2005-03-02

Flexural Behavior of Carbon/Epoxy IsoTruss Reinforced-Concrete Beam-Columns

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FLEXURAL BEHAVIOR OF CARBON/EPOXY ISOTRUSS®-REINFORCED
CONCRETE BEAM-COLUMNS

by

Monica Joy Ferrell

A thesis submitted to the faculty of
Brigham Young University
in partial fulfillment of the requirements for the degree of

Master of Science

Department of Civil and Environmental Engineering
Brigham Young University
April 2005
of a thesis submitted by

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This thesis has been read by each member of the following graduate committee and by majority vote has been found to be satisfactory.

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ABSTRACT

FLEXURAL BEHAVIOR OF CARBON/EPOXY ISOTRUS®-REINFORCED CONCRETE BEAM-COLUMNS

Monica Joy Ferrell

Department of Civil and Environmental Engineering

Master of Science

This thesis quantifies the flexural behavior (strength, stiffness and failure) of IsoTruss®-reinforced concrete beam-columns for use in deep foundation pile applications. Four-point bending tests were performed in the laboratory on two instrumented carbon/epoxy IsoTruss® reinforced concrete piles (IRC piles) and two instrumented steel-reinforced concrete piles (SRC piles). The piles were approximately 14 ft (4.3 m) in length and 14 in (36 cm) in diameter and were loaded to failure while monitoring load, deflection, and strain data. The steel and IsoTruss® reinforcement cages were designed to have equal flexural stiffness to permit a relative strength comparison. Moment-curvature diagrams reveal that the stiffness values were indeed close, verifying the design objective. At failure, the IsoTruss®-reinforced concrete beams held nearly twice the
bending moment as the steel-reinforced concrete beams [1,719 kip-in vs. 895 kip-in (194 kN-m vs. 101 kN-m)], although the failure modes were quite different. The SRC piles exhibited the traditional ductile failure behavior, as expected, while the IRC piles lacked ductility. The IRC pile deflections remained linear to failure, while the SRC piles yielded significantly. At 35 kips (165 kN), the maximum load on the SRC piles, the ductility of the SRC piles was twice that of the IRC piles (0.0084 and 0.0042, respectively).

Toughness measurements reveal that due to the lack of ductility in the IRC piles, the SRC piles absorbed approximately twice as much energy as the IRC piles. Further investigations are required to explain the absence of ductility in the IRC piles, since ductility has been observed in other IsoTruss®-reinforced concrete structures in flexure. Even with this low level of ductility, the IRC piles are substantially stronger than the SRC piles and provide an alternative for use in deep foundation environments. Not only is the IRC pile strong enough for the job, but the IsoTruss® reinforcement is approximately 62% lighter, more rigid, and more corrosion resistant than the steel reinforcement.
ACKNOWLEDGMENTS

It would be incredibly selfish to presume that this entire thesis was due to the outstanding efforts of me alone. First and foremost, I would like to thank my parents for their constant support and encouragement over the years. I love you both and could not have done it without you. Secondly, I would like to thank Dr. Jensen and Dr. Rollins for not only believing in me and giving me this opportunity, but for guiding me along the way. Also, I am very grateful to UDOT and FWHA for partially funding this research project.

To Dave Anderson, Luke Heiner, and Chris Parshall, I say thank you for all of your help in the laboratory. It made testing go so smoothly and I appreciate all of your expertise and advice. Thanks also to the CASC employees for helping and supporting me through this process. Finally, I want to thank the girls of the BYU swim and dive team for remaining my friends through it all.
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1 Introduction

In a world where change is inevitable, the newest technology will continue to rise and take its place over things of the past. Many across the world are constantly performing the art of research and development with one goal in mind: to come up with the next big idea. From submarines to space shuttles and bicycles to bathing suits, one can be sure that someone has put a lot of time and effort into showing the world that their idea is superior to the rest.

The research presented in this thesis explores the possibility of using an IsoTruss® structure as a possible method of reinforcement for concrete deep foundation piles. The flexural behavior of these IsoTruss® reinforced concrete piles (IRC piles) is compared to similar piles reinforced with steel (SRC piles). The ultimate goal is to demonstrate that the IRC pile is comparable in many ways to the standards currently in use and is capable of becoming the next big idea in deep foundation piles.

1.1 Description of Research

The research contained within this thesis deals with the testing of reinforced concrete deep foundation piles in a laboratory environment. Four lab tests were performed in a steel reaction frame where the pile specimens were positioned horizontally and subjected
to four-point bending within a 12 ft (3.7 m) region. Two of the samples were reinforced with steel rebar and the remaining samples were reinforced with a specially-designed carbon composite IsoTruss® lattice structure.

This thesis compares the flexural strength and stiffness of SRC piles to IRC piles. The objective is to evaluate the feasibility of the IRC pile as an alternative in the future of deep foundation piles. With the comparable strength of the IsoTruss® to steel as a method of concrete reinforcement, its many other unique and desirable qualities, such as corrosion resistance, light weight and superb rigidity, make the IsoTruss® a better choice for concrete pile reinforcement applications.

1.2 OVERVIEW OF DEEP FOUNDATION PILES

Deep foundation piles are used throughout the world to support structures founded on soft soils. They are structural members, typically of steel or reinforced concrete, which are driven into the ground to act as part of the foundation support system. Piles are most often used in high rise buildings and bridge structures where the loads would cause excessive settlement if supported by a mat foundation or spread footings. There are also many soil conditions that require the use of deep foundation piles [Vessic, 1977]. For example:

1) Piles can be used to transmit load to underlying bedrock or stronger soil layers when the ground layers immediately beneath a structure are too weak to support
its weight. If there is no bedrock layer, the resistance of the structural load can come from the frictional resistance provided at the soil-pile interface.

2) Pile foundations can resist horizontal forces while still supporting the vertical loads transmitted by the structure. This is very useful in areas where there are high winds or seismic forces.

3) Piles can be extended past the active zones of expansive or collapsible soil regions. In areas such as these, shallow foundations would suffer severe damage.

4) Piles can be used to resist the uplift forces.

5) Piles are useful in supporting bridge abutments and piers where shallow foundations would be damaged by the soil erosion at the ground surface.

These are a few of the many uses of deep foundation piles. While piles can be made of many different materials, the focus of this document is directed towards reinforced concrete piles. The typical reinforcement material used is steel. Unfortunately, steel can rust, and can be very costly and heavy. The IsoTruss® offers a better alternative to long-term reinforcement because it is lightweight, corrosion resistant, and is more rigid than the steel cage.

1.3 INTRODUCING THE ISOTruss®

The IsoTruss® is a combination of fiber and resin that is wound in interlocking patterns and cured to form a three-dimensional lattice structure. The interlocking pattern is composed of helical members that follow a diagonal path wrapping around the structure, and longitudinal members that run along the length of the structure. These members are
displayed in Figure 1-1. By varying the number of members and the winding pattern, different strengths of IsoTruss® structures can be created and transformed into many different types of supporting structures.

![Figure 1-1 Helical and Longitudinal Members of an IsoTruss® Structure](image)

The IsoTruss® design incorporates primarily axial force members oriented at angular intervals (30 and 60 degrees) to form stable triangular cells – much like a simple truss system [Jensen, D.W., et al., 1996]. This is the key to its exceptional strength-to-weight ratio. The longitudinal members resist axial and flexural loads while the helical members carry torsion and shear loads. While high strength and stiffness are obvious qualities desired in all structural members, there are many other components that are just as important. Some of these include ease of transport and maintainability.

One of the finer qualities of the IsoTruss® is that it is incredibly lightweight. While a forklift must handle a sample of steel reinforcement, the same size sample of IsoTruss®
can be carried by hand with ease. Many would be fooled and think that this fact would impair its strength, but that is not true. The IsoTruss®, as mentioned earlier, has an incredible strength-to-weight ratio which comes from its truss-like design and use of interlocking helical and longitudinal members. The fact that it is so lightweight creates many benefits such as ease of transportation, handling and installation.

Another characteristic of a carbon-fiber based IsoTruss®, which is especially useful when considering deep foundation piles, is that it will not corrode. Many times reinforced concrete piles are used in areas subject to large amounts of water and salt. When these come into contact with the steel reinforcement, rust results and causes induced tensile stresses to build within the member. Eventually, these tensile stresses are higher than the concrete can handle and the concrete begins to crack and spall. In fact, the deterioration of timber, concrete, and steel piling costs the United States nearly $1 billion per year for repair and replacement [Lampo, et al., 1998].

The research center of the Florida Department of Transportation conducted a study on how corrosion damage affects a pile’s axial and flexural capacity. The results of the test indicate that under moderate damage, capacity reductions ranged between 35% (axial loading) to 55% (under bending). In the case of severe damage, reductions were much greater, varying between 40% (axial) to 85% (bending) [FDOT, 1999]. Corrosion damage would never be an issue with a carbon-fiber IsoTruss® reinforced concrete pile, offering considerable savings in terms of structural degradation and replacement.
An IsoTruss® reinforced concrete deep foundation pile had never been produced or tested until this research. It is always exciting to be a part of something new, and watch an interesting idea become a reality. The IRC pile is that new idea and has the potential to catch on as a more efficient form of reinforcement for deep foundation piles.

1.4 Scope of Research

This research compares the flexural behavior of four pre-fabricated reinforced concrete piles. The piles were approximately 14 ft (4.3 m) in length and 14 in (36 cm) in diameter. Two of the samples were reinforced with steel rebar and the other two were reinforced with an IsoTruss® structure. The four-point bending tests were performed in a laboratory setting and data acquisition devices measured load, deflection, and strain. From these quantities, the mid-plane strains and curvatures were calculated and compared. The data and results within this report demonstrate that the IRC pile does have the strength and stiffness to compete as a deep foundation pile.
2 Pile Design, Fabrication and Characteristics

The production of the IsoTruss® structures used in the IRC piles was a tedious and detailed process performed by others. This chapter contains a brief description of the design, production and characteristics of the reinforcement as well as the piles themselves. A more detailed report of these processes is given in [McCune, 2005].

2.1 REINFORCEMENT DESIGN

The design procedure for the pile reinforcement consisted of three main steps:

1) Rounded node design;
2) IsoTruss® reinforcement design; and,
3) Steel reinforcement design.

This chapter describes the processes used in each of the three steps and the final reinforcement design.

2.1.1 ROUNDED NODE DESIGN

A node of an IsoTruss® structure is the location where opposing helical members intersect. The nodes typically meet at a point, resulting in pyramidal sections, as shown
in Figure 2-1. To optimize the flexural performance of the longitudinal members, the helical member area needs to be maximized while limiting any increase in diameter of the IsoTruss®. The use of rounded nodes allows a larger diameter for the helical members without any increase in overall diameter. This allows the reinforcement structure to fit in the 14 in (36 cm) diameter pile forms.

The elliptical shape of the rounded nodes was designed using a system of equations that described the path of the helical members of the IsoTruss®. Constraints such as the degree of curvature and slope magnitude refined the design. An illustration of the rounded nodes is shown in Figure 2-2, where “δɝ” is the inner radius of the IsoTruss®.
2.1.2 ISOTruss® Reinforcement Design

The longitudinal members of the IsoTruss® were designed to match the bending stiffness of their steel counterpart. Bending stiffness is proportional to the product EI, where “E” is the modulus of elasticity and “I” is the moment of inertia. The modulus of elasticity of the IsoTruss® can be determined using the rule of mixtures. Based on the modulus of elasticity of the steel [29,000 ksi (20,000 kN/cm²)], the required moment of inertia of the IsoTruss® can be easily determined.

Using the solver function in an Excel spreadsheet, with constraints such as inner and outer diameter of the structure, degree of curvature, slope of the helicals, and diameter to bay length ratio, the necessary winding passes, or tows, for each longitudinal member was determined to be 133 tows. The grid structure geometry consisted of an outer diameter of 13.0 in (33.0 cm), an inner diameter of 11.4 in (28.9 cm) and a bay length of 7.44 in (18.9 cm). The individual helical members were 0.36 in (0.91 cm) in diameter and the individual longitudinal members were 0.44 in (1.1 cm) in diameter.

An IsoTruss® structure typically employs only about 1/2 to 2/3 as many tows in the helical members as the number of tows in the longitudinal members. For these structures, 89 tows in each helical member, which is 2/3 of the 133 tows in each longitudinal member, was arbitrarily selected. The transverse steel reinforcement was designed to have a stiffness equivalent to the selected IsoTruss® reinforcement design. A depiction of a section of the IsoTruss® reinforcement is shown in Figure 2-3.
2.1.3 STEEL REINFORCEMENT DESIGN

Matching the effective bending stiffness of the steel reinforcement to that of the carbon fiber IsoTruss® yielded a direct method for obtaining the dimensions of the longitudinal steel reinforcement. A total of 8 bars were chosen to closely simulate the 8 longitudinal members of the IsoTruss®. Further, #4 Grade 60 bars were selected to prevent overly-large loads during testing. Using the parallel axis theorem, the moment of inertia for this configuration of bars was calculated. Combining this with the modulus of elasticity of steel, the effective bending stiffness was determined. The stiffness was used to calculate the required cross-sectional area of the longitudinal members of the IsoTruss® reinforcement.

Designing the transverse reinforcement was slightly more complicated. The contributions of the helical members of the IsoTruss® were compared to circular steel
ties. Vector calculations were performed to account for the orientation of the helical members and from this, #2 Grade 60 steel ties spaced at ½ of the bay length of the IsoTruss®, or 3.72 in (9.45 cm) was used. The inner diameter of these hoops was 9 in (23 cm). A summary of the cage diameters and the individual member areas of both types of materials is given in Table 2-1.

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Steel [in (cm)]</th>
<th>IsoTruss® [in (cm)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outer Cage Diameter</td>
<td>9.0 (23)</td>
<td>13.0 (33.0)</td>
</tr>
<tr>
<td>Longitudinal Member diameter</td>
<td>0.5 (1.3)</td>
<td>0.44 (1.1)</td>
</tr>
<tr>
<td>Transverse/Helical Member Diameter</td>
<td>0.25 (0.64)</td>
<td>0.36 (0.91)</td>
</tr>
</tbody>
</table>

2.2 Fabrication

2.2.1 IsoTruss® Reinforcement

The IsoTruss® structures in this study were manufactured from T300C 200NT 12K tow carbon fiber pre-impregnated with TCR UF3325-95 epoxy resin. The tows were hand wound on an aluminum mandrel with wooden dowels and vinyl ester heads, as shown in Figure 2-4. The mandrel rotation was automated by a Howell 3-phase, 220 volt, 3 Hp motor. The speed was manually controlled using an Omron 3G3MV inverter.

Figure 2-4 Section of Assembled Mandrel with Partially Wound IsoTruss®
2.2.1.1 WINDING

The fiber was wound in bundles of four to six tows, rather than a single tow, to cut down the production time. The entire process required two to three people to hold spools of fiber, one person to bundle the tows together and lead them on the right path, and a final person to operate the motor to rotate the mandrel, as shown in Figure 2-5. Constant tension on the fiber was essential during the winding process to ensure the fibers were straight and consolidated as densely as possible.

![Figure 2-5 Manual Filament Winding Process](image)

Because the winding was done by hand, keeping constant tension on the carbon fiber was difficult. Even though the specimens were subsequently consolidated with shrink tape, a loose wind contributes to non-straight fibers, which reduces the strength of the specimen. Though visual checks were made to the straightness as the winding progressed, an estimated error of 5 degrees was possible, which would result in an error of 1.4%. Error also occurred due to inadequate consolidation. The fiber-volume fraction of the
specimen was measured as 50.2% where the predicted design fiber volume fraction was 52.6%. This indicates that the actual specimens had more voids and that the loose consolidation created an error of 4.8%. These factors led to a 20% reduction in compressive strength of the members [McCune, 2005]. Ideally, the structures would have been wound automatically, however the resources were not available and so the hand winding was done as carefully as possible.

2.2.1.2 CONSOLIDATION

Upon completion of the winding process, the IsoTruss® samples were consolidated with Dunstone Hi-shrink tape. One side of the tape was release-coated with Teflon® to aid in its removal once the curing process was complete. The tape was wound tightly around each member and anchored by tying, as demonstrated in Figure 2-6.

Figure 2-6 Consolidation of IsoTruss® Members using Shrink Tape
2.2.1.3 Curing

Curing took place in a hand-made oven. The oven was open at each end where heaters blew in hot air, and a fan was used to facilitate air movement. The curing procedure was based on the requirements for the UF 3325-95 epoxy resin used in the carbon fiber. The structures were heated to a temperature of 290°F (140°C) in intervals of 5°F (9°C) per minute. This temperature was held for two hours, and then cooled at the same rate until the temperature reached 150°F (66°C).

2.2.1.4 Cutting

The final step in the production process was cutting the two 30 ft (9 m) specimens to the desired lengths for testing. A 3.1 ft (0.94 m) piece of one end of each sample was cut off to be used in a compression strength test, leaving a pile length of 26.9 ft (8.20 m).

One of the samples will be tested in the field. Since this sample needed to be 30 ft (9 m) in length, a small steel reinforcing cage was spliced onto the IsoTruss® section to replace the section removed for the compression strength test. The steel section is shown in Figure 2-7.

The steel piece was the same configuration as that of the steel reinforcement used in the other piles except for a slightly larger steel ring diameter. The larger diameter was necessary to provide a compatible fit with the diameter of the IsoTruss®. The other 26.9 ft (8.20 m) sample was cut in half and, after removing some scrap so the final cuts were located at a joint before a node, the two lab sample lengths were obtained. Each pile had a final length of 13.3 ft (4.05 m).
2.2.2 **STEEL REINFORCEMENT**

The steel reinforcement cage consisted of eight #4 Grade 60 bars with #2 Grade 60 ties spaced at 3.72 in (9.45 cm). The longitudinal members were anchored to the transverse hoops using steel tie wire. The cage lengths for the lab samples were 13.3 ft (4.04 m), which was chosen to match the lengths of the IsoTruss® samples. The field test sample needed to be 30 ft (9 m) long, and since the standard rebar length is 20 ft (6 m), a standard splice was required. The splices were alternated so that half of them appeared at one end, and the rest were at the other end of the pile.

2.2.3 **STRAIN GAGES**

2.2.3.1 **PLACEMENT OF GAGES**

Eighteen TML WFLA-6-11 strain gages were applied to the longitudinal members of the steel and IsoTruss® reinforcement structures. These particular gages had lead wires already attached. The locations of the gages on the samples were sanded and cleaned with acetone in preparation for their mounting, and were applied using Cyanoacrylate (CN) adhesive.
To waterproof the gages, the lead wires were insulated from the reinforcement with vacuum tape and hot wax was poured over the gage-wire combination. A second layer of vacuum tape was then applied over the wax. To provide mechanical protection during the concrete pour, the waterproofed gage was covered in Standard Araldite A Epoxy. All of the lead wires were attached to the longitudinal members of the reinforcement using electrical tape. A representation of the location of the internal strain gages is shown in Figure 2-8.
2.2.3.2  STRAIN GAGE LOCATIONS

Before the steel and IsoTruss® cages were enveloped with concrete, there needed to be a way to locate the center of the strain gages across the length of the pile so the location of each gage would be constant for the tests. Because the lengths of the cages were different, some additional measurements needed to be made. The distance from the non-wire end of the cage to the center gage was recorded, as shown in Table 2-2. The cage was inserted as far into the concrete form as was possible.

<table>
<thead>
<tr>
<th>Sample Type</th>
<th>Pile</th>
<th>Length [ft (m)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lab</td>
<td>SRC 1</td>
<td>13.6 (4.15)</td>
</tr>
<tr>
<td></td>
<td>SRC 2</td>
<td>13.6 (4.14)</td>
</tr>
<tr>
<td></td>
<td>IRC 1</td>
<td>13.4 (4.09)</td>
</tr>
<tr>
<td></td>
<td>IRC 2</td>
<td>13.5 (4.10)</td>
</tr>
<tr>
<td>Field</td>
<td>SRC</td>
<td>30.3 (9.23)</td>
</tr>
<tr>
<td></td>
<td>IRC</td>
<td>30.3 (9.24)</td>
</tr>
</tbody>
</table>

2.2.4  INCLINOMETER PIPE

Slope indicator standard pipe was placed in the samples so that an inclinometer could be used to obtain deflection measurements. The pipe has an outer diameter of 2.75 in (6.99 cm) and an inner diameter of 2.32 in (5.89 cm) and is held into place with tie wire at regularly spaced intervals along the reinforcement cage. The pipes were sized to accommodate a vertical inclinometer for field testing rather than a horizontal version for lab testing. The pipes appear in all samples to ensure similar behavior between lab and field samples.
2.3 CONCRETE PLACEMENT

2.3.1 DESCRIPTION OF POUR

The reinforcement for the two 30 ft (9 m) field samples and four 13 ft (4 m) lab samples was shipped to Eagle Precast Company in Salt Lake City for the placement of the concrete. Half of the field and lab samples were steel reinforced while the other half contained IsoTruss® reinforcement.

The reinforcement cages were encased in concrete using 14 in (36 cm) diameter Kolumn Form™ forms purchased from Caraustar®. The forms were 30 ft (9 m) long and two were cut to accommodate the lab samples. The concrete was placed with the forms lying horizontal to avoid the possibility of damage to the reinforcement. Wooden cradles and collars supported the forms during placement, as shown in Figure 2-9. Rectangular holes were cut into the tops of the forms to provide an entrance for the concrete. Two holes were cut in each of the long forms at ¼ and ¾ of the length, and one in each of the shorter forms at the center. Wooden troughs were fit around the holes to guide the concrete into the forms.

![Figure 2-9 Forms Prepared for Concrete Pour](image)

Figure 2-9 Forms Prepared for Concrete Pour
Self-consolidating concrete was chosen to facilitate flow through the horizontal forms. Eagle Precast estimated the strength of the concrete to be 12 ksi (83 MPa), which was a little high, however, as the 30-day measured strength was 7.22 ksi (49.8 MPa) based on four samples. A further description of the concrete compression strength results is given later. Small holes were drilled into the tops of the forms at several locations to indicate when the forms were full. Six 6.0 in (15 cm) diameter cylindrical samples were also cast to provide strength data throughout testing.

2.3.2 Problem with Pour

A problem that arose during the pour dealt with the fact that the forms were horizontal. The concrete did not flow the full length of the forms and was preventing the forms from completely filling. To solve this problem, additional rectangular holes were cut into the tops of the forms near the ends, creating additional entrances for the concrete. This seemed to solve the problem, but inspection of the cured piles after removal of the forms revealed slight dips along the top at greater distances from the insertion hole.

2.4 Pile Specifications

2.4.1 General Characteristics

The finished product consisted of two 30 ft (9 m) field piles and four lab samples about 13 ft (4 m) long. One of the field piles and two of the lab piles were reinforced with steel and the others were reinforced with the IsoTruss®. The lengths of each pile are reported in Table 2-3. Each pile had a diameter of approximately 14 in (36 cm).
Table 2-3 Lengths of All Piles

<table>
<thead>
<tr>
<th>Pile</th>
<th>Length [ft (m)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>SRC 1</td>
<td>6.58 (2.01)</td>
</tr>
<tr>
<td>SRC 2</td>
<td>6.58 (2.01)</td>
</tr>
<tr>
<td>IRC 1</td>
<td>6.67 (2.03)</td>
</tr>
<tr>
<td>IRC 2</td>
<td>6.65 (2.03)</td>
</tr>
</tbody>
</table>

It is interesting to note the difference in weight between the steel and IsoTruss® reinforcements, as displayed in Table 2-4. As mentioned in the first chapter, one of the benefits of the IsoTruss® is that it is very light and easy to handle. The IsoTruss® reinforcement is about 38% the weight of the steel reinforcement, even though they are approximately the same length and diameter.

Table 2-4 Weights of Reinforcement in Pile Samples

<table>
<thead>
<tr>
<th>Sample Type</th>
<th>Reinforcement</th>
<th>Pile</th>
<th>Weight [lb (kg)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lab</td>
<td>Steel</td>
<td>1</td>
<td>97 (44)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>97 (44)</td>
</tr>
<tr>
<td></td>
<td>IsoTruss®</td>
<td>1</td>
<td>37 (17)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>37 (17)</td>
</tr>
<tr>
<td>Field</td>
<td>Steel</td>
<td>1</td>
<td>232 (104)</td>
</tr>
<tr>
<td></td>
<td>IsoTruss® w/o steel piece</td>
<td>1</td>
<td>76 (34)</td>
</tr>
<tr>
<td></td>
<td>IsoTruss® w/ steel piece</td>
<td>1</td>
<td>110 (50)</td>
</tr>
</tbody>
</table>

2.4.2 SURFACE QUALITY DEFECTS

As mentioned previously, the concrete forms were not completely filled resulting in a disruption in the smoothness of the concrete. Slight dips were observed at the top of each pile where the concrete did not come into contact with the form, as shown in Figure 2-10. The maximum depth of these dips was 0.5 in (1.3 cm).
2.4.3 **Circularity**

Another imperfection occurred with the circularity of the pile. Webster defines circularity as the quality or state of being circular. Because the piles were poured and cured horizontally, gravity pushed down on the length of the forms and caused their cross-sections to compress slightly. The measurements of the cross-sections of the lab piles, taken in four directions and three locations with large calipers are summarized in Appendix A. In the table, the “wire end” refers to the end of the piles where the bundles of wires from the strain gages exited out. Figure 2-11 explains the angles at which measurements were taken and Figure 2-12 portrays the compressed cross-section of a pile.
Figure 2-12 Compressed Circularity of Concrete Pile Cross-Section with Circular Dashed Line Superimposed on Photograph for Reference
3 Lab Test Set-up

Preparation for the lab tests included designing and building the test fixture, setting up each element of data acquisition, and performing several other checks before each test. This chapter documents these processes in detail.

3.1 Test Fixture Components

The test fixture consisted of three main components:

1) The reaction frame;
2) The point load system; and,
3) The data acquisition equipment.

3.1.1 Reaction Frame

The reaction frame employed in this research is composed of multiple 14x398 steel I-beams and 1.5 in (3.8 cm) gussets welded together using 0.75 in (1.9 cm) fillet welds. The depth and width of the I-beams are 18.5 in (47.0 cm) and 16.6 in (42.2 cm), respectively. The frame is constrained from movement by a combination of 2.38 in (6.03 cm) and 1.56 in (3.97 cm) diameter bolts which anchor the frame to the floor. Figure 3-1 provides a depiction of the reaction frame. Complete dimensions of the entire frame are included in Appendix B.
3.1.2 POINT LOAD SYSTEM

The lab tests simulated a 12 ft (4 m) simply-supported beam subjected to two point loads. The surface area of the loading devices was too large to know exactly where the load was being applied. In order to better simulate pinned connections and point loads, additional pieces were fabricated.

The idea was to create a system that forced the load to follow a path straight through its center and into the pile. This was accomplished by combining a system of grooved steel plates with a cylindrical bar wedged in between. The plates rested on a cradle that fit
against the pile which restricted movement. Load applied to the first steel plate was directed through the bar, second steel plate, cradle, and directly into the pile. A point-load system was located at each reaction point and at each load point. At the load points, additional steel bars were used to hold the load cells in place. The point load system is shown in Figures 3-2 and 3-3.

![Figure 3-2 Front and Side View of Complete Point Load System at Load Points](image)

![Figure 3-3 Point Load System: a) Close-up; and, b) As Installed in Test Set-up](image)
3.1.3 Data Acquisition Equipment

Different types of sensors were used in the tests to measure load, deflection, and strain. This section describes the types of devices used, how they were set-up, and where each was located along the pile.

3.1.3.1 Loading Devices

Two different loading devices were used during testing. For the testing of the first SRC pile, two Power Team 150-ton (1300 kN) hydraulic jacks were used. The jacks were located 4 ft (1 m) in from each reaction point. The disadvantage of using hydraulic jacks for testing rather than a hydraulic actuator was that there was no way to control the loading rate. Throughout the test, keeping the rate constant between the two jacks was difficult and this weakness was displayed on the computer screen as the loading was monitored.

For the remainder of the tests, the jacks were replaced with a single MTS 100-kip (400 kN) hydraulic servo-valve actuator with a built-in load cell and swivel head. The single actuator allowed full control of loading rate and eliminated the possibility of one device loading faster than the other. The actuator applied load through a point load system to a 14x398 steel I-beam. The load was transferred through another set of point load systems in contact with the pile at 4 ft (1 m) from each reaction point. Figures 3-4 thru 3-7 show these loading devices and the path of the load from the actuator.
Figure 3-4 Loading System with Hydraulic Jacks

Figure 3-5 Hydraulic Jacks Used in Test of SRC Pile #1
Figure 3-6 Loading System: a) Actuator; and, b) Load Path Through the I-beam

Figure 3-7 Hydraulic Actuator with Beam
3.1.3.2 Load Cells

RST Instruments model SG300 300-kip (1300 kN) capacity load cells with a tolerance of +/- 0.1% were used to monitor the loads applied to the piles. The load cells rested in a compartment created by steel bars welded on the top of the point load system. For the testing of the first SRC pile, a load cell was located at the end of each jack. For the other tests, a load cell was located on each point load system to record the actual load applied to the pile through the beam. Measurements were also taken from the load cell that was built into the actuator.

3.1.3.3 Linear Motion Transducers

Deflection measurements were made by Patriot Sensors linear motion transducers, model P-20A. The tolerance of the transducers was +/- 0.1% with a range of motion of 20 in (51 cm). While the transducer was clamped to a steel beam, an attached string was pulled out and hooked to a piece of wire that was strung around the pile. The deflection of the pile caused the string to retract which equaled the deflection in the pile at that point.

Within the 12 ft (4 m) test length, deflection measurements were taken at 1 ft (0.3 m) increments from 2 ft (0.6 m) to 10 ft (3 m), resulting in nine transducers total. Three points around the circumference of the pile were indicated at each foot mark to align the attachment wires around the pile. The location and complete attachment of the linear motion transducers to the piles are displayed in Figures 3-8 and 3-9.
Figure 3-8 Locations of Deflection Measurements

Figure 3-9 Attachment of Linear Motion Transducers
The method used to attach the linear motion transducer to the pile could be a source of error. First, most of the pre-test checks were made visually. Human judgment is not perfect, and would present error when making sure everything was lined up. Also, the tension in the wire encircling the pile could have caused the wire to stretch and throw off the deflection measurements.

Assuming a maximum offset in the location of the wires around the pile of 1.0 in (1.3 cm) and a distance of 16 in (41 cm) from the pile to the linear motion transducers, the error incurred is less than 0.2%. This error is not sufficient to provide any major discrepancies. Figure 3-10 depicts the error. In the future, a more precise alternative would be to bond a hook to the pile. This way, the transducer could be hooked to the pile directly and error due to distortion of the string around the pile would be prevented. Also, more precise measurements could be made to ensure the transducers were lined up more accurately.
3.1.3.4 LVDTs

Another device used to measure deflection was a linear variable differential transducer, or LVDT. RDP Group model ACT500A with a tolerance of +/- 0.25% was used and consists of a cylindrical shaft with a probe that projects from its surface. The LVDT was clamped to a wooden structure and the probe was allowed to rest on the surface of the pile. As movement occurred, the probe compressed in proportion to the deflection of the pile. One LVDT was placed at each reaction point.

In a perfect simply-supported beam, no movement should occur at the reaction points. The purpose of the LVDTs was to indicate how closely the lab tests resembled this ideal: the smaller the deflection measurements at the reaction points, the closer to a properly supported beam which indicates the effectiveness the point load system.

3.1.3.5 Surface Strain Gages

In addition to the strain gages mounted on the reinforcement, two 4.5 in (11 cm) TML PL-90-11-1L self temperature compensating strain gages were applied to the surface of the pile. The gage tolerance was +/- 0.85 (µm/m)/ºC. The gages were located at the center of the testing area oriented longitudinally and located circumferentially 90º on either side of the top of the pile. To ensure each gage was placed at the precise location, the horizontal axis of the cross section at each end was located. A chalk line was pulled along the length of the pile and lined up with each axis point to provide the horizontal axis of the pile. This was repeated for both sides of the pile. The center of the strain
gages was located, using measurements taken before the reinforcement was encased with concrete. The strain gages were placed at the intersection of these two reference lines.

To apply the gages, a layer of Devcon® Polystrate™ 5-minute epoxy gel was applied to the surface of the pile. The cured epoxy was sanded to provide a smooth and flat surface. A second layer of epoxy was applied, and the gage was carefully placed, making sure no air bubbles remained. This epoxy secured the gages to the pile.

### 3.2 Test Preparation

Before each test was run, several checks were performed as precautions to ensure the best possible accuracy in the data. These checks can be classified into three categories:

1) Equipment check;

2) Sensor alignment; and,

3) Strain gage orientation.

Each of these checks is described below.

#### 3.2.1 Equipment Check

The internal strain gages required a set of checks to ensure each was functioning correctly. The handling between concrete placement and testing was sufficient to provide opportunities for damage to occur. Each pair of strain gage lead wires was tested with a voltmeter and this device confirmed that all of the gages were intact. Once this was completed, all of the sensors were attached to the data acquisition. The final equipment
check ensured that all of the devices registered in the computer and were responding properly.

### 3.2.2 SENSOR ALIGNMENT

The first alignment check performed was to make sure that the center of the strain gages was in the center of the test length using data from Table 2-1. The next alignment check ensured that the reactions and loading points were at their proper locations. The reaction points needed to be exactly 12.0 ft (3.7 m) apart and exactly 6.0 ft (1.8 m) from the center strain gages. The center of the point load systems at each loading point needed to be exactly 4.0 ft (1.2 m) from each other, 4.0 ft (1.2 m) from the closest reaction point, and 2.0 ft (0.6 m) from the center of the strain gages.

For the tests of SRC Pile #2 and the IRC piles, one additional check was made to ensure the actuator was in the center of the I-beam that transferred the load, and that the point load systems were equidistant from this center point. The point load systems also needed to be in their correct location of 4.0 ft (1.2 m) from each reaction point.

The final alignment check ensured that all of the wires connected to the linear motion transducers were horizontal and straight. The wires were lined up with the three marks around the circumference at each axial location. The strings connecting the wire loops to the transducers were also leveled visually.
3.2.3 STRAIN GAGE ORIENTATION

Accurate strain measurements required the internal strain gages to be on a horizontal plane. To ensure a horizontal plane, the pile needed to be perfectly oriented on its supports. Unfortunately, the only reference points available to that plane were the two bundles of strain gage lead wires that exited the end of the pile. A mark was made 2 in (5 cm) up from each of the bundles and these points were connected to make a line. Before each test, a level was taken to this line to make sure the pile was oriented properly.

Any error in testing due to improper orientation would have been caused by misalignment of the wire bundles. Measurements taken on one of the samples after testing revealed that the orientation was off by approximately 16° which is a 4.4% error. In future testing, a more exact reference point is needed to properly locate the horizontal plane of the strain gages. One idea would be to mark the blue inclinometer pipe to indicate the locations of the gages.

3.3 TEST PROCEDURE

3.3.1 FLEXURE TESTS

Prior to each test, the computer and all of the data acquisition equipment were given a chance to warm up for a minimum of 30 minutes. All of the components of the test fixture were closely inspected for proper alignment and orientation. Each pile was loaded monotonically to failure. Failure was defined as the point where the piles experienced a significant decrease in load and maintained that lower load for at least one minute. The jacks applied the load on the first SRC pile at a rate of approximately 1.8 kips/sec (8.0
kN/sec). The applied actuation caused the load from the other tests to increase at a rate of 0.5 kips/sec (2.2 kN/sec). The similarity between results, however, demonstrates that the difference in loading rates had no effect on the outcome of the tests. Each test followed the same procedure because if a test is repeated with similar results, the credibility of the research is increased.

Another reason for a constant loading procedure was because the hydraulic jacks were replaced with a single actuator to provide additional control. The previously used loading procedure was preformed for the remainder of the tests to ensure no deviation due to this factor. This way, at least one set of data from the steel piles could be directly compared to the IRC pile data.

3.3.2 Compression Tests

The concrete compression strength tests were performed in a Baldwin 300,000 lb (1,300 kN) capacity testing machine. Each end of the concrete sample was dipped in liquid sulfur which hardened to form a smooth surface. Capping the specimens in this manner is essential when performing a compression test to provide a smooth and even surface. This allows the application of load to be uniform over the entire surface area of the sample.

3.4 Data Consolidation

The first step in the data reduction process was to condense each data set into a smaller, more manageable one. This was accomplished using a proven Excel macro developed
by members of the Center for Advanced Structural Composites, or CASC. The macro allows the user to specify two columns of data and choose one as the x values and the other as the y values. The data set designated as “x” is stepped according to the user entered starting value and step size. The macro uses the method of least-squares to find the best-fit “y” value corresponding to the new stepped “x” value.

The method of least squares uses the equation of a line:

\[ y = a + bx \]  

(3.1)

to approximate a given set of data, \((x_1, y_1), (x_2, y_2), \ldots, (x_n, y_n)\). The curve of best fit, \(f(x)\), has the least square error, i.e.,

\[
\prod = \sum_{i=1}^{n} [y_i - f(x_i)]^2 = \sum_{i=1}^{n} [y_i - (a + bx_i)]^2 = \text{min} 
\]

(3.2)

To find the least square error, the derivative of the unknown coefficients \(a\) and \(b\) is taken:

\[
\frac{\partial \prod}{\partial a} = -2\sum_{i=1}^{n} [y_i - (a + bx_i)] = 0 
\]

(3.3)

\[
\frac{\partial \prod}{\partial b} = -2\sum_{i=1}^{n} x_i [y_i - (a + bx_i)] = 0 
\]

(3.4)

Expanding these equations gives:

\[
\sum_{i=1}^{n} y_i = a \sum_{i=1}^{n} 1 + b \sum_{i=1}^{n} x_i 
\]

(3.5)

\[
\sum_{i=1}^{n} x_i y_i = a \sum_{i=1}^{n} x_i + b \sum_{i=1}^{n} x_i^2 
\]

(3.6)
Solving for the unknown coefficients yields:

\[
a = \frac{\left( \sum_{i=1}^{n} y \right) \left( \sum_{i=1}^{n} x^2 \right) - \left( \sum_{i=1}^{n} x \right) \left( \sum_{i=1}^{n} xy \right)}{n \sum_{i=1}^{n} x^2 - \left( \sum_{i=1}^{n} x \right)^2}
\]  \hspace{1cm} (3.7)

\[
b = \frac{\sum_{i=1}^{n} xy - \left( \sum_{i=1}^{n} x \right) \left( \sum_{i=1}^{n} y \right)}{n \sum_{i=1}^{n} x^2 - \left( \sum_{i=1}^{n} x \right)^2}
\]  \hspace{1cm} (3.8)

The macro takes the two columns of data, and runs this process over a local region to find the equation of the line of best fit. At each new stepped x value, the corresponding y value is determined based on the equation of the best-fit local line.

In some cases, the data was divided into sections and a cut-and-paste method was used to consolidate these sections. The program did not work when the data went negative or changed direction. When this happened, the negative sections were made positive, the line was fit, and the sign was changed back. This allowed the sections that backtracked to match the original data set.

This program was applied to all of the data prior to any further reduction. Consolidation was performed for load vs. time, load vs. deflection, load vs. strain, moment vs. curvature, and moment vs. mid-plane strain. In all cases, the first set of data was the y value and the second set was designated as the x values. For example, in load vs.
deflection, the deflection values were stepped and the corresponding load was determined. For each consolidation process, a graph was produced of the consolidated curve and the original curve, to compare and make sure the program worked properly. An example is shown in Figure 3-11.

![Graph showing original and consolidated load vs. deflection curves](image)

**Figure 3-11 Original and Consolidated Load vs. Deflection Curves**

### 3.5 Derivation of Curvature and Mid-Plane Strain Equations

The mid-plane strain is the strain that resides at the neutral axis of a symmetric member. One of the fundamental principles of simple beam theory is that in pure bending, cross-sections of a beam remain plane. The mid-plane strain is therefore a measure of how close a beam is to pure bending.
Figure 3-12 displays two strains: one above, \( \varepsilon_u \), and one below, \( \varepsilon_l \), the neutral axis. Each strain has a specific location in the x-y plane. These strains are proportional to the curvature of the beam and vary linearly with the distance y from the neutral axis. In pure bending,

\[
\varepsilon_{(x,y)} = -y\kappa_{(s)}
\]  

(3.9)

where \( \kappa \) is the curvature. In an imperfect testing situation, the strain at the neutral axis is not zero. This changes the equation for strain to:

\[
\varepsilon_{(x,y)} = \varepsilon^0_{(s)} - y\kappa_{(s)}
\]  

(3.10)

where \( \varepsilon^0_{(s)} \) is the strain at the neutral axis. Substituting values for \( \varepsilon_u \) and \( \varepsilon_l \) gives:

\[
\varepsilon_u = \varepsilon^0 - \left(\frac{h}{2}\right)\kappa
\]  

(3.11)

\[
\varepsilon_l = \varepsilon^0 + \left(\frac{h}{2}\right)\kappa
\]  

(3.12)

Solving this system of equations gives an expression for the curvature and the mid-plane strain at any location:

\[
\varepsilon^0 = \frac{\varepsilon_l + \varepsilon_u}{2}
\]  

(3.13)
Equations 3.13 and 3.14 were used to calculate the values for the mid-plane strain and curvature. The mid-plane strain was calculated by simply averaging the two strain values. For the curvature, the difference of the two strain values at each location was taken and divided by the distance between them. This “h” value was 9 in (23 cm) for the SRC piles and 12 in (31 cm) for the IRC piles. The strain data was consolidated using the macro previously described. The mid-plane strain and curvature were stepped and the corresponding total load was interpolated.

From this total load, an Excel look-up function was used to find the values at each loading point. Using statics, the loads at the reaction points were calculated and the bending moment was calculated at each strain gage location along the pile. The bending moment was plotted on the y-axis because the moment vs. curvature curve gives the stiffness of the piles. According to strength of materials concepts:

\[ \kappa = \frac{\varepsilon_i - \varepsilon_u}{h} \]  

where \( \varepsilon_i \) and \( \varepsilon_u \) are the strain values at the inner and outer surfaces of the pile, respectively, and \( h \) is the thickness of the pile.

Rearranging this equation gives:

\[ M = EI\kappa \]  

The effective flexural stiffness of the pile is equal to the slope of the moment vs. curvature plot. One of the assumptions in the design of these piles was that the stiffness
of the two types of reinforcement was equal. By plotting the moment as a function of curvature, the accuracy of the design assumption can be determined.
4 Lab Tests

The SRC piles were tested first, followed by the two IRC piles. Immediately following each test, a compressive strength test was performed on a 6.0 in (15 cm) diameter concrete cylinder. This chapter outlines each test, focusing on observations made throughout the testing, including post-failure observations. Some minor alterations were made to the test fixture as the tests progressed, specifically in the loading devices. Those changes are also detailed here.

4.1 STEEL REINFORCED CONCRETE PILE #1

4.1.1 TEST PROCEDURE AND OBSERVATIONS

SRC Pile #1 was tested to failure on the 44th day of concrete curing. For the first minute of testing, Jack #2 consistently applied a greater load on the pile than Jack #1 by as much as 1 kip (4 kN). Small tension cracks began to appear and continued to grow and multiply in the zone between the two reaction points, with the majority being closest to Jack #2 due to the load discrepancy. The first large cracks appeared at approximately 35 kips (156 kN) into testing, which caused a decrease in the loads applied to the pile.

The load continued to decrease and at approximately 25 kips (111 kN) another problem occurred. After the large cracks at 35 kips (156 kN), Jack #1 started to exceed the load of
Jack #2. This discrepancy, along with the weakened condition of the pile due to the cracking, caused the pile to shift. This shift caused Jack #2 to lose contact with the pile and Jack #1 became the only load point in contact. This loss of contact caused the reading on Load Cell #1 to increase significantly, while Load Cell #2 dwindled down to zero. Testing was terminated due to excessive cracking and the stabilization of the load registered on Load Cell #1. A pre-test and post-test view of the pile is shown in Figure 4-1.

![Figure 4-1 Test Set-up of SRC Pile #1: a) Prior to Loading; and, b) After Loading](image)

4.1.2 POST-TEST OBSERVATIONS

The first crack appeared at 55 in (140 cm) from the wire end of the pile (the end where the strain gage end wires exited the pile). Cracks continued from this point to 105 in (265
cm) with the largest crack occurring at 96 in (240 cm) and measuring 0.25 in (0.64 cm) in width. Figure 4-2 displays these cracks.

Accompanying the cracks was a region where the concrete crushed and broke off. The crushed region was from 65 in (170 cm) to 109 in (277 cm) with complete break-off from 78 in (200 cm) to 109 in (277 cm) from the wire end of the pile. The smallest diameter of the pile within the break-off region was 11.5 in (29.2 cm) at a distance of 100 in (250 cm) from the wire end of the pile. Figure 4-3 depicts this region. A complete listing of crack locations and diameters within the break-off region is in Appendix C.
4.1.3 **FIRST COMPRESSION TEST**

Upon completion of the pile test, a compression test was performed on one of the cylindrical strength samples, as shown in Figure 5.4. The results of the test gave a concrete strength of 7.66 ksi (5.28 kN/cm$^2$). The broken sample is shown in Figure 4-4.

![Figure 4-4 First Compression Test](image)

4.2 **STEEL REINFORCED CONCRETE PILE #2**

4.2.1 **TEST PROCEDURE AND OBSERVATIONS**

SRC Pile #2 was tested to failure on the 50$^{th}$ day of concrete curing. Once again, cracking started in the region between the loading points. A set of slightly larger cracks appeared at approximately 35 kips (155 kN), but the load kept increasing. The first major cracks appeared at approximately 38 kips (169 kN), and caused the load to start decreasing. Further cracking and concrete crushing continued as the load continued to decline.
After the first set of large cracks, movement in the point load system between the actuator and the beam was observed. The actuator swivel head began to pivot on the point load system, creating uneven loading of the beam. At approximately 27 kips (120 kN), the swivel head completely pivoted and caused a drastic change in the loading pattern as the angle of load application had been considerably changed. After evidence that the load was not increasing or decreasing, the test was ended and the actuator was unloaded. Figure 4-5 gives a pre-test and post-test view.

![Figure 4-5 Test Set-up of SRC Pile #2: a) Prior to Loading; and, b) After Loading](image)

4.2.2 Post-Test Observations

SRC Pile #2 cracked in a similar pattern as SRC Pile #1, as expected. The major difference was that the cracks and crushed region were more evenly distributed between
the load points than in SRC Pile #1. This was largely due to the fact that the loading was more evenly distributed until the rotation of the swivel head.

The first cracks appeared at 49 in (120 cm) and continued at approximately 5 in (13 cm) increments until 116 in (295 cm). The largest crack was at 93 in (240 cm) and was 0.25 in (0.64 cm) in width. These cracks are shown in Figure 4-6. As in SRC Pile #1, a crushed region occurred from 72 in (180 cm) to 105 in (267 cm) with complete concrete break-off from 88 in (220 cm) to 105 in (267 cm), as shown in Figure 4-7. Throughout this break off region, the smallest diameter occurred at 100 in (250 cm) and was recorded as 11.9 in (30.2 cm). A complete listing of the locations of cracks and diameters within the broken region is in Appendix C.

Figure 4-6 Tension Cracks of SRC Pile #2
4.2.3 SECOND COMPRESSION TEST

Another compression test was performed to obtain a concrete strength for this test. Results from the test confirm that on this day, the concrete had a strength of 7.83 ksi (5.40 kN/cm²) which was very close in value to the results from the first compression test. The broken cylinder is shown in Figure 4-8.
4.3 IsoTruss® Reinforced Concrete Pile #1

4.3.1 TEST PROCEDURE AND OBSERVATIONS

The first IRC pile was tested on the 52nd day of concrete curing. The same configuration of actuator with point load systems that was used in the testing of second SRC pile was used in this test as well.

From the very beginning of load application the IRC piles behaved differently than the SRC piles. At loads where the SRC piles had yielded and were heavily cracked throughout the region between load points, the IRC pile had much smaller deflections, and hence, much smaller hair-line cracks. The IRC pile seemed to be able to take the load much better and maintain its shape until loads much higher than the total capacity of the SRC piles.

As the test continued, the actuator swivel head started to pivot on the point load system toward the end of the pile where the lead wires were located. At approximately 65 kips (289 kN), the swivel head completely pivoted in a manner similar to the previous test. This occurred at a load that was approximately twice the total capacity of the SRC piles. The movement in the swivel head caused the loads to drop suddenly. Once the loads started to increase again, the loading points transferred unequal loads due to the angle of load application. The load point closest to the wire-end of the pile was taking a considerably larger load than the other point, which caused further cracking on this end of the pile.
The load continued increasing until failure occurred. The manner of failure of the IRC pile was very different than the SRC piles. In this test, the failure was more of a sudden rupture than a failure due to excessive cracking. Both the tension and compression side of the pile experienced a region where the concrete broke off, revealing the damaged IsoTruss\textsuperscript{®} reinforcement within. Although the IRC pile sustained approximately twice the load as the SRC piles and remained in one piece, the IRC pile failed in a much more brittle manner. Once failure occurred, the load decreased quickly and remained very low. After a few more minutes, the actuator was unloaded and the test was completed. A pre-test and post-test view of the pile is shown in Figure 4-9.
4.3.2 Post-Test Observations

A closer inspection of the pile after removal from the test fixture revealed that there were spalled regions on the tension side and crushing in the compression side of the pile. The spalling in the tension side did not occur in the SRC piles. There were also many more cracks in the IRC pile although the cracks were much smaller than those in the SRC piles.

The first crack appeared at 27 in (69 cm) and the final crack was located at 128 in (325 cm) from the end of the pile where the lead wires exit. The largest crack occurred at 82 in (210 cm) and was approximately 0.13 in (0.32 cm) in width. A full listing of the crack locations is included in Appendix C. Clearly, the unexpected rotation of the swivel head caused an uneven cracking pattern, with the majority of the cracks located around the load point closest to the wire-end of the pile.

As mentioned earlier, there was a region where the pile spalled and broke off. The tension spall began at 82 in (210 cm) and ended at 91 in (230 cm) with the region where the concrete completely broke off located from 83 in (210 cm) to 91 in (230 cm). This is represented in Figure 4-10. The compression crushing, which in this case was the same as the break-off region, started at 79 in (200 cm) and ended at 90 in (230 cm). Within these regions, the smallest diameter of the pile was 12.1 in (31 cm) and was located at 88 in (220 cm). The compression break is shown in Figure 4-11. A complete listing of pile diameters within the broken regions is in Appendix C.
Figure 4-10 Tension Side of IRC Pile #1

Figure 4-11 Compression Side of IRC Pile #1
4.3.3 **Third Compression Test**

After the test, another compression cylinder was capped and tested in the Baldwin testing machine. Unfortunately, this specimen was not capped evenly, causing the compression forces to be unevenly distributed. One side of the cylinder broke off at approximately one-half the capacity of the other cylinders, thus invalidating the test. Unfortunately the test can not be excluded, as shown later using Chauvenet’s Criterion. A view of this cylinder is given in Figure 4-12.

![Figure 4-12 Third Compression Test](image)

4.4 **IsoTruss® Reinforced Concrete Pile #2**

4.4.1 **Test Procedure and Observations**

IRC Pile #2 was also tested on the 52\textsuperscript{nd} day of concrete curing. Due to the over-rotation of the swivel head in previous tests, the actuator was mounted directly to the I-beam, as
shown in Figure 4-13. This gave the actuator a flat surface to push against, rather than
the circular shaft in the point load system.

![Diagram of actuator setup](image)

*Figure 4-13 Actuator Set-up for IRC Pile #2 Test*

This test went very smoothly and had no problems, as in the other tests. The pile was
loaded by the actuator in a very constant manner. Cracking began around 21 kips (93
kN) and by approximately 32 kips (140 kN), the pile was fully cracked. Partial failure
occurred at approximately 65 kips (289 kN) and the load dropped slightly, but soon after
continued to increase. At about 66 kips (294 kN) another failure occurred, this one
causing a loud popping sound, and the load dropped off completely. The total load
hovered around 1 kip (4 kN) until the actuator was unloaded and the test completed.

Once again, the failure was more dynamic than the SRC piles. The pile in its pre-test and
post-test form is shown in Figure 4-14.
4.4.2 Post-Test Observations

As in the first IRC pile, multiple small cracks and both a tension spalled region and compression crushed region were observed. The first crack appeared at 34 in (86 cm) and the final was located at 124 in (315 cm). The largest crack was at 64 in (160 cm) and was 0.13 in (0.32 cm) in width. Once again, all measurements were taken from the lead wire end of the pile.
On the tension side of the pile, failure occurred and caused a section of the pile to break away, as shown in Figure 4-15. This region began at 67 in (170 cm) and ended at 80 in (200 cm). On the compression side, a similar region existed and was located from 72 in (180 cm) to 91 in (230 cm), as represented in Figure 4-16. Throughout these regions, the smallest diameter of the pile was located at 74 in (190 cm) and was 11.9 in (30.2 cm).
4.4.3 **FOURTH COMPRESSION TEST**

Following testing, preparation for the second compression test of the day began.

Fortunately, the test went very smoothly and the result gave a concrete strength of 8.65 ksi (5.97 kN/cm²). This cylinder is shown in Figure 4-17.

![Figure 4-17 Fourth Compression Test](image)

4.5 **REVIEW OF RESULTS**

The third compression test specimen was capped incorrectly and resulted in a bad test and low strength. Chauvenet’s criterion states that for four readings, a reading may be rejected when the ratio of maximum acceptable deviation to standard deviation, \( d_{\max}/\sigma \) is greater than 1.54 [Holman and Gajda, 1989]. The value \( d_{\max} \) for the strength tests is equal to 2.48 ksi (1.71 kN/cm²). Therefore, the ratio for Chauvenet’s criterion is equal to:
\[
\frac{d_{\text{max}}}{\sigma} = \frac{2.48}{1.71} = 1.45
\]  

(4.1)

This value is not greater than the limit ratio of 1.54 for four readings and so the concrete compressive strength of test three cannot be omitted based on Chauvenet’s criterion.

Table 4-1 shows a summary of the compression strength test results.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Concrete Cure Day</th>
<th>Max Load [kips (kN)]</th>
<th>Strength [ksi (kN/cm²)]</th>
<th>d/\sigma</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>44</td>
<td>216 (961)</td>
<td>7.66 (5.28)</td>
<td>0.26</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
<td>221 (983)</td>
<td>7.83 (5.40)</td>
<td>0.36</td>
</tr>
<tr>
<td>3</td>
<td>52</td>
<td>134 (596)</td>
<td>4.74 (3.27)</td>
<td>1.45</td>
</tr>
<tr>
<td>4</td>
<td>52</td>
<td>245 (1090)</td>
<td>8.65 (5.96)</td>
<td>0.84</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>204 (908)</td>
<td>7.22 (4.98)</td>
<td></td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>48.35 (215)</td>
<td>1.71 (1.18)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4-2 gives the values of the concrete compression strength if test 3 had been thrown out. Figure 4-18 depicts the maximum load of each compressive strength test plotted against the number of days the concrete had cured when each test was performed. The trend line with the negative slope reflects the data series with the bad data point, and the other trend line is the behavior of the concrete if the bad point were thrown out.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Concrete Cure Day</th>
<th>Max Load [kips (kN)]</th>
<th>Strength [ksi (kN/cm²)]</th>
<th>d/\sigma</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>44</td>
<td>216 (961)</td>
<td>7.66 (5.28)</td>
<td>0.26</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
<td>221 (983)</td>
<td>7.83 (5.40)</td>
<td>0.36</td>
</tr>
<tr>
<td>4</td>
<td>52</td>
<td>245 (1090)</td>
<td>8.65 (5.96)</td>
<td>0.84</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>227 (1010)</td>
<td>8.04 (5.55)</td>
<td></td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>15.5 (68.9)</td>
<td>0.53 (0.37)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 4-3 summarizes the strength of the piles as well as the averages and standard deviations for the SRC piles and the IRC piles, showing the consistency of the results. There were no issues with repeatability between tests which gives validity to the results.

Table 4-3 Summary of Results for Lab Tests

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Ultimate Load [kips (kN)]</th>
<th>SRC Piles</th>
<th>IRC Piles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>36.2 (161)</td>
<td>63 (280)</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>37.9 (169)</td>
<td>65.4 (291)</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>37.1 (165)</td>
<td>64.2 (286)</td>
<td></td>
</tr>
<tr>
<td>St. Dev.</td>
<td>1.20 (5.66)</td>
<td>1.70 (7.78)</td>
<td></td>
</tr>
</tbody>
</table>
5 Lab Test Results

The lab tests provided substantial data to work with and transform into useful results. For each pile, there were a total of 33 locations of data acquisition with an average of approximately 8,000 data points at each location. The data provides some great insight into the comparative flexural performance of the SRC and IRC piles.

5.1 Load

Plotting load vs. time provided some valuable insight as to the exact times of the problems with the loading systems. These graphs also displayed noticeable differences in the behavior of the two types of reinforcement in the piles as they were loaded in bending. These differences are in small part because of the difference in loading rates of the different tests. For SRC Pile #1, as shown in Figure 5-1, it is very clear where Jack #2 lost contact with the pile, thus causing Jack #1 to double in load as it picked up the slack. Similarly, the graphs of SRC Pile #2, as seen in Figure 5-2, and IRC Pile #1, as shown in Figure 5-3, depict when the swivel head rotated, resulting in uneven loading between the two contact points on the pile. The load vs. time graph of IRC Pile #2, shown in Figure 5-4, is the cleanest of the four due to the fact that no major problems occurred during testing.
Figure 5-1 Load vs. Time for SRC Pile #1

Figure 5-2 Load vs. Time for SRC Pile #2
Figure 5-3 Load vs. Time for IRC Pile #1

Figure 5-4 Load vs. Time for IRC Pile #2
5.2 DEFLECTION

Differences in the behavior of the IRC piles and the SRC piles were further displayed in the deflection measurements. In an ideal four-point bending test, deflection would be symmetric from one side of the pile to the other, with the region between the load points being equal. This is due to the symmetric behavior of the moment across the pile, as demonstrated in Figure 5-5.

![Shear Diagram](image1)

![Bending Moment Diagram](image2)

**Figure 5-5 Shear and Moment Diagram for Four-Point Loading of the Piles**

Deflection data were taken at each reaction point, and at 1.0 ft (0.3 m) increments along the length of the pile. From these measurements many different results were examined:

1) Load vs. deflection at each location for each test, as well as averages of the SRC piles and the IRC piles;
2) A comparison of load vs. deflection for corresponding points left and right of center along the pile; and,

3) A look at the deflection along the entire pile at different load intervals.

5.2.1 **TOTAL LOAD VS. LOCAL DEFLECTION**

The first set of deflection data examined was simply load vs. deflection curves for each test. To condense the number of curves, the average of the deflection values was calculated for the two SRC piles and the two IRC piles at each location of measurement. Plotting these curves on the same chart demonstrates that the behavior of the IRC piles was different from the SRC piles, as shown in Figure 5-6.

![Figure 5-6 Average Deflections of All Piles](image-url)
5.2.2 **AVERAGE DEFLECTION AT CORRESPONDING POINTS**

The deflections of corresponding points left and right of center were plotted against each other to determine how closely the lab tests performed to this ideal. It is clearly evident in Figures 5-7 thru 5-11 show that the corresponding deflections on either side of center of the pile were very close in value, thus demonstrating the effectiveness of the point load systems in providing the loads and reactions at their correct location. These charts also show good repeatability of the tests which adds credibility to the results. In these graphs, “L#” and “R#” refer to the location of the deflection measurement along the pile; with the # corresponding to distance in feet to the left (L) or right (R) of center. Following the location is an “SRC” for steel reinforced concrete pile, or an “IRC” for the IRC pile.

![Graph showing average deflections at locations L4 and R4 for all piles.](image-url)

*Figure 5-7 Average Deflections at Locations L4 and R4 for All Piles*
Figure 5-8 Average Deflection at Locations L3 and R3 for All Piles

Figure 5-9 Average Deflections at Locations L2 and R2 for All Piles
Figure 5-10 Average Deflections at Locations L1 and R1 for All Piles

Figure 5-11 Center Deflections for All Piles
5.2.3 DEFLECTION ALONG THE PILE

The final aspect of the deflection data that was examined was the deflection along the pile as loading progressed which showed the symmetry of loading along the pile, with the center as the mirror line. These deflections were determined at 5.0 kip (22 kN) increments using an Excel spreadsheet to interpolate between data points.

Figure 5-12 is a plot of these deflections from 5.0 kips (22 kN) to 35.0 kips (156 kN), which was the maximum load experienced by the SRC piles. This load was chosen to provide a comparison of equal loads of the two types of piles. It is evident that the deflections across the piles were very symmetric, even at high loads, providing yet another source of verification to the accuracy of the system used in the lab tests. A closer look at these graphs shows that as the load progresses, the deflection in the SRC piles is greater than those in the IRC piles. This once again indicates the higher stiffness of the IRC piles. Figure 5-13 displays the deflections along the pile up to the maximum of 60 kips (270 kN).
Figure 5-12 Deflection Along the Piles at Equivalent Loads

Figure 5-13 Deflection Along the Piles at All Loads
5.3 Strain

The other set of data obtained from the tests was strain. There were 18 internal strain gages, with 9 mounted on each side of the reinforcement. In addition to this, an additional strain gage was mounted on each outer side of the center of the piles. Unfortunately, during testing, there were many problems with these face gages. The surface gages either failed to work from the beginning of testing, or were severed due to cracking of the concrete early on in the loading process. The strain data acquired was transformed into load vs. strain diagrams and provided the means to calculate the mid-plane strain and the curvature of the piles.

5.3.1 Total Load vs. Strain

The load vs. strain curves provide some valuable information about the piles. At first look, there seems to be a vast difference between the SRC piles and the IRC piles. The strain curves for the SRC piles reveal that they do not experience as much strain as the IRC piles. Also, the IRC piles held twice as much total load as the SRC piles which create additional differences in the appearance of the curves. Figures 5-14 and 5-15 display the differences.
Figure 5-14 Average Total Load vs. Strain for the SRC Piles

Figure 5-15 Average Total Load vs. Strain for the IRC Piles
At each location, one gage was located on the tension side and one gage was located on the compression side of the reinforcement. It was critical that these gages were aligned on the horizontal plane and that the load was acting perpendicular to this same plane. If the orientation was correct, then at each location, the two sets of strain data should be opposite each other. A close-up of the initial region of the strain curves gives a better look into this occurrence. Figure 5-16 clearly shows that the corresponding strain curves are very close to mirror images of each other. It is interesting to note that the IRC piles experienced a higher strain at lower loads than the SRC piles.

![Figure 5-16 Close-up of Average Total Load vs. Strain for All Piles](image-url)
5.4 Mid-Plane Strain

As stated earlier, in the case of perfect bending, the mid-plane strain of the piles would be zero at any value of pure flexural loading. This was not the case in our tests.

It appears that the strain at the neutral axis does remain relatively close to zero initially, and then steadily increases as the loads approach failure. As in the case of deflection, the SRC piles have a much higher rate of strain per load. The strain at the neutral axis increases greatly with little increase in load as these loads approach ultimate. The IRC pile curves tended to remain linear throughout the entire testing period.

A closer look at the initial region of the curves demonstrates the IRC piles resistance to strain compared to the SRC piles. The SRC piles lose their capacity to take strain almost immediately as the slopes of the curves flatten out around 2 microstrain. The IRC piles, however, continue to strain less per unit moment. Comparisons of the moment vs. mid-plane strain curves of the piles are given in Figures 5-17 and 5-18.
Figure 5-17 Moment vs. Mid-Plane Strain for All Piles

Figure 5-18 Close-up of Moment vs. Mid-Plane Strain for All Piles
5.5 Review of Failure

When the piles were examined after the completion of the tests, the longitudinal members were observed as the members that failed. The helical members did not have the strength to carry the total load, and so when the longitudinal members failed, the strength was gone. Looking back, the helical members may have been under designed. This would explain the complete loss in strength once the longitudinal members were compromised. The steel ties that were chosen were very small compared to what is normally used and the helical members were designed to approximately match the ties, although three additional conservative factors were applied in the SRC piles. These factors are:

1) Nominal strength was used rather than average strength for the steel;
2) An overlap of 46% in the steel ties existed, but was not accounted for in the design of the helical members of the IsoTruss®;
3) The steel hoops were oriented perpendicular to the cross-section which resists containment failure better than the angled helical members of the IsoTruss®; and,
4) The effective diameter of the helical members is much smaller than the diameter of the steel hoops.

5.6 Evaluation of Tests

In spite of all of the possible sources of error for these tests, the results were very repeatable. Looking at the standard deviations of the two tests of each type of pile, the difference in ultimate load was very small. This is not only a product of good design and fabrication, but of the test set-up as well. The moment-curvature diagrams show that the
two types of piles had similar stiffness and allowed an accurate comparison in strength. The minimal movement of the LVDTs at the reaction points shows that a simply supported pinned-pinned connection was very nearly achieved. Due to these precautions and extra time spent on details, the tests ran relatively smoothly and the data was clear and workable.
6 Discussion of Results

From the lab test data, the behavior of the SRC and IRC piles can be examined and compared. Many interesting things occurred and this chapter takes a closer look at the loads, deflections, moment vs. curvature and toughness.

6.1 LOADS

The behavior of the IRC piles under the loading conditions is very different from that of the SRC counterparts. Figure 6-1 shows the loading of all four lab tests. The differences in time of maximum values are due to the different loading rates of each test. It is clear, however, that the IRC piles sustained approximately twice as much load as the steel reinforced concrete piles and sustained a very linear path prior to failure. The reason for the higher strength of the IRC piles is partially due to the difference in ultimate strength [195 ksi (134 kN/cm²) for IRC and 67.8 ksi (47 kN/cm²) for SRC], the different geometries of the reinforcement cages, and different individual member areas. A summary of the different member and cage areas is presented in Table 2-1. These different characteristics in the IRC piles worked together to provide exceptional strength. Also, inspection of the load vs. time curves shows that the IRC piles did not yield as loading progressed, like that observed in the SRC piles.
The SRC piles behaved as expected. In the initial stages of testing, the loading followed a fairly linear path until yielding in the steel occurred due to cracking in the concrete. After this, the curve flattened out until the sample was unloaded. The IRC piles, however, don’t have this flattened region where the deflection continues to increase with very small difference in load. The load continues to grow in a linear manner until ultimate failure occurs, at which point the pile can no longer take any load, and drops down to zero.

Others have researched the use of the IsoTruss® as a method of reinforcement in concrete beams. Testing of the flexural performance of the IsoTruss® alone demonstrated brittle characteristics locally, with ductile global behavior. “Locally, each longitudinal member in compression failed in a brittle manner. Because of the redundancy of the IsoTruss® members, however, a ductile failure occurred globally” [Jensen, C, 2000]. Similar
conclusions were also obtained from testing a rectangular IsoTruss® as a method of reinforcement for rectangular reinforced concrete beams. “Overall the rectangular IsoTruss® specimens exhibited greater ultimate strength while the steel reinforced beams showed greater toughness and ductility” [Jarvis, 2001].

6.2 DEFLECTION

6.2.1 DIFFERENCES FROM PREVIOUS RESEARCH

Figure 6-2 shows the deflections of all the piles at all locations. In the SRC piles, it is clear that once the reinforcement reached its yield point, permanent damage continued with very little increase in load. These curves initially have an elastic region, where if the load were removed, theoretically there would be no permanent damage. Past this point, however, the slope of the deflection curves diminish almost to zero, as it continues to deflect with no increase in load. The slope even reaches negative towards the end, as deflection occurs even as the load begins to drop.
The IRC pile deflection curves follow those of the SRC piles at the beginning; however, the main difference comes once the latter curves begin to flatten out. The IRC pile curves show that as the load continues to increase, the rate of deflection does not change. The load vs. deflection curve remains linear until ultimate failure. There are both pros and cons to this fact. On a positive note, the IRC piles had less damage at higher loads due to deflection than the SRC piles. The IsoTruss® structure combines with the strength of the concrete to resist lateral bending better than a typical SRC Pile.
Figure 6-3 shows a close-up of the average deflection chart. Both the SRC and IRC piles exhibit bi-linear behavior. The deflections at the supports are minimal, which was desired to simulate a pinned-pinned connection.

Other research on the IsoTruss® has shown that deflection curves followed a linear path until failure of the members began, similar to the tests performed for this research. Deflection curves “end very abruptly indicating a brittle failure and little ability to dissipate energy” [Jarvis, 2001]. The results of this test are shown in Figure 6-4.
The results of this research are also consistent with other tests of a similar nature. Tests on the flexural performance of concrete beams reinforced with fiber-reinforced plastic grids found that “the results demonstrate the brittle nature of FRP materials. No evidence of yielding in the reinforcing was observed as the ultimate strength was approached. Both deflection and strain remained linear with respect to applied load at ultimate. The amount of cracking that characterized the failed sections was very small” [Yost, et al., 2001].

Both of these sources, however, used a rectangular composite structure. The difference in geometry produces quite a significant difference in strength and stiffness properties. A test that was more closely related to the research provided in this study displayed much different results. Testing of the flexural behavior of spirally-consolidated IsoTruss® reinforced concrete beams produced the results in Figure 6-5 [Jones, 2000]. These samples were the most similar to the samples used in this research.
Figure 6-5 shows that the IsoTruss® samples did exhibit considerable ductility, unlike the samples in this research. This raises the question as to why the IRC piles showed no ductility. Further research will need to be performed to understand why there is such a great difference in the ductility of the samples investigated in these two studies.

### 6.2.2 Average Deflections at Corresponding Points

The average deflections at corresponding points are all compiled on one chart in Figure 6-6 with the center deflections displayed on Figure 6-7. The data displayed on this chart shows the average of not only the two samples of each type of pile, but also the average of the corresponding points left and right of center. For example, 1-SRC shows the average of the load vs. deflection data for 1.0 ft (0.30 m) left and right of the center of the pile. One thing that is evident is the closeness of the center curve to the 1.0 ft (0.30 m) curve. This was expected because of the nature of the four-point bending test and the
region of constant bending moment between the loading points. Also, this chart shows that the deflections increase as the location moves closer to the center of the pile, which is also expected. The lack of ductility of the IRC piles is also displayed.
6.2.3 DEFLECTION ALONG THE PILE

Figures 6-8 and 6-9 give a different view of the deflection at different locations along the pile. Figure 6-8 shows the difference in load at a constant deflection. For the same maximum deflection experienced at 35 kips (156 kN) in the SRC piles, the IRC piles held 55 kips (245 kN). Figure 6-9 shows the deflection at a similar load. It is very clear that the IRC piles experience much less deflection than the SRC piles and considerably higher loads at ultimate.

![Graph of deflection along the pile](image)

**Figure 6-8 Corresponding Loads at Equal Deflections of the SRC and IRC Piles**
The second derivatives of the curves in Figure 6-9 as well as the deflection curves of the SRC and IRC piles at maximum loads provide some interesting insight regarding the ductility of the structure. In pure bending, strain is equal to the distance from the centroid to the extreme fiber multiplied by the curvature; i.e., $\varepsilon = zd$. This strain is a measure of flexural ductility. The second derivative of the deflection is the bending curvature. Two different z-values were used in order to look at ductility of the structure as a whole and ductility based solely on the reinforcement. The deflection data was fitted to a best fit curve and discretized over the 12.0 ft (3.6 m) test length in 0.1 in (0.25 cm) increments. The derivatives, taken by numerical methods, are shown in Appendix D. This process was performed for the SRC and IRC piles at 35 kips (165 kN) and at maximum loads of 35 kips (165 kN) for the SRC piles and 60 kips (267 kN) for the IRC piles. Table 6.1 gives the results of these calculations. Figures 6-10 and 6-11 show the comparison of the curvatures of the two piles at 35 kips (165 kN) and maximum loads using a z-value of 7.0
in (18 cm) for the surface-mounted strain gages, 6.0 in (15 cm) for the IsoTruss®
reinforcement, and 4.5 in (11.4 cm) for the steel reinforcement.

| Table 6-1 Comparison of Strain-based Ductility of SRC and IRC Piles in Flexure |
|-----------------------------------|-----------------|------------------|----------------|-----------------|
| Pile Type                        | Load            | Surface Train     | Reinforcement  |
|                                  | [kips (kN)]     | [in (cm)]         | [microstrain]  | [in (cm)]       | [microstrain]  |
| Equal Load                       |                 |                  |                |                 |                |
| SRC                              | 35 (165)        | 7 (18)           | 8,400          | 4.5 (11)        | 5,400          |
| IRC                              | 35 (165)        | 7 (18)           | 4,200          | 6 (15)          | 3,600          |
| Failure                          |                 |                  |                |                 |                |
| SRC                              | 35 (165)        | 7 (18)           | 8,400          | 4.5 (11)        | 5,400          |
| IRC                              | 60 (267)        | 7 (18)           | 8,400          | 6 (15)          | 7,200          |

Figure 6-10 Bending Curvature vs. Distance Along Pile at 35 kips (165 kN)
These results show that at the maximum load point of the SRC piles, 35 kips (165 kN), the IRC piles exhibited much less ductility. At their respective maximum load points, however, the IRC reinforcement experienced higher strain based ductility than the SRC reinforcement while the ductility based on strain at the surface of the piles is equal in both the SRC and IRC piles. Figure 6-11 shows that the failure appeared to be controlled by the curvature in the beam rather than the maximum strain in the reinforcement. Failure occurred in both piles as they approached the limit curvature of approximately 0.0084 strain/in. Further investigation is required to explore the possibility that the curvature of the concrete is controlling the ultimate failure.
6.3 Moment vs. Curvature

One of the most important features of a moment vs. curvature chart is the slope. As shown earlier, the slope of this curve is the effective flexural stiffness, EI, of the sample. By comparing the SRC pile plots to those of the IRC piles, the relative stiffness can be compared. This provides some information as to the accuracy of the design of the reinforcement structures.

One common trend continues in the pattern of these curves. Once again, the SRC pile plots level out while those of the IRC piles continue to grow in a linear fashion, as shown in Figure 6-12.
There is a significant difference between the calculated strain-based curvatures and the derived deflection-based curvatures. In fact, the curvature calculated from the strain data yields less than $\frac{1}{2}$ of the derived curvature from the deflection data. One contributing factor in this situation is the fact that many of the strain gages stopped functioning before the test specimens failed. Due to the incomplete strain data, it is difficult to obtain complete curvature results. Also, the cracking in the specimens created non-linear behavior and the curvatures cannot be completely represented by the linear analysis methods presented in this study.

Exploring the behavior of carbon fiber-based rods as flexural reinforcement demonstrates moment-curvature behavior that is similar to the tests performed in this research. For example, “…the steel reinforced beam reaches a certain moment capacity and sustains it for a long increase in curvature before failure. On the other hand, the [carbon-fiber] rods reach a peak capacity before failing” [Thiagarajan, 2003]. Also, “the [carbon-fiber] rods have to undergo high strains to develop high stresses” [Thiagarajan, 2003].

To accurately calculate the stiffness of the piles, the slope of the moment-curvature lines was calculated at the most linear portion, which occurred after the initial noise. The region from 100 microstrain/in (40 microstrain/cm) to 140 microstrain/in (55 microstrain/cm) was examined to get a better idea of the behavior of the samples. As shown in Figure 6-13, the slopes of the curves of the two types of piles are very similar. This demonstrates that the design was fairly accurate and the piles are of similar stiffness.
A more complete analysis of the slopes of the curves was performed using a linear regression function in Excel. For the SRC piles, that slope is 3.8 kip-in$^2$ (109 kN-cm$^2$), and the average slope of the IRC piles is 3.4 kip-in$^2$ (98 kN-cm$^2$). The SRC piles are slightly stiffer than the IRC piles in this region. The real difference, however, comes as the load increases. After the SRC piles reach their capacity, the IRC piles continue to carry additional load.

Figure 6-13 Close-up of Moment vs. Curvature for All Piles

Figure 6-14 gives a final view of the stiffness measurements of the piles, showing the moment vs. EI curves based on the average of the strain gage readings between the load application points. The moments and the stiffness values used in this chart were averaged from the center regions of the two types of piles. It is interesting to see the behavior of the stiffness as the load progresses. As damage to the concrete occurs, the
stiffness decreases accordingly until it is nearly constant. In Figure 6.14, the average values of the flexural stiffness, EI, from the first local minimum to the last local peak is 3.9 kip-in² (44 kN·cm²) for the SRC piles and 3.1 kip-in² (35 kN·cm²) for the IRC piles.

Figure 6-14 Average Moment vs. EI for Both Types of Piles in the Center Region (gages 4-8)

6.4 TOUGHNESS

Toughness describes the energy required to fracture a material. The toughness of the two types of piles was determined by computing the area under the load-deflection curves at the center of the pile. This area is equal to the energy in the pile at failure. If failure is defined as the point where the test was stopped, the toughness of the SRC piles is 168 kip-in (1900 kN·cm) and the toughness of the IRC piles is 83 kip-in (940 kN·cm). The IRC piles absorb much less total energy than the SRC piles. This is not very desirable. Further research needs to be performed to determine why the IRC piles were lacking in ductility in this study. With this ductility in place as it should be, the toughness would be
greater than the toughness of the SRC piles and would be one more advantage of using the IRC piles as reinforcement for piles. It is interesting to note that at the maximum loads, respectively, the toughness of the IRC piles is 83 kip-in (940 kN-cm), but the toughness of the SRC piles is only 74.0 kip-in (836 kN-cm). The toughness up to this maximum load condition is more realistic because most structures are subject to given loads, rather than specified deflections, as in these tests.

6.5 REVIEW OF RESULTS

Table 6-2 summarizes the stiffness, moment, curvature, and toughness information.

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<tr>
<th>Property</th>
<th>SRC</th>
<th>IRC</th>
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<td>Flexural Stiffness [kip-in² (kN-cm²)]</td>
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<td>3.4 (98)</td>
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<tr>
<td>Maximum Moment [kip-in (kN-m)]</td>
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<td>1719 (194)</td>
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<td>Maximum Curvature from Strain Gage [μE/in (μE/cm)]</td>
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<td>505 (199)</td>
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<td>Maximum Curvature from Deflections [μE/in (μE/cm)]</td>
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<td>1200 (472)</td>
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<td>7200 (7200)</td>
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<td>Toughness at Maximum Displacement [kip-in (kN-m)]</td>
<td>168 (1900)</td>
<td>83 (940)</td>
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<tr>
<td>Toughness at Maximum Loads [kip-in (kN-m)]</td>
<td>74 (836)</td>
<td>83 (940)</td>
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</table>
7 Conclusions and Recommendations

Two steel-reinforced concrete (SRC) piles and two IsoTruss\textsuperscript{®}-reinforced concrete (IRC) piles were tested to compare flexural properties. The purpose of these tests was to show that the IRC pile is worthy to compete as a form of reinforcement for deep foundation piles. The IsoTruss\textsuperscript{®} lattice structure as a form of reinforcement is advantageous in many aspects including weight, which allows easy installation and repair; rigidity; and most importantly, superb resistance to corrosion. These factors, combined with the strength necessary to resist the imposed loads, makes the IsoTruss\textsuperscript{®} a preferred alternative for pile reinforcement. This chapter provides a comprehensive summary of the significance of the test results as well as recommendations for improvements in future testing.

7.1 Conclusions

1) Moment-curvature charts reveal the flexural stiffness of the two types of piles and show that they are very nearly equal in value. This validates the design requirement of equal stiffness so an accurate comparison in flexural strength is possible.

2) The IRC piles clearly displayed superior ultimate strength over the steel reinforced piles by supporting approximately twice as much flexural load.
3) IRC piles exhibit linear elastic behavior to failure. Although IRC piles form cracks during loading, the cracks were not nearly as visible as those in the SRC piles. The failure of the IRC Piles was very quick and explosive, characteristic of a brittle material.

With adequate factors of safety during the design process to ensure the loads never approach ultimate, the IRC pile would be more than adequate in supporting lateral loads and, with its other desirable qualities, would be a better alternative to the reinforcement of deep foundation piles.

7.2 RECOMMENDATIONS FOR FUTURE TESTS

There is still so much to learn about the capabilities of the IsoTruss® as a form of deep foundation pile reinforcement, namely an understanding of the absence of ductility observed in the tests performed in this project. For example, field tests are in preparation to determine the properties of the piles in an actual pile application. The set-up for these tests is described in Appendix E. The results are reported in a separate thesis [Richardson, 2005]. If tests similar to the ones performed in this study were conducted again, a few improvements might provide even better results. These improvements include:

1) Pour concrete with the piles on end, i.e., vertically;

2) Provide a clearer reference point as to the location of the strain gages;

3) Attach the actuator directly to the pushing beam; and,

4) Change the attachment method of the linear motion transducers to the piles.
7.2.1 **Vertical Pour**

The horizontal pour of the concrete piles created some change in the cross-sectional shape. Rather than being perfectly circular, the effects of gravity pushed down on the forms during the curing process and resulted in more of an oval shape. The effects this property had on the testing is not known, but could be a source of error. It definitely opens the door to increased error if the orientation of the piles was not exact. The reason the piles were not poured vertically to begin with was a fear that the falling concrete could damage the reinforcement or, more importantly, the strain gages and lead wires that were mounted on the reinforcement cages. If these wires could be protected somehow so that the falling concrete wouldn’t impact them, then a vertical pour would be possible and the circularity of the piles would not be affected.

A vertical concrete pour would also have resulted in smooth, circular piles. The concrete had a hard time completely filling the forms and as a result, the tops of the piles contained dips and voids where the concrete didn’t reach. This would not have been an issue with a vertical pour and all voids would have been filled.

7.2.2 **Clearer Reference**

Correct orientation of the strain gages was crucial to provide correct strain measurements. The location where the lead wires exited the piles was the only reference as to the location of the strain gages. All measurements and calculations were taken from the provided reference. In future tests, a more exact method would be helpful. For example, the inclinometer pipe could be marked and used as a reference point before the pour
began. Since it was braced and supported by the cage itself, minimal movement would occur and the location of the strain gages would be clearer and more precise.

7.2.3 ATTACHMENT OF ACTUATOR

Different configurations were used with the actuator throughout testing. The last test employed the best method. During the testing of SRC Pile #2 and IRC Pile #1, the actuator pushed on a point load system which attached to the pushing beam. In both cases, the point load system caused the swivel head on the actuator to rotate and skew the loading. This was evident in the load vs. time graphs and caused some error. For the testing of IRC Pile #2, the actuator was bolted to the beam directly and this proved to be the best solution. There was no rotation and the loading was constant and smooth. Therefore, the direct connection of the actuator to the beam was the best method of loading.

7.2.4 IMPROVE DEFLECTION MEASUREMENTS

To attach the linear motion transducers to the piles, a wire was wrapped around the circumference and ended in a hook. The transducer attached to the hook to provide the deflection measurements. A better alternative would be to bond a hook directly to the pile using epoxy. That way, the transducer could be directly attached to the pile and there would be no extra deflections. The only problem with this method, however, would be the possibility of the hooks breaking off during testing.
References


Appendix A: Circularity Measurements
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<th>IRC Diameter [in (cm)]</th>
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Appendix B: Reaction Frame Dimensions
NOTE: When welding I-Beams leave the ends of the flanges unwelded.

NOTE: All stiffeners to be welded with 0.5" (1.3 cm) wide full penetration weld

All I-Beam designation unless otherwise noted:
14x390

Depth - 18.5" (47.0 cm)
Width - 16-5/8" (42.2 cm)

Figure B-1 Plan View of Reaction Frame for Laboratory Tests
Figure B-2 Side View of Reaction Frame for Laboratory Tests

Mounting Plate - Test Specimen Side

Mounting Plate - Actuator Side

Figure B-3 Hole Dimensions for Reaction Frame Mounting Plates
Figure B-4 Gusset Dimensions for Reaction Frame Mounting Plates
Figure B-5 Short I-beam Dimensions for Reaction Frame
Appendix C: Locations of Cracks and Spalling
### Table C-1 Locations of Cracks on All Piles

<table>
<thead>
<tr>
<th>SRC Pile #1</th>
<th>SRC Pile #2</th>
<th>IRC Pile #1</th>
<th>IRC Pile #2</th>
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<tr>
<td>[in (cm)]</td>
<td>[in (cm)]</td>
<td>[in (cm)]</td>
<td>[in (cm)]</td>
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<tr>
<td>55 (140)</td>
<td>49 (125)</td>
<td>27 (69)</td>
<td>34 (86)</td>
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<tr>
<td>61 (155)</td>
<td>55.5 (141)</td>
<td>35 (89)</td>
<td>37 (94)</td>
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<tr>
<td>64 (163)</td>
<td>60 (152)</td>
<td>39 (99)</td>
<td>41 (104)</td>
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<tr>
<td>72 (183)</td>
<td>67 (170)</td>
<td>42 (107)</td>
<td>48 (123)</td>
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<td>76 (193)</td>
<td>73 (185)</td>
<td>44 (112)</td>
<td>53 (135)</td>
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<td>80.5 (205)</td>
<td>78 (198)</td>
<td>49 (125)</td>
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<td>88.5 (225)</td>
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<td>91.5 (232)</td>
<td>95.5 (243)</td>
<td>60 (152)</td>
<td>80 (203)</td>
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<td>93 (236)</td>
<td>102 (259)</td>
<td>64 (163)</td>
<td>84 (213)</td>
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<td>96 (244)</td>
<td>103 (262)</td>
<td>72 (183)</td>
<td>99 (251)</td>
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<td>107 (272)</td>
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<td>104 (264)</td>
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<td>91 (231)</td>
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## Table C-2 Break-off Regions for All Piles

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<th>Diameter [in (cm)]</th>
<th>Location [in (cm)]</th>
<th>Diameter [in (cm)]</th>
<th>Location [in (cm)]</th>
<th>Diameter [in (cm)]</th>
<th>Location [in (cm)]</th>
<th>Diameter [in (cm)]</th>
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</thead>
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<td>88 (224)</td>
<td>14.1 (35.9)</td>
<td>78 (198)</td>
<td>14.1 (35.9)</td>
<td>66 (168)</td>
<td>14.1 (35.9)</td>
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<tr>
<td>80 (203)</td>
<td>13.6 (34.6)</td>
<td>90 (229)</td>
<td>14.0 (35.6)</td>
<td>80 (203)</td>
<td>13.9 (35.2)</td>
<td>68 (173)</td>
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<tr>
<td>82 (208)</td>
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<td>92 (234)</td>
<td>13.5 (34.3)</td>
<td>82 (208)</td>
<td>13.5 (34.3)</td>
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</tr>
<tr>
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<td>94 (239)</td>
<td>13.3 (33.7)</td>
<td>84 (213)</td>
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<td>72 (183)</td>
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<td>98 (249)</td>
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<td>88 (224)</td>
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<td>76 (193)</td>
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<td>106 (269)</td>
<td>14.3 (36.4)</td>
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</table>
Appendix D: Additional Charts: Load vs. Deflection, Load vs. Strain, Moment vs. Mid-Plane Strain, Moment vs. Curvature
Figure D-1 Total Transverse Load vs. Deflection for SRC Pile #1

Figure D-2 Total Transverse Load vs. Deflection for SRC Pile #2
Figure D-3 Total Transverse Load vs. Deflection for IRC Pile #1

Figure D-4 Total Transverse Load vs. Deflection for IRC Pile #2
Figure D-5 Average Load vs. Deflection for SRC Piles and IRC Piles at Location S1

Figure D-6 Average Load vs. Deflection for SRC Piles and IRC Piles at Location L4
Figure D-7 Average Load vs. Deflection for SRC Piles and IRC Piles at Location L3

Figure D-8 Average Load vs. Deflection for SRC Piles and IRC Piles at Location L2
Figure D-9 Average Load vs. Deflection for SRC Piles and IRC Piles at Location L1

Figure D-10 Average Load vs. Deflection for SRC Piles and IRC Piles at Location C
Figure D-11 Average Load vs. Deflection for SRC Piles and IRC Piles at Location R1

Figure D-12 Average Load vs. Deflection for SRC Piles and IRC Piles at Location R2
Figure D-13 Average Load vs. Deflection for SRC Piles and IRC Piles at Location R3

Figure D-14 Average Load vs. Deflection for SRC Piles and IRC Piles at Location R4
Figure D-15 Average Load vs. Deflection for SRC Piles and IRC Piles at Location S2

Figure D-16 Deflection Along the Piles at 5 kips (22 kN)
Figure D-17 Deflection Along the Piles at 10 kips (44 kN)

Figure D-18 Deflection Along the Piles at 15 kips (67 kN)
Figure D-19 Deflection Along the Piles at 20 kips (89 kN)

Figure D-20 Deflection Along the Piles at 25 kips (111 kN)
Figure D-21 Deflection Along the Piles at 30 kips (133 kN)

Figure D-22 Deflection Along the Piles at 35 kips (156 kN)
Figure D-23 Total Transverse Load vs. Strain for SRC Pile #1

Figure D-24 Total Transverse Load vs. Strain for SRC Pile #2
Figure D-25 Total Transverse Load vs. Strain for IRC Pile #1

Figure D-26 Total Transverse Load vs. Strain for IRC Pile #2
Figure D-27 Moment vs. Mid-Plane Strain for SRC Pile #1

Figure D-28 Moment vs. Mid-Plane Strain for SRC Pile #2
Figure D-29 Moment vs. Mid-Plane Strain for IRC Pile #1

Figure D-30 Moment vs. Mid-Plane Strain for IRC Pile #2
Figure D-31 Moment vs. Curvature for SRC Pile #1

Figure D-32 Moment vs. Curvature for SRC Pile #2
Figure D-33 Moment vs. Curvature for IRC Pile #1

Figure D-34 Moment vs. Curvature for IRC Pile #2
Figure D-35 Deflection of Piles Along Their Length at 35 kips (156 kN)

Figure D-36 Slope of Deflection of Piles Along Their Length at 35 kips (165 kN)
Figure D-37 Bending Curvature of the Piles Along Their Length at 35 kips (165 kN)

Figure D-38 Deflection of Piles Along Their Length at Max Loads
Figure D-39 Deflection of Piles Along Their Length at Max Loads

Figure D-40 Bending Curvature of the Piles Along Their Length at Max Loads
Appendix E: Field Test Set-Up
The two field test samples were 30 ft (9.14 m) in length: one reinforced with steel and the other with the IsoTruss® structure. These piles were driven into the ground and will be tested in bending.

E.1 Test Site

The field tests were performed at a site located near South Temple in Salt Lake City, Utah [Richardson, In Preparation]. The site is located under a set of freeways and between two sets of railroad tracks, near the crossroads of other railroad tracks. This site was chosen partly because of an existing concrete freeway support foundation that will provide a static structure to push the actuator against.

The area was excavated so part of the concrete block was exposed. The driving locations of the piles were placed approximately 10 ft (3 m) apart to prevent the areas of disrupted soil from intersecting. Also, the piles were positioned to provide reaction surfaces for the actuators. For the IRC pile, that surface will be the concrete block. For the SRC pile, a steel pile filled with concrete will provide the reaction. Figure E-1 gives a representation of the test site.
E.2 Pile Driving

E.2.1 Preparation

Before the piles were driven, an accelerometer and a strain gage were attached to the pile. The accelerometer measurements can be integrated to find the velocity of the pile as a function of time. The strain gage data can be converted into a force vs. time function. Members of the Utah Department of Transportation were interested in these measurements, and made the necessary installations. Unfortunately, in the SRC pile,
holes were drilled directly in line with the internal strain gage wires. Realizing their mistake, the remaining holes were drilled with an orientation rotated 90° from the wires to prevent the possibility of damaging the internal devices. These initial holes were only approximately 1.0 in (2.5 cm) deep, so damage to the wires is unlikely.

A cushion was also attached to the end of the pile to prevent the driving hammer from crushing the concrete locally. This cushion consisted of multiple plywood disks with the same diameter as the piles. For the steel reinforced pile, wooden wedges were used to brace the cushion around the circumference of the piles so that the disks wouldn’t fall off during the driving process. This method didn’t work very well and so pieces of cardboard concrete forms were wrapped around the cushion for the IRC pile. This proved to work much better. Figure E-2 displays this latter method.

![Figure E-2 Cardboard Cushion for IRC Pile](image-url)
E.2.2 PILE DRIVING AND COMPLICATIONS

The piles were driven using an IHC S-70 pile hammer on July 19, 2004. There were a couple of problems that resulted: one minor, and one that proved to be a little more complicated. As mentioned earlier, the wooden wedges used to support the cushion on the SRC pile were not very effective. As a result, the cushion did not stay attached through the duration of the driving process. This caused some damage to the top of the SRC pile. Fortunately, the damage was minimal and should not cause any problems in the testing process.

The other problem arose after driving the IRC pile. For the strain gages to provide the most relevant data, the load path needs to pass directly through the set of gages at each point along the pile. In other words, the pile must be aligned so that the actuator can push perpendicular to the plane of the strain gages. To aid in this process, markings were made that instructed the drivers on the correct orientation of the piles. The SRC pile was driven with the correct alignment but the IRC pile rotated and ended up skewed approximately 16 degrees clockwise from due north. Relative to a line projected along the plane of the strain gages, the actuator would miss the concrete reaction block. Figure E-3 depicts this dilemma.
E.3 SOLUTION TO ORIENTATION PROBLEM

The rotation of the IRC pile created some complications that needed to be resolved before testing could begin: the concrete block needed to be extended and the actuator needed to be rotated to the same angle as the strain gages so it could push directly through them. A steel beam was attached to the concrete block and extended out to the projected location of the actuator. A wedge with the correct angle will assure the correct orientation of the load.

E.3.1 BEAM

A steel I-beam was selected to provide the extra length necessary for the actuator. The beam chosen had been used in previous projects and had some pre-existing alterations. These included pre-drilled holes and additional plates welded to the center. Stiffeners
were also included across the length of the beam. Dimensions are provided in Figure E-4.

Figure E-4 Dimensions of Beam Chosen to Extend Concrete Block: a) Stiffener Dimensions; b) Hole Dimensions; c) Plan View and Thickness Dimensions; and, d) Front View Thickness Dimensions

E.3.2 LOADS

To design the necessary attachments and components of the beam structure, a maximum load of 200,000 lb (1,000 kN) was assumed. This load is highly conservative, but
ensures a safe design. The load acts at an angle of 16.5° clockwise with respect to the concrete reaction block, and so x and y components of this load were calculated.

\[ P_y = P \cos \theta = 200,000 \text{ lb} [\cos (16.5^\circ)] = 192,000 \text{ lb} (853 \text{ kN}) \quad (E.1) \]
\[ P_x = P \sin \theta = 200,000 \text{ lb} [\sin (16.5^\circ)] = 56,700 \text{ lb} (252 \text{ kN}) \quad (E.2) \]

The y-component of the force extends past the edge of the concrete and creates a moment, M:

\[ M = P_y \times y = 192,000 \text{ lb} \times (13 \text{ in}/12 \text{ in/ft}) = 208,000 \text{ lb-ft} (282 \text{ kN-m}) \quad (E.3) \]

where y is the moment arm of the force. These loads and moment are the basis of all further calculations.

E.3.3 ANGLED ACTUATOR WEDGE

An additional component was designed to apply the load through the necessary plane. This piece is composed of a base plate and a top plate supported by three triangular stiffener plates in between. The triangular stiffener plates create an angle of 16.5° to match the required angle for the load path.

A 14 in (35 cm) square base plate was used. The initial thickness was chosen as 0.5 in (1.3 cm). The main concern was to prevent any buckling of the triangular pieces. To do this, a 1.0 in (2.5 cm) section at the tall end of one of the stiffeners was examined. Figure E-5 displays the critical section.
The critical section is 4.14 in (10.5 cm) by 1.0 in (2.5 cm) by 0.5 in (1.3 cm). Two equations can be used to determine the allowable buckling stress for this section. To decide which one, the slenderness ratio must be calculated and compared to the critical value. The slenderness ratio is equal to the effective length, $kl$, divided by the radius of gyration, $r$:

$$C = \frac{kl}{r}$$  \hspace{1cm} (E.4)

The effective length, $kl$, is simply a factor, which is dependent on the type of end connection, multiplied by the length of the sample. The radius of gyration is calculated using:

$$r = \sqrt{\frac{I}{A}}$$  \hspace{1cm} (E.5)

where $I$ is the moment of inertia of the sample and $A$ is the cross-sectional area. The critical slenderness ratio, $C_c$, is found using the equation:
\[ C_c = \frac{2\pi^2 E}{\sqrt{F_y}} \]  \hspace{1cm} \text{(E.6)}

where $E$ is the modulus of elasticity and $F_y$ is the yield strength of the material. In the case of the steel used in this piece, $E = 29 \times 10^6$ psi ($2.0 \times 10^7$ kN/m$^2$) and $F_y = 36,000$ psi ($2.5 \times 10^5$ kN/m$^2$). When the slenderness ratio is less than the critical value, axial crushing is the method of failure. When this is the case, AISC Equation E2-1 gives the allowable axial stress as:

\[
F_a = \frac{\left[1 - \left(\frac{kl}{r}\right)^2\right] F_y}{\frac{5 + \frac{3}{C_c^2}}{3} \left(\frac{kl}{r}\right)^2 - \frac{8}{8} \left(\frac{kl}{r}\right)^3} \hspace{1cm} \text{(E.7)}
\]

When $kl/r$ exceeds $C_c$, buckling governs and the allowable stress becomes:

\[
F_a = \frac{12\pi^2 E}{23(kl/r)^2} \hspace{1cm} \text{(E.8)}
\]

according to AISC Equation E2-2. To analyze the stiffener, the values for this case are simply substituted into these equations:

\[ I = \frac{bh^3}{12} = \frac{(1)(1/2)^3}{12} = 0.0104 \text{ in}^4 (0.434 \text{ cm}^4) \hspace{1cm} \text{(E.9)} \]

\[ A = bh = (1)(1/2) = 0.5 \text{ in}^2 (1.3 \text{ cm}^2) \hspace{1cm} \text{(E.10)} \]

From these, the radius of gyration can be calculated:

\[ r = \sqrt{\frac{0.0104}{0.5}} = 0.14 \text{ in} (0.37 \text{ cm}) \hspace{1cm} \text{(E.11)} \]

Using a conservative $k$ value of 1 for a pinned connection at both ends and an unbraced length of 4.14 in (10.5 cm), the slenderness ratio calculates to:
\[
\frac{kl}{r} = \frac{(1)(4.14)}{0.144} = 29 \quad (E.12)
\]

The limit \( C_c \) is:

\[
C_c = \sqrt{\frac{2\pi^2 (29*10^6)}{36,000}} = 126 \quad (E.13)
\]

Therefore, the unbraced length is less than \( C_c \) and the allowable axial load can be computed:

\[
F_a = \frac{1 - \frac{(29)^2}{2(126)^2}}{\frac{5}{8(126)} - \frac{(29)}{8(126)}^3} \times 36,000 = 20,000 \text{ psi} \ (1\times10^5 \text{ kN/m}^2) \quad (E.14)
\]

The actual axial stress is simply the force divided by the area over which it acts:

\[
f_a = \frac{P_y}{A} = \frac{6570}{0.5} = 13,100 \text{ psi} \ (90,600 \text{ kN/m}^2) \quad (E.15)
\]

The force acting on this ramp has both an \( x \) and \( y \) component. This means that in addition to the axial force calculated above, there is also a shear force. AISC equation F4-1 gives the equation for the allowable shear force:

\[
F_v = 0.4F_y = 0.4(36,000) = 14,400 \text{ psi} \ (9,930 \text{ N/cm}^2) \quad (E.16)
\]

The actual shear force, similar to the axial, is the shear component of the force divided by the area over which it acts:

\[
f_v = \frac{P_x}{A} = \frac{1940}{0.5} = 3,880 \text{ psi} \ (2,680 \text{ N/cm}^2) \quad (E.17)
\]

With these forces, the combined stress can be calculated:

\[
\frac{f_a}{F_a} + \frac{f_v}{F_v} < 1 \quad (E.18)
\]
\[
\frac{13,100}{20,000} + \frac{3,880}{14,400} = 0.93 < 1.0 \quad (E.19)
\]

Since this expression holds true, the structure is sufficient to hold the applied loads.

Therefore, the final design of the ramp consists of a base and top plate with three evenly spaced triangular stiffeners, as shown in Figure E-6. All material is 0.5 in (1.3 cm) thick steel plate.

![Figure E-6 Angled Actuator Wedge](image)

**E.3.4 BEAM CONNECTIONS**

The design of the beam connections to the concrete block was challenging in that very large loads will be applied at an angle. Not only is there a y-component of the force that will create a moment, but also an x-component which acts as a sliding force across the front of the concrete. To resist against the y-component and resulting moment, a wedge and a side plate were designed. The wedge attaches to the concrete and the beam directly behind the actuator pushing point. The side plate is bolted to the concrete on the opposite side, as shown in Figure E-7.
E.3.4.1 WEDGE DESIGN

The wedge is similar to the ramp in that there are two base plates with triangular stiffeners in between. The base plates are 15 in (38 cm) wide and 14 in (36 cm) in depth. An initial thickness of 0.5 in (1.3 cm) was selected. Figure E-8 depicts the design.
The source of load that affects the wedge is the moment caused by the y-component of the actuator. As calculated earlier, that moment is 208,000 lb-ft (282 kN-m). This load can be distributed across the face of the concrete block, which has a length of 4 ft (1 m). The resulting force, $P_w$, is the compression force that acts on the wedge:

$$P_w = \frac{208,000}{4} = 52,000 \text{ lb (230 kN)} \quad (E.20)$$

Since there are a total of four stiffener plates, the load is distributed between them. Thus, the tributary load on the innermost plates is equal to:

$$P_p = \frac{52,000}{3} = 17,300 \text{ lb (77.0 kN)} \quad (E.21)$$

The outer plates carry half this amount. A buckling check is conducted on one of the inner plates because this represents the worst case scenario. A representation of the buckling plane is represented in Figure E-9.

![Figure E-9 Plan View of Wedge Showing a 1 in (2.5 cm) Strip for Wedge Buckling Calculations](image-url)
The load per inch on this strip is equal to:

\[
P_{ps} = \frac{P_p}{l \sin 45} = \frac{17,300}{15 \sin 45} = 1,630 \text{ lb/in (2,860 N/cm)} \tag{E.22}
\]

With this load, the buckling stresses can be calculated as before:

\[
I = \frac{(l)(0.5)^3}{12} = 0.0104 \text{ in}^4 (0.433 \text{ cm}^4) \tag{E.23}
\]

\[
A = (0.5)(l) = 0.5 \text{ in}^2 (3.2 \text{ cm}^2) \tag{E.24}
\]

\[
r = \sqrt{\frac{0.0104}{0.5}} = 0.144 \text{ in (0.366 cm)} \tag{E.25}
\]

\[
\frac{kl}{r} = \frac{(l)(21.2)}{0.144} = 147 \tag{E.26}
\]

where 21.2 in (53.8 cm) is the length of the hypotenuse of the 15 in by 15 in (38 cm by 38 cm) triangle. Since the same type of steel is used again as before, the critical slenderness ratio is equal to 126. In this case, the slenderness ratio is larger than this limit, so the allowable stress equation for buckling is used:

\[
F_a = \frac{12\pi^2 E}{23(47)^2} = \frac{12\pi^2 (29 \times 10^6)}{23(47)^2} = 6,910 \text{ psi (4,770 N/cm}^2) \tag{E.27}
\]

The actual stress on the buckling plane of the wedge is:

\[
f_a = \frac{P_{ps}}{A} = \frac{1,633}{0.5} = 3,270 \text{ psi (2,250 N/cm}^2) \tag{E.28}
\]

Since \(f_a<F_a\), the stiffeners will be sufficient to hold the loads. The margin of safety is calculated by:

\[
M.S. = \frac{F_a}{f_a} - 1 = \frac{6,910}{3,270} - 1 = 1.11 \tag{E.29}
\]
The base plates were also checked to make sure the 0.5 in (1.3 cm) thickness of the steel was sufficient. The critical section of the plate is where the bolts are located, since the holes weaken the plate. The tributary load to the center row of bolts, $P_b$ is half of the total force on the plate:

$$P_b = P_w/2 = 26,000 \text{ lb (120 kN)} \quad (E.30)$$

The actual force on the plate is equal to the force over the area that it acts. In the case of the steel plate, the area is calculated taking bolt holes into account:

$$A = \left(15 - \frac{7}{8} - \frac{7}{8} \frac{1}{2}\right) = 6.6 \text{ in}^2 (43 \text{ cm}^2) \quad (E.31)$$

The actual force on the steel plate, $P_{plate}$, is equal to:

$$P_{plate} = \frac{26,000}{6.6} = 3,920 \text{ psi (2,700 N/cm}^2\text{)} \quad (E.32)$$

The allowable stress, which was calculated in the previous wedge design, is equal to 14,400 psi (9,930 N/cm$^2$). The 0.5 in (1.3 cm) thick plate is more than enough to support the loads on the wedge. The margin of safety is equal to:

$$M.S. = \frac{14,400}{3,920} - 1 = 2.67 \quad (E.33)$$

The final aspect of the wedge design was the type and number of bolts needed to secure the beam to the concrete. As calculated previously, the design load for the structure is equal to 52,000 lb (230 kN). Two different options were considered: epoxy bolts or concrete anchors. Study of an ICBO evaluation report on the two systems showed that the epoxy bolts were superior in strength and would support the beam with fewer bolts. Hilti HY-150 adhesive epoxy bolts were selected. The initial sizing used A-325 bolts or
ASTM A-193 grade B-7 rods with a diameter of 0.75 in (1.9 cm). The embedment was 6.63 in (16.8 cm), with an 8 in (20 cm) edge distance and 5 in (13 cm) spacing. The concrete strength of the block at the site was conservatively assumed to be 4,000 psi (3,000 kN/cm²).

The ICBO evaluation report for the Hilti HY-150 Adhesive Anchor Systems gives a table of allowable values and requirements for these allowables. The allowable force per bolt in shear for an A-325 bolt or an ASTM A-193 grade B-7 rod with 0.75 in (1.9 cm) diameter, 6.63 in (16.8 cm) embedment, 10.0 in (25.4 cm) edge distance, and 13.3 in (33.7 cm) spacing is 8060 lb (35.9 kN). This value, which is based on the concrete strength, was chosen over the less conservative allowable based on steel strength. The allowable of 8,060 lb (35.9 kN), however, must be adjusted for a decrease in spacing and edge distance. The allowable load for A-325 bolts in shear is 7,510 lb (33.4 kN), and governs if the reduced allowable based on the concrete strength is larger than the allowable load for the ASTM A-193 grade B-7 rods.

Table 3 in the ICBO report gives reduction factors for reduced spacing and edge distance in normal-weight and light-weight concrete. Since the applied load is directed away from the closest edge of the plate, the reduction factor for spacing is 1.0. In other words, no reduction is necessary. There is a reduction for the edge distance, however. According to Table 3, the minimum edge distance is equal to 33% of the required edge distance, $e$, which, in this case, is 10 in (25 cm):

$$e_{\min} = 0.33e = 3.3 \text{ in (8.4 cm)}$$

(E.34)
The edge distance reduction factor for bolts in shear when the load is directed away from the edge is 0.6. Since the edge distance of 8 in (20 cm) is greater than the minimum, but less than the required, the factor can be interpolated:

\[
\text{Edge Reduction} = \left( \frac{8 - 3.3}{10 - 3.3} \right) \times 0.4 + 0.6 = 0.88 \quad (E.35)
\]

The new allowable load is equal to:

\[
V = (8,060)(0.88) = 7100 \text{ lb (32 kN)} \quad (E.36)
\]

The wedge works together with the side plate to secure the beam to the concrete block. To balance the load between the two, six bolts were selected to secure the wedge. The calculated strength that these bolts provide was reduced to 80%. This ensures that the load transfers over to the side plate before the wedge bolts fail. The strength provided by the bolts in the concrete, \( F_b \), is equal to the number of bolts multiplied by the individual strength. The first column of bolts provide the reduced allowable of 7,100 lb (32 kN) due to the 8 in (20 cm) edge distance. Because the second column of bolts is located greater than 10 in (25 cm) from the edge of the concrete, no reduction is necessary. The decision had not been made whether to use the A-325 bolts or the ASTM A-193 grade B-7 rods, so the shear capacity for the A-325 bolts of 7,510 lb (33.4 kN) was used to be conservative:

\[
F_b = (7,098)(3) + (7,510)(3) = 43,800 \text{ lb (195 kN)} \quad (E.37)
\]

\[
0.8F_b = (0.8)(43,824) = 35,100 \text{ lb (156 kN)} \quad (E.38)
\]

The design force for the side plate, \( F_{sp} \), is equal to the total force in the concrete, \( P_w \), minus the resistance provided by the wedge bolts, \( F_b \):
\[ F_{sp} = 52,000 - 35,100 = 16,900 \text{ lb (75.1 kN)} \]  

(E.39)

Therefore, the side plate must be designed to support 16,999 lb (75.1 kN) of load.

E.3.4.2 Side Plate Design

The side plate is located at the edge of the concrete furthest from the applied load and will work with the wedge to attach the steel beam to the concrete block. The plate’s dimensions are 14.8 in (37.5 cm) high by 17 in (43 cm) wide by 0.5 in (1.3 cm) thick. Similarly to the wedge, the thickness of the plate was checked for strength:

\[ P_{sp} = \frac{52,000/2}{(14.75 - 7/8 - 7/8 - 7/8)(0.5)} = 4,280 \text{ psi (2,950 N/cm}^2) \]  

(E.40)

This is much less than the allowable value of 14,400 psi (9,928 N/cm\(^2\)) and is sufficient to withstand the anticipated loads. The margin of safety is equal to:

\[ M.S. = \frac{14,400}{4,280} - 1 = 2.36 \]  

(E.41)

As determined in the previous section, the bolts in this plate need to be able to resist 16,900 lb (75.1 kN) of force. For this application, Hilti HY-150 adhesive epoxy bolts are also be used. Once again, 0.75 in (1.9 cm) diameter bolts with 6.63 in (16.8 cm) embedment are chosen, and while the spacing is taken again as 5 in (13 cm) the edge distance is increased to 10 in (25 cm). This time, the force is directed towards the edge of the concrete, and so a reduction is applied to account for the decreased spacing. Since the full 10 in (25 cm) edge distance is used this time, there is no reduction. The minimum spacing is equal to 25\% of the required value:

\[ s_{min} = 0.25s = (0.25)(13.2) = 3.3 \text{ in (8.4 cm)} \]  

(E.42)
Interpolating once again, the reduction factor necessary for this case is 0.75:

\[
\text{Spacing Reduction} = \left( \frac{5 - 3.3}{13.3 - 3.3} \right) \times 0.3 + 0.7 = 0.75 \quad (E.43)
\]

The allowable force per bolt therefore is:

\[
V_{allow} = (0.75)(8,060) = 6,050 \text{ lb (26.9 kN)} \quad (E.44)
\]

The number of bolts necessary is equal to:

\[
N_{bolts} = \frac{16,900}{6,050} = 3 \text{ bolts} \quad (E.45)
\]

Therefore, at least three bolts must be used at the end plate to carry the remainder of the force.

E.3.4.3 SLIDING FORCE RESISTANCE

The final design consideration was to resist the sliding force imposed on the beam due to the x-component of the initial force by bolting the beam to the front face of the concrete block. As calculated in Section 6.3.2, the sliding force, \( F_x \), is equal to 56,700 lb (252 kN). Hilti HY-150 epoxy bolts are used once again. This time, however, the bolts are 1.0 in (2.5 cm) in diameter with an embedment of 8.25 in (21.0 cm). The edge distance and spacing requirements for this set are 12.5 in (31.8 cm) and 16.5 in (41.9 cm) respectively. For this case, an edge distance of 11 in (28 cm) and spacing of 8 in (20cm) will be used. Three columns of two bolts was the initial decision before checking. The load is directed toward the edge of the concrete, so a reduction for spacing is required and the first column of bolts must also be reduced for edge distance. The minimum edge distance is: 4.12 in (10.5 cm) and the minimum spacing is the same:

\[
s_{\text{min}} = 0.25s = 4.12 \text{ in (10.5 cm)} \quad (E.46)
\]
\[ e_{min} = 0.33e = 4.12 \text{ in (10.5 cm)} \]  

(E.47)

Interpolating gives the reduction factors for spacing and edge distance as 0.79 and 0.85, respectively:

\[
\text{Spacing Reduction} = \left( \frac{8 - 4.13}{16.5 - 4.13} \right) \times 0.3 + 0.7 = 0.79 \quad \text{(E.48)}
\]

\[
\text{Edge Reduction} = \left( \frac{11 - 4.13}{12.5 - 4.13} \right) \times 0.8 + 0.2 = 0.85 \quad \text{(E.49)}
\]

The allowable load per bolt before reductions for a 1.0 in (2.5 cm) Hilti HY-150 epoxy bolt is 12,800 lb (57.1 kN). The total strength provided by this bolt setup is:

\[ F_{sf} = (2)(12,800)(0.79)(0.85) + (4)(12,800)(0.79) = 58,000 \text{ lb (260 kN)} \quad \text{(E.50)}
\]

The actual load of 56,670 lb (252 kN) is less than the allowable, so the beam will be able to handle the sliding force. The margin of safety is equal to:

\[ M.S. = \frac{58,000}{56,670} - 1 = 0.02 \quad \text{(E.51)}
\]

In summary, the beam is connected to the concrete block using a system of three connections. The wedge is fastened to the concrete using six 0.75 in (1.9 cm) diameter Hilti HY-150 Epoxy bolts. The bolts have a 6.63 in (16.8 cm) embedment, 8 in (20 cm) edge distance and 5 in (13 cm) spacing. The side plate is attached using three 0.75 in (1.9 cm) diameter Hilti HY-150 Epoxy bolts. The bolts have a 6.63 in (16.8 cm) embedment, 10 in (25 cm) edge distance and 5 in (13 cm) spacing. The front face of the beam is attached using 1.0 in (2.5 cm) diameter Hilti HY-150 epoxy bolts. These bolts have an embedment of 8.25 in (21.0 cm), edge distance of 11 in (28 cm), and spacing of 8 in (20 cm).
cm). Table E-1 summarizes the margins of safety for the design of the beam connections.

<table>
<thead>
<tr>
<th>Component</th>
<th>Load Type</th>
<th>Actual Load [psi (N/cm²)]</th>
<th>Allowable Load [psi (N/cm²)]</th>
<th>Margin of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wedge</td>
<td>Plate Strength</td>
<td>3,920 (2,700)</td>
<td>14,400 (9,930)</td>
<td>2.67</td>
</tr>
<tr>
<td></td>
<td>Buckling</td>
<td>3,270 (2,250)</td>
<td>6,910 (4,770)</td>
<td>1.11</td>
</tr>
<tr>
<td>Side Plate</td>
<td>Plate Strength</td>
<td>4,280 (2,950)</td>
<td>14,400 (9,930)</td>
<td>2.36</td>
</tr>
<tr>
<td>Front Face</td>
<td>Sliding Resistance</td>
<td>56,670 (252)</td>
<td>58,000 (260)</td>
<td>0.02</td>
</tr>
</tbody>
</table>

A summary of the pertinent loads and resisting forces is shown in Figure E-10.