Manufacturing Quality of Carbon/Epoxy IsoTruss (R) Reinforced Concrete Structures

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MANUFACTURING QUALITY OF CARBON/EPOXY ISOTRUSS® REINFORCED CONCRETE STRUCTURES

by

David T. McCune

A thesis submitted to the faculty of

Brigham Young University

in partial fulfillment of the requirements for the degree of

Master of Science

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BRIGHAM YOUNG UNIVERSITY

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ABSTRACT

MANUFACTURING QUALITY OF CARBON/EPOXY ISOTRUSS® REINFORCED CONCRETE STRUCTURES

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Department of Civil and Environmental Engineering

Master of Science

This thesis explores the quality of hand-manufactured carbon-epoxy IsoTruss® grid structures for use as reinforcement in concrete piles. Large IsoTruss® grid structures were manufactured and embedded in 14.0” (35.6 cm) diameter concrete to create IsoPiles™. The IsoPiles™ were designed to have flexural characteristics similar to steel reinforced concrete piles of equal diameter. Bending stiffness was matched based on the longitudinal members. A method for comparing transverse steel reinforcement to helical IsoTruss® members was developed, along with equations to facilitate the design of IsoTruss® structures with rounded nodes.

Compression tests were performed on 3.0 ft (0.91 m) long sections taken from the ends of each of the two 30 ft (9.14 m) long IsoTruss® grid structures manufactured. Fiber volume fraction, void fraction, and cross section area inspections were performed on IsoTruss® samples to determine quality. The strength, stiffness, and fiber volume
fraction data obtained from these tests are compared to values obtained previously [1] for the same consolidation method. The quality of hand-manufactured large IsoTruss® grid structures was quantified by performing microscopic inspection of the members, by testing the reinforcement cage in compression, and by testing short section of IsoTruss® and steel reinforced concrete piles in compression. Compression tests were performed on short sections taken from the ends of the IsoPile™ specimens. These were compared with compression tests performed on equivalent steel-reinforced piles to evaluate the viability of the IsoTruss® as reinforcement in concrete piles.

Insufficient tension on the fiber during manufacturing and insufficient radial compression during the cure resulted in an average fiber volume fraction 13% lower than previously obtained, causing the ultimate compressive strength and Young’s modulus of the IsoTruss® reinforcement cages to be 51% and 22% lower, respectively, than previous data. The IsoTruss®-reinforced piles had an ultimate compressive load that was within 4% of the ultimate compressive load of the steel-reinforced piles.
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CHAPTER 1 – INTRODUCTION

This thesis documents the quality of hand-manufactured carbon-epoxy IsoTruss® grid structures for use as reinforcement in concrete piles. Throughout this document, these piles are referred to as IsoPiles™. Large diameter IsoTruss® grid structures were manufactured for use in IsoPiles™. The IsoTruss® reinforcement was designed to compare to steel reinforced concrete piles of equal diameter. Bending stiffness was matched based on the longitudinal members. A method for comparing transverse steel reinforcement to helical IsoTruss® members was developed. Also, equations for the design of IsoTruss® structures with rounded nodes were derived.

This introductory chapter describes the IsoTruss® grid structure and discusses previous research performed on the IsoTruss® that is relevant to this research. Next, corrosion of concrete reinforcement is discussed, explaining the benefit of polymer composites as concrete reinforcement. Lastly, the scope of investigation of this research is defined.

Two main investigations determined the quality of the manufactured specimens. First, compression tests were performed on 3 ft (0.9 m) long sections taken from the ends of each of the two IsoTruss® grid structures. Second, fiber volume fraction inspections were performed to determine compaction quality. The strength, stiffness, and fiber volume fraction data obtained from these tests are compared to the values obtained by
Hansen [1] for the same consolidation method in order to quantify the quality of hand-manufactured large IsoTruss® grid structures.

Flexural testing of the shorter IsoPile™ specimens will demonstrate viability of the IsoTruss® as reinforcement in concrete piles. Maximum moment will be determined from four-point bending tests and compared to identical tests of steel reinforced concrete piles (see Ferrell [25]).

1.1 DESCRIPTION OF ISOTRUSS® STRUCTURES

The IsoTruss® is a patented composite grid structure (see Figure 1.1). The IsoTruss® has been the subject of extensive research due to its recognized high strength-to-weight and high strength-to-stiffness ratios. The geometry of the IsoTruss® is such that maximum load can be resisted by minimum material. The specimens in this research were manufactured using carbon-epoxy, but other fibers have been used including E-glass, aramid, and basalt (see Scoresby [2]).

![Figure 1.1 Isometric View of 8-Node IsoTruss® Grid Structure](image)
In an IsoTruss®, the longitudinal members run parallel to the long axis of the structure (x-axis in Figure 1.1). The longitudinal members primarily resist structural axial and bending loads. Helical members spiral around the structure, intersecting the longitudinal members at regular intervals. The helical members serve as bracing for the longitudinal members, shortening the buckling length. The helical members also resist the shear and torsion loads. The exact geometry of the structure depends on the truss diameter, number of nodes around the circumference, bay length (longitudinal distance between nodes), and member diameters. A detailed description of the geometry of the IsoTruss® with defining equations has been documented in Winkel [3].

1.2 PREVIOUS RESEARCH

Extensive research has been performed on IsoTruss® structures. Axial strength tests were performed by Weaver [4], McCune [5], and Rackliffe [6]. Weaver [4] manufactured IsoTruss® grid structures using Akzo Nobel-Fortafil® 3C continuous carbon fiber and Shell Epon® 826 resin. The longitudinal members were 0.147 in (0.373 cm) in diameter. The specimens were consolidated under combined vacuum pressure with a flexible elastomeric internal mandrel. The specimens were tested in compression, tension, and torsion. Weaver achieved average compressive and tensile strengths of 40.8 ksi (281 MPa) and 44.3 ksi (306 MPa), respectively. The average torque resisted by Weaver’s larger specimens was 4.8 kip-in (543 N-m).

McCune [5] manufactured 8-node, 5 inch (12.7 cm) diameter IsoTruss® grid structures using 12K T300 C 200 NT carbon fiber and Shell Epon® 826 resin. The largest specimen had a member diameter of 0.173 in (0.441 cm). The members in these
specimens were compacted by hand tying with Kevlar® fiber. These specimens had a fiber volume fraction 14% lower than those manufactured by Weaver [4], who achieved an average fiber volume fraction of about 56%. The average ultimate compressive stress value for McCune’s specimens was 33.0 ksi (228 MPa), 74% of Weaver’s. The compression specimens exhibited both crushing and local buckling of the longitudinal members as failure modes. The ultimate tensile stress, however, was not able to be measured. The tension specimens experienced premature failure at the ends due to kinking of the fibers.

Rackliffe [6] fabricated extremely lightweight and delicate IsoTruss™ grid structures designed for space applications. The largest specimen had a longitudinal member diameter of 0.12 in (0.32 cm). Rackliffe’s specimens were made using IM7 6K tow carbon fiber pre-impregnated with Thiokol UF3325-95 epoxy resin. The specimens were consolidated by hand tying with Kevlar® fiber. All of Rackliffe’s specimens exhibited buckling failure modes.

Other research related to this thesis was conducted by Keller [7]. Keller performed bending, torsion, and tension tests on small diameter IsoTruss® grid structures that were pressure consolidated on a water-soluble mandrel. The largest structure had a longitudinal member area of 0.0043 in² (0.029 cm²). This translates into an equivalent diameter of 0.08 in (0.19 cm) for a circular cross-section. Due to the consolidation method, however, the longitudinal members were flattened. These structures transitioned to a cylindrical structure at each end instead of winding around an endplate. Aluminum plugs were pressure fit in each cylinder to serve as loading points. This eliminated the fiber kinking problem in McCune [5]. Keller’s grid structures had an average tensile
strength of 245 ksi (1,691 MPa), and a maximum moment capacity of 2,117 lb-in (239 N-m). The average maximum torque of Keller’s specimens was 959 lb-in (108 N-m).

Tests of composite reinforced concrete were performed by Earl [8], Hancock[9], Blake [10], Jones [11], and Jarvis [12]. Earl [8] performed tests on large concrete cylinders that were reinforced longitudinally by pultruded fiberglass rods, and circumferentially by carbon fiber rings. Testing different containment percentages and patterns determined that containment can significantly delay failure in composite reinforced concrete design. Spacing of the circumferential (hoop) reinforcement should be a major design consideration, along with strength of the circumferential reinforcement. This approach was applied in this research.

Hancock [9] studied external reinforcement of concrete columns. Pultruded fiberglass staves were positioned around the outside of concrete cylinders and wrapped with carbon fiber for circumferential reinforcement. The composite reinforcement increased the axial compressive strength of concrete up to 250%. Just as in Earl [8], the strength was a function of the number of staves (longitudinal reinforcement) and the distribution of the circumferential reinforcement.

Blake [10] tested IsoTruss® reinforced concrete beams in compression and flexure. Due to fabrication problems, the compressive strength of the IsoTruss® reinforced specimens was approximately equal to that of the plain concrete specimens. In flexure, the IsoTruss® reinforced specimens resisted an ultimate load 24.5 kips (109 kN), which was about twice that of steel reinforced specimens with equivalent stiffness.

Jones [11] also tested IsoTruss® reinforced concrete beams in flexure. These tests showed that IsoTruss® reinforced concrete has a ductile failure mode, which is
traditionally preferred in concrete design. Jarvis [12] developed a rectangular IsoTruss®
grid structure to reinforced concrete beams. The rectangular IsoTruss® reinforced
concrete beams were 55% stronger in flexure than the standard 6-node IsoTruss®
reinforced concrete beams tested by Jones (see Jarvis [12]).

Hansen [1] performed compression tests on carbon-epoxy members specially
manufactured to simulate a longitudinal member of the IsoTruss®. Specimens were
manufactured with and without joints using various consolidation methods. A
microscopic examination was performed to determine fiber volume fraction. This
research determined that consolidation of specimens using a sleeve provides consistent
consolidation and results in high strength and stiffness. Local compressive strength of a
composite IsoTruss® structure is directly related to the straightness of the longitudinal
member tows at the joints.

The investigation discussed in this thesis examines the quality of large, hand-
manufactured IsoTruss® grid structures. Tests performed on IsoPiles™ address the
viability of IsoTruss®-reinforced concrete piles. Axial compressive strength and
stiffness are calculated. The results can be used to design IsoTruss® structures for
reinforcement in concrete structures.

1.3 CORROSION

Corrosion of steel is a major problem in reinforced concrete. Corrosion occurs
when the passive film on steel breaks down due to chloride attack or carbonation (see
Qian [13]). The products of corrosion cause induced stresses in the concrete which
eventually exceed its tensile strength. Under these conditions, the concrete will begin to
spall off. This can result in a loss of bond and reduction in structural strength (see Stewart [14]).

Because of rising construction costs, engineers are constantly looking for new materials that will increase the design life of structures. Many state highway agencies have set a design life goal of 100 years on larger highway bridges (see Concrete Construction [15]). In order to meet such goals, a more corrosion resistant reinforcement must be used. Fiber reinforced polymer composite materials appear to be an ideal solution. Polymer composites have a high corrosion resistance, especially compared to steel (see Cosenza [16]). The IsoTruss® grid structure has demonstrated a very high strength-to-weight ratio. This structural efficiency combined with the incredible corrosion resistance of polymer composite materials makes the IsoTruss® an attractive alternative to steel reinforcement.

1.4 SCOPE OF INVESTIGATION

The investigation detailed in this document determines the quality of hand-manufactured, carbon-epoxy IsoTruss® grid structures for use as reinforcement in concrete piles. The IsoTruss® specimens investigated for this research have the largest member diameter manufactured to date. The design diameter of the helical members was 0.35 in (0.88 cm), while the design diameter of the longitudinal members was 0.42 in (1.07 cm). The members are large in order to have the same flexural stiffness as standard #4 steel reinforcement. New design features were incorporated into the IsoTruss® specimens, including more substantial rounding of the nodes. Also, a technique for
comparing the helical member strength of IsoTruss® grid structures to steel hoops in reinforcement cages was developed.

Two 30 ft (9.14 m) long IsoTruss® reinforcement cages were fabricated. One was cut in half to create two 13.25 ft (4.04 m) long specimens. These IsoTruss® structures were embedded in concrete to create three IsoPiles™. Steel reinforced concrete piles of the same lengths were also manufactured for comparison. All piles were 14.0 in (35.6 cm) in diameter.

The steel reinforcement was designed to match the estimated stiffness of the IsoTruss® reinforcement. The piles were manufactured to be tested in flexure. The shorter piles were manufactured for laboratory tests in a four-point bending fixture. The longer piles were manufactured for in-situ field testing.

Short sections, approximately 3 ft (0.9 m) long, were cut off the ends of each IsoTruss® structure and tested in pure axial compression. The cross-sections of pieces of the IsoTruss® removed while making these cuts were polished and investigated under a microscope to determine the cross-sectional area, void content, and fiber volume fraction. The results of the microscopic examination and the data from the compression tests of the IsoTruss® reinforcement quantified the quality of the hand-manufacturing process used for these specimens. Short sections, 3 ft (0.9 m) long, were also cut off each end of the short IsoPile™ specimens after the laboratory bending tests and tested in pure axial compression.
CHAPTER 2 – EXPERIMENTAL APPROACH

The experimental approach for this research is outlined in this chapter. First, a description of the experiment is detailed. Next the specimens manufactured for this research are described followed by the specific tests performed on the specimens. Lastly, test matrices presenting the tests to be performed on the specimens manufactured for this research are presented.

2.1 DESCRIPTION OF EXPERIMENT

Test specimens were manufactured to demonstrate the use of hand-manufactured IsoTruss® grid structures as reinforcement in concrete piles. To assess the manufactured quality, short sections, about 3 ft (0.9 m) long, of IsoTruss® (IT) structure were tested in pure axial compression. Other IsoTruss® structures were encased in concrete to create IsoTruss®-reinforced concrete (IRC) piles, referred to as IsoPiles™. Steel (rebar) reinforced concrete (SRC) piles were manufactured for comparison to the IRC specimens. Short sections, about 3 ft (0.9 m) long of both the IRC and the SRC specimens were tested in pure axial compression. Longer IRC and SRC specimens were manufactured for bending tests in the lab (see Ferrell [24]) and in the field (see Richardson [25]). During the manufacturing process, sections of the longitudinal members of the IsoTruss® reinforcement were polished and examined under a microscope to determine
the consolidation quality. A more detailed description of the specimens and their corresponding tests is in the following sections.

2.2 DESCRIPTION OF SPECIMENS

IsoTruss®, IsoPile™, steel reinforced pile and steel rebar specimens were tested. Sections of the longitudinal members of the IsoTruss® were also examined under a microscope. The specimen parameters are described in the following sections.

2.2.1 IsoTruss® Specimens (IT)

The 8-node IsoTruss® grid structures manufactured for this research are composed of 12k T-300 carbon fiber tows pre-impregnated with TCR UF3325 resin. There are 133 carbon fiber tows in the longitudinal members and 89 tows in the helical members. Based on the manufacturer’s specifications, the predicted longitudinal member diameter was 0.42 in (1.1 cm) and the predicted helical member diameter was 0.35 in (0.9 cm). The nodes of the specimens were rounded (see Figure 2.1) and the outer diameter was nominally 13.4 in. (34.0 cm). The IsoTruss® (IT) specimens are approximately 33 in (84 cm) long with a bay length of 7.44 in. (18.9 cm). Table 2.1 details the measured parameters of the IsoTruss® compression specimens. The minor variations between the two specimens are due to inaccuracies in the hand-manufacturing process.
Figure 2.1 Isometric View of IsoTruss® Reinforcement Section with Rounded Nodes

Table 2.1 IsoTruss® Reinforcement Specimen Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>IT-1</th>
<th>IT-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall Diameter, D</td>
<td>13.5 in (34.3 cm)</td>
<td>13.5 in (34.3 cm)</td>
</tr>
<tr>
<td>Bay Length, L_b</td>
<td>7.44 in (18.9 cm)</td>
<td>7.44 in (18.9 cm)</td>
</tr>
<tr>
<td>Design Helical Area, A_H</td>
<td>0.094 in² (0.604 cm²)</td>
<td>0.094 in² (0.604 cm²)</td>
</tr>
<tr>
<td>Design Longitudinal Area, A_L</td>
<td>0.140 in² (0.903 cm²)</td>
<td>0.140 in² (0.903 cm²)</td>
</tr>
<tr>
<td>Actual Longitudinal Area, A_L</td>
<td>0.181 in² (1.171 cm²)</td>
<td>0.181 in² (1.171 cm²)</td>
</tr>
<tr>
<td>Longitudinal Net Area</td>
<td>0.163 in² (1.052 cm²)</td>
<td>0.163 in² (1.052 cm²)</td>
</tr>
<tr>
<td>Design Axial Stiffness, EA</td>
<td>25,554 kip (113,670 kN)</td>
<td>25,554 kip (113,670 kN)</td>
</tr>
<tr>
<td>Length, L</td>
<td>32.75 in (83.19 cm)</td>
<td>32.50 in (82.55 cm)</td>
</tr>
<tr>
<td>Weight (without caps)</td>
<td>7.54 lb (3.42 kg)</td>
<td>7.50 lb (3.40 kg)</td>
</tr>
<tr>
<td>Weight (with caps)</td>
<td>25.2 lb (11.43 kg)</td>
<td>26.0 lb (11.80 kg)</td>
</tr>
</tbody>
</table>
2.2.2 IsoTruss®-Reinforced Concrete (IRC) Specimens

IsoTruss® structures were encased in self-consolidating concrete with a manufacturer’s specified design strength of 12.0 ksi (82.7 MPa) to form IsoPile™ specimens. The parameters of the IsoTruss® structures used as reinforcement in the IsoPiles™ are identical to the IT specimens, except for length. The IRC compression specimens were not of equal lengths due to the process used to cut the piles (see Table 2.2)

<table>
<thead>
<tr>
<th>Sample</th>
<th>Length [in (cm)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>IRC-1</td>
<td>32.9 (83.5)</td>
</tr>
<tr>
<td>IRC-2</td>
<td>32.8 (83.3)</td>
</tr>
<tr>
<td>IRC-3</td>
<td>33.1 (84.0)</td>
</tr>
<tr>
<td>IRC-4</td>
<td>33.1 (84.1)</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>33.0 (83.8)</strong></td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>0.2 (0.4)</td>
</tr>
<tr>
<td></td>
<td>0.5%</td>
</tr>
</tbody>
</table>

The IRC specimens manufactured for the lab bending test were 13.4 ft (6.2 m) long, while the specimen manufactured for the field bending test was 30 ft (9.1 m) long. The IsoTruss® reinforcement in the lab bending test IsoPiles™ extended through the entire length of the specimens. The IsoTruss® reinforcement for the field bending test IsoPile™ was 26.9 ft (8.2 m) long. A steel reinforcement section employing the same
size bars described in Sub-Section 2.1.2 was spliced onto what would be the bottom end of the IsoTruss® reinforcement. The total length of the IsoTruss® and steel reinforcement extended through the entire length of the field bending test specimen. The diameter of all IRC specimens was 14.0 in (35.6 cm).

2.2.3 Steel-Reinforced Concrete (SRC) Specimens

Steel-reinforced piles were manufactured to compare to the IsoPiles™. Eight #4 steel reinforcing bars were used for the longitudinal reinforcement. Concrete cover on the longitudinal bars was 2.5” (6.4 cm). Transverse reinforcement was composed of steel circular ties (see Figure 2.2) spaced at one half the bay length of the IsoPiles™, or 3.72 in. (9.45 cm). The ties are #2 reinforcing bar. The steel reinforcement extended through the entire length of all SRC specimens. The SRC specimens were manufactured to be the same lengths as the IRC specimens. The lengths of the SRC compression specimens varied slightly (see Table 2.3).

![Figure 2.2 Transverse Steel Reinforcing Bar for Steel Reinforced Concrete Pile Specimens](image-url)
Table 2.3 Lengths of SRC Compression Specimens

<table>
<thead>
<tr>
<th>Sample</th>
<th>Length [in (cm)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>SRC-1</td>
<td>32.3 (81.9)</td>
</tr>
<tr>
<td>SRC-2</td>
<td>32.9 (83.5)</td>
</tr>
<tr>
<td>SRC-3</td>
<td>33.1 (84.0)</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>32.8 (83.2)</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Standard Deviation</th>
<th>0.4 (1.1)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.3%</td>
</tr>
</tbody>
</table>

The lab bending test SRC specimens were 13.4 ft (6.2 m) long, and the field bending test SRC specimen was 30 ft (9.1 m) long. The diameter of all SRC specimens was 14.0 in (35.6 cm). The lap splices of the transverse steel required by the American Concrete Institute (see MacGregor [19]) give an advantage to the steel reinforcement by adding extra circumferential containment strength that the IsoTruss® reinforcement doesn’t have. A full explanation of the comparison of transverse steel reinforcement to helical members of the IsoTruss® is detailed in Chapter 3.

2.2.4 Quality Control (QC) Specimens

Six sections of the longitudinal members of the IsoTruss® structures were obtained during manufacturing of the other specimens. One end of each of the QC specimens was encased in resin and polished for viewing under a microscope. These microscopic investigations enabled measurements of the cross-sectional areas, the fiber volume fraction, and the void fraction.
2.3 DESCRIPTION OF STRENGTH AND STIFFNESS TESTS

Simple compression strength and stiffness tests were performed on the IsoTruss®, IsoPile™, and steel-reinforced pile compression specimens. Bending tests were performed on IsoPile™ and steel-reinforced pile specimens. The compression and bending tests are described in this section.

2.3.1 Compression Strength Tests

Compression tests were performed on the IsoTruss® (IT) grid structure specimens in the Baldwin 300,000 lb (1,330 kN) capacity testing machine shown in Figure 2.3. The ends of the specimens were encased in 1.88 in. (4.8 cm) thick, chopped carbon fiber reinforced vinyl ester resin rings. The caps were added to prevent brooming at the ends of the longitudinal members. A swivel head assembly was used to partially compensate for the ends not being perfectly parallel. An 18.0 in (45.7 cm) by 0.5 in (1.3 cm) thick steel plate weighing 44.6 lb (198.5 N) was placed on top of the upper resin cap to transfer load from the swivel head to the specimen.

Nine surface-mounted strain gages were used to collect strain data and help identify local failure modes. Four linear motion transducers were placed around the specimens to measure axial deflection. The exact location and type of instrumentation is described more fully in Chapters 4 and 5.
Simple axial compression tests were also performed on the 33 in (84 cm) long IRC and SRC specimens using a self-reacting steel frame (see Figure 2.4). The frame incorporated a 600,000 lb (2,700 kN) capacity actuator. Polychloroprene (neoprene) pads were used in lieu of vinyl ester resin caps. Some of the specimens were equipped with
four internal strain gages, and all of them had two surface strain gages bonded to the concrete. Axial deflection was measured by two linear motion transducers. Results of the compression tests are presented in Chapter 5.

![Test Fixture for Axial Compression Tests of IRC and SRC Specimens](image)

**Figure 2.4** Test Fixture for Axial Compression Tests of IRC and SRC Specimens

2.3.2 Bending Test in Lab

Four-point bending tests were performed in the lab on IsoPile™ and steel reinforced pile specimens. These specimens are about 13.25 ft (4.04 m) long (see Chapter 3). It was therefore logical to make the test length 12.0 ft (3.66 m). This leaves 4.0 ft. (1.22 m) between the supports and the loads (see Figure 2.5). Steel saddle pieces were machined for the load points and the supports in order to prevent slippage. The results of this test are reported in a separate document (see Ferrell [24]).
2.3.3 Bending Test in Field

The two 30 ft. (9.14 m) long pile specimens were tested in the field. One is an IsoPile™ and the other is a steel reinforced concrete pile (see Figure 2.6). The piles were driven into the ground at a site in Salt Lake City, Utah, leaving the top 2 ft (0.6 m) of pile length exposed. The results of these bending tests are reported in Ferrell [24].
2.4 DESCRIPTION OF QUALITY CONTROL TESTS

The cross-sections of the QC specimens were analyzed under two microscopes to determine the consolidation quality of the IsoTruss® structures manufactured for this research. The area, fiber volume fraction, and void fraction measurements are described in the following sections.

2.4.1 Area Measurements

The nominal cross-sectional area was estimated using the fiber area values for 12k T-300 carbon fiber. The actual area was measured using the IA-32 program (LECO image analysis software), with a LECO Olympus SZH photographic microscope (see Figure 2.7a). Photographs of the specimens were taken using the digital camera attached to the microscope. The member area of the specimens was too large to be viewed all in one image. The cross-section was divided into eight sections and the area of each section was measured separately, and summed to determine the total cross-sectional area. The photographs were modified using Paint Shop Pro software to enhance the contrast between the lighter background and the darker cross-section. This enhanced image was imported into IA-32, threshold controls allowed darker colors representing the cross sectional area to be colored red. Also, the IA-32 program measured the total area of the image. The area of the cross section was calculated by multiplying the total area of the image by the fraction of the image that was red.

2.4.2 Fiber Volume Fraction Measurements

The nominal fiber volume fraction was assumed from the resin content of the pre-impregnated fiber obtained from TCR Composites. The actual fiber volume fraction was measured using the IA-32 program in conjunction with the LECO Olympus PME3
photographic microscope (see Figure 2.7b). The specimens were encased in resin and polished (see Chapter 3). The cross sections were photographed at ten random locations at a magnification of 100X. These locations were chosen to exclude any obvious voids. In the photographs, fibers can be seen as white ovals, while the resin appears gray. As described previously, manual thresholding in IA-32 colored the dark areas of the image red. In addition, the light areas were colored green. Because of the low contrast between white and gray in the image, slightly different area fractions were obtained each time a photograph was measured. In order to increase statistical accuracy of the measurement, the fiber fraction was measured three times in each image. There were a total of 45 measurements taken (15 images measured three times each). The average value and results are presented in Chapter 5. The fiber area fraction is the same as the fiber volume fraction if constant properties are assumed in the longitudinal direction.

Figure 2.7 LECO Olympus Photographic Microscopes: (a) SZH; and, (b) PME3
2.4.3 Void Fraction Measurements

Void fraction was also investigated. The process for void fraction measurement was very similar to fiber volume measurement. One difference is the void fraction photographs were taken at 20X magnification instead of 100X. The cross-sections were photographed at 10 locations instead of 15. The actual measurement taken in IA-32 was also slightly different. Threshold controls in IA-32 were used to color the dark portions of the image red. At 20X magnification, the voids are the only dark spaces. The fraction of the picture colored red was measured. Three measurements were taken at each location for a total of 30 measurements. The average was taken and is reported as the void fraction in Chapter 5.

2.5 TEST MATRIX

Tables 2.4 and 2.5 show the test matrices for all tests related to this thesis. Due to the labor-intensive manufacturing, only a few specimens were made. Due to the small number of specimens, variation of design parameters between specimens was not possible. For a complete description of specimen design, see Chapter 3.
### Table 2.4 Test Matrix

<table>
<thead>
<tr>
<th>Test</th>
<th>Specimen</th>
<th>Notation</th>
<th>Number of Specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression</td>
<td>IsoTruss®</td>
<td>IT</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>IsoPile™</td>
<td>IRC</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Steel-Reinforced Pile</td>
<td>SRC</td>
<td>3</td>
</tr>
</tbody>
</table>

**Area**

- Fiber Volume Fraction: Quality Control (QC) 6
- Void Fraction

### Table 2.5 Specimens Manufactured for Related Tests

<table>
<thead>
<tr>
<th>Test</th>
<th>Specimen</th>
<th>Notation</th>
<th>Number of Specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lab</td>
<td>IsoPile™</td>
<td>IRC</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Steel-Reinforced Pile</td>
<td>SRC</td>
<td>2</td>
</tr>
<tr>
<td>In-Situ</td>
<td>IsoPile™</td>
<td>IRC</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Steel-Reinforced Pile</td>
<td>SRC</td>
<td>1</td>
</tr>
</tbody>
</table>
CHAPTER 3 – SPECIMEN DESIGN

The IsoTruss® specimens and the steel cages were designed to reinforce concrete piles. Therefore, design of the specimens centered around the creation of an IsoTruss® structure that could be easily compared to a typical, pre-selected steel reinforcement cage.

Unique design challenges existed in this project due to the need for large members and rounded nodes. The specimens have the largest diameter members of any IsoTruss® structure produced to date. The longitudinal members have a design diameter of 0.42 in (1.07 cm). The previous largest structure was a 47 ft. (14.3 m) long carbon-epoxy IsoTruss® with a design longitudinal member diameter of 0.25 in (0.64 cm). The nodes were rounded more substantially than traditional IsoTruss® structures in order to increase the effective cross-sectional moment of inertia of the longitudinal members, when placed in a fixed-diameter concrete structure.

Design decisions regarding the dimensions of the piles and the strength of the concrete are described in this chapter. First the IsoTruss® specimen design is presented, followed by the design of the IsoPiles™ and the steel-reinforced piles.

3.1 ISOTRUSS® REINFORCEMENT

Use of an IsoTruss® structure as reinforcement in the IsoPile™ is a new application. Equations were developed to describe the more substantially rounded nodes of the IsoTruss® specimens. These equations and a description of the design process
regarding the longitudinal members, helical members, and length of the IsoTruss® specimens are presented in the following sections. Lengths of the IsoTruss® design in this section became the IsoTruss® (IT) compression specimens, as well as reinforcement for the bending and compression test IsoPiles™ (IRC).

3.1.1 Design of Rounded Node

On a standard IsoTruss® structure, the nodes are pointed. Helical members are generally straight members passing through a plane that includes two longitudinal members. The node occurs when a helical member crosses another helical member from an adjacent plane. Figure 3.1 shows two views of a section of an 8-node IsoTruss® grid structure. The node is the point where the helical members intersect.

![Figure 3.1 Views Identifying a Node of an IsoTruss® Structure: (a) Isometric View; and, (b) End View](image)
The node is farther away from the center of the cross-section than the longitudinal members. The helical members form pyramids with the apex being the node and the sides of the base defined by the two nearest longitudinal members (see Figure 3.2).

![Figure 3.2 Helical Member Pyramid with Intersecting Longitudinal Members](image)

The IsoTruss® specimens are the reinforcement in the IsoPiles™. The primary loading on the IsoPiles™ is flexure. For this reason, the bending stiffness of the IsoPile™ is compared to that of the steel reinforced pile. Bending stiffness can be represented by the product of modulus of elasticity, $E$, and moment of inertia, $I$ (see Gere [17]). Both reinforcements (steel and composite IsoTruss®) are composed of eight longitudinal members. The modulus of elasticity is fixed for each material, so the only way to change structural bending stiffness is to change the moment of inertia.

The parallel axis theorem can be used to determine the total moment of inertia of several shapes about a specific axis. The theorem is defined as:

$$ I = \sum_{i=1}^{n} \left[ I_{i0} + A_{i} d_{i}^{2} \right] $$

(3.1)
where \( I \) is the total moment of inertia about a given axis, \( I_{0i} \) is the moment of inertia of each component about its own parallel axis, \( A_i \) is the cross-sectional area of each component, and \( d_i \) is the distance between the component’s own axis and the given axis.

In both reinforcements, the longitudinal members are in a circular pattern. All the members are equally spaced from the center of the cross section. When Equation 3.1 is applied to this research, \( A_i \) is the area of the individual members, and the largest \( d_i \) is the radius of the reinforcement group, \( r_s \), sometimes referred to as the inner radius of a standard IsoTruss®. Figure 3.3 compares a standard IsoTruss® cross-section to the cross-section of an IsoTruss® with rounded nodes.

![Figure 3.3 IsoTruss® End Views: (a) Standard IsoTruss®; and, (b) IsoTruss® with Rounded Nodes](image)

In order to make the longitudinal members of the IsoTruss® as efficient as possible in bending, the area of the members must be minimized, while the inner
diameter, \( d_s = 2r_s \), is maximized. The overall diameter of a standard IsoTruss® grid structure, \( D \), is the distance between opposite nodes. Therefore, the inner diameter cannot be increased without also increasing the overall diameter, \( D \), of the entire IsoTruss®.

The concept of rounded nodes allows a larger inner diameter, \( d_r = 2r_r \), without increasing the overall diameter. If the helical members make a gradual, rounded turn, the ratio of \( d_r \) to total diameter, \( D \), is larger than the ratio of \( d_s \) to \( D \). Figure 3.4 shows the end view of a standard IsoTruss® superimposed over the end view of an IsoTruss® with rounded nodes.

![Figure 3.4 End Views of Standard and Rounded-Node IsoTruss® Grid Structures with Same Overall Diameter](image)

From Figure 3.4, a relationship between the increase in inner diameter, \( \Delta d \), and the reduction in node height, \( \delta \), was developed. To begin, the difference in diameter is related to the radii by:
\[ \Delta d = d_x - d_z = 2\Delta r = 2(r_i - r_s) \]  \hspace{1cm} (3.2)

By simple geometry:

\[ r_i - r_s = \frac{\delta}{\cos \frac{\theta}{2}} \]  \hspace{1cm} (3.3)

Therefore,

\[ \Delta d = \frac{2\delta}{\cos \frac{\theta}{2}} \]  \hspace{1cm} (3.4)

where \( \delta \) is the reduction in the height of the pyramid and \( \theta \) is the reference angle as described in Winkel [3]. For an 8-node IsoTruss® structure, \( \theta = 45^\circ \). Equation 3.4 only applies when the rounded section has the same overall diameter as the original section, as shown in Figure 3.4.

Many factors were considered in the design of the rounded node to account for the complex geometry of the IsoTruss®. First, the curvature of the node was considered. McCune [5] observed a lower than expected strength in the composite due to the sharp bend where the fibers wrapped around an endplate. An elliptical path was selected for the tops of the pyramids, in order to maximize the local radius of the helical member. The straight portion of the helical members meets the elliptical shape at a tangent in order to transition smoothly. Figure 3.5 shows an isometric view of half of an 8-node
IsoTruss®. Only two helical members and four longitudinal members are shown for clarity. Different views of Figure 3.5 are shown in the following paragraphs to graphically represent the design of the rounded node.

**Figure 3.5 Isometric View of 8-Node IsoTruss® Pyramid Defined by Nodes A, C, E, F, and G**

The simplest way to achieve the design of the rounded node was to set up a two-dimensional coordinate system containing one helical member. The path of one helical member lies on Plane $ACE$ in Figure 3.5. The helical members were modeled with algebraic equations representing the path of the fibers in the plane. Equations for angles
(see Figure 3.6) defining the direction of helical members of a standard IsoTruss® grid structure were derived. These equations were extremely helpful in the design of the elliptical portion of the rounded node, as well as tooling design (Chapter 4).

\[
\begin{align*}
\epsilon &= \tan^{-1}\left(\frac{\sqrt{L^2 + R^2}}{R}\right) \\
&= \tan^{-1}\left(\frac{2L}{R}\right)
\end{align*}
\]  

Figure 3.6 Projected Views of Node: (a) Top View; (b) Plane ACE; and, (c) Side View

From simple geometry, Equations 3.3-3.5 can be derived using Figure 3.6:
\[ \gamma = \tan^{-1} \left( \frac{R}{L_b} \right) \]  

(3.6)

\[ \varphi = \tan^{-1} \left( \frac{L_b}{R} \right) \]  

(3.7)

where \( R \) is the overall radius of the IsoTruss® and \( L_b \) is the bay length as defined in Winkel [3] and Figure 3.7.

Figure 3.7 Side View of One Bay of an IsoTruss®

The algebraic equations for direction of the helical member were set up in plane \( BCD \) as described in Figure 3.5. This plane is shown in a two-dimensional view in Figure 3.8. The origin is at point \( B \) and the positive \( \xi \)-direction extends toward point \( D \). The straight portion of the helical member is modeled as a line. The curved portion is
modeled as an ellipse with horizontal center at the midpoint between $B$ and $D$. All other parameters of the ellipse are variables. The variables, $h$ and $k$, are the coordinates of the center of the ellipse and $p$ is the local height of the node.

$$\frac{(\xi - h)^2}{a^2} + \frac{(\eta - k)^2}{b^2} = 1 \quad (3.8)$$

Figure 3.8 Local Coordinate System (Plane BCD) for Rounded Node Design

In order for the straight section of the helical member to meet the curved section at a tangent, both sections must have the same slope at that point. The slope of the straight section in Figure 3.8 is easily obtained when $\epsilon$ is known. Salas [18] describes the equation of an ellipse in the $\xi$-$\eta$ plane as:
where $h$ and $k$ are the $\xi$ and $\eta$ coordinates, respectively, of the center of the ellipse, and $a$ and $b$ are the major and minor radii of the ellipse, respectively. Solving Equation 3.8 for $\eta$ yields:

$$\eta = \frac{b\sqrt{a^2 - (\xi - h)^2}}{a} + k$$

(3.9)

To find an equation for the slope of the curved portion of the helical member, the derivative of Equation 3.9 is taken with respect to $\xi$. The result is:

$$\frac{d\eta}{d\xi} = \frac{-b(\xi - h)}{a\sqrt{a^2 - (\xi - h)^2}}$$

(3.10)

The slope of the straight portion of the helical member is:

$$\frac{d\eta}{d\xi} = \tan(90 - \epsilon)$$

(3.11)

where $\epsilon$ is the direction definition angle defined in Equation 3.5 in degrees.

During fabrication, the members of the IsoTruss® are in tension. This enables the fibers that make up the grid to hold their shape while being cured. The helical members can not hold the elliptical shape of the rounded node without resting against some kind of tooling. The location of the point of tangency of the straight helical member to the elliptical section is important in order to design the tooling. At the point of tangency, the
curved and straight portions of the helical member have the same slope. Therefore, the point of tangency is found by first setting Equations 3.10 and 3.11 equal to each other and squaring each side to obtain:

\[
\frac{b^2(\xi - h)^2}{a^2 - a^2(\xi - h)^2} = \tan^2(90 - \varepsilon)
\]  

(3.12)

Solving Equation 3.12 for \(\xi\) yields:

\[
\xi = h \pm \frac{a^2 \tan(90 - \varepsilon)}{\sqrt{b^2 + a^2 \tan^2(90 - \varepsilon)}}
\]  

(3.13)

where \(\xi\) is the \(\xi\)-coordinate for the point of tangency. As explained in the previous section, the size of the longitudinal members depends on the design of the rounded node. In order to optimize the IsoTruss® grid structure reinforcement, the size of the longitudinal members and the shape of the rounded node were designed simultaneously using the solver function in Microsoft Excel®.

3.1.2 Design of IsoTruss® Longitudinal Members

The longitudinal members were designed to match the bending stiffness of the IsoTruss® reinforcement to the bending stiffness of the steel reinforcement. The size of the longitudinal members is determined by the total moment of inertia of the IsoTruss®. The moment of inertia is a function of the longitudinal member area and distance from the center of the IsoTruss®. The distance of the members from the center of the IsoTruss® depends on geometric parameters including shape of the rounded node.
The modulus of elasticity for the composite was obtained using the rule of mixtures. The fiber volume fraction was not known at the time of design, so it was assumed to be 0.65. This is consistent with past experience with pre-impregnated fibers. The rule of mixtures for modulus of elasticity in the fiber direction, \(E\), is defined by Barbero [20] as:

\[
E = E_f V_f + E_m (1 - V_f)
\]

where \(E_f\) is the modulus of elasticity of the fiber, \(E_m\) is the modulus of elasticity of the matrix, and \(V_f\) is the fiber volume fraction. Barbero [20] lists the modulus of elasticity of T300 carbon fiber as 33,400 ksi (230,000 MPa). Thiokol lists the modulus of elasticity of UF3325-95 resin as 410.0 ksi (2,830 MPa). When these values are substituted into Equation 3.9 with an assumed fiber volume fraction of 0.65, the combined modulus of elasticity of the composite is 21,800 ksi (150,000 MPa).

Using the parallel axis theorem (Equation 3.1), the moment of inertia of the steel reinforcement group was calculated as 14.21 in\(^4\) (591.5 cm\(^4\)). The modulus of elasticity of steel is approximately 29,000 ksi (200,000 MPa) (see Gere [17]). Therefore, the bending stiffness, \(EI\), for the steel reinforcement is 412,100 kip-in\(^2\) (11,830 MN-cm\(^2\)). The bending stiffness of the steel reinforcement was divided by the modulus of elasticity of the composite to obtain the required moment of inertia of the longitudinal members of the IsoTruss® reinforcement. The moment of inertia obtained from this calculation is 18.88 in\(^4\) (785.9 cm\(^4\)).

Microsoft Excel® was used to optimize the IsoTruss® design. A spreadsheet was set up to calculate all the geometric parameters of the IsoTruss® (see Figure 3.9). The
The solver determined the geometry of the IsoTruss® separate from the nodes as if it were a standard IsoTruss® grid structure. The solver also designs the rounded head and calculates the size of the longitudinal members required to match the aforementioned bending stiffness.

<table>
<thead>
<tr>
<th>Normal IsoTruss Parameters</th>
<th>Benefit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter, 2r = D</td>
<td>14.87 inches</td>
</tr>
<tr>
<td>Bay Length, L_b</td>
<td>7.44 inches</td>
</tr>
<tr>
<td>d/L_b</td>
<td>2.00</td>
</tr>
<tr>
<td>Nodes</td>
<td>8</td>
</tr>
<tr>
<td>Reference Angle, θ</td>
<td>45 degrees</td>
</tr>
<tr>
<td>Bay Angle, φ</td>
<td>35.26 degrees</td>
</tr>
<tr>
<td>Min Inner Diameter, 2r_{min}</td>
<td>10.52 inches</td>
</tr>
<tr>
<td>Inner Diameter, d_i</td>
<td>11.38 inches</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Benefit</th>
</tr>
</thead>
<tbody>
<tr>
<td>El to Match</td>
</tr>
<tr>
<td>Longi Tows</td>
</tr>
<tr>
<td>Increase in d,</td>
</tr>
<tr>
<td>Reduction in Tows</td>
</tr>
</tbody>
</table>

| Length | 360 inches |
| Bays   | 48.41 |
| Bay Length Fraction | 0.41 |
| Whole Bays | 48 |

<table>
<thead>
<tr>
<th>Head Angles</th>
</tr>
</thead>
<tbody>
<tr>
<td>θ</td>
</tr>
<tr>
<td>r</td>
</tr>
<tr>
<td>ϕ</td>
</tr>
<tr>
<td>Height of Node, p</td>
</tr>
<tr>
<td>True Diameter, 2(p+r_{min})</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bend Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major Axis, a</td>
</tr>
<tr>
<td>Minor Axis, b</td>
</tr>
<tr>
<td>Horizontal Center, h</td>
</tr>
<tr>
<td>Vertical Center, k</td>
</tr>
<tr>
<td>a/b ratio</td>
</tr>
<tr>
<td>Fiber-Head Contact, x</td>
</tr>
<tr>
<td>Slope at Contact Point</td>
</tr>
<tr>
<td>Slope of Straight Heli</td>
</tr>
<tr>
<td>Head Material Length</td>
</tr>
</tbody>
</table>

**Figure 3.9 Microsoft Excel® Worksheet for IsoPile™ Design**

The solver was allowed to vary the diameter of the normal IsoTruss® (without the rounded nodes). The solver was also allowed to vary the ellipse parameters a, b, and k in Figure 3.8. The actual diameter of the IsoTruss® was restricted to 13.0 in (33.0 cm) because of the 14.0 in (35.6 cm) diameter of the concrete forms. Concrete cover is
provided mainly for corrosion control. Because of the corrosion resistance of the carbon-epoxy composite used, the diameter of the IsoTruss® could be increased to virtually eliminate concrete cover. The degree of curvature of the elliptical section was also controlled by limiting the $a/b$ ratio to 6.0. The solver was instructed to maximize the inner diameter, $d_r$, in order to minimize the material required to match the steel reinforcing bending stiffness. The inner diameter was also limited to a value less than or equal to 12.0 in (30.5 cm). This was done to prevent the solver from rounding off the nodes completely by the inner diameter to 13.0 in (33.0 cm). Also, for simplicity, the diameter to bay length ratio was set at two. The diameter in this ratio is what the diameter of the IsoTruss® would be without the rounded nodes.

Figure 3.9 shows the design selected by the solver. The geometric parameters are summarized in Table 3.1. The parameters refer to the geometry of the IsoTruss® with members modeled as lines (no member area). The actual IsoTruss® members have area which makes the actual overall diameter slightly larger.

![Figure 3.9 IsoTruss® Design Selected by Solver](image)

<table>
<thead>
<tr>
<th>Table 3.1 IsoTruss® Reinforcement Design Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter</td>
</tr>
<tr>
<td>-----------</td>
</tr>
<tr>
<td>Diameter, D</td>
</tr>
<tr>
<td>Bay Length, $L_b$</td>
</tr>
<tr>
<td>Inner Diameter, $2r_i$</td>
</tr>
</tbody>
</table>

Equation 3.1 is used to solve for the required area of one longitudinal member. Because the longitudinal members are identical, the moment of inertia, $I_{0i}$, and the cross-
sectional area, $A_i$, are the same for each member, and their values are $\pi r_m^4/4$ and $\pi r_m^2$, respectively, where $r_m$ is the radius of the longitudinal member. From Figure 3.10, the distance between the longitudinal members’ parallel axis and the axis of bending, $d_i$, for the two longitudinal members lying on the neutral axis is zero. The distance, $d_i$, for the two outer-most longitudinal members is equal to the inner radius, $d_3 = d_7 = r_r$, obtained by the solver. The distance, $d_i$, for the other four longitudinal members is equal to $d_2 = d_4 = d_6 = d_8 = r_r \sin \theta$. Table 3.2 details the summation performed for each longitudinal member in order to obtain the total moment of inertia.

![Diagram](image-url)

**Figure 3.10** Distance Values for Parallel Axis Theorem
Table 3.2 IsoTruss® Geometric Parameters (Input Values for Equation 3.1)

<table>
<thead>
<tr>
<th>Longitudinal Member, $i$</th>
<th>Parameter</th>
<th>$I_{0i}$</th>
<th>$A_i$</th>
<th>$d_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$\pi \frac{r_m^4}{4}$</td>
<td>$\pi r_m^2$</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>$\pi \frac{r_m^4}{4}$</td>
<td>$\pi r_m^2$</td>
<td>$r, \sin \theta$</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>$\pi \frac{r_m^4}{4}$</td>
<td>$\pi r_m^2$</td>
<td>$r_r$</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>$\pi \frac{r_m^4}{4}$</td>
<td>$\pi r_m^2$</td>
<td>$r, \sin \theta$</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>$\pi \frac{r_m^4}{4}$</td>
<td>$\pi r_m^2$</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>$\pi \frac{r_m^4}{4}$</td>
<td>$\pi r_m^2$</td>
<td>$r, \sin \theta$</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>$\pi \frac{r_m^4}{4}$</td>
<td>$\pi r_m^2$</td>
<td>$r_r$</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>$\pi \frac{r_m^4}{4}$</td>
<td>$\pi r_m^2$</td>
<td>$r, \sin \theta$</td>
<td></td>
</tr>
</tbody>
</table>

The required moment of inertia, 18.88 in$^4$ (785.9 cm$^4$) is substituted into Equation 3.1 with the values from Table 3.2 in order to solve for the member radius, $r_m$. The member radius, in turn, is used to find the required member area, which is 0.15 in$^2$ (0.97 cm$^2$).

12k tows of pre-impregnated carbon fiber were available for manufacturing. 12k means there are 12,000 individual fibers in each tow of carbon fiber. Data from the manufacturer was used to calculate the fiber area of one tow as $7.16 \times 10^{-4}$ in$^2$ (4.62 x 10$^{-3}$ cm$^2$). The total area of an IsoTruss® member can be described by:

$$A = \frac{NA}{V_f}$$

(3.15)
where $N$ is the number of tows, $A_t$ is the area of one tow, and $V_f$ is the fiber volume fraction. The required member area was put into Equation 3.15 with a theoretical fiber volume fraction of 0.65, and the tow area calculated above in order to obtain the number of tows required. Rounding up to the whole tow, 133 fiber tows per longitudinal member are required to produce the moment of inertia required to match the bending stiffness of the pre-selected steel reinforcement.

3.1.3 Design of IsoTruss® Helical Members

The helical members were designed first, and the transverse steel was designed to match. The number of tows in the helical members of a typical IsoTruss® structure are typically about one-half to two-thirds the number of tows in the longitudinal members. Following this convention, the helical members were designed to have two-thirds the amount of tows in the longitudinal members. Two-thirds of the longitudinal tows (133) rounded up to the whole tow is 89, which equates to an area of 0.10 in$^2$ (0.63 cm$^2$).

3.2 CONCRETE PILE SPECIMENS

Steel-reinforced (SRC) and IsoTruss®-reinforced (IRC) concrete piles were manufactured for bending and compression tests. The reinforcement was identical in all SRC and IRC piles, respectively. Length of the specimens, as detailed in Chapter 2, varied according to which test would be conducted. A diameter of 14.0 in (35.6 cm) was chosen for all piles because it is a common diameter for circular pile forms. The concrete was provided and placed by Eagle Precast of Salt Lake City, Utah. For manufacturing purposes, a self-consolidating concrete was chosen. The mix had an expected strength of 12 ksi (83 MPa). The actual strength and the concrete placement procedure are detailed
in Chapter 4. The reinforcement for the IsoPileSTM is identical to the IsoTruss®
reinforcement designed in Section 3.1. The steel reinforcement design is explained in
detail in the following sections.

3.2.1 Longitudinal Reinforcement for SRC

Longitudinal reinforcement for foundations cast against soil is usually given 3.0
in (7.62 cm) of cover (see MacGregor [19]) to minimize corrosion. Because the piles for
this research were cast in forms, indoors, the cover was reduced to 2.5 in (6.35 cm).
Eight #4 grade 60 steel reinforcing bars were chosen for the longitudinal reinforcement.
#4 was chosen in order to avoid extremely large loads in testing. A quantity of eight was
selected to be easily comparable to an 8-node IsoTruss® grid structure.

Lap splices were required due to the long length of the field test specimens. The
required length of a standard lap splice is explained by MacGregor [19]. For #6 bars and
smaller, the relevant equation is:

\[
\frac{l_d}{d_b} = \frac{f_y \alpha \beta \lambda}{25 \sqrt{f'_c}}
\]  

(3.16)

where \(l_d\), \(d_b\), and \(f_y\) are the development length, bar diameter, and yield strength,
respectively, of the spliced bar; \(f'_c\) is the strength of the concrete; and \(\alpha\), \(\beta\), and \(\lambda\) are the
bar location factor, coating factor, and lightweight concrete factor, respectively. The bars
used for the steel reinforced concrete have location and coating factors of 1.0. The
lightweight concrete factor is 1.0, because normal weight concrete was to be used in
construction. The diameter of the #4 bar is 0.5 in (1.3 cm), and the reported yield
strength is 60.0 ksi (413.7 MPa). The actual yield strength is slightly higher (see Chapter
6). At the time of construction of the steel reinforcing cages, the concrete strength was assumed to be 4.0 ksi (27.6 MPa), but the concrete used in construction had a strength much higher than this. The use of 4.0 ksi (27.6 MPa) as concrete strength and 60.0 ksi (413.7 MPa) as yield strength of the steel makes the design of the steel splices extra conservative. This gives another advantage to the steel reinforced piles over the IsoTruss®-reinforced piles.

When all values are substituted into Equation 3.16, \( l_d \) is 19.0 in (48.1 cm). This value is multiplied by 1.3 to constitute a class B lap splice. The new value was 24.7 in (62.6 cm). The lap splice of the longitudinal bars was rounded up to 25.0 in (63.5 cm).

3.2.2 Transverse Reinforcement for SRC

The transverse steel reinforcing was designed to match the helical members of the IsoTruss® reinforcement. In order to do this, a method for comparing helical members of the IsoTruss® to transverse steel members was developed.

One difficulty in comparing helical members to transverse steel reinforcement is the difference in orientation. Steel hoops are oriented parallel to the cross-section of the pile. Helical members are oriented at a certain angle to the cross-section of the pile. In order to compute the strength of a composite in a direction other than longitudinal or transverse, the angle between the fibers and the direction in question must be determined.

The fiber direction is the path of the helical members. The new direction for which the strength must be computed is the path of the steel hoops. If the two directions are treated as vectors, the dot product can be used to compute the angle between them. Figure 3.11 shows the coordinate system that was defined to compute the angle between the vectors, \( \lambda \), on part of an IsoTruss® grid structure. Half of the longitudinal members
and most of the helical members have been removed for clarity. The figure also shows a steel hoop with the same diameter as the maximum diameter of the IsoTruss® grid structure. The origin of the coordinate system is at the center of the steel hoop. The $y$-$z$ plane and the steel hoop are co-planar.

The helical members are straight and the steel hoops are curved. This causes the angle between them to vary around the circumference. $\lambda$ was measured along a helical member between two nodes. This covers the full range of angles, because the remaining helical members repeat the same geometry.

![Global Coordinate System for Measuring Stress Transformation Angle](image)

Figure 3.11  Global Coordinate System for Measuring Stress Transformation Angle
The helical member forming one side of the angle $\lambda$ in Figure 3.11 can be described as the following vector in the $x$-$y$-$z$ coordinate system:

$$\vec{H} = L_0 \hat{i} + \{D \sin \theta \} \hat{k}$$  \hspace{1cm} (3.17)

where $D$ is the overall diameter of the standard IsoTruss® grid structure, $\theta$ is the reference angle, and $L_B$ is the bay length, as defined by Winkel [3]. Figure 3.12 shows the parameters $D$ and $\theta$ on the end view of a standard 8-node IsoTruss® structure.

![Figure 3.12 End View of Standard 8-Node IsoTruss® Structure](image)

The vector representing the steel hoop is a function of $x$. The equation of a circle in the $y$-$z$ plane, with its center at the origin, is:
\[ y^2 + z^2 = R^2 \]  

(3.18)

where \( R \) is the radius of the circle. Solving Equation 3.18 for \( y \) yields:

\[ y = \sqrt{R^2 - z^2} \]  

(3.19)

Taking the derivative of Equation 3.19 with respect to \( x \) gives the slope of the steel hoop in the \( y-z \) plane:

\[ \frac{dy}{dz} = \frac{-z}{\sqrt{R^2 - z^2}} \]  

(3.20)

From the slope in Equation 3.20, the following vector was obtained to describe the direction of the steel hoop:

\[ \vec{S} = -z\hat{j} + \sqrt{R^2 - z^2} \hat{k} \]  

(3.21)

\( R \) is the radius of the steel hoop and the overall radius of the IsoTruss®, and \( z \) is the \( z \)-coordinate in the defined system.

Salas [18] defines the dot product of two vectors as:

\[ \vec{P} \cdot \vec{Q} = \|P\|\|Q\|\cos \lambda \]  

(3.22)
\[ \vec{P} \cdot \vec{Q} = P_i Q_i + P_j Q_j + P_k Q_k \]  

(3.23)

where \(|P|\) and \(|Q|\) are the magnitudes of vectors \(\vec{P}\) and \(\vec{Q}\), respectively, and \(\lambda\) is the angle between vectors \(\vec{P}\) and \(\vec{Q}\). The values \(P_i, P_j,\) and \(P_k\) represent the components of the vector \(\vec{P}\) in the x-, y-, and z-directions, respectively. The magnitudes of the vectors described by Equations 3.17 and 3.21 were easily obtained using geometry. The magnitude of vector \(\vec{H}\) is \(\sqrt{L^2 + D^2 \sin^2 \theta}\). The magnitude of vector \(\vec{S}\) is \(R\).

Substituting these values into Equation 3.22 yields:

\[ \vec{H} \cdot \vec{S} = R \cos \lambda \sqrt{L^2 + D^2 \sin^2 \theta} \]  

(3.24)

The directional values \(i, j,\) and \(k\), of the vectors described by Equations 3.17 and 3.21 were substituted into Equation 3.23:

\[ \vec{H} \cdot \vec{S} = D \sin \theta \sqrt{R^2 - z^2} \]  

(3.25)

Setting Equations 3.24 and 3.25 equal to each other yields:

\[ R \cos \lambda \sqrt{L^2 + D^2 \sin^2 \theta} = D \sin \theta \sqrt{R^2 - z^2} \]  

(3.26)

An equation for the angle between the helical member and the steel hoop was obtained by solving for \(\lambda\) and substituting \(2R\) for \(D\):
\[ \lambda = \cos^{-1} \left( \frac{2 \sin \theta \sqrt{R^2 - z^2}}{\sqrt{L_b^2 + 4R^2 \sin^2 \theta}} \right) \] 

(3.27)

where \( R \) is the radius of the steel hoop and the overall radius of the IsoTruss®, \( \theta \) is the reference angle (45° for an 8-node IsoTruss®), \( L_b \) the bay length, 7.44 in (18.9 cm); and \( z \) is the \( z \)-coordinate in the system defined in Figure 3.11.

A different equation must be derived to account for the rounded node in the IsoTruss® reinforcement. Equation 3.27 applies to the straight section of the helical member. Equation 3.10 describes the slope of the curved portion of the helical member in the coordinate system defined in Figure 3.8. From this equation, a vector describing the path of the rounded portion of the helical member can be computed:

\[
\vec{N} = \left\{ a \sqrt{a^2 - (\xi - h)^2} \right\} \hat{i} - \left\{ b(\xi - h) \right\} \hat{j}
\]

(3.28)

where \( a, b, \) and \( h \) are the geometric parameters describing the ellipse, and \( \xi \) is the value in the local coordinate system \((\xi, \eta)\) defined in Figure 3.8. This vector is described by \( \hat{i} \) and \( \hat{j} \), to differentiate from the global coordinate system used by vectors \( \vec{H} \) and \( \vec{S} \) in Equations 3.17 and 3.21.

In order to describe the entire helical member, vector \( \vec{N} \) in Equation 3.28 must be described in the same coordinate system as vectors \( \vec{H} \) and \( \vec{S} \). Groesberg [21] describes the linear transformation of vectors between coordinate systems as:
\[
    x = Lx',
\]

(3.29)

\[
    \begin{bmatrix}
        x_1 \\
        x_2 \\
        x_3
    \end{bmatrix} =
    \begin{bmatrix}
        l_{11} & l_{12} & l_{13} \\
        l_{21} & l_{22} & l_{23} \\
        l_{31} & l_{32} & l_{33}
    \end{bmatrix}
    \begin{bmatrix}
        x_1' \\
        x_2' \\
        x_3'
    \end{bmatrix}
\]

(3.30)

where rows \( x_1, x_2, \) and \( x_3 \) are coordinates in the global system, \( x_1', x_2', \) and \( x_3' \) are the equivalent coordinates in the local coordinate system, and \( l_{ij} = \cos \angle(x_i, x_j') \) is the direction cosines defining the transformation between the two coordinate systems.

The direction cosines can easily be obtained if the angles of rotation between systems are known. Two rotations are made to change the global coordinate system to the local system. First, the global coordinate system in Figure 3.11 is rotated by \( \theta = 45^\circ \) clockwise about the \( x \)-axis (see Figure 3.13). Then the intermediate coordinate system in Figure 3.13(a) is rotated by \( \gamma = 45^\circ \) about the \( y' \)-axis. The resulting \( x'' \)-axis and \( y''' \)-axis in Figure 3.13(b) are in the same direction as the \( \xi \)-axis and \( \eta \)-axis, respectively, from Figure 3.8.
Figure 3.13 Views of IsoTruss® Showing Rotation to Obtain Local Coordinate System: (a) End View – First Rotation; and, (b) Side View – Second Rotation

The direction cosines for the first and second rotation can easily be found when the angles are known (see Groesberg [21]):

\[
L_1 = \begin{bmatrix}
1 & 0 & 0 \\
0 & \cos \theta & -\sin \theta \\
0 & \sin \theta & \cos \theta
\end{bmatrix}
\]

(3.31)

\[
L_2 = \begin{bmatrix}
\cos \gamma & 0 & \sin \gamma \\
0 & 1 & 0 \\
-\sin \gamma & 0 & \cos \gamma
\end{bmatrix}
\]

(3.32)

where $\theta$ and $\gamma$ as defined in Figure 3.13 are negative relative to the sign convention employed to determine the transformation matrices in Equations 3.31 and 3.32. The
value for $\gamma$ is defined in Equation 3.6 and $\theta$ is 45° for an 8-node IsoTruss®. Note that
the direction cosines are incorrect in Figure 1.11 of Groesberg [21]. As explained in
Figure 3.13:

$$x = L_1 x'$$ (3.33)

$$x' = L_2 x''$$ (3.34)

Substituting Equation 3.34 into Equation 3.33:

$$x = L_1 L_2 x''$$ (3.35)

where $x$ is the coordinates in the global system, and $x''$ is the coordinates in the local
system described in Figures 3.8 and 3.13. Finally, substituting Equations 3.31 and 3.32
into Equation 3.35, along with vector $\bar{N}$ from Equation 3.28:

$$\begin{bmatrix} i \\ j \\ k \end{bmatrix} = \begin{bmatrix} \cos \gamma & 0 & -\sin \gamma \\ \sin \theta \sin \gamma & \cos \theta & \sin \theta \cos \gamma \\ \sin \gamma \cos \theta & -\sin \theta & \cos \theta \cos \gamma \end{bmatrix} \begin{bmatrix} a\sqrt{a^2 - (\xi - h)^2} \\ -b(\xi - h) \\ 0 \end{bmatrix}$$ (3.36)

where $\xi$ is the local coordinate of the curved portion of the helical member; and $a$, $b$, and
$h$ are the ellipse parameters described previously.
The angle between the curved portion of the helical member and the steel hoop is measured by comparing their corresponding vectors at a given radial location (see Figure 3.14). The members are analyzed from \(-\theta\) to \(\theta\). The global and local coordinates of the node located at an angle of \(-\theta\) from the global \(y\)-axis in Figure 3.14 can be found using IsoTruss® geometry. The change in global coordinates of any point along the curved portion of the helical member is found by substituting the change in local coordinates for \(\xi''\) in Equation 3.35. The radial location corresponding to the global coordinates of the curved portion of the helical member is found through simple trigonometry.

The directional vectors of the helical member are found by substituting the local coordinate \(\xi\) into Equation 3.36. The directional vectors of the steel hoop are found with Equation 3.21, and the corresponding radial location is found through simple trigonometry. Lastly, the angle between the helical member and steel hoop directional vectors is found by simultaneously solving Equations 3.23 and 3.24, substituting \(\bar{R}\) and \(\bar{S}\) at the same radial locations for \(\bar{P}\) and \(\bar{Q}\), respectively. Figure 3.15 shows the angle between the helical member and the transverse steel reinforcement, \(\lambda\), as a function of the reference angle described in Figure 3.14.

![Figure 3.14 End View of IsoTruss® with Rounded Nodes](image-url)
The discussion of the angle between the helical member and the steel member has the object of estimating the strength of a helical member in the direction of the steel ring. When transforming the strength of a composite from the longitudinal direction, larger angles between the longitudinal direction and the new direction will yield lower strength values. Figure 3.15 shows the strength transformation angles along the helical members of one plane of the IsoTruss®. The redundant nature of the IsoTruss® grid structure is such that other helical members exist within the same interval on the adjacent planes. In order to be conservative in the strength estimate, the angles of these helical members should also be considered.

Figure 3.16 shows the graph of another helical member within the same interval already discussed. All helical members within the interval have the geometry of one of
the two helical members in Figure 3.16. Figure 3.17 shows an envelope that includes the
greatest angle at any point in the interval.

The angles shown in Figure 3.17 were used to calculate the strength of the helical
members in the circumferential direction. Barbero [20] describes the transformation of
local stresses to global stresses in a composite by the equation:

\[
\begin{bmatrix}
\sigma_1 \\
\sigma_2 \\
\sigma_6
\end{bmatrix} = \begin{bmatrix}
m^2 & n^2 & 2mn \\
n^2 & m^2 & -2mn \\
-mn & mn & m^2 - n^2
\end{bmatrix}\begin{bmatrix}
\sigma_x \\
\sigma_y \\
\sigma_{xy}
\end{bmatrix}
\]

(3.37)

where \( \sigma_1, \sigma_2, \) and \( \sigma_6 \) are the global longitudinal, transverse, and shear stresses, and \( \sigma_x, \sigma_y, \) and \( \sigma_{xy} \) are the local longitudinal, transverse, and shear stresses, respectively. The
variables \( m \) and \( n \) are defined as \( \cos \lambda \) and \( \sin \lambda \), respectively, where \( \lambda \) is the angle
between the global and local directions. In this case, \( \lambda \) is the angle between the helical
members and the transverse steel reinforcement. The strength in the circumferential
direction was substituted into Equation 3.37 as \( \sigma_x \), while \( \sigma_y \) and \( \sigma_{xy} \) were set to zero. The
global material stresses were obtained in terms of \( \lambda \) and \( \sigma_x \).
Barbero [20] defines the Tsai-Wu failure criterion as:
\[ F_\sigma_1 + F_\sigma_2 + F_\sigma_2 + F_{\sigma_1}^2 + F_{\sigma_2}^2 + F_{\sigma_6}^2 \sigma_2 = 1 \] (3.38)

where \( F_1, F_2, F_{11}, F_{22}, \) and \( F_{66} \) are parameters defined by the material strengths of the composite. The equations for these parameters are:

\[ F_1 = \frac{1}{F_{1t}} - \frac{1}{F_{1c}} \] (3.39)

\[ F_2 = \frac{1}{F_{2t}} - \frac{1}{F_{2c}} \] (3.40)

\[ F_{11} = \frac{1}{F_{1t} F_{1c}} \] (3.41)

\[ F_{22} = \frac{1}{F_{2t} F_{2c}} \] (3.42)

\[ F_{66} = \frac{1}{F_6^2} \] (3.43)

where \( F_{1t} \) and \( F_{2t} \) are the longitudinal tensile strength and transverse tensile strength, respectively; and \( F_{1c}, F_{2c}, \) and \( F_6 \) are the longitudinal compressive strength, transverse compressive strength, and in-plane shear strength, respectively. The values in Table 3.3 are substituted into Equations 3.39-3.43, which are in turn substituted into Equation 3.38 with the values obtained from Equation 3.37 for \( \sigma_1, \sigma_2, \) and \( \sigma_6 \). Equation 3.38 can be
solved for $\sigma_x$ in terms of $\lambda$. The largest angle between the helical member and the circumferential directions was used in order to be conservative. This angle is 47.7°. This yields a $\sigma_x$ value of 11.5 ksi (79.4 MPa). Figure 3.18 shows $\lambda$ plotted versus $\sigma_x$.

Table 3.3 Strength Values for Typical Carbon/Epoxy Composite (see Barbero [20])

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Notation</th>
<th>Value [ksi (MPa)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal Tensile Strength</td>
<td>$F_{1t}$</td>
<td>265.4 ksi (1,830 MPa)</td>
</tr>
<tr>
<td>Transverse Tensile Strength</td>
<td>$F_{2t}$</td>
<td>8.3 ksi (57 MPa)</td>
</tr>
<tr>
<td>Longitudinal Compressive Strength</td>
<td>$F_{1c}$</td>
<td>159.0 ksi (1,096 MPa)</td>
</tr>
<tr>
<td>Transverse Compressive Strength</td>
<td>$F_{2c}$</td>
<td>33.1 ksi (228 MPa)</td>
</tr>
<tr>
<td>In-Plane Shear Strength</td>
<td>$F_6$</td>
<td>10.3 ksi (71 Mpa)</td>
</tr>
</tbody>
</table>

Figure 3.18 Tensile Strength vs. Offset Angle
Earl [8] showed that spacing of transverse reinforcement is the primary concern in containment failure of concrete columns. Strength of the reinforcement is the next concern. An inspection of the geometry of the IsoTruss® grid structure shows that rings are simulated by the helical members. Figure 3.19 shows two bays of an 8-node IsoTruss® grid structure with parts of the helical members removed to show this concept. The longitudinal members are shown as dashed lines. Portions of two different helical members form one effective “ring” which stretches across two bays. In an 8-node IsoTruss® grid structure, there are eight of these rings in a two-bay length. This would mean that on average there are four rings per bay. Because the helical members cross so often, each pair of helical members was matched with one steel ring. This makes the spacing of the steel rings one half bay length, or 3.72 in. (9.45 cm).

Figure 3.19 Helical Member “Rings”
The design of the IsoPile™ reinforcement has a helical member area of 0.094 in² (0.604 cm²). Therefore, the allowable force on one of the simulated helical member rings is twice the helical area times the strength in the circumferential direction. This yields a force of 2.02 kips (9.0 kN). When this force is divided by the strength of the steel, the area of the steel rings is obtained.

The company which supplied the steel had #2 bars available. These have a diameter of 0.25 in. (0.64 cm) and a reported strength of 45 ksi (310.3 MPa). With this strength, the required steel area according to the method described previously is 0.045 in² (0.290 cm²). This area has a diameter of 0.24 in. (0.607 cm), making the #2 bar a good choice. Therefore, the final design of the transverse reinforcement was #2 steel bars spaced at 3.72 in. (9.45 cm) throughout the pile.

The required splice length for the transverse steel can be computed in the same manner as in sub-section 3.2.1. All the values are the same except for the bar diameter, which is 0.25 in (0.64 cm), and the yield strength, which is 45 ksi (310.3 MPa). When these values are put into Equation 3.16, the development length is 7.11 in (18.1 cm). When multiplied by 1.3 to constitute a class B lap splice, the required splice length is 9.25 in (23.5 cm).

3.2.3 Advantages Given to Steel Reinforcement

The comparison of helical members to transverse steel members is not complete without mentioning the advantages that steel hoops have in this experiment. There are three main advantages. First is orientation. The helical members are at an angle to the cross-section of the pile. The steel hoops are parallel to the cross-section, which gives them the ideal orientation to confine the concrete. The second advantage is strength. The
design analysis assumed that the strength of the steel was 45 ksi (310.3 MPa). This is a design strength, however, which has factors of safety built in to ensure safe design when used in industry. The actual strength is much higher. By contrast, the strength of the carbon composite is based on an experimental average strength with no factors. The third advantage is circumferential containment. Earl [8] showed that spacing and area of circumferential reinforcement are critical to control containment failure of concrete cylinders. The required lap splice of the steel hoops adds 45% extra area to the containment reinforcement in the steel reinforced pile. This should be considered when comparing results of the IRC and SRC tests.
CHAPTER 4 – SPECIMEN FABRICATION

The manufacturing process of the test specimens is detailed in this chapter. Past experience with fabrication of large diameter IsoTruss® grid structures is briefly presented to show how the manufacturing method was chosen for the IsoPile™ reinforcement. The IsoTruss® was filament wound by hand on a mandrel and cured in a temporary wooden oven. The steel reinforcement was tied in the same manner as is done in industry. Several strain gages were mounted on the specimens with appropriate mechanical protection. The instrumented reinforcement cages were inserted in cylindrical cardboard concrete forms. Holes cut in one side of the horizontal forms were used to place the concrete.

The IsoTruss® reinforcement test specimens were manufactured and prepared for testing in a similar fashion as previous testing (see Scoresby [2] and Rackliffe [7]). Vinyl ester resin caps reinforced with chopped fiber were used on the IsoTruss® reinforcement compression specimens (IT). This chapter explains lessons learned from past manufacturing experience, and details the manufacturing of all test specimens for this research: IsoTruss® reinforcement (IT), IsoPiles™ (IRC), steel-reinforced piles (SRC), and the microscopic investigation specimens (QC).
4.1 PAST MANUFACTURING EXPERIENCE – SOLAR BOOM

In the summer of 2003, Renewable Energy Corporation commissioned Brigham Young University’s Center for Advanced Structural Composites (CASC) to design and manufacture an IsoTruss® boom that would hold a prototype of their newest solar energy collector. The main section of the boom was approximately 20 ft (6.1 m) with a 5 ft (1.5 m) extension at a 45° angle from the main section. Without a continuous interweaving IsoTruss® manufacturing machine, the only way to achieve fiber interweaving is through filament winding. When a filament winding process is conducted, an internal mandrel must be used. The mandrel must provide support at the nodes and have endplates suitable for anchoring the fiber during each pass of the filament winding process.

For several years, CASC has used vinyl ester resin to manufacture heads that can be used to support the nodes of the IsoTruss® before curing. The heads are designed in such a way that grooves make it easy to guide the fiber into place without damage. Vinyl ester has sufficient strength properties to endure the manufacturing process and the curing temperature. The heads are usually placed on dowels inserted into a pipe running down the center of the IsoTruss® grid structure. Therefore, the internal mandrel consists of a pipe, dowels, and heads. When the dowels are inserted into the pipe, the overall weight of the mandrel is increased because an internal core is required for the dowels to rest on. Also, the versatility of the mandrel is limited because the drilled holes limit the ability to adjust the bay length; thus only one specific IsoTruss® configuration can be manufactured on this type of pin mandrel.
4.1.1 New Mandrel Concept

A mandrel concept developed previously by CASC was used to manufacture the solar boom IsoTruss® grid structure. Collars with drilled holes are placed around the outside of the pipe. The pipe acts as the core when the dowels are inserted in the holes in the collars. This decreases overall weight of the mandrel, which is a concern for deflection control in horizontal filament winding. This also makes the mandrel useful for manufacturing IsoTruss® grid structures of various bay lengths.

A 4.0 in (10.16 cm) diameter aluminum tube was selected for the mandrel core. An analysis of the tube was performed to ensure undesirable deflection would not occur during manufacturing. The pipe had to be detachable at the joint in order to facilitate mandrel removal after curing. In order to do this, plates with bolt holes were welded on the ends of each section of the pipe at the 45° joint. Figure 4.1 shows the assembled pipe with the pin-positioning collars and end plates.

![Figure 4.1 Pipe for Solar Boom Mandrel](image)

Figure 4.1 Pipe for Solar Boom Mandrel
The black rings in Figure 4.1 are the collars that position the node pins. The collars were made out of ABS pipe with an inner diameter approximately equal to the outer diameter of the aluminum tube. Because the inner diameter of the ABS pipe did not allow the collar to slide freely over the aluminum pipe, the pre-drilled collars were slit as shown in Figure 4.2. A better method would be to machine the inner diameter of the collars to provide the proper clearance.

![Figure 4.2 Drawing of End View of Pin-Positioning ABS Collar](image)

Wooden dowels were used to support the heads because they are relatively easy to cut for removal upon completion of fabrication. The IsoTruss® grid structure, cured, is quite rigid. The mandrel is removed by moving the heads inward away from the nodes, and pulling the mandrel out one end. A mandrel with automatically collapsing pins significantly reduces manufacturing time by removing the steps of cutting new dowels for each new truss and cutting the dowels for mandrel removal. Such mandrels, however, are too expensive for prototype development.
4.1.2 Connection Hardware

The solar boom needed to be able to mate with specific hardware already designed for the solar collector. To accomplish this, endplates were designed around which the fiber could be wound during the manufacturing. The hardware was welded to a small section of aluminum pipe which was welded to the endplates. This enabled the connection hardware to be in place on the mandrel before manufacturing of the IsoTruss® began. The welded pipe was integrated into the final structure; therefore, it had to pass a strength analysis. Also, the endplates had to be designed with rounded edges where the fiber passed over them to minimize fraying of the fiber during manufacturing. Figure 4.3 shows the base plate integrated into the Solar Boom IsoTruss® for simple attachment to the dish assembly. Figure 4.4 shows the end plate for attachment of the collector.

![Figure 4.3 Solar IsoTruss® Boom Base Plate](image)
4.1.3 Manufacturing Process

The filament winding was slightly automated by a motor connected to a gearbox, which was in turn connected to the mandrel. The gearbox allowed slower rotation of the mandrel with more torque. Fiber rolls were held manually while the fiber bundles were manually guided over the heads. The process used to manufacture the IsoPile™ reinforcement, which was very similar to the IsoBoom™ manufacturing process, is explained in greater detail in later sections.

TCR T300C 200 NT 12K tow carbon fiber pre-impregnated with Thiokol UF3325-95 epoxy resin was used for the solar boom IsoTruss®. The IsoBoom™ was consolidated using Dunstone Hi-Shrink Tape. This allowed greater consolidation of fiber as the tape shrunk at curing temperature. Some of the tape was release-coated with Teflon®. Because not all of the tape was release-coated, plastic cling wrap was placed
over the fiber before the shrink tape in an attempt to prevent the resin from bonding to the tape. The entire IsoTruss® grid structure was cured in an oven made of particle board by blowing hot air in each end using gas heaters. This process is explained in greater detail in later sections. The final weight of the structure without end fixtures was 26.10 lb (11.97 kg). The endplates and hardware added significantly to the weight. The weight of the entire structure was 66.75 lb (30.16 kg) (see Figures 4.5 and 4.6). Table 4.1 compares the IsoTruss® boom to an equivalent steel design.

![Completed Solar IsoBoom™](image)

**Figure 4.5 Completed Solar IsoBoom™**

**Table 4.1 Comparison of IsoBoom™ to Equivalent Steel Structure**

<table>
<thead>
<tr>
<th>Structure</th>
<th>Weight [lb (kg)]</th>
<th>Guyed</th>
</tr>
</thead>
<tbody>
<tr>
<td>IsoBoom™</td>
<td>66.75 (30.16)</td>
<td>No</td>
</tr>
<tr>
<td>Steel Boom</td>
<td>165.0 (74.84)</td>
<td>Yes</td>
</tr>
</tbody>
</table>
4.1.4 Lessons Learned From Solar IsoTruss® Boom

Several important lessons were learned from the manufacturing of the solar boom IsoTruss® grid structure. First, ABS pipe is not a strong enough material to be used for the collars of the mandrel. The ABS pipe is not thick enough, and the ABS softens slightly during curing. This caused parts of the IsoTruss® to be slightly offline or deformed. Aluminum collars with set screws to hold them in place would be a better design. While a collapsible pin would be preferable, wood dowels performed satisfactorily. The second lesson learned was that the cling wrap did not prevent bonding of the resin to the shrink tape. Great effort and time was still required to remove all the shrink tape from the structure. Test cures since the fabrication of the solar boom have shown that Teflon® coated shrink tape releases at least as well as, if not better than, the combination of cling wrap and shrink tape.
4.2 FABRICATION OF ISOTRUSS® REINFORCEMENT

The IsoTruss® grid structures used for reinforcement in the IsoPile™ were manufactured almost exactly as the IsoBoom™. The mandrel and process were similar, incorporating minor changes according to the lessons learned. The entire IsoTruss® reinforcement manufacturing process is detailed in this section: mandrel, filament winding process, consolidation, curing, and cutting.

4.2.1 Mandrel

The mandrel consisted of an aluminum tube core, wooden dowels, vinylester heads, aluminum collars, and special aluminum endplates.

4.2.1.1 Core

Aluminum tube was selected for the mandrel core because of its relatively high stiffness and low weight. Titanium would be even better, except for the cost. The diameter and wall thickness of the pipe were determined by considering two factors. The first consideration was what sizes were readily available and could be shipped within a few days. A 6.0 in (15.24 cm) outer diameter pipe with a 0.13 in (0.32 cm) wall thickness was available.

The second consideration was deflection. The pipe had to be stiff enough to keep deflections within an acceptable range during the manufacturing process. This was not a trivial consideration as the unsupported length of the pipe would be more than 30 ft (9.14 m) and it was desired that the deflection be limited to less than 0.5 in (1.27 cm).

An estimate of the loads on the mandrel during fabrication is detailed in Table 4.2 below. The continuous loads are applied along the entire length of the pipe. The point load is applied in the middle of the pipe, as that location produces the most severe
moment. The fiber tension load is based on an assumption of 5.0 lb (22.4 N) per tow and a maximum of six tows being applied at once.

Table 4.2 Summary of Manufacturing Loads on Mandrel

<table>
<thead>
<tr>
<th>Source</th>
<th>Load Type</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipe</td>
<td>Distributed Gravity</td>
<td>2.76 lb/ft (40.3 N/m)</td>
</tr>
<tr>
<td>Collars</td>
<td>Distributed Gravity</td>
<td>1.09 lb/ft (15.9 N/m)</td>
</tr>
<tr>
<td>Heads and Dowels</td>
<td>Distributed Gravity</td>
<td>2.00 lb/ft (29.2 N/m)</td>
</tr>
<tr>
<td>Fiber</td>
<td>Distributed Gravity</td>
<td>2.60 lb/ft (37.9 N/m)</td>
</tr>
<tr>
<td>Total Distributed Load</td>
<td></td>
<td>8.45 lb/ft (123.3 N/m)</td>
</tr>
<tr>
<td>Fiber Tension</td>
<td>Point</td>
<td>30.0 lb (133.4 N)</td>
</tr>
</tbody>
</table>

The distributed loads are gravity loads. The manual filament winding process causes the fiber tension point load to be applied parallel to the ground (see Figure 4.7). AISC [23] defines the maximum deflection of a fixed-pinned beam with a uniform load as:

\[ \delta_{\text{max}} = \frac{qL^4}{185EI} \]  

where q is the distributed load, L is the length of the beam, E is the modulus of elasticity of the material, and I is the moment of inertia of the beam. The pipe was clamped at both ends during the manufacturing process, but modeled as a fixed-pinned beam to be more realistic. The moment of inertia of a 6.0 in (15.2 cm) outer diameter pipe with a 0.13 in
(0.32 cm) wall thickness is 9.96 in\(^4\) (414.50 cm\(^4\)). The distributed load is the total of all the distributed loads in Table 4.2, which is 8.45 lb/ft (123.3 N/m). The length is 30.25 ft (9.22 m). This length was chosen to be conservative. The pipe is 6061-T6511 aluminum which has a modulus of elasticity of 9,990 ksi (68,900 MPa). Putting all these values into Equation 4.1, the maximum deflection due to distributed loads is 0.66 in (1.69 cm).

![Distributed Loads](image1)

**Figure 4.7 Manufacturing Loads on Mandrel**

AISC [23] defines the deflection of a fixed-pinned beam due to a point load in the center as:

\[
\delta_{\text{max}} = \frac{PL^3}{48EI\sqrt{5}}
\]  

(4.2)
where P is the point load. When the properties of the pipe are put into Equation 4.2 with the point load from Table 4.2, the deflection due to fiber tension during manufacturing is 0.13 in (0.34 cm). The total deflection is the resultant of the deflection due to gravity computed from Equation 4.1 and the deflection due to the fiber tension computed from Equation 4.2. This resultant is 0.67 in (1.71 cm). Although more than desired, this is an acceptable deflection that shouldn’t significantly damage the fiber during fabrication.

4.2.1.2 Dowels

A quick analysis and past experience showed that 0.375 in (0.953 cm) diameter wood dowels would be sufficient. The length of the dowels was determined by the diameter of the pipe, the diameter of the IsoTruss®, and the geometry of the head. The head design has 0.204 in (0.518 cm) of material between the bottom of the dowel hole and the top of the head. By using Equation 3.10 and simple geometry, the theoretical diameter of the helical members was determined to be 0.35 in (0.88 cm). Assuming less than perfect consolidation, the distance from the top of the head to the middle of the helical member should be approximately 0.20 in (0.51 cm). These dimensions yield a dowel length of 3.10 in (7.86 cm) (see Figure 4.8).
4.2.1.3 Heads

The heads were cast out of vinyl ester resin in a silicone mold. The geometry of the head was determined from the solver values explained in sub-Section 3.1.2. The grooves were rounded and enlarged as shown to encourage the fiber to form a round cross-section and to provide a cross-sectional area slightly larger than the design value to allow for manufacturing deviations. The diameter of the groove for the helical members is 0.2 in (0.6 cm), which is 16% larger than the design diameter of the helical members.
A model of the design was created in Catia® (see Figure 4.9). The drawing was input into a rapid prototype machine to create a prototype of the head made of starch coated in resin (see Figure 4.10).

Figure 4.9 Drawing of Head Design

Figure 4.10 Rapid Prototyped Head
The prototype head was mounted on an aluminum plate and the plate was attached by screws to a short section of ABS pipe (see Figure 4.11a). Using the pipe and the head as a mold, a silicone mold with a single chamber was created. Using the mold in Figure 4.10(a), nine prototype heads were cast in a manner similar to the process shown in Figure 4.11(b). The prototype heads were used to create a larger mold with nine chambers, as shown in Figure 4.12.

![Figure 4.11 Single Chamber Silicone Mold: (a) Mold; and, (b) Pouring Prototype Head](image1)

![Figure 4.12 Prototype Heads in Mold Before Silicone is Poured](image2)
Figure 4.12 shows that many of the prototype heads were metallic (Cerrobend®). A mold was made using these metallic heads; however, contraction upon cooling caused imperfections and led to inferior mold quality. Therefore, another nine chamber mold was made with vinylester heads. This larger mold made it possible to manufacture many heads relatively quickly. The mold was carefully manufactured to ensure that any head made in the large mold had the proper dimensions.

4.2.1.4 Collars

The collars were made from aluminum to prevent deformation at the curing temperature. Eight holes are evenly spaced around each collar. There is one collar for each bay of the IsoTruss®. Each hole corresponds to one node (see Figure 4.13). The holes are 0.375 in (0.953 cm), to match the diameter of the dowels. The wooden dowels deform slightly when entering the holes, creating a tight fit. The collars are 0.475 in (1.21 cm) thick in order to provide lateral stability to the dowels. The inner diameter of the collars is slightly larger than the other diameter of the pipe. This ensures that the collars will slide to the desired position on the mandrel. Each collar was secured to the pipe with four set screws.
4.2.1.5 Endplates

Special aluminum endplates were designed for the manufacturing process (see Figure 4.14). The IsoTruss® was designed to have nodes at each end. This requires separate grooves for each longitudinal member and each pair of helical members. The entire endplates, including the grooves, were gradually rounded on both sides to prevent damage to the fiber such as the fraying that was experienced during the manufacturing of the IsoBoom™.

![Figure 4.14 Drawing of Endplate Design](image)

The location of the grooves for the IsoTruss® members was easily obtained from the geometry equations defined by Winkel [3]. The grooves were 0.6 in (1.5 cm) in diameter which provides for an area 50% larger than the design area of two helical members computed using Equation 3.10 to allow for manufacturing deviations.
4.2.2 Filament Winding Process

The IsoTruss® specimens were filament-wound in the same manner as the IsoBoom™ described in Section 4.1. The winding pattern, automation, and manual aspect of the filament winding is detailed below.

4.2.2.1 Winding pattern

The IsoTruss® was wound using the common pattern used in past manufacturing. In this pattern, the tows forming the helical members lay across the grooves in the heads. From one bay to the next, the members skip a node. The longitudinal tows follow a straight line parallel to the mandrel and lay on top of intersections of the helical members. Both the longitudinal and helical members are wound around the endplates through their corresponding slots. This is a continuous winding process. The 16 helical members require eight trips down the mandrel and back to place one layer of helicals. Standard practice is to alternate layers of helical and longitudinal members in order to achieve maximum tow interweaving at the joints and increase the overall integrity of the lattice. The process begins and ends with helical members. For a more detailed explanation and diagrams on the standard winding pattern, see Rackliffe [6].

A bundle of tows (instead of just one tow) is placed on each pass along the mandrel in order to reduce manufacturing time. One person is required for every two tows being placed. This is because much of the manufacturing process is not automated. A shortage of personnel made it difficult to keep the number of tows being placed per pass constant. An effort was made to be as consistent as possible. Four or six tows were generally used in each bundle. For a few layers only, two tows were used in a bundle due to lack of personnel. Consequently, the helical members were placed in groups of two,
four or six tows, and the longitudinal members were placed in groups of four, six, or twelve tows.

4.2.2.2 Automation

The only automation in the filament winding process was the rotation of the mandrel using a Howell 3-phase, 220 volt, 3 HP motor. An Omron 3G3MV inverter was used to control the speed of the motor. The motor was also attached to a gear box with a reduction ratio of 20:1. This means that the mandrel could be rotated up to 20 times slower than with the motor, and 20 times more torque was available. The motor was connected to the gear box through existing connections, and the other end of the gear box was bolted to the mandrel (see Figure 4.15).

![Figure 4.15 End of Mandrel with Motor, Gear Box, and Operator](image)

4.2.2.3 Manual filament winding

A long filament winding machine was not available, so the fiber was placed on the mandrel manually. In order to keep the correct angle of departure from the roll of
pre-impregnated fiber, one person could carry a maximum of two rolls of fiber. Without the proper angle of departure, the fiber would fray and eventually break completely. An extra person was also needed to bundle all the tows together and guide them onto the mandrel in the proper pattern. When six tows were placed at once, five people were required: one to run the motor, one to bundle and place the tows, and three to carry two spools of fiber each (see Figure 4.16).

![Figure 4.16 Manual Filament Winding Process](image)

4.2.3 Fiber Consolidation

In order to achieve greater compressive strength, the fiber tows must be properly consolidated (see Hansen [1]). Barbero [20] describes the main mode of compressive failure as fiber micro-buckling. In order to avoid micro-buckling, the fiber must be as straight as possible. To accomplish this, an effort was made to keep tension on the fiber
during the filament winding process. Also, consolidating the fiber straightens its path and allows the fibers to support one another.

Consolidation was accomplished using Dunstone Hi-Shrink Tape. One side of the shrink tape was release coated with Teflon® to prevent bonding with the resin. The tape was wrapped tightly around each member in long strips and anchored by masking tape (see Figure 4.17). The tape was removed from the longitudinal members after curing.

![Figure 4.17 Consolidation of IsoTruss® Members Using Shrink Tape](image)

4.2.4 Curing

The IsoTruss® was cured in a rudimentary particle board oven. The oven ran the length of the structure and was open at each end (see Figure 4.18). Hot air was blown in each end with gas heaters and vented out the middle with the assistance of a fan. Temperature in the oven was monitored by thermometers at regular intervals. The temperature was controlled by altering the distance of the heaters and the fan from the openings in the oven. Prior to curing, the IsoTruss®/mandrel assembly was carefully
leveled and verified by surveying equipment to ensure the IsoTruss® grid structure produced was straight.

![Image](image1.jpg)

(a)

![Image](image2.jpg)

(b)

**Figure 4.18 Plywood Oven for IsoTruss® Curing:** (a) IsoTruss® Grid Structure in Oven; and, (b) Monitoring Oven Temperature During Curing

The resin in the pre-impregnated T-300 carbon fiber was Thiokol UF 3325-95.

The curing specifications for this resin call for a ramp-up of 5°F/min (9°C/min) to a temperature of 290°F (143.3°C). This temperature was held for two hours, followed by a cool-down of 5°F/min (2.8°C/min). Once the temperature during the ramp-down reached 150°F (65.6°C), the specimen was removed from the oven. Although precisely controlling the temperature was difficult in such a primitive oven, the required temperature was achieved and maintained for the proper time. An average temperature
gradient of about 30°F (16.7°C) existed between the ends of the oven and the middle. Tables showing recorded temperatures from the cures are contained in Appendix C.

4.2.5 Cutting IsoTruss® Specimens

The mandrel was designed to produce two IsoTruss® grid structures exactly 30.0 ft (9.14 m) long. A short length was cut off each of the IsoTruss® structures to test in axial compression. The most desirable location to cut an IsoTruss® grid structure is at the triple joints, as shown in Figure 4.19.

![Figure 4.19 Preferred End Cut Locations on an IsoTruss® Grid Structure](image)

The two original 30.0 ft (9.14 m) long IsoTruss® structures shown in Figures 4.20(a) and 4.21(a) were cut into the lengths shown in Figures 4.20(b) and 4.21(b). The shortest resulting IsoTruss® structures became the IsoTruss® compression specimens (IT). The longer specimen in Figure 4.20(b) became the in-situ IRC bending test specimen, and the two longer specimens in Figure 4.21(b) became the IRC lab bending test specimens. The QC specimens were all taken from the second IsoTruss® Specimen in Figure 4.21(a). This should be considered when analyzing the microscopic inspection results in Chapter 6. It is assumed that the properties summarized in the inspection are
uniform for all IsoTruss® specimens in this research, because they were all manufactured using the same process.

![Figure 4.20](image1)

**Figure 4.20** Cutting of First IsoTruss® Structure:
(a) As Manufactured; and, (b) As Tested

![Figure 4.21](image2)

**Figure 4.21** Cutting of Second IsoTruss® Structure:
(a) As Manufactured; and, (b) As Tested

4.2.6 IsoTruss® Compression Specimens

In a simple compression test, it is imperative that all the longitudinal members are equally loaded. This is accomplished by ensuring that the cuts of the longitudinal members are straight and in the same plane. The swivel head on the testing machine can
compensate if the two planes at each end are not exactly parallel. The ends of the IsoTruss® (IT) specimens were sanded on a disk sander to make all the cuts even and coplanar.

4.2.7 In-Situ Bending Test IRC Reinforcement

The first IsoPile™ was tested in the field [25]. Figure 4.20 shows the cuts that were made to create the IRC in-situ bending test specimen. The IsoTruss® grid structure used for reinforcing the in-situ specimen was about 26.90 ft (8.20 m) long. A short rebar cage was spliced onto one end of the field bending test specimen IsoTruss® grid structure (see Figure 4.22) to make the final specimen exactly 30 ft (9.14 m) long after removal of the IsoTruss® tests specimen. The longitudinal steel bars were overlapped on the inside of the longitudinal members of the IsoTruss®. The splice length used was the same as determined in Section 3.2. The sizes of the longitudinal and transverse steel were the same as in the steel reinforced piles. The spacing of the transverse steel was also the same. The diameter of the steel rings was slightly larger because of the larger diameter of the IsoTruss® longitudinal member group.

Figure 4.22 Steel Extension to IsoTruss® Reinforcement
The final weight of the field bending test specimen without the steel extension is 75.5 lb (34.2 kg) or 2.8 lb/ft (4.2 kg/m). When the steel was added to the end, the weight increased to 107.1 lb (48.6 kg) or 4.0 lb/ft (5.9 kg/m).

4.2.8 Lab Bending Test IRC Reinforcement

The second original 30 ft (9.14 m) long IsoTruss® was cut into three pieces and the scrap was removed (see Figure 4.21). The resulting two IsoTruss® structures intended for use in the lab bending test IRC specimens were 13.3 ft (4.05 m) long. The weights of the two specimens were almost identical. The specimen that was used in IRC-1 weighed 37.1 lb (16.8 kg). The specimen that was used in IRC-2 weighed 37.2 lb (16.9 kg). These weights are equivalent to 2.8 lb/ft (4.2 kg/m).

4.3 FABRICATION OF STEEL REINFORCEMENT

The dimensions and type of steel reinforcement selected were explained in Chapter 3. The longitudinal bars were cut to the desired length using a standard rebar cutter and anchored to the transverse hoops by tie wire in an evenly spaced, 8-bar pattern. Obtaining the proper cage lengths for the field specimen was not difficult. The transverse steel hoops were manufactured by Bowman & Kemp Steel and Supply in Ogden, Utah. As discussed in Chapter 3, the inner diameter of the hoops is 9.0 in (22.86 cm). The assembly of the steel reinforcement cages for the steel reinforced concrete pile specimens is detailed in this section.

4.3.1 In-Situ Bending Test SRC Reinforcement

The in-situ bending test specimens were 30.0 ft (9.14 m) long. The #4 bars used for longitudinal reinforcement had a maximum available length of 20.0 ft (6.10 m). This
required standard splices to be made in the longitudinal bars. The splices were alternated each bar so four of the splices were at one end of the pile, and four were at the other end.

The weight of the steel cages was estimated from the known weights of the bars. The total weight of the assembled field bending test cage (see Figure 4.23) was approximately 219 lb (99.2 kg) or 7.3 lb/ft (10.9 kg/m).

Figure 4.23 Steel Reinforcing Cage for Bending Test Specimens

4.3.2 Lab Bending Test SRC Reinforcement

The reinforcement cage for the lab bending test steel reinforced specimens was assembled as described in Section 4.4. The length of the two specimens was 13.3 ft (4.0 m), the same as the lab bending test IRC specimens. The total weight of the steel reinforcement cages in the lab bending test SRC specimens was approximately 97 lb (44.0 kg) or 7.3 lb/ft (10.9 kg/m).
4.4 REINFORCEMENT STRAIN GAGES

This Section describes the strain gages used on the reinforcement in this research and the mounting process. First, the IsoTruss® compression specimen gages are discussed, followed by the IsoPile™ and steel-reinforced concrete pile strain gages.

4.4.1 IsoTruss® (IT) Compression Specimens

Strain gages were applied to the longitudinal members of the specimens. The surface of the longitudinal members was prepared by sanding lightly and rinsing with water. Micro Measurements CEA-06-250UN-350 gages were affixed to the longitudinal members using cyanoacrylate (CN) adhesive. Waterproofing and mechanical protection were not required for these gages because the compression specimens were not encased in concrete. These gages did not have integral lead wires, therefore wires had to be soldered to the terminals on the gages.

4.4.2 Pile Specimens (IRC and SRC)

The lab and field test specimens were fitted with Texas Measurements FLA-3-11-3LT strain gages. The gages were mounted on the longitudinal members of the IsoTruss® reinforcement. These particular gages have lead wires already attached, eliminating the need for soldering.

Application of the gages was a multi-step process. First, the area where the gage was to be applied was prepared. In order to function properly, the gages require a smooth area to which to adhere. The surface preparation was not difficult to accomplish on the IsoTruss® structures. The carbon composite structure was sanded lightly (to avoid damaging the fibers) and cleaned with a small amount of acetone applied with a cloth. Preparation of the longitudinal steel bars for strain gage application was more time-
-consuming than the preparation of the IsoTruss®. Steel reinforcing bars have ribs that were removed with a power grinder before sanding (see Figure 4.24). The sanded area on the steel bars was cleaned using acetone. The gages were applied to the prepared areas using cyanoacrylate (CN) adhesive (see Figure 4.25).

Figure 4.24  Preparing Steel for Strain Gages

Figure 4.25  Application of Strain Gage Using Cyanoacrylate
Mechanical protection and waterproofing was required for these strain gages since the reinforcement cage was to be encased in concrete. First, the exposed lead wires were insulated from the reinforcement by vacuum tape. Hot wax was painted onto the strain gage and lead wires. Finally, vacuum tape was used to cover the wax. Mechanical protection was applied to the gages after waterproofing. A thin layer of Standard Araldite A epoxy was mixed and applied to the waterproofed strain gage. Upon hardening, the epoxy provided protection from potential damage during concrete pouring (see Figure 4.26). Electrical tape was wrapped around the lead wires for protection. This protection extended along the entire length of the wires that would come into contact with concrete. All the lead wires exit the same end of the piles.

![Figure 4.26 Waterproofed Strain Gage](image)

4.5 PILE MANUFACTURING

This section details the manufacturing process for the SRC and IRC piles after the reinforcement was completed with strain gages attached. The installation of inclinometer casing is presented first, followed by concrete placement.
4.5.1 Inclinometer Casing

Special casing was installed in the center of the bending test specimens to take inclinometer readings. The casing allows an inclinometer to pass through the middle of the pile and give an accurate slope profile. The casing was Slope Indicator® standard casing with an outer diameter of 2.75 in (7.0 cm) and an inner diameter of 2.32 in (5.89 cm). The casing was centered using temporary wood blocks (see Figures 4.27 and 4.28). The inclinometer casing is continuous throughout the center of the In-Situ bending test IRC specimen and was anchored to three sides of the reinforcement at regular intervals using steel tie wire. The size of the casing employed only allows a vertical inclinometer reading. For this reason, the inclinometer is only able to be used in the field pile tests. The casing was also installed in the lab specimens, however, to maintain consistency between the in-situ and lab tests.

Figure 4.27 Installation of Inclinometer Casing Showing Temporary Wood Spacers
4.5.2 Concrete Placement

The steel and IsoTruss® reinforcement cages were encased in concrete to complete the fabrication process. First, the cages were properly centered in the column forms. The ends of the forms were capped and the concrete was poured through holes cut in the side of the forms. The forms are described below along with the concrete placement process.

4.5.2.1 Pile forms

Cardboard Kolumn Forms™ were purchased from Caraustar®, with an inner diameter of 14.0 in (35.6 cm). The forms were ordered in 30.0 ft (9.14 m) lengths, the length of the In-Situ bending test specimens.

Chairs were used to center the IsoTruss® in the forms (see Figure 4.29). The chair height was equal to the concrete cover on the longitudinal bars. For the steel reinforcement this dimension is 2.5 in (6.35 cm). In the IsoPile™, the cover was about 1.25 in (3.18 cm). The chairs were attached to the longitudinal bars by tie wire. Longitudinal movement of the cage was prevented by tying it to the form at each end.

Figure 4.28 Inclinometer Casing Centered Inside IsoTruss® Reinforcement
4.5.2.2 Concrete

Eagle Precast Company placed the concrete in the piles. After the reinforcement was secured, the forms and reinforcement were transported by truck to Eagle’s manufacturing facility in Salt Lake City. Six piles were poured, including two 30 ft (9.14 m) piles (one each with IsoTruss® and steel reinforcement); and four piles that were just over 13 ft (4.0 m) (two each with IsoTruss® and steel reinforcement).

Eagle Precast employees were concerned that the reinforcement cages or instrumentation would be damaged if the piles were oriented vertically during placement of the concrete. The only way to pour the 30 ft (9.14 m) piles vertically would be to insert a tube to the bottom and slowly raise it as the form fills with concrete. This minimizes damage from falling concrete. There were concerns, however, that the tube could become entangled as it was extracted. This could have severed tie wires and caused the cage or the inclinometer pipe to be off center. Therefore, the concrete was poured with the piles oriented horizontally.
The first step was to create cradles to stabilize the forms. Also, several collars were positioned along the length to prevent distortion of the form. The forms are designed to be poured vertically and therefore are sufficiently strong in the radial direction. When laid on their side, however, the weight of the concrete causes the forms to distort into an oval shape. The reinforcing collars helped to minimize distortion. Also, caps were made for the ends of the piles. The cradles, collars and caps were all made from plywood by Eagle Precast employees.

Rectangular holes were cut in the top of each of the forms through which the concrete was placed. One was cut in each of the shorter forms, and two in each of the longer forms. Special troughs were constructed out of plywood and construction lumber to contain and guide the concrete into the forms. These troughs also created a small hydraulic head to force the concrete to the ends of the forms. The forms, caps, and troughs can be seen in Figure 4.30 after their preparation for concrete pour.

Figure 4.30  Forms Prepared for Concrete Pour
Self-consolidating concrete was the logical choice for these piles. This type of concrete flows enough to fill the horizontal forms and has minimal contraction during curing. Eagle Precast reported that the strength of the concrete mix would be approximately 12 ksi (83 MPa). The actual strength, however, was much lower (see Chapter 5). Standard tests were performed on the concrete mix before pouring (see Figure 4.31). The concrete spread test takes the place of a slump test when using self-consolidating concrete. The spread was 24.0 in (61.0 cm). The air entrainment was 7.5% and the temperature was 70°F (21°C).

![Concrete Spread Test](image)

**Figure 4.31 Concrete Spread Test**

The concrete placement is shown in Figure 4.32. Small fill indicator holes were drilled in the top of the forms every few feet over the length of the piles. Concrete oozing out of the holes indicated that the form was full at that location. When concrete flowed through all the holes, pouring was complete (see Figure 4.33). Six 6.0 in (15.24 cm)
cm) diameter concrete cylinders were also made to verify the compression strength of the concrete. The results are discussed in Chapter 5.

Figure 4.32  Concrete Pour

Figure 4.33  Concrete Oozing Out of an Indicator Hole
4.6 PILE COMPRESSION SPECIMENS

The SRC and IRC pile compression specimens are discussed separately because they were taken from other pile specimens. After the lab bending tests, compression strength tests were performed on short sections of the IsoPiles™ and steel reinforced concrete (SRC) piles. These specimens were the same length as the short IsoTruss® compression specimens. As expected, the piles tested in the lab failed in the middle. The ends showed some slight cracking, but were left relatively undamaged. The ends of each of the lab piles were cut off to create the concrete compression specimens. A concrete saw was used to make these cuts. An unsuccessful attempt was made with a saw that was not large enough. This attempt was made on one of the steel reinforced piles. The remaining piles were transported to Eagle Precast and cut on a large concrete saw.

The lead wires for the strain gages on the reinforcement all extend out of the same end of the piles. The specimens that were cut from that end still have functioning strain gages that can be used for data acquisition during testing. When the specimens were cut from the other end, however, the lead wires were severed, preventing internal strain measurements on those specimens.

Four IsoPile™ (IRC) compression specimens and three steel reinforced concrete (SRC) compression specimens were created. Two of the IRC and one of the SRC specimens had functioning internal strain gages.

Small amounts of concrete were chipped off at the edges of the specimens during cutting. Devcon® 5-minute epoxy was used to fill in the holes (see Figure 4.34) and ensure that the entire end of the specimen would contact the fixture during the compression tests.
4.7 QUALITY CONTROL SPECIMENS

The fiber fraction, void fraction and area measurement specimens were made by taking a short section of longitudinal member from the scrapped sections and making a perpendicular cut using the jig manufactured by Hansen [1]. One end of the specimen was encased in Devcon 5-minute epoxy (see Figure 4.35). The polishing was accomplished using 320, 600, 1200, and 2400 grit LECO polishing paper progressively. The cross-section was observed through the microscope to verify surface quality. The polishing process was repeated until clear digital photographs could be taken to conduct the inspections described in Section 2.4.
Figure 4.35 QC-6 After Epoxy Application and Polishing


CHAPTER 5 – TESTING

Compression strength and stiffness testing was performed on sections of the IsoTruss® reinforcing cage as well as sections of the IsoPile™. The IsoTruss® reinforcing sections were tested in pure axial compression on a 300,000 lb (1,330 kN) Baldwin testing machine. The IsoPile™ sections were tested in a specially designed, self-reacting frame that incorporated a 600 kip (2,670 kN) actuator. These two tests determined the strength of the hand-manufactured IsoTruss® reinforcement that was used in the construction of the IsoPiles™.

A microscopic inspection was also performed on cross sections of the longitudinal members of the IsoTruss® reinforcement to determine specimen quality. Photographs of the polished cross-sections were taken and analyzed using computer software. The cross-sectional area of these sections was measured to compare with the design cross-sectional area (see Chapter 3). The void fraction and fiber volume fraction were also measured for comparison with machine-manufactured specimens made by Hansen [1].

The results of all tests and investigations are presented in Chapter 6. This chapter describes the compression testing performed on IsoTruss® reinforcement, the SRC and IRC compression tests, and the microscopic investigation results.
5.1 ISOTRUSS® REINFORCEMENT COMPRESSION TESTS

As explained in Chapter 4, the IsoTruss® reinforcement for the IsoPiles™ was manufactured in two 30 ft (9.1 m) lengths. A section was cut off of the end of each of these lengths to perform axial compression tests. The test specimens were approximately 3 ft (0.9 m) long. Each end of the specimens was encased in a chopped carbon fiber-reinforced vinyl ester resin ring to ensure proper load transfer and control brooming of the longitudinal members during testing. The test fixture and instrumentation are described in the following sections.

5.1.1 Test Fixture

Compression tests of the IsoTruss® reinforcement were performed on the 300,000 lb (1,300 kN) capacity Baldwin testing machine (see Figure 5.1). This machine is equipped with a swivel head to compensate when the two ends of the specimen are slightly less than parallel. A steel plate was used to transfer load between the swivel head and the donut-shaped resin ring. A similar steel plate was used to transfer load between the other resin cap and the base of the test fixture.
5.1.2 End Caps

End caps were manufactured from vinyl ester resin to prevent brooming of the longitudinal members during testing. An aluminum mold was used to fabricate the end caps. The IT specimen was placed in the mold and the resin was poured around the specimen. The resin forming the end caps was reinforced with chopped carbon fiber to minimize cracking of the cap during curing. The cap was a tapered donut shape (see Figure 5.2) with a thickness of 2.0 in (5.1 cm). The mold was tapered for easy removal of the specimen after the cap cured. The outer diameter was 14.13 in (35.9 cm) at the top of the mold and tapered to 14.0 in (35.6 cm) at the bottom. The inner diameter was 5.88 in (14.9 cm) at the top and 6.0 in (15.2 cm) at the bottom.
5.1.3 Instrumentation

As explained in Section 4.7, each IsoTruss® specimen was equipped with nine Micro Measurements CEA-06-250UN-350 strain gages. The gages were applied to the longitudinal members at exactly the midpoint of the length. Eight of the gages were mounted on the outside of the members, and one gage was applied to the inside of one of the members as shown in Figure 5.3. Four Patriot Sensors model P-20A linear motion transducers were used to measure axial displacement. These transducers, with an error of ±0.1%, were attached to the metal plates at each end and spaced evenly around the outside of the specimen. The wires from the transducers can be seen in Figure 5.1.
Figure 5.3 End View of IsoTruss® Compression Specimens Showing Locations of Strain Gages and Linear Motion Transducers

5.2 SRC AND IRC COMPRESSION STRENGTH TESTS

The IsoTruss®-reinforced concrete (IRC) and steel reinforced concrete (SRC) piles manufactured for this research were intended for flexure tests. The in-situ bending test specimens and the lab bending test specimens were tested, and the results are reported separately (see Ferrell [25] and Richardson [26]). After the short specimens were tested, each end was cut off to be tested in compression. The nature of a four-point bending test ensured that the ends of the piles were relatively undamaged. The ends that were cut off became the IRC and SRC compression strength specimens. The concrete and steel material strength tests, the pile compression test fixture, the end caps, and the instrumentation of the IRC and SRC compression specimens are described in this section.

5.2.1 Material Strength Tests

Material strength tests were performed to facilitate ultimate load predictions for the IRC and SRC specimens. The concrete compression and steel rebar tension tests are described in the following sections.
5.2.1.1 Concrete

At the time of the concrete pour, four 6.0 in (15.2 cm) diameter concrete specimens were made to determine the concrete strength. These specimens were capped with standard sulfur caps. Testing was performed on the Baldwin testing machine mentioned in Section 5.1.1. The results are summarized in Chapter 6.

5.2.1.2 Steel

Three sections of grade 60 #4 steel rebar were tested in tension to determine the material strength of the steel used to manufacture the SRC piles. The rebar sections were about 12.0 in (30.5 cm) long. The steel was tested in an MTS Systems 110 kip (489 kN) capacity testing machine. The rebar was secured using the MTS 647 hydraulic wedge grip on the testing machine (see Figure 5.4). Test results are summarized in Chapter 6.

Figure 5.4 Rebar in MTS Testing Machine
5.2.2 Compression Test Fixture

A self-reacting steel frame, constructed to test composite columns, was easily modified to test the piles in axial compression. Figure 5.5 shows the test fixture with a 600,000 lb (2,700 kN) capacity actuator. All the specimens except SRC-1 were tested with this configuration. The actuator pushes against a beam that is pinned on one side of the frame. The center of the specimen was placed 60.25 in (153.0 cm) from the pin. The total length of the beam between the pin and the actuator is 134.2 in (340.9 cm). This is a ratio of 2.23 to 1. Consequently, the capacity of the test fixture is 600 kip (2,668.9 kN) times 2.23, or 1,340 kip (5,950 kN).

Figure 5.5 Test Fixture for Concrete Compression Specimens
The first steel-reinforced concrete compression strength specimen, SRC-1, was tested in the same fixture without the actuator. Instead, two 1,000,000 lb (4,450 kN) capacity jacks (see Figure 5.6) were positioned in the middle of the beam. The pin was removed to allow the beam to slide freely along the fixture. This configuration was used only once because of difficulties keeping the load equal between the two jacks.

![Figure 5.6 1,000,000 lb (4,450 kN) Capacity Jacks](image)

There were other slight variations of the test fixture between tests. For example, the first tests began with the lever arm perpendicular to the face of the specimen. In later tests, the actuator was started further back so that failure of the specimens occurred about the time the lever arm achieved perpendicularity with the specimen. All the variations along with their possible implications are explained and discussed in Chapter 6.
5.2.3 End Caps

Capping the specimens with vinyl ester or sulfur would have proved very difficult due to their weight and size; therefore, an un-bonded rubber surface was used. The rubber deflects enough to allow uniform distribution of the load along the surface of the specimen. ASTM International has published specifications that allow the use of un-bonded caps in the determination of compressive strength of concrete cylinders [22]. The specification requires that the pads be made of 0.5 in (1.27 cm) thick polychloroprene (neoprene). The concrete strength of the specimens requires a neoprene pad hardness of 70 durometer. The retaining rings for the pads were 15.0 in (38.1 cm) in diameter and 1.0 in (2.54 cm) high. This diameter is 107% of the diameter of the specimens and pads. The specification states that the retaining ring diameter shall not be more than 107% of the pad diameter. A maximum number of reuses of the pads is also specified. This condition did not apply because new pads were used for each test, due to extensive damage of the pads.

All concrete compression tests strictly adhered to the specifications set forth except for the pad thickness. SRC-1 was tested with 0.5 in (1.27 cm) thick pads. The remainder of the specimens used 1.0 in (2.54 cm) thick pads because of limited neoprene availability.

5.2.4 Instrumentation

The wires leading to the internal strain gages on the lab flexure specimens all exit the pile on the same end. When the IRC and SRC specimens were cut from one end, the wires leading to the internal strain gages on the other end were severed. Therefore, only two each of the IRC and SRC compression specimens had operational internal gages.
These internal gages are the same ones applied in Section 4.3.1 (Texas Measurements FLA-3-11-3LT).

All pile compression specimens were also equipped with two surface gages mounted exactly opposite each other. The surface gages were TML PL-90-11-IL strain gages. The surface gages were named North and South according to their position in the lab during the test. The internal gages have their original names from the IRC and SRC lab flexure tests. Two linear motion transducers identical to the ones used in the pile compression tests were also used in the concrete compression tests. The transducers were attached between the lever arm and one side of the frame on either side of the specimens. Deflection was determined by averaging the movement recorded by the two transducers. All instrumentation on the pile compression specimens is detailed in Figure 5.7.

Figure 5.7 End and Side View of Instrumentation on IRC and SRC Specimens
5.3 MICROSCOPIC INSPECTION

As explained in Chapter 2, a microscopic inspection was performed to determine the quality of the hand-manufactured IsoTruss® reinforcing specimens. Cross-sectional area, fiber volume fraction, and void fraction were measured using LECO microscopes and IA-32 image analysis software. These three measurements are described below.

5.3.1 Area Measurements

Cross-sectional area measurements were performed using a LECO Olympus SZH photographic microscope (see Figure 5.8). Photographs were taken of the cross-sections of the QC specimens. The cross-section was too large to fit in one photograph, so the area was divided into eight regions (see Figures 5.9 and 5.10). The black line on the right side of Figure 5.10 is the dividing line between Region 7 and 8 of the cross-section of QC-3. The line was colored black to ensure it wasn’t counted in the area measurement of sector 7 (it was already counted in the measurement of Sector 8). The photographs were altered in Paint Shop Pro to include only the area of the region in question. Paint Shop Pro was also used to enhance the photographs to make the background light in comparison to the specimen area. These enhanced photographs were imported into the IA-32 program. Threshold controls were used in IA-32 to make the cross-section of the specimen red. The percentage of the photograph that was red was calculated. This percentage was measured three times and averaged to reduce the error introduced by the variability in measurements. The area of the entire photograph is 0.096 in² (0.619 cm²). The specimen area in each photograph was calculated by multiplying the percentage of the photograph that was red by the area of the entire photograph. The total specimen area was calculated by adding the areas of each of the eight photographs.
Figure 5.8 LECO Olympus SZH Photographic Microscope

Figure 5.9 Photograph of Part of Cross-Section of QC-3
5.3.2 Fiber Volume Fraction

The polished cross-sections of the QC specimens were viewed through a LECO Olympus PME3 Photographic Microscope at 100X magnification (see Figure 5.11). The specimens had to be polished sufficiently so the fibers could be seen clearly. The fibers appear as white, kidney-shaped objects (see Figure 5.12). The resin and voids are darker colors. A total of 15 photographs per specimen were taken at random locations on the cross-section. The photos were imported to Photoshop® to adjust the contrast using the auto levels feature to increase the accuracy of IA-32 measurements (see Figure 5.13).

Figure 5.11 LECO Olympus PME3 Photographic Microscope
Figure 5.12  Raw Photograph of Cross-Section of QC-4 at 100X Magnification

Figure 5.13  Enhanced Photograph of Cross-Section of QC-4 at 100X Magnification
The enhanced fiber volume fraction photos were imported into the IA-32 software, where the threshold controls were modified. The controls were modified so the fibers appeared green, and the percentage of the screen that appeared green was measured. The threshold controls were modified three times for each picture. This process yielded a total of 45 fiber volume measurements for each QC sample (three measurements per photograph). The 45 measurements were averaged to obtain an average fiber volume fraction for each QC specimen. An average of the values for each QC specimen gives the approximate fiber volume fraction for the IsoTruss® reinforcement.

5.3.3 Void Fraction

Voids account for some of the cross-sectional area of the longitudinal members of the IsoTruss®. QC specimens required no further preparation for void fraction measurements once the fiber volume measurements were taken. Photographs were taken at a magnification of 20X using the same LECO Olympus PME3 Photographic Microscope.

A total of 10 photographs were taken at random locations in the cross section of each QC specimen. These photographs were imported directly into the IA-32 software. Threshold controls were used to color the voids red. The IA-32 software was used to measure the percentage of the photograph that was colored red. The threshold controls were adjusted three times to increase reliability. This process yielded a total of 30 void fraction measurements for each specimen. The average of these measurements yields the approximate void fraction for each QC specimen. The voids appear black in the photos. The dark gray areas are resin concentrations. The voids in Figure 5.14 are not visible
with the naked eye. Most of the specimens contain a large concentration of voids near the center of the member. This tubing effect, shown in Figure 5.15, is easily seen without being magnified under the microscope.

Figure 5.14 Photograph of Cross-Section of QC-4 at 20X Magnification

Figure 5.15 Photograph showing Concentration of Voids in Center of Specimen QC-3
CHAPTER 6 – TEST RESULTS

The results of all tests conducted for this research are presented and discussed, along with statistical information. First, the IsoTruss® compression test results are presented and analyzed, followed by the pile compression test results. Lastly, the microscopic investigations that determined compaction quality are presented.

6.1 STATISTICAL OBSERVATIONS

Statistical observations were made for each set of results in order to determine the reliability of the averages obtained. Construction of confidence and reliability intervals and Chauvenet’s criterion for rejecting a data value are explained below. A more detailed discussion of these and other statistical fundamentals are contained in Appendix D.

6.1.1 Confidence and Reliability

In the figures presenting the data, a probable range of average values is desired. The range represents 90% reliability with a 95% confidence. This means that it can be stated with 95% confidence that 90% of the data falls within the range. Unfortunately, too few test specimens combined with relatively large standard deviations made the range difficult to plot for the IsoTruss® compression specimens and the steel reinforced concrete compression specimens. The reasons for the difficulty are explained in later
sections. In these situations, a range of an integer multiple of the standard deviation (e.g., 3s) is plotted.

6.1.2 Application of Chauvenet’s Criterion

Each data set was examined to determine if any samples were sufficiently inconsistent to be excluded from the set average and standard deviation. Chauvenet’s Criterion could not be applied to the IsoTruss® compression tests, because there was only two data values.

6.2 ISOTRUS® REINFORCEMENT COMPRESSION TESTS

Test data from the IsoTruss® compression tests is summarized in Table 6.1. The ultimate compressive stress and Young’s modulus is explained in greater detail in the following sub-sections.

<table>
<thead>
<tr>
<th>Table 6.1 Test Results of IsoTruss® Compression Specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>IT-1</td>
</tr>
<tr>
<td>IT-2</td>
</tr>
<tr>
<td>Average</td>
</tr>
<tr>
<td>Standard Deviation</td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>
6.2.1 Ultimate Compressive Stress

The average ultimate compressive stress of the IsoTruss® specimens was 49.6 ksi (342 MPa). This value is based on the average measured cross-sectional area of the longitudinal members. This is about half of what Hansen [1] achieved in testing IsoTruss® longitudinal members with interwoven joints consolidated with a polyester shrink tape sleeve. The average ultimate stress that Hansen achieved was 101.3 ksi (698 MPa). The ultimate stress of the IsoTruss® specimens was less than one-third the value for a typical carbon-epoxy composite reported in Barbero [20]: 159.0 ksi (1,096 MPa). The compressive ultimate stresses for the two IsoTruss® specimens were within 5.1 ksi (35 MPa), however, demonstrating consistency. Obviously, voids do not support load. If the void area is removed from the cross-sectional area prior to the stress calculations, the average ultimate stress of the IsoTruss® specimens is 53.8 ksi (371 MPa). The stress values are summarized and compared to Hansen and Barbero in Table 6.2.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Total Area</th>
<th>Net Area Excluding Voids</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ultimate Stress [ksi (MPa)]</td>
<td>% of Hansen</td>
</tr>
<tr>
<td>IT1</td>
<td>47.0 (324)</td>
<td>46.4</td>
</tr>
<tr>
<td>IT2</td>
<td>52.1 (359)</td>
<td>51.4</td>
</tr>
<tr>
<td>Average</td>
<td><strong>49.6 (342)</strong></td>
<td><strong>48.9</strong></td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>3.6 (25)</td>
<td>3.5</td>
</tr>
</tbody>
</table>

1 101.3 ksi (698 MPa)
2 159.0 ksi (1,096 MPa)
Another reason that the ultimate stress values were lower than expected was premature failure of the end caps. The vinyl ester resin on the outside of the longitudinal members spalled off before failure (see Figures 6.1 and 6.2). This caused some brooming of the longitudinal members and most likely led to lower ultimate stress values.

Figure 6.1 Failure of Vinyl Ester End Cap in IT-1

Figure 6.2 Delamination of Composite at Joint of IT-2
Figures 6.3 and 6.4 show the stress-strain curves for IT-1 and IT-2 based on total area. Figures 6.5 and 6.6 show the stress-strain curves for IT-1 and IT-2 based on net area (excluding voids). All strain gages are on different longitudinal members except for gage numbers 8 and 9, which are on opposite sides of the same longitudinal member. Buckling of the members is monitored by checking the strain difference between gage numbers 8 and 9. Figures 6.3 through 6.7 show that the stress-strain curve for gage number 9 on IT-1 deviates slightly from the rest of the gage curves. The deviation could indicate some buckling of the members, but the main failure mode appears to be material crushing. An examination of the failed specimens shows that the material delaminated and crushed at the joints closest to the top of the specimen (see Figure 6.2).

![Graph showing compressive stress vs. strain plots for IT-1 (Gross Area)](image)

**Figure 6.3** Compressive Stress vs. Strain Plots for IT-1 (Gross Area)
Figure 6.4 Compressive Stress vs. Strain Plots for IT-2 (Gross Area)

Figure 6.5 Compressive Stress vs. Strain Plots for IT-1 (Net Area Excluding Voids)
Figures 6.7 and 6.8 show the compressive stress versus strain curves for both IsoTruss® specimens, together with an average of both specimens. Placing a confidence interval at the mean ultimate strength for the IsoTruss® compression specimens was problematic. Even if an interval as broad as 90% confidence and 90% reliability is considered, the $k$ factor from Table D.1 is 15.92. When the $k$ factor and the sample mean and standard deviation for Figure 6.7 of 49.6 ksi (342 MPa) and 3.6 ksi (25 MPa), respectively, are entered into Equation D.3, the confidence interval that results extends below zero. The difficulty of computing a confidence interval indicates there are insufficient data points to reliably estimate within what range of values a large percentage of future specimens would fail. One or two more tests would significantly shorten the confidence interval and make it useful as a data analysis tool. For this reason, an interval representing 2 times the sample standard deviation, $2s$, on either side of the sample mean
is plotted. Figure 6.7 plots the data measuring stress with gross area, and Figure 6.8 plots the data measuring stress with the net area minus the voids. Figure 6.9 compares the averages from Figures 6.7 and 6.8. The slopes of these two lines are 14.3 Mpsi (98 GPa) and 15.5 Mpsi (107 GPa), with an average slope of 14.9 Mpsi (103 MPa).

**Figure 6.7** Compressive Stress vs. Strain Plots for IT Specimens (Gross Area)
Figure 6.8 Compressive Stress vs. Strain Plots for IT Specimens (Net Area Excluding Voids)

Figure 6.9 Average Compressive Stress vs. Strain Plots for IT Specimens (Gross and Net Area Excluding Voids)
6.2.2 Young’s Modulus

The Young’s modulus of the IsoTruss® reinforcement can be predicted using the rule of mixtures as described in Equation 3.9. The modulus of elasticity of the fiber provided by the manufacturer is 33.4 Mpsi (230 GPa). The modulus of the resin is 410 ksi (2,827 MPa). The average fiber volume fraction of the IsoTruss® reinforcement is 50.7%. The average resin fraction is 41.5%. When these values are put into Equation 3.9, the predicted Young’s modulus is 17.1 Mpsi (118 GPa).

The average Young’s modulus for the IsoTruss® reinforcement was 16.0 Mpsi (110 GPa). This value is 78.4% of the Young’s modulus obtained by Hansen and 77.6% of the value reported by Barbero [20]. The difference between the Young’s modulus values for the two IsoTruss® specimens is more pronounced than the difference between the ultimate stress values. IT-2 had a Young’s modulus of 19.0 Mpsi (131 GPa), which is 93.1% of Hansen’s average value and 92.3% of Barbero’s value. The lower value of IT-1’s modulus significantly lowered the average. The Young’s modulus values obtained in the IsoTruss® tests are compared to other modulus values in Table 6.3.
Table 6.3 Young’s Modulus Values for IsoTruss® Tests

<table>
<thead>
<tr>
<th>Sample</th>
<th>Measured Young’s Modulus [Mpsi (Gpa)]</th>
<th>Predicted Modulus$^1$ [%]</th>
<th>Hansen [1]$^2$ [%]</th>
<th>Barbero [20]$^3$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>IT-1</td>
<td>13.1 (90)</td>
<td>77</td>
<td>64</td>
<td>64</td>
</tr>
<tr>
<td>IT-2</td>
<td>16.0 (110)</td>
<td>94</td>
<td>78</td>
<td>78</td>
</tr>
<tr>
<td>Average</td>
<td>14.5 (100)</td>
<td>85</td>
<td>71</td>
<td>71</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>3.9 (14)</td>
<td>12</td>
<td>10</td>
<td>10</td>
</tr>
</tbody>
</table>

$^1$ 17.1 Mpsi (117.9 GPa)
$^2$ 20.4 Mpsi (140.7 GPa)
$^3$ 20.6 Mpsi (142.0 GPa)

A possible explanation for the discrepancy in Young’s modulus between the two IsoTruss® specimens is the specimen preparation. Both specimens were initially cut with a band saw. Each member of IT-1 was sanded and filed individually to ensure that every cross section was on the same plane. IT-2 was sanded on a disk sander and filed by hand. The disk sander allowed the cross sections of the longitudinal members to be more co-planar by sanding all members simultaneously. This allowed the load to be distributed more evenly throughout each member, increasing the apparent modulus.

6.3 CONCRETE PILE COMPRESSION TESTS

Compression tests were performed on the steel reinforced concrete (SRC) and IsoTruss® reinforced concrete (IRC) specimens. These specimens were cut from the ends of the SRC and IRC lab bending test specimens as described in Chapter 4. The
specimens were tested in the self-reacting steel frame described in Chapter 5. The maximum axial load for a concrete column is defined by MacGregor [19] as:

\[
P_0 = 0.85 f'_c (A_g - A_r) + f_y A_r
\]  

(6.1)

where \( f'_c \) is the concrete strength, \( A_g \) is the gross area of the column, \( A_r \) is the area of the reinforcement, and \( f_y \) is the strength of the reinforcement. Table 6.4 shows the input values needed for Equation 6.1 and the results obtained for the predicted ultimate loads for the SRC and IRC specimens. The average compression strength of the IsoTruss® compression specimens was used as \( f_y \) when predicting the strength of the IRC specimens. For the SRC specimens, \( f_y \) is the average yield strength obtained from the steel rebar tension tests, and is reported in the following section with the concrete strength test results. The results of the material strength tests are presented below, followed by the results of the SRC and IRC compression tests. A comparison of the SRC compressive strength to the IRC compressive strength is also presented.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( f'_c ) [ksi (MPa)]</th>
<th>( f_y ) [ksi (MPa)]</th>
<th>( A_g ) [in(^2) (cm(^2))]</th>
<th>( A_r ) [in(^2) (cm(^2))]</th>
<th>( P_0 ) [kip (kN)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>SRC</td>
<td>7.2 (56)</td>
<td>67.8 (467)</td>
<td>153.9 (993)</td>
<td>1.6 (10.3)</td>
<td>1,040 (4,640)</td>
</tr>
<tr>
<td>IRC</td>
<td>7.2 (56)</td>
<td>49.6 (342)</td>
<td>153.9 (993)</td>
<td>1.5 (9.3)</td>
<td>1,010 (4,480)</td>
</tr>
</tbody>
</table>

Table 6.4 Compression Strength Prediction Parameters
6.3.1 Material Strength

The results of the material strength tests are presented in this section. The concrete compression test results are discussed first, followed by the steel rebar tension tests.

6.3.1.1 Concrete

Results of the compression strength tests of the concrete used to manufacture the SRC and IRC specimens are summarized in Table 6.5. The ultimate stress of Sample 3 was significantly less than the other samples. However, the ultra-conservative Chauvenet’s criterion did not allow for the removal of this obviously flawed data from the analysis.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Ultimate Load [kip (kN)]</th>
<th>Ultimate Stress [ksi (MPa)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>216 (963)</td>
<td>7.7 (52.8)</td>
</tr>
<tr>
<td>2</td>
<td>221 (985)</td>
<td>7.8 (54.0)</td>
</tr>
<tr>
<td>3</td>
<td>134 (596)</td>
<td>4.7 (32.7)</td>
</tr>
<tr>
<td>4</td>
<td>245 (1,088)</td>
<td>8.7 (59.7)</td>
</tr>
<tr>
<td>Average</td>
<td><strong>204 (909)</strong></td>
<td><strong>7.2 (55.5)</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Standard Deviation</th>
<th>Ultimate Load [kip (kN)]</th>
<th>Ultimate Stress [ksi (MPa)]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>48 (215)</td>
<td>1.7 (3.8)</td>
</tr>
<tr>
<td></td>
<td>23.7%</td>
<td>24.1%</td>
</tr>
<tr>
<td>Chauvenet Envelope</td>
<td>130 (576)</td>
<td>4.6 (31.3)</td>
</tr>
<tr>
<td></td>
<td>278 (1,239)</td>
<td>9.9 (68.3)</td>
</tr>
</tbody>
</table>
6.3.1.2 Steel

Results of the steel rebar tension strength tests are shown in Table 6.6. Grade 60 steel has a reported yield strength of 60 ksi (414 MPa). The tests of the #4 rebar exhibited yield strength values that were on average 13% higher than the reported yield strength of the steel.

Table 6.6 Results of Steel Rebar Tension Strength Tests

<table>
<thead>
<tr>
<th>Sample</th>
<th>Ultimate Load [kip (kN)]</th>
<th>Ultimate Stress [ksi (MPa)]</th>
<th>Yield Stress [ksi (MPa)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20.7 (92)</td>
<td>105.3 (726)</td>
<td>66.3 (457)</td>
</tr>
<tr>
<td>2</td>
<td>20.6 (92)</td>
<td>105.1 (725)</td>
<td>66.6 (459)</td>
</tr>
<tr>
<td>3</td>
<td>21.3 (95)</td>
<td>108.4 (747)</td>
<td>70.5 (486)</td>
</tr>
<tr>
<td>Average</td>
<td>20.9 (93)</td>
<td>106.3 (733)</td>
<td>67.8 (467)</td>
</tr>
<tr>
<td></td>
<td>Standard Deviation</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.4 (2)</td>
<td>1.9 (13)</td>
<td>2.3 (16)</td>
</tr>
<tr>
<td></td>
<td>1.7%</td>
<td>1.7%</td>
<td>3.4%</td>
</tr>
<tr>
<td>Chauvenet Envelope</td>
<td>20.4 (91)</td>
<td>103.7 (715)</td>
<td>64.6 (445)</td>
</tr>
<tr>
<td></td>
<td>21.4 (95)</td>
<td>108.8 (750)</td>
<td>71.0 (490)</td>
</tr>
</tbody>
</table>

6.3.2 SRC Specimens

Table 6.7 shows the results of the SRC compression tests with a comparison to the predicted load. SRC-2 and SRC-3 failed at loads close to the predicted load. SRC-1 brought the average ultimate load down, failing at only 69.3% of the predicted load. Despite this low value, Chauvenet’s criterion did not allow for the exclusion of SRC-1 from the analysis. The lower ultimate compressive strength value of SRC-1 inflates the standard deviation, which causes the 95% reliability, 90% confidence interval to extend
off the chart. For this reason, an interval representing the sample standard deviation on either side of the sample mean is plotted. More tests would facilitate a better prediction. The structural modulus was calculated using the average strain from all available strain gages and the stress (load divided by gross area). On SRC-1 and SRC-3, only two concrete strain gages were available. SRC-2 had four gages on the longitudinal steel reinforcement in addition to the concrete surface-mounted gages.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Ultimate Load [kip (kN)]</th>
<th>Deviation from Predicted Load&lt;sup&gt;1&lt;/sup&gt; [%]</th>
<th>Ultimate Stress [ksi (MPa)]</th>
<th>Structural Modulus [Mpsi (GPa)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>SRC-1</td>
<td>723.2 (3,217)</td>
<td>69.3</td>
<td>4.9 (33.7)</td>
<td>5.2 (35.5)</td>
</tr>
<tr>
<td>SRC-2</td>
<td>998.8 (4,443)</td>
<td>95.7</td>
<td>6.7 (46.5)</td>
<td>3.8 (26.0)</td>
</tr>
<tr>
<td>SRC-3</td>
<td>999.9 (4,448)</td>
<td>95.8</td>
<td>6.8 (46.6)</td>
<td>3.9 (26.8)</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>907.3 (4,036)</strong></td>
<td><strong>87.0</strong></td>
<td><strong>6.1 (42.3)</strong></td>
<td><strong>4.3 (29.4)</strong></td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>159.4 (709)</td>
<td>15.3</td>
<td>1.1 (7.4)</td>
<td>0.8 (5.3)</td>
</tr>
<tr>
<td></td>
<td>17.6%</td>
<td>17.5%</td>
<td>17.6%</td>
<td>17.8%</td>
</tr>
<tr>
<td>Chauvenet Envelope</td>
<td>687.3 (3,057)</td>
<td>65.9</td>
<td>4.6 (32.0)</td>
<td>3.2 (22.1)</td>
</tr>
<tr>
<td></td>
<td>1,127.3 (5,014)</td>
<td>108.1</td>
<td>7.6 (52.5)</td>
<td>5.3 (36.8)</td>
</tr>
</tbody>
</table>

<sup>1</sup> 1,040 kip (4,640 kN)

SRC-2 and SRC-3 were both cut from opposite ends of the same lab bending test specimen (lab bending test SRC-2). An examination of Table 6.7 reveals that the results for SRC-2 and SRC-3 are very similar. SRC-1 was cut from one end of the other lab bending test specimen (lab bending test SRC-1). Table 6.8 compares SRC-2 to SRC-3.
Removing SRC-1 from the analysis lowers the standard deviation as percentage of the mean from 17.6% to virtually zero.

### Table 6.8 Strength and Stiffness Results for SRC-2 and SRC-3

<table>
<thead>
<tr>
<th>Sample</th>
<th>Ultimate Load [kip (kN)]</th>
<th>Fraction of Predicted Load[^1] [%]</th>
<th>Ultimate Stress [ksi (MPa)]</th>
<th>Structural Modulus [Mpsi (GPa)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>SRC-2</td>
<td>998.8 (4,443)</td>
<td>95.7</td>
<td>6.7 (46.5)</td>
<td>3.8 (26.0)</td>
</tr>
<tr>
<td>SRC-3</td>
<td>999.9 (4,448)</td>
<td>95.8</td>
<td>6.8 (46.6)</td>
<td>3.9 (26.8)</td>
</tr>
<tr>
<td>Average</td>
<td><strong>999.4 (4,445)</strong></td>
<td><strong>95.8</strong></td>
<td><strong>6.1 (42.3)</strong></td>
<td><strong>3.9 (26.5)</strong></td>
</tr>
<tr>
<td></td>
<td>0.8 (3)</td>
<td>0.1</td>
<td>0.1 (0.5)</td>
<td>0.1 (0.5)</td>
</tr>
<tr>
<td></td>
<td>0.1%</td>
<td>0.1%</td>
<td>0.1%</td>
<td>0.1%</td>
</tr>
</tbody>
</table>

[^1]: 1,040 kip (4,640 kN)

Figure 6.10 shows the stress vs. strain plots with the average for all SRC specimens. The unequal loading of SRC-1 actually increased the apparent Young’s modulus and made the average higher than it would have been otherwise. SRC-2 and SRC-3 had essentially the same modulus. Additional observations for each individual SRC compression test are detailed below.
6.3.2.1 SRC-1

As explained in Chapter 5, SRC-1 used a different test fixture configuration than the other concrete compression tests. The SRC-1 was tested with two 1,000,000 lb (4,450 kN) capacity jacks side by side. There was no actuator to control the load. Instead, constant pressure to both jacks was used to attempt to load the specimen evenly. Unfortunately, the load did not remain equal between the two jacks. Figure 6.11 is a plot of total load versus the load difference between the two jacks. The load difference between the two jacks stays mostly in a range between zero and 4.5% of the sum of the load of both jacks. This doesn’t seem like much, but the difference was enough to overload the north side of the specimen and cause premature failure. The stress vs. strain plot for SRC-1 is shown in Figure 6.12. The stress vs. strain plot for the two strain gages
diverge due to unequal loading leading to initial failure on the north side at about 4.0 ksi (27.6 MPa). Figure 6.13 plots load against deflection for SRC-1.

![Graph](attachment:graph.png)

**Figure 6.11 Total Load vs. Load Difference Plot for SRC-1 Test**
Figure 6.12 Compressive Stress vs. Strain Plots for SRC-1

Figure 6.13 Compressive Load vs. Deflection Plots for SRC-1
Figure 6.11 shows that the load difference between the two jacks dropped about 2.5 kip (11.1 kN) when the total load was about 600 kip (2,670 kN). This corresponds to the partial failure of the specimen at about 4.0 ksi (27.6 MPa) shown in Figure 6.12. As seen in Table 6.5, the ultimate load for SRC-1 was 723.2 kip (3,217 kN) which translates into an ultimate stress of 4.9 ksi (33.7 MPa).

6.3.2.2 SRC-2

SRC-2 was loaded using the test fixture and 600 kip (2,670 kN) capacity actuator described in Chapter 5. The test began with the lever arm perpendicular to the cross section of the specimen. The loading was relatively even as shown by the stress-strain behavior (see Figures 6.14 and 6.15). However, problems with the hydraulics for the actuator resulted in numerous test attempts. The first, second, and fourth attempts were able to reach a load of about 400 kips (1,780 kN). The third attempt attained a load of approximately 60 kips (267 kN). SRC-2 was equipped with four internal strain gages on the reinforcing bars and two gages on the surface of the concrete. Therefore, it was possible to compute the average strain in the concrete and the average strain in the steel. Figure 6.14 shows the stress vs. average steel strain and average concrete strain for each of the first four test attempts. The figure shows that the modulus of elasticity remains constant throughout all the tests. Also, no visible damage occurred during the first four tests.

Figure 6.15 shows the stress vs. strain curve for the fifth test attempt on SRC-2. The hydraulics were fixed on this test, so the specimen was able to be tested to failure. Ultimate load was 999 kip (4,440 kN), and the ultimate stress was 6.8 ksi (47 MPa).
Figure 6.14 Compressive Stress vs. Strain Plots for SRC-2 (First Four Test Attempts)

Figure 6.15 Compressive Stress vs. Strain Plots for Final Test of SRC-2
Figure 6.16 shows load plotted versus deflection for the SRC-2 specimen. Upon initial failure at about 999 kip (4,440 kN), the deflection increased about 0.6 in (1.5 cm) more before stopping the test with a residual strength of about 200 kip (890 kN).

Figure 6.16  Compressive Load vs. Deflection Plot for SRC-2

6.3.2.3 SRC-3

There were no unusual loading conditions during the SRC-3 test. The fixture was set up the same as SRC-2. The hydraulics were working, so only one test attempt was required. Figure 6.17 shows the stress-strain curves and Figure 6.18 shows the load versus deflection curve for SRC-3. The deflection behavior was similar to that exhibited by SRC-2. SRC-3 experienced initial failure at 1,000 kip (4,450 kN), before failing completely at a residual strength of about 700 kip (3,110 kN). The ultimate stress was 6.8 ksi (46.6 MPa).
Figure 6.17 Compressive Stress vs. Strain Plots for SRC-3

Figure 6.18 Compressive Load vs. Deflection Plot for SRC-3
6.3.3 IRC Specimens

Table 6.9 shows the results of the IRC compression tests and compares the actual load to the predicted load. The IRC specimens failed on average at 86% of the predicted load. On IRC-1 and IRC-3, only two concrete strain gages were available. IRC-2 and IRC-4 had four gages on the longitudinal steel reinforcing in addition to the concrete gages. There were three different test fixture configurations used in the testing of the IRC specimens. The differences are minor and are explained below. The change did not seem to dramatically impact the ultimate load or the structural modulus.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Ultimate Load ([\text{kip (kN)}])</th>
<th>Fraction of Predicted Load ([%])</th>
<th>Ultimate Stress ([\text{ksi (Mpa)}])</th>
<th>Structural Modulus ([\text{Mpsi (GPa)}])</th>
</tr>
</thead>
<tbody>
<tr>
<td>IRC-1</td>
<td>834.4 (3,712)</td>
<td>82.8</td>
<td>5.6 (38.9)</td>
<td>3.9 (27.1)</td>
</tr>
<tr>
<td>IRC-2</td>
<td>882.9 (3,927)</td>
<td>87.6</td>
<td>6.0 (41.1)</td>
<td>4.1 (28.3)</td>
</tr>
<tr>
<td>IRC-3</td>
<td>859.9 (3,825)</td>
<td>85.3</td>
<td>5.8 (40.1)</td>
<td>3.5 (24.0)</td>
</tr>
<tr>
<td>IRC-4</td>
<td>889.2 (3,958)</td>
<td>88.3</td>
<td>6.0 (41.4)</td>
<td>3.6 (24.9)</td>
</tr>
<tr>
<td>Average</td>
<td><strong>866.6 (3,855)</strong></td>
<td><strong>86.0</strong></td>
<td><strong>5.9 (40.4)</strong></td>
<td><strong>3.8 (26.1)</strong></td>
</tr>
<tr>
<td></td>
<td>24.9 (111)</td>
<td>2.5</td>
<td>0.2 (1.2)</td>
<td>0.3 (2.0)</td>
</tr>
<tr>
<td></td>
<td>2.9%</td>
<td>2.9%</td>
<td>2.9%</td>
<td>7.7%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Chauvenet Envelope</th>
<th>Ultimate Load ([\text{kip (kN)}])</th>
<th>Fraction of Predicted Load ([%])</th>
<th>Ultimate Stress ([\text{ksi (Mpa)}])</th>
<th>Structural Modulus ([\text{Mpsi (GPa)}])</th>
</tr>
</thead>
<tbody>
<tr>
<td>IRC-1</td>
<td>828.2 (3,684)</td>
<td>82.2</td>
<td>5.6 (38.6)</td>
<td>3.3 (23.0)</td>
</tr>
<tr>
<td>IRC-4</td>
<td>905.0 (4,026)</td>
<td>89.9</td>
<td>6.1 (42.2)</td>
<td>4.2 (29.2)</td>
</tr>
</tbody>
</table>

\[^{1}\text{1,010 kip (4,480 kN)}\]
IRC-1 and IRC-3 were cut from the same specimen (lab bending test specimen #1). IRC-2 and IRC-4 were cut from lab bending test specimen #2. An examination of Table 6.9 reveals that the ultimate compressive strengths of IRC-1 and IRC-3 are nearly identical, and the ultimate compressive strengths of IRC-2 and IRC-4 are nearly identical. The structural moduli for the specimens are not equal according to their corresponding lab bending test specimen. Instead, equal Young’s moduli corresponded to the location on the lab bending test specimen from which they were taken. IRC-1 and IRC-2 have almost identical structural moduli, and they were both taken from the top of the piles. IRC-3 and IRC-4 have almost identical structural moduli, and they were both cut from the bottom of the piles. Tables 6.10 and 6.11 compare the ultimate compressive load and stress of IRC-1 to IRC-3, and IRC-2 to IRC-4, respectively.

**Table 6.10 Strength Results for IRC-1 and IRC-3**

<table>
<thead>
<tr>
<th>Sample</th>
<th>Ultimate Load [kip (kN)]</th>
<th>Fraction of Predicted Load</th>
<th>Ultimate Stress [ksi (Mpa)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>IRC-1</td>
<td>834.4 (3,712)</td>
<td>82.8</td>
<td>5.6 (38.9)</td>
</tr>
<tr>
<td>IRC-3</td>
<td>859.9 (3,825)</td>
<td>85.3</td>
<td>5.8 (40.1)</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>847.2 (3,768)</strong></td>
<td><strong>83.9</strong></td>
<td><strong>5.7 (39.5)</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Standard Deviation</th>
<th>Fraction of Predicted Load</th>
<th>Ultimate Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>18.1 (80)</td>
<td>2.0</td>
<td>0.1 (0.8)</td>
</tr>
<tr>
<td><strong>2.1%</strong></td>
<td><strong>2.1%</strong></td>
<td><strong>2.1%</strong></td>
</tr>
</tbody>
</table>

1 1,010 kip (4,480 kN)
### Table 6.11 Strength Results for IRC-2 and IRC-4

<table>
<thead>
<tr>
<th>Sample</th>
<th>Ultimate Load [kip (kN)]</th>
<th>Fraction of Predicted Load [%]</th>
<th>Ultimate Stress [ksi (Mpa)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>IRC-2</td>
<td>882.9 (3,927)</td>
<td>87.6</td>
<td>6.0 (41.1)</td>
</tr>
<tr>
<td>IRC-4</td>
<td>889.2 (3,958)</td>
<td>88.3</td>
<td>6.0 (41.4)</td>
</tr>
<tr>
<td>Average</td>
<td><strong>886.1 (3,942)</strong></td>
<td><strong>87.7</strong></td>
<td><strong>6.0 (41.3)</strong></td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>4.5 (20)</td>
<td>0.8</td>
<td>0.0 (0.2)</td>
</tr>
<tr>
<td></td>
<td>0.5%</td>
<td>0.5%</td>
<td>0.5%</td>
</tr>
</tbody>
</table>

1 1,010 kip (4,480 kN)

Figure 6.19 shows the stress vs. strain plots for all IRC specimens, including the average and confidence interval. This figure verifies that the behavior of all IRC specimens are very similar, as reported in Table 6.9. Additional observations for each IRC compression test are presented in the following sections.
6.3.3.1 IRC-1 and IRC-2

IRC-1 and IRC-2 are discussed together first because they used the same test fixture configuration. This configuration was also used for SRC-2 and SRC-3. The IRC-1 test had problems with the hydraulics, similar to the SRC-2 test. Only two attempts needed to be made this time, because the second test was to failure. Figure 6.20 shows the stress-strain behavior for both loadings of IRC-1, while Figure 6.21 shows the load versus deflection curve for IRC-1. IRC-1 experienced initial failure at 834 kip (3,710 kN) before stopping the test at a residual load of approximately 240 kip (1,060 kN).
Figure 6.20  Compressive Stress vs. Strain Plots for IRC-1

Figure 6.21  Compressive Load vs. Deflection Plots for IRC-1
Figure 6.22 shows the stress-strain curves and Figure 6.23 shows the load versus deflection curve for IRC-2. IRC-2 experienced initial failure at 883 kip (3,925 kN), before the test was stopped at a residual load of about 450 kip (2,000 kN).

Figure 6.22  Compressive Stress vs. Strain Plots for IRC-2
6.3.3.2 IRC-3

The test fixture was modified slightly for the IRC-3 test. In all previous tests using the self-reacting frame, the lever arm began perpendicular to the cross section of the specimens. Failure occurred after the arm rotated about the pivot. One side of the specimen would deflect more than the other, which may have caused premature failure. Therefore, for this test the actuator was backed up about 4.0 in (10.2 cm) because that was the deflection reading on the actuator when initial failure began on the previous two IRC tests. This 4.0 in (10.2 cm) offset placed the lever at an angle of about 1.7° to the cross section of the specimens. By beginning this IRC-3 test with the arm at an angle, failure was able to occur when the arm was nearly perpendicular to the end cross-section of the specimens. This should have ensured a more uniform compression failure, possibly leading to higher ultimate loads.
Figure 6.24 shows the stress-strain curves and Figure 6.25 shows the load versus deflection plot for IRC-3. IRC-3 failed initially at a load of 860 kip (3,825 kN) before failing at a residual load of about 710 kip (3,160 kN).

Figure 6.24  Compressive Stress vs. Strain Plots for IRC-3
6.3.3.3 IRC-4

The IRC-4 test was conducted nearly identically to IRC-3. The swivel head being used in all the tests to evenly distribute the load had been observed to rotate too much. This could also contribute to premature failure due to uneven loading. Therefore, a plate was placed between the two plates on either side of the swivel head, in order to stop additional rotation from becoming excessive. This slight change in test fixture did not increase the ultimate load or structural modulus significantly, but a clean compression failure mode was demonstrated for the first time. The failure is shown in Figure 6.26. Figure 6.27 shows the stress-strain curves for IRC-4 and Figure 6.28 shows the load versus deflection curve. IRC-4 failed initially at 889 kip (3,960 kN) before the test was stopped at a residual strength of about 250 kip (1,110 kN).

Figure 6.25 Compressive Load vs. Deflection Plot for IRC-3
Figure 6.26 Compression Failure Mode, IRC-4

Figure 6.27 Compressive Stress vs. Strain Plots for IRC-4
Figure 6.28 Compressive Load vs. Deflection Plot for IRC-4

6.3.4 Steel vs. IsoTruss® Reinforced Piles

Table 6.12 compares the average properties of the SRC specimens to the average properties of the IRC specimens. The SRC specimens held 2.3% more of their expected load than the IRC specimens. The ultimate load and stress of the IRC specimens was 95.5% of the ultimate load and stress of the SRC specimens. The structural modulus of the IRC specimens was 88.6% of the modulus of the SRC specimens. This was expected because the carbon fiber composite IsoTruss® reinforcement had a significantly lower Young’s modulus, 16.0 Mpsi (110 GPa), than steel, 29.0 Mpsi (200 GPa). Figure 6.29 compares the average stress-strain plots for the IRC and the SRC specimens.
Table 6.12 Average Results Comparison for Concrete Compression Specimens

<table>
<thead>
<tr>
<th>Sample</th>
<th>Ultimate Load [kip (kN)]</th>
<th>Fraction of Predicted Load [%]</th>
<th>Ultimate Stress [ksi (MPa)]</th>
<th>Structural Modulus [Mpsi (GPa)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>SRC</td>
<td>907.3 (4,036)</td>
<td>88.3</td>
<td>6.0 (41.4)</td>
<td>3.6 (24.9)</td>
</tr>
<tr>
<td>IRC</td>
<td>866.6 (3,855)</td>
<td>86.0</td>
<td>5.9 (40.4)</td>
<td>3.8 (26.1)</td>
</tr>
</tbody>
</table>

1 SRC = 1,040 kip (4,640 kN), IRC = 1,010 kip (4,480 kN)

Figure 6.29 Average Stress vs. Strain Plots for Concrete Compression Specimens

6.4 MICROSCOPIC INSPECTION

One immediate observation of the microscopic investigation is that sufficient pressure was not applied. This resulted in higher void and lower fiber volume fractions
than desired. On future specimens, the shrink tape must overlap more in order to achieve sufficient consolidation. The main reason for the inferior physical properties of the hand-manufactured specimens is the hand manufacturing process itself. Constant tension of the fiber is extremely important during manufacturing. This keeps the fiber path straight which facilitates better consolidation. When winding by hand, it is impossible to keep tension constant, and occasionally tension is released entirely. Hansen was able to achieve constant tension with the specimen fabrication machine [1]. A 5.0 lb (22.2 N) weight was hung from each longitudinal member tow. Proper tension and adequate consolidation pressure on the tows are clearly two key factors in manufacturing high quality specimens. In the following sections, the cross-sectional area measurements are presented, followed by a more detailed analysis of the void fraction and fiber volume fraction results.

6.4.1 Area Measurement

The design area of the longitudinal members of the IsoTruss® from which the QC specimens were obtained was 0.140 in² (0.90 cm²). The actual average measured cross sectional area was 0.181 in² (1.17 cm²). This is 29% greater than the design area. Hansen [1] performed an inspection of smaller diameter longitudinal members specimens consolidated with polyester shrink tape. Those specimens had an average measured cross sectional area of 0.038 in² (0.24 cm²), almost exactly the same as the design area. Table 6.13 and Figure 6.30 summarize the area measurements for the QC specimens.
Table 6.13 Area Measurements for QC Specimens

<table>
<thead>
<tr>
<th>Sample</th>
<th>Area [in² (cm²)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>QC-1</td>
<td>0.169 (1.09)</td>
</tr>
<tr>
<td>QC-2</td>
<td>0.203 (1.31)</td>
</tr>
<tr>
<td>QC-3</td>
<td>0.161 (1.04)</td>
</tr>
<tr>
<td>QC-4</td>
<td>0.197 (1.27)</td>
</tr>
<tr>
<td>QC-5</td>
<td>0.171 (1.10)</td>
</tr>
<tr>
<td>QC-6</td>
<td>0.188 (1.21)</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.181 (1.17)</strong></td>
</tr>
</tbody>
</table>

| Standard Deviation | 0.017 (0.11) |
| Chauvenet Envelope | 0.152 (0.98) |
| 9.3%               | 0.211 (1.36) |

Figure 6.30 Graph of Cross-sectional Area of QC Specimens
6.4.2 Void Fraction

The cross-section of the QC specimens contained significant voids. The microscopic analysis revealed that the hand manufacturing process produced specimens with an average void fraction of 7.9% (see Table 6.14 and Figure 6.31). This translates into 0.014 in$^2$ (0.09 cm$^2$). The voids account for about 35% of the increase in area from the nominal area. By comparison, Hansen’s specimens had an average void fraction of only 1.1% [1].

Table 6.14 Void Fraction Measurements for QC Specimens

<table>
<thead>
<tr>
<th>Sample</th>
<th>Void Fraction [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>QC-1</td>
<td>7.6</td>
</tr>
<tr>
<td>QC-2</td>
<td>8.9</td>
</tr>
<tr>
<td>QC-3</td>
<td>8.6</td>
</tr>
<tr>
<td>QC-4</td>
<td>9.9</td>
</tr>
<tr>
<td>QC-5</td>
<td>5.7</td>
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<tr>
<td>QC-6</td>
<td>6.5</td>
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<tr>
<td><strong>Average</strong></td>
<td><strong>7.9</strong></td>
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<table>
<thead>
<tr>
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<td></td>
<td>1.6</td>
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<td><strong>Chauvenet Envelope</strong></td>
<td><strong>19.9%</strong></td>
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<table>
<thead>
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<th>Chauvenet Envelope</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2</td>
</tr>
</tbody>
</table>

|                      | 13.7               |
6.4.3 Fiber Volume Fraction

As expected from the increased void fraction, fiber volume fraction was lower. The QC specimens had an average fiber volume fraction of 50.7% when void area is considered (see Table 6.15 and Figure 6.32). The actual fiber area calculated from the manufacturer’s specifications was 0.095 in$^2$ (0.614 cm$^2$). This area divided by the measured cross sectional area yields a predicted fiber volume fraction of 52.6%. When the measured fiber volume fraction is added to the measured void fraction, the result is 58.6%. This means the resin fraction would be 41.5%. The average resin fraction reported by the manufacturer on the spools of pre-impregnated fiber was 31.5%. Hansen reported a fiber volume fraction of 63.6%.

Figure 6.31 Void Fraction Measurements for QC Specimens
Table 6.15 Fiber Volume Fraction Measurements for QC Specimens

<table>
<thead>
<tr>
<th>Sample</th>
<th>Fiber Volume Fraction [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>QC-1</td>
<td>52.0</td>
</tr>
<tr>
<td>QC-2</td>
<td>49.9</td>
</tr>
<tr>
<td>QC-3</td>
<td>49.6</td>
</tr>
<tr>
<td>QC-4</td>
<td>52.0</td>
</tr>
<tr>
<td>QC-5</td>
<td>52.1</td>
</tr>
<tr>
<td>QC-6</td>
<td>48.5</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td>50.7</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>1.6</td>
</tr>
<tr>
<td>Chauvenet Envelope</td>
<td>48.0</td>
</tr>
</tbody>
</table>

Figure 6.32 Fiber Volume Fraction Measurements for QC Specimens
6.5 SUMMARY OF DISCUSSION

The following observations were made from the manufacturing and testing:

1) Improper tensioning during hand manufacturing of IsoTruss® grid structures achievement of compressive strength. When results are based on total cross sectional area, specimens tested in this research had an average compressive strength of 50 ksi (340 MPa). This is only 49% of the average strength that Hansen [1] achieved testing smaller longitudinal members of identical material and consolidation method fabricated on a specimen manufacturing machine. Also, 50 ksi (340 MPa) is only 31% of the compressive strength of a typical carbon-epoxy composite, as reported by Barbero [20]. The 50 ksi (340 MPa) value, however, is 49% higher than the average compressive strength value obtained by McCune [5] in testing hand-manufactured IsoTruss® grid structures.

2) Young’s modulus was not as adversely affected by tensioning during hand-manufacturing as was compressive strength. Average Young’s modulus for the IsoTruss® reinforcement was 16.0 Mpsi (110 GPa). This is 78% of Hansen’s value, and 94% of the predicted Young’s modulus using manufacturer’s specifications and the rule of mixtures. It is likely that the Young’s modulus achieved in this research would have been higher if IT-1 had been better prepared for testing.

3) The low compressive strength of the IsoTruss® reinforcement can be partially attributed to premature failure of the vinyl ester resin cap. In spite of design efforts to prevent this, the cap failure allowed brooming of the longitudinal members.

4) Low compressive strength values are primarily due to inferior consolidation of fibers. The fiber volume fraction of the longitudinal members of the IsoTruss®
reinforcement was only 50.7%, with a void fraction of 7.9%. These compare to a fiber volume fraction of 63.6% and a void fraction of 1.1% achieved by Hansen [1]. Hansen used the same shrink tape to consolidate his specimens, but kept constant tension on the fibers using weights. Hansen also used the same amount of shrink tape, although his specimens had a much smaller nominal cross-sectional area of 0.04 in$^2$ (0.25 cm$^2$) compared to 0.14 in$^2$ (0.90 cm$^2$) for the specimens in this research. Voids were introduced into the specimens in this research through inadequate tension and pressure.

5) The ultimate compression load achieved by IsoTruss® reinforced concrete piles was lower than expected. The IRC specimens averaged 86% of the predicted load. This is not necessarily based on improper tensioning during the manufacturing process, as the prediction was based on the compressive strength value obtained in the IT tests. If Hansen’s value is used as the expected strength of the IsoTruss® the percentage is 73% of the predicted load.

The IRC specimens also compared well to the steel reinforced concrete (SRC) specimens. The SRC specimens were designed to approximate a typical reinforced concrete pile and the IRC specimens were designed to match the bending stiffness of the SRC specimens. The IRC specimens had an ultimate compression load that was 98% of the ultimate load of the SRC specimens. This shows that on a relative basis, the IsoTruss® performed extremely well and can be a viable alternative to steel in reinforcing concrete piles.
CHAPTER 7 – CONCLUSIONS AND RECOMMENDATIONS

The conclusions in Section 7.1 were made possible by the following contributions to the state of the art:

1) The first IsoTruss® reinforced concrete piles were manufactured and tested in compression. Bending test results are presented in other documents (see Ferrell [24] and Richardson [25]). The IsoTruss® grid structures used to reinforce the piles were the largest (in terms of member diameter) fabricated so far. This research shows that further investigation of IsoTruss® grid structures as reinforcement of concrete piles is warranted. The viability of replacing steel reinforcement with IsoTruss® grid structures appears promising in piles. The natural corrosion resistance of polymer composites provides additional benefit.

2) Equations for rounded nodes were developed and employed in the design of the IsoTruss® reinforcement. The rounded node allows a greater bending stiffness in a diameter-limited application by moving the longitudinal members away from the center of the cross-section without increasing the overall diameter of the IsoTruss®.

3) Several other equations that are useful in IsoTruss® design and analysis were developed. The node definition equations (Equations 3.3-3.5) facilitate development of tooling. Other equations use vector transformation in order to compare helical member strength to steel hoops.
7.1 CONCLUSIONS

Analysis of the data obtained in this research led to the following conclusions:

1) Sufficient and constant tension on the composite tows during manufacturing of IsoTruss® grid structures is essential to obtain desirable strength and stiffness properties. The tension of the fiber tows of the IsoTruss® structures manufactured for this research was maintained by hand. Therefore, constant tension was impossible to maintain and occasionally, tension was released entirely.

Insufficient tension of the carbon fiber tows during hand manufacturing of IsoTruss® structures results in compressive strength values that average 51% lower than specimens manufactured with adequate tension. The Young’s modulus of IsoTruss® specimens manufactured with insufficient tension averages 22% lower than specimens manufactured with adequate tension and consolidation pressure.

2) Insufficient pressure on the members of the IsoTruss® during consolidation resulted in a fiber volume fraction that was 13% lower than specimens manufactured by Hansen [1] using the same consolidation method. The design area of the longitudinal members of the IsoTruss® structures in this research is 3.7 times larger than that of the specimens manufactured by Hansen [1]. These larger members require more pressure to achieve adequate consolidation.

3) The ultimate compressive load of the IsoTruss® reinforced concrete piles was 4% lower than the ultimate compressive load of the steel reinforced concrete piles. The IsoTruss® structures used to reinforce the concrete piles had a compressive strength that was 51% lower than smaller specimens manufactured with adequate tension. Sufficient
tension of the fiber tows of the IsoTruss® structures during manufacturing should increase the compressive strength of carbon-epoxy IsoTruss® reinforced concrete piles.

7.2 RECOMMENDATIONS

The following recommendations could improve manufacturing and testing in further research:

1) The geometry of the IsoTruss® reinforcement was more constant than past hand manufactured specimens, but could still be improved. Irregularities in the geometry were caused primarily by the flexibility of the wood dowels used to support the heads on the mandrel. A collapsible mandrel with metal pins should be developed. This mandrel would make the manufacturing process more efficient. More importantly, metal pins would result in more uniform geometry of the IsoTruss® by minimizing distortions due to manufacturing loads. The lower deformations would result in straighter fibers, which are a significant factor in compressive strength according to Barbero [20] and Hansen [1].

2) A method for maintaining tension on fiber tows during manufacturing of IsoTruss® structures should be developed. Control of the tension on the fibers is one of the main obstacles to producing higher-performance, hand-manufactured IsoTruss® grid structures.

3) Higher IsoTruss® compressive strength values could be achieved with higher quality end caps for the tests. The IsoTruss® (“IT”) specimens failed close to the ends. One end of the IsoTruss® specimens should be cut off and new vinyl ester caps should be cast in preparation for retesting. Carbon fiber should be wrapped around the outside of the caps for reinforcement. This should prevent premature spalling of the vinyl ester and
increase compressive strength results. Higher quality end caps would determine if the
compressive strength values for hand-manufactured IsoTruss® specimens with improper
tension are accurate.

4) Future compression tests of IRC specimens should use the test fixture
configuration that was used for IRC-4. Specifically, the lever arm should be started back
far enough so failure occurs as the arm becomes perpendicular to the cross section of the
specimen. Also, plates should be placed as bracing in the fixture (as in Figure 6.26) to
prevent over-rotation of the swivel head.
REFERENCES


APPENDIX
APPENDIX A – TEST FIXTURE DRAWINGS

This appendix contains the drawings for the self-reacting steel frame used to test the steel-reinforced concrete (SRC) and IsoTruss®-reinforced concrete (IRC) compression specimens. Shims and minor modifications made to accommodate linear motion transducers are not shown in the drawings.
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 A.1 Plan View of Reaction Frame for Laboratory Tests
Figure A.2 Side View of Reaction Frame for Laboratory Tests

Mounting Plate - Test Specimen Side

Mounting Plate - Actuator Side

Figure A.3 Hole Dimensions for Reaction Frame Mounting Plates
Figure A.4 Gusset Dimensions for Frame Mounting Plates
Figure A.5 Short I-beam Dimensions for Reaction Frame
APPENDIX B – FIBER VOLUME FRACTION AND VOID FRACTION MEASUREMENTS

This appendix contains all the measurements taken for the fiber volume fraction and void fraction of the QC specimens. For each specimen, 45 fiber volume fraction and 30 void fraction measurements were taken.
### Table B.1 Fiber Volume Fraction Measurements for QC Specimens

<table>
<thead>
<tr>
<th>Photograph</th>
<th>QC-1</th>
<th>QC-2</th>
<th>QC-3</th>
<th>QC-4</th>
<th>QC-5</th>
<th>QC-6</th>
</tr>
</thead>
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<td>[%]</td>
<td>[%]</td>
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APPENDIX C – ISOTRUSS® CURE RECORDS

This appendix contains a record of the temperature maintained in the plywood oven during curing of the IsoTruss® specimens. The temperature was measured at five locations. The temperature was hotter at each end where the air entered the oven, and coolest in the middle where the air exited the oven.
Table C.1 Record of Curing Temperatures [°F (°C)] for First IsoTruss® Specimen

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<td>265</td>
<td>196 (91.1)</td>
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<tr>
<td>270</td>
<td>177 (80.6)</td>
</tr>
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</table>

**Table C.2 Record of Curing Temperatures [°F (°C)] for Second Isotruss® Specimen**
APPENDIX D – STATISTICAL CONSIDERATIONS

This appendix contains a review of fundamentals of statistics that are used in the statistical observations reported in Chapter 6. Standard deviation and normal distribution are defined. Also, explanations of confidence intervals and Chauvenet’s criterion for rejection of a data point are given.
D.1 STANDARD DEVIATION

The standard deviation of an incomplete data set is defined by Christensen [26] as:

$$s = \sqrt{\frac{1}{n-1} \sum_{i=1}^{n} (x_i - x_m)^2}$$  (D.1)

where $n$ is the number of samples in the set, $x_i$ is a specific sample, and $x_m$ is the mean of the data set. Equation D.1 was used to calculate all standard deviations reported in this document.

D.2 CONFIDENCE AND RELIABILITY

This analysis assumes that the data follows a normal distribution. Table C in the appendix of Christensen [26] gives cumulative values for the standard normal distribution. The table shows the percentage of data that is less than any given number of standard deviations, $z$, away from the mean. For example, a $z$ value of 1.645 corresponds to 95% of the data (50% less than the mean and 45% greater than the mean (see Figure D.1). Therefore, an interval centered at the mean containing a certain percentage of the data is constructed by:

$$I_R = \mu \pm z_{(R/2 + 0.5)} \sigma$$  (D.2)
where $\mu$ is the population mean, $\sigma$ (equal to 1 for a standard normal distribution) is the population standard deviation and $z$ is the cumulative value of the standard distribution from Christensen [26]. The percentage of data that falls in the interval is referred to as the reliability (see Figure D.2).

Figure D.1  Example of Cumulative Value of the Standard Normal Distribution

Figure D.2  90% Reliability Interval of Standard Normal Distribution
The 90% reliability interval constructed by Equation D.2 assumes the mean and standard deviations are known. In testing, only the sample mean and sample standard deviation can be known, which differ from the actual population mean and population standard deviation due to statistical variations. This additional uncertainty is introduced by the sample size. As the number of samples approaches infinity, the sample mean and sample standard deviation approach the actual population mean and population standard deviation. Therefore, a correction to the interval must be made taking into account the sample size and the confidence level desired.

Odeh [27] describes the formation of confidence and reliability intervals as

\[ I_{k,c} = x_m \pm ks \]  \hspace{1cm} (D.3)

where \( k \) is a factor based on sample size, desired confidence, and desired reliability.

Odeh [27] compiled tables containing calculated values of \( k \) (see Table D.1).
Table D.1 Sample Size $k$ Factors for Calculation of Confidence and Reliability Intervals

<table>
<thead>
<tr>
<th>Number of Samples</th>
<th>Confidence = 90%</th>
<th>Confidence = 95%</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>$R = 90%$</td>
<td>$R = 95%$</td>
</tr>
<tr>
<td>2</td>
<td>15.92</td>
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<td>5.85</td>
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<td>4.94</td>
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<td>3.13</td>
<td>3.72</td>
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<td>3.45</td>
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</tr>
<tr>
<td>15</td>
<td>2.28</td>
<td>2.71</td>
</tr>
</tbody>
</table>

$R =$ Reliability Level

Values of $k$ for data sets with two samples are not given in Odeh [27]. The $k$ values for data sets with two samples were calculated using the formula set forth by the National Institute of Standards and Technology (NIST). The Engineering Statistics Handbook compiled by NIST gives the value of $k$ for computation of confidence and reliability intervals as:
\[ k = \sqrt{\left( n - 1 \right) \left( 1 + \frac{1}{n} \right) z^2_{(R/2+0.5)}} \]

where \( n \) is the number of samples, \( C \) is the desired confidence, \( R \) is the desired reliability, \( z_{(R/2+0.5)} \) is the critical \( z \) value of the standard distribution which is an upper bound for \( R/2+0.5 \) of the data, and \( \chi^2_{1-C,n-1} \) is the critical value of the chi-square distribution which is an upper bound for \( 1-C \) of the data with \( n-1 \) degrees of freedom (see NIST [28]).

D.3 CHAUVENET’S CRITERION

Chauvenet’s criterion [29] sets forth an envelope based on the ratio of the maximum acceptable deviation from the mean to the standard deviation. The ratios, set forth in Table D.2 [29], differ according to the number of samples in a set. Samples that fall outside Chauvenet’s envelope are excluded from the data analysis.

<table>
<thead>
<tr>
<th>( n )</th>
<th>( d_{max}/s )</th>
</tr>
</thead>
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<tr>
<td>3</td>
<td>1.38</td>
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<tr>
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<td>7</td>
<td>1.80</td>
</tr>
<tr>
<td>10</td>
<td>1.96</td>
</tr>
</tbody>
</table>

\( n = \) number of samples in a data set
\( d_{max} = \) maximum acceptable deviation from mean
\( s = \) standard deviation