to the point-of-inflection required that the specimen be placed on a concrete block so that the vertical actuator could fit beneath the tie beam.

The specimen was supported by a pivoting steel collar that represented the in-ground point of inflection generated from pile bending below grade. The dominating behavior expected was yielding of the dowel bars, degradation of the welds and subsequent opening and closing of the pile-to-deck interface.

As previously mentioned for the prestressed concrete column connection, the specimen for this test is also a part of a long berth and therefore would be restrained from moving out of plane. The large moment frame was also used in this test set up to prevent out of plane sway of the specimen to correctly simulate the boundary conditions in the field. The specimen is allowed to move vertically slightly in order to accommodate the decrease of the system height as it displaced horizontally. In this
way no artificial constrains, that are not present in the real structure, are introduced in the system. The deck however remains always horizontal due to the slaved vertical actuator.
Figure 3-8: Test Set-up and Free Body Diagram for Specimen A.2
3.2.2 Connection Detailing

The reinforcing details differ slightly along the length of any berth so a representative reinforcing detail was chosen to most accurately simulate the behavior of the system. The typical steel piles used by the Port of Los Angeles at the time construction were HP14x117, a standard rolled shape. This HP section is connected to a deep girder section that run longitudinally along the length of the berth by (8) #11 bars. These bars are welded 8” along the flanges of the HP14. The dowel bars are straight with no hooks and extend from the soffit into the deck 44”. The deep section of deck is connected to the existing superstructure by means of 3’x 3’ tie beam that is heavily reinforced. These conditions were reproduced in the specimen as shown in the plans of construction in Figure 3-9.
A primary concern was the weld strength that provides the connection between the HP14x117 flanges and the (4) #11 bars on each flange. It was decided to reduce the welds from their original 5/16 size to 3/16 to conservatively simulate the difference between shop and field weld quality.

The tie beams that connect the new piles and longitudinal pile cap beam to the existing superstructure were heavily reinforced, having (10) #11 bars both top and bottom. Additionally, (4) #8 bars run along the tie beam (Figure 3-10). Shear stirrups of #5 bars were present in both the pile cap and tie beam. (8) #9 bars reinforced the longitudinal direction of the berth at both the top and bottom (Figure 3-11). Four
additional bars were present at the bottom, and (2) #11 bars were vertically spaced equidistant between the top and bottom row of (8) #9 bars.

The seismic behavior of the pile-to-deck connection was controlled by the (8) #11 bars that were welded to the flanges of the pile and extended 44” into the deck. The dowel bars were straight with no hooks, heads, or plates on the end. The steel pile was embedded 8” into the deck and the dowels were welded along the entire embedded 8 inches. The piles are spaced 6’in the longitudinal direction in the actual Berth; however, the pile cap beam was extended beyond its tributary width of 6’ to allow for longitudinal reinforcing to receive more development length.

Figure 3-10: Reinforcement for Specimen A.2
3.2.3 External and Internal Instrumentation

Three main types of data were collected including lateral displacement, curvature of the pile, and strain in the pile dowels (Figure 3-12). Several linear potentiometers (pots) were placed to monitor the support movement in order to ensure the fixed boundary conditions remained fixed and that in the event of slip, the data could be corrected. Linear pots were also placed to monitor bending in the 3’x3’ tie beam. Strain gauges were placed along the length of two #11 dowels; a center dowel on each flange was chosen. Vertical string potentiometers were attached to the head of the gauged bars to monitor slip and pull out of the bars.

Figure 3-11: Plan View of Reinforcement for Specimen A.1
Figure 3-12: Primary Internal and External Instrumentation

Notice that the first strain gauge is located 10” above the soffit of the specimen, which is 2” above the steel pile top that is embedded 8” into the deck. The first set of strain gauges were placed 10” above the steel pile to prevent stress concentrations from distorting the readings. However, it is important to remember that the peak strain in the dowels will occur at the steel pile tip (top interface into the cap between the pile and the concrete).

3.2.4 Loading Protocol and Test Control

Each test target was based on absolute displacement at the request of Port of Los Angeles. The load protocol (Figure 4-19) is designed to determine whether or not the steel pile-to-deck connection can still support the necessary load after deforming with the rest of the berth in a seismic event.
The lateral load was applied with a single horizontal actuator. A single vertical actuator was slaved to the 4 string potentiometers at the four corners of the deck. The slaved actuators responded to differences in the four corners and kept the deck level throughout the test. Load, displacement, and strain were monitored real-time to ensure safe behavior of the system. Due to the sudden and drastic loss in moment capacity the test had to be stopped several times in order to take safety measures. The last displacement targets were met with straps wrapped around the specimen being attached to an overhead crane (Figure 3-14).
The vertical load was applied by using steel beams supported on the deck’s top and pulled down with thread rod running from beam to the floor and prestressed with 4 separate hydraulic jacks (Figure 3-15). These jacks have internal load cells that were used to monitor the load.

Figure 3-14: Specimen with Safety Straps In Case of Sudden Failure

Figure 3-15: Typical Hydraulic Jack Used to Apply Vertical Load
4 Construction

4.1 Specimen A.1 – Prestressed octagonal concrete pile

4.1.1 Procedure

The pile used for this test (Figure 4-1) came from a group of piles manufactured at a precast yard per Port of Los Angeles specifications. The piles were built specifically for this test and previous tests conducted by Krier (Krier et al., 2007). This pile was prestressed and precast. The lower portion of this pile was roughed to ensure mechanical friction between the pile and the grout used to hold the pile in place within a metal collar.

A pivoting metal collar was used to simulate a pin connection at the base of the pile (Figure 4-2). Two weld beads were placed around the inside of the collar to provide mechanical friction for the grout. The column was temporarily secured inside
the collar and formwork was placed to allow pouring of the high strength grout. After the pile was secured in the collar, the pile dowels were grouted into the pile using a 0.4 w/c grout. Super plasticizer was added for workability. Pile bars were held in place until the grout cured.

Falsework was then built up around the pile bars and pile. Rebar was laid in traditional top and bottom mats with stirrups and transverse reinforcing (Figure 4-3). Also PVC tubes for the actuator connections, joint deformation monitoring, and anchors for pick-up points and fall protection, were placed before pouring the concrete (Figure 4-4).
Figure 4-3: Specimen Slab Prior to Pouring Concrete

Figure 4-4: PVC and Pick-Up Points in Specimen
4.1.2 Material Properties

The deck concrete and dowel grout are parameters that directly influence the test results. Hence, the material characteristics were expected to replicate what would
have been used in the field during the construction of Berth 145 at the Port of Los Angeles as shown in Table 4-1.

Table 4-1: Concrete Compressive Strengths

<table>
<thead>
<tr>
<th>Material</th>
<th>Age</th>
<th>Compressive Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck</td>
<td>21 days Day of Test</td>
<td>5.08 ksi</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5.24 ksi @ 23 days</td>
</tr>
<tr>
<td>Pile Grout</td>
<td>Day of Test</td>
<td>3.67 ksi @ 41 days</td>
</tr>
<tr>
<td>Precast Pile</td>
<td>28 days</td>
<td>7.5 ksi</td>
</tr>
</tbody>
</table>

It is important to note that the precast pile was cast in mid May of 2005 and that the test was carried out more than seven months later. Significant compression strength increase would have occurred over those seven months but not extra material from the casting yard was available to obtain exact values. In the theoretical analysis it was assumed that the precast pile had a compressive strength of 8 ksi.

The reinforcing pattern for the concrete deck required four different sized bars. The strengths of samples of these bars are listed in Table 4-2. Reinforcing bars, stirrups, and dowels were A615 grade 60 steel.
Table 4-2: Steel Strengths

<table>
<thead>
<tr>
<th>Bar size</th>
<th>$f_y$ (ksi)</th>
<th>$f_{su}$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td># 4</td>
<td>78.23</td>
<td>120.57</td>
</tr>
<tr>
<td># 8</td>
<td>65.53</td>
<td>108.90</td>
</tr>
<tr>
<td># 9</td>
<td>66.18</td>
<td>108.55</td>
</tr>
<tr>
<td># 10</td>
<td>71.69</td>
<td>97.32</td>
</tr>
</tbody>
</table>

4.2 Specimen A.2 – Steel pile

4.2.1 Procedure

The pile used for this test was obtained from a private company. A used piece (but unyielded) of HP14x117 was purchased in 8’ length to allow coupons to be cut from the pile before cutting to the desired size (Figure 4-7). The pile was inspected to ensure no yielding had previously occurred. The pile had been used as shoring in the past, but was still in good condition. Due to the nature of construction practices and variation in quality, this component is probably a very good representation of what was used in the field during the time of construction of Berth 226. Standard ASTM coupons were cut from the lower 2’ of the steel pile to be tested for yield and ultimate stress (Figure 4-8).
After coupons were cut, a plate and roller were welded to the base of the steel pile to provide a second pin connection, in addition to the pivoting steel collar. Bearing plates were also welded into the open sides of the HP section where it would
be grouted in the collar. Flat bars were also welded to the flanges and added bearing plates to create more area for mechanical interlock (Figure 4-9 and Figure 4-10).

The (8) #11 bars were welded with 3/16” flare groove bevel welds per AWS D1.4 along the 8 inches that overlapped the steel pile (Figure 4-11). The certified welder was asked to replicate field conditions instead of aiming for the highest quality.

Figure 4-9: Components Added to 6’ Steel Pile
Figure 4-10: Steel Pile with Bearing Plate, Flat Bars, and Roller Welded

Figure 4-11: Steel HP14x117 Pile with welded #11 dowels and welded flat bars for mechanical friction

Two rows of bent flat bars were placed around the inside of the collar to provide mechanical friction for the concrete. A hydraulic jack was placed directly
beneath the steel pile to provide a bearing surface for the roller welded to the bottom of the pile (Figure 4-12).

![Figure 4-12: Interior View of Pivoting Steel Collar](image)

The soffit level falsework was built up and the steel pile was lowered into the collar and held in place until the pour. The remaining falsework was assembled after the steel pile with welded dowels had been set (Figure 4-13). The rebar cages for each piece were built to 90% completion separately and then lifted and merged on top of the placed specimen (Figure 4-14 to Figure 4-16). PVC tubes for the actuator connections, slip monitoring, and anchors for pick-up points of the specimen were placed before casting the concrete (Figure 4-17). The same concrete used for the deck was used to secure the steel pile within the collar. The strength of the concrete was
not a primary concern because the steel roller supported by the hydraulic jack was in place to provide additional bearing capacity if required.

Figure 4-13: Shoring, Collar, and Side-Sway-Inhibiting Frame

Figure 4-14: Tie Beam Reinforcing Cage 90% Assembled
Figure 4-15: Load Stub Reinforcing Cage, Foreground, 90% Assembled

Figure 4-16: Reinforcing Cages Merged, 98% Assembled
After falsework was removed, the specimen was painted to allow for increased visibility of cracks during testing. The specimen was shored and actuators were placed. Reinforced steel beams were placed on top of the pile cap with thread roads connected to hydraulic jacks to provide the 300 kips of axial load (Figure 4-18).
The linear pots were attached to ¼” thread-rod that was tack welded directly to the flange of the steel pile. A target, whose anchor was embedded in confined core concrete, helped prevent inaccurate measurements when spalling began to occur (Figure 4-19).
4.2.2 Material Properties

The deck concrete and collar concrete strength is presented in Table 4-3. The goal was to replicate what would have been used in the field during the construction of Berth 226 at the Port of Los Angeles.

**Table 4-3: Concrete Compressive Strengths**

<table>
<thead>
<tr>
<th>Material</th>
<th>Age</th>
<th>Compressive Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck</td>
<td>Day of Test 1, truck 1</td>
<td>8.32 ksi @ 40 days</td>
</tr>
<tr>
<td>Deck</td>
<td>Day of Test 1, truck 2</td>
<td>8.69 ksi @ 40 days</td>
</tr>
</tbody>
</table>

The concrete used came from two trucks the first batch had a slump of 4.5” and the second batch had a slump of 4”. The concrete strength was higher than
expected. The minimum concrete strength specified for the deck of Berth 226 was 5000 psi. Note that stronger concrete will tend to slightly reduce the strain penetration of the reinforcing bars welded to the HP pile. Therefore for a given rotation demand a larger strain could be expected if the concrete strength increased.

The strengths for the deck reinforcing tested samples are listed in Table 4-2. Reinforcing bars, stirrups, and dowels were A615 grade 60 steel. The strengths of the coupons removed from steel pile are listed in Table 4-3.

Table 4-4: Steel Reinforcing Strengths

<table>
<thead>
<tr>
<th>Bar size</th>
<th>(f_y) (ksi)</th>
<th>(f_{su}) (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td># 8</td>
<td>59.50</td>
<td>95.07</td>
</tr>
<tr>
<td># 9</td>
<td>66.28</td>
<td>100.05</td>
</tr>
<tr>
<td># 11</td>
<td>64.26</td>
<td>98.81</td>
</tr>
</tbody>
</table>

Table 4-5: Steel Pile Strength

<table>
<thead>
<tr>
<th>Location</th>
<th>(f_y) (ksi)</th>
<th>(f_{su}) (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Web</td>
<td>52.23</td>
<td>75.95</td>
</tr>
<tr>
<td>Flange</td>
<td>53.29</td>
<td>75.64</td>
</tr>
</tbody>
</table>
5 Prediction of Monotonic Response

This chapter describes theoretical predictions made prior to testing. Assumptions, simplifications, and methods used are described.

5.1 Specimen A.1 – Prestressed octagonal concrete pile

5.1.1 Mechanisms of Inelastic Deformation

Deformation in the system is forced into the pile and, subsequently, the pile-to-deck connection (Figure 5-1). This is to ensure a ductile response. The plastic hinge region at the top of the column and opening of the connection will govern the system’s lateral displacement. The inelastic deformation that takes place in the pile-to-deck interface was modeled. The moment curvature relationship was estimated using the software Columna (Kuebitz, 2002). The rotation, force and displacements were obtained from these results, the assumption of the plastic hinge length and the test set up geometry.
5.1.2 Main Components of Lateral Displacements

The two main contributions to the system’s lateral displacement are the connection and pile flexibility. The flexibility of the connection, $\delta_c$, was expected to be the most significant component of lateral displacement, while the flexibility of the pile, $\delta_p$, contribute little in comparison (Figure 5-2). The flexibility of the deck is negligible due to additional reinforcing being added to simulate physical boundary conditions of the retrofit.
To compute total displacement we use
\[ \Delta = \delta_c + \delta_p \quad (5-1) \]

Where
\[ \delta_c = \theta h \quad (5-2) \]
and
\[ \delta_p = \frac{Vh^3}{EI} \quad (5-3) \]

\[ \theta = \varphi l_p \quad (5-4) \]

Priestley et al (1996) \[ l_p = 0.3d_b f_y \quad (5-5) \]

5.1.3 Moment Curvature Analysis of Critical Sections

The moment curvature analyses was carried out considering accurate dimensioning and placement of dowel bars and confining steel. Because prestressing bars are present, the dowel bars are not lined up against the confining steel. Figure 5-3 shows a photograph of the pile used and the model used in the Moment Curvature software. The dowel bars are grouted into ducts shown.

![Figure 5-3: Photograph of Pile and Corresponding Model](image)
The section analysis directly returns a moment-curvature relation for a defined axial load. However, in the real structure as well as in the test set up, such load varies as the lateral displacement changes. This requires an iterative section analysis until agreement between moment, curvature, and induced axial load is reached (Figure 5-4). After converging on lateral and axial forces the curvature in equations (5-4) and (5-2) is used to predict the displacement due to the connection opening. A detailed explanation of this procedure is given by Krier et al (2007) Similarly, the lateral force obtained from the moment analysis and the specimen geometry is used to predict the displacement due to pile bending. Finally, the force-displacement relationship is obtained (Figure 5-5).
5.1.4 Effect of Axial Load and Reverse Cycling

During testing the axial load induced by the test setup fluctuated cyclically between tension and compression. This increase or decrease in induced axial load was expected to influence the lateral force capacity of the system. It was also expected to have some unsymmetrical influence from the P-D effect caused by the axial load fluctuation.

5.1.5 Predicted Moment – Rotation Responses

The moment rotation estimation was obtained using the plastic hinge length calculated from equation (5-5) (Figure 5-6). This estimation allows comparing the experimental and analytical results for verifying the value of the plastic hinge used.
Figure 5-6: Moment - Rotation Prediction
5.2 Specimen A.2 – Steel Pile

This chapter describes possible failure modes and behavioral concerns.

5.2.1 Weld Strength

The weld strength was suspected as a possible failure mode due to the lack of a ductility criterion for the weld design, the difficulty of verifying quality, and difficulty of the welding conditions.

Weld fracture was a concern in part due to the rebar material being A615. A615 steel is not considered weldable by today’s standards. While technically it can be welded it requires preheating and maintaining of temperature during welding which is difficult to control. Even when performed by an experienced licensed welder the consistency is poor.

The ASTM specification for A615 steel does not limit the carbon content like A706. A706 steel is limited to 0.30% carbon. A615, not having a limit is often around 0.45% and sometimes as high as 0.55% making it a medium to high carbon steel. Medium to high carbon steel has poor resistance to impact, does not bend well, and does not weld consistently.

Despite the common practice of specifying A706 rebar in welding applications today, A615 was used in specimen A.2 to match the existing conditions of the wharf.
5.2.2 Dowel Anchorage

The lack of mechanical hook, bulb or plate at the termination of the dowel bars caused concern that proper anchorage would not be achieved since the dowels only extended 44” into the deck. For this reason, the gauged bars were attached to vertical string potentiometers that monitored slip during testing.
5.2.3 Dowel Fracture

A high stress concentration is created in the dowel bars at the termination of the steel pile where there is an abrupt drop in effective area. It is very likely that necking will occur in this region and eventually cause fracture of the bars after repeated elongation and shortening of the dowel bars.

5.2.4 Moment Curvature Analysis

The moment curvature estimation was carried out assuming a transversal section as shown in Figure 5-7. The geometrical properties of the bars and the H section are described in section 3.2. Figure 5-8 shows the results for the moment curvature analysis of the section.

The moment rotation (Figure 5-9) and force displacements (Figure 5-10) envelopes were estimated following the equations 5-1 to 5-4 given in the previous section. Due to the characteristics of the connection, the plastic hinge length was obtained as \( l_p = 0.15d_b f_y \). This value is half of that given in equation 5-5 given that the strain penetration will only occur on the dowels extending out from the welding end into the joint cap.
Figure 5-7: Transversal section for Moment Curvature Analysis
Figure 5-8: Moment - Curvature Prediction

Figure 5-9: Moment - Rotation Prediction
Figure 5-10: Moment - Displacement Prediction
5.3 Acknowledgement

Contributions to Section 5.2.4 Moment Curvature Analysis were made by Carlos A. Blandon including some text and Figures 5-6 through 5-10.
6 Test Results

6.1 Specimen A.1

6.1.1 Overall Behavior

A precast, prestressed concrete pile connected to a 3 ft 6 in. deep reinforced concrete deck was tested. The connection consisted of 8 #9 dowel bars anchored 35 in. into the deck and grouted 5 ft. in the pile. The #9 dowel bars were bent with 90 hooks in the deck and the bars were positioned with the hooks outwards. This test was performed to evaluate the capacity to resist lateral loads and to characterize overall hysteretic behavior of the connection. Test specimen A1 was built as an exterior pile-to-deck connection. Figure 6-1 shows an overall view of the specimen prior testing.

Overall, the Test specimen A1 responded very well. The test was completed up to a displacement ductility of 18 when a number of dowel bars had fractured. The yielding displacement was defined as 0.34 in. based on the strains measured in the instrumented dowels at the connection.

The OLE structural performance limit state occurred when a pile dowel attained 1% strain. This limit-state occurred when the test specimen had been loaded laterally .51 in. at a displacement ductility of 1.5. At this stage there were hairline cracks and slight delamination of the south side of the deck (Figure 6-2 and Figure 6-3.) No crushing occurred on the north side of the soffit. Cracking was observed on
the deck. Cracking at the pile-to-deck interface probably occurred but was hidden by the formwork sealant left from construction.

At a displacement ductility of 2, diagonal shear cracks were noticed running through the deck above the pile (Figure 6-4). This was monitored and did not significantly affect the behavior of the specimen. The shear cracks that developed were minor and were not continuous through the deck. A possible joint failure was anticipated because of the anchorage of the dowel bars, but the test conclusively showed otherwise. Delamination continued, but still no crushing on the north side was observed.

The CLE structural performance limit-state was governed by the strain limit on the pile dowels of 5%. This limit-state was attained at displacement ductility of 6. Crushing, delamination, and spalling of the cover concrete around the connection were seen (Figure 6-5 and Figure 6-6)

The general dominating behavior of the specimen was opening and closing of the pile-deck interface. This behavior was similar to that observed in a previous test on a seismically design connection (Krier et al, 2007). While a number of cracks were present throughout the specimen, they were small. The cyclic loading caused crushing and spalling at the compression and tension sides of the pile. This behavior continued until almost 3 cycles of nominal displacement ductility of 18 were met. The test was stopped when half of the pile’s longitudinal reinforcing steel had fractured (Figure 6-7 and Figure 6-8).
Figure 6-1: Specimen before loading

Figure 6-2: Delmatination of deck at OLE south of pile
Figure 6-3: Delamination Continues

Figure 6-4: Flexural cracks at $\mu = 2$
Figure 6-5: Crushing on north side of pile at CLE

Figure 6-6: Delamination at CLE
6.1.2 Analysis of the Hysteretic Response

The lateral force-displacement hysteretic response of this test specimen is shown in Figure 6-9 and the respective moment rotation at the connection in Figure
Positive loading was in the push direction and negative in the pull direction. In the push direction the pile was subjected to axial compression, which increased its capacity but caused the connection to degrade more than for the tensile cycles as the displacement was applied. The opposite occurred in the pull direction; notice that the force is smaller but it drops very little even at large displacements. The predicted envelope shows a sharp reduction of the maximum capacity and slightly under predicts the experimental moment. The larger experimental capacity can be explained by the cyclic hardening of the steel that is not included in the analytical estimation.

Figure 6-9: Lateral Force-Displacement Prediction and Measured Results
6.1.3 Strain Penetration of Dowel Bars

Figure 6-11 and Figure 6-12 plot the strain distribution in the dowel bars for different cycles. Peak strains were observed at the pile-deck interface as expected. These two figures clearly indicate that there was significant yield strain penetration and that the extent of this penetration towards the pile or towards the deck was either similar or greater in the deck.
6.2 Specimen A.2

6.2.1 Overall Behavior

The displacement targets set in the load protocol were met throughout the test, but due to slip of the collar in the pull direction, a relative displacement of 2” was not
met in the pull direction until the cyclic loading had drastically reduced the moment capacity of the connection.

The specimen connection shown in (Figure 6-13) performed very well up through absolute displacements of 2” in both directions. Minor cracks were seen in the soffit at initial displacements (Figure 6-14 to Figure 6-16) and propagated, but remained narrow up through an absolute displacement of 2” in both the push and pull directions. Along the contact surface, where the pile flanges beared on the concrete, the additional cracking was seen.

Figure 6-13: Specimen Prior to Testing
Figure 6-14: Minor Cracks Observed, 3/4" Displacement Pictured

Figure 6-15: Onset Spalling of Cover Concrete Under Pile Bearing
At displacements of 2”, the specimen was held in place and a 300 kip load was applied for 2 minutes using 4 hydraulic jacks (Figure 6-17). While supporting the vertical load necessary to operate the crane, it was noticed that the base of the steel collar was deforming from high overturning forces (Figure 6-18).

The presence of the roller and jack beneath the pile removed the resisting moment due to the self weight of the specimen. The result was that the collar was subjected to high tension forces on one side, resulting in the slip of the collar in the pull direction.

Since the collar base plate uplift only occurred in the pull direction, the specimen was still pushed to a 2” displacement and the 300 kip vertical load was applied (Figure 6-19). The specimen was able to carry the load, and no additional damage was noticed. After the collar hold-downs were reinforced (Figure 6-20) testing was resumed.
Figure 6-17: Specimen Displaced 2" in Pull Direction with 300 kips Vertical load

Figure 6-18: Collar Uplift

Figure 6-19: Specimen Displaced 2" in Push Direction with 300 kips Vertical load

Figure 6-20: Square Tubes Are Used to Prevent Uplift of the Collar Base Plate
After the push cycles to 2” displacement, there was significant residual displacement of the specimen. During the process of attempting to pull the specimen to a displacement of 3” several dowels were heard to fail and the moment capacity of the pile dropped dramatically with each dowel failing. The first drop in lateral force capacity was accompanied by extreme spalling of the soffit of the deck (Figure 6-21). After the specimen was pulled to 3” of displacement extensive spalling had occurred (Figure 6-22).

Figure 6-21: Widespread Splitting of Cover Concrete
The specimen was unloaded with almost no force and the residual displacement was approximately zero. The vertical load of 300 kips was applied and maintained for 2 minutes.

Due to the sudden and violent failure that occurred while pulling the specimen to a displacement of 3” it was decided to place safety straps at the longitudinal pile cap beam before pushing the specimen to 3” of displacement.

Unlike, in the pull direction in which the dowels fractured individually, in the push direction, all four dowels fractured at once resulting in an almost instantaneous loss of lateral capacity and extensive spalling (Figure 6-23). The unit was unloaded and held at its resting position and the 300 kip vertical load was applied and successfully maintained for 2 minutes.
6.2.2 Analysis of the Hysteretic Response

The hysteretic response of this test unit consisted of stable well-defined loops similar to a well-detailed reinforced concrete unit. Most of the hysteresis was due to yielding of the dowel bars inside the deck and localized crushing of concrete.

The horizontal reaction at the support pin is transferred through the pile to the joint cap along the contact surface between the pile and the cap. Note that the experimental moment at the connection and the analytical lateral force have to be computed indirectly by assuming an effective length \((L_e)\) (Figure 6-24) between the pin at the base and the point of application of the lateral load at the cap. If the clear distance between the pin and the cap bottom (54 in.) is assumed as the effective
length, the analytical lateral force will be larger than if the total length of the pile (from the pin to the pile top inside the deck-62 in.) is assumed. In the other hand, the experimental moment capacity will be smaller for the first case (pin to cap bottom) than for the second case (pin to pile tip).

The experimental response and the numerical predictions for moment -rotation and lateral force-displacement for both extreme effective pile lengths are shown in Figure 6-25 and Figure 6-26. It is clear that the analytical and experimental results have a better match for the shortest effective length (pin to cap bottom). This indicates that most of the lateral force (V) is being transmitted between the pile and the deck at a location very close to the cap bottom. The experimental results still show a larger strength that predicted, but this can be explained by the cyclic hardening of the steel dowels at the connection.
This test unit showed a lateral displacement capacity in excess of the target 2”. In fact, fracture of the dowel bars was observed after an excursion to 2.6” corresponding to 4.2% drift ratio (Figure 6-26). From this standpoint, the component met the displacement objective stated for the retrofit of Berth 226. The drift ratio reported in Figure 6-26 was estimated as the displacement at the deck level divided by the total pile length, this is, the length from the support pin to the rotation center.
located inside the joint, 8 in. from the cap soffit. The center of rotation corresponds to the location of the H steel pile tip.

Figure 6-25: Moment - Rotation for Specimen A.2
6.2.3 Strain Penetration of Dowel Bars

Due to the bearing of the steel pile on the core concrete strain penetration of the dowel bars was able to occur. The closest gauge is 10” from the soffit and 2” above the end of the pile. Some strain penetration can be seen as far up as 22” from the soffit, but the dowel strain peaks at the pile top (Figure 6-27 and Figure 6-28).
Figure 6-27: Peak Tensile Strains at Displacement Targets in the Push Direction (Pile in compression)

Figure 6-28: Peak Tensile Strains at Displacement Targets in the Pull Direction (Pile in tension)
The yield strain penetrated 21 in. above the deck’s soffit which corresponds to 15 $d_b$. No record of end bar slip was recorded in the test.

6.3 Post Testing Inspection

The concrete in the lower 8” of the longitudinal pile cap beam that provided the bearing surface for the steel pile was crushed exposing the bottom most layer of longitudinal bars and shirt stirrups as shown in Figure 6-29. Inspection of the specimen after testing and removal of lose spalled soffit concrete revealed that all (8) #11 bars fractured at the top of the steel pile (Figure 6-30 and Figure 6-31). It appears that no welds were fractured and that the pile itself has remained in the elastic range. The lateral deformation that occurred resulted from cyclic elongation and shortening of the dowel bars and crushing of the cover concrete.

Figure 6-29: Exposed Bottom Reinforcing After Spalling and Crushing
Figure 6-30: Underside of Specimen with Visible Fractured Dowels

Figure 6-31: Upper Segment of Steel Pile with Visible Fractured Dowels
6.4 Acknowledgement

Contributions to Section 6.2.2 Analysis of the Hysteretic Response were made by Carlos A. Blandon including some text and Figures 6-24 through 6-26.
7 Summary and Conclusions

Two existing pile-to-deck connection were tested for the Port of Los Angeles. In the first test, a precast, prestressed, 24” octagonal pile was connected using 8 #9 dowel bars to the deck of berth 145. A new line of seismically designed columns were placed to allow for the addition of crane rail at this berth. With this addition to the structure the columns in question became a redundant structure but still important in the behavior of the wharf as a whole. The pile-to-deck connection attained a displacement ductility greater than eight in both directions. This corresponds to about a 5% drift ratio of the structure. It is hoped that this performance will facilitate the decision to refrain from cutting the existing piles and using further connective elements than are already in place in berth 145 at the Port of Los Angeles.

In the second test, a steel pile that is a typical retrofits element to berths built between 1981 and 1989 at the Port of Los Angeles was tested to confirm vertical load capacity of the pile-deck system after specified displacement targets.

The steel pile-to-deck connection was able to maintain the full 300 kips vertical load capacity at an OLE level displacement of 2 inches. The damage to the system was minor and acceptable at this limit state. The pile was still able to maintain vertical load capacity even after all dowels had fractured at a horizontal displacement above 2”. Damage at this level is extensive, and because the soffit is below grade the damage is difficult to identify and repair.
8 Lessons Learned

Having a clearly defined project goal and revisiting that goal on a regular basis will help to focus energy on important areas. The construction aspect of the project can be the most detailed, time-consuming, and frustrating. Much time is spent discussing and understanding the theory and setup of the project while the translation of the theory into reality can be less developed.

In authoring a research paper, being able to progress effectively from theory to testing to conclusion is paramount. On paper, the construction process is overshadowed by the test results. There are exceptions. Errors or changes in the test resulting from construction related problems or discoveries are often noted and can play a substantial role in the paper. However, it is more common that problems arise and after time, energy, and stress are poured in them they are moot.

During the research process, construction can easily become the most time and energy consuming aspect of the project. On a day-to-day basis it requires constant attention and coordination. For example, you may have thoroughly planned the location of strain gauges considering system behavior, possible localized effects, reliability of readings, and many other factors. During construction you are confronted with what to do with the wires. Unlike the several days or weeks you had to spend hashing out and considering every factor you could think of regarding strain gauge location, you now have 24 hours to decide which path the wires should follow before concrete is poured. As the specimen cracks any wires along a crack plane will be potentially stretched and their readings affected. Running wires along rebar could
compromise their bonding with the concrete. It is these types of construction issues that arise and can be daunting during a research project. When you are processing the strain gauge readings for analysis, you are no longer concerned with details of their wire locations, assuming you developed a solution that did not compromise your results. And so something that, at one point in time, was important is no longer.

Constantly recalibrating focus on the end goal of the project instead of allowing focus to shift to the roadblocks and bottlenecks along the way can significantly aid the research process. This can be further facilitated by spending the time to intricately develop and detail the construction process. Unfortunately, seeing the big picture and anticipating the problems along the way requires construction experience. Asking many questions and coordinating with lab staff becomes the most effective tool for a student trying to developing a workable construction plan.
References


Weismair, M. Private communication, 2005