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Section 01
Introducing CNT Foundations LLC

We are the Professionals

CNT Foundations has over 15 years of building experience. As professional builders, we help you understand how the entire building process works. We don't just sell you a product...we work together with you to find the solution to your problem.

- CNT Foundations is not a franchise. We have local ownership, management and staff. Our storefront and warehouse are located right here in South Carolina. Our workers are trained to have the most comprehensive knowledge base in the foundation repair industry.
- We promise to provide you competitive pricing with no gimmicks. We do not offer discounts on inflated prices.
- You will receive exceptional service from our office staff and our on-site employees.
- We offer superior products that are properly and professionally installed.
- CNT is fully licensed and bonded as general contractors and residential builders.
- Your project is completely covered under our General Liability Insurance and all of our employees are covered by Workman's Compensation Insurance.
- There is no building too big or too small for CNT Foundations to cure. Our company has a full arsenal of foundation solutions that include helical piers, slab brackets, wood rot replacement, crawlspace solutions, and concrete reinforcement. It's only a problem if it doesn't have a solution…and we have your solution!
- All CNT Foundation piers come with an industry-leading LIFETIME TRANSFERABLE WARRANTY. Because we ensure you get the right product installed properly the first time, we consider this the "warranty you will never use."

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CMU Piers
Concrete Footers
Helical Piers
Crawl Space Repairs
Moisture Control
Structual Repairs

COMMERCIAL

The Strongest Helical Piers on the Market for a Wide Variety of Applications
Helical Micro Pulldown Piles
Solar Foundations

NEW CONSTRUCTION

Helical Piers for New Construction
Concrete for New Construction

CRAWL SPACE

Crawlspace Moisture Control
Encapsulation / Vapor Barrier
Crawlspace Insulation
Crawlspace Structual Repairs

References

Clancy & Theys Construction
Shaun Foreman
757-873-6869
Front Street Flats
$128,187.00
52 Piles
Pile Depth 77'

Huss Construction
Richard Huss
843-937-0023
Citadel War Memorial
$38,500.00
18 Micro Piles
Pile Depth 60'

ADD + Dwelling Group
John Ferrell
843-324-4721 8
5 Henrietta
$82,463.00
43 Piles 3
Pile Depth 35' / 9Kips

Turner Construction
Zach Olsen
404-991-0764
Clemson University
Little John Colliseum
230 Soil Nails Pile
Shot Crate Wall

Hill Construction
Don Houghton
843-259-7372
Ashley Shafts Tunnel
$80,000.00
38 Micro Piles
Pile Depth 77'

Newport Construction
Rob Hamill
843-614-2467
Shem Creek Inn
$5,256.00
5 Piles
Pile Depth 50'

NBM Construction
Will Danielson
843-566-9738
1 Meeting Street
$11,768.00
4 Piles
Pile Depth 148'

McBreairty Construction
Dale McBreairty
43-881-6584
315 Ashley Avenue
$51,315.00
9 Micro Piles
Pile Depth 55'

JE Dunn Construction
Doug Paasch
912-661-6929
$60,000.00
2 Micro Piles
Depth 56'

Huss Construction
Jake Oleksak
843-810-5269
Citadel War Memorial
$42,000.00
22 Micro Piles
Pile Depth 80'

References

Huss Construction
Jake Oleksak
843-810-5269
The Citadel Deas Hall
$97,000.00
65 Pile
Pile Depth Range: 13' – 97' Pile

Chastain Construction
Liston Guerry
843-722-4555
577/579 King Street
$8,500.00
7 Piles
Pile Depth Range: 45'

NBM Construction
Scott Smith
843-566-9738
Gibbes Museum of Art
$30,000.00
23 Piles
Pile Depth Range: 50'
Included Pile Testing

Emery Construction
Ken Emery
813-417-2487
Burwells Restautant
$80,000.00
65 Piles
Pile Depth Range 48'-178'

Dow Construction
Scott Dow
843-308-0600
Hibernia Hall
$15,000.00
12 Piles
Depth Range: 50'
***4" Material for elevator shaft*

Simmons Construction
Marvin Simmons
423-626-4518
Big Lots – Chesapeake, VA
$6,500.00
2 Piles
Pile Depth Range: 21'
***Truck Dock Wall*

Meadors Construction
James Meadors
843-460-5261
$24,000.00
7 Piles
Pile Depth Range: 103'
Under Historical Struction
***In 3' Head Room*

Ramsey Development
Randy Templeton
936-714-6494
Port Royal Senior
$60,888.00
58 Piles
Pile Depth 50'

Residential Business License # 23096
General Contractors License # G 114384

Section 02
ICC Report

ICC-ES Evaluation Report

ESR-1559

Reissued December 2020

This report is subject to renewal December 2021.

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A Subsidiary of the International Code Council®

DIVISION: 31 00 00—EARTHWORK
Section: 31 63 00—Bored Piles

REPORT HOLDER:

CANTSINK MANUFACTURING, INC.

EVALUATION SUBJECT:

CANTSINK HELICAL PILE FOUNDATION SYSTEMS

1.0 EVALUATION SCOPE

Compliance with the following codes:

- 2015, 2012, 2009 and 2006 *International Building Code®* (IBC)
- 2015, 2012, 2009 and 2006 *International Residential Code®* (IRC)
- 2013 *Abu Dhabi International Building Code* (ADIBC)†

†The ADIBC is based on the 2009 IBC. 2009 IBC code sections referenced in this report are the same sections in the ADIBC.

Property evaluated:

Structural and geotechnical

2.0 USES

Cantsink Helical Pile Foundation Systems are used either to underpin foundations of existing structures or to form deep foundations for new structures; and are designed to transfer compression and tension loads from the supported structure to suitable soil bearing strata. Underpinning of existing foundations is generally achieved by attaching the helical piles to the repair brackets, which support compression loads. Deep foundations for new construction are generally obtained by attaching the helical piles to new construction brackets that are embedded in concrete pile caps or grade beams, which support both compression and tension loads.

When helical piles are installed under the IRC, an engineered design is required in accordance with IRC Section R301.1.3.

3.0 DESCRIPTION

3.1 GENERAL:

The Cantsink Helical Pile Foundation Systems consist of a helical pile lead section, optional extension sections, and a bracket that allows attachment to the foundation of the supported structure. The helical pile is screwed into the ground by applying torsion to a desired depth and suitable soil bearing strata. The bracket is then installed to connect the pile to the concrete foundation of the supported structure.

3.2 System Components:

3.2.1 Shafts: The lead sections consist of the following shaft sizes: $2^7/_8$-inch-outside-diameter (73 mm) round steel tubing (round HSS2.875x0.203) having a nominal wall thickness of 0.203 inch (5.16 mm) and 1.5-inch (38 mm) solid-round-cornered square (RCS) steel bar. The helical plates, which are factory-welded to the lead, allow advancement into the soil as the pile is rotated. For $2^7/_8$-inch-outside-diameter (73 mm) round steel tubing leads and extension with helical plates, the helical plates are hexagonally shaped with an area equal to a helix of either 8-, 10-, 12-, 14-, 16-, or 19-inch outside diameter (203, 254, 305, 356, 406 or 483 mm), and are made from $^3/_8$-inch-thick (9.5 mm) steel plates. For 1.5-inch RCS steel bar leads and extension with helical plates, the helical plates are hexagonally shaped with an area equal to a helix of either 8-, 10-, 12-, 14-, or 16-inch outside diameter (203, 254, 305, 356 or 406 mm), and are made from $^3/_8$-inch-thick (9.5 mm) steel plates. The helical plates are formed with either a 3-inch (76 mm) pitch for the $2^7/_8$-inch-outside-diameter (73 mm) round steel tubing or 2.9-inch (74 mm) pitch for the 1.5-inch (38 mm) RCS steel bar and radially stiffening ribs, and are then factory-welded to the lead. The extension sections are available in shafts with and without helical plates. Extension sections for the $2^7/_8$-inch-outside-diameter (73 mm) round steel tubing have a factory cold-formed bell-shaped end segment to fit over the lead shaft or other extension sections. Connection of the $2^7/_8$-inch-outside-diameter (73 mm) round steel tubing extension sections to the lead shaft or other extension sections is made by through-bolted connections with three $^3/_4$-inch-diameter (19 mm) steel bolts through the extension bell-shaped segment and the connected lead or other extensions. Extension sections for the 1.5-inch RCS steel bar have a forged upset socket at one end of the steel bar, which allows the upper end of the lead shaft or other end without the upset socket of an extension to be fitted. Connections of the 1.5-inch RCS steel bar lead shaft and extensions sections is made by through-bolted connection of a single $^7/_8$-inch-diameter (22.2 mm) bolt through the upset socket and end of the shaft. Leads and extensions may be either bare steel or hot-dipped galvanized in accordance with ASTM A123, with a minimum coating thickness of 0.005 inch per side (0.127 mm). See Table 1A, 1B, 1C and Figures 1B, 1C, 5 and 6 of this report.

3.2.2 Foundation Attachments (Brackets):

3.2.2.1 Repair Bracket: A foundation repair bracket is used to transfer compressive loads from existing concrete foundations to a helical pile. The main body of the bracket is cut from square HSS10x10x$^3/_8$ inch (254 by 254 by 9.5 mm) steel tubing. The 12-inch-wide-by-11-inch-high (305 mm by 279 mm) steel shelf and supporting tabs are cut from ¼-inch-thick (6 mm) ASTM A36 steel and are factory-welded to the HSS main body. A lifting T-pipe consists of a rectangular HSS3 x 2 x $^1/_4$ inch (76 by 51 by 6 mm) steel tubing and a 2$^3/_8$-inch-outside-diameter (60 mm) round steel tubing (round HSS2.375 x 0.154) having a nominal wall thickness of 0.154 inch (4 mm) and a length of 48 inches (1219 mm). When connecting the repair bracket to the 1.5-inch RCS shaft, the lifting T-pipe is the same as described above, except that it has a 2$^3/_8$-inch-outside-diameter (60 mm) round steel tubing (round HSS2.375 x 0.213) having a nominal wall thickness of 0.213 inch (5.4 mm) and a length of 48 inches (1219 mm). The lifting T-pipe is connected to the bracket main body through two $^7/_8$-inch-diameter (22 mm) steel threaded rods with matching steel nuts and matching steel washers at each end of the threaded rod. See Figures 2 and 3 of this report. The repair bracket may be either bare steel or hot-dipped galvanized in accordance with ASTM A123 with a minimum coating thickness of 0.005-inch per side.

3.2.2.2 2$^7/_8$ HSS Shaft New Construction Bracket (NCB-TC): This bracket is embedded into concrete footings, grade beams, or pile caps. The bracket consists of a 6-by-6-by-$^1/_2$-inch (152 by 152 by 12.7 mm) steel plate factory-welded to 3½-inch-outside-diameter (89 mm) steel round tubing (round HSS3.5 x 0.216) having a nominal wall thickness of 0.216 inch (5.49 mm) and a length of 6.5 inches (165 mm). The bracket includes three predrilled holes used to receive three (3) ¾-inch-diameter (19 mm) bolts as described in Section 3.3.5 of this report. The bracket comes in bare steel or hot-dipped galvanized steel in accordance with ASTM A123. See Figure 2B.

3.2.2.3 1.5 RCS Shaft New Construction Bracket: This bracket is embedded into concrete footings, grade beams, or pile caps. The bracket consists of a 6-by-6-by-$^1/_2$-inch (152 by 152 by 12.7 mm) steel plate factory-welded to 2.38-inch-outside-diameter (60 mm) steel round tubing (round HSS2.375 x 0.218) having a nominal wall thickness of 0.218 inch (5.54 mm) and a length of 6 inches (152 mm). The bracket comes with one predrilled hole used to receive one $^7/_8$-inch diameter (22.2 mm) bolt as described in Section 3.3.5 of this report. The bracket comes in bare steel or hot-dipped galvanized steel in accordance with ASTM A123. See Figure 2A.

3.3 Material Specifications:

3.3.1 Helical Plates: The carbon steel plates conform to ASTM A36, having a minimum yield strength of 36,000 psi (248 MPa) and a minimum tensile strength of 58,000 psi (400 MPa). The helical plates and the shafts to which they are factory-welded may be either bare steel or hot-dipped galvanized in accordance with ASTM A123 with a minimum coating thickness of 0.005 inch per side (0.127 mm).

3.3.2 Leads and Extension Sections: The leads and extension sections of the 2$^7/_8$ HSS shafts are carbon steel round tubes that conform to ASTM A500, Grade B, except for having a minimum yield strength of 50,000 psi (345 MPa) and a minimum tensile strength of 65,000 psi (448 MPa). The leads and extension sections of the 1.5-inch RCS steel bar conform to 1530M2 and/or 1045M2, having a minimum yield strength of 85 ksi (586 MPa) and a minimum tensile strength of 100 ksi (689 MPa). The leads and extension sections may be either bare steel or hot-dipped galvanized in accordance with ASTM A123 with a minimum coating thickness of 0.005 inch per side (0.127 mm).

3.3.3 Repair Bracket: The square steel tubing used to fabricate the repair bracket main body conforms to ASTM A500 Grade B, having a minimum yield strength of 46,000 psi (317 MPa) and a minimum tensile strength of 58,000 psi (400 MPa). The round steel tubing which is part of the lifting T-pipe is made from steel conforming to ASTM A500 Grade B, having a minimum yield strength of 42,000 psi (290 MPa) and a minimum tensile strength of 58,000 psi (400 MPa). The rectangular steel tubing which is part of the lifting T-pipe is made from steel conforming to ASTM A500 Grade B, except for having a minimum yield strength of 50,000 psi (345 MPa) and a minimum tensile strength of 58,000 psi (400 MPa). The steel shelf plate and supporting tabs are made from ASTM A36 steel having a minimum yield strength of 36,000 psi (248 MPa) and a minimum tensile strength of 58,000 psi (400 MPa). The steel threaded rods conform to ASTM A307 Grade A. ASTM A563 nuts and ASTM F844 washers are used to fasten the threaded rods to the bracket. The repair bracket may be either bare steel or hot-dipped galvanized in accordance with ASTM A123 with a minimum coating thickness of 0.005-inch per side.

3.3.4 New Construction Bracket: The steel plate conforms to ASTM A572 Grade 50, having a minimum yield strength of 50,000 psi (345 MPa) and a minimum tensile strength of 65,000 psi (448 MPa). The round steel tubing conforms to ASTM A500, Grade B, and has a minimum yield strength of 50,000 psi (345 MPa) and a tensile strength of 65,000 psi (448 MPa). The new construction bracket is made from bare steel or galvanized in accordance with ASTM A123 with a minimum coating thickness of 0.005 inch per side (0.127 mm).

3.3.5 Bolts for Coupling and for New Construction Brackets: The bolts, used to connect the 2$^7/_8$ HSS lead and extension sections, conform to ASTM A307 GRADE A and the matching hex nuts conform to ASTM A563, Grade A or ASTM A194, Grade 2H. The bolts used to connect the 1.5-inch RCS steel bar lead and extension sections conform to ASTM F3125 Grade A325 and the matching hex nuts conform to ASTM A563, Grade DH. The bolts used to connect the 2$^7/_8$ HSS and 1.5-inch RCS shafts to the new construction brackets conform to ASTM F3125 Grade A325 with matching hex nuts conforming to ASTM A563, Grade DH. Bolts and nuts can be either bare steel or hot-dipped galvanized in accordance with ASTM A153.

4.0 DESIGN AND INSTALLATION

4.1 Design:

4.1.1 General: Engineering calculations and drawings, prepared by a registered design professional, must be submitted to the code official for each project, must be based on accepted engineering principles as described in IBC Section 1604.4, and must conform to 2015, 2012 and 2009 IBC Section 1810 (2006 IBC Section 1808). The load capacities shown in this report are based on allowable stress design (ASD), described in IBC Section 1602 and AISC 360 Section B3.4. The engineering analysis must address helical foundation system performance related to structural and geotechnical requirements. The calculations must address the ability (considering strength and stiffness) of the supported foundation and structure to transmit the applied loads to the helical foundation system and the ability of the helical piles and surrounding soils to support the loads applied by the supported foundation and structure. The structural analysis must consider all applicable internal forces (shear, bending moments and torsional moments, if applicable) due to applied loads, load transfer between the bracket and the pile segments (leads and extensions) and maximum span(s) between helical foundations. The result of the analysis and the structural capacities must be used to select a helical foundation system. The minimum embedment depth for various loading conditions must be included, based on the most stringent requirements of the following: engineering analysis; tested conditions described in this report; a site-specific geotechnical investigation report; and site-specific load tests, if applicable. A soil investigation report must be submitted to the code official as part of the required submittal documents, prescribed in 2015, 2012 and 2009 IBC Section 107 (2006 IBC Section 106), at the time of permit application. The geotechnical report must include, but is not limited to, all of the following:

1. A plot showing the location of the soil investigation.
2. A complete record of the soil boring and penetration test logs and soil samples.
3. A record of soil profile.
4. Information on groundwater table, frost depth and corrosion-related parameters, as described in Section 5.5 of this report.
5. Soil design parameters, such as shear strength, soil bearing pressure, unit weight of soil, deformation characteristics and other parameters affecting pile support conditions as defined in 2015, 2012 and 2009 IBC Section 1810.2.1 (2006 IBC Section 1808.2.9).
6. Confirmation of the suitability of helical foundation systems for the specific project.
7. Recommendations for design criteria, including, but not limited to, mitigation of effects of differential settlement and varying soil strength and effects of adjacent loads.
8. Recommended center-to-center spacing of helical pile foundations, if different from spacing noted in Section 5.14 of this report; and reduction of allowable loads due to the group action, if necessary.
9. Field inspection and reporting procedures (to include procedures for verification of the installed bearing capacity, when required).
10. Load test requirements.
11. Any questionable soil characteristics and special design provisions, as necessary.
12. Expected total and differential settlement.
13. The axial compression and axial tension load soil capacities, if values cannot be determined from this evaluation report.

The allowable axial compressive or tensile load of the helical pile system must be based on the least of the following in accordance with 2015, 2012 and 2009 IBC Section 1810.3.3.1.9:

- Area of the helical bearing plate affixed to the pile shaft times the ultimate bearing capacity of the soil or rock comprising the bearing stratum divided by a safety factor of at least 2. This capacity will be determined by a registered design professional based on site-specific conditions.
- Allowable capacity determined from well-documented correlations with installation torque. Section 4.1.5 of this report includes torque correlation factors used to establish pile capacities based on documented correlations.
- Allowable capacity predicted by dividing the ultimate capacity determined from load tests by a safety factor of at least 2.0. This capacity will be determined by a registered design professional for each site-specific condition.
- Allowable axial capacity of pile shaft and pile shaft couplings. Section 4.1.3 of this report includes the smaller of pile shaft and shaft coupling capacities.
- Allowable axial capacity of helical bearing plates affixed to the pile. Section 4.1.4 of this report includes helical plate axial capacities.
- Allowable axial capacity of the bracket connecting to the foundation. Section 4.1.2 of this report includes bracket capacities.

4.1.2 Bracket Capacity: The concrete foundation must be designed and justified to the satisfaction of the code official with due consideration to the direction and eccentricity of applied loads, including reactions provided by the brackets, acting on the concrete foundation. Only localized limit states of supporting concrete, including punching shear and bearing, have been considered in this evaluation report. Other limit states are outside the scope of this evaluation report and must be determined by the registered design professional. The effects of reduced lateral sliding resistance due to uplift from wind or seismic loads must be considered for each project. Reference Table 2 for the allowable bracket capacities.

4.1.3 Pile Shaft Capacity: The top of the shafts must be braced as described in 2015, 2012 and 2009 IBC Section 1810.2.2 (2006 IBC Section 1808.2.5). In accordance with 2015, 2012 and 2009 IBC Section 1810.2.1 (2006 IBC Section 1808.2.9), any soil other than fluid soil must be deemed to afford sufficient lateral support to prevent buckling of the systems that are braced, and the unbraced length is defined as the length of piles standing in air, water, or in fluid soils plus an additional 5 feet (1524 mm) when embedment is into firm soil, or an additional 10 feet (3048 mm) when embedment is into soft soil. Firm soils must be defined as any soil with a Standard Penetration Test (SPT) blow count of five or greater. Soft soils must be defined as any soil with a SPT blow count greater than zero and less than five. Fluid soils must be defined as any soil with a SPT blow count of zero [weight of hammer (WHO) or weight of rods (WOR)]. Standard Penetration Test blow count must be determined in accordance with ASTM D1586. The ASD shaft capacities recognized in Table 4 of this evaluation report are for helical pile shafts that are installed in fully braced conditions as noted in this section, including piles not standing in air, water, or fluid soils. The shaft capacity of the helical foundation systems in air, water or fluid soils must be determined by a registered design professional.

The elastic shortening/lengthening of the pile shaft will be controlled by the strength and section properties of the shaft sections and the shaft couplers, as applicable. The mechanical properties of the shaft sections are shown in Tables 3A and 3B and can be used to calculate the shortening of the pile shaft. The slip of the $2^7/_8$-inch HSS shaft helical pile coupler is 0.0625 inch (1.6 mm) at rated allowable compression/tensile load per coupling. The slip of the 1.5-inch RCS shaft helical pile coupler is 0.124 inch (3.2 mm) at rated allowable compression/tensile load per coupling.

4.1.4 Helix Plate Capacity: The allowable axial compressive/tensile load capacities of the hexagonally shaped helical plates described in Section 3.2.1 of this report are included in Table 5.

4.1.5 Soil Capacity: Table 6 describes the geotechnical related properties of the piles. The allowable axial compressive or tensile soil capacity must be determined by a registered design professional in accordance with a site-specific geotechnical report, as described in Section 4.1.1, combined with the individual helix bearing method (Method 1), or from field load tests conducted under the supervision of a registered design professional (Method 2). For either Method 1 or Method 2, the predicted axial load capacities must be confirmed during the site-specific production installation, such that the axial load capacities predicted by the torque correlation method are equal to or greater than that is predicted by Method 1 or 2 as described above. The individual bearing method (the individual bearing method is defined as the area of the helical bearing plate times the ultimate bearing capacity of the soil or rock comprising the bearing stratum, divided by a safety factor of at least 2) must be used by a registered design professional when the appropriate soils information is available for the site. The design allowable axial load must be determined by dividing the total ultimate axial load capacity predicted by either Method 1 or 2, above, by a safety factor of at least 2. The torque correlation method must be used to determine the ultimate capacity (Q_{ult}) of the pile (Equation 1). A factor of safety of at least 2 must be applied to the ultimate capacity to determine the allowable soil capacity (Q_{all}) of the pile (Equation 2).

$Q_{ult} = K_t T$ *(Equation 1)*

$Q_{all} = 0.5\ Q_{ult}$ *(Equation 2)*

where:

Q_{ult} = Ultimate axial compressive or tensile capacity (lbf or N) of helical pile, which must be limited to the maximum ultimate values noted in Table 6.

K_t = Torque correlation factors are described in Table 6.

T = Final installation torque in ft-lbf or N-m. The final installation torque is the torque measurement recorded at the final installation depth.

4.2 Installation:

The Cantsink Foundation Systems must be installed by a Cantsink-certified installer, in accordance with Section 4.2 of this report; 2015, 2012 and 2009 IBC Section 1810.4.11; the Cantsink published installation instructions; and approved site-specific construction documentation. In case of conflict, the most stringent requirement governs.

4.2.1 Helical Piles: The helical piles must be installed and located in accordance with the approved plans and specifications. The helical piles are typically installed using hydraulic rotary motors having forward and reverse capabilities, as recommended by Cantsink Manufacturing, Inc. The installation torque must not exceed the values shown in Table 6. Helical piles must be installed vertically into the ground with a maximum allowable angle of inclination of 1 degree from vertical. The helical piles must be rotated clockwise in a continuous manner with the lead section advancing at the helix pitch. Extensions (number and length) are selected based on the approved plans as specified per the site conditions by a registered design professional. The extensions and the lead section must be connected by the use of coupling bolts and nuts as described in Section 3.3.5. Coupling bolts must be snug-tightened as defined in Section J3 of AISC 360. The final installation torque must equal or exceed that specified by the torque correlation method, to support the allowable design loads of the structure. The helical piles must be installed to the minimum depth described in the approved plans, but with the helical plate not less than 5 feet (1.53 m) below the bottom of the supported concrete foundation. For tension application, the helical pile must be installed such that the minimum depth from the ground surface to the uppermost helix is 12D, where D is the diameter of the largest helix.

4.2.2 Foundation Attachments:

4.2.2.1 Repair Bracket: The repair bracket must be installed as specified in the approved plans. The repair bracket is installed by excavating the bottom of the footing or foundation a minimum of 30 inches wide (762 mm) x 30 inches deep (762 mm) from final face of the footing. Footing is chipped back so exposed footing from the wall is less than the footing depth. The excavation is extended under the footing for 12 inches (305 mm) and below the footing for 12 inches (305 mm) The underside of the footing for the bracket bearing plate is cleaned and chipped if highly irregular. Existing concrete footing capacity must not be altered, such as with notching of concrete or cutting of reinforcing steel, without the approval of the registered design professional and the code official. The helical pile is installed vertically and located 2 to 3 inches (51 to 76 mm) from the footing face. The repair bracket is installed over the pile facing, away from the concrete footing. The bracket is rotated into place under the footing and raised into position. The pile is cut off squarely 3 inches (76 mm) above the bracket in the raised position. The T-pipe is installed over the pile shaft, and threaded rods, nuts and washers are added to hold the bracket in position. Coupling nuts, jacking bracket and lifting jack are installed to raise the foundation to the desired elevation. Any lifting of the existing structure must be verified by a registered design professional and is subject to approval of the code official to ensure that the foundation, superstructure and helical piles are not overstressed. Once the foundation has been raised to its desired elevation, the hex nuts over brackets are tightened and jacking brackets and lifting jacks are removed. The threaded-rod nuts must be snug tightened. The field cutting and bolting must be in accordance with the most restrictive requirements as described in this evaluation report, the IBC, AISC 360, and the manufacturer’s written instructions. The excavation must be backfilled in accordance with 2015, 2012 and 2009 IBC Section 1804 (2006 IBC Section 1803).

4.2.2.2 New Construction Bracket: New construction brackets must be placed over the top of the helical piles. The top of pile elevation must be established and must be consistent with the specified elevation. If necessary, the top of the pile may be cut off level to the required length in accordance with the manufacturer's instructions and AISC 360 requirements so as to ensure full, direct contact (bearing) between the top of the pile shaft and the bracket. For tensile load applications of the New Construction Bracket (NCB-TC), up to three ¾-inch-diameter (19 mm) bolts and matching nuts as described in Section 3.3.5 of this report must be installed. For tensile load applications of the New Construction Bracket (NBC-TC 1.5 RCS), one $^7/_8$-inch-diameter (22.2 mm) bolt and matching nut as described in Section 3.3.5 of this report must be installed. The bolts must be snug-tightened as defined in Section J3 of AISC 360. The embedment and edge distance of the bracket into the concrete foundation must be as described in the approved plans and as indicated in Table 2 of this report. The concrete foundation must be cast around the bracket in accordance with the approved construction documents.

4.3 Special Inspection: Continuous special inspection in accordance with 2015 and 2012 IBC Section 1705.9 (2009 IBC Section 1704.10 and 2006 IBC Section 1704.9) must be provided for the installation of the helical piles and foundation brackets. Where on-site welding is required, special inspection in accordance with 2015 and 2012 IBC Section 1705.12 (2009 and 2006 IBC Section 1704.3) is also required. Items to be recorded and confirmed by the special inspector must include, but are not necessarily limited to, the following:

1. Verification of product manufacturer and the manufacturer's certification of the installers.
2. Product configurations and identification (including catalog number) for lead sections, extensions, brackets, bolts, nuts, and washers, if applicable.
3. Installation equipment and written installation procedures.
4. Required target installation torque of piles and depth of helical foundation system.
5. Inclination and position of helical piles; top of pile extension in full contact with bracket; full-surface contact of foundation brackets with concrete; tightness of all bolts and threaded rods.
6. Verification that supported foundation is in a condition adequate to resist applied loads resulting from installation of repair bracket.
7. Compliance of installation with the approved construction documents and this evaluation report.

5.0 CONDITIONS OF USE

The Cantsink Helical Pile Foundation Systems described in this report comply with, or are suitable alternatives to what is specified in, those codes indicated in Section 1.0 of this report, subject to the following conditions:

5.1 The Cantsink Helical Pile Foundation Systems are manufactured, identified and installed in accordance with this report, the manufacturer's written installation instructions (which must be available at the jobsite at all times during installation), and the approved construction documents. In the event of a conflict, the most restrictive requirement governs.

5.2 The Cantsink Helical Pile Foundation Systems have been evaluated for support of structures assigned to Seismic Design Categories (SDCs) A, B and C in accordance with IBC Section 1613. Use of the systems to support structures assigned to SDC D, E, or F or that are located in Site Class E or F are outside the scope of this report, and are subject to the approval of the building official, based upon submission of a design in accordance with the code by a registered design professional.

5.3 Both the repair bracket and the new construction bracket must be used only to support structures that are laterally braced as defined in 2015, 2012 and 2009 IBC Section 1810.2.2 (2006 IBC Section 1808.2.5). Shaft couplings must be located within firm or soft soil as defined in Section 4.1.3.

5.4 Installation of the helical foundation systems is limited to regions of concrete members where analysis indicates no cracking will occur at service load levels.

5.5 Use of the helical foundation systems in conditions that are indicative of potential pile deterioration or corrosion situations, as defined by the following: (1) soil resistivity less than 1,000 ohm-cm; (2) soil pH less than 5.5; (3) soils with high organic content; (4) soil sulfate concentrations greater than 1,000 ppm; (5) soils located in landfill; or (6) soil containing mine waste; is beyond the scope of the evaluation report.

5.6 Zinc-coated steel and bare steel components must not be combined in the same system, unless, they are designed as bare steel elements. All helical foundation components must be galvanically isolated from concrete reinforcing steel, building structural steel, or any other metal building components.

5.7 Special inspection is provided in accordance with Section 4.3 of this report.

5.8 The helical piles must be installed vertically into the ground with a maximum allowable angle of inclination of 1 degree from vertical. To comply with the requirements found in 2015, 2012 and 2009 IBC Section 1810.3.1.3 (2006 IBC Section 1808.2.8.8), the superstructure must be designed to resist the effects of helical pile eccentricity.

5.9 A geotechnical investigation report in accordance with Section 4.1.1 of this report must be submitted to the code official for approval.

5.10 The load combinations prescribed in Section 1605.3.2 of the IBC must be used to determine the applied loads. When using the alternative basic load combinations prescribed in Section 1605.3.2, the allowable stress increases permitted by material chapters of the IBC (Chapters 19 through 23, as applicable) or the referenced standards are prohibited.

5.11 Engineering calculations and drawings in accordance with recognized engineering principles as described in IBC Section 1604.4, and in compliance with Section 4.1 of this report, are prepared by a registered design professional and approved by the code official.

5.12 The applied loads must not exceed the allowable capacities described in Section 4.1 of this report.

5.13 The adequacy of the concrete structures that are connected to the brackets must be verified by a registered design professional in accordance with applicable code provisions, and is subject to the approval of the code official.

5.14 In order to avoid group efficiency effects, an analysis prepared by a registered design professional must be submitted where the center-to-center spacing of the helical piles is less than three times the diameter of the helical plate at the depth of bearing.

5.15 Compliance with 2015, 2012 and 2009 IBC Section 1810.3.11.1 (2006 IBC Section 1808.2.23.1.1) for buildings assigned to SDC C, and with 2012 and 2009 IBC Section 1810.3.6 (2006 IBC Section 1808.2.7) for all buildings, is outside the scope of this report. Such compliance must be addressed by a registered design professional for each site, and is subject to approval of the code official.

5.16 Settlement of the helical pile is outside the scope of this report and must be determined by a registered design professional, as required in 2015, 2012 and 2009 IBC Section 1810.2.3 (2006 IBC Section 1808.2.12).

5.17 The Cantsink Helical Pile Foundation Systems are manufactured at the Cantsink Manufacturing, Inc., facility located in Winder, Georgia, under a quality-control program with inspections by ICC-ES.

6.0 EVIDENCE SUBMITTED

Data in accordance with the ICC-ES Acceptance Criteria for Helical Foundation Systems and Devices (AC358), dated October 2016 (editorially revised June 2017).

7.0 IDENTIFICATION

7.1 The Cantsink Helical Pile Foundation System components are identified by a tag or label bearing the name and address of Cantsink Manufacturing, the catalog number and the evaluation report number (ESR-1559).

7.2 The report holder's contact information is the following:

CANTSINK MANUFACTURING, INC.
71 FIRST AVENUE
LILBURN, GEORGIA 30047
(678) 280-7453
www.cantsink.com
info@cantsink.com

TABLE 1A—HELICAL PILE FOUNDATION SYSTEM COMPONENTS

PRODUCT DESCRIPTION	SHAFT TYPE	CATALOG NUMBER
5-foot lead with 8-inch helix	$2^7/_8$ HSS Lead Sections	2.5-40L05-8
7-foot lead with 8-inch helix		2.5-40L07-8
10-foot lead with 8-inch helix		2.5-40L10-8
5-foot lead with 10-inch helix		2.5-40L05-10
7-foot lead with 10-inch helix		2.5-40L07-10
10-foot lead with 10-inch helix		2.5-40L10-10
5-foot lead with 12-inch helix		2.5-40L05-12
7-foot lead with 12-inch helix		2.5-40L07-12
10-foot lead with 12-inch helix		2.5-40L10-12
5-foot lead with 14-inch helix		2.5-40L05-14
7-foot lead with 14-inch helix		2.5-40L07-14
10-foot lead with 14-inch helix		2.5-40L10-14
5-foot lead with 16-inch helix		2.5-40L05-16
7-foot lead with 16-inch helix		2.5-40L07-16
10-foot lead with 16-inch helix		2.5-40L10-16
5-foot lead with 19-inch helix		2.5-40L05-19
7-foot lead with 19-inch helix		2.5-40L07-19
10-foot lead with 19-inch helix		2.5-40L10-19
5-foot lead with 8 and 10-inch helix		2.5-40L05-8-10
7-foot lead with 8 and 10-inch helix		2.5-40L07-8-10
10-foot lead with 8 and 10-inch helix		2.5-40L10-8-10
5-foot lead with 8 and 14-inch helix		2.5-40L05-8-14
7-foot lead with 8 and 14-inch helix		2.5-40L07-8-14
10-foot lead with 8 and 14-inch helix		2.5-40L10-8-14
5-foot lead with 10 and 12-inch helix		2.5-40L05-10-12
7-foot lead with 10 and 12-inch helix		2.5-40L07-10-12
10-foot lead with 10 and 12-inch helix		2.5-40L10-10-12
5-foot lead with 12 and 14-inch helix		2.5-40L05-12-14
7-foot lead with 12 and 14-inch helix		2.5-40L07-12-14
10-foot lead with 12 and 14-inch helix		2.5-40L10-12-14
5-foot lead with 8, 10 and 12-inch helix		2.5-40L05-8-10-12
7-foot lead with 8, 10 and 12-inch helix		2.5-40L07-8-10-12
10-foot lead with 8,10 and 12-inch helix		2.5-40L10-8-10-12
5-foot lead with 10, 12 and 14-inch helix		2.5-40L05-10-12-14
7-foot lead with 10, 12 and 14-inch helix		2.5-40L07-10-12-14
10-foot lead with 10,12 and 14-inch helix		2.5-40L10-10-12-14
10-foot lead with 8, 10,12 and 14-inch helix		2.5-40L10-8-10-12-14
10 foot lead with 10, 12, 14 and 16-inch helix		2.5-40L10-10-12-14-16

TABLE 1A—HELICAL PILE FOUNDATION SYSTEM COMPONENTS (Continued)

5 foot lead with 8-inch helix	**1.5 RCS Lead Sections**	1.5RCSL05-8
5 foot lead with 10-inch helix		1.5RCSL05-10
5 foot lead with 12-inch helix		1.5RCSL05-12
5 foot lead with 14-inch helix		1.5RCSL05-14
5 foot lead with 16-inch helix		1.5RCSL05-16
5 foot lead with 8 and 10-inch helix		1.5RCSL05-8-10
5 foot lead with 8, 10 and 12-inch helix		1.5RCSL05-8-10-12
7 foot lead with 8-inch helix		1.5RCSL07-8
7 foot lead with 10-inch helix		1.5RCSL07-10
7 foot lead with 12-inch helix		1.5RCSL07-12
7 foot lead with 14-inch helix		1.5RCSL07-14
7 foot lead with 16-inch helix		1.5RCSL07-16
7 foot lead with 8 and 10-inch helix		1.5RCSL07-8-10
7 foot lead with 10 and 12-inch helix		1.5RCSL07-10-12
7 foot lead with 12 and 14-inch helix		1.5RCSL07-12-14
7 foot lead with 8, 10 and 12-inch helix		1.5RCL07-8-10-12
7 foot lead with 10, 12 and 14-inch helix		1.5RCSL07-10-12-14
10 foot lead with 8-inch helix		1.5RCSL10-8
10 foot lead with 10-inch helix		1.5RCSL10-10
10 foot lead with 12-inch helix		1.5RCSL10-12
10 foot lead with 14-inch helix		1.5RCSL10-14
10 foot lead with 16-inch helix		1.5RCSL10-16
10 foot lead with 8 and 10-inch helix		1.5RCSL10-8-10
10 foot lead with 8 and 14-inch helix		1.5RCSL10-8-14
10 foot lead with 10 and 14-inch helix		1.5RCSL10-10-14
10 foot lead with 8, 10 and 12-inch helix		1.5RCL10-8-10-12
10 foot lead with 10, 12 and 14-inch helix		1.5RCSL10-10-12-14
10 foot lead with 8, 10, 12 and 14-inch helix		1.5RCSL10-8-10-12-14
10 foot lead with 10, 12, 14 and 16-inch helix		1.5RCSL10-10-12-14-16
2-foot extension	**$2^7/_8$ HSS Extension Sections**	2.5-40X02
3-foot extension		2.5-40X03
4-foot extension		2.5-40X04
5-foot extension		2.5-40X05
7-foot extension		2.5-40X07
3.5 foot bolt-on extension with 14-inch helix		2.5-40X3.5-14
3.5 foot bolt-on extension with 16-inch helix		2.5-40X3.5-16
3.5 foot bolt-on extension with 19-inch helix		2.5-40X3.5-19
7 foot bolt-on extension with 14 and 16-inch helix		2.5-40X07-14-16
7 foot bolt-on extension with 16 and 19-inch helix		2.5-40X07-16-19
10-foot extension		2.5-40X10
3-foot extension	**1.5 RCS Extension Sections**	1.5RCSX03
4-foot extension		1.5RCSX04
5-foot extension		1.5RCSX05
7-foot extension		1.5RCSX7
10-foot extension		1.5RCSX10
3.0 foot bolt-on extension with 14-inch helix		1.5RCSX03-14
3.0 foot bolt-on extension with 16-inch helix		1.5RCSX03-16
5.0 foot bolt-on extension with 14-inch helix		1.5RCSX05-14
5.0 foot bolt-on extension with 16-inch helix		1.5RCSX05-16
5.0 foot bolt-on extension with 14 and 16-inch helix		1.5RCSX05-14-16
New Construction Bracket	**$2^7/_8$ HSS**	NCB-TC
New Construction Bracket	**1.5 RCS**	NCB-TC 1.5 RCS
Foundation Repair Bracket	**$2^7/_8$ HSS**	UPB-D
Foundation Repair Bracket	**1.5 RCS**	UPB-1.5RCS

For **SI:** 1 inch = 25.4 mm, 1 foot = 305 mm.

TABLE 1B—2.875 HSS HELICAL PILE LEAD AND EXTENSIONS WITH HELICAL PLATES

CATALOG NUMBER	HELICAL PILE AND EXTENSIONS WITH PLATE SPACING (inches)						
	L	M	N	O	P	Q	R
2.5-40L05-8	63	58	-	-	-	-	-
2.5-40L07-8	84	79	-	-	-	-	-
2.5-40L10-8	126	121	-	-	-	-	-
2.5-40L05-10	63	58	-	-	-	-	-
2.5-40L07-10	84	79	-	-	-	-	-
2.5-40L10-10	126	121	-	-	-	-	-
2.5-40L05-12	63	58	-	-	-	-	-
2.5-40L07-12	84	79	-	-	-	-	-
2.5-40L10-12	126	121	-	-	-	-	-
2.5-40L05-14	63	58	-	-	-	-	-
2.5-40L07-14	84	79	-	-	-	-	-
2.5-40L10-14	126	121	-	-	-	-	-
2.5-40L05-16	63	58	-	-	-	-	-
2.5-40L07-16	84	79	-	-	-	-	-
2.5-40L10-16	126	121	-	-	-	-	-
2.5-40L05-19	63	58	-	-	-	-	-
2.5-40L07-19	84	79	-	-	-	-	-
2.5-40L10-19	126	121	-	-	-	-	-
2.5-40L05-8-10	63	34	58	-	-	-	-
2.5-40L07-8-10	84	55	79	-	-	-	-
2.5-40L07-10-12	84	49	79	-	-	-	-
2.5-40L07-12-14	84	43	79	-	-	-	-
2.5-40L10-8-10	126	97	121	-	-	-	-
2.5-40L05-8-14	63	34	28	-	-	-	-
2.5-40L07-8-14	84	55	79	-	-	-	-
2.5-40L10-8-14	126	97	121	-	-	-	-
2.5-40L05-10-12	63	28	58	-	-	-	-
2.5-40L07-10-12	84	49	79	-	-	-	-
2.5-40L10-10-12	126	91	121	-	-	-	-
2.5-40L05-12-14	63	22	58	-	-	-	-
2.5-40L07-12-14	84	43	79	-	-	-	-
2.5-40L10-12-14	126	85	121	-	-	-	-
2.5-40L05-8-10-12	63	10	34	58	-	-	-
2.5-40L07-8-10-12	84	16	52	79	-	-	-
2.5-40L10-8-10-12	126	67	97	121	-	-	-
2.5-40L07-10-12-14	84	13	49	79	-	-	-
2.5-40L10-10-12-14	126	55	91	121	-	-	-
2.5-40L10-8-10-12-14	126	19	61	94	121	-	-
2.5-40L10-10-12-14-16	126	16	54	90	120	-	-
2.5-40X3.5-14	42	-	-	-	-	13	-
2.5-40X3.5-16	42	-	-	-	-	13	-
2.5-40X3.5-19	42	-	-	-	-	13	
2.5-40X07-14-16	82	-	-	-	-	11	53
2.5-40X07-16-19	82	-	-	-	-	14	53

For **SI**: 1inch=25.4 mm.

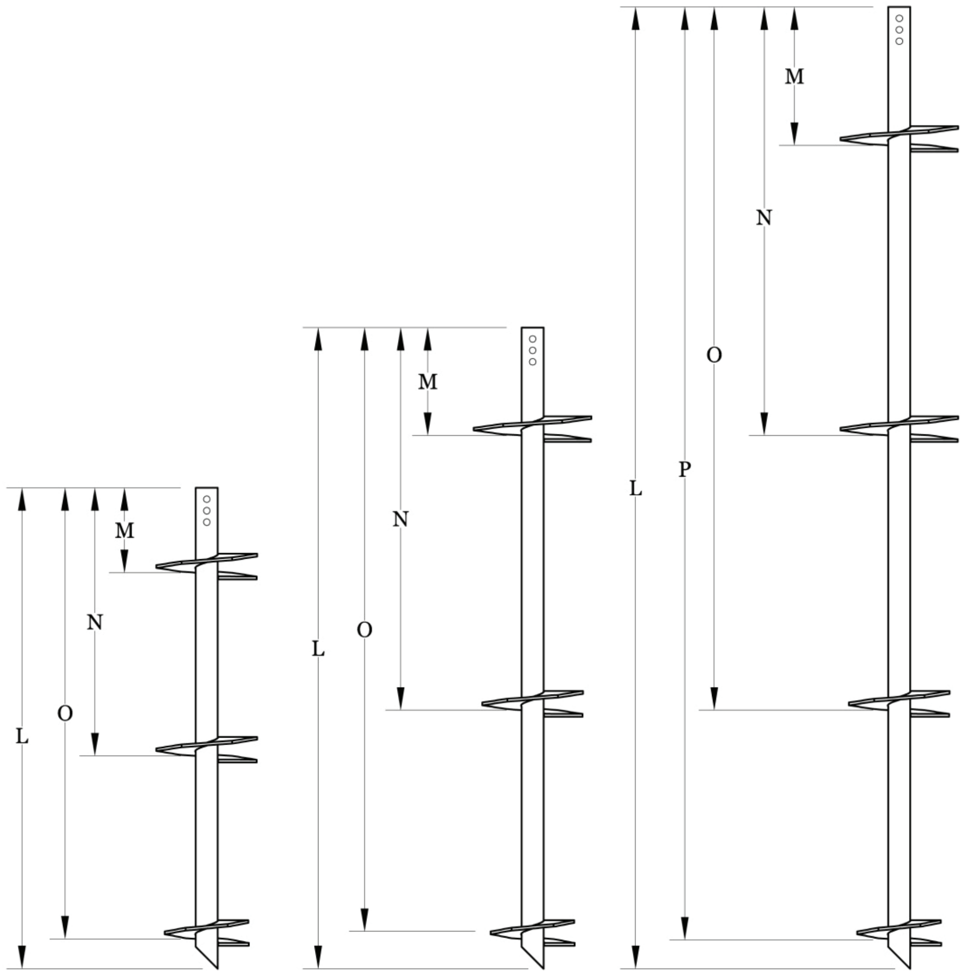


FIGURE 1B—2.875 HSS HELICAL PILE LEAD

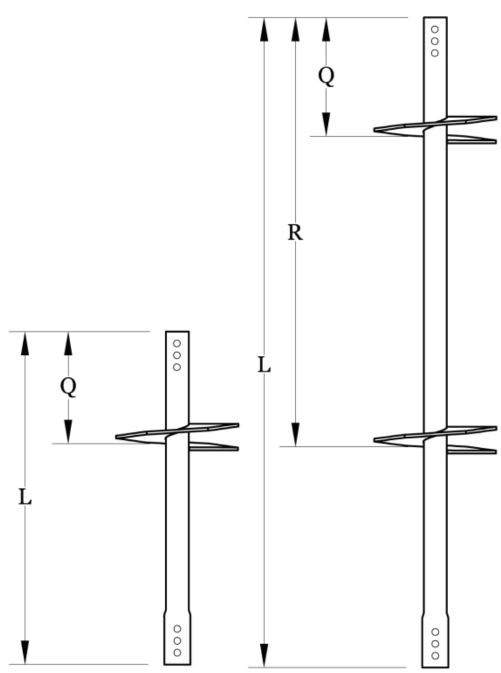


FIGURE 1B—2.875 HSS EXTENSIONS WITH HELICAL PLATES

TABLE 1C—1.5 RCS HELICAL PILE LEAD AND EXTENSIONS WITH HELICAL PLATES

CATALOG NUMBER[1]	HELICAL PILE AND EXTENSIONS WITH PLATE SPACING (inches)						
	L	M	N	O	P	Q	R
1.5RCSL05-8	60	54	-	-	-	-	-
1.5RCSL05-10	60	54	-	-	-	-	-
1.5RCSL05-12	60	54	-	-	-	-	-
1.5RCSL05-14	60	54	-	-	-	-	-
1.5RCSL05-16	60	54	-	-	-	-	-
1.5RCSL05-8-10	60	30	54	-	-	-	-
1.5RCSL05-8-10-12	60	11	35	56	-	-	-
1.5RCSL07-8	84	54	-	-	-	-	-
1.5RCSL07-10	84	54	-	-	-	-	-
1.5RCSL07-12	84	54	-	-	-	-	-
1.5RCSL07-14	84	54	-	-	-	-	-
1.5RCSL07-16	84	54	-	-	-	-	-
1.5RCSL07-8-10	84	54	78	-	-	-	-
1.5RCSL07-8-10-12	84	21	51	78	-	-	-
1.5RCSL07-10-12-14	84	15	51	78	-	-	-
1.5RCSL10-8	120	54	-	-	-	-	-
1.5RCSL10-10	120	54	-	-	-	-	-
1.5RCSL10-12	120	54	-	-	-	-	-
1.5RCSL10-14	120	54	-	-	-	-	-
1.5RCSL10-16	120	54	-	-	-	-	-
1.5RCSL10-8-10	120	90	114	-	-	-	-
1.5RCSL10-8-14	120	87	114	-	-	-	-
1.5RCSL10-10-14	120	84	114	-	-	-	-
1.5RCSL10-8-10-12	120	60	90	114	-	-	-
1.5RCSL10-10-12-14	120	48	84	114	-	-	-
1.5RCSL10-8-10-12-14	120	15	51	84	114	-	-
1.5RCSL10-10-12-14-16	120	15	57	87	114	-	-
1.5RCSX03-14	36			-	-	14	-
1.5RCSX03-16	36			-	-	14	-
1.5RCSX05-14	60					14	-
1.5RCSX05-16	60					14	-
1.5RCSX05-14-16	60					14	50

For **SI:** 1 inch = 25.4 mm, 1 foot = 305 mm.

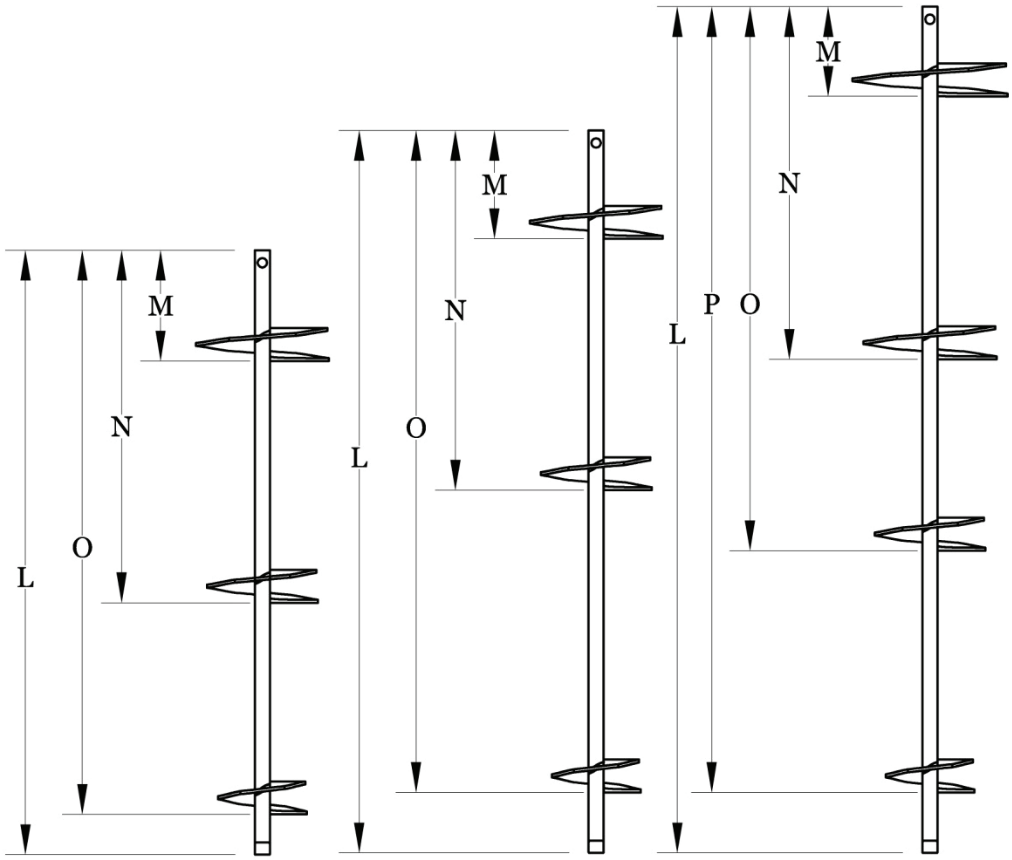


FIGURE 1C—1.5 RCS HELICAL PILE LEAD

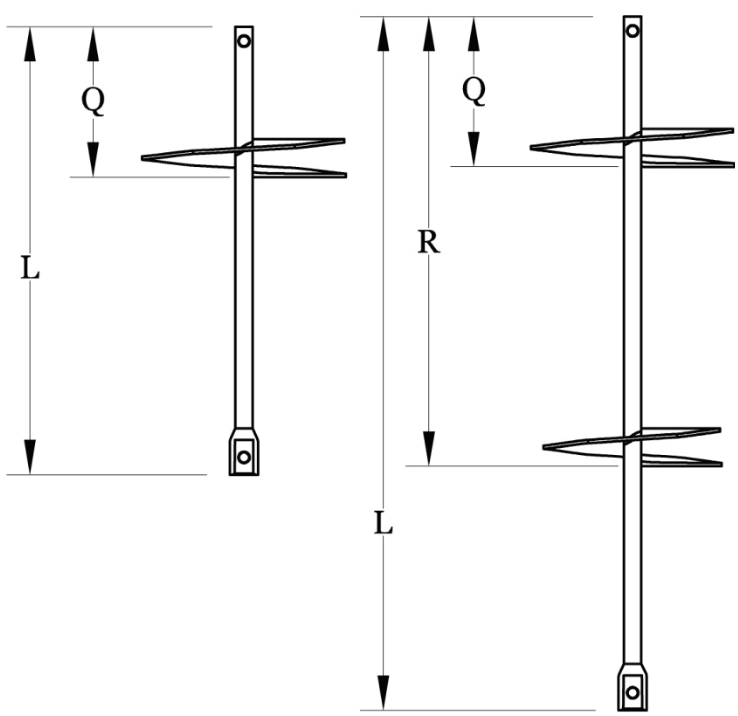


FIGURE 1C—1.5 RCS EXTENSIONS WITH HELICAL PLATES

TABLE 2—ALLOWABLE LOAD CAPACITY OF BRACKETS[6]

BRACKET TYPE[7]	ALLOWABLE AXIAL COMPRESSIVE LOAD CAPACITY (kips)	ALLOWABLE AXIAL TENSILE LOAD CAPACITY (kips)
UPB-BD	23.5[1]	-
UPB-RCS1.5	22.0[1]	-
NCB-TC	30[2]	30[3]
NCB-TC 1.5 RCS	30[4]	30[5]

For **SI:** 1 kip (1000 lbf)=4.48 kN.

[1]Load capacity is based on full scale load tests per AC358 with an installed 5′-0 unbraced pile length having a maximum of one coupling per 2015, 2012 and 2009 IBC Section 1810.2.1 (2006 IBC Section 1808.2.9.2). Repair bracket must be concentrically loaded. Minimum specified compressive strength of concrete is 2500 psi.
[2]The allowable capacity is based on limit states associated with steel strength, concrete punching shear and concrete bearing strength. The allowable capacity has been determined assuming minimum reinforcement has been provided as specified by ACI 318-14 Section 9.6.1.2 and ACI 318-11 Section 10.5.1. The bracket must be installed with a minimum of 8 inches of concrete cover measured from the top of the bracket plate to the centerline of top reinforcement of concrete footing. The concrete footing must have a minimum width of 16 inches and a minimum depth of 12 inches, and must be normal-weight concrete having a minimum specified compressive strength of 2500 psi.
[3] The allowable capacity is based on limit states associated with steel strength, concrete punching shear and concrete bearing strength. The allowable capacity has been determined assuming minimum reinforcement has been provided as specified by ACI 318-14 Section 9.6.1.2 and ACI 318-11 Section 10.5.1. The bracket must be installed with a minimum embedment of the bracket plate of 8 inches. The embedment of the bracket plate is measured from the bottom of the plate to the centerline of bottom reinforcement of the concrete footing. The concrete footing must have a minimum width of 20 inches and a minimum depth of 20 inches, and must be normal-weight concrete having a minimum specified compressive strength of 2500 psi. Three (3) ¾-inch diameter bolts with matching nuts must be installed in accordance with Section 4.2.2.2 of this report. Threads excluded from shear plane.
[4]The allowable capacity is based on limit states associated with steel strength, concrete punching shear and concrete bearing strength. The allowable capacity has been determined assuming minimum reinforcement has been provided as specified by ACI 318-14 Section 9.6.1.2 and ACI 318-11 Section 10.5.1. The bracket must be installed with a minimum of 7 inches of concrete cover measured from the top of the bracket plate to the centerline of top reinforcement of concrete footing. The concrete footing must have a minimum width of 18 inches and a minimum depth of 16 inches, and must be normal-weight concrete having a minimum specified compressive strength of 2500 psi.
[5] The allowable capacity is based on limit states associated with steel strength, concrete punching shear and concrete bearing strength. The allowable capacity has been determined assuming minimum reinforcement has been provided as specified by ACI 318-14 Section 9.6.1.2 and ACI 318-11 Section 10.5.1. The bracket must be installed with a minimum embedment of the bracket plate of 6 inches. The embedment of the bracket plate is measured from the bottom of the plate to the centerline of bottom reinforcement of the concrete footing. The concrete footing must have a minimum width of 20 inches and a minimum depth of 20 inches, and must be normal-weight concrete having a minimum specified compressive strength of 2500 psi. One (1) $^7/_8$-inch diameter bolt with matching nut must be installed in accordance with Section 4.2.2.2 of this report. Threads excluded from shear plane.
[6]The capacities listed in Table 2 assume the pile foundation system is sidesway fully braced and complies with requirements described in Section 4.1.3 of this report.
[7] Allowable capacities are based on bare steel losing 0.036-inch (318 μm) steel thickness as indicated in Section 3.9 of AC358 for a 50-year service life.

TABLE 3A—MECHANICAL PROPERTIES AFTER CORROSION LOSS OF 2.875-INCH HSS HELICAL PILE SHAFT AND EXTENSIONS

PARAMETER	VALUE	
	Bare Steel[1]	Galvanized Steel[2]
Steel yield strength, Fy	50 ksi	
Steel tensile strength, Fu	65 ksi	
Modulus of Elasticity, E	29,000 ksi	
Design wall thickness	0.153 inch	0.181 inch
Outside diameter	2.839 inch	2.872 inch
Inside diameter	2.533 inch	2.510 inch
Cross-sectional area	1.291 inch^2	1.530 inch^2

For **SI:** 1 inch = 25.4; 1 ksi =6.89 MPa.

[1]Dimensional properties are based on bare steel losing 0.036-inch steel thickness as indicated in Section 3.9 of AC358 for a 50-year service life.
[2]Dimensional properties are based on hot-dipped galvanized steel with a minimum coating thickness of 0.005 inch per side and losing 0.013-inch steel thickness as indicated in Section 3.9 of AC358 for a 50-year service life.

TABLE 3B—MECHANICAL PROPERTIES AFTER CORROSION LOSS OF 1.5-INCH RCS HELICAL PILE SHAFT AND EXTENSIONS

PARAMETER	VALUE	
	Bare Steel[1]	Galvanized Steel[2]
Steel yield strength, Fy	85 ksi	
Steel tensile strength, Fu	100 ksi	
Modulus of Elasticity, E	29,000 ksi	
Design shaft depth	1.464 inch	1.494 inch
Cross-sectional area	2.097 inch^2	2.179 inch^2
Moment of Inertia	0.361 inch^4	0.390 inch^4
Radius of Gyration	0.415 inch	0.423 inch
Section Modulus	0.385 inch^3	0.409 inch^3
Plastic Section Modulus	0.656 inch^3	0.693 inch^3

For **SI:** 1 inch = 25.4; 1 ksi =6.89 MPa.

[1]Dimensional properties are based on bare steel losing 0.036-inch steel thickness as indicated in Section 3.9 of AC358 for a 50-year service life.
[2]Dimensional properties are based on hot-dipped galvanized steel with a minimum coating thickness of 0.005 inch per side and losing 0.013-inch steel thickness as indicated in Section 3.9 of AC358 for a 50-year service life.

TABLE 4—SHAFT ALLOWABLE CAPACITY [1,2]

SHAFT TYPE	STEEL TYPE	COMPRESSION (KIPS)	TENSION (KIPS)	TORQUE RATING (FT-LB)
2.875 HSS	BARE[3]	24.63	22.71	6,000
	GALVANIZED[4]	29.95	27.71	
1.5 RCS	BARE[3]	60	29.12	6,687
	GALVANIZED[4]	60	35.57	

For **SI:** 1 inch=25.4 mm; 1 kip (1000 lbf)=4.48 kN; 1 ft-lb= 1.356 N-m

[1] Capacity based on shaft fully braced condition in that the pile length is fully embedded in firm or soft soil and the supported structure is braced in accordance with 2015, 2012 and 2009 IBC Section 1810.2.2 (2006 IBC Section 1808.2.5).
[2] Capacity based on shafts and extensions connected with bolts and matching nuts installed in accordance with Section 3.2.1 of this report.
[3] Allowable capacity based on bare steel losing 0.036-inch steel thickness as indicated in Section 3.9 of AC358 for a 50-year service life.
[4] Allowable capacity based on hot-dipped galvanized steel with a minimum coating thickness of 0.005 inch per side and losing 0.013-inch steel thickness as indicated in Section 3.9 of AC358 for a 50-year service life.

TABLE 5—ALLOWABLE AXIAL COMPRESSION/TENSION CAPACITY OF HELICAL PLATES (lbf)[1,2,3]

SHAFT TYPE	HELICAL PLATE DIAMETER (inches)					
	8	10	12	14	16	19
2.875 HSS	52,225	52,013	49,011	42,490	51,922	51,323
1.5 RCS	30,727	30,244	31,433	30,723	31,237	N/A

For **SI:** 1 inch=25.4 mm; 1 lbf=4.48 N

[1] Helical plates must comply with Section 3.2 of this report.
[2] For helical piles with more than one helix, the allowable capacity for the helical foundation system, may be taken as the sum of the least allowable capacity of each individual helix.
[3] Allowable capacity based on bare steel losing 0.036-inch steel thickness as indicated in Section 3.9 of AC358 for a 50-year service life.

TABLE 6—SOIL CAPACITY AXIAL TENSION AND COMPRESSION[1]

GEOTECHNICAL RELATED PROPERTIES	2.875-inch HSS helical pile[1]		1.5-inch RCS helical pile	
	Compression	Tension	Compression	Tension
Maximum Torsion Rating (ft-lbs)	6,000	6,000	6,687	6,687
Torque Correlation Factor, K_t (ft^{-1})	9	8	10	8
Maximum Ultimate Soil Capacity/ Maximum Allowable Soil Capacity from Torque Correlation (lbf)	54,000/27,000	48,000/24,000	64,000/32,000	53,500/26,750

[1] Helical pile system must be installed in accordance with Section 4.2.1 of this report.

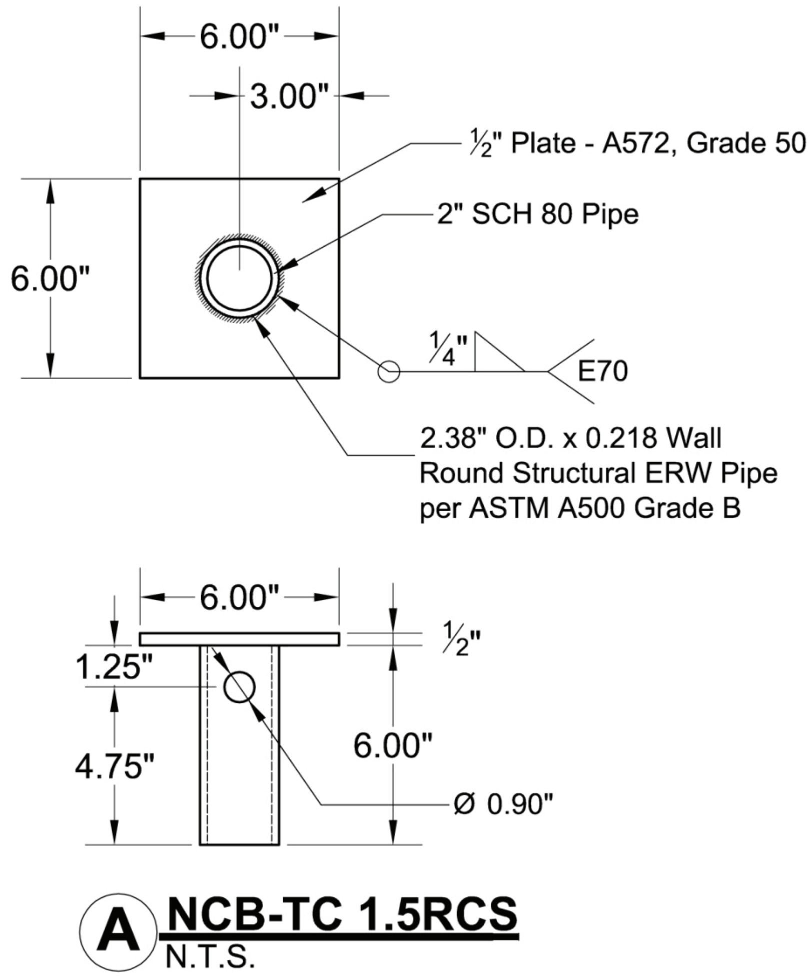


FIGURE 2A—1.5 RCS SHAFT NEW CONSTRUCTION BRACKET (NCB-TC 1.5 RCS)

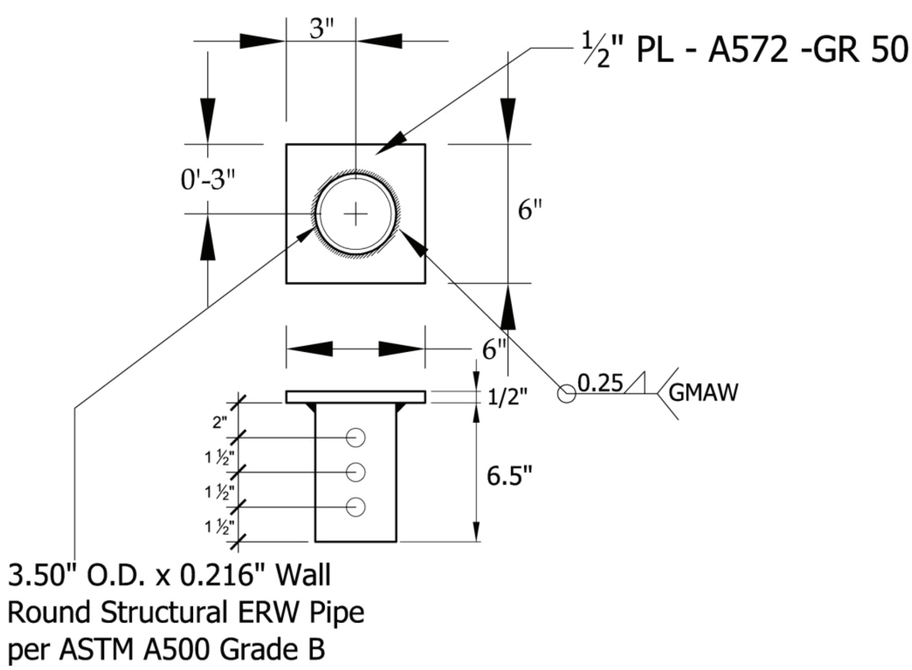


FIGURE 2B—2$^{7}/_{8}$ HSS SHAFT NEW CONSTRUCTION BRACKET (NCB-TC)

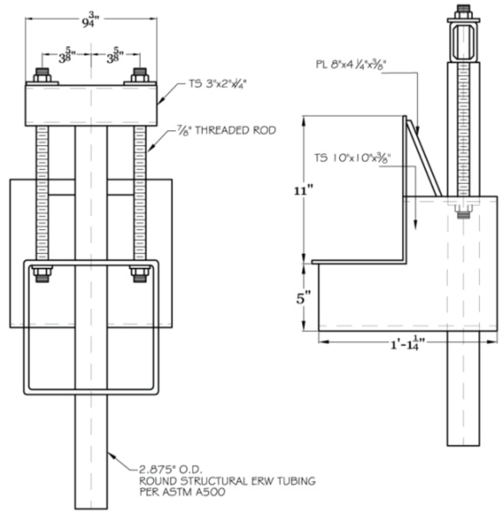


FIGURE 3—FOUNDATION REPAIR BRACKET (UPB-BD SHOWN)

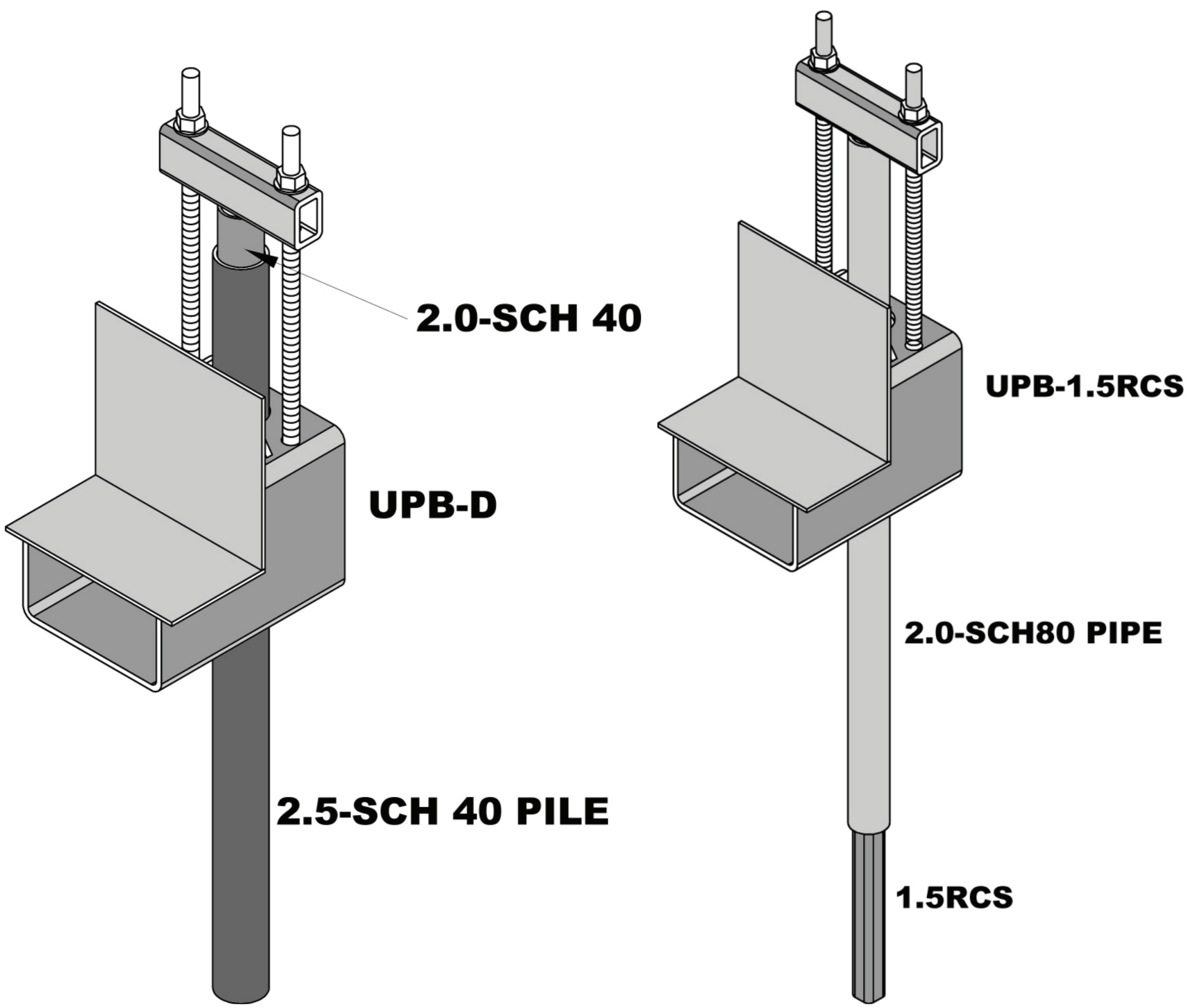
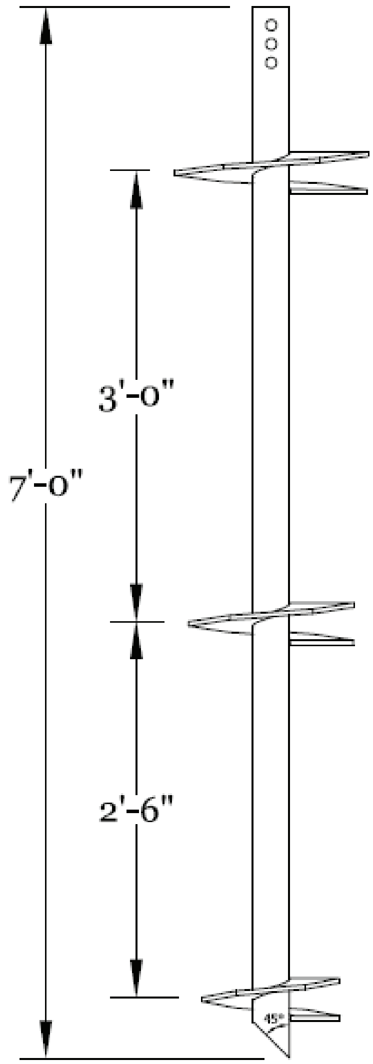


FIGURE 4—FOUNDATION REPAIR BRACKET (UPB-BD AND UPB-RCS1.5)



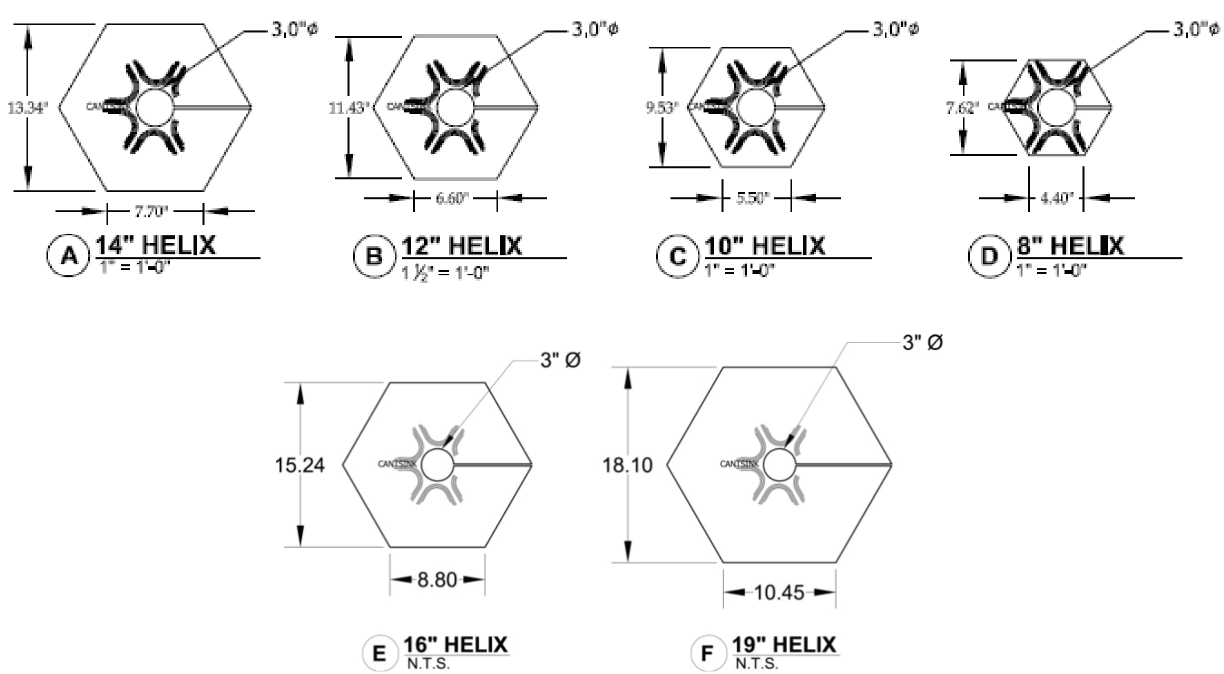


FIGURE 5— (CONTINUED BELOW)

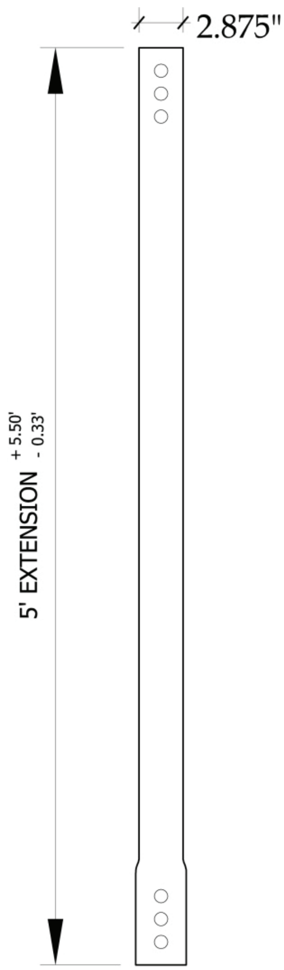


FIGURE 5—2.875-INCH DIAMETER SHAFT HELICALPILE LEAD SECTION, EXTENSION SECTION AND HELICAL PLATES (TYPICAL)

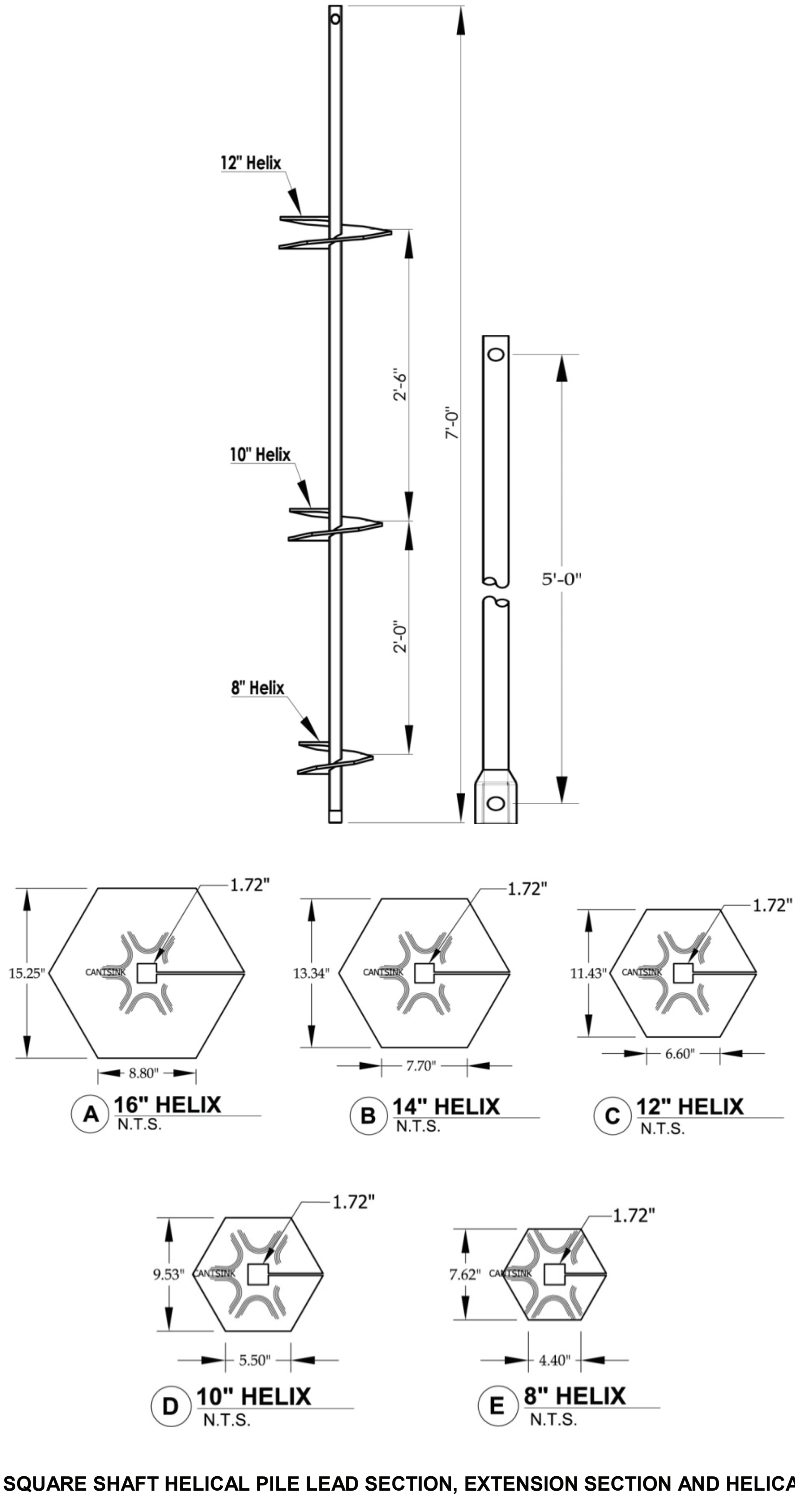


FIGURE 6—1.5-INCH SQUARE SHAFT HELICAL PILE LEAD SECTION, EXTENSION SECTION AND HELICAL PLATES (TYPICAL)

ICC-ES Evaluation Report

ESR-1559 FBC Supplement

Reissued December 2020

This report is subject to renewal December 2021.

www.icc-es.org | (800) 423-6587 | (562) 699-0543 *A Subsidiary of the International Code Council®*

DIVISION: 31 00 00—EARTHWORK
Section: 31 63 00—Bored Piles

REPORT HOLDER:

CANTSINK MANUFACTURING, INC.

EVALUATION SUBJECT:

CANTSINK HELICAL PILE FOUNDATION SYSTEMS

1.0 REPORT PURPOSE AND SCOPE

Purpose:

The purpose of this evaluation report supplement is to indicate that the Cantsink Helical Pile Foundation Systems, described in ICC-ES evaluation report ESR-1559, have also been evaluated for compliance with the codes noted below.

Applicable code editions:

- 2017 *Florida Building Code—Building*
- 2017 *Florida Building Code—Residential*

2.0 CONCLUSIONS

The Cantsink Helical Pile Foundation Systems, described in Sections 2.0 through 7.0 of the evaluation report ESR-1559, comply with the *Florida Building Code—Building* and the *Florida Building Code—Residential*, provided the design and installation are in accordance with the 2015 *International Building Code®* provisions noted in the evaluation report and the following condition applies:

Design wind loads must be based on Section 1609 of the *Florida Building Code—Building* or Section 301.2.1.1 of the *Florida Building Code—Residential*, as applicable.

Use of the Cantsink Helical Pile Foundation Systems for compliance with the High-Velocity Hurricane Zone provisions of the *Florida Building Code—Building* and the *Florida Building Code—Residential* has not been evaluated, and is outside the scope of this evaluation report.

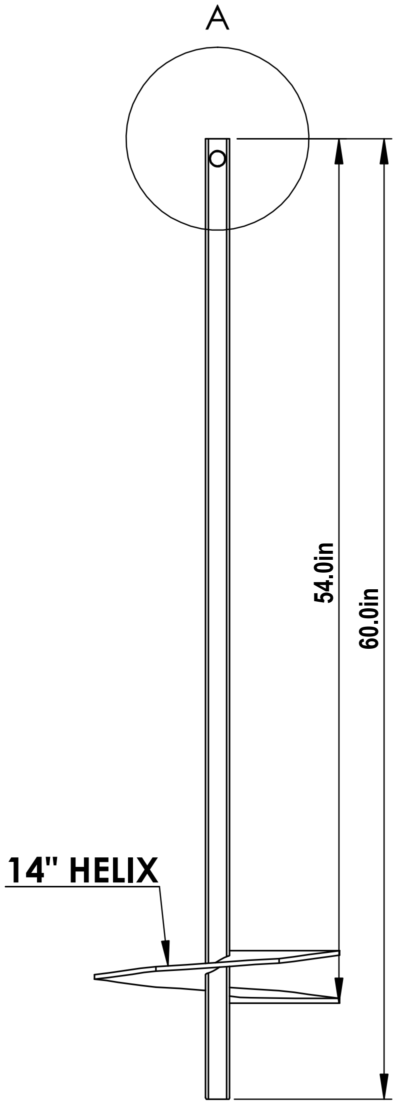
For products falling under Florida Rule 9N-3, verification that the report holder's quality-assurance program is audited by a quality-assurance entity approved by the Florida Building Commission for the type of inspections being conducted is the responsibility of an approved validation entity (or the code official when the report holder does not possess an approval by the Commission).

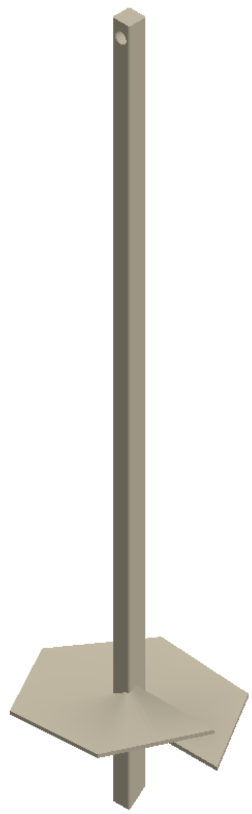
This supplement expires concurrently with the evaluation report, reissued December 2020.

Section 03
Square Shaft Cut Sheets

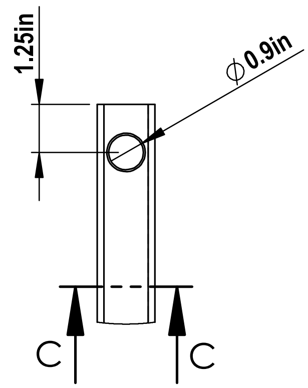
NOTES:

1. SHAFT: 1.5" SQUARE ROUND CORNER SOLID BAR
   GRADE 1530M2 OR GRADE 1045M2
   MINIMUM YIELD 85 KSI
   MINIMUM TENSILE STRENGTH 100 Ksi
2. TORQUE STRENGTH RATING = 6,700 FT-LB
3. LOAD TO TORQUE RATIO (KT) = 10
4. USE ONE $\frac{7}{8}$" A325 GR 5 BOLT TO ATTACH TO AN EXTENSION.
5. AVAILABLE NOMINAL LENGTHS ARE 5', 7' AND 10.5'
6. AVAILABLE HELIX SIZE ARE 8, 10, 12, 14 AND 16"
7. HELIXES ARE A36 STEEL, WITH PITCH OF 2.90" ±0.375"





1 1.5" RCS SINGLE HELIX
1" = 1' 0"



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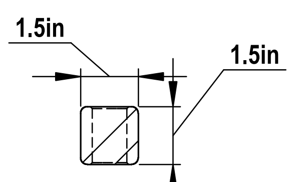

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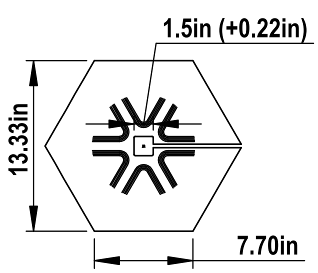

2 14" HELIX
N.T.S.

CANTSINK MANUFACTURING, INC.
CANTSINK
71 First Avenue, N.W.
Lilburn, Georgia 30047
678-280-PILE(7453)
FAX:678-280-3476

Title:	CANTSINK HELICAL PILE LEAD						
				Catalog No.	1.5RCSL05-14		
Revision:	RV-12-12/26/2017	Scale:	N.T.S.	Drawn by:	LXSS	Sheet:	1/1

NOTES:

1. SHAFT: 1.5" SQUARE ROUND CORNER SOLID BAR
   GRADE 1530M2 OR GRADE 1045M2
   MINIMUM YIELD 85 KSI
   MINIMUM TENSILE STRENGTH 100 Ksi
2. TORQUE STRENGTH RATING = 6,700 FT-LB
3. LOAD TO TORQUE RATIO (KT) = 10
4. USE ONE 7/8" A325 GR 5 BOLT TO ATTACH TO AN EXTENSION.
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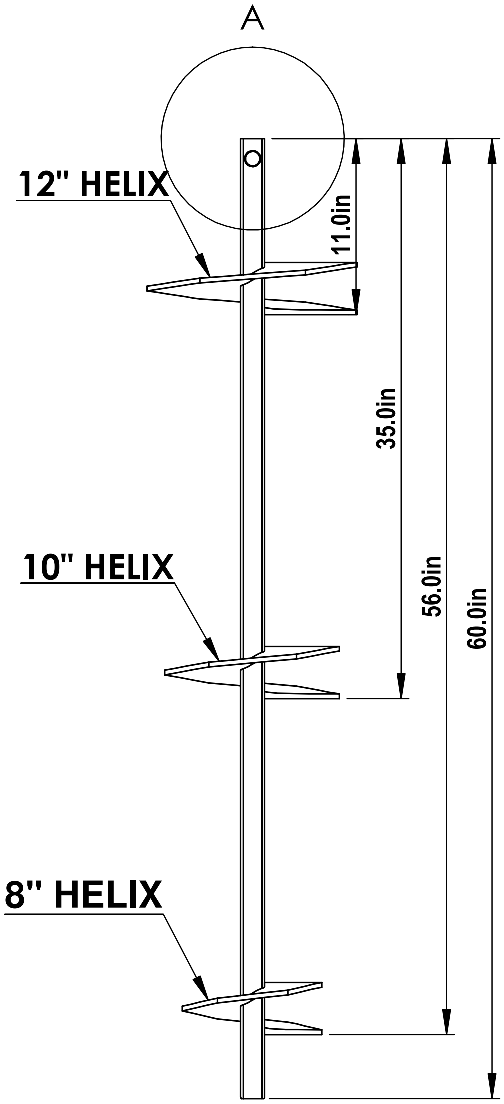
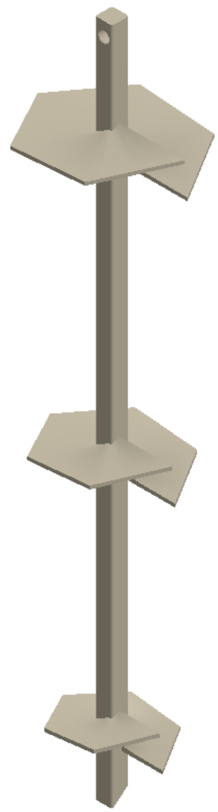
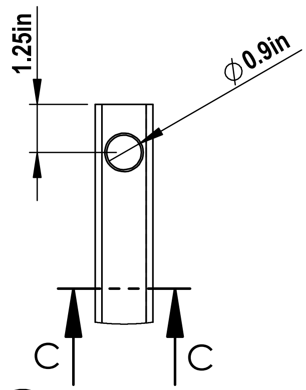
6. AVAILABLE HELIX SIZE ARE 8, 10, 12, 14 AND 16"
7. HELIXES ARE A36 STEEL, WITH PITCH OF 2.90" ±0.375"



1 1.5" RCS TRIPLE HELIX
1" = 1' 0"





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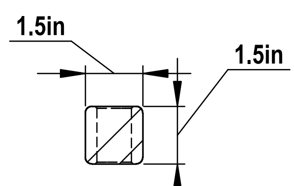

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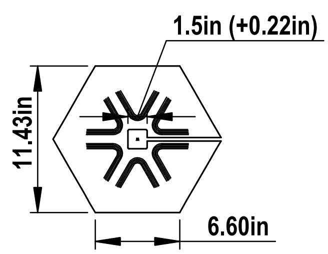

4 12" HELIX
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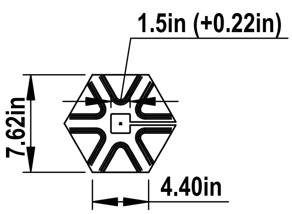

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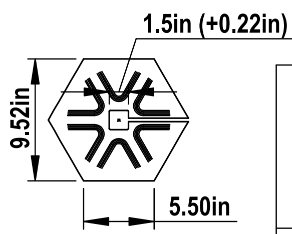

3 10" HELIX
N.T.S.

CANTSINK MANUFACTURING, INC.

CANTSINK

71 First Avenue, N.W.
Lilburn, Georgia 30047
678-280-PILE(7453)
FAX:678-280-3476

Title: CANTSINK HELICAL PILE LEAD

Catalog No. 1.5RCSL05-8-10-12

Revision: RV-12-12/26/2017 | Scale: N.T.S. | Drawn by: LXSS | Sheet: 1/1

NOTES:

1. SHAFT: 1.5" SQUARE ROUND CORNER SOLID BAR
   GRADE 1530M2 OR GRADE 1045M2
   MINIMUM YIELD 85 KSI
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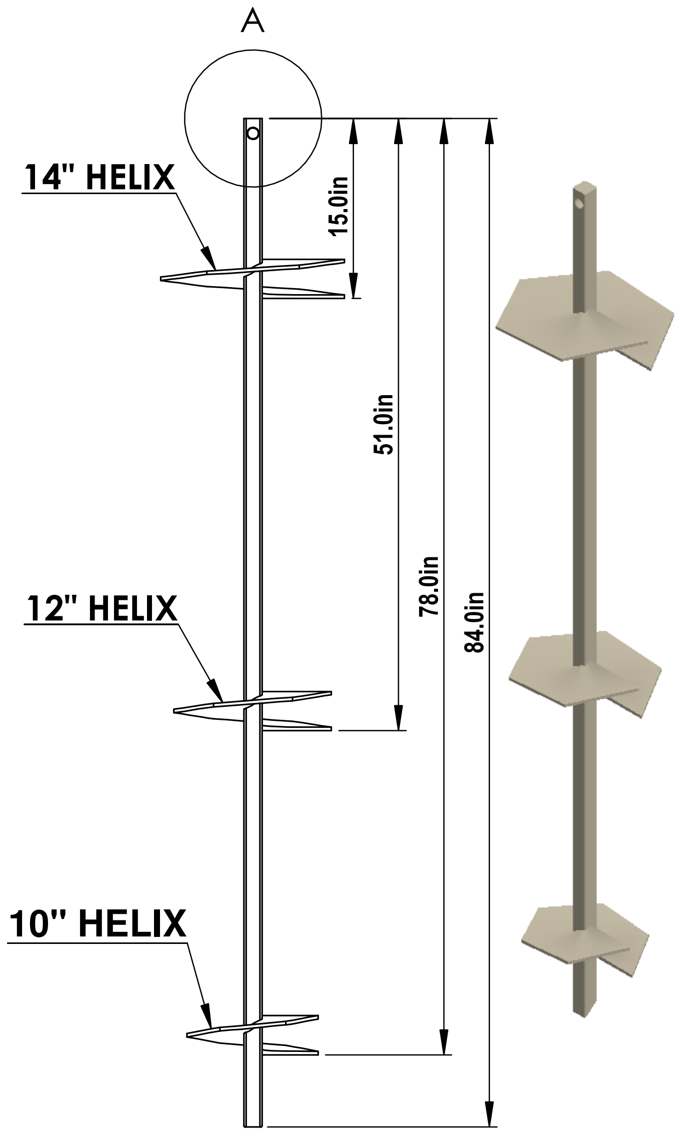
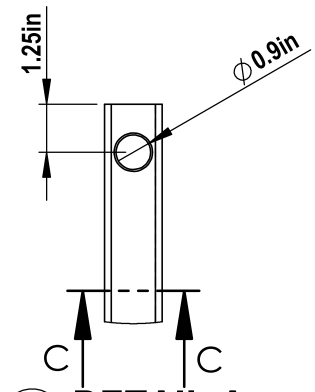


1 1.5" RCS TRIPLE HELIX
3/4" = 1' 0"



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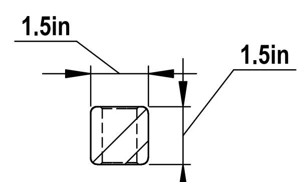

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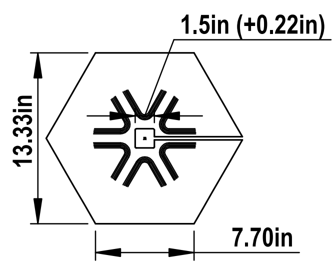
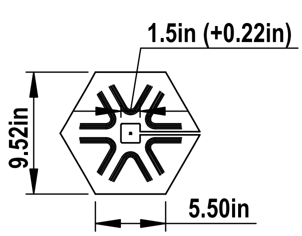


4 14" HELIX
N.T.S.



2 10" HELIX
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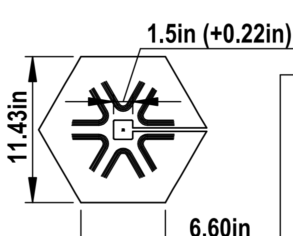

3 12" HELIX
N.T.S.

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CANTSINK

71 First Avenue, N.W.
Lilburn, Georgia 30047
678-280-PILE(7453)
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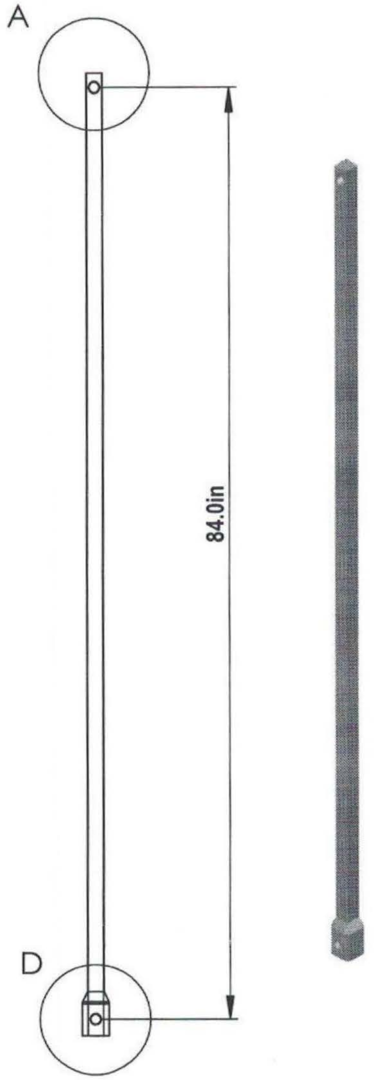
Title:	CANTSINK HELICAL PILE LEAD						
				Catalog No.	1.5RCSL07-10-12-14		
Revision:	RV-12-12/26/2017	Scale:	N.T.S.	Drawn by:	LXSS	Sheet:	1/1



2 1.5" RCS EXTENSION
¾" = 1' 0"

NOTES:

1. SHAFT: 1.5" SQUARE ROUND CORNER SOLID BAR
   GRADE 1530M2 OR GRADE 1045M2
   MINIMUM YIELD 85 KSI
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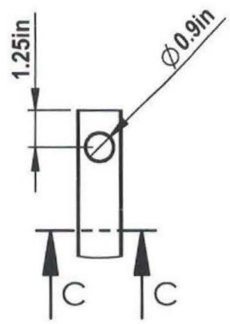

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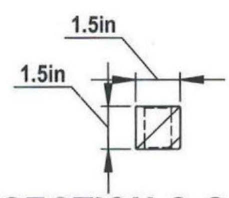

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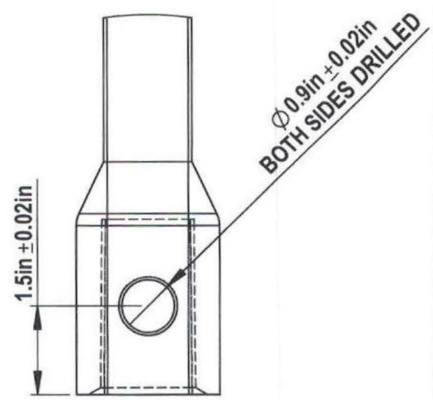

C DETAIL D EXTRUDED EXTENSION
N.T.S.

CANTSINK MANUFACTURING, INC.
CANTSINK
71 First Avenue, N.W.
Lilburn, Georgia 30047
678-280-PILE(7453)
FAX:678-280-3476

Title: CANTSINK HELICAL PILE EXTENSION			
		Catalog No. 1.5RCSX07	
Revision: RV-12-12/26/2017	Scale: N.T.S.	Drawn by: LXSS	Sheet: 1/1

Section 04
Grout Mix

Portland Cement

Safety Data Sheet

According To Federal Register / Vol. 77, No. 58 / Monday, March 26, 2012 / Rules And Regulations And According To The Hazardous Products Regulation (February 11, 2015).

Revision Date: 06/17/2019 Date of Issue: 06/17/2019 Version: 4.0

SECTION 1: IDENTIFICATION

1.1. Product Identifier

Product Form: Mixture

Product Name: Lafarge Portland Cement

Synonyms: Cement, Portland Cement, Hydraulic Cement, Oil Well Cement, Antique White Cement, Portland Limestone Cement, Portland Cement Type I, IA, IE, I/II, II, IIA, II L.A., III, IIIA, IV, IVA, V, VA, 10, 20, 30, 40, 50, GU, GUL, MS, MH, HE, LH, HS, OWH, OWG Cement, OW Class G HSR, ONECEM®, INFINICEM®

Note: This SDS covers many types of Portland cement. Individual composition of hazardous constituents will vary between types of Portland cement.

1.2. Intended Use of the Product

Cement is used as a binder in concrete and mortars that are widely used in construction. Cement is distributed in bags, totes and bulk shipment.

1.3. Name, Address, and Telephone of the Responsible Party

Company
Lafarge US
8700 West Bryn Mawr Avenue, Suite 300
Chicago, IL 60631
Information: 773-372-1000 (9am to 5pm CST)
Email: SDSinfo@Lafarge.com
Website: www.lafargeholcim.us

Company
Lafarge Canada

Eastern Canada
6509 Airport Road
Mississauga, ON L4V 157
Phone: (905) 738-7070

Western Canada
#300 115 Quarry Park Road SE
Calgary, AB T2C 5G9
Phone: (403) 271-9110

Website:www.lafarge.ca

1.4. Emergency Telephone Number

Emergency Number : Chemtrec 1-800-424-9300 (24 hours)

SECTION 2: HAZARDS IDENTIFICATION

2.1. Classification of the Substance or Mixture

GHS-US/CA Classification

Skin Corr. 1C	H314
Eye Dam. 1	H318
Skin Sens. 1	H317
Carc. 1A	H350
STOT SE 3	H335

Full text of hazard classes and H-statements : see Section 16.

2.2. Label Elements

GHS-US/CA Labeling

Hazard Pictograms (GHS-US/CA) :

GHS05 GHS07 GHS08

Signal Word (GHS-US/CA) : Danger

Hazard Statements (GHS-US/CA) : H314 - Causes severe skin burns and eye damage.
H317 - May cause an allergic skin reaction.
H318 - Causes serious eye damage.
H335 - May cause respiratory irritation.

H350 - May cause cancer (Inhalation).

Precautionary Statements (GHS-US/CA) : P201 - Obtain special instructions before use.
P202 - Do not handle until all safety precautions have been read and understood.
P260 - Do not breathe dust.
P264 - Wash hands, forearms, and other exposed areas thoroughly after handling.
P271 - Use only outdoors or in a well-ventilated area.
P272 - Contaminated work clothing should not be allowed out of the workplace.
P280 - Wear protective gloves, protective clothing, and eye protection.
P301+P330+P331 - IF SWALLOWED: Rinse mouth. Do NOT induce vomiting.
P303+P361+P353 - IF ON SKIN (or hair): Take off immediately all contaminated clothing. Rinse skin with water.
P304+P340 - IF INHALED: Remove person to fresh air and keep comfortable for breathing.
P305+P351+P338 - IF IN EYES: Rinse cautiously with water for several minutes. Remove contact lenses, if present and easy to do. Continue rinsing.
P308+P313 - If exposed or concerned: Get medical advice/attention.
P310 - Immediately call a POISON CENTER or doctor.
P321 - Specific treatment (see Section 4 on this SDS).
P333+P313 - If skin irritation or rash occurs: Get medical advice/attention.
P362+P364 - Take off contaminated clothing and wash it before reuse.
P403+P233 - Store in a well-ventilated place. Keep container tightly closed.
P405 - Store locked up.
P501 - Dispose of contents/container in accordance with local, regional, national, territorial, provincial, and international regulations.

2.3. Other Hazards

Exposure may aggravate pre-existing eye, skin, or respiratory conditions. Individuals with lung disease (e.g. bronchitis, emphysema, COPD, pulmonary disease) or sensitivity to hexavalent chromium can be aggravated by exposure.

2.4. Unknown Acute Toxicity (GHS-US/CA)

No data available

SECTION 3: COMPOSITION/INFORMATION ON INGREDIENTS

3.2. Mixture

Name	Product Identifier	% *	GHS Ingredient Classification
Cement, portland, chemicals	(CAS-No.) 65997-15-1	100	Skin Irrit. 2, H315 Eye Dam. 1, H318 Skin Sens. 1, H317 STOT SE 3, H335
Limestone	(CAS-No.) 1317-65-3	<= 15	Not classified
Gypsum (Ca(SO4).2H2O)	(CAS-No.) 13397-24-5	2 - 10	Not classified
Calcium oxide	(CAS-No.) 1305-78-8	<= 5	Skin Irrit. 2, H315 Eye Dam. 1, H318 STOT SE 3, H335 Aquatic Acute 3, H402
Magnesium oxide (MgO)	(CAS-No.) 1309-48-4	<= 4	Not classified
Quartz	(CAS-No.) 14808-60-7	<= 0.2	Carc. 1A, H350 STOT SE 3, H335 STOT RE 1, H372

Full text of H-phrases: see Section 16.

*Percentages are listed in weight by weight percentage (w/w%) for liquid and solid ingredients. Gas ingredients are listed in volume by volume percentage (v/v%).

SECTION 4: FIRST AID MEASURES

4.1. Description of First-aid Measures

General: Never give anything by mouth to an unconscious person. If you feel unwell, seek medical advice (show the label where possible).

Inhalation: Remove to fresh air and keep at rest in a position comfortable for breathing. Immediately call a POISON CENTER or doctor/physician.

Skin Contact: Remove contaminated clothing. Immediately flush skin with plenty of water for at least 30 minutes and continue flushing throughout emergency transport, if needed. Immediately call a poison center or physician. Wash contaminated clothing before reuse.

Eye Contact: Get medical attention immediately and begin flushing eyes with plenty of water for at least 30 minutes and continue flushing eyes throughout emergency transport. Immediately call a poison center or physician. Occasionally lift the upper and lower eyelids during flushing. Remove any contact lenses, if possible. Chemical burns should be treated promptly by a physician.

Ingestion: Rinse mouth. Do NOT induce vomiting. Obtain emergency medical attention.

4.2. Most Important Symptoms and Effects Both Acute and Delayed

General: May cause respiratory irritation. Causes severe skin burns and eye damage. Skin sensitization. May cause cancer.

Inhalation: Irritation of the respiratory tract and the other mucous membranes. May be corrosive to the respiratory tract. The three types of silicosis include: 1) Simple chronic silicosis – which results from long-term exposure (more than 20 years) to low amounts of respirable crystalline silica. Nodules of chronic inflammation and scarring provoked by the respirable crystalline silica form in the lungs and chest lymph nodes. This disease may feature breathlessness and may resemble chronic obstructive pulmonary disease (COPD); 2) Accelerated silicosis – occurs after exposure to larger amounts of respirable crystalline silica over a shorter period of time (5-15 years); 3) Acute silicosis – results from short-term exposure to very large amounts of respirable crystalline silica. The lungs become very inflamed and may fill with fluid, causing severe shortness of breath and low blood oxygen levels. Inflammation, scarring, and symptoms progress faster in accelerated silicosis than in simple silicosis. Progressive massive fibrosis may occur in simple or accelerated silicosis, but is more common in the accelerated form. Progressive massive fibrosis results from severe scarring and leads to the destruction of normal lung structures.

Skin Contact: Concrete may cause dry skin, discomfort, irritation, severe burns, and dermatitis. Exposure of sufficient duration to wet concrete can cause serious, potentially irreversible damage to skin, eye, respiratory and digestive tracts due to chemical (caustic) burns, including third degree burns. A skin exposure may be hazardous even if there is no pain or discomfort. Unhardened concrete is capable of causing dermatitis by irritation and allergy. Skin affected by dermatitis may include symptoms such as, redness, itching, rash, scaling, and cracking. Irritant dermatitis is caused by the physical properties of concrete including alkalinity and abrasion. Allergic contact dermatitis is caused by sensitization to hexavalent chromium (chromate) potentially present in concrete. The reaction can range from a mild rash to severe skin ulcers. Persons already sensitized may react to the first contact with wet concrete. Others may develop allergic dermatitis after years of repeated contact with wet concrete. May cause an allergic skin reaction.

Eye Contact: Potentially causes permanent damage to the cornea, iris, or conjunctiva. Airborne dust may cause immediate or delayed irritation or inflammation. Eye contact with large amounts of dry powder or with wet cement can cause moderate eye irritation, chemical burns and blindness. Eye exposures require immediate first aid and medical attention to prevent significant damage to the eye.

Ingestion: May cause burns or irritation of the linings of the mouth, throat, and gastrointestinal tract.

Chronic Symptoms: May cause cancer.

4.3. Indication of Any Immediate Medical Attention and Special Treatment Needed

If exposed or concerned, get medical advice and attention. If medical advice is needed, have product container or label at hand.

SECTION 5: FIRE-FIGHTING MEASURES

5.1. Extinguishing Media

Suitable Extinguishing Media: Water spray, dry chemical, foam, carbon dioxide.

Unsuitable Extinguishing Media: Do not use a heavy water stream. Use of heavy stream of water may spread fire.

5.2. Special Hazards Arising From the Substance or Mixture

Fire Hazard: Not considered flammable but may burn at high temperatures.

Explosion Hazard: Product is not explosive.

Reactivity: May react exothermically with water releasing heat. Adding an acid to a base or base to an acid may cause a violent reaction.

5.3. Advice for Firefighters

Precautionary Measures Fire: Exercise caution when fighting any chemical fire.

Firefighting Instructions: Use water spray or fog for cooling exposed containers.

Protection During Firefighting: Do not enter fire area without proper protective equipment, including respiratory protection.
Hazardous Combustion Products: Silicon oxides.

Reference to Other Sections

Refer to Section 9 for flammability properties.

SECTION 6: ACCIDENTAL RELEASE MEASURES

6.1. Personal Precautions, Protective Equipment and Emergency Procedures

General Measures: Do not breathe dust. Do not get in eyes, on skin, or on clothing. Do not handle until all safety precautions have been read and understood.

6.1.1. For Non-Emergency Personnel

Protective Equipment: Use appropriate personal protective equipment (PPE).
Emergency Procedures: Evacuate unnecessary personnel.

6.1.2. For Emergency Personnel

Protective Equipment: Equip cleanup crew with proper protection.
Emergency Procedures: Upon arrival at the scene, a first responder is expected to recognize the presence of dangerous goods, protect oneself and the public, secure the area, and call for the assistance of trained personnel as soon as conditions permit. Ventilate area.

6.2. Environmental Precautions

Prevent entry to sewers and public waters.

6.3. Methods and Materials for Containment and Cleaning Up

For Containment: Contain solid spills with appropriate barriers and prevent migration and entry into sewers or streams. As an immediate precautionary measure, isolate spill or leak area in all directions.
Methods for Cleaning Up: Clean up spills immediately and dispose of waste safely. Recover the product by vacuuming, shoveling or sweeping. Transfer spilled material to a suitable container for disposal. Contact competent authorities after a spill. Cautiously neutralize spilled solid. Vacuum clean-up is preferred. If sweeping is required use a dust suppressant.

6.4. Reference to Other Sections

See Section 8 for exposure controls and personal protection and Section 13 for disposal considerations.

SECTION 7: HANDLING AND STORAGE

7.1. Precautions for Safe Handling

Additional Hazards When Processed: May release corrosive vapors. Repeated or prolonged exposure to respirable (airborne) crystalline silica dust will cause lung damage in the form of silicosis. Symptoms will include progressively more difficult breathing, cough, fever, and weight loss. Heavy material- proper lifting methods or equipment.
Precautions for Safe Handling: Wash hands and other exposed areas with mild soap and water before eating, drinking or smoking and when leaving work. Avoid contact with eyes, skin and clothing. Do not get in eyes, on skin, or on clothing. Handle empty containers with care because they may still present a hazard. Do not breathe dust. Obtain special instructions before use. Do not handle until all safety precautions have been read and understood.
Hygiene Measures: Handle in accordance with good industrial hygiene and safety procedures.

7.2. Conditions for Safe Storage, Including Any Incompatibilities

Technical Measures: Comply with applicable regulations.
Storage Conditions: Keep container closed when not in use. Store in a dry, cool place away from incompatible materials. Store in original container or corrosive resistant and/or lined container.
Incompatible Materials: Acids. Oxidizers. Ammonium salts. Aluminum metal. Diazomethane. Phosphorus.
Storage Temperature: Unlimited.

7.3. Specific End Use(s)

Cement is used as a binder in concrete and mortars that are widely used in construction. Cement is distributed in bags, totes and bulk shipment.

SECTION 8: EXPOSURE CONTROLS/PERSONAL PROTECTION

8.1. Control Parameters

For substances listed in Section 3 that are not listed here, there are no established Exposure limits from the manufacturer, supplier, importer, or the appropriate advisory agency including: ACGIH (TLV), AIHA (WEEL), NIOSH (REL), OSHA (PEL), Canadian provincial governments, or the Mexican government.

Quartz (14808-60-7)		
Mexico	OEL TWA (mg/m^3)	0.1 mg/m^3 (respirable fraction)

USA ACGIH	ACGIH TWA (mg/m³)	0.025 mg/m³ (respirable particulate matter)
USA ACGIH	ACGIH chemical category	A2 - Suspected Human Carcinogen
USA OSHA	OSHA PEL (TWA) (mg/m³)	50 µg/m³
USA NIOSH	NIOSH REL (TWA) (mg/m³)	0.05 mg/m³ (respirable dust)
USA IDLH	US IDLH (mg/m³)	50 mg/m³ (respirable dust)
Alberta	OEL TWA (mg/m³)	0.025 mg/m³ (respirable particulate)
British Columbia	OEL TWA (mg/m³)	0.025 mg/m³ (respirable)
Manitoba	OEL TWA (mg/m³)	0.025 mg/m³ (respirable particulate matter)
New Brunswick	OEL TWA (mg/m³)	0.1 mg/m³ (respirable fraction)
Newfoundland & Labrador	OEL TWA (mg/m³)	0.025 mg/m³ (respirable particulate matter)
Nova Scotia	OEL TWA (mg/m³)	0.025 mg/m³ (respirable particulate matter)
Nunavut	OEL TWA (mg/m³)	0.05 mg/m³ (respirable fraction)
Northwest Territories	OEL TWA (mg/m³)	0.05 mg/m³ (respirable fraction)
Ontario	OEL TWA (mg/m³)	0.1 mg/m³ (designated substances regulation-respirable)
Prince Edward Island	OEL TWA (mg/m³)	0.025 mg/m³ (respirable particulate matter)
Québec	VEMP (mg/m³)	0.1 mg/m³ (respirable dust)
Saskatchewan	OEL TWA (mg/m³)	0.05 mg/m³ (respirable fraction)
Yukon	OEL TWA (mg/m³)	300 particle/mL

Limestone (1317-65-3)		
Mexico	OEL TWA (mg/m³)	10 mg/m³
Mexico	OEL STEL (mg/m³)	20 mg/m³
USA OSHA	OSHA PEL (TWA) (mg/m³)	15 mg/m³ (total dust) 5 mg/m³ (respirable fraction)
USA NIOSH	NIOSH REL (TWA) (mg/m³)	10 mg/m³ (total dust) 5 mg/m³ (respirable dust)
Alberta	OEL TWA (mg/m³)	10 mg/m³
British Columbia	OEL STEL (mg/m³)	20 mg/m³ (total dust)
British Columbia	OEL TWA (mg/m³)	10 mg/m³ (total dust) 3 mg/m³ (respirable fraction)
New Brunswick	OEL TWA (mg/m³)	10 mg/m³ (particulate matter containing no Asbestos and <1% Crystalline silica)
Nunavut	OEL STEL (mg/m³)	20 mg/m³
Nunavut	OEL TWA (mg/m³)	10 mg/m³
Northwest Territories	OEL STEL (mg/m³)	20 mg/m³
Northwest Territories	OEL TWA (mg/m³)	10 mg/m³
Québec	VEMP (mg/m³)	10 mg/m³ (Limestone, containing no Asbestos and <1% Crystalline silica-total dust)
Saskatchewan	OEL STEL (mg/m³)	20 mg/m³
Saskatchewan	OEL TWA (mg/m³)	10 mg/m³
Yukon	OEL STEL (mg/m³)	20 mg/m³
Yukon	OEL TWA (mg/m³)	30 mppcf 10 mg/m³

Cement, portland, chemicals (65997-15-1)		
Mexico	OEL TWA (mg/m³)	10 mg/m³
Mexico	OEL STEL (mg/m³)	20 mg/m³
USA ACGIH	ACGIH TWA (mg/m³)	1 mg/m³ (particulate matter containing no asbestos and <1% crystalline silica, respirable particulate matter)
USA ACGIH	ACGIH chemical category	Not Classifiable as a Human Carcinogen
USA OSHA	OSHA PEL (TWA) (mg/m³)	15 mg/m³ (total dust) 5 mg/m³ (respirable fraction)
USA NIOSH	NIOSH REL (TWA) (mg/m³)	10 mg/m³ (total dust) 5 mg/m³ (respirable dust)

USA IDLH	US IDLH (mg/m^3)	5000 mg/m^3
Alberta	OEL TWA (mg/m^3)	10 mg/m^3
British Columbia	OEL TWA (mg/m^3)	1 mg/m^3 (particulate matter containing no Asbestos and <1% Crystalline silica-respirable particulate)
Manitoba	OEL TWA (mg/m^3)	1 mg/m^3 (particulate matter containing no Asbestos and <1% Crystalline silica-respirable particulate matter)
New Brunswick	OEL TWA (mg/m^3)	10 mg/m^3 (particulate matter containing no Asbestos and <1% Crystalline silica)
Newfoundland & Labrador	OEL TWA (mg/m^3)	1 mg/m^3 (particulate matter containing no Asbestos and <1% Crystalline silica-respirable particulate matter)
Nova Scotia	OEL TWA (mg/m^3)	1 mg/m^3 (particulate matter containing no Asbestos and <1% Crystalline silica-respirable particulate matter)
Nunavut	OEL STEL (mg/m^3)	20 mg/m^3
Nunavut	OEL TWA (mg/m^3)	10 mg/m^3
Northwest Territories	OEL STEL (mg/m^3)	20 mg/m^3
Northwest Territories	OEL TWA (mg/m^3)	10 mg/m^3
Ontario	OEL TWA (mg/m^3)	1 mg/m^3 (containing no Asbestos and <1% Crystalline silica-respirable)
Prince Edward Island	OEL TWA (mg/m^3)	1 mg/m^3 (particulate matter containing no Asbestos and <1% Crystalline silica-respirable particulate matter)
Québec	VEMP (mg/m^3)	10 mg/m^3 (containing no Asbestos and <1% Crystalline silica-total dust) 5 mg/m^3 (containing no Asbestos and <1% Crystalline silica-respirable dust)
Saskatchewan	OEL STEL (mg/m^3)	20 mg/m^3
Saskatchewan	OEL TWA (mg/m^3)	10 mg/m^3
Yukon	OEL STEL (mg/m^3)	20 mg/m^3
Yukon	OEL TWA (mg/m^3)	30 mppcf 10 mg/m^3

Gypsum (Ca(SO4).2H2O) (13397-24-5)		
Mexico	OEL TWA (mg/m^3)	10 mg/m^3 (inhalable fraction)
USA ACGIH	ACGIH TWA (mg/m^3)	10 mg/m^3 (inhalable particulate matter)
USA OSHA	OSHA PEL (TWA) (mg/m^3)	15 mg/m^3 (total dust) 5 mg/m^3 (respirable fraction)
USA NIOSH	NIOSH REL (TWA) (mg/m^3)	10 mg/m^3 (total dust) 5 mg/m^3 (respirable dust)
Alberta	OEL TWA (mg/m^3)	10 mg/m^3
British Columbia	OEL STEL (mg/m^3)	20 mg/m^3 (total dust)
British Columbia	OEL TWA (mg/m^3)	10 mg/m^3 (total dust) 3 mg/m^3 (respirable fraction)
Manitoba	OEL TWA (mg/m^3)	10 mg/m^3 (inhalable particulate matter)
Newfoundland & Labrador	OEL TWA (mg/m^3)	10 mg/m^3 (inhalable particulate matter)
Nova Scotia	OEL TWA (mg/m^3)	10 mg/m^3 (inhalable particulate matter)
Ontario	OEL TWA (mg/m^3)	10 mg/m^3 (inhalable)
Prince Edward Island	OEL TWA (mg/m^3)	10 mg/m^3 (inhalable particulate matter)
Québec	VEMP (mg/m^3)	10 mg/m^3 (containing no Asbestos and <1% Crystalline silica-total dust) 5 mg/m^3 (containing no Asbestos and <1% Crystalline silica-respirable dust)
Saskatchewan	OEL STEL (mg/m^3)	20 mg/m^3
Saskatchewan	OEL TWA (mg/m^3)	10 mg/m^3
Yukon	OEL STEL (mg/m^3)	20 mg/m^3

Yukon	OEL TWA (mg/m^3)	30 mppcf 10 mg/m^3
Calcium oxide (1305-78-8)		
Mexico	OEL TWA (mg/m^3)	2 mg/m^3
USA ACGIH	ACGIH TWA (mg/m^3)	2 mg/m^3
USA OSHA	OSHA PEL (TWA) (mg/m^3)	5 mg/m^3
USA NIOSH	NIOSH REL (TWA) (mg/m^3)	2 mg/m^3
USA IDLH	US IDLH (mg/m^3)	25 mg/m^3
Alberta	OEL TWA (mg/m^3)	2 mg/m^3
British Columbia	OEL TWA (mg/m^3)	2 mg/m^3
Manitoba	OEL TWA (mg/m^3)	2 mg/m^3
New Brunswick	OEL TWA (mg/m^3)	2 mg/m^3
Newfoundland & Labrador	OEL TWA (mg/m^3)	2 mg/m^3
Nova Scotia	OEL TWA (mg/m^3)	2 mg/m^3
Nunavut	OEL STEL (mg/m^3)	4 mg/m^3
Nunavut	OEL TWA (mg/m^3)	2 mg/m^3
Northwest Territories	OEL STEL (mg/m^3)	4 mg/m^3
Northwest Territories	OEL TWA (mg/m^3)	2 mg/m^3
Ontario	OEL TWA (mg/m^3)	2 mg/m^3
Prince Edward Island	OEL TWA (mg/m^3)	2 mg/m^3
Québec	VEMP (mg/m^3)	2 mg/m^3
Saskatchewan	OEL STEL (mg/m^3)	4 mg/m^3
Saskatchewan	OEL TWA (mg/m^3)	2 mg/m^3
Yukon	OEL STEL (mg/m^3)	4 mg/m^3
Yukon	OEL TWA (mg/m^3)	2 mg/m^3
Magnesium oxide (MgO) (1309-48-4)		
Mexico	OEL TWA (mg/m^3)	10 mg/m^3 (fume)
USA ACGIH	ACGIH TWA (mg/m^3)	10 mg/m^3 (inhalable particulate matter)
USA ACGIH	ACGIH chemical category	Not Classifiable as a Human Carcinogen
USA OSHA	OSHA PEL (TWA) (mg/m^3)	15 mg/m^3 (fume, total particulate)
USA IDLH	US IDLH (mg/m^3)	750 mg/m^3 (fume)
Alberta	OEL TWA (mg/m^3)	10 mg/m^3 (fume)
British Columbia	OEL STEL (mg/m^3)	10 mg/m^3 (respirable dust and fume)
British Columbia	OEL TWA (mg/m^3)	10 mg/m^3 (fume, inhalable) 3 mg/m^3 (respirable dust and fume)
Manitoba	OEL TWA (mg/m^3)	10 mg/m^3 (inhalable particulate matter)
New Brunswick	OEL TWA (mg/m^3)	10 mg/m^3 (fume)
Newfoundland & Labrador	OEL TWA (mg/m^3)	10 mg/m^3 (inhalable particulate matter)
Nova Scotia	OEL TWA (mg/m^3)	10 mg/m^3 (inhalable particulate matter)
Nunavut	OEL STEL (mg/m^3)	20 mg/m^3 (inhalable fraction)
Nunavut	OEL TWA (mg/m^3)	10 mg/m^3 (inhalable fraction)
Northwest Territories	OEL STEL (mg/m^3)	20 mg/m^3 (inhalable fraction)
Northwest Territories	OEL TWA (mg/m^3)	10 mg/m^3 (inhalable fraction)
Ontario	OEL TWA (mg/m^3)	10 mg/m^3 (inhalable)
Prince Edward Island	OEL TWA (mg/m^3)	10 mg/m^3 (inhalable particulate matter)
Québec	VEMP (mg/m^3)	10 mg/m^3 (fume)
Saskatchewan	OEL STEL (mg/m^3)	20 mg/m^3 (inhalable fraction)
Saskatchewan	OEL TWA (mg/m^3)	10 mg/m^3 (inhalable fraction)
Yukon	OEL STEL (mg/m^3)	10 mg/m^3 (fume)
Yukon	OEL TWA (mg/m^3)	10 mg/m^3 (fume)

8.2. Exposure Controls

Appropriate Engineering Controls: Emergency eye wash fountains and safety showers should be available in the immediate vicinity of any potential exposure. Ensure adequate ventilation, especially in confined areas. Ensure all national/local regulations are observed.

Personal Protective Equipment: Gloves. Protective clothing. Protective goggles. Insufficient ventilation and/or dust generation: wear respiratory protection.

Materials for Protective Clothing: Chemically resistant materials and fabrics. Corrosion-proof clothing.

Hand Protection: Wear protective gloves.

Eye and Face Protection: Chemical safety goggles and face shield.

Skin and Body Protection: Wear suitable protective clothing.

Respiratory Protection: If exposure limits are exceeded or irritation is experienced, approved respiratory protection should be worn. In case of inadequate ventilation, oxygen deficient atmosphere, or where exposure levels are not known wear approved respiratory protection.

Other Information: When using, do not eat, drink or smoke.

SECTION 9: PHYSICAL AND CHEMICAL PROPERTIES

9.1. Information on Basic Physical and Chemical Properties

Physical State	:	Solid
Appearance	:	Gray, Off White or White Powder
Odor	:	Odorless
Odor Threshold	:	Not available
pH	:	12 - 13 (in water)
Evaporation Rate	:	Not available
Melting Point	:	Not available
Freezing Point	:	Not available
Boiling Point	:	> 1000 °C (> 1832 °F)
Flash Point	:	Not available
Auto-ignition Temperature	:	Not available
Decomposition Temperature	:	Not available
Flammability (solid, gas)	:	Not available
Lower Flammable Limit	:	Not available
Upper Flammable Limit	:	Not available
Vapor Pressure	:	Not available
Relative Vapor Density at 20°C	:	Not available
Specific Gravity	:	3.15 (Water = 1)
Solubility	:	Water: 0.1 - 1 % (slightly soluble)
Partition Coefficient: N-Octanol/Water	:	Not available
Viscosity	:	Not available

SECTION 10: STABILITY AND REACTIVITY

10.1. Reactivity: May react exothermically with water releasing heat. Adding an acid to a base or base to an acid may cause a violent reaction.

10.2. Chemical Stability: Stable under recommended handling and storage conditions (see Section 7).

10.3. Possibility of Hazardous Reactions: Hazardous polymerization will not occur.

10.4. Conditions to Avoid: Incompatible materials.

10.5. Incompatible Materials: Acids. Oxidizers. Ammonium salts. Aluminum metal. Diazomethane. Phosphorus.

10.6. Hazardous Decomposition Products: None expected under normal conditions of use.

SECTION 11: TOXICOLOGICAL INFORMATION

11.1. Information on Toxicological Effects - Product

Acute Toxicity (Oral): Not classified
Acute Toxicity (Dermal): Not classified
Acute Toxicity (Inhalation): Not classified
LD50 and LC50 Data: Not available

Skin Corrosion/Irritation: Causes severe skin burns and eye damage.
pH: 12 - 13 (in water)
Eye Damage/Irritation: Causes serious eye damage.
pH: 12 - 13 (in water)
Respiratory or Skin Sensitization: May cause an allergic skin reaction.
Germ Cell Mutagenicity: Not classified
Carcinogenicity: May cause cancer (Inhalation).
Specific Target Organ Toxicity (Repeated Exposure): Not classified
Reproductive Toxicity: Not classified
Specific Target Organ Toxicity (Single Exposure): May cause respiratory irritation.
Aspiration Hazard: Not classified
Symptoms/Injuries After Inhalation: Irritation of the respiratory tract and the other mucous membranes. May be corrosive to the respiratory tract. The three types of silicosis include: 1) Simple chronic silicosis – which results from long-term exposure (more than 20 years) to low amounts of respirable crystalline silica. Nodules of chronic inflammation and scarring provoked by the respirable crystalline silica form in the lungs and chest lymph nodes. This disease may feature breathlessness and may resemble chronic obstructive pulmonary disease (COPD); 2) Accelerated silicosis – occurs after exposure to larger amounts of respirable crystalline silica over a shorter period of time (5-15 years); 3) Acute silicosis – results from short-term exposure to very large amounts of respirable crystalline silica. The lungs become very inflamed and may fill with fluid, causing severe shortness of breath and low blood oxygen levels. Inflammation, scarring, and symptoms progress faster in accelerated silicosis than in simple silicosis. Progressive massive fibrosis may occur in simple or accelerated silicosis, but is more common in the accelerated form. Progressive massive fibrosis results from severe scarring and leads to the destruction of normal lung structures.
Symptoms/Injuries After Skin Contact: Concrete may cause dry skin, discomfort, irritation, severe burns, and dermatitis. Exposure of sufficient duration to wet concrete can cause serious, potentially irreversible damage to skin, eye, respiratory and digestive tracts due to chemical (caustic) burns, including third degree burns. A skin exposure may be hazardous even if there is no pain or discomfort. Unhardened concrete is capable of causing dermatitis by irritation and allergy. Skin affected by dermatitis may include symptoms such as, redness, itching, rash, scaling, and cracking. Irritant dermatitis is caused by the physical properties of concrete including alkalinity and abrasion. Allergic contact dermatitis is caused by sensitization to hexavalent chromium (chromate) potentially present in concrete. The reaction can range from a mild rash to severe skin ulcers. Persons already sensitized may react to the first contact with wet concrete. Others may develop allergic dermatitis after years of repeated contact with wet concrete. May cause an allergic skin reaction.
Symptoms/Injuries After Eye Contact: Potentially causes permanent damage to the cornea, iris, or conjunctiva. Airborne dust may cause immediate or delayed irritation or inflammation. Eye contact with large amounts of dry powder or with wet cement can cause moderate eye irritation, chemical burns and blindness. Eye exposures require immediate first aid and medical attention to prevent significant damage to the eye.
Symptoms/Injuries After Ingestion: May cause burns or irritation of the linings of the mouth, throat, and gastrointestinal tract.
Chronic Symptoms: May cause cancer.

11.2. Information on Toxicological Effects - Ingredient(s)

LD50 and LC50 Data:

Quartz (14808-60-7)	
LD50 Oral Rat	> 5000 mg/kg
LD50 Dermal Rat	> 5000 mg/kg
Calcium oxide (1305-78-8)	
LD50 Oral Rat	> 2000 mg/kg
LD50 Dermal Rabbit	> 2500 mg/kg
Magnesium oxide (MgO) (1309-48-4)	

LD50 Oral Rat	3870 mg/kg

Quartz (14808-60-7)	
IARC Group	1
National Toxicology Program (NTP) Status	Known Human Carcinogens.
OSHA Hazard Communication Carcinogen List	In OSHA Hazard Communication Carcinogen list.

SECTION 12: ECOLOGICAL INFORMATION

12.1. Toxicity

Ecology - General: High pH (alkalinity) of product may be harmful to aquatic life.

Calcium oxide (1305-78-8)	
LC50 Fish 1	50.6 mg/l

12.2. Persistence and Degradability

Trinity® White Cement	
Persistence and Degradability	Not established.

12.3. Bioaccumulative Potential

Trinity® White Cement	
Bioaccumulative Potential	Not established.

Calcium oxide (1305-78-8)	
BCF Fish 1	(no bioaccumulation)

12.4. Mobility in Soil

Not available

12.5. Other Adverse Effects

Other Information: Avoid release to the environment.

SECTION 13: DISPOSAL CONSIDERATIONS

13.1. Waste treatment methods

Waste Disposal Recommendations: Dispose of contents/container in accordance with local, regional, national, and international regulations.

Additional Information: Container may remain hazardous when empty. Continue to observe all precautions.

Ecology - Waste Materials: Avoid release to the environment.

SECTION 14: TRANSPORT INFORMATION

The shipping description(s) stated herein were prepared in accordance with certain assumptions at the time the SDS was authored, and can vary based on a number of variables that may or may not have been known at the time the SDS was issued.

14.1. In Accordance with DOT Not regulated for transport

14.2. In Accordance with IMDG Not regulated for transport

14.3. In Accordance with IATA Not regulated for transport

14.4. In Accordance with TDG Not regulated for transport

SECTION 15: REGULATORY INFORMATION

15.1. US Federal Regulations

Trinity® White Cement	
SARA Section 311/312 Hazard Classes	Health hazard - Serious eye damage or eye irritation Health hazard - Specific target organ toxicity (single or repeated exposure) Health hazard - Carcinogenicity Health hazard - Skin corrosion or Irritation Health hazard - Respiratory or skin sensitization

Quartz (14808-60-7)
Listed on the United States TSCA (Toxic Substances Control Act) inventory

Limestone (1317-65-3)
Listed on the United States TSCA (Toxic Substances Control Act) inventory

Cement, portland, chemicals (65997-15-1)

Listed on the United States TSCA (Toxic Substances Control Act) inventory

Calcium oxide (1305-78-8)
Listed on the United States TSCA (Toxic Substances Control Act) inventory

Magnesium oxide (MgO) (1309-48-4)
Listed on the United States TSCA (Toxic Substances Control Act) inventory

15.2. US State Regulations

Quartz (14808-60-7)	
U.S. - California - Proposition 65 - Carcinogens List	WARNING: This product contains chemicals known to the State of California to cause cancer.

Quartz (14808-60-7)
U.S. - Massachusetts - Right To Know List U.S. - New Jersey - Right to Know Hazardous Substance List U.S. - Pennsylvania - RTK (Right to Know) List

Limestone (1317-65-3)
U.S. - Massachusetts - Right To Know List U.S. - New Jersey - Right to Know Hazardous Substance List U.S. - Pennsylvania - RTK (Right to Know) List

Cement, portland, chemicals (65997-15-1)
U.S. - Massachusetts - Right To Know List U.S. - New Jersey - Right to Know Hazardous Substance List U.S. - Pennsylvania - RTK (Right to Know) List

Gypsum (Ca(SO4).2H2O) (13397-24-5)
U.S. - New Jersey - Right to Know Hazardous Substance List U.S. - Pennsylvania - RTK (Right to Know) List

Calcium oxide (1305-78-8)
U.S. - Massachusetts - Right To Know List U.S. - New Jersey - Right to Know Hazardous Substance List U.S. - Pennsylvania - RTK (Right to Know) List

Magnesium oxide (MgO) (1309-48-4)
U.S. - Massachusetts - Right To Know List U.S. - New Jersey - Right to Know Hazardous Substance List U.S. - Pennsylvania - RTK (Right to Know) List

15.3. Canadian Regulations

Quartz (14808-60-7)
Listed on the Canadian DSL (Domestic Substances List)

Limestone (1317-65-3)
Listed on the Canadian NDSL (Non-Domestic Substances List)

Cement, portland, chemicals (65997-15-1)
Listed on the Canadian DSL (Domestic Substances List)

Gypsum (Ca(SO4).2H2O) (13397-24-5)
Listed on the Canadian DSL (Domestic Substances List)

Calcium oxide (1305-78-8)
Listed on the Canadian DSL (Domestic Substances List)

Magnesium oxide (MgO) (1309-48-4)
Listed on the Canadian DSL (Domestic Substances List)

SECTION 16: OTHER INFORMATION, INCLUDING DATE OF PREPARATION OR LAST REVISION

Date of Preparation or Latest Revision : 04/23/2018

Other Information : This document has been prepared in accordance with the SDS requirements of the OSHA Hazard Communication Standard 29 CFR 1910.1200 and Canada's Hazardous Products Regulations (HPR) SOR/2015-17.

GHS Full Text Phrases:

Aquatic Acute 3	Hazardous to the aquatic environment - Acute Hazard Category 3
Carc. 1A	Carcinogenicity Category 1A
Eye Dam. 1	Serious eye damage/eye irritation Category 1
Skin Corr. 1C	Skin corrosion/irritation Category 1C
Skin Irrit. 2	Skin corrosion/irritation Category 2
Skin Sens. 1	Skin sensitization, Category 1
STOT RE 1	Specific target organ toxicity (repeated exposure) Category 1
STOT SE 3	Specific target organ toxicity (single exposure) Category 3
H314	Causes severe skin burns and eye damage
H315	Causes skin irritation
H317	May cause an allergic skin reaction
H318	Causes serious eye damage
H335	May cause respiratory irritation
H350	May cause cancer
H372	Causes damage to organs through prolonged or repeated exposure
H402	Harmful to aquatic life

An electronic version of this SDS is available: for Canada on www.lafarge.ca under the Health and Safety Section, and for US on www.lafargeholcim.us under the Our Solutions and Products Section. Please direct any inquiries regarding the content of this SDS to SDSinfo@Lafarge.com.

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NO WARRANTY IS MADE, EXPRESS OR IMPLIED, OF MERCHANTABILITY, FITNESS FOR A PARTICULAR PURPOSE, OR OTHERWISE.

NA GHS SDS 2015 (Can, US, Mex)

Section 05
Manufacturing Certification

CANTSINK™
SUPPORTING INNOVATION

71 First Avenue NW Lilburn, GA 30047

678-280-PILE (7453)

January 29, 2015

To Whom it Might Concern:

RE: Fabrication of Helical Piles and other accessories

We are writing to confirm that our company has been manufacturing the above referenced products since 2003 and most recently received the prestigious accreditation by the international recognized agency ICC with report number 1559.

Let us know if there is any other information that we need to provide to help achieve your goals.

Sincerely,

P Hutchinson

Patrick Hutchinson

Manager

Section 06
Design Engineer Certification

The South Carolina

State Board of Registration

for

Professional Engineers and Surveyors

This is to certify that

Philip E. Slemons

having given satisfactory evidence of the necessary qualifications
as required by Title 40, Chapter 22, Code of Laws of
South Carolina, 1976, has been duly registered and
is hereby authorized to practice

Engineering

in the State of South Carolina.

REGISTRATION FOR PROFESSIONAL ENGINEERS & LAND SURVEYORS

No. 27217

In Testimony Whereof Witness the signature of the Chairman and Secretary under the seal of the Board this the seventeenth day of March 1009.

Gene L. Dinkins

Gene L. Dinkins. PLS, PE - Chairman

Theresa H. Hodge

Theresa H. Hodge. PE - Secretary

Section 07
Installer Certification

CANTSINK
SUPPORTING INNOVATION

CERTIFICATE

This document certifies that

CNT Foundations LLC

has successfully completed the Cantsink helical pile installation course and is qualified in the methods, procedures and techniques necessary to install Cantsink helical pile products. Given at Winder, Georgia on this the 18th day of November 2011.

P Hutchinson

Manager

Ed Adams

Instructor

Section 08
ICC Requirements Per Code

1810.3.1.5 Helical Piles. Helical piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by installation into the ground and service loads.

1810.3.5.3.3 Helical Piles. Dimensions of the central shaft and the number, size and thicknesses of helical bearing plates shall be sufficient to support the design loads.

1810.4.10 Helical piles. Helical piles shall be installed to specified embedment depth and torsional resistance criteria as determined by a registered design professional. The torque applied during installation shall not exceed the maximum allowable installation torque of the helical pile.

1810.4.1011 Special inspection. Special inspections in accordance with Sections 1704.8 and 1704.9 shall be provided for driven and cast-in-place deep foundation elements, respectively. Special inspections in accordance with Section 1704.10 shall be provided for helical piles.

1704.10 Helical Pile Foundations. Special inspections shall be performed continuously during installation of helical pile foundations. The information recorded shall include installation equipment used, pile dimensions, tip elevations, final depth, final installation torque, and other pertinent installation data as required by the registered design professional in responsible charge. The approved geotechnical report and the documents prepared by the registered design professional shall be used to determine compliance.

17.3 PRODUCT EVALUATION REPORTS

The International Building Code requires that all manufactured products used in construction, including helical foundations, have a current product evaluation report. ICC-Evaluation Services, Inc. (ICC-ES), the product evaluation arm of the ICC, publishes these reports. Evaluation reports contain product capacity information as well as design and installation guidelines.

There are currently over 50 helical foundation manufacturers in the world. Manufacturing quality varies considerably. Most do not have ICC-ES product evaluation reports. A small number of manufacturers have a "legacy" evaluation report. Legacy reports are those published prior to the formation of ICC-ES by old code agencies, such as the International Conference of Building Officials (ICBO) or Building Officials and Code Administrators International (BOCA). However, a standard guideline for evaluation of helical foundations did not exist when these reports were written. As a result, they contain limited information. Manufacturing quality is not calibrated

Exception: Where a specific design is not provided, masonry-unit footings supporting walls of light-frame construction shall be permitted to be designed in accordance with Table 1809.7.

1809.9.1 Dimensions. Masonry-unit footings shall be laid in Type M or S mortar complying with Section 2103.2.1 and the depth shall be not less than twice the projection beyond the wall, pier or column. The width shall be not less than 8 inches (203 mm) wider than the wall supported thereon.

1809.9.2 Offsets. The maximum offset of each course in brick foundation walls stepped up from the footings shall be $1^1/_2$ inches (38 mm) where laid in single courses, and 3 inches (76 mm) where laid in double courses.

1809.10 Pier and curtain wall foundations. Except in *Seismic Design Categories* D, E and F, pier and curtain wall foundations shall be permitted to be used to support light-frame construction not more than two *stories above grade plane*, provided that the following requirements are met:

1. All load-bearing walls shall be placed on continuous concrete footings bonded integrally with the *exterior wall* footings.
2. The minimum actual thickness of a load-bearing masonry wall shall be not less than 4 inches (102 mm) nominal or $3^5/_8$ inches (92 mm) actual thickness, and shall be bonded integrally with piers spaced 6 feet (1829 mm) on center (o.c.).
3. Piers shall be constructed in accordance with Chapter 21 and the following:
   - 3.1. The unsupported height of the masonry piers shall not exceed 10 times their least dimension.
   - 3.2. Where structural clay tile or hollow concrete masonry units are used for piers supporting beams and girders, the cellular spaces shall be filled solidly with concrete or Type M or S mortar.

     Exception: Unfilled hollow piers shall be permitted where the unsupported height of the pier is not more than four times its least dimension.
   - 3.3. Hollow piers shall be capped with 4 inches (102 mm) of solid masonry or concrete or the cavities of the top course shall be filled with concrete or grout.
4. The maximum height of a 4-inch (102 mm) load-bearing masonry foundation wall supporting wood frame walls and floors shall not be more than 4 feet (1219 mm) in height.
5. The unbalanced fill for 4-inch (102 mm) foundation walls shall not exceed 24 inches (610 mm) for solid masonry, nor 12 inches (305 mm) for hollow masonry.

1809.11 Steel grillage footings. Grillage footings of *structural steel elements* shall be separated with *approved* steel spacers and be entirely encased in concrete with not less than 6 inches (152 mm) on the bottom and not less than 4 inches (102 mm) at all other points. The spaces between the shapes shall be completely filled with concrete or cement grout.

1809.12 Timber footings. Timber footings shall be permitted for buildings of Type V construction and as otherwise *approved* by the *building official*. Such footings shall be treated in accordance with AWPA U1 (Commodity Specification A, Use Category 4B). Treated timbers are not required where placed entirely below permanent water level, or where used as capping for wood piles that project above the water level over submerged or marsh lands. The compressive stresses perpendicular to grain in untreated timber footings supported on treated piles shall not exceed 70 percent of the allowable stresses for the species and grade of timber as specified in the ANSI/AWC NDS.

1809.13 Footing seismic ties. Where a structure is assigned to *Seismic Design Category* D, E or F, individual spread footings founded on soil defined in Chapter 20 of ASCE 7 as *Site Class* E or F shall be interconnected by ties. Unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade, ties shall be capable of carrying, in tension or compression, a force equal to the lesser of the product of the larger footing design gravity load times the seismic coefficient, S_{DS}, divided by 10 and 25 percent of the smaller footing design gravity load.

SECTION 1810 DEEP FOUNDATIONS

1810.1 General. Deep foundations shall be analyzed, designed, detailed and installed in accordance with Sections 1810.1 through 1810.4.

1810.1.1 Geotechnical investigation. Deep foundations shall be designed and installed on the basis of a geotechnical investigation as set forth in Section 1803.

1810.1.2 Use of existing deep foundation elements. Deep foundation elements left in place where a structure has been demolished shall not be used for the support of new construction unless satisfactory evidence is submitted to the *building official*, which indicates that the elements are sound and meet the requirements of this code. Such elements shall be load tested or redriven to verify their capacities. The design load applied to such elements shall be the lowest allowable load as determined by tests or redriving data.

1810.1.3 Deep foundation elements classified as columns. Deep foundation elements standing unbraced in air, water or fluid soils shall be classified as columns and designed as such in accordance with the provisions of this code from their top down to the point where adequate lateral support is provided in accordance with Section 1810.2.1.

Exception: Where the unsupported height to least horizontal dimension of a cast-in-place deep foundation element does not exceed three, it shall be permitted to design and construct such an element as a pedestal in accordance with ACI 318.

1810.1.4 Special types of deep foundations. The use of types of deep foundation elements not specifically mentioned herein is permitted, subject to the approval of the *building official*, upon the submission of acceptable test data, calculations and other information relating to the structural properties and load capacity of such elements. The allowable stresses for materials shall not in any case exceed the limitations specified herein.

1810.2 Analysis. The analysis of deep foundations for design shall be in accordance with Sections 1810.2.1 through 1810.2.5.

1810.2.1 Lateral support. Any soil other than fluid soil shall be deemed to afford sufficient lateral support to prevent buckling of deep foundation elements and to permit the design of the elements in accordance with accepted engineering practice and the applicable provisions of this code.

Where deep foundation elements stand unbraced in air, water or fluid soils, it shall be permitted to consider them laterally supported at a point 5 feet (1524 mm) into stiff soil or 10 feet (3048 mm) into soft soil unless otherwise *approved* by the *building official* on the basis of a geotechnical investigation by a *registered design professional*.

1810.2.2 Stability. Deep foundation elements shall be braced to provide lateral stability in all directions. Three or more elements connected by a rigid cap shall be considered to be braced, provided that the elements are located in radial directions from the centroid of the group not less than 60 degrees (1 rad) apart. A two-element group in a rigid cap shall be considered to be braced along the axis connecting the two elements. Methods used to brace deep foundation elements shall be subject to the approval of the *building official*.

Deep foundation elements supporting walls shall be placed alternately in lines spaced not less than 1 foot (305 mm) apart and located symmetrically under the center of gravity of the wall load carried, unless effective measures are taken to provide for eccentricity and lateral forces, or the foundation elements are adequately braced to provide for lateral stability.

Exceptions:

1. Isolated cast-in-place deep foundation elements without lateral bracing shall be permitted where the least horizontal dimension is not less than 2 feet (610 mm), adequate lateral support in accordance with Section 1810.2.1 is provided for the entire height and the height does not exceed 12 times the least horizontal dimension.
2. A single row of deep foundation elements without lateral bracing is permitted for one- and two-family dwellings and lightweight construction not exceeding two *stories above grade plane* or 35 feet (10 668 mm) in *building height*, provided that the centers of the elements are located within the width of the supported wall.

1810.2.3 Settlement. The settlement of a single deep foundation element or group thereof shall be estimated based on *approved* methods of analysis. The predicted settlement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any element to be loaded beyond its capacity.

1810.2.4 Lateral loads. The moments, shears and lateral deflections used for design of deep foundation elements shall be established considering the nonlinear interaction of the shaft and soil, as determined by a *registered design professional*. Where the ratio of the depth of embedment of the element to its least horizontal dimension is less than or equal to six, it shall be permitted to assume the element is rigid.

1810.2.4.1 Seismic Design Categories D through F. For structures assigned to *Seismic Design Category* D, E or F, deep foundation elements on *Site Class* E or F sites, as determined in Section 1613.2.2, shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soil-foundation-structure interaction coupled with foundation element deformations associated with earthquake loads imparted to the foundation by the structure.

Exception: Deep foundation elements that satisfy the following additional detailing requirements shall be deemed to comply with the curvature capacity requirements of this section.

1. Precast prestressed concrete piles detailed in accordance with Section 1810.3.8.3.3.
2. Cast-in-place deep foundation elements with a minimum longitudinal reinforcement ratio of 0.005 extending the full length of the element and detailed in accordance with Sections 18.7.5.2, 18.7.5.3 and 18.7.5.4 of ACI 318 as required by Section 1810.3.9.4.2.2.

1810.2.5 Group effects. The analysis shall include group effects on lateral behavior where the center-to-center spacing of deep foundation elements in the direction of lateral force is less than eight times the least horizontal dimension of an element. The analysis shall include group effects on axial behavior where the center-to-center spacing of deep foundation elements is less than three times the least horizontal dimension of an element. Group effects shall be evaluated using a generally accepted method of analysis; the analysis for uplift of grouped elements with center-to-center spacing less than three times the least horizontal dimension of an element shall be evaluated in accordance with Section 1810.3.3.1.6.

1810.3 Design and detailing. Deep foundations shall be designed and detailed in accordance with Sections 1810.3.1 through 1810.3.13.

1810.3.1 Design conditions. Design of deep foundations shall include the design conditions specified in Sections 1810.3.1.1 through 1810.3.1.6, as applicable.

1810.3.1.1 Design methods for concrete elements. Where concrete deep foundations are laterally supported in accordance with Section 1810.2.1 for the entire height and applied forces cause bending moments not greater than those resulting from accidental eccentricities, structural design of the element using the load combinations of Section 1605.3 and the allowable stresses specified in this chapter shall be permitted. Otherwise, the structural design of concrete deep foundation elements shall use the load combinations of Section 1605.2 and *approved* strength design methods.

1810.3.1.2 Composite elements. Where a single deep foundation element comprises two or more sections of different materials or different types spliced together, each section of the composite assembly shall satisfy the applicable requirements of this code, and the maximum allowable load in each section shall be limited by the structural capacity of that section.

1810.3.1.3 Mislocation. The foundation or superstructure shall be designed to resist the effects of the mislocation of any deep foundation element by not less than 3 inches (76 mm). To resist the effects of mislocation, compressive overload of deep foundation elements to 110 percent of the allowable design load shall be permitted.

1810.3.1.4 Driven piles. Driven piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by handling, driving and service loads.

1810.3.1.5 Helical piles. Helical piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by installation into the ground and service loads.

1810.3.1.6 Casings. Temporary and permanent casings shall be of steel and shall be sufficiently strong to resist collapse and sufficiently water tight to exclude any foreign materials during the placing of concrete. Where a permanent casing is considered reinforcing steel, the steel shall be protected under the conditions specified in Section 1810.3.2.5. Horizontal joints in the casing shall be spliced in accordance with Section 1810.3.6.

1810.3.2 Materials. The materials used in deep foundation elements shall satisfy the requirements of Sections 1810.3.2.1 through 1810.3.2.8, as applicable.

1810.3.2.1 Concrete. Where concrete is cast in a steel pipe or where an enlarged base is formed by compacting concrete, the maximum size for coarse aggregate shall be $^3/_4$ inch (19.1 mm). Concrete to be compacted shall have a zero slump.

1810.3.2.1.1 Seismic hooks. For structures assigned to *Seismic Design Category* C, D, E or F, the ends of hoops, spirals and ties used in concrete deep foundation elements shall be terminated with seismic hooks, as defined in ACI 318, and shall be turned into the confined concrete core.

1810.3.2.1.2 ACI 318 Equation (25.7.3.3). Where this chapter requires detailing of concrete deep foundation elements in accordance with Section 18.7.5.4 of ACI 318, compliance with Equation (25.7.3.3) of ACI 318 shall not be required.

1810.3.2.2 Prestressing steel. Prestressing steel shall conform to ASTM A416.

1810.3.2.3 Steel. Structural steel H-piles and structural steel sheet piling shall conform to the material requirements in ASTM A6. Steel pipe piles shall conform to the material requirements in ASTM A252. Fully welded steel piles shall be fabricated from plates that conform to the material requirements in ASTM A36, ASTM A283, ASTM A572, ASTM A588 or ASTM A690.

1810.3.2.4 Timber. Timber deep foundation elements shall be designed as piles or poles in accordance with ANSI/AWC NDS. Round timber elements shall conform to ASTM D25. Sawn timber elements shall conform to DOC PS-20.

1810.3.2.4.1 Preservative treatment. Timber deep foundation elements used to support permanent structures shall be treated in accordance with this section unless it is established that the tops of the untreated timber elements will be below the lowest ground-water level assumed to exist during the life of the structure. Preservative and minimum final retention shall be in accordance with AWPA U1 (Commodity Specification E, Use Category 4C) for round timber elements and AWPA U1 (Commodity Specification A, Use Category 4B) for sawn timber elements. Preservative-treated timber elements shall be subject to a quality control program administered by an *approved agency*. Element cutoffs shall be treated in accordance with AWPA M4.

1810.3.2.5 Protection of materials. Where boring records or site conditions indicate possible deleterious action on the materials used in deep foundation elements because of soil constituents, changing water levels or other factors, the elements shall be adequately protected by materials, methods or processes *approved* by the *building official*. Protective materials shall be applied to the elements so as not to be rendered ineffective by installation. The effectiveness of such protective measures for the particular purpose shall have been thoroughly established by satisfactory service records or other evidence.

1810.3.2.6 Allowable stresses. The allowable stresses for materials used in deep foundation elements shall not exceed those specified in Table 1810.3.2.6.

1810.3.2.7 Increased allowable compressive stress for cased mandrell-driven cast-in-place elements. The allowable compressive stress in the concrete shall be permitted to be increased as specified in Table 1810.3.2.6 for those portions of permanently cased cast-in-place elements that satisfy all of the following conditions:

1. The design shall not use the casing to resist any portion of the axial load imposed.
2. The casing shall have a sealed tip and be mandrel driven.

Acceptable engineering practices are outlined in AC358

1810.3.3.1.2 Load tests. Where design compressive loads are greater than those determined using the allowable stresses specified in Section 1810.3.2.6, where the design load for any deep foundation element is in doubt, or where cast-in-place deep foundation elements have an enlarged base formed either by compacting concrete or by driving a precast base, control test elements shall be tested in accordance with ASTM D1143 or ASTM D4945. One element or more shall be load tested in each area of uniform subsoil conditions. Where required by the *building official*, additional elements shall be load tested where necessary to establish the safe design capacity. The resulting allowable loads shall not be more than one-half of the ultimate axial load capacity of the test element as assessed by one of the published methods listed in Section 1810.3.3.1.3 with consideration for the test type, duration and subsoil. The ultimate axial load capacity shall be determined by a *registered design professional* with consideration given to tolerable total and differential settlements at design load in accordance with Section 1810.2.3. In subsequent installation of the balance of deep foundation elements, all elements shall be deemed to have a supporting capacity equal to that of the control element where such elements are of the same type, size and relative length as the test element; are installed using the same or comparable methods and equipment as the test element; are installed in similar subsoil conditions as the test element; and, for driven elements, where the rate of penetration (for example, net displacement per blow) of such elements is equal to or less than that of the test element driven with the same hammer through a comparable driving distance.

1810.3.3.1.3 Load test evaluation methods. It shall be permitted to evaluate load tests of deep foundation elements using any of the following methods:

1. Davisson Offset Limit.
2. Brinch-Hansen 90-percent Criterion.
3. Butler-Hoy Criterion.
4. Other methods *approved* by the *building official*.

1810.3.3.1.4 Allowable shaft resistance. The assumed shaft resistance developed by any uncased cast-in-place deep foundation element shall not exceed one-sixth of the bearing value of the soil material at minimum depth as set forth in Table 1806.2, up to 500 psf (24 kPa), unless a greater value is allowed by the *building official* on the basis of a geotechnical investigation as specified in Section 1803 or a greater value is substantiated by a load test in accordance with Section 1810.3.3.1.2. Shaft resistance and end-bearing resistance shall not be assumed to act simultaneously unless determined by a geotechnical investigation in accordance with Section 1803.

1810.3.3.1.5 Uplift capacity of a single deep foundation element. Where required by the design, the uplift capacity of a single deep foundation element shall be determined by an *approved* method of analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D3689. The maximum allowable uplift load shall not exceed the ultimate load capacity as determined in Section 1810.3.3.1.2, using the results of load tests conducted in accordance with ASTM D3689, divided by a factor of safety of two.

> **Exception:** Where uplift is due to wind or seismic loading, the minimum factor of safety shall be two where capacity is determined by an analysis and one and one-half where capacity is determined by load tests.

1810.3.3.1.6 Allowable uplift load of grouped deep foundation elements. For grouped deep foundation elements subjected to uplift, the allowable uplift load for the group shall be calculated by a generally accepted method of analysis. Where the deep foundation elements in the group are placed at a center-to-center spacing less than three times the least horizontal dimension of the largest single element, the allowable uplift load for the group is permitted to be calculated as the lesser of:

1. The proposed individual allowable uplift load times the number of elements in the group.
2. Two-thirds of the effective weight of the group and the soil contained within a block defined by the perimeter of the group and the length of the element, plus two-thirds of the ultimate shear resistance along the soil block.

1810.3.3.1.7 Load-bearing capacity. Deep foundation elements shall develop ultimate load capacities of not less than twice the design working loads in the designated load-bearing layers. Analysis shall show that soil layers underlying the designated load-bearing layers do not cause the load-bearing capacity safety factor to be less than two.

1810.3.3.1.8 Bent deep foundation elements. The load-bearing capacity of deep foundation elements discovered to have a sharp or sweeping bend shall be determined by an *approved* method of analysis or by load testing a representative element.

1810.3.3.1.9 Helical piles. The allowable axial design load, P_a, of helical piles shall be determined as follows:

$$P_a = 0.5\, P_u \qquad \textbf{(Equation 18-4)}$$

where P_u is the least value of:

1. Sum of the areas of the helical bearing plates times the ultimate bearing capacity of the soil or rock comprising the bearing stratum.
2. Ultimate capacity determined from well-documented correlations with installation torque.
3. Ultimate capacity determined from load tests.

Well documented correlations are tested in accordance with AC 358 & documented by the ICC or IAPMO wall thickness of not less than $^3/_8$ inch (9.5 mm) and a suitable steel driving shoe welded to the bottom; the diameter of the rock socket shall be approximately equal to the inside diameter of the casing.

Exceptions:

1. There is no minimum diameter for steel pipes or tubes used in micropiles.
2. For mandrel-driven pipes or tubes, the minimum wall thickness shall be $^1/_{10}$ inch (2.5 mm).

1810.3.5.3.5 Helical piles. Dimensions of the central shaft and the number, size and thickness of helical bearing plates shall be sufficient to support the design loads.

1810.3.6 Splices. Splices shall be constructed so as to provide and maintain true alignment and position of the component parts of the deep foundation element during installation and subsequent thereto and shall be designed to resist the axial and shear forces and moments occurring at the location of the splice during driving and for design load combinations. Where deep foundation elements of the same type are being spliced, splices shall develop not less than 50 percent of the bending strength of the weaker section. Where deep foundation elements of different materials or different types are being spliced, splices shall develop the full compressive strength and not less than 50 percent of the tension and bending strength of the weaker section. Where structural steel cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded.

Splices occurring in the upper 10 feet (3048 mm) of the embedded portion of an element shall be designed to resist at allowable stresses the moment and shear that would result from an assumed eccentricity of the axial load of 3 inches (76 mm), or the element shall be braced in accordance with Section 1810.2.2 to other deep foundation elements that do not have splices in the upper 10 feet (3048 mm) of embedment.

1810.3.6.1 Seismic Design Categories C through F. For structures assigned to *Seismic Design Category* C, D, E or F splices of deep foundation elements shall develop the lesser of the following:

1. The nominal strength of the deep foundation element.
2. The axial and shear forces and moments from the seismic load effects including overstrength factor in accordance with Section 2.3.6 or 2.4.5 of ASCE 7.

1810.3.7 Top of element detailing at cutoffs. Where a minimum length for reinforcement or the extent of closely spaced confinement reinforcement is specified at the top of a deep foundation element, provisions shall be made so that those specified lengths or extents are maintained after cutoff.

1810.3.8 Precast concrete piles. Precast concrete piles shall be designed and detailed in accordance with Sections 1810.3.8.1 through 1810.3.8.3.

1810.3.8.1 Reinforcement. Longitudinal steel shall be arranged in a symmetrical pattern and be laterally tied with steel ties or wire spiral spaced center to center as follows:

1. At not more than 1 inch (25 mm) for the first five ties or spirals at each end; then
2. At not more than 4 inches (102 mm), for the remainder of the first 2 feet (610 mm) from each end; and then
3. At not more than 6 inches (152 mm) elsewhere.

The size of ties and spirals shall be as follows:

1. For piles having a least horizontal dimension of 16 inches (406 mm) or less, wire shall not be smaller than 0.22 inch (5.6 mm) (No. 5 gage).
2. For piles having a least horizontal dimension of more than 16 inches (406 mm) and less than 20 inches (508 mm), wire shall not be smaller than 0.238 inch (6 mm) (No. 4 gage).
3. For piles having a least horizontal dimension of 20 inches (508 mm) and larger, wire shall not be smaller than $^1/_4$ inch (6.4 mm) round or 0.259 inch (6.6 mm) (No. 3 gage).

1810.3.8.2 Precast nonprestressed piles. Precast nonprestressed concrete piles shall comply with the requirements of Sections 1810.3.8.2.1 through 1810.3.8.2.3.

1810.3.8.2.1 Minimum reinforcement. Longitudinal reinforcement shall consist of not fewer than four bars with a minimum longitudinal reinforcement ratio of 0.008.

1810.3.8.2.2 Seismic reinforcement in Seismic Design Categories C through F. For structures assigned to *Seismic Design Category* C, D, E or F, precast nonprestressed piles shall be reinforced as specified in this section. The minimum longitudinal reinforcement ratio shall be 0.01 throughout the length. Transverse reinforcement shall consist of closed ties or spirals with a minimum $^3/_8$ inch (9.5 mm) diameter. Spacing of transverse reinforcement shall not exceed the smaller of eight times the diameter of the smallest longitudinal bar or 6 inches (152 mm) within a distance of three times the least pile dimension from the bottom of the pile cap. Spacing of transverse reinforcement shall not exceed 6 inches (152 mm) throughout the remainder of the pile.

1810.3.8.2.3 Additional seismic reinforcement in Seismic Design Categories D through F. For structures assigned to *Seismic Design Category* D, E or F, transverse reinforcement shall be in accordance with Section 1810.3.9.4.2.

1810.3.8.3 Precast prestressed piles. Precast prestressed concrete piles shall comply with the requirements of Sections 1810.3.8.3.1 through 1810.3.8.3.3.

Has to be correlated through testing outlined in AC358 able quality returns at the top of the element. The following requirements apply to specific installation methods:

1. For micropiles grouted inside a temporary casing, the reinforcing bars shall be inserted prior to withdrawal of the casing.The casing shall be withdrawn in a controlled manner with the grout level maintained at the top of the element to ensure that the grout completely fills the drill hole. During withdrawal of the casing, the grout level inside the casing shall be monitored to verify that the flow of grout inside the casing is not obstructed.
2. For a micropile or portion thereof grouted in an open drill hole in soil without temporary casing, the minimum design diameter of the drill hole shall be veri-fied by a suitable device during grouting.
3. For micropiles designed for end bearing, a suitable means shall be employed to verify that the bearing surface is properly cleaned prior to grouting.
4. Subsequent micropiles shall not be drilled near elements that have been grouted until the grout has had sufficient time to harden.
5. Micropiles shall be grouted as soon as possible after drilling is completed.
6. For micropiles designed with a full-length casing, the casing shall be pulled back to the top of the bond zone and reinserted or some other suitable means employed to ensure grout coverage outside the casing.

1810.4.11 Helical piles. Helical piles shall be installed to specified embedment depth and torsional resistance crite-ria as determined by a *registered design professional*. The torque applied during installation shall not exceed the maximum allowable installation torque of the helical pile.

Installation torque or torque correlation can only be determined through testing in accordance with AC538 & documented by ICC or IAPMO

1810.4.12 Special inspection. *Special inspections* in accordance with Sections 1705.7 and 1705.8 shall be provided for driven and cast-in-place deep foundation elements, respectively. *Special inspections* in accordance with Section 1705.9 shall be provided for helical piles.

Section 09
AC358

ICC EVALUATION SERVICE, INC.
Evaluate ■ Inform ■ Protect

ACCEPTANCE CRITERIA FOR HELICAL FOUNDATION SYSTEMS AND DEVICES

AC358

Approved June 2007

Effective July 1, 2007

PREFACE

Evaluation reports issued by ICC Evaluation Service, Inc. (ICC-ES), are based upon performance features of the International family of codes and other widely adopted code families, including the Uniform Codes, the BOCA National Codes, and the SBCCI Standard Codes. Section 104.11 of the *International Building Code*® reads as follows:

> The provisions of this code are not intended to prevent the installation of any materials or to prohibit any design or method of construction not specifically prescribed by this code, provided that any such alternative has been approved. An alternative material, design or method of construction shall be approved where the building official finds that the proposed design is satisfactory and complies with the intent of the provisions of this code, and that the material, method or work offered is, for the purpose intended, at least the equivalent of that prescribed in this code in quality, strength, effectiveness, fire resistance, durability and safety.

Similar provisions are contained in the Uniform Codes, the National Codes, and the Standard Codes.

This acceptance criteria has been issued to provide all interested parties with guidelines for demonstrating compliance with performance features of the applicable code(s) referenced in the acceptance criteria. The criteria was developed and adopted following public hearings conducted by the ICC-ES Evaluation Committee, and is effective on the date shown above. All reports issued or reissued on or after the effective date must comply with this criteria, while reports issued prior to this date may be in compliance with this criteria or with the previous edition. If the criteria is an updated version from the previous edition, a solid vertical line (❙) in the margin within the criteria indicates a technical change, addition, or deletion from the previous edition. A deletion indicator (➜) is provided in the margin where a paragraph has been deleted if the deletion involved a technical change. This criteria may be further revised as the need dictates.

ICC-ES may consider alternate criteria, provided the report applicant submits valid data demonstrating that the alternate criteria are at least equivalent to the criteria set forth in this document, and otherwise demonstrate compliance with the performance features of the codes. Notwithstanding that a product, material, or type or method of construction meets the requirements of the criteria set forth in this document, or that it can be demonstrated that valid alternate criteria are equivalent to the criteria in this document and otherwise demonstrate compliance with the performance features of the codes, ICC-ES retains the right to refuse to issue or renew an evaluation report, if the product, material, or type or method of construction is such that either unusual care with its installation or use must be exercised for satisfactory performance, or if malfunctioning is apt to cause unreasonable property damage or personal injury or sickness relative to the benefits to be achieved by the use of the product, material, or type or method of construction.

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ACCEPTANCE CRITERIA FOR HELICAL FOUNDATION SYSTEMS AND DEVICES

1.0 INTRODUCTION

1.1 Purpose: The purpose of this acceptance criteria is to establish requirements for helical foundation systems and helical foundation devices to be recognized in ICC Evaluation Service, Inc. (ICC-ES), evaluation reports under the 2006 *International Building Code*® (IBC) and the 1997 *Uniform Building Code*™ (UBC). Bases for recognition are IBC Section 104.11 and UBC Section 104.2.8.

The reason for the development of this acceptance criteria is to supplement general requirements for pile foundations in the IBC and UBC to permit evaluation of helical foundation systems and devices.

1.2 Scope: This criteria provides methods to establish the allowable load and deformation capacities of helical foundation systems and devices used to resist axial compression, axial tension or lateral loads. This criteria applies to helical foundation systems and devices as defined in Section 1.4 and includes provisions for determining soil embedment and soil capacity.

This criteria is limited to helical foundation systems and devices used under the following conditions:

1.2.1 Support of structures in IBC Seismic Design Categories A, B, or C, or UBC Seismic Zones 0, 1 or 2, only.

1.2.2 Exposure conditions to soil that are not indicative of potential pile deterioration or corrosion situations as defined by the following: (1) soil resistivity less than 1,000 ohm-cm; (2) soil pH less than 5.5; (3) soils with high organic content; (4) soil sulfate concentrations greater than 1,000 ppm; (5) soils located in landfills, or (6) soil containing mine waste.

1.2.3 Helical products manufactured from carbon steel, with optional zinc or powder coatings.

1.3 Codes and Referenced Standards: Where standards are referenced in this criteria, these standards shall be applied consistently with the code (IBC, and UBC) upon which compliance is based in accordance with Table 1.

1.3.1 2006 *International Building Code*® (IBC), International Code Council.

1.3.2 1997 *Uniform Building Code* (UBC)™.

1.3.3 ICC-ES Acceptance Criteria for Inspection Agencies (AC304).

1.3.4 ANSI/AF&PA NDS, National Design Specification for Wood Construction (NDS), American Forest & Paper Association.

1.3.5 ACI 318-05, Building Code Requirements for Structural Concrete, American Concrete Institute.

1.3.6 Specification for Structural Steel Buildings, Load and Resistance Factor Design, 3rd Edition, American Institute of Steel Construction (AISC LRFD).

1.3.7 Specification for Structural Steel Buildings, Allowable Stress Design, American Institute of Steel Construction (AISC ASD).

1.3.8 ANSI/ASME Standard B18.2.1-1996, Square and Hex Bolts and Screws, Inch Series, American Society of Mechanical Engineers.

1.3.9 ANSI/AWS D1.1/D1.1M, Structural Welding Code—Steel (AWS D1.1/D1.1M), American Welding Society.

1.3.10 ASTM A 123-02, Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products, ASTM International.

1.3.11 ASTM A 153-05, Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware, ASTM International.

1.3.12 ASTM B 633-07 Standard Specification for Electro deposited Coatings of Zinc on Iron and Steel, ASTM International.

1.3.13 ASTM B 695-04 Standard Specification for Coatings of Zinc Mechanically Deposited on Iron and Steel, ASTM International.

1.3.14 ASTM C 31-98, Standard Practice for Making and Curing Concrete Test Specimens in the Field, ASTM International.

1.3.15 ASTM C 39-03, Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens, ASTM International.

1.3.16 ASTM D 1143-81(1994)e1, Standard Test Method for Piles Under Static Axial Compressive Load, ASTM International.

1.3.17 ASTM D 1586-99, Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils, ASTM International.

1.3.18 ASTM D 3689-90(1995), Standard Test Method for Individual Piles under Static Axial Tensile Load, ASTM International.

1.3.19 ASTM D 3966-90(1995), Standard Test Method for Piles under Lateral Loads, ASTM International.

1.3.20 ICC-ES Acceptance Criteria for Corrosion Protection of Steel Foundation Systems Using Polymer (EAA) Coatings (AC228).

1.4 Definitions: Terminology herein is based on the Glossary of the AISC LRFD and the following definitions:

1.4.1 Angle Bracket: A side load bracket with horizontal bearing plate extending below and supporting a concrete foundation.

1.4.2 Helical Foundation System: A factory-manufactured steel foundation designed to resist axial compression, axial tension, and/or lateral loads from structures, consisting of a central shaft with one or more helical-shaped bearing plates, extension shafts, and a bracket that allow for attachment to structures. The shafts with helix bearing plates are screwed into the ground by application of torsion and the shaft is extended until a desired depth or a suitable soil or bedrock bearing stratum is reached.

1.4.3 Helical Foundation Device: For purposes of this criteria, a helical foundation device is any part or component of a helical foundation system.

1.4.4 Lateral Resistance: Capacity of a helical foundation system or device to resist forces acting in a direction that is perpendicular to the longitudinal direction of the shaft.

1.4.5 Conventional Design: Methods for determining design capacities of the helical foundation system that are prescribed by and strictly in accordance with standards and codes referenced in Section 1.3.

1.4.6 Special Analysis: Methods for determining design capacities of the helical foundation system that incorporate finite element modeling, discrete element modeling, strain compatibility, or other conventional analytical/numerical techniques. Computer software developed for the analysis of laterally loaded piles, which incorporate methods of analysis considering the nonlinear interaction of the shaft with soil, is an example of special analysis.

2.0 BASIC INFORMATION

2.1 General: The following information shall be submitted with ICC-ES evaluation report applications:

2.1.1 Summary Document: A tabulated list of the helical foundation systems, devices, and combinations thereof to be included in the ICC-ES evaluation report, along with proposed structural capacities. All systems and devices shall be clearly identified in the documentation with distinct product names and/or product numbering.

2.1.2 Product Description: Helical products shall be manufactured from carbon steel, with optional zinc or powder coatings. Complete information pertaining to the helical foundation systems or devices, including material specifications and drawings showing all dimensions and tolerances, and the manufacturing processes. All materials, welding processes and manufacturing procedures used in helical foundation systems and devices shall be specified and described in quality documentation complying with Section 5.2. All material specifications shall comply with ASTM, ACI, NDS, AISC, UBC, or IBC requirements. Material composition, grade, and sizes of bolts and fasteners shall be based on criteria in AISC, ASTM, or ANSI requirements.

2.1.3 Installation Instructions: Procedures and details regarding helical foundation system or device installation, including product-specific requirements, exclusions, limitations, and inspection requirements, as applicable.

2.1.4 Packaging and Identification: A description of the method of packaging and field identification of each helical foundation system device. Identification provisions shall include the manufacturer's name and address, product name and model number, evaluation report number and name or logo of the inspection agency.

2.1.5 Design Calculations: Clear and comprehensive calculations of ASD or LRFD structural capacities for system or device, based on requirements of the IBC or UBC and this criteria. Calculations shall be sealed by a registered design professional.

2.2 Testing Laboratories: Testing laboratories shall comply with Section 2.0 of the ICC-ES Acceptance Criteria for Test Reports (AC85) and Section 4.2 of the ICC-ES Rules of Procedure for Evaluation Reports.

2.3 Test Reports: Reports of tests required under Section 3.0 of this criteria shall comply with AC85 and reporting requirements in referenced standards.

2.4 Product Sampling: Sampling of devices for tests under this criteria shall comply with Section 3.1 of AC85.

3.0 DESIGN, TEST, AND PERFORMANCE REQUIREMENTS

3.1 General: The helical foundation systems and devices shall be evaluated for resistance to axial compression, axial tension, or lateral loads, or a combination of these loads. The required capacities shall be evaluated by considering four primary structural elements of the helical foundation system as shown in Figures 1 through 4. These elements are described as Bracket Capacity (P1), Shaft Capacity (P2), Helix Capacity (P3), and Soil Capacity (P4). The allowable capacity of a helical foundation system or device shall be the lowest value of P1, P2, P3, and P4, from each application illustrated in Figures 1 through 4. For evaluation of helical foundation devices subject to combined lateral loads and axial compression or axial tension, the allowable lateral capacity and allowable axial capacity shall be determined and reported separately. The allowable strength under combined load conditions shall be determined using the interaction equation provided in the AISC referenced standard.

3.2 P1 Bracket Capacity: The P1 bracket capacity is the maximum load that can be sustained by the bracket device of a helical foundation system based on strength in accordance with Section 3.10.

3.3 P2 Shaft Capacity: The P2 shaft capacity is the specified load that can be sustained by the shaft or coupling elements of a helical foundation device based on strength in accordance with Section 3.11.

3.4 P3 Helix Capacity: The P3 helix capacity is the specified load that can be sustained by the helix element of a helical foundation device based on strength or deformation in accordance with Section 3.12.

3.5 P4 Soil Capacity: The P4 soil capacity is the specified load that can be sustained by the soil or bedrock bearing stratum supporting the foundation system or device based on strength and settlement or pullout in accordance with Section 3.13.

3.6 Determination of Allowable Design Capacities: In accordance with Section 3.7 and Section 3.8, the allowable design capacities of helical foundation elements P1 and P2 shall be evaluated based on Conventional Design with no testing required, Special Analysis with verification tests, or solely on tests. All load tests shall be conducted in accordance with Section 4.0. The allowable capacity P3 shall be determined through load testing only as prescribed in Section 3.12. The allowable capacity P4 shall be determined by registered design professional or through installation torque correlations as specified in Section 3.13.

3.7 Design Methods:

3.7.1 Conventional Design: For conventional design of steel, either Allowable Stress Design (ASD) or Load and Resistance Factor Design (LRFD) methods referenced in the IBC or UBC may be used to calculate the allowable design capacity, P'. For design of concrete, strength design methods referenced in ACI 318 (IBC) or the UBC shall be used to calculate the design capacity.

3.7.1.1 ASD Method: When using the ASD method, the allowable design capacity, P', shall be taken as the allowable strength, P_a, and shall be determined in accordance with the applicable code or referenced standard (Eq-3).

$$P' = P_a \quad \text{(ASD)} \qquad \text{(Eq-3)}$$

3.7.1.2 LRFD Method: When using the LRFD method, the allowable design capacity, P', shall be taken as 0.7 times the design strength, ϕP_n, ϕP_n determined in accordance with the applicable code or referenced standard (Eq-4).

$$P' = 0.7\phi P_n \quad \text{(LRFD)} \qquad \text{(Eq-4)}$$

3.7.2 Special Analysis: Where special analysis is used, the allowable capacity P' shall be taken as 0.6 times the resistance based on yield strength (P_y) or, when stress concentrations are prevalent, P' shall be 0.5 times the resistance based on maximum strength (P_{max}) (Eq-5).

$$P' = 0.6P_y \text{ or } 0.5P_{max} \text{ (Special Analysis)} \qquad \text{(Eq-5)}$$

3.7.3 Direct Measurement: Where load testing only is used and the number of samples is not specified, the allowable capacity shall be reported as the average allowable strength determined in accordance with Section 4.0 from tests conducted on at least five specimens, provided all test results are within 15 percent (±15%) of the average. Otherwise, the allowable capacity from testing only shall be based on the least test result. For direct measurement of helical foundation device capacities, testing shall be conducted in accordance with the applicable test procedure described in Section 4.0. The allowable capacity, P,' shall be taken as 0.6 times the resistance based on yield strength (P_y) or 0.5 times the maximum strength (P_{max}), whichever yields the lowest value (Eq-6).

$$P' = 0.6P_y \text{ or } 0.5P_{max} \text{ (Direct Measurement)} \qquad \text{(Eq-6)}$$

For direct measurement of soil capacity, testing shall be conducted in accordance with Section 4.4.1.2. For determination of allowable soil capacity, a factor of safety equal to 2 or greater shall be applied to the maximum measured soil capacity.

3.8 Capacity Limits: For conventional design, the maximum allowable design capacity of helical foundation systems and devices is 60 kips (266.9 kN) in axial tension and axial compression and 6 kips (26.7 kN) in lateral resistance. Helical foundation systems or devices with allowable design capacities greater than these normal capacity limits require special analysis with additional verification testing as prescribed in Sections 3.10 to 3.13.

3.9 Corrosion: Helical foundation systems and devices shall be bare steel, powder-coated steel or zinc-coated steel. Powder coatings shall comply with the ICC-ES Acceptance Criteria for Corrosion Protection of Steel Foundation Systems Using Polymer (EAA) Coatings (AC228) and the coating thickness shall be at least 450 μm (0.018 inch). Zinc coatings shall comply with ASTM A 123, A 153, B 633, or B 695, as applicable. Loss in steel thickness due to corrosion shall be accounted for in determining structural capacities by reducing the thickness of all helical foundation components by the sacrificial thickness over a period, t, of 50 years. The design thickness, T_d, of helical foundation components used in capacity calculations and testing shall be computed by Eq.-6. For purposes of design calculations and fabrication of test specimens, the thickness of each component shall be reduced by $^1/_2\ T_s$ on each side, for a net reduction in thickness of T_s.

$$T_d = T_n - T_s \qquad \text{(Eq-6)}$$

where T_n is nominal thickness and T_s is sacrificial thickness (t = 50 yrs).

$$T_d \le \text{base steel thickness}$$

Zinc-coated steel: $T_s = 25\ t^{0.65}$ = 318 μm (0.013 in)

Bare steel, $T_s = 40\ t^{0.80}$ = 915 μm (0.036 in)

Powder coated steel:

$$T_s = 40(t\text{-}16)^{0.80} = 671\ \mu\text{m (0.026 in)}$$

For bare steel and powder-coated steel, T_n shall be the base-steel thickness. For zinc-coated steel, T_n may be the sum of the base-steel thickness and zinc coating thickness, provided the minimum zinc coating thickness is 86 μm (0.0034 in). Otherwise, the sacrificial thickness, T_s, shall be determined by linear interpolation between bare steel and zinc coated steel using the actual specified zinc coating thickness.

For powder-coated steel, the life of powder coating is taken as 16 years maximum. Hence, t has been reduced by 16 in the determination of T_s.

For verification of Special Analysis or for determination of allowable capacity through testing only, test specimens shall be constructed using steel thickness equal to T_d. Alternatively, unaltered test specimens may be used and the resulting allowable strength shall be reduced by multiplying the result by a scaling factor that takes into account corrosion and the observed failure mode. Thus, a tension failure result shall be scaled by the area of the fracture surface, while a flexural failure would be scaled by the reduced section modulus. The testing laboratory shall determine the appropriate scaling method and identify the failure mode.

Corrosion loss shall be accounted for regardless of whether devices are below or above ground or embedded in concrete. Zinc-coated steel and bare steel components shall not be combined in the same system. Powder coated steel may be combined with zinc-coated steel and bare steel components. All helical foundation components shall be galvanically isolated from concrete reinforcing steel, building structural steel, or any other metal building components.

3.10 P1 Bracket Capacity: Helical foundation brackets shall be classified as one of four types: side vertical load, direct load, slab support compressive load and tension anchor load. These types of brackets are illustrated in Figures 1 through 4. Bracket capacity shall be evaluated separately for each type. At a minimum, evaluation of P1 shall include determination of strength of the connection of the bracket to the structure, the internal strength of the bracket itself, and the strength of connection of the bracket to the helical foundation shaft. The frictional resistance of concrete on a horizontal bracket component shall be determined using a coefficient of friction of 0.4 or less. The shear strength of concrete also shall be calculated in accordance with the applicable code. Brackets may be evaluated for compression, tension, and/or lateral strengths, depending on the type. The angle of the shaft with respect to the bracket recommended by the installation instructions shall be accounted for in the calculations. The evaluation shall include an allowance for a tolerance of 1 degree from the permissible angle of inclination. Effects of helical foundation shaft inclination relative to vertical shall be accounted for in the analysis for axial compression or axial tension loads by incorporating a lateral component of forces in the analysis of the bracket, helical foundation shaft, and bracket connections. The shaft and the bracket shall be attached by a mechanical connection. Installation shall be limited to support of uncracked concrete, as determined in accordance with the applicable code. In order for the shaft to be considered side sway braced, the structure shall provide lateral restraint to the shaft equal to or greater than 0.4 percent of the shaft's allowable axial compression load.

3.10.1 Type A Side Load: Type A brackets are illustrated in Figure 1 and support tensile or compressive loads that are not concentric with the primary axis of the helical foundation shaft. Use of Type A brackets for supporting lateral loads is outside the scope of this criteria. Rotational moments caused by load eccentricity shall be subdivided into two components, bracket eccentricity and structure eccentricity, as illustrated in Figure 5. The shaft and the connected bracket components, consisting of the connected bracket, connection of the bracket to the shaft, and connection of the bracket to the structure, shall resist bracket eccentricity. Structure eccentricity varies with application and is generally resisted by the internal strength of the structure to which the bracket is attached. Therefore, resistance to structure eccentricity shall be determined on a case-by-case basis. For purposes of bracket eccentricity and internal strength design, the location of the resultant vertical compression force of the concrete structure on an angle bracket shall be taken as the centroid of an area defined by the uniform concrete bearing stress, taken as $0.35f'_c$ for ASD and $0.55f'_c$ for LRFD as shown in Figure 5. Type A brackets shall only be used to support structures that are braced as defined in IBC Section 1808.2.5. The strength of connected bracket components, shafts shall be evaluated based on one of two methods of proportioning moment between helical foundation shaft and connected bracket components. The first method is based on allowable stress design and is described in Section 3.10.1.1. The second method is based on limit state analysis and is described in Section 3.10.1.2.

3.10.1.1 Allowable Stress Design: This method of evaluation assumes the resistance to overturning moment is proportioned between the helical foundation shaft and the connected bracket components based on relative stiffness. The overturning moment caused by bracket eccentricity shall be proportioned between helical foundation shaft and connected bracket components using Eq-7a.

$$G = E_pI_p/E_bI_b \qquad \text{(Eq-7a)}$$

where:

I_p = Moment of inertia of helical foundation shaft (in^4 or mm^4).

E_p = Modulus of elasticity of helical foundation shaft (psi or MPa).

I_b = Moment of inertia of connected bracket components (in^4 or mm^4).

E_b = Modulus of elasticity of connected bracket components (psi or MPa).

If G >10 Method a applies.

If G < 0.1 Method b applies.

If $0.1 \leq G \leq 10$ Method c applies.

The stiffness of the helical foundation shaft can be increased by reinforcing the top section of shaft with an outer sleeve, T-pipe, or other means. Based on the resulting value of G, the corresponding method in Sections 3.10.1.1.1 to 3.10.1.1.3 shall apply.

3.10.1.1.1 Method a: Rigid Shaft: This method of evaluation assumes the shaft and its connection to the bracket are relatively rigid compared to the connection of the bracket to the structure. By this method, the shaft shall resist the moment due to bracket eccentricity. A free body diagram of the bracket based on this method is illustrated in Figure 5(a). The free body diagram is statically determinate. Separate evaluation of helical foundation bracket devices by this method shall include evaluation of P2 for all specified helical foundation shafts to be used with the bracket. In the analysis of the shaft, a moment shall be applied to the top of the shaft equal to the eccentricity of the bracket times the axial load.

3.10.1.1.2 Method b: Flexible Shaft: This method of evaluation assumes the helical foundation shaft and/or its connection to the bracket are relatively flexible compared to the connection of the bracket to structure. By this method, the connection of the bracket to the structure is required to resist the moment due to bracket eccentricity. Axial loads are transmitted concentrically to the helical foundation shaft. A free body diagram of the bracket based on this method is illustrated in Figure 5(b). The free body diagram is statically determinate.

3.10.1.1.3 Method c: Combined Stiffness: This method of evaluation assumes the shaft and the connection of the bracket to the structure are of similar stiffness. In this case, both the shaft and structure contribute to resisting the moment due to bracket eccentricity. A free body diagram of the bracket based on this method is illustrated in Figure 5(c). The free body diagram is statically indeterminate. Numerical analysis, finite element modeling, strain compatibility, or other Special Analysis shall be used to determine allowable capacity. Alternatively, the moment exerted on the shaft and the connection of the bracket to the structure can be proportioned using G, and the capacity of the bracket can be statically determined using Conventional Design described in Section 3.7. Evaluation of P1 bracket capacity by this method shall include a specified shaft and is necessarily coupled with evaluation of P2 shaft capacity. In the analysis of the shaft, a moment shall be applied to the top of the shaft equal to the eccentricity of the bracket times the appropriate proportion ($G/(G+1)$) of axial load.

3.10.1.2 Limit State Design: This method of evaluation assumes at failure that the connection between the bracket and structure reaches a maximum limit state and the helical foundation shaft has a plastic hinge. Based on these assumptions, the rotational stability of a side load bracket is statically determinate. The nominal load capacity of the bracket shall be determined by simultaneous solution of static equilibrium equations. In the static analysis, the moment at the connection of the helical foundation shaft to the bracket or T-pipe shall be set equal to the moment resistance of the shaft based on combined axial and flexural loading. The shear at the connection of the helical foundation shaft to the bracket or T-pipe shall be determined by Eq-7b.

$$V_p = M_p/d \qquad \text{(Eq-7b)}$$

where

M_p = Moment resistance of helical foundation shaft from combined axial and flexural load analysis (in–lbf or N-mm).

V_p = Shear in helical foundation shaft at the connection to the bracket or T-pipe (lbf or N).

d = 60 inches (1524 mm).

3.10.1.3 Connection to the Structure: Axial compression, axial tension, or lateral load connection capacities shall be determined in accordance with the IBC, UBC, or a current ICC-ES evaluation report. For purposes of evaluation, the structure shall be modeled as a mass of structural plain concrete, semi-infinite in extent, with varying strength. The structure shall be assumed to be fixed in translation and rotation, but can move freely in the vertical direction. At a minimum, design of the connection shall be based on normal-weight concrete with a specified compressive strength of 2,500 psi (17.22 MPa). Other concrete strengths, structural lightweight concrete, masonry and other materials also can be included in the evaluation at the option of the bracket manufacturer. For all combinations of concrete strength and/or material compositions, details regarding connection of the bracket to the structure types (i.e., anchor bolt placement, grouting, surface preparation, etc.) shall be prescriptively specified.

3.10.2 Type B: Direct Load: Type B brackets illustrated in Figure 2, support axial compressive or axial tension loads that are concentric with the primary axis of the helical foundation shaft and may be used to support lateral loads. The strength of bracket components and connections shall be evaluated in accordance with Section 3.10.2.1 or Section 3.10.2.2 depending on whether the structure to be supported by the bracket is side sway braced.

3.10.2.1 Method 1: Sidesway Braced: This method of evaluation assumes the connection of the bracket to the structure provides lateral but not rotational bracing for the top of the helical foundation shaft so that the top of the shaft is essentially a pinned connection.

3.10.2.2 Method 2: Sidesway Unbraced: This method of evaluation assumes the structure provides neither lateral nor rotational bracing for the top of the helical foundation shaft, so that the top of the shaft is essentially a free connection.

3.10.2.3 Connection to the Structure: The structures that Type B brackets are used to support may be concrete, steel, wood or other material. Evaluation shall include specifications for connection to structures, such as material strength, embedment depth, edge distance, welds, bolts, bearing area, and bracing. Connection of the bracket to each type of structure (grade beams, walls, steel beams, posts, etc.) for which evaluation is being sought shall be detailed and analyzed separately. At a minimum, design of the connection shall be based on normal-weight concrete with a specified compressive strength of 2,500 psi (17.22 MPa). The analysis shall include considerations of internal shear and moment within concrete elements, as applicable. Analysis of wood, steel, and concrete shall be based on the IBC, UBC, AISC LRFD, AISC ASD, AF&PA, NDS, or ACI 318, as applicable.

3.10.3 Type C: Slab Support: Type C brackets support concrete flatwork. These brackets shall support axial compression loads concentrically. Use of Type C brackets for supporting tension or lateral loads is outside the scope of this criteria. Calculations shall be performed proving whether the bracket can be considered sidesway braced. Evaluation shall comply with Section 3.10.2.1 of the criteria for Type B direct load brackets, Method 1, and shall include analysis of punching shear based on ACI 318 in concrete slabs of different strength and different thickness slabs, along with recommended bracket spacing for slabs supporting 40 psf (1915 Pa) to 100 psf (4788 Pa) uniform live loads. At a minimum, evaluation shall include 4-, 6-, and 8-inch-thick (102, 152, and 203 mm), unreinforced slabs containing normal-weight concrete with minimum specified compressive strength of 2,500 psi (17.22 MPa). Other concrete strengths and structural lightweight concrete also can be included in the evaluation at the option of the bracket manufacturer.

3.10.4 Type D: Tension Anchor: Type D brackets are used to support axial tension loads only. These brackets shall support loads concentrically and shall not be evaluated for lateral load resistance. Evaluation shall comply with Section 3.10.2 of the criteria for Type B direct load brackets. The connection to the existing structure shall be evaluated, including the range of acceptable shaft installation angles proposed by the manufacturer.

3.10.5 Test Requirements: Verification tests shall not be required for evaluation of foundation brackets provided all analysis is accomplished using Conventional Design as set forth in Section 3.7 and allowable capacities are within the range of Normal Capacity Limits as set forth in Section 3.8. A minimum of three verification load tests shall be conducted in each load direction (axial compression, axial tension, and lateral) on any component of a bracket or bracket/shaft system evaluated using Special Analysis and for brackets exceeding Normal Capacity Limits. Where tests are required for verification of lateral resistance, tests shall be conducted to verify lateral resistance in all directions for which lateral resistance is being claimed. Bracket tests shall be conducted in accordance with Section 4.1 for compression and tension and Section 4.4.2 for lateral resistance.

3.11 P2 Shaft Capacity: At a minimum, helical foundation shaft capacities shall be evaluated for torsion and either axial compression, axial tension, or both. Shafts may also be evaluated for lateral resistance with consideration of combined lateral and axial loading. Evaluation of shafts shall include connections between shafts. All shaft connections shall be made via a mechanical coupling.

3.11.1 Tension: Shaft evaluation for tension shall include yielding on the gross area and fracture at any couplings. At couplings, there shall be consideration of fracture on the net area of the main member, fracture on the net area of the sleeve, bearing of fasteners such as pins or bolts on the net areas of fastener holes, shearing of the fasteners, block shearing of the main member and sleeve, and the attachment of the sleeve to the main member.

3.11.2 Compression: Shaft evaluation for compression shall include buckling resistance, yielding on the gross area, and yielding at any couplings. At couplings, there shall be consideration of bearing of the fasteners such as pins or bolts on the net area of the fastener holes, shearing of the fasteners, and the attachment of the sleeve to the main member. A bending moment shall be applied to the top of the shaft in buckling calculations in accordance with Section 3.10 and Section 3.11.2.3.

3.11.2.1 Unsupported Length: Unsupported shaft lengths shall include the length of the shaft in air, water, or in fluid soils. For unbraced systems, the lengths specified in IBC Section 1808.2.9.2 shall apply unless determined otherwise by Special Analysis. In accordance with IBC Section 1808.2.9.1, any soil other than fluid soil shall be deemed to afford sufficient lateral support to prevent buckling of systems that are braced. Bracing shall comply with IBC Section 1808.2.5. Firm soils shall be defined as any soil with a Standard Penetration Test blow count of five or greater. Soft soils shall be defined as any soil with a Standard Penetration Test blow count greater than zero and less than five. Fluid soils shall be defined as any soil with a Standard Penetration Test blow count of zero [weight of hammer (WOH) or weight of rods (WOR)]. Standard Penetration Test blow count shall be determined in accordance with ASTM D 1586.

3.11.2.2 Effective Length: Effective lengths shall be determined using the unsupported length defined in Section 3.11.2.1 and the appropriate effective length factor, K, determined in accordance with the AISC referenced standard. Slenderness ratio limitations as specified by the AISC referenced standards do not apply.

3.11.2.3 Coupling Rigidity: Coupling rigidity shall be considered for all cases except braced systems in firm or soft soils. To account for coupling rigidity, the eccentricity of the axial compressive load applied to the shaft shall be increased by a distance, $n \cdot e_c$, where n is the number of couplings possible in the unsupported length and e_c is the maximum lateral deflection of the unsupported length of shaft due to flexure of the coupling under an applied lateral load of 0.4 percent of the applied axial compressive load. Maximum lateral deflection of the shaft due to coupling flexure shall be determined in accordance with Section 4.2.4.

3.11.3 Torsion: Torsion resistance shall be determined by testing in accordance with Section 4.2.2. A minimum of 12 samples, with an equal number of samples from four or more separate heats, shall be used for the basis of testing. The mean ultimate (maximum) torsion resistance and standard deviation shall be determined from the test population. Based on test results, maximum installation torque shall be reported as two standard deviations below the mean ultimate (maximum) torque from the sample population. Torsional strength need not be evaluated for corrosion losses.

3.11.4 Lateral Resistance: Lateral resistance of the shaft is necessarily coupled with soil capacity and shall be determined in accordance with Section 3.13. Shaft area, moment of inertia, and elasticity shall be used as inputs in the analysis. Maximum bending moment and shear stress determined from the analysis shall be limited by the allowable bending and shear resistance of the shaft or the shaft couplings, whichever is less. Deflection of shaft couplings shall be included in lateral resistance analysis.

3.11.5 Elastic Shortening or Lengthening: Methods (equations) shall be provided for estimation of elastic shortening/lengthening of the shaft under the allowable axial load plus any slip in the couplings. These methods shall be based upon Conventional Design.

3.11.6 Combined Stresses: Shaft evaluation shall include combined stresses. Combinations of tension, compression, bending, and lateral loads shall be considered as applicable.

3.11.7 Test Requirements: Verification tests shall not be required for evaluation of shaft tension, compression, and bending moment provided all analysis is accomplished using Conventional Design in accordance with Section 3.1 and allowable capacities are within the range of Normal Capacity Limits as set forth in Section 3.8. A minimum of three verification load tests shall be conducted on separate specimens in each direction (compression, tension, bending) on any component of a shaft evaluated using Special Analysis and for shafts that exceed Normal Capacity limits as set forth in Section 3.8. Tests are required to determine torsion resistance of all shafts and coupling rigidity as described in Sections 3.11.2.3 or 3.11.3. Tests for shaft capacity shall be conducted in accordance with Section 4.2.

3.12 P3 Helix Capacity: Helix capacities shall be evaluated for torsional resistance, punching flexure, weld flexure, and weld shear in tension and compression. Evaluation shall be based solely on testing. The allowable helix capacity, P3, for helical foundation systems and devices with multiple helices shall be taken as the sum of the least design allowable capacity of each individual helix. The allowable capacity of the helix in torsion shall be considered acceptable provided it exceeds the torsional strength of the shaft.

3.12.1 Lateral Capacity: The determination of the lateral capacity of the helix is not permitted. The lateral capacity of a helical foundation system is based on the resistance of the shaft only and is not significantly affected by the presence of helix bearing plates.

3.12.2 Torsion: Torsion resistance of helix bearing plates can be determined in conjunction with shaft torsion or independently. In either case, testing shall be conducted in accordance with Section 4.2.2 using the number of samples and the same procedures described in Section 3.11.3.

3.12.3 Test Requirements: Each diameter, thickness, steel grade, pitch, and edge geometry helix, for which evaluation is being sought, shall be tested. The allowable capacity for each size and type of helix shall be reported as the average result of at least three test specimens. In order to allow the mean values, individual results determined from testing shall be within 15 percent of the average of tests. Otherwise, the least test result shall apply. At least one laboratory test shall be conducted to verify the torsional shear strength of each helix for installation purposes. Helix punching, weld flexure, and weld shear tests shall be conducted in accordance with Section 4.3. Helix torsion resistance shall be tested in accordance with Section 4.2.2.

3.13 P4 Soil Capacity: Soil capacity includes the tension, compression, and/or lateral resistance of a helical foundation embedded in ground, as applicable.

3.13.1 Axial Capacity Verification: For all helical foundation systems, full-scale field installation and load tests shall be conducted to verify the axial capacity on specimens installed to the maximum installation torque determined in accordance with Section 3.11.3. The tests shall be regarded as a successful verification of installation and allowable capacity, provided the maximum allowable torque is achieved during installation without significant damage to the helical foundation shaft and all full-scale axial load tests exceed the allowable capacity of the system by a factor of safety of at least 2.0.

At least two specimens of each type of helical foundation shaft shall be tested in each load direction (tension or compression) for which evaluation is being sought. Variations in shaft size and material strengths, as well as helix pitch, helix thickness, and edge geometry, shall constitute a different type of specimen. Two separate specimens shall be tested in each direction (compression and/or tension) for which evaluation is being sought. Test specimens shall consist of a shaft, at least one shaft coupling, and a single helix. The helix size shall include the smallest available helix diameter for one test and the largest available helix diameter for the other test. The test specimen may include a bracket. All verification tests shall be conducted at sites described in Section 3.13.4. Additional information on testing is provided in Section 3.13.5. The determination of soil capacity, P4, on any specific site or with any configuration of helical bearing plates other than the test site and test specimen is outside the scope of this acceptance criteria. The evaluation report shall indicate that soil capacity shall be determined by a registered design professional for each site considering groundwater and other geotechnical conditions. As an alternative, torque correlations for specific soil conditions may be determined in accordance with Section 3.13.2.

3.13.2 Torque Correlations: Evaluation reports may include a correlation between final installation torque, T, and ultimate (maximum) axial capacity, Q, given by Eq-8:

$$Q = K_t\, T \qquad \text{(Eq-8)}$$

where K_t is the axial tensile or compressive load capacity to torque ratio for a given helical foundation type. The allowable capacity, Q_a, shall be computed by Eq-9:

$$Q_a = 0.5Q \qquad \text{(Eq-9)}$$

If included in the evaluation report, the parameter K_t shall be verified by full-scale field installation and load tests. The number of tests required depends on whether the helical foundation system is conforming or nonconforming. Separate torque correlations are required for shafts with differing geometry and outside dimensions and for each helix plate style (pitch, thickness, geometry). Field tests may be conducted at any site provided a geotechnical engineering report is obtained for the site in accordance with Section 3.13.4 and the soil profile generally matches that shown in Table 2.

3.13.2.1 Conforming Systems: Systems shall be considered conforming based on compliance with the criteria given in Table 3. The following capacity to torque ratios (K_t) shall be reported for conforming products.

1.5-inch- and 1.75-inch-square shafts	$K_t = 10\ \text{ft}^{-1}$
2.875-inch outside diameter round shafts	$K_t = 9\ \text{ft}^{-1}$
3.0-inch outside diameter round shafts	$K_t = 8\ \text{ft}^{-1}$
3.5-inch outside diameter round shafts	$K_t = 7\ \text{ft}^{-1}$

The number of tests required to verify capacity to torque ratios for conforming products shall be as shown in Table 2. The correlation between torque and capacity shall be deemed verified if all of the ultimate (maximum) soil capacities determined from load tests conducted in accordance with Section 3.13.2 exceed the allowable capacity determined using the forgoing K_t values and provided the average ratio of ultimate (maximum) soil capacity determined in field tests to predicted allowable capacity determined using K_t is equal to or greater than two (2.0). If verification is not obtained, these helical foundation systems and devices shall be deemed as non-conforming and shall be subject to the additional testing as set forth in Section 3.13.2.2.

3.13.2.2 Nonconforming Systems: Systems that fail to comply with the criteria in Table 3 or that fail verification tests given in Section 3.13.2.1 shall be deemed nonconforming. Conforming systems also may be deemed non-conforming if values of K_t higher than provided in Section 3.13.2.1 are desired. In order to establish K_t values for these systems, at least eight additional field tests shall be conducted in compression and six additional tests shall be conducted in tension in addition to the quantity shown in Table 2. These tests shall involve a range of at least three different helix combinations and at least three different soil types. The subsurface profile at each test site shall be determined in accordance with Section 3.13.4.

Test sample population shall be plotted versus the ratio Q_f/Q, where Q_f is ultimate (maximum) soil capacity determined through full-scale field tests and Q is ultimate (maximum) soil capacity determined by correlations with torque using a constant K_t. An iterative approach shall be used to determine the value of K_t such that the mean value of Q_f/Q is equal to 1.0. The K_t value shall be considered valid if 94 percent of the data have a Q_f/Q ratio greater than 0.5. Otherwise, a correlation between capacity and torque is invalid for that product and cannot be reported.

3.13.3 Lateral Resistance: Allowable soil capacity in the lateral direction shall be determined through load tests on specimens installed in different soil conditions. The allowable soil capacity shall be determined based on deflection criteria set forth in Section 4.4.2. In order to be valid, allowable capacities determined for each type of specimen in each soil type shall be within 15 percent of the average allowable capacity for those tests.

A minimum of four specimens of each type of helical foundation shaft shall be tested in each soil type for which evaluation is being sought. Variations in shaft size, shaft geometry, and material strength shall constitute a different type of specimen. Variations in helix size, geometry, pitch, material strength, thickness, and number do not require separate tests. Four separate specimens shall be tested in each transverse direction for which evaluation is being sought if the shaft is not axially symmetric. Test specimens shall consist of a shaft, at least one shaft coupling located within the manufacturer’s smallest extension length from the ground surface, and one or more helix bearing plates. The test may include a bracket.

At a minimum, evaluation shall include tests in firm clay soils. Additional tests may be conducted in different soil conditions from other sites. The subsurface profile at all test sites shall be characterized in a soil investigation by a registered design professional. Additional information on testing is provided in Section 3.13.4. Allowable soil capacity for different specimens in different soil categories shall be tabulated in the evaluation report. The evaluation report shall contain a statement that soil capacity for lateral resistance in soils conditions that substantially differ from actual test sites included in the evaluation shall be determined by a registered professional engineer on a case-by-case basis.

3.13.4 Test Requirements: Axial compressive, tensile, and lateral allowable load capacity shall be verified through field load tests as provided in Section 3.13.3. At least two verification tests are required for axial compression and at least two verification tests are required for axial tension. If a ratio between final installation torque and capacity is specified, then at least eight tests are required for axial compression verification and at least six tests are required for axial tension verification for each shaft size for which evaluation is being sought. The two verification tests required for compression and tension may be included in the tests for torque correlations. No additional tests are required for establishing torque correlations for conforming products, whereas nonconforming products will require eight additional tests in compression and six additional tests in tension for each shaft size. If evaluation of lateral resistance is requested, four verification tests are required for each shaft size, shaft geometry, and soil type.

Tests for axial compression and tension soil capacity shall be conducted in accordance with Section 4.4.1 and tests for lateral resistance shall be conducted in accordance with Section 4.4.2. Tension and compression verification load tests are required to be conducted at the facility or field station of a testing laboratory complying with Section 2.2. The subsurface profile at other test sites shall be characterized in a soil report by a registered design professional. Subsurface profile characterization shall include soil borings, standard penetration resistance tests, and basic laboratory classification tests essential for soil classification according to the Unified Soil Classification System. All field penetration tests, laboratory tests, and soil classifications shall be conducted in accordance with ASTM D 1586.

4.0 TEST METHODS

4.1 P1 Bracket Capacity: Where specified herein, each size and configuration of the bracket shall be tested. The configuration of the bracket and direction of applied loads in the test apparatus shall be as close to actual field conditions as practical. Pertinent data such as maximum load applied, maximum bracket rotation, failure mode, etc. shall be reported.

4.1.1 Type A Side Load:

4.1.1.1 Setup: Compression and tension tests can be conducted in a horizontal configuration, as illustrated in Figure 6. The bracket shall be mounted to a block of plain concrete of known strength that is fixed with respect to translation and rotation. The connection of the bracket to the concrete shall be in accordance with manufacturer's installation instructions. Load shall be applied to the bracket using a 60 inch (1524 mm) long section of helical foundation shaft secured to the bracket in a manner that duplicates actual field conditions. The loaded end of the shaft shall be rotationally fixed. Axial load shall be applied in the direction of the longitudinal axis of the helical foundation shaft. Any eccentricity inherent in the bracket configuration and manufacturer-recommended angle of the shaft to bracket shall be accounted for and shall be modeled to match the anticipated design purpose.

4.1.1.2 Procedure: Axial deflection shall be recorded as a function of applied load at regular intervals equal to or less than 20 percent of the anticipated allowable load. The rate of load application shall be sufficiently slow to simulate static conditions. Each load increment shall be held for a minimum of 1 minute. Yield strength and ultimate (maximum) strength of the bracket shall be determined using conventional analysis of a plot of load versus deflection. The allowable strength of the bracket shall be determined from yield or ultimate (maximum) strength using the equations provided in Section 3.7.3, whichever formula results in the lowest value. Compression tests shall be conducted within 24 hours of the bracket test on concrete cylinders cast at the same time as the test specimen to establish concrete compressive strength. Cylinders shall be stored and cured according to Section 9.3.1 of ASTM C 31 (field cure). The tested concrete compressive strength shall be within 15 percent of the specified compressive strength. Concrete cylinder compression tests shall be conducted in accordance with ASTM C 39.

4.1.2 Type B: Direct Load:

4.1.2.1 Setup: The test bracket shall be mounted to a fixture that is substantially similar to the structure for which the bracket is intended to support. The fixture representing the structure shall be translationally and rotationally fixed as appropriate to simulate field conditions, as illustrated in Figure 7. The connection of the bracket to the fixture shall be in accordance with manufacturer's installation instructions. The load shall be applied to the bracket using a 60-inch-long (1524 mm) section of helical foundation shaft secured to the bracket in a manner that duplicates actual field conditions. The loaded end of the shaft shall be rotationally fixed. Axial load shall be applied in the direction of the longitudinal axis of the helical foundation shaft. Any inclination of the shaft with respect to the structure shall be modeled to match the anticipated design purpose. For tests of the lateral capacity of a bracket and the connection of the bracket to a structure, the load test shall be set-up as described herein, except that the load shall be applied normal to the shaft at a location as close to the base of the cap as possible. In order to avoid application of flexure to the shaft during loading, a roller guide shall be used to facilitate load application as shown in Figure 7.

4.1.2.2 Procedure: Depending on the purpose of the test, axial or lateral deflection shall be recorded as a function of applied load at regular intervals equal to or less than 20 percent of the anticipated allowable load. The rate of load application shall be sufficiently slow to simulate static conditions. Each load increment shall be held for a minimum of 1 minute. Yield strength and ultimate (maximum) strengths of the bracket shall be determined using conventional analysis of a plot of load versus deflection. The allowable strength of the bracket shall be determined from yield or ultimate (maximum) strength and the equations provided in Section 3.7.3, whichever formula results in a lower value. If a concrete structure is used in the load test, the strength of the concrete shall be tested in accordance with the procedures in Section 4.1.1.2.

4.1.3 Type C: Slab Support:

4.1.3.1 Setup: Compression tests shall be conducted by casting a concrete slab with specified thickness and dimensions equal to the manufacturer's recommended helical foundation shaft spacing for that thickness slab and anticipated loading. The slab support bracket and a section of helical foundation shaft shall be mounted in an inverted fashion over the slab, as illustrated in Figure 8. A hole consistent with manufacturer's recommendations shall be cored through the slab in the bracket location and subsequently filled with cementitious grout. The slab shall be supported on a flexible air diaphragm sufficient to withstand the imposed loads. The length of the helical shaft used in the test shall be at least six times the diameter of the shaft. As an alternative, the slab, bracket, shaft, and air diaphragm may be mounted in a horizontal load frame.

4.1.3.2 Procedure: Downward compression loads shall be applied axially to the end of the shaft. Axial deflections shall be recorded as a function of applied load at regular intervals not exceeding 20 percent of the anticipated allowable load. The rate of load application shall be sufficiently slow to simulate static conditions. Each load increment shall be held for a minimum of 1 minute. Yield strength and ultimate (maximum) strengths of the bracket shall be determined using conventional analysis of a plot of load versus deflection and may depend heavily on slab shear. The allowable strengths of the bracket shall be determined from yield or ultimate (maximum) strength and the equations provided in Section 3.7.3, whichever formula results in the lowest value. The compressive strength of the concrete shall be verified in accordance with the procedures described in Section 4.1.1.2.

4.1.4 Type D: Tension Anchor:

4.1.4.1 Setup: Load tests shall be conducted on Type D anchor brackets by attaching the bracket to a short section of helical foundation shaft following the evaluation report applicant’s recommendations. The bracket shall be cast into a concrete test specimen or otherwise attached to a structure that substantially conforms to the manufacturer’s recommended connection details including minimum washer plate size, concrete cover, and concrete reinforcement as applicable. The specimen shall be placed in tension in a laboratory load frame, as illustrated in Figure 9. Deflection of the anchor bracket shall be measured with a dial gauge. The load shall be determined with a calibrated load cell. The length of the shaft used in the test shall be at least six times the shaft diameter.

4.1.4.2 Procedure: The specimen shall be loaded in increments not exceeding 20 percent of the calculated allowable capacity. The rate of load application shall be sufficiently slow to simulate static conditions. Each load increment shall be held for a minimum of 1 minute. Deflections and loads at the completion of the hold period for each increment shall be measured. The specimens shall be loaded until plastic yielding or brittle fracture occurs. The failure mode shall be reported. A plot of deflection versus load shall be reported. The allowable strength of the bracket shall be determined from yield or ultimate (maximum) strength and the equations provided in Section 3.7.3, whichever equation results in a lower value, along with the corresponding deflection as determined from the load-deflection plot. If applicable, the strength of the concrete shall be verified in accordance with the procedures described in Section 4.1.1.2.

4.2 P2 Shaft Capacity:

4.2.1 Axial Tension and Compression:

4.2.1.1 Setup: Tension and compression tests shall be conducted on a section of shaft with a coupling located approximately at the midpoint of the shaft specimen. The test specimen shall be mounted to a vertical or horizontal load frame with one end attached to a fixed platform and the other end attached to a mobile platform with the capability to apply the load to the specimen in the axial direction. The coupling connection shall be done in accordance with manufacturer’s specific published recommendations. Direction of loading shall be coaxial with the longitudinal axis of the shaft. The testing apparatus shall provide sufficient rigidity as to minimize any slip or deformation not associated with the test specimen. The shaft shall have sufficient length (each side of coupling) to allow a uniform tensile or compressive force to develop in the shaft prior to reaching the connection. To evaluate buckling resistance, compression specimens shall have a minimum length equal to or greater than the effective length as specified in Section 3.11.2.2.

4.2.1.2 Procedure: Loads shall be applied to the specimen in increments not exceeding 20 percent of the design allowable load of the specimen. Each load increment shall be held for a minimum of one minute. The specimen shall be loaded to failure. Application of the load shall be performed at a slow rate to simulate a statically applied load. Pertinent data such as maximum load applied, maximum shaft or connection deformation, failure mode, etc. shall be reported. Yield strength and ultimate (maximum) strength of the shaft and coupling shall be determined using conventional analysis of a plot of load versus deflection. The allowable strength of the shaft and coupling shall be determined from yield or ultimate (maximum) strength and the equations provided in Section 3.7.3, whichever equation results in a lower value.

4.2.2 Torsion:

4.2.2.1 Setup: Torsion testing shall be performed on a section of shaft with a minimum length of 36 inches (914 mm) or 12 times the maximum outside cross sectional dimension of the shaft; whichever is greater. The shaft shall have a standard manufactured coupling located approximately midway between the ends of the shaft specimen and a helix affixed to the end of the shaft. The specimen shall be fixed at the helix end and attached to a torque motor on the other end. The helix shall be fixed about the outside edge using six bolt clamps. The tests shall be conducted in a load frame that allows for measurement of the angle of twist, as illustrated in Figure 10. Torque shall be applied to a short section of shaft attached to the helix. The test setup shall include a means of measuring shaft coupling bolt hole elongation during the test. Alternatively, the helix may be tested separately at the evaluation report applicant’s option. In the shaft torsion test without a helix, the specimen shall be fixed at one end of the shaft and attached to the torque motor on the other end. In the helix torsion test, the specimen shall consist of a short section of shaft attached to a helix plate. The helix shall be fixed about the outside edge as previously described herein and torsion shall be applied to the end of the shaft.

4.2.2.2 Procedure: As applicable depending on the test specimen configuration, the maximum torsion resistance shall be defined as that required to achieve 0.5 shaft revolution per foot (1.6 revolutions per meter) of shaft length, that which causes failure of the helix, coupling, or shaft, that which damages the coupling to an extent that it cannot be decoupled effectively, or that which elongates the coupling bolt hole 0.25 inch (6.4 mm), whichever occurs first. The rotation rate shall not exceed 20 rpm.

4.2.3 Bending:

4.2.3.1 Setup: Bending tests shall be conducted on a section of shaft that is horizontally arranged in a compression load frame, as illustrated in Figure 11. For shafts with a non-circular cross section, as a minimum, the tests shall be conducted with the least resistant orientation. The distance between shaft supports shall be at least 36 inches (914 mm) or 12 times the maximum outside cross-sectional dimension of the shaft, whichever is greater. A coupling shall be located approximately in the center of the specimen. Loads shall be applied using a two point test where the load points straddle the coupling so that a uniform bending moment is produced in the coupling.

4.2.3.2 Procedures: Load shall be applied and deflections measured at intervals of less than or equal to 20 percent of the load corresponding to the theoretical allowable bending moment. Application of load shall be performed at a slow rate to simulate a statically applied load. Pertinent data such as maximum load applied, maximum shaft or coupling deformation, failure mode, etc. shall be reported. Yield strength and ultimate (maximum) strength of the shaft and coupling shall be determined using conventional analysis of a plot of load versus deflection. The allowable bending strength of the shaft and coupling shall be determined from yield or (maximum) strength and the equations provided in Section 3.7.3, whichever equation results in a lower value.

4.2.4 Coupling Rigidity:

4.2.4.1 Setup: The maximum lateral deflection of shafts due to coupling flexure shall be determined using a section of shaft with length equal to the Unsupported Length [60 or 120 inches (1524 or 3048 mm) as specified by Section 1808.2.9.2 of the IBC]. The shaft shall have the maximum number of couplings possible over its length based on the available shaft sections. The shaft shall be horizontally or vertically arranged in a load frame at the evaluation report applicant's option with one end fixed and the other end unsupported, as illustrated in Figure 12. A load shall be applied perpendicularly to the unsupported end of the shaft.

4.2.4.2 Procedures: A vertical load equal to 0.4 percent of the allowable compression load on the helical foundation shaft system shall be applied. The total deflection of the loaded end of the shaft, including any free deflection, shall be measured relative to a horizontal plane extending from the fixed end. The total deflection shall be reported and used in shaft eccentricity computations.

4.2.5 Shear Strength:

4.2.5.1 Setup: The maximum shear strength of shafts and couplings shall be determined using specimens with lengths as appropriate for the test apparatus. The specimen shall be horizontally or vertically arranged in a load frame with one end fixed and the other end free. A load shall be applied normal to the shaft or coupling using a roller or slide to avoid inducing flexure into the system.

4.2.5.2 Procedure: The loads shall be applied in increments not exceeding 20 percent of the allowable shear load on the shaft or coupling. The total deflection of the shaft or coupling at the point of load application shall be measured at each increment. Load shall be applied at a slow rate to simulate statically applied load. Each load increment shall be held for a minimum of one minute. Yield and ultimate (maximum) strength of the shaft or coupling shall be determined using a conventional analysis of a plot of load versus deflection.

4.3 P3 Helix Capacity:

4.3.1.1 Setup: Helix capacity tests shall be performed by placing a short section of shaft with a helix in a laboratory load frame, as illustrated in Figure 13. The helix plate shall bear on an adjustable mandrill with five or more pins or a helix-shaped fixture. The line of bearing shall be located at a distance from the central axis of the shaft equal to one-half the outer radius of the helix, R_b, plus the radius of the shaft, R_s. For non-circular shafts, R_s shall be the radius of a circle circumscribed about the outer extent of the shaft's cross-section. Direction of loading shall be coaxial with the longitudinal axis of the shaft and normal to the bearing plane of the helix.

4.3.1.2 Procedures: Load shall be applied and deflection recorded at intervals equal to 20 percent of the theoretical punching strength of the helix. Application of load shall be done at a slow enough rate as to simulate a statically applied load. Pertinent data such as maximum load applied, maximum helix deformation, failure mode, etc., shall be reported. Load shall be plotted as a function of deflection. Maximum strength of the helix shall be the peak load sustained by the helix. The allowable strength of the helix shall be determined from the maximum strength in accordance with Section 3.7.3.

4.4 P4 Soil Capacity:

4.4.1 Full-scale Load Tests:

4.4.1.1 Setup: Full-scale load tests shall be conducted in accordance with ASTM D 1143 for axial compression and ASTM D 3689 for axial tension. The quick load test procedure set forth in Section 5.6 of ASTM D 1143 shall be used in compression tests. Installation of the helical piers shall be done in accordance with the installation instructions. The brand, model number, and maximum torque capacity of the installation device shall be reported. All test piers shall be installed as close to vertical as possible. Pertinent data such as helical foundation shaft depth and final installation torque achieved shall be reported. Torque should be measured with a calibrated in-line indicator, or calibrated hydraulic torque motor via differential pressure. Calibration of torque motors and/or torque indicators shall be performed on equipment whose calibration is traceable back to NIST (National Institute of Standards and Technology). For tension tests, the helical foundation shaft shall be installed such that the minimum depth from the ground surface to the uppermost helix is 12*D*, where *D* is the diameter of the largest helix.

4.4.1.2 Procedures: Direction of loading shall be coaxial with the longitudinal axis of the pier. Application of load shall be done at a slow rate to simulate a statically applied load. Piers shall be installed to the depth interval recommended for the designated helical foundation shaft test sites. Maximum load capacity shall be that which is achieved when plunging of the helix plate occurs or when net deflection exceeds 10 percent of the helix plate diameter, whichever occurs first. Net deflection shall be total deflection minus shaft elastic shortening or lengthening. For multiple helix configurations, the average helix diameter shall be used in this criterion.

4.4.2 Lateral Load Tests:

4.4.2.1 Setup: Lateral load tests shall be conducted in accordance with ASTM D 3966. These tests can be performed in two ways. If verification of lateral resistance of brackets is required, the test setup shall consist of a helical foundation representative of a standard installation with a bracket above the ground surface. The bracket shall be connected to a structure constructed from wood, steel, or concrete depending on the particular detail for which evaluation is being sought. The test setup shall be such that lateral load is applied to the structure being supported immediately above the bracket elevation. The tests shall be conducted with a free head arrangement in accordance with ASTM D 3966. Where the bracket is intended to support a structure that is rotationally restrained, the test may be conducted using fixed head or free head arrangements in accordance with ASTM D 3966.

If verification of bracket capacity is not required, as in the case of Conventional Design, then the tests shall be conducted with the helical foundation shaft extending a minimum of 12 inches (304.8 mm) from the ground surface. The lateral load shall be applied to the helical foundation shaft immediately above the ground surface. Depending on whether the helical foundation shaft is intended to support a structure that is rotationally restrained, the test may be conducted using fixed head or free head arrangements in accordance with ASTM D 3966.

Bracket and helical foundation installation shall be done in accordance with the standards set forth in manufacturer's specific published recommendations. All test piers shall be installed within the manufacturer's specified tolerances for angle of installation for the bracket type. Where brackets are not used, the shaft shall be installed within the manufacturer's specified tolerances for plumbness. The minimum depth of the uppermost helix shall be 180 inches (4572 mm) unless the helical foundation system is only available in a shorter length.

4.4.2.2 Procedures: For tests including brackets or shafts that are non symmetrical, separate specimens shall be loaded in all lateral directions for which evaluation is being sought. Application of load shall be done at a slow rate to simulate a statically applied load. The allowable load capacity reported shall be equal to half the load required to cause $^{3}/_{4}$ inch (19.1 mm) of lateral deflection at the ground surface.

4.5 General Testing Requirements: Test equipment shall be adequate to impose anticipated maximum loads. If loading is not carried to failure, the highest value achieved will be considered the maximum load.

5.0 QUALITY CONTROL

5.1 Manufacturing: All products shall be manufactured under an approved quality control program with inspections by an inspection agency accredited by the International Accreditation Service (IAS) or otherwise acceptable to ICC-ES.

5.2 Quality Control Documentation: Quality documentation complying with the ICC-ES Acceptance Criteria for Quality Documentation (AC10) shall be submitted.

6.0 EVALUATION REPORT RECOGNITION

6.1 General: The evaluation report shall include a description of the helical foundation device or system, typical applications, and limitations. The evaluation report shall state that (1) the device or system shall be limited to support of structures in IBC Seismic Design Categories A, B, and C or UBC Seismic Zones 0, 1, and 2, only; (2) the device or system shall not be used in conditions that are indicative of a potential pile corrosion situation as defined by soil resistivity less than 1,000 ohm-cm, pH less than 5.5, soils with high organic content, sulfate concentrations greater than 1,000 ppm, landfills, or mine waste.

System and device descriptions shall include the dimensions of primary components as well as engineering drawings of the product. Any bracket connections to structures shall be prescriptively specified in construction details, including type and condition of structure to be supported, drill holes, bolts, washer plates, field welds, minimum concrete cover, concrete reinforcement, and leveling grout, as applicable. The recommended angle of shaft installation and maximum permissible departure from that angle shall be specified for each bracket. Construction details for bracket connections shall indicate that materials with different corrosion protection coatings shall not be combined in the same system and that helical foundation devices and systems shall not be placed in electrical contact (galvanically isolated) with structural steel, reinforcing steel, or any other metal building components.

A table of allowable capacities (tension, compression, and/or lateral) for all elements (P1, P2, P3, and P4, as applicable) shall be provided with listings for each system or device and all possible combinations and configurations. The evaluation report shall state that the allowable capacity of a helical foundation device or system shall be governed by the least allowable capacity, P1 through P4, as applicable.

If lateral resistance is included in the evaluation report, a table of soil capacity in the lateral direction based on load tests shall be provided for each type of shaft in each test soil condition. The evaluation report shall indicate that soil capacity in the lateral direction needs to be determined by a registered design professional unless the soil conditions for the site in question are generally consistent with soil types described in the evaluation report. For any helical foundation device subject to combined lateral and axial compression or axial tension, the evaluation report shall contain the maximum allowable lateral strength and the maximum allowable axial strength and shall state that the strength of the device is governed by the interaction equation given in the AISC reference standard.

The evaluation report shall provide a discussion of elastic shortening/lengthening, anticipated settlements, and typical elastic deflections, as applicable, depending the end use. The discussion shall contain design values from analysis or load tests.

6.2 Brackets: Bracket capacities, P1, shall include reference to the type of shaft and shall include provisions for, P2, shaft capacity. The table of side load bracket capacities also shall include a list of values or an equation for determining the maximum overturning moment specific to that type of bracket as a function of axial load supported. The allowable capacities of brackets connected to or embedded in concrete shall provide values for systems installed in the different concrete strengths that were evaluated. Installation shall be limited to uncracked concrete as defined in the applicable code. Allowable capacities for direct load brackets shall clearly identify the construction details for which those capacities are applicable. For slab support brackets, a table shall be provided showing recommended bracket spacing for support of different slabs under different loading conditions as described in Section 3.10.3. The table of capacities for brackets and shafts shall indicate whether the structure to be supported has to be sidesway braced or rotationally fixed based on assumptions used in the design and testing of the product.

6.3 Shafts: Shaft capacities shall be tabulated for each size of shaft for the conditions of being braced or unbraced in soft and firm soils as applicable. The evaluation report shall define these conditions by reference to Chapter 18 of the IBC. Standard penetration resistance blow count ranges for firm and soft soils described in Section 3.11.2.1 of this criteria shall be repeated in evaluation reports. The evaluation report shall state that the shaft capacity of helical foundations in fluid soils shall be determined by a registered professional engineer. For evaluation reports including provisions for lateral resistance, the structural properties of the shaft shall be provided including gross area, section modulus, modulus of elasticity, maximum allowable bending moment, and maximum allowable shear.

6.4 Helices: Helix compression and tension capacities shall be tabulated for each diameter, thickness, edge geometry, pitch, and material strength available. The evaluation report shall indicate that the capacities shall be added together for products with multiple helix plates.

6.5 Soil Capacity: If a soils capacity-to-torque ratio was validated, it shall be listed in the evaluation report along with the equations set forth in this acceptance criteria. Otherwise, the evaluation report shall indicate that soil capacity in compression or tension needs to be determined by a registered design professional. For lateral soil resistance, the evaluation report shall contain a table of capacities for all soil types used in the lateral load testing. The evaluation report shall state that lateral soil resistance shall be determined by a registered design professional for soil conditions that differ from those shown in the table.

6.6 Materials: The evaluation report shall list the material composition, including steel grades, of system and device components. Minimum material specifications for structures to be supported on brackets included in the evaluation report shall be included, as applicable.

6.7 Design: The evaluation report shall describe general procedures for design and application of the helical foundation system or device and state whether bracket capacity is based on a braced or unbraced helical system or device in accordance with IBC Section 1808. An explanation of the structural analysis that shall be performed by the design professional for proper application of the system or device including consideration of the internal shears and moment due to structure eccentricity and maximum span between helical foundations shall be provided. The magnitude of shear and moment forces exerted on the structure due to the connection of the structure to the helical foundation or device shall be provided. The results of this analysis and the structural capacities shall be used to select a helical foundation system. The evaluation report shall state a minimum helical foundation shaft spacing of four helix plate diameters to avoid group efficiency effects. The minimum embedment depth for various loading conditions shall be included based on analysis and tested conditions. The evaluation report shall indicate that Section 1808 of the IBC shall apply to these products.

6.8 Foundation and Soils Investigation Report: The evaluation report shall indicate that a site-specific foundation and soils investigation report is required for proper application of these products. The foundation and soils investigation report shall address corrosive properties of the soil to ensure that a potential pile corrosion situation does not exist. The foundation and soils investigation report shall address the support conditions for the shaft. The foundation and soils investigation report shall address the axial compression, axial tension, and lateral load soil capacities if values cannot be determined from the evaluation report. The foundation and soils investigation report shall address effects of groundwater and other questionable characteristics.

6.9 Installation: The evaluation report shall note any special training or certification required for installation professionals, equipment required for installation, and a detailed description of proper installation techniques. Requirements and procedures for quality assurance inspection of product installation shall be described, including procedures for field verification of ultimate maximum soil capacity for tension and compression through correlations with final installation torque, as applicable. The evaluation report shall state that the torque induced in the shaft shall not exceed the maximum installation torque. The evaluation report shall state that for tension applications, the pier shall be installed such that the minimum depth from the ground surface to the uppermost helix is 12D, where D is the diameter of the largest helix.

6.10 Special Inspection: For installation, the evaluation report shall state that special inspection in accordance with Section 1704.9 of the IBC or Section 1701.5.11 of the UBC is required. Where on-site welding is required, the evaluation report shall state that special inspection in accordance with Section 1704.3 of the IBC or Section 1701.5.5 of the UBC is required. The evaluation report shall state the items to be observed by the special inspector. At a minimum, these items shall include verification of manufacturer, helical pier and bracket configuration, the installation torque and depth of the foundation, and compliance of the installation of helical foundation system with the approved construction documents and this evaluation report. In lieu of continuous special inspection, periodic special inspection in accordance with IBC Section 1701.6.2 may be permitted when structural observations in accordance with IBC Section 1702, a periodic inspection schedule (prepared by the registered design professional), and evidence of installer training by the report holder are provided to the code official.

6.11 Identification: The evaluation report shall describe the identification method used by the manufacturer as set forth in Section 2.1.4.

6.12 Findings: The evaluation report shall list approved manufacturing facilities and their inspection agencies.■

TABLE 1—REFERENCE STANDARD EDITIONS

STANDARD	IBC	UBC
ANSI AF&PA NDS	2005	1991 revised
AISC ASD	AiSC 360-05	June 1, 1989
AISC LRFD	AISC 360-05	March 16, 1991
AWS D1.1	2004	1992

TABLE 2—SOIL CAPACITY ANALYSIS/TEST REQUIREMENTS[1]

HELIX COMBINATION	NUMBER OF HELICES	SAND	CLAY	HARD BEDROCK	NUMBER OF COMPRESSION TESTS	NUMBER OF TENSION TESTS
Smallest diameter	1	C/T		C	2	1
Largest Diameter	1		C/T	C	2	1
Any two diameters	2	C/T	C/T		2	2
Any three diameters	3	C/T	C/T		2	2
Minimum Number of Tests Required					8	6

[1]C = Compression; T = Tension.

TABLE 3—TORQUE CORRELATION CONFORMANCE CRITERIA

	CRITERIA
1	Square shafts with dimensions between 1.5 inches by 1.5 inches and 1.75 inches by 1.75 inches, or round shafts with outside diameters between 2.875 inches and 3.5 inches
2	True helix shaped plates that are normal with the shaft such that the leading and trailing edges that are within $^1/_4$ inch of parallel.
3	Capacity is within normal capacity limits
4	Helix plate diameters between 8 inches and 14 inches with thickness between $^3/_8$ inch and $^1/_2$ inch.
5	Helix plates and shafts are smooth and absent of irregularities that extend more than $^1/_{16}$ inch from the surface excluding connecting hardware and fittings.
6	Helix spacing along the shaft shall be between 2.4 to 3.6 times helix diameter.
7	Helix pitch is 3 inches ± $^1/_4$ inch.
8	All helix plates have the same pitch.
9	Helical plates are arranged such that they theoretically track the same path as the leading helix.
10	For shafts with multiple helices, the smallest diameter helix shall be mounted to the leading end of the shaft with progressively larger diameter helices above.
11	Helical foundation shaft advancement equals or exceeds 85% of helix pitch per revolution at time of final torque measurement.
12	Helix piers shall be installed at a rate less than 25 revolutions per minute.
13	Helix plates have generally circular edge geometry.

For **SI:** 1 inch = 25.4 mm.

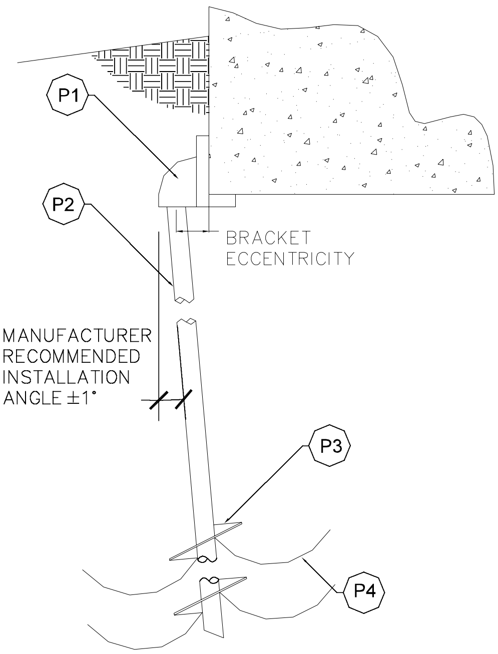


FIGURE 1—TYPE A SIDE LOAD APPLICATION

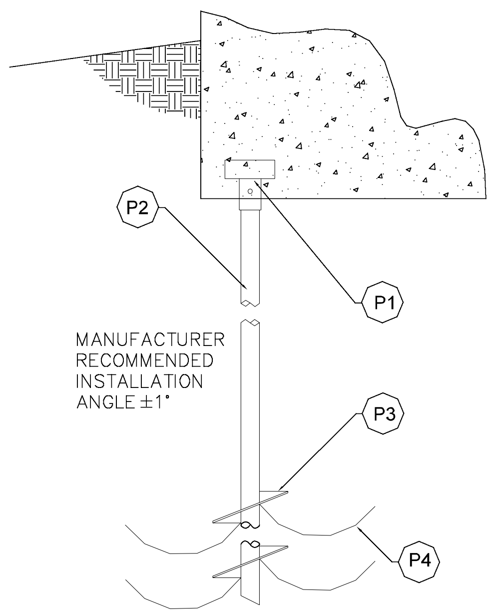


FIGURE 2—TYPE B DIRECT LOAD APPLICATION

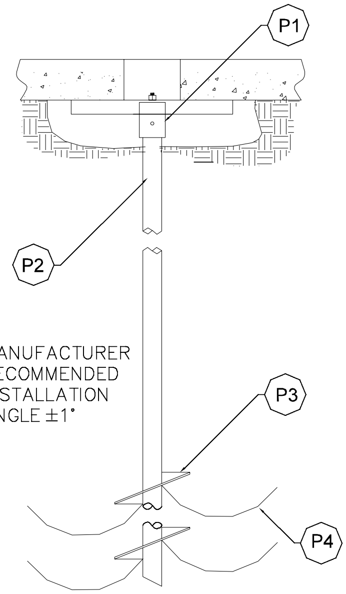


FIGURE 3—TYPE C SLAB SUPPORT APPLICATION

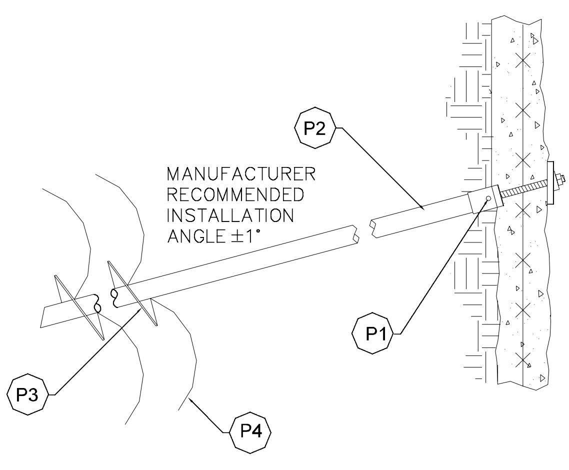


FIGURE 4—TYPE D TENSION ANCHOR APPLICATION

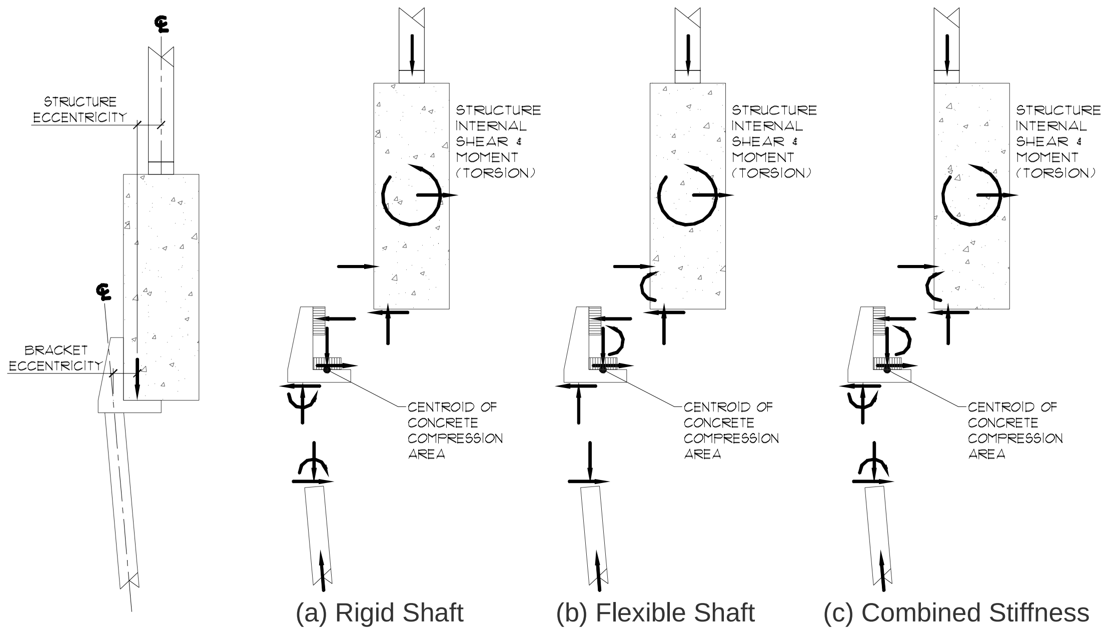
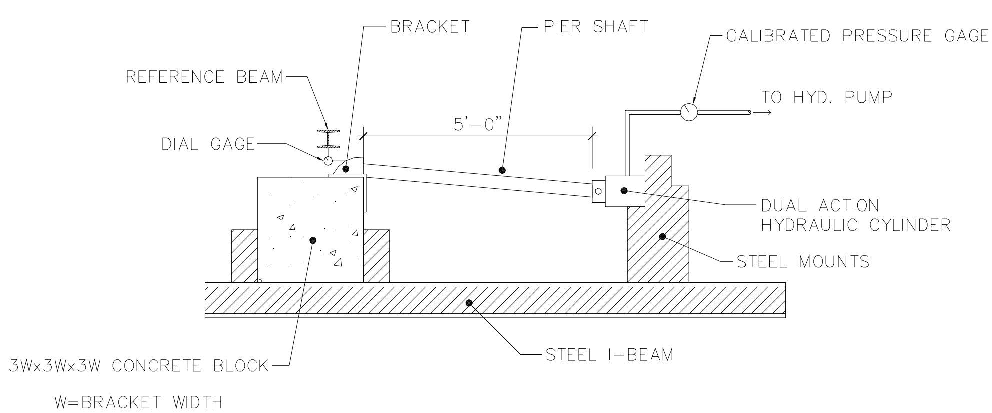
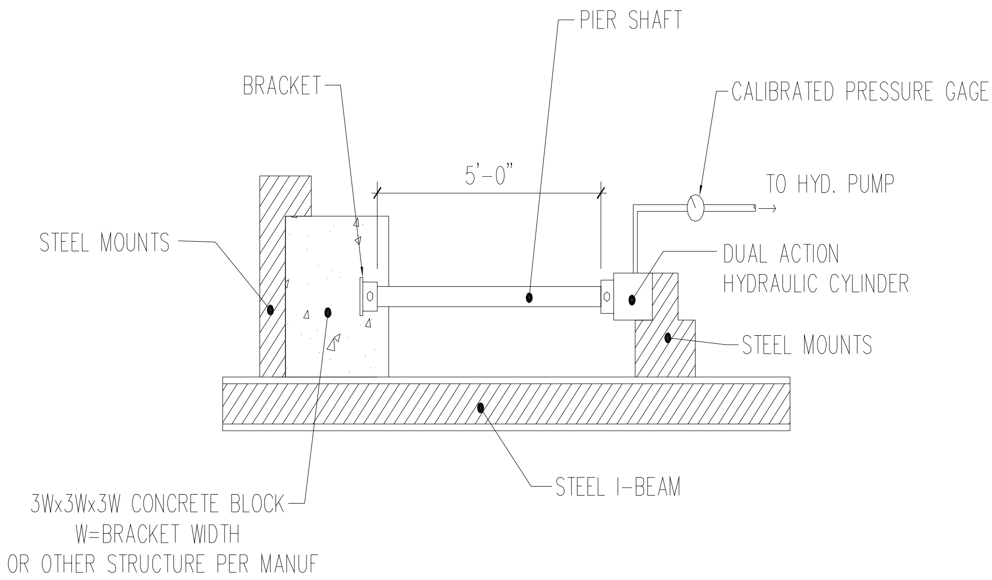


FIGURE 5—TYPE A BRACKET FREE BODY DIAGRAMS

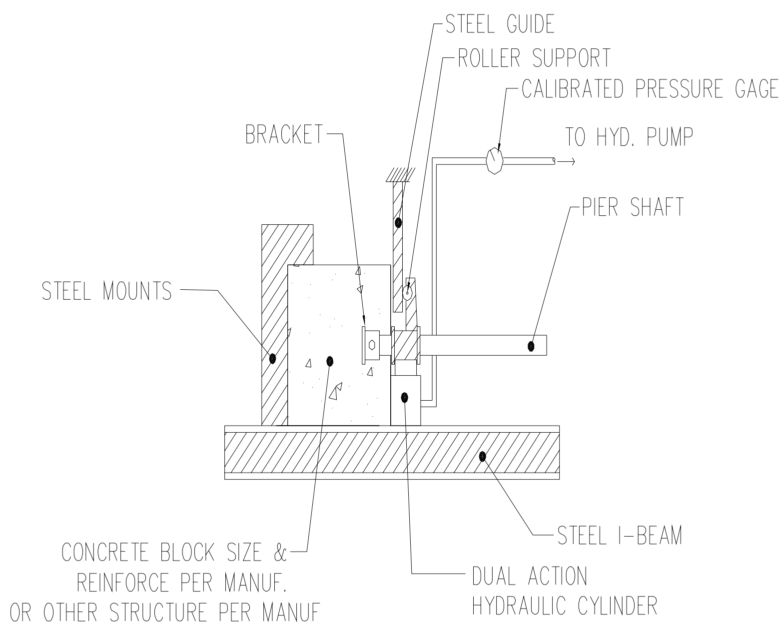


For **SI:** 1 inch = 25.4 mm.

FIGURE 6—TYPE A BRACKET EXAMPLE LABORATORY TEST SET-UP

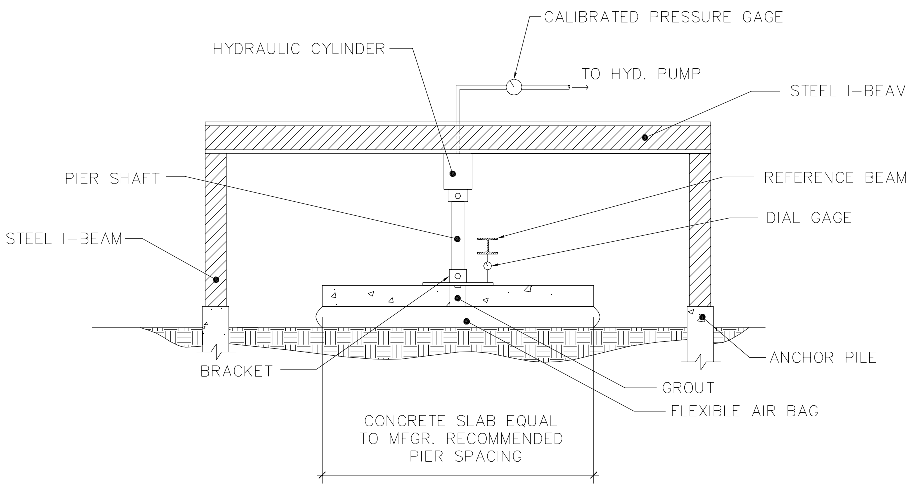


(AXIAL LOAD SET-UP)



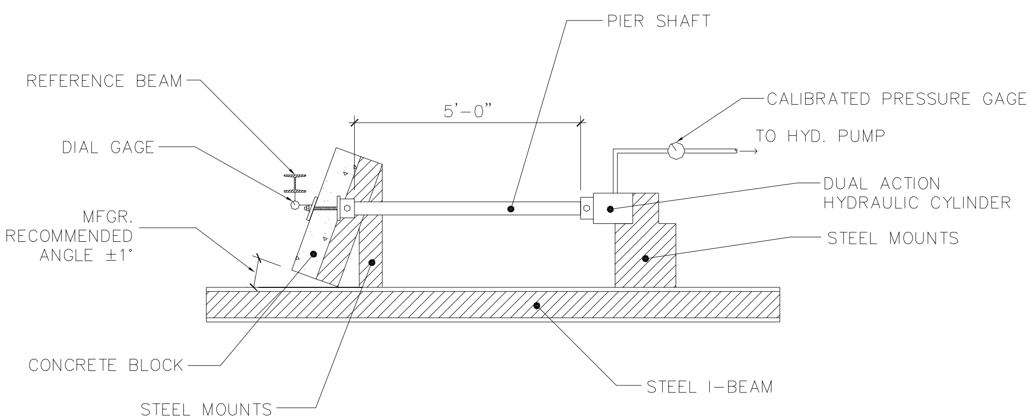
(LATERAL LOAD SET-UP)

FIGURE 7—TYPE B BRACKET EXAMPLE LABORATORY TEST SET-UP



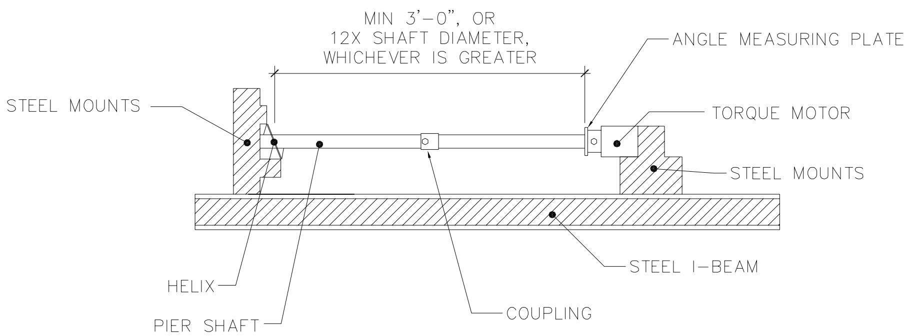
For **SI:** 1 inch = 25.4 mm.

FIGURE 8—TYPE C BRACKET EXAMPLE LABORATORY TEST SET-UP



For **SI:** 1 inch = 25.4 mm.

FIGURE 9—TYPE D BRACKET EXAMPLE TEST SET-UP



For **SI:** 1 inch = 25.4 mm.

FIGURE 10—SHAFT TORSION EXAMPLE LABORATORY TEST SET-UP

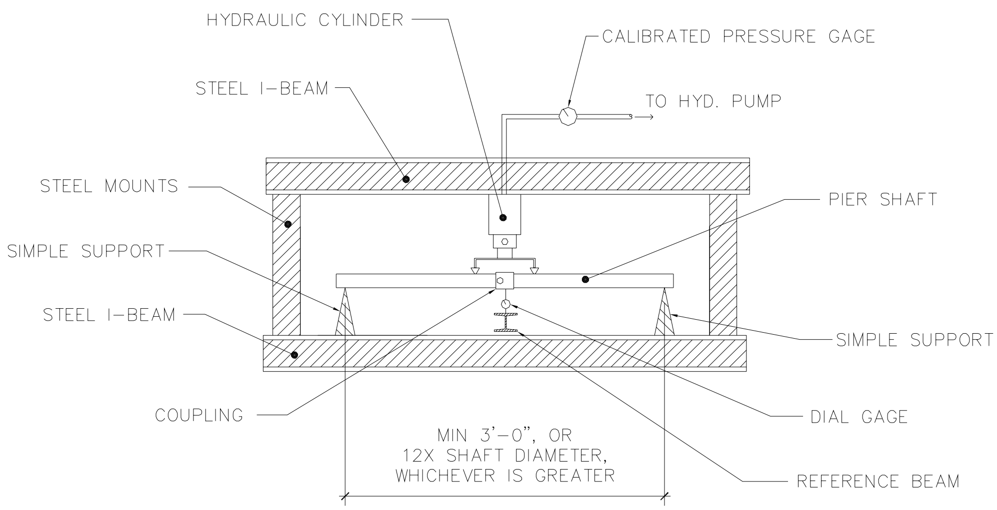


FIGURE 11—SHAFT BENDING EXAMPLE LABORATORY TEST SET-UP

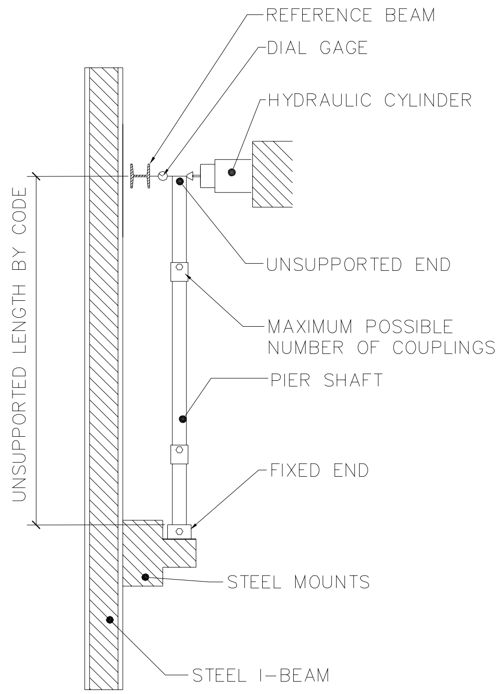


FIGURE 12—COUPLING RIGIDITY EXAMPLE LABORATORY TEST SET-UP

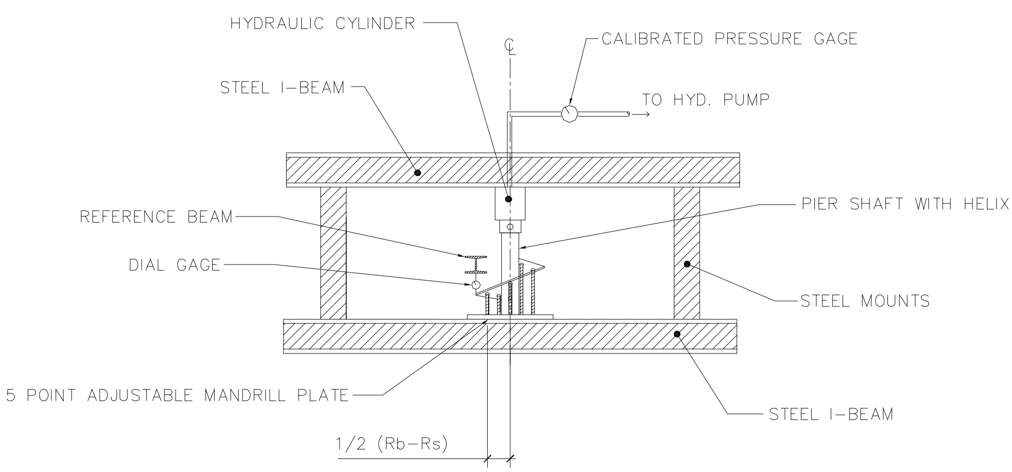


FIGURE 13—HELIX EXAMPLE LABORATORY TEST SET-UP

Section 10
Grouted Pile Illustration

TECHNICAL BULLETIN

MPS Civil Products

MacLean Vortex Pile (MVP)

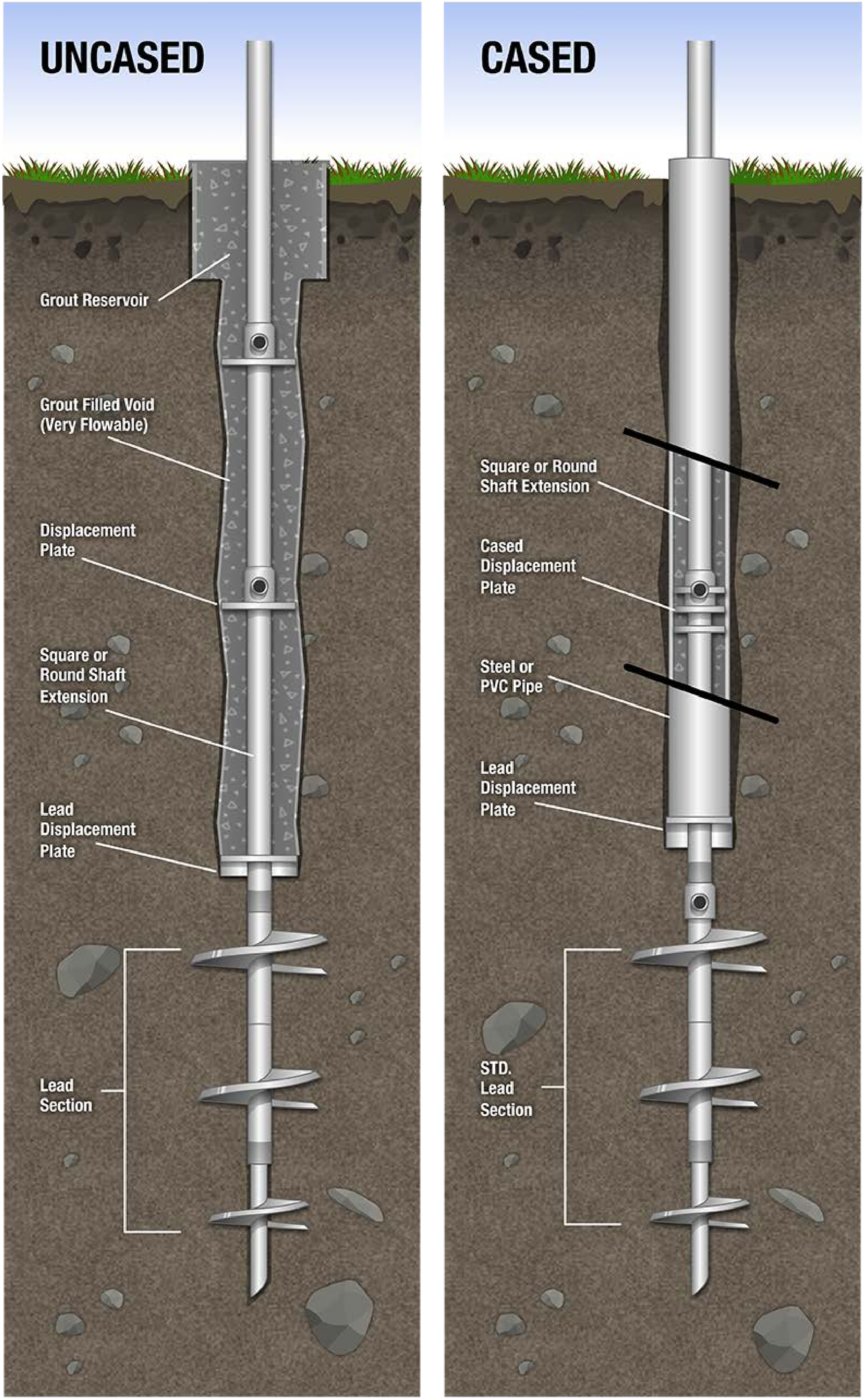
THE MACLEAN VORTEX PILE HAS ARRIVED

This month MacLean Power Systems - Civil Products Group is pleased to announce the MacLean Vortex Pile (MVP). The MVP is an easily installed grouted helical pipe pile. Applicable to a wide variety of soil conditions, the MVP provides the rapid and simple installation expected of a helical pile, but delivers the higher buckling resistance and lateral capacity associated with concrete piling systems.

Typical MVP Installation

THE MVP ADVANTAGE

- Grouted pipe pile provides structural steel reinforcement provides increased bending strength for higher lateral capacity.
- Helical flights plus skin friction along the grouted column allows for up to 98% increase in capacity over similarly sized helical piles.
- Concrete column construction with no spoils to truck off site
- Installed with same tooling as traditional helical piles
- Micropile magnitude loads with very low mobilization costs


UNCASED
Grout Reservoir
Grout Filled Void (Very Flowable)
Displacement Plate
Square or Round Shaft Extension
Lead Displacement Plate
Lead Section
CASED
Square or Round Shaft Extension
Cased Displacement Plate
Steel or PVC Pipe
Lead Displacement Plate
STD. Lead Section


HUBBELL

Section 11
Sample Theoretical Calcs

Mr. Travis Bedson October 6, 2020
CNT Foundations, LLC
3362 Navajo Street
North Charleston SC 29405
Office: (843)-577-7268
Cell: (843)-343-9068
Email: travis@yourfoundationexperts.com

RE: NUCOR Degasser
Darlington, SC

Dear Mr. Bedson:

Per the request by CSD Structural Engineers for a 280K pile we have estimated the previous 224K pile with a 16″ grout collar for the first ten feet in place of the 12″ collar and a 10″ grout collar to 72 feet deep.

The 4″ SCH 40 (4.50″ O.D.) 12/14/16″ 79′ proposed pile with a 16″ grout column for the top 10′, then a 10″ grout column for the next 62′ will have an ultimate capacity of:
If do pile with 16″ plate for first 10′, then 62′ of 10″ plate
$Q_T = Q_{Tip} + Q_{friction}$

$Q_{Tip} = 100{,}000 lbs$
Maximum capacity of the end-bearing for a 4.0″ shaft with triple helix

The adhesion of the grouted column is estimated as follow (NAVFAC DM7.2):

$Q_{ult} = \Sigma_{H=H_0}^{H=H_0+D} K_{HT} \times P_0 \times tan\delta \times S \times H$
$P_O = \gamma \times H$ Effective vertical stress
$Q_{1-Ult} = K_{HC} \times \gamma \times H \times tan\delta \times D \times \pi$
S = Surface Area
K_{HC}=1.25 ☛ See Earth Pressure Coefficients, Driven single displacement piles
Grout shall have a minimum compressive strength of 4,500 psi

Note 1 says friction increases up to a limiting depth of 20B.
Table 1 says that the friction angle can be estimated as 3/4Φ

$$Q_{friction} = K_{HC} \times \frac{\gamma \times H}{2} \times tan\left(\frac{3}{4} \times \emptyset\right) \times 20B \times \left(Dia_{grout}\right) \times \pi$$
$$+ K_{HC} \times \gamma \times H \times tan\left(\frac{3}{4} \times \emptyset\right) \times Depth \times \left(Dia_{grout}\right) \times \pi =$$
$$Q_{friction} = 1.25 \times \frac{1340}{2} \times tan\left(\frac{3}{4} \times 33\right) \times 10 \times \left(\frac{16}{12}\right) \times \pi$$
$$+ 1.25 \times 1340 \times tan\left(\frac{3}{4} \times 33\right) \times 62 \times \left(\frac{10}{12}\right) \times \pi =$$
$$Q_{friction} = 1.25 \times 670 \times tan(24.75) \times 10 \times \left(\frac{4}{3}\right) \times \pi$$
$$+ 1.25 \times 1340 \times \tan(24.75) \times 62 \times \left(\frac{5}{6}\right) \times \pi = 16{,}173 + 125{,}338$$
$$Q_{friction} = 141{,}511 lbs$$

$$\boldsymbol{Q_T} = 100{,}000 + 141{,}511 \cong 241{,}500\ lbs$$

The contribution of the helices is limited to 100,000 pounds as the available torque is 20,000 lb-ft and the Kt is 5.5.
This is only an estimate and typically grouted piles have performed better than the geotechnical estimates.

For calculations for pile top plate to transfer load to pile cap, axial, bending, and torque capacities of the shaft we present.

Helical pile top plates

For the 120K pile on 3.5″ OD pipe

$b_o d = 4(10" + 10") \times 10" = 800\ in^2$

Shear strength for punching shear action

$V_c = 4 \times \sqrt{f_c'} \times b_o \times d$

$V_c = 4\sqrt{3000}\ PSI \times 800\ in^2 = 175{,}270\ lb$

$\emptyset V_c = 0.75 \times V_c \times 0.7 = 92{,}000\ lb$

$P_u < \emptyset V_c \quad 60{,}000\ lb < 92{,}000\ lb \therefore O.K.$

Bending of the embedded plate into concrete

Depth;N = 10″

Breadth; B = 10″

Design strength; F_y = **50.0** ksi

Axial Load; Pa =**60,000** lb

Safety factors

Flexure; Ω_b = **1.67**

Plate cantilever dimensions

Area of base plate; A_1 = B x N = **100** in^2

$D_0 = 3.5\ in$

Bending line cantilever distance m; m = (N - 0.80 x D_o) / 2 = **3.6** in

Bending line cantilever distance n; n = (B - 0.80 x D_o) / 2 = **3.6** in

Maximum bending line cantilever; l = max (m, n) = **3.6** in

Required plate thickness;

$$t_p = \left(l\times \sqrt{\frac{2\times\Omega_b\times P_a}{F_y\times B\times N}} \right) = \left(3.6\times \sqrt{\frac{2\times1.67\times60}{50\times10\times10}} \right) = 0.72"$$

Use 10"x10"x3/4" plate with 10" of cover

For the 280K pile on 4.5" OD pipe

280/(0.35x3)=267 SI

$b_o d = 4(14" + 12") \times 12" = 1248\ in^2$

Shear strength for punching shear action

$V_c = 4 \times \sqrt{f_c'} \times b_o \times d$

$V_c = 4\sqrt{3000}\ PSI \times 1248\ in^2 = 273{,}420\ lb$

$\emptyset V_c = 0.75 \times V_c \times 0.7 = 143{,}550\ lb$

$P_u < \emptyset V_c \ \ 140{,}000\ lb < 143{,}550\ lb \therefore O.K.$

Bending of the embedded plate into concrete

Depth;N = 14"

Breadth; B = 14"

Design strength; F_y = **50.0** ksi

Axial Load; Pa =**140,000** lb

Safety factors

Flexure; Ω_b = **1.67**

Plate cantilever dimensions

$D_0 = 4.5\ in$

Bending line cantilever distance m; m = (N - 0.80x D_o) / 2 = **5.2** in

Bending line cantilever distance n;n = (B - 0.80x D_o) / 2 = **5.2** in

Maximum bending line cantilever;l = max (m, n) = **5.2**in

Required plate thickness;

$$t_p = \left(l\times \sqrt{\frac{2\times\Omega_b\times P_a}{F_y\times B\times N}} \right) = \left(5.2\times \sqrt{\frac{2\times1.67\times140}{50\times14\times14}} \right) = 1.13"$$

Use 14"x14"x1 ¼" plate with 12" of cover

4.5" shaft axial compression and tension=2.96x0.6x50,000=88,800#

Three (3) - A325 7/8" bolts double shear=122.7K

Geotechnical compression and tension for the helical pile

$Q = A \times \gamma H \times N_q = (0.79 + 1.07 + 1.4) \times 1691 \times 54.5 = 300{,}440$

This cannot be achieved as torque limits embedment, so uplift it limited to 100 kips of the helical pile.

Bolt bearing=2.4x0.221x0.75x65,000x6=155,140 lbs

Bolt torque=155,140x2.25/12=29.1K-ft

Shaft Torque

$$Shaft_{Torque} = \frac{efficiency \times 0.75 \times F_u \times J}{O.D/2} =$$

$$Shaft_{Torque} = \frac{0.82 \times 0.75 \times 65{,}000 \times 13.6}{4.5/_2} = 241{,}626in - lb \; \cong 20{,}130ft - lb$$

Helix weld two ¼" welds (top and bottom of the helix)

Perimeter around this pipe is πx(4.5)" = 14.14"

$$R_n = 0.6F_{Exx} \times \frac{\sqrt{2}}{2} \times \frac{D}{16} \times l \times (1.0 + 0.5 \times sin^{1.5}\theta)$$

$$\Omega = 2.0$$

$$\frac{R_n}{\Omega} = 2 \times \frac{0.6 \times 70000 \times \frac{\sqrt{2}}{2} \times \frac{4}{16} \times \pi \times 4.5(1 + 0.5sin^2 90)}{2} = 157{,}444lb$$

This weld capacity is per helix.

Helix AC358 testing of a 14" helix on a 2.88" OD shaft averaged 104.4K ultimate before the shaft failed. The helix did not fail due to radial ribs.

Shaft buckling per Davisson for sand with an N of 3 at 20 feet

k_h=30.5

$$R = \sqrt[4]{\frac{EI}{k_h d}} = \sqrt[4]{\frac{29{,}000{,}000 \times 6.82}{30.5 \times 4.5}} = 34$$

For pinned-pinned connection

U_{cr}=2

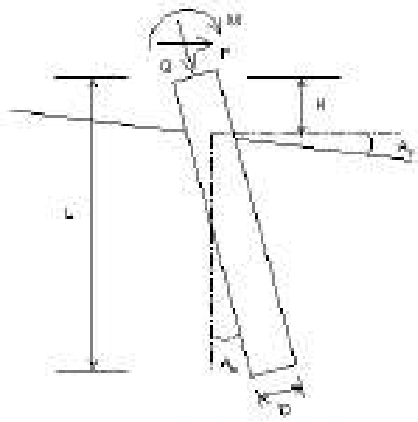
$$P_{cr} = \frac{U_{cr} \times EI}{R^2} = \frac{2 \times 29{,}000{,}000 \, ^{lb}/_{in^2} \times 6.82in^4}{(34in)^2} 342{,}180lb$$

The capacity is limited to structural capacity of pile shaft

Lateral by Allpile for 2K is

LATERAL ANALYSIS

Figure 2



Driving Steel Pile (Open end)

Loads:
Load Factor for Vertical Loads= 1.0
Load Factor for Lateral Loads= 1.0
Loads Supported by Pile Cap= 0 %
Shear Condition: Static

(with Load Factor)
Vertical Load, Q= 140.0 -kp
Shear Load, P= 2.0 -kp
Moment, M= 0.0 -kp-f

Profile:
Pile Length, L= 80.0 -ft
Top Height, H= 0 -ft
Slope Angle, As= 0
Batter Angle, Ab= 0
Free Head Condition

Soil Data:

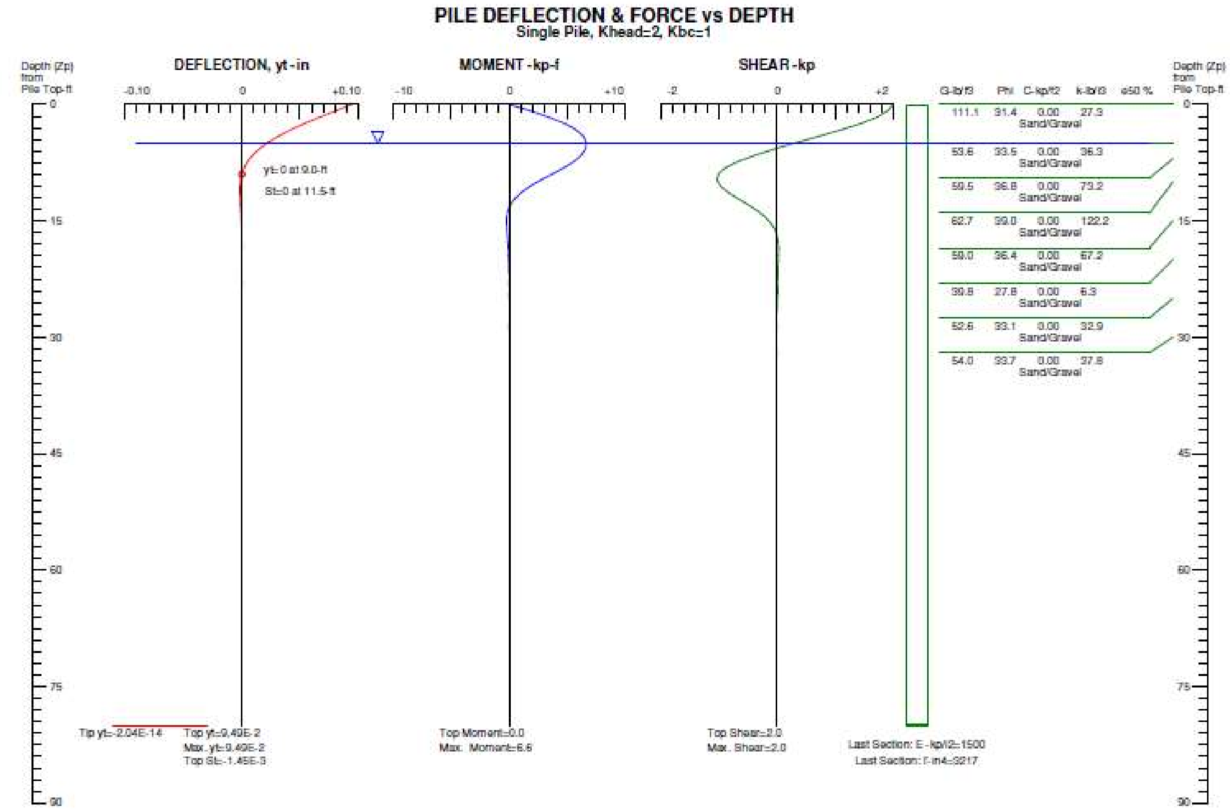
Depth -ft	Gamma -lb/f3	Phi	C -kp/f2	K -lb/i3	e50 or Dr %	Nspt
0	111.1	31.4	0.00	27.3	27.04	7
5	53.6	33.5	0.00	38.3	37.24	11
7	59.5	36.8	0.00	73.2	56.64	22
10	62.7	39.0	0.00	122.2	75.23	40
15	59.0	36.4	0.00	67.2	53.92	20
20	39.8	27.8	0.00	6.3	11.50	3
25	52.6	33.1	0.00	32.9	35.04	10
30	54.0	33.7	0.00	37.8	38.19	11

Pile Data:

Depth -ft	Width -in	Area -in2	Per. -in	I -in4	E -kp/i2	Weight -kp/f
0.0	4.5	201	50	3217	1500	210
80.0						

Single Pile Lateral Analysis:
Top Deflection, yt= 0.09490-in
Max. Moment, M= 6.60-kp-f
Top Deflection Slope, St= -0.00145
OK! Top Deflection, 0.0949-in is less than the Allowable Deflection= 1.00-in

Note: If the program cannot find a result or the result exceeds the upper limit. The result will be displayed as 99999.
The Max. Moment calculated by program is an internal force from the applied load conditions. Structural engineer has to check whether the pile has enough capacity to resist the moment with adequate factor of safety. If not, the pile may fail under the load conditions.



CivilTech Software

Figure 2

K_t Extrapolating the K_t formula from AC358 $K_t=22.285(4.5)^{-0.9195}=5.5$

We appreciate the opportunity to be of service to you on this project and trust that you will call this office with any questions that you may have.

Sincerely,

Philip E. Slemons, P.E.
THE FOUNDATION FIRM

THE FOUNDATION FIRM

September 19, 2019

Mr. Travis Bedson
CNT Foundations, LLC
3362 Navajo Street
North Charleston SC 29405
Tel: (843)-577-7268
Cell: (843)-343-9068

Subject: Helical Grouted Pile - Estimation of Load Bearing Capacity
Dominium Energy - LNG
2400 Bushy Park Road, Goose Creek, South Carolina 29445

Mr. Bedson,

At your request The Foundation Firm, LLC performed a review of the geotechnical study of the above refererenced project in order to provide a recommendation of a grouted helical pile.

We have finished the review and we are glad to present to you justification and anticipated capacity for an 8″ grouted round 2.5″ SCH 40 helical pile and a 4.5″ SCH 40 helical based on what we know of the site with the information given in the geotechnical study by WPC dated September 12, 2018. The piles we analyzed have total depth of 63 feet for the 4.50″ shaft and 42 feet for the 2.5″ shaft. Ultimate bearing capacity for the 4.5″ shaft is 71 kip and 45.9 kip for the 2.5 shaft.
Since the lateral capacity for this type of pile is less than one kip, we are recommending that four additional piles be installed on each side of the continuous footing (shed structure) at a batter angle of 25 degrees from the vertical plane. We anticipated these battered piles to reach a depth of 21 feet where soils are more dense. Single 14″ helical piles will be adequate for the batter and vertical piles.
For the pipe support structure footings we are recommending the use of 4.0″ SCH 40 grouted helical piles with single 14″ helix in order to support the anticipated lateral and axial loads. Attached is an analysis in ALLPILE that shows deflections will be kept below half of an inch with the size shaft which is adequate for similar structures.

We appreciate the opportunity to be of service to you on this project and trust that you will call this office with any questions that you may have.

Respectfully,

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER
No. 27217
PHILIP E SLEMONS
9/19/19

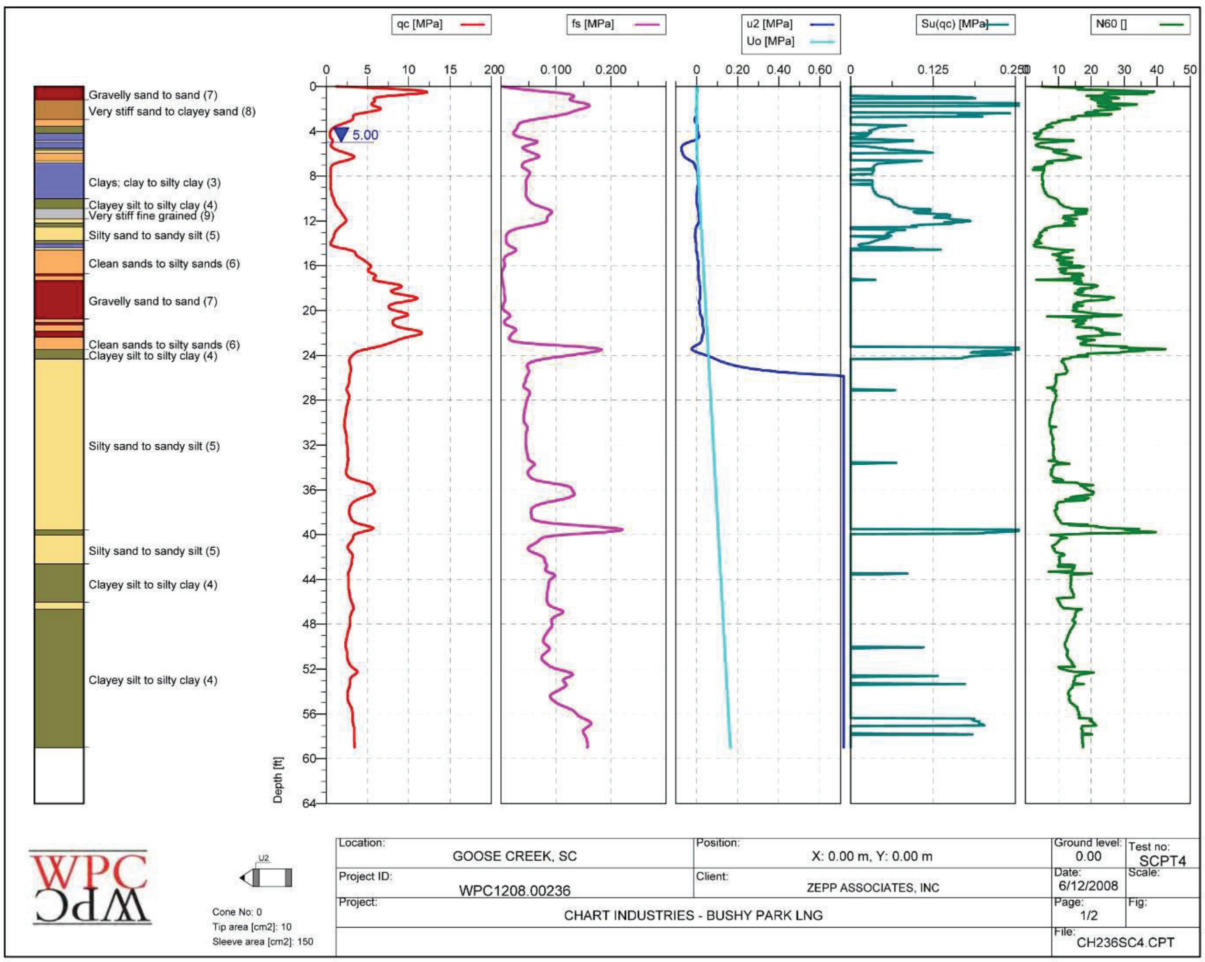
THE FOUNDATION FIRM

Philip E Slemons, P.E.
Senior Engineer

Pipe Support Structure

Analysis of a 4.5" SCH 40 round shaft square shaft with single 14" helix to 63 feet in depth with an 8" grout column to 56 feet is as follows.

This calculations make reference to NAVFAC DM7.2, below is a snapshot of the basic principle of friction that is going to be used in this set.

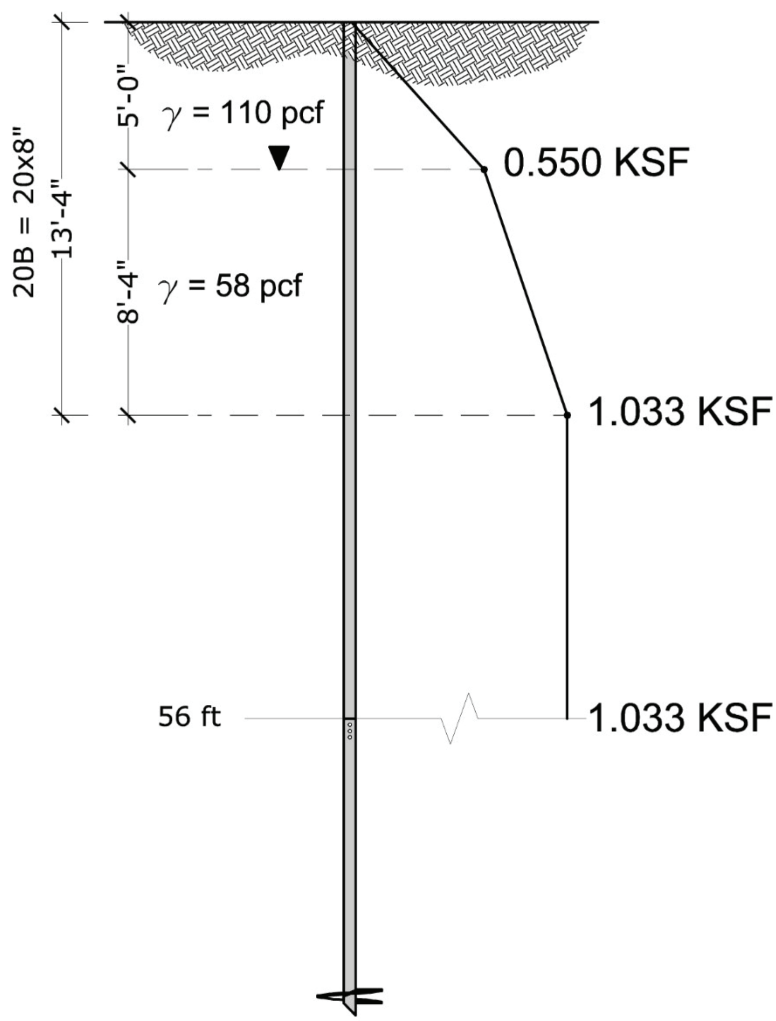


Note 1 says friction increases up to a limiting depth of 20B.

Table 1 says that the friction angle can be estimated as 3/4Φ

The adhesion of the grouted column is estimated as follow:

$$Q_{ult} = \Sigma_{H=H_0}^{H=H_0+D} K_{HT} \times P_0 \times tan\delta \times S \times H$$



Properties of soils for analysis

Type of soil	Layer Depth (ft)	Angle Φ	K+HC factor	Adhesion, C_A (psf)
Sand and Concrete	0-40	27	1.5	--
Clay and Steel	40	--		900

Tables taken from NAVFAC DM7.2

EARTH PRESSURE COEFFICIENTS K+HC, AND K+HT,

PILE TYPE	K+HC,	K+HT,
DRIVEN SINGLE H-PILE	0.5 - 1.0	0.3 - 0.5
DRIVEN SINGLE DISPLACEMENT PILE	1.0 - 1.5	0.6 - 1.0
DRIVEN SINGLE DISPLACEMENT TAPERED PILE	1.5 - 2.0	1.0 - 1.3
DRIVEN JETTED PILE	0.4 - 0.9	0.3 - 0.6
DRILLED PILE (LESS THAN 24" DIAMETER)	0.7	0.4

Table 1 (NAVFAC DM7.2)

FRICTION ANGLE - [delta]

PILE TYPE	[delta]
STEEL	20 deg.
CONCRETE	3/4 [phi]
TIMBER	3/4 [phi]

Friction Angles

RECOMMENDED VALUES OF ADHESION

PILE TYPE	CONSISTENCY OF SOIL	COHESION, C PSF	ADHESION, C_A PSF
TIMBER AND CONCRETE	VERY SOFT	0 - 250	0 - 250
	SOFT	250 - 500	250 - 480
	MED. STIFF	500 - 1000	480 - 750
	STIFF	1000 - 2000	750 - 950
	VERY STIFF	2000 - 4000	950 - 1300
STEEL	VERY SOFT	0 - 250	0 - 250
	SOFT	250 - 500	250 - 460
	MED. STIFF	500 - 1000	460 - 700
	STIFF	1000 - 2000	700 - 720
	VERY STIFF	2000 - 4000	720 - 750

Adhesion Values

Friction capacity (0-58')

$Q_{1-Ult} = K_{HC} \times P_o \times tan\delta \times S \times H$

$P_O = \gamma \times D$

$Q_{1-U} \;\; = K_{HC} \times \gamma \times H \times tan\delta \times D \times \pi$

S = Surface Area

$$Q_{1-Ult} = 1.5 \times \frac{5 \times 110 + 8.33 \times 58}{2} \times tan\left(\frac{3}{4} \times 27\right) \times 13.33 \times \left(\frac{8}{12}\right) \times \pi$$
$$+ 1.5 \times 1033 \times tan\left(\frac{3}{4} \times 27\right) \times 42.67 \times \left(\frac{8}{12}\right) \times \pi = 7{,}980 + 51{,}080$$
$$= 59{,}060\ lb$$

End bearing capacity for the 14-helix helical pile

$$Q_{2-Ult} = 1.07 \times 9 \times 125 \times 10 = 12{,}030\ lb$$

Total ultimate pile capacity

$$Q_{ult} = \sum Q_n = 59{,}060 + 12{,}030 = 71{,}090\ lb$$

$$Q_{allow} = \frac{Q_{ult}}{2} = \frac{71{,}090}{2} = 35{,}545\ lb$$

Shed Support Foundation Helical Piles

The load for the helical piles of the shed structure is 30 ASD factored load. The helical pile does not need to be as deep as the helical piles for the pipe support structure. Below is analysis of a 2.5" SCH 40 round shaft grouted pile to a depth of 42 feet with an 8" grout column to 35 feet.

$$Q_{1-Ult} = 1.5 \times \frac{5 \times 110 + 8.33 \times 58}{2} \times tan\left(\frac{3}{4} \times 27\right) \times 13.33 \times \left(\frac{8}{12}\right) \times \pi + 1.5 \times 1033 \times tan\left(\frac{3}{4} \times 27\right) \times 21.67 \times \left(\frac{8}{12}\right) \times \pi = 7{,}980 + 25{,}940 = 33{,}920\ lb$$

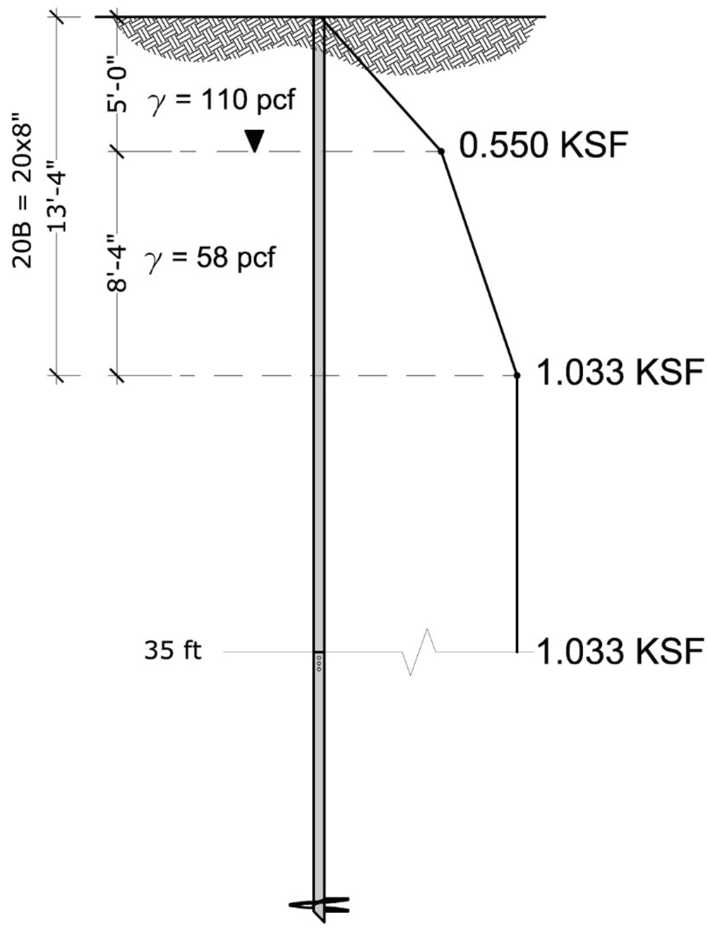
End bearing capacity for the 14-helix helical pile

$$Q_{2-Ult} = 1.07 \times 9 \times 125 \times 10 = 12{,}030\ lb$$

Total ultimate pile capacity

$$Q_{ult} = \sum Q_n = 33{,}920 + 12{,}030 = 45{,}950\ lb$$

$$Q_{allow} = \frac{Q_{ult}}{2} = \frac{45{,}950}{2} = 22{,}975\ lb$$



20B = 20x8"
13'-4"
5'-0"
γ = 110 pcf
0.550 KSF
8'-4"
γ = 58 pcf
1.033 KSF
35 ft
1.033 KSF

THE FOUNDATION FIRM

February 13, 2019

Mr. Travis Bedson
CNT Foundation
3362 Navajo Street
North Charleston, SC 29405
Tel.: (843)-577-7268

RE: MUSC Wellness Center
Grouted Helical Pile Submittal Justification
45 Courtenay Dr., Charleston, SC

Dear Mr. Bedson,

At your request The Foundation Firm, LLC is providing revised calculations of a 1.5" solid square shaft 6" grouted helical pile to a maximum depth of 100 feet.

In this document we provide the anticipated load capacity based on data is known from the geotechnical study by SCI Construction Materials.

We appreciate the opportunity to be of service to you on this project and trust that you will call this office with any questions that you may have.

Sincerely,

THE FOUNDATION FIRM, LLC

Philip E Slemons, P.E.
Senior Engineer

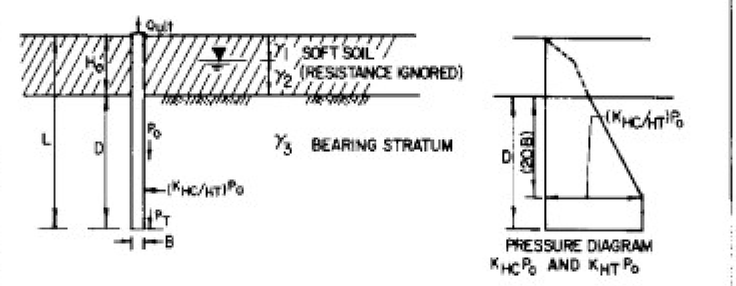
SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 27217 PHILIP E SLEMONS
2/13/19

71 1st Avenue, NW - Lilburn, GA 30047
T 770-564-6949 - F 678-280-(FIRM)
info@foundationfirm.com
www.foundationfirm.com

Analysis of a 1.5” solid square shaft with single 12” helix to 100 ft in depth.

This calculations make reference to NAVFAC DM7.2, below is a snapshot of the basic principle of friction that is going to be used in this set.

Friction



(A) ULTIMATE LOAD CAPACITY IN COMPRESSION

$$Q_{ult} = P_T N_q A_T + \sum_{H=H_0}^{H=H_0+D} (K_{HC})(P_0)(\tan\delta)(S)$$

WHERE Q_{ult} = ULTIMATE LOAD CAPACITY IN COMPRESSION
P_T = EFFECTIVE VERTICAL STRESS AT PILE TIP (SEE NOTE 1)
N_q = BEARING CAPACITY FACTOR (SEE TABLE, FIGURE 1 CONTINUED)
A_T = AREA OF PILE TIP
K_{HC} = RATIO OF HORIZONTAL TO VERTICAL EFFECTIVE STRESS ON SIDE OF ELEMENT WHEN ELEMENT IS IN COMPRESSION.
P_0 = EFFECTIVE VERTICAL STRESS OVER LENGTH OF EMBEDMENT, D (SEE NOTE 1)
δ = FRICTION ANGLE BETWEEN PILE AND SOIL (SEE TABLE, FIGURE 1 CONTINUED)
S = SURFACE AREA OF PILE PER UNIT LENGTH

FOR CALCULATING Q_{all}, USE F_S OF 2 FOR TEMPORARY LOADS, 3 FOR PERMANENT LOADS (SEE NOTE 2)

(B) ULTIMATE LOAD CAPACITY IN TENSION

$$T_{ult} = \sum_{H=H_0}^{H=H_0+D} (K_{HT})(P_0)(\tan\delta)(S)(H)$$

WHERE: T_{ult} = ULTIMATE LOAD CAPACITY IN TENSION, PULLOUT
K_{HT} = RATIO OF HORIZONTAL TO VERTICAL EFFECTIVE STRESS ON SIDE OF ELEMENT WHEN ELEMENT IS IN TENSION

FOR CALCULATING T_{all}, USE F_S = 3 ON T_{ult} PLUS THE WEIGHT OF THE PILE (W_P), THUS $T_{all} = \frac{T_{ult}}{3} + W_P$ (SEE NOTE 2)

NOTE-1: EXPERIMENTAL AND FIELD EVIDENCE INDICATE THAT BEARING PRESSURE AND SKIN FRICTION INCREASE WITH VERTICAL EFFECTIVE STRESS P_0 UP TO A LIMITING DEPTH OF EMBEDMENT. DEPENDING ON THE RELATIVE DENSITY OF THE GRANULAR SOIL AND POSITION OF THE WATER TABLE. BEYOND THIS LIMITING DEPTH (10B ± TO 40B ±) THERE IS VERY LITTLE INCREASE IN END BEARING, AND INCREASE IN SIDE FRICTION IS DIRECTLY PROPORTIONAL TO THE SURFACE AREA OF THE PILE. THEREFORE, IF D IS GREATER THAN 20 B, LIMIT P_0 AT THE PILE TIP TO THAT VALUE CORRESPONDING TO D = 20B.

NOTE-2: IF BUILDING LOADS AND SUBSURFACE CONDITION ARE WELL DOCUMENTED IN THE OPINION OF THE ENGINEER, A LESSER FACTOR OF SAFETY CAN BE USED BUT NOT LESS THAN 2.0 PROVIDED PILE CAPACITY IS VERIFIED BY LOAD TEST AND SETTLEMENTS ARE ACCEPTABLE.

FIGURE 1
Load Carrying Capacity of Single Pile in Granular Soils

Figure 1

Note 1 says friction increases up to a limiting depth of 20B.
Table 1 says that the friction angle can be estimated as 3/4Φ

The adhesion of the grouted column is estimated as follow:

$$Q_{ult} = \Sigma_{H=H_0}^{H=H_0+D} K_{HT} \times P_0 \times tan\delta \times S \times H$$

Properties of soils for analysis

Type of soil	Layer Depth (ft)	Angle Φ	K+HC factor	Adhesion, C_A (psf)
Sand and Steel	0-30	20	1.15	--
Sand and Concrete	30-50	27	1.10	--
Sand and Concrete	50-60	30	1.20	--
Silt and Concrete	60-85	--		500
Silt and Concrete	85-90	--		1300
Silt and Concrete	90-113	--		900

Tables taken from NAVFAC DM7.2

EARTH PRESSURE COEFFICIENTS K+HC, AND K+HT,

PILE TYPE	K+HC,	K+HT,
DRIVEN SINGLE H-PILE	0.5 - 1.0	0.3 - 0.5
DRIVEN SINGLE DISPLACEMENT PILE	1.0 - 1.5	0.6 - 1.0
DRIVEN SINGLE DISPLACEMENT TAPERED PILE	1.5 - 2.0	1.0 - 1.3
DRIVEN JETTED PILE	0.4 - 0.9	0.3 - 0.6
DRILLED PILE (LESS THAN 24" DIAMETER)	0.7	0.4

Table 1 (NAVFAC DM7.2)

FRICTION ANGLE - [delta]

PILE TYPE	[delta]
STEEL	20 deg.
CONCRETE	3/4 [phi]
TIMBER	3/4 [phi]

Friction Angles

RECOMMENDED VALUES OF ADHESION

PILE TYPE	CONSISTENCY OF SOIL	COHESION, C PSF	ADHESION, C_A PSF
TIMBER AND CONCRETE	VERY SOFT	0 - 250	0 - 250
	SOFT	250 - 500	250 - 480
	MED. STIFF	500 - 1000	480 - 750
	STIFF	1000 - 2000	750 - 950
	VERY STIFF	2000 - 4000	950 - 1300
STEEL	VERY SOFT	0 - 250	0 - 250
	SOFT	250 - 500	250 - 460
	MED. STIFF	500 - 1000	460 - 700
	STIFF	1000 - 2000	700 - 720
	VERY STIFF	2000 - 4000	720 - 750

Adhesion Values

Grouted Helical Pile

The 6" diameter grouted pile is going to be at 93', the single 12" helix will be at 100 feet.

Friction capacity (0-20')

$$Q_{1-Ult} = K_{HC} \times P_o \times tan\delta \times S \times H$$
$$P_O = \gamma \times D$$
$$Q_{1-Ul} \ = K_{HC} \times \gamma \times H \times tan\delta \times D \times \pi$$

S = Surface Area

$$Q_{1-Ul} \ = 1.15 \times 1{,}100 \times tan\left(\frac{3}{4} \times 20\right) \times \left(\frac{6}{12}\right) \times \pi \times 20 = 10{,}649lb$$

Friction capacity (20-50')

$$Q_{2-Ult} = 1.10 \times 1{,}100 \times tan\left(\frac{3}{4} \times 27\right) \times \left(\frac{6}{12}\right) \times \pi \times 30 = 22{,}948lb$$

Friction capacity (50-60')

$$Q_{3-Ult} = 1.20 \times 1{,}100 \times tan\left(\frac{3}{4} \times 30\right) \times \left(\frac{6}{12}\right) \times \pi \times 10 = 8{,}588lb$$

Friction capacity (60-93')

$$Q_{4-Ult} = C_A \times 2 \times \pi \times R \times H = C_A \times \pi \times D \times H$$

$$Q_{4-Ult} = (500 \times 25 + 1300 \times 5 + 900 \times 3) \times \left(\frac{6}{12}\right) \times \pi = 34{,}086lb$$

End bearing capacity for the single helix helical pile

Single 12"

$$Q_{5-Ult} = A_1 \times 9 \times N_c$$

$$Q_{5-Ult} = 0.79 \times 9 \times (30 \times 125) \quad = 26{,}663lb$$

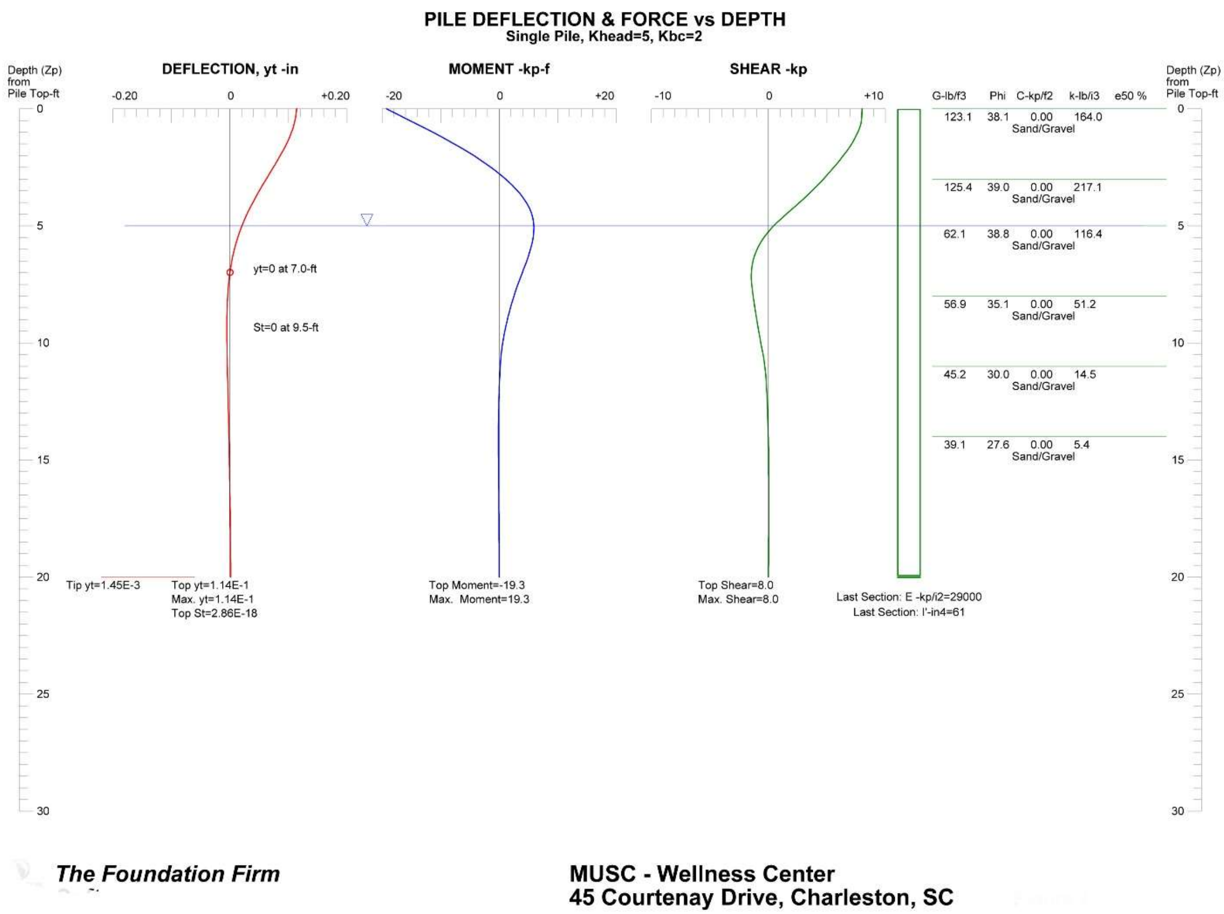
Total ultimate pile capacity

$$Q_{ult} = \sum Q_n = 10{,}648 + 22{,}984 + 8{,}588 + 34{,}086 + 26{,}663 = 102{,}696lb$$

$$Q_{allow} = \frac{Q_{ult}}{2} = \frac{102{,}969}{2} = 51{,}485lb$$

Per ALLPILE lateral load analysis 20 feet of 8" API pipe casing will be necessary to support the flexure of the bending moment. See diagram below.

Grouted Helical Pile



Section 12
Load Test Equipment Certification

Model 3000

Load Cell

Instruction Manual

Document Revision: M | Release date: 11/30/20

Electrical Resistance Type Load Cells

Applications

The Model 3000 Electrical Resistance Strain Gauge Type Load Cells are used for…

- Monitoring loads in tie-backs and rock bolts in the walls of excavations
- Monitoring loads in steel arch tunnel supports
- Monitoring loads in cross lot struts
- Measurement of loads during pile testing

- *Model 3000 Series Load Cells.*

- *Closeup of cable insertion showing **Kellems®** wire mesh grip.*

- *Model GK-502 Readout for use with the Model 3000 Series Load Cells.*

Operating Principle

The Model 3000 Load Cell is designed primarily for use on tiebacks and rockbolts. They may also be used during pile load tests and for monitoring loads in crosslot struts and tunnel supports, etc.

In most situations, the Model 3000 is used in conjunction with bearing plates, positioned on either side of the load cell.

Where load cells are used to check the load as determined by the hydraulic pressure applied to the jack, during proof-testing on tiebacks, rockbolts, etc., the user should be aware that, due to the annular design and the many variables in load distribution, the agreement cannot be guaranteed better than ±15%.

In use, load cells are positioned so that the tensile load in the tieback or rockbolt produces a compressive load in the load cell. This is done by trapping the load cell between bearing plates positioned between the jack and the structure, either below the anchor plate for permanent installations or above the anchor plate for proof-testing.

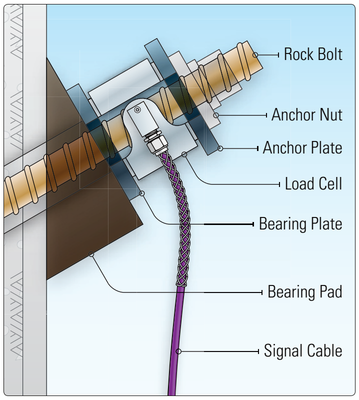
Advantages and Limitations

The Model 3000 Load Cell is made from an annulus of high strength steel or aluminum. Electrical resistance strain gauges are cemented around the outside of the annulus and connected in a Wheatstone Bridge circuit so that there is a single mV/V output. Remote sensing techniques are used to minimize cable effects. Solid load cells are also available.

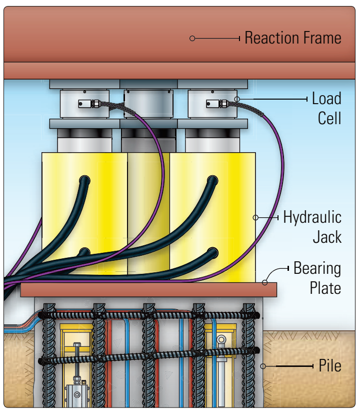
An outer shell protects the gauges from damage and 'O'-rings on either side of the gauges ensure that the load cell is fully waterproof.

The cable is attached to the cell through a waterproof gland. A strain relief, in the form of a **Kellem**'s grip, prevents the cable from being pulled out of the cell. Cables have thick PVC jackets and can be terminated in a 10 pin connector to mate with the GK-502 Readout.

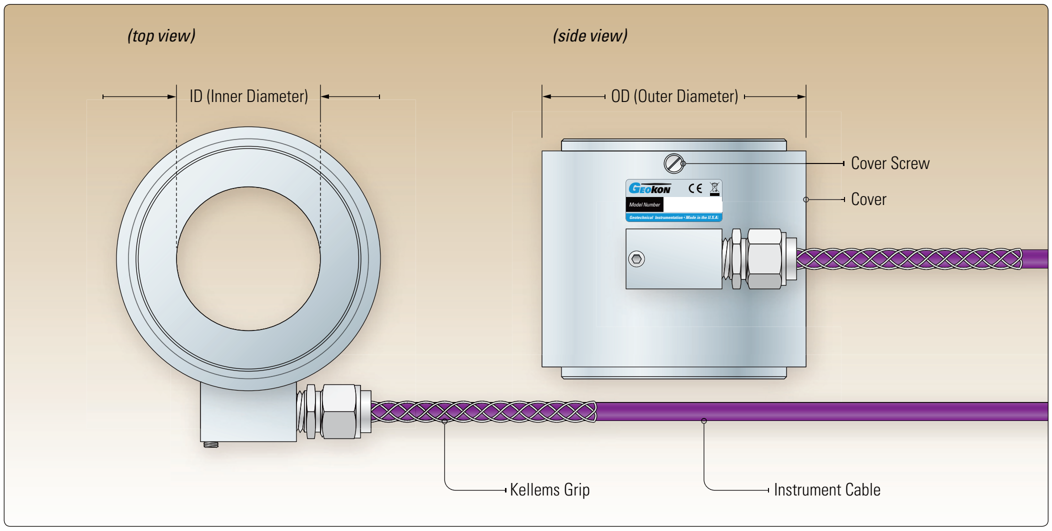
The calibration of annular shaped load cells is very dependent on the end loading conditions, i.e. on the flatness and thickness of the bearing surfaces and on any mismatch in size between the load cell and the hydraulic ram which could cause bearing plates to bend. Calibration variations of as much as 15% have been observed. For best accuracy, calibrations should be performed while duplicating or simulating actual field conditions.



• *Load cell used to monitor performance of tie-back in excavation.*



• *Load cells used in pile test.*



• *Model 3000 Series Load Cell components.*

System Components

Signals from the load cell are transmitted to the readout location by means of a multi-conductor shielded cable, which may be armored for extra protection. **Kellems** grips prevent the cable from being pulled from the load cell. Larger size load cells are supplied with lifting lugs.

To minimize eccentric and uneven loading, the use of the thick machined-flat bearing plates and centralizer bushings (where necessary) are recommended.

Bearing plates should be machined flat and large enough to totally cover the load bearing surface of the load cell. The thickness is related to the load cell/ hydraulic jack size mismatch: the greater the size disparity the thicker the bearing plate. Typical thickness ranges from 25 to 75 mm.

If the size of the tie-back or rock bolt is more than 20 mm smaller than the internal diameter of the load cell, then centralizer bushings are recommended.

Readout of the Model 3000 Load Cells is achieved using the Model GK-502 Readout or with the Micro-800/1000 Dataloggers.

Technical Specifications

Rated Capacities[1]	100 to 10,000 kN
Over Range[2]	150% F.S.
Resolution	0.025% F.S.
Accuracy[3]	±0.5% F.S.
Output	1.5 to 2.5 mV/V @ F.S.
Temperature Range[4]	−20 °C to +80 °C
Cables	Multi-conductor shielded pairs with PVC outer jacket
Internal Diameters[1]	solid, 25, 50, 75, 100, 125, 150, 200, 250 mm

[1]*Other capacities and diameters available on request.*
*Calibrations that exceed **GEOKON**'s NIST traceable capacity of approximately 10,675 kN are subcontracted to an accredited testing laboratory.*
[2]*With no calibration shift.*
[3]*Established under laboratory conditions. System accuracy depends on end loading conditions.*
[4]*Other ranges available on request.*

GEOKON®

Load Cell Calibration Report

Model Number: 3000-200-2

Calibration Date: February 11, 2020

Serial Number: 1532440

This calibration has been verified/validated as of 02/11/2020

Max. Range (lbs): 200000

Calibration Instruction: CI-3000 Tinius

Cable Length: 15 feet

Technician:

Initial Cycling Data

Load (lbs):	0	0	300000	0
Reading:	2668	2654	22388	2660

Calibration

Applied Load in lbs	Readings from GK-501 or GK-502 readout box				Linearity % Max Load	Polynomial Error (%FS)
	Cycle 1	Cycle 2	Average	Change		
0	2660	2660	2660		-1.00	-0.72
20000	4083	4083	4083	1423	-0.17	0.00
40000	5419	5419	5419	1336	0.00	0.06
60000	6726	6727	6727	1308	-0.05	-0.08
80000	8050	8058	8054	1327	0.05	-0.03
100000	9381	9378	9380	1326	0.14	0.04
120000	10689	10691	10690	1310	0.12	0.01
140000	12001	12001	12001	1311	0.09	0.02
160000	13301	13301	13301	1300	-0.01	-0.04
180000	14614	14611	14613	1312	-0.03	0.03
200000	15901	15914	15908	1295	-0.18	-0.01
0	2655	2657	2656			

GK-501 or GK-502 Readout

Linear Gauge Factor (G): 15.22 lbs/digit

Regression Zero (R_0):* 2791

Polynomial Gauge Factors: **A: 0.00001552** **B: 14.91** **C: -41150**

Polynomial, $L = AR_1^2 + BR_1 + C$

Full Scale mV/ V: 3.312 mV/ V

Calculate C by setting L=0 and R_1 = initial field zero reading in the polynomial equation

* Note: The above calibration uses a linear regression method. The Regression Zero Reading shown is ideal for straight line computation and does not usually agree with the actual no-load reading.

The above named instrument has been calibrated by comparison with standards traceable to the NIST, in compliance with ANSI Z540-1.

GEOKON

Load Cell Calibration Report

Model Number: 3000-400-4

Calibration Date: September 01, 2020

Serial Number: 1909475

This calibration has been verified/validated as of 10/05/2020

Max. Range (lbs): 400000

Calibration Instruction: CI-3000 Tinius

Cable Length: 40 feet

Technician:

Initial Cycling Data

Load (lbs):	0	0	400000	0
Reading:	-1063	-1058	3287	-1056

Calibration

Applied Load in lbs	Readings from GK-501 or GK-502 readout box				Linearity % Max Load	Polynomial Error (%FS)
	Cycle 1	Cycle 2	Average	Change		
0	-1056	-1058	-1057		0.62	0.62
40000	-643	-645	-644	413	0.07	0.07
80000	-212	-212	-212	432	-0.04	-0.04
120000	225	227	226	438	-0.02	-0.01
160000	658	658	658	432	-0.13	-0.12
200000	1097	1105	1101	443	0.01	0.03
240000	1536	1544	1540	439	0.06	0.08
280000	1967	1975	1971	431	-0.08	-0.06
320000	2412	2414	2413	442	0.04	0.05
360000	2853	2853	2853	440	0.11	0.12
400000	3279	3282	3281	428	-0.10	-0.11
0	-1056	-1056	-1056			

GK-501 or GK-502 Readout

Linear Gauge Factor (G): 91.55 lbs/digit

Regression Zero (R_0):* -1084

Polynomial Gauge Factors: **A: -0.00002053** **B: 91.61** **C: 99270**

Polynomial, $L = AR_1^2 + BR_1 + C$

Full Scale mV/ V: 1.085 mV/ V

Calculate C by setting L=0 and R_1 = initial field zero reading in the polynomial equation

* Note: The above calibration uses a linear regression method. The Regression Zero Reading shown is ideal for straight line computation and does not usually agree with the actual no-load reading.

The above named instrument has been calibrated by comparison with standards traceable to the NIST, in compliance with ANSI Z540-1.

The World Leader in Vibrating Wire Technology

48 Spencer Street
Lebanon, NH 03766, USA
Tel: 603•448•1562
Fax: 603•448•3216
E-mail: geokon@geokon.com
http://www.geokon.com

Instruction Manual

Model GK-502

Load Cell Readout

(Doc Rev C, 02/13)

Load Cell Readout

Applications

The Model GK-502 Load Cell Readout is designed to read the **GEOKON**® Model 3000 full bridge resistance strain gauge type load cells. The rugged and reliable, user-friendly GK-502 features the following…

- Easy-to-use control panel
- Output displayed in engineering units
- 16×2 LCD display
- 10-pin load cell connector
- USB communications port
- Internal real-time clock
- Non-volatile memory
- Rechargeable battery
- Data storage capability
- Cold weather operation

● Model GK-502 Load Cell Readout.

● Close-up of the Model GK-502 Load Cell Readout control panel.

Operating Principle

The Model GK-502 Load Cell Readout is a portable battery powered instrument designed to read all full bridge resistance strain gauge type load cells, including the **GEOKON** Model 3000 Load Cells.

The readout incorporates a 12 Volt, 1.4 Ahr Sealed Lead Acid (SLA) battery, 16×2 graphic liquid crystal display (LCD) with backlight, membrane keypad, and battery charger circuit. Two side-mounted 10-pin, military style **Bendix**® connectors are provided; the first is used to connect the load cell, and the second is used for communications, via a USB connection (COM port), and for charging of the battery, via the battery charger.

The GK-502 supplies a precision 2.048 VDC excitation to the full bridge Load Cell and displays the output in Digits, mV, mV/V, or by entering a Gauge Factor and Zero Reading, in engineering units (lbs, kg, kips, Tons, etc.).

An internal Real-Time Clock/Calendar (RTCC) and non-volatile memory allows storage for up to 999 time-stamped readings, which can be displayed via the LCD display, or downloaded to a computer via the COM port for further analysis.

The GK-502 is designed to read both 4-wire and 6-wire remote-sense full bridge load cells.

Power consumption of the GK-502 is very low (300 mW), and will allow continuous operation for up to 48 hours under normal conditions. Continuous battery monitoring is included to warn the user when the battery is low and requires recharging.

● *Model GK-502 Load Cell Readout, showing load cell and communication connectors.*

● *Model GK-502 Load Cell Readout shown with the Model 3000 Electrical Resistance Load Cell.*

Advantages and Limitations

The Model GK-502 is designed to be user-friendly with push button operation for all functions.

The display shows digits, mV, mV/V or engineering units.

Readings, including reading number, date and time, can be stored by pressing the "STORE" button.

To power off the GK-502, press the "ON/OFF" switch. Alternatively, the GK-502 will automatically shut off after five minutes of remaining idle.

Load cells are easily connected via the side-mounted 10-pin connector, or via the supplied patch cord with alligator clips.

Remote sense capabilities for added accuracy with long cable lengths.

Stored data can be downloaded through the 10-pin USB port for use in spreadsheet applications.

System Components

The Model GK-502 is supplied complete with battery charger, USB cable, USB driver (CD format), patch cord with alligator clips, for connection to load cell cables without 10-pin connectors, and manual.

Technical Specifications

Display Resolution	1 uV (mV, mV/V); 1 digit (Dg); 1 lb (lbs.); 1 kg (kg); 0.01 kip (kips); 0.01 ton (tons); 0.01 metric ton (metric tons); 0.01 kN (kilonewton)
Accuracy	±0.05% F.S. (±30 digits)
Range (S+S–)	±16 mV (±31,250 digits)
ADC	Differential 24 bit Sigma Delta
ADC Resolution	1.9 nV
Excitation Voltage/ ADC Reference	2.048 V (± 0.001 V) 3 ppm/°C
Display	16 × 2 graphic LCD with backlight
Connectors	*Bulkhead:* **Bendix** PT02A-12-10S *Mating:* **Bendix** PY06A-12-10P(SR)
Operating Temperature	–30 °C to +50 °C
Power Requirements	12 VDC @ 22 mA (operation) 12 VDC @ 16 µA (off)
AC Adaptor	120/230 VAC: 50-60 Hz, 18 VDC, 1.66 A
Battery	*(type)* Lead acid 12 volt, 1.4 Ahr *(operating time)* 48 hours
Materials	Aluminum case and lid
Weight	2.3 kg
L × W × H	165 × 102 × 216 mm

Section 13
Drive Head Torque Correlation

SERVICE MANUAL
76 SERIES DRIVE HEADS

Example Part Number

76	BA	6	4	F	71	B
Model	Ratio	Output Shaft	Bail Boss	Motor Supplier	Motor Number	Shift Pressure

THIS SERVICE MANUAL IS EFFECTIVE:
S/N: 126302 TO CURRENT
DATE: 7-29-2014 TO CURRENT
VERSION: SM76_0714

NOTE: Individual customer specifications may vary from exploded drawing and standard part numbers shown. If applicable, refer to customer drawing for details.

SERVICE MANUAL
50 SERIES DIGGER MODELS

Example Part Number

50	05	2	1	F	54
Model	Ratio	Shaft	Bail Boss	Motor Supplier	Motor Number

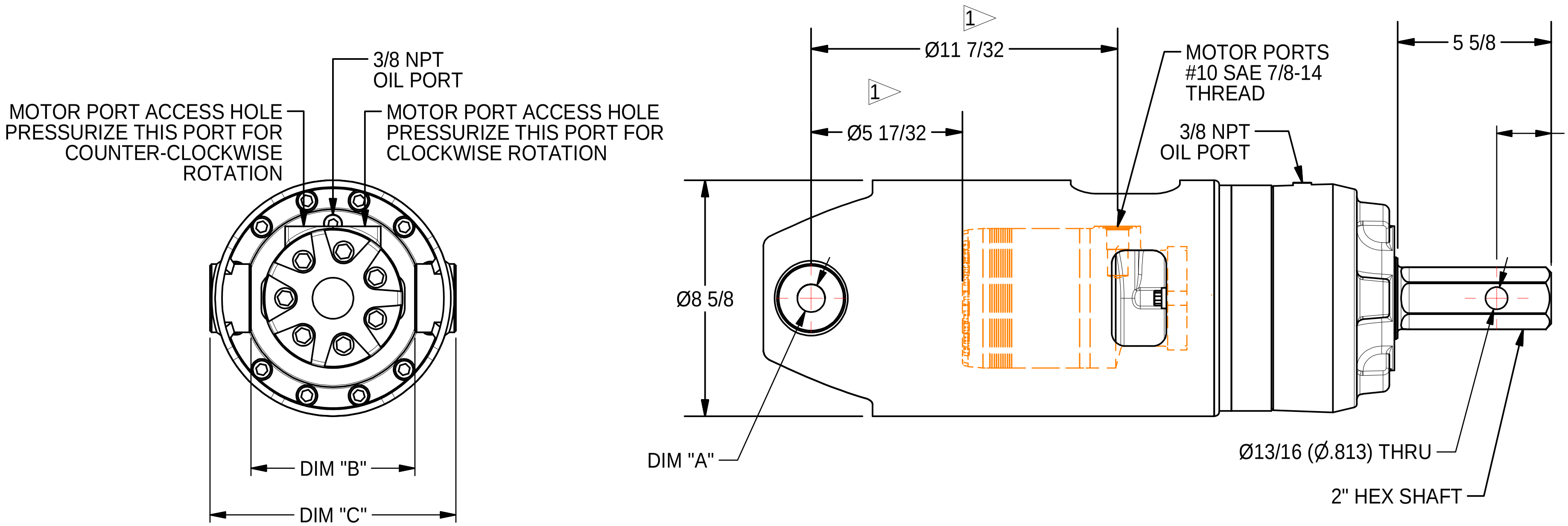
THIS SERVICE MANUAL IS EFFECTIVE:
S/N: 58670 TO CURRENT
DATE: 9-2003 TO CURRENT
VERSION: SMD50L-AC

NOTE: Individual customer specifications (spindle mounting, sprocket pilot, brake assembly, etc.) may vary from exploded drawing and standard part numbers shown. If applicable, refer to customer drawing for details.

50 SERIES DRIVE HEAD

2,800 ft-lb (3,796 N-m) to 5,000 ft-lb (6,780 N-m) Maximum
Typical Applications: Utility and Construction Industries

(913) 782-1238 (Tel)
(913) 782-4206 (Fax)
Sales@Eskridgeinc.com
www.Eskridgeinc.com
FORM: PS D50-AC



NOTES:

1. SUBJECT TO CHANGE WITH DIFFERENT MOTORS.
2. VALUES ARE SHOWN IN INCHES.

	A	B	C
CODE 1 BAIL BOSS	Ø1.01	6	9
CODE 2 BAIL BOSS	Ø1.26	6 3/8	11 3/8

ESKRIDGE

50 SERIES DRIVE HEAD

2,800 ft-lb (3,796 N-m) to 5,000 ft-lb (6,780 N-m) Maximum
Typical Applications: Utility and Construction Industries

(913) 782-1238 (Tel)
(913) 782-4206 (Fax)
Sales@Eskridgeinc.com
www.Eskridgeinc.com
FORM: PS D50-AC

Performance Ratings

Maximum Intermittent Output Torque:	5,000 ft-lb (6,780 N-m)
Shaft Pressure Load:	15,000 lb (6,800 kg)
Shaft Pullout Load:	10,000 lb (4,540 kg)
*Shaft Side loading Not Recommended	
Maximum Input Speed:	3,000 RPM
Maximum Operating Ambient Temperature:	110°F (43°C)
Minimum Operating Ambient Temperature:	-20°F (-28°C)

Consult your Eskridge representative to determine ratings for your specific application or configuration

Oil Capacities

5005	5016
2.0 PINTS 1.0 LITERS	3.0 PINTS 1.4 LITERS

ALL UNITS WITH MOTORS SHIPPED WITH OIL
Use EP or API GL-5 Designated Lubricants

Ordering Information
Example Part Number: 5005-21F55

MODEL		OUTPUT SHAFT	BAIL BOSS	MOTOR SUPPLIER	MOTOR PART NUMBER
5005	—	2	1	F	55
5005- 5:1 REDUCTION 5016- 16:1 REDUCTION		2- 2" HEX	1- Ø1 2- Ø1 1/4	F- FACTORY U- USER	54- 10.2 CU. IN. 55- 17.1 CU. IN.

Motor Mount Dimensions

PILOT DIMENSIONS	BOLT CIRCLE
SAE 'A' Ø3.251/3.255 ↧.32	2X 1/2-13 UNC-2B ↧.79 ON Ø4.188 BC

Motor Specifications:

Single Speed Hydraulic Vane Motors
Maximum System Pressure: 3000 psi (207 bar)
No case drain required

Approximate Unit Weight

With Motor: 150 lb (68 kg)

50 SERIES DRIVE HEAD

2,800 ft-lb (3,796 N-m) to 5,000 ft-lb (6,780 N-m) Maximum
Typical Applications: Utility and Construction Industries

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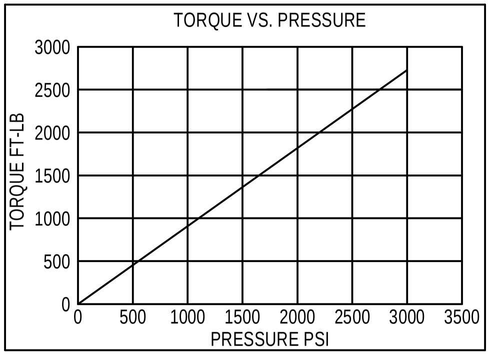
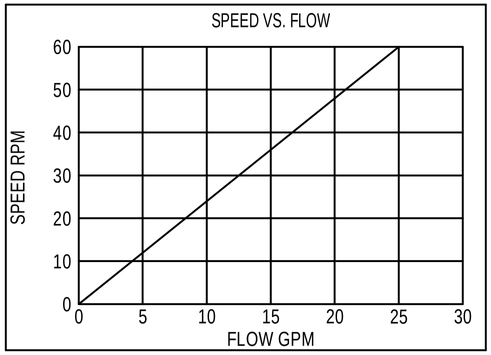
FORM: PS D50-AC

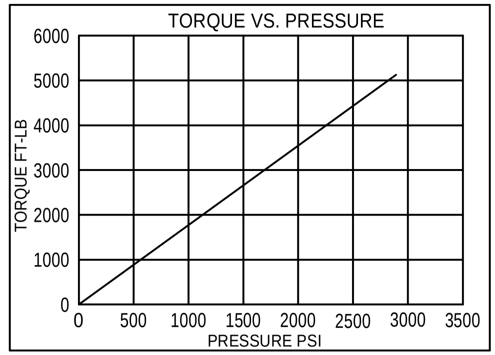
Torque and Speed Charts:

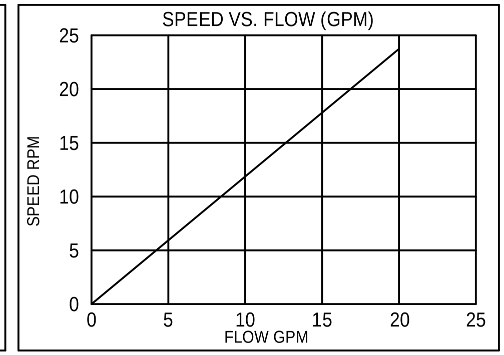
5005 With 55 Motor



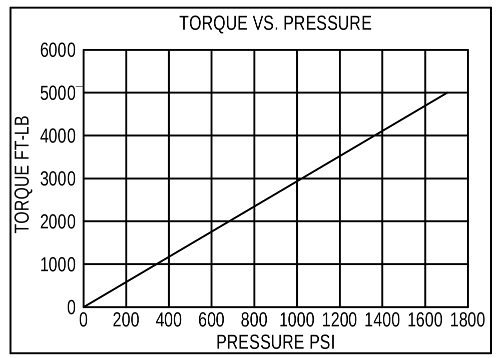


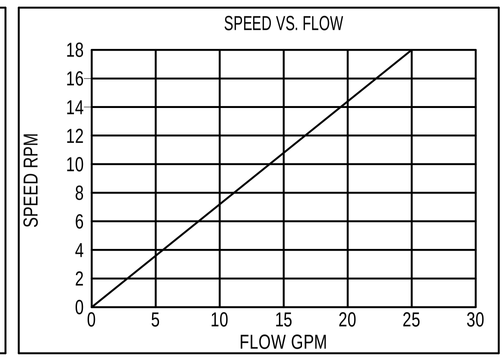
5016 With 54 Motor





5016 With 55 Motor





* Starting torques are approximately 10% less than running torque.
* Torque and speed charts are based on analytical calculations that use average gearbox and motor efficiencies.
* System back pressure will reduce running torque proportionally.
* Consult Eskridge Representative for Torque and Speed Charts for your specific application.

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Drive Head Performance Specifications

Customer: Cantsink   Sales Person: Kevin Schumm   Date: 1/13/2017

Drive Head Model				
50-16				
Gearbox Low Ratio	Low Ratio Efficiency	Gearbox High Ratio	High Ratio Efficiency	Rated Output Torque (ft-lbs)
16.65	0.95	0.00	0.00	5,000

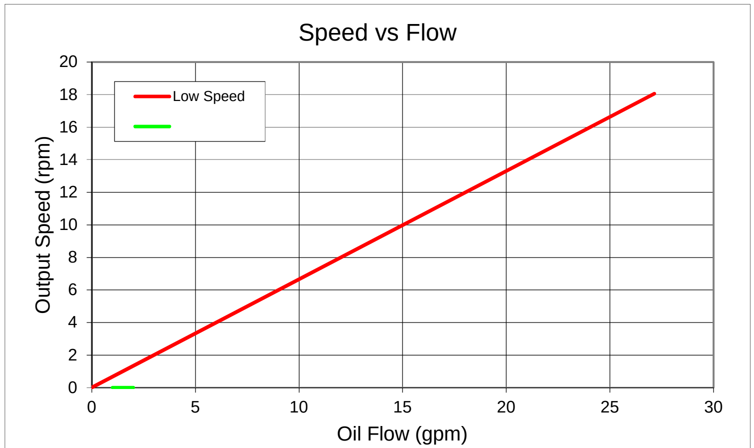
Maximum Motor Flow		
Set to ZERO for Calculated Maximum	0.0	gpm

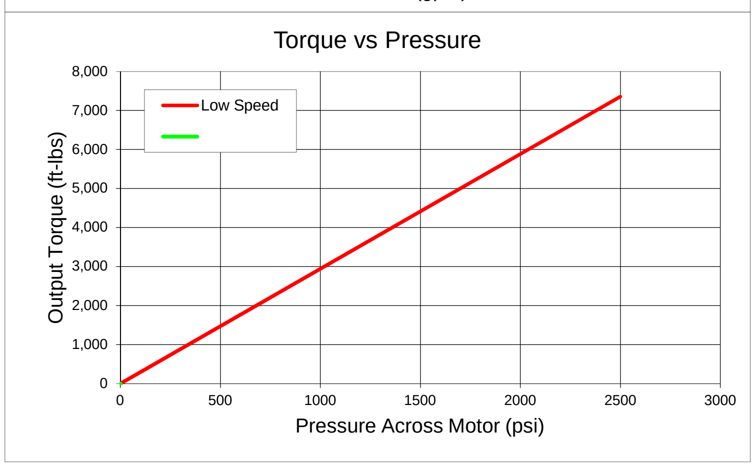
Motor Description and Information								
Code 55 - ROSS MB18 MTR TF280AS010AAAB								
Motor Part Number	Displacement, Low Speed (in^3)	Mechanical Efficiency, Low Speed	Displacement, High Speed (in^3)	Mechanical Efficiency, High Speed	Maximum Rated Pressure (psi)	Maximum Rated Speed, Low Speed (rpm)	Maximum Rated Speed, High Speed (rpm)	Maximum Rated Oil Flow (gpm)
01-304-0550	17.10	0.82	0.00	0.00	3000	330		0

Low Speed			
Pressure (psi)	Torque (ft-lbs)	Pressure (psi)	Torque (ft-lbs)
0	0		
2500	7,354		
Flow (gpm)	Speed (rpm)	Flow (gpm)	Speed (rpm)
0	0		
27.1	18.1		

50-16 Motor Code 55			
Pressure (psi)	Torque, Low Speed (ft-lbs)	Torque, High Speed (ft-lbs)	Torque, Low Speed (in-lbs)
0	0	N/A	0
0	0		0
100	294		3,530
200	588		7,060
300	882		10,590
400	1,177		14,120
500	1,471		17,650
600	1,765		21,180
700	2,059		24,710
800	2,353		28,240
900	2,647		31,770
1000	2,942		35,299
1100	3,236		38,829
1200	3,530		42,359
1300	3,824		45,889
1400	4,118		49,419
1500	4,412		52,949
1600	4,707		56,479
1700	5,001		60,009
1800	5,295		63,539
1900	5,589		67,069
2000	5,883		70,599
2100	6,177		74,129
2200	6,472		77,659
2300	6,766		81,189
2400	7,060		84,719
2500	7,354		88,249

Highlighted values exceed Rated Output Torque





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Drive Head Performance Specifications

Customer: Cantsink   Sales Person: Ralph   Date: 1/4/2017

Drive Head Model				
50-16				
Gearbox Low Ratio	Low Ratio Efficiency	Gearbox High Ratio	High Ratio Efficiency	Rated Output Torque (ft-lbs)
16.65	0.95	0.00	0.00	5,000

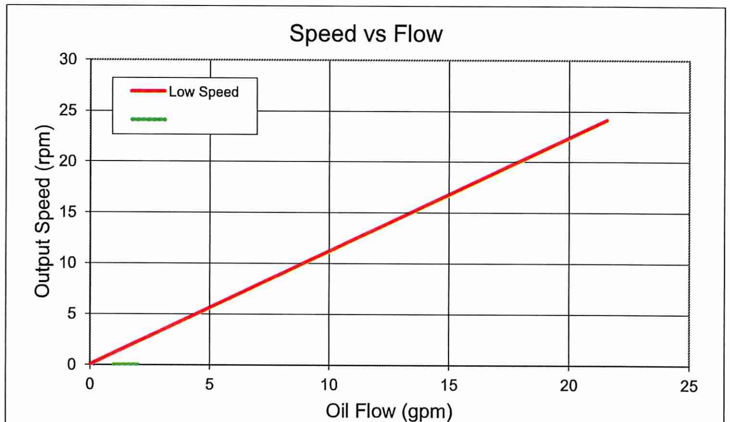
Maximum Motor Flow		
Set to ZERO for Calculated Maximum	0.0	gpm

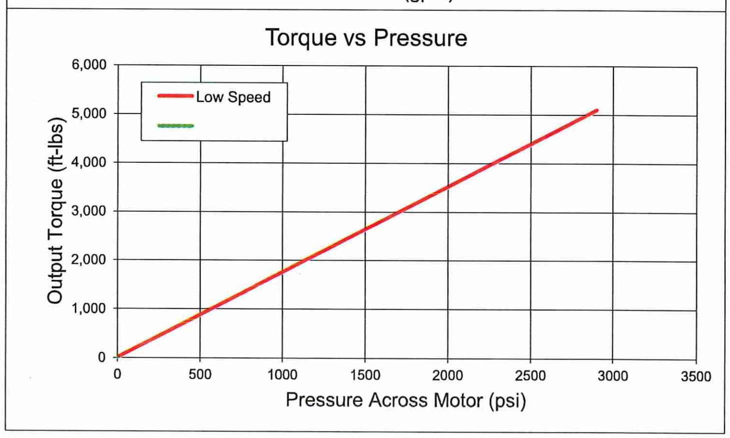
Motor Description and Information								
Code 54 - ROSS MB10 MTR -TF0170AS010AAAB								
Motor Part Number	Displacement, Low Speed (in^3)	Mechanical Efficiency, Low Speed	Displacement, High Speed (in^3)	Mechanical Efficiency, High Speed	Maximum Rated Pressure (psi)	Maximum Rated Speed, Low Speed (rpm)	Maximum Rated Speed, High Speed (rpm)	Maximum Rated Oil Flow (gpm)
01-304-0540	10.20	0.82	0.00	0.00	3000	440		0

Low Speed			
Pressure (psi)	Torque (ft-lbs)	Pressure (psi)	Torque (ft-lbs)
0	0		
2900	5,088		
Flow (gpm)	Speed (rpm)	Flow (gpm)	Speed (rpm)
0	0		
21.6	24.1		

50-16 Motor Code 54			
Pressure (psi)	Torque, Low Speed (ft-lbs)	Torque, High Speed (ft-lbs)	Torque, Low Speed (in-lbs)
0	0	N/A	0
400	702		8,422
500	877		10,528
600	1,053		12,633
700	1,228		14,739
800	1,404		16,845
900	1,579		18,950
1000	1,755		21,056
1100	1,930		23,161
1200	2,106		25,267
1300	2,281		27,373
1400	2,457		29,478
1500	2,632		31,584
1600	2,807		33,689
1700	2,983		35,795
1800	3,158		37,900
1900	3,334		40,006
2000	3,509		42,112
2100	3,685		44,217
2200	3,860		46,323
2300	4,036		48,428
2400	4,211		50,534
2500	4,387		52,640
2600	4,562		54,745
2700	4,738		56,851
2800	4,913		58,956
2900	5,088		61,062

Highlighted values exceed Rated Output Torque





Model T20K

Pro-Dig "T" Series are a Two Speed shift on the fly unit giving exceptional productivity benefits They are available in the following options.

1. **Manual Switching** - Controlled remotely by cab mounted switch
2. **Auto Kick down** - The unit always starts in a High Speed-Low Torque Mode as the system builds pressure generated by ground conditions the unit will automatically switch into a High Torque - Low Speed Mode. The Kick down system can be adjusted to suit the operators preference.

T20K	
Motor Mount	SAE "C" 4 Bolt
Motor	227.8cc Bidirectional Speed
Min. Hydraulic Flow	35 GPM
Max. Hydraulic Flow	75 GPM
Max. Continuous Pressure	2500 PSI
Unit Weight	678 Lbs.
Output Shaft	3" Hex

OUTPUT SPEED (LOW SPEED)

FLOW GPM (LPM)	SPEED RPM
35 (133)	10.1
40 (151)	11.5
45 (170)	13
50 (189)	14.4
60 (227)	17.2
65 (246)	18.6
70 (265)	20
75 (284)	21.4

OUTPUT SPEED (HIGH SPEED)

FLOW GPM (LPM)	SPEED RPM
35 (133)	20.1
40 (151)	23
45 (170)	25.9
50 (189)	28.8
60 (227)	34.4
65 (246)	37.3
70 (265)	40.2
75 (284)	43.1

OUTPUT TORQUE (LOW TORQUE)

PRESSURE PSI	TORQUE FT/LBS (Nm)
1000	4049 (5490)
1250	5061 (6863)
1500	6073 (8235)
1750	7085 (9608)
2000	8098 (10981)
2250	9110 (12353)
2375	9616 (13039)
2500	10122 (13725)

OUTPUT TORQUE (HIGH TORQUE)

PRESSURE PSI	TORQUE FT/LBS (Nm)
1000	8098 (6578)
1250	10123 (8222)
1500	12147 (9868
1750	14172 (11511)
2000	16196 (13156)
2250	18221 (14800)
2375	19233 (26080)
2500	20245 (27452)

Reliability Under Pressure

Model X12K5

X12K5	
Motor Mount	SAE "CC" 4 Bolt
Motor	400.0cc Bidirectional Speed
Min. Hydraulic Flow	10 GPM
Max. Hydraulic Flow	40 GPM
Max. Continuous Pressure	3000 PSI
Unit Weight	366 Lbs.
Output Shaft	2 1/2" Hex

OUTPUT SPEED

FLOW GPM (LPM)	SPEED RPM
10 (38)	5.9
12 (45)	7.1
15 (57)	8.8
17 (64)	10
20 (76)	11.8
22 (83)	12.9
25 (96)	14.7
27 (102)	15.9
30 (114)	17.6
32 (121)	18.8
35 (133)	20.5
40 (151)	23.5

OUTPUT TORQUE

PRESSURE PSI (BAR)	TORQUE FT/LBS (Nm)
1000 (69)	4204 (5701)
1200 (83)	5045 (6841)
1400 (96)	5886 (7981)
1600 (110)	6726 (9121)
1800 (124)	7567 (10261)
2000 (138)	8408 (11401)
2200 (152)	9249 (12541)
2400 (166)	10090 (13681)
2600 (179)	10930 (14822)
2800 (193)	11771 (15962)
3000 (207)	12612 (17102)

Note: 1 Ftlb = 1.356 Nm

Reliability Under Pressure

Section 14
Sample Load Test

August 09, 2019

Mr. Travis Bedson
CNT Foundations, LLC
3362 Navajo Street
North Charleston SC 29405
Tel: (843)-577-7268
Cell: (843)-343-9068

Subject: Static Axial Compressive and Lateral Load Test Report
8 Franklin Street, Charleston, South Carolina 29401

Mr. Bedson,

At your request The Foundation Firm, LLC witnessed the compression test of one Cantsink Manufacturing 2.5″ SCH 40 round shaft helical pile on Tuesday August 06, 2019 at the above referenced address. This pile was installed seven days prior to our visit to allow for the grout to properly cure.

The tested helical pile consisted of an eight-inch diameter partially grouted pile (we were not present during the installation of the pile and placement of the grout). The helical pile itself consisted of a 2.5″ SCH 40 round shaft with a 14″ helix and subsequent 2.5″ round shaft seven-foot extensions thereafter. The final embedment depth of the pile was around 70 feet at the tip of the pile. We understand the final installation torque of the helical pile was 4,200 ft-lb according to the installation log of the contractor, using a pressure gauge yielding 2,400 psi and a drive head motor manufactured by Eskridge (model 5016-21F54). The purpose for this test was to evaluate the capacity of this particular pile at this particular site and in these particular soil conditions.
For this compression test a set of four reaction piles were installed to a maximum depth of 100 feet. Steel beams were set up to create a frame to be able to apply load to the test pile. The test helical pile was monitored for deflection (pile movement) under axial compressive load. Pile deflection was measured with two dial gauges.
Static Axial Compressive Load Test Monitoring

One static axial compression test was performed on August 06, 2019. Procedure for the testing was based on guidelines of ASTM D1143-07 quick load test procedure section 8.1.2. A target test load of 100 kips was used for the test pile. The test load was determined using a safety factor of 2.0 of the design service load capacity of 50 kips. The pile was preloaded to 2 kips to provide proper alignment to the reaction beams and take the all the gaps out of the reaction frame (alignment load), a start reading was taken at this load and was used as the initial reading on the dial gauges.

THE FOUNDATION FIRM

71 First Avenue NW
Lilburn, GA 30047
Office: (770) 564-6949
Fax: (678) 280-3476

The test pile was then loaded in load increments of 5.0 kips at a time (approximate 5% of the target load). Each load interval was kept constant for a time interval of 5 minutes, with readings being recorded at beginning and at the end of the holding time. At the request of Perryman Engineering the pile was loaded to 50 kips, the load was maintained for 30 minutes and readings were taken every five minutes. At the end of the thirty minutes the pile was unloaded in five equal decrements to determine an amount of rebound. Once the pile was completely unloaded a new cycle of loads were created starting with 25 percent of the expected ultimate load up to the design load. After the design load the pile was loaded in increments of 5 kips to the expected ultimate load of 100 kips with holding times of 5 minutes per load increments. After the final load, the pile was unloaded in five equal decrements to determine rebound on this new loading cycle.

A digital load cell was used to determine the applied loads. See Exhibit III for calibration sheet of this equipment. Two three-and-half-inch continuous reading dial indicators, AGD 4, dial gauge with 0.001-inch accuracy were used to measure pile head deflection under the applied loads.
The comprehensive pile load test results are presented graphically and tabulated as an attachment on Exhibit I, while a summary of the test results including pile embedment are presented below.

PILE NUMBER	NET PILE HEAD DEFLECTION (IN.) AT 50 kips*	NET PILE HEAD DEFLECTION (IN.) AT 100 kips	PILE EMBEDMENT (FT)
Test Pile	0.477	1.05	70’-0”

* As identified on the load vs. deflection curve.

Static Lateral Load Test Monitoring

One static lateral test was performed on August 06, 2019. Procedure for the testing was based on guidelines of ASTM D3966-07 standard procedure. The test was conducted with modified loading times. The pile was loaded in increments of 1 kips and load was maintained for ten minutes, readings were taken at the beginning and at the end of the holding time.
Final load was 4,500 lbs. as the dial indicator ran out of stroke and no further reading were able to be taken. At this point the pile had experienced movement in excess of 3.5” at grade. See attached picture for lateral load testing setup and graph of load versus deflection curve.

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Lilburn, GA 30047
Office: (770) 564-6949
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The setup consisted of 3/8" plate that serve as a ring to hug the pile to be tested, a 5 ton hydraulic pull cylinder, a 10 kip capacity tension load cell, a vertical pile to tie a chain to the load cell and also a chain to a battered helical pile to keep the vertical pile straight and allow the lateral load to be horizontal and perpendicular to the test pile.

Evaluation and Recommendations

The Foundation Firm has no review structural drawings for this project. At this point allowable pile deflection by the structural engineer is not known. Based on our on-site observations and the tabulated data provided in Exhibit I, the pile performed well as maximum axial deflection didn't reach what we would call excessive deflection but the engineer of record should review the data presented on this report and evaluate if this would satisfy design requirements.
The lateral load test presented significant lateral movement, the design lateral loads are unknown at this time, the engineer of record should review these results and evaluated the adequacy of the lateral capacity.

The allowable axial deflection of helical piles is typically accepted as ten percent of the diameter of the helix per guidelines of AC358 "Acceptance Criteria for Helical Piles Systems and Devices".

The production piles will need to have similar final installation torques and minimum embedment depths to perform similarly under these loading conditions.

We appreciate the opportunity to be of service to you on this project and trust that you will call this office with any questions that you may have.

Respectfully Submitted,

THE FOUNDATION FIRM

Philip E Slemons

Philip E Slemons, P.E.
Senior Engineer

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 27217 PHILIP E SLEMONS
Philip E Slemons 8/9/19

Lester Jonis Salinas

Lester Salinas
Project Engineer

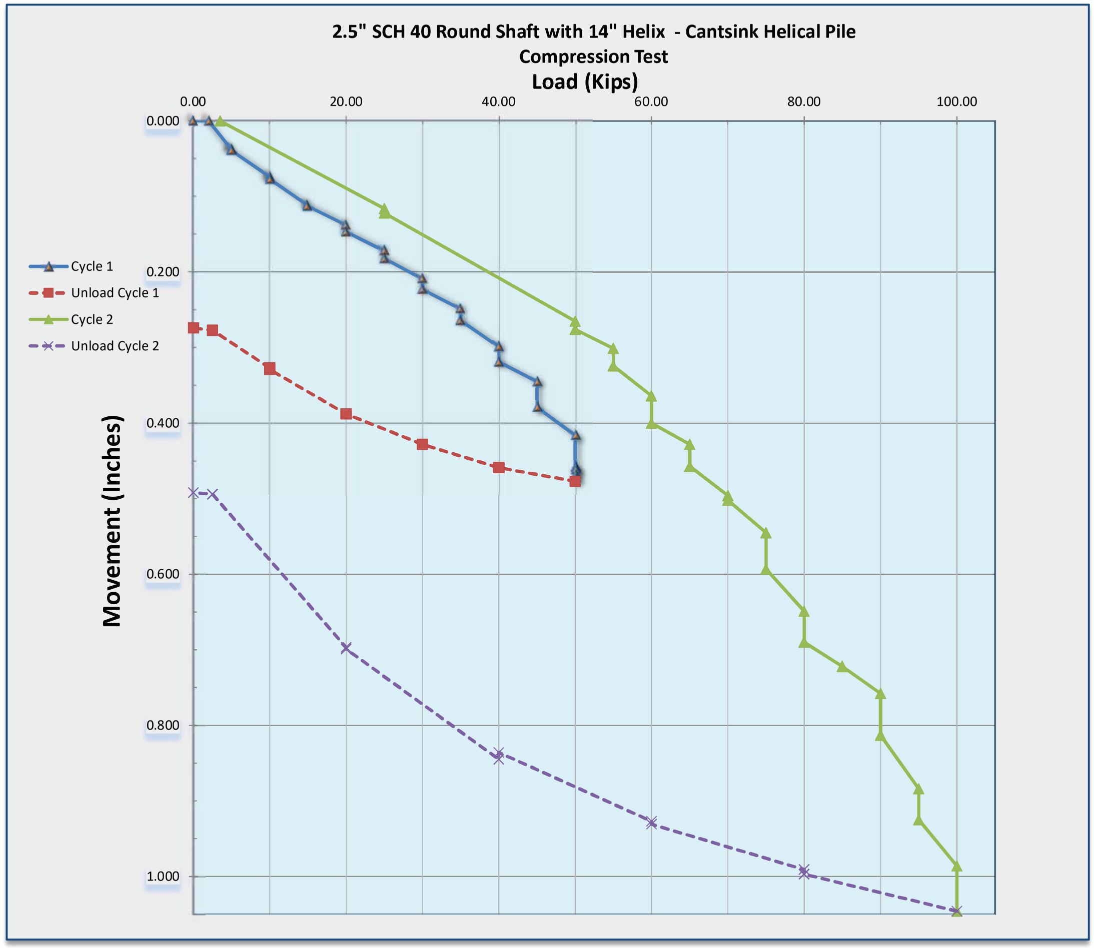
Exhibit I

2.5" Round Shaft Cantsink Manufacturing Helical Pile Compression Load Test
Charleston - South Carolina

Project Name: 8 Franklin Street
Customer: CNT Foundations
Test Site: 8 Franklin Street
Date Tested: 08/06/2019

Coordenates:	N	32.776701
	W	-79.937434

Load (kips)	Gage No. 1 (Inches)	Gage No. 2 (Inches)	Load (kips)	Gage No. 1 (Inches)	Gage No. 2 (Inches)	Net Deflection (Inches)	
0.00	First Cycle			Second Cycle		Cycle 1	Cycle 2
2.00	2.980	2.999	3.50	2.722	2.862	0.000	0.000
5.00	2.942	3.007	25.00	2.607	2.746	0.038	0.116
5.00	2.941	3.010	25.00	2.591	2.740	0.039	0.122
10.00	2.906	3.002	50.00	2.461	2.597	0.074	0.265
10.00	2.903	3.002	50.00	2.450	2.586	0.077	0.276
15.00	2.868	2.983	55.00	2.433	2.561	0.112	0.301
15.00	2.867	2.981	55.00	2.412	2.538	0.113	0.324
20.00	2.842	2.959	60.00	2.380	2.498	0.138	0.364
20.00	2.833	2.951	60.00	2.380	2.462	0.147	0.400
25.00	2.808	2.927	65.00	2.311	2.434	0.172	0.428
25.00	2.798	2.919	65.00	2.284	2.405	0.182	0.457
30.00	2.771	2.894	70.00	2.245	2.366	0.209	0.496
30.00	2.757	2.881	70.00	2.228	2.360	0.223	0.502
35.00	2.731	2.854	75.00	2.196	2.317	0.249	0.545
35.00	2.716	2.841	75.00	2.145	2.269	0.264	0.593
40.00	2.681	2.807	80.00	2.099	2.213	0.299	0.649
40.00	2.661	2.797	80.00	2.045	2.172	0.319	0.690
45.00	2.635	2.769	85.00	2.012	2.140	0.345	0.722
45.00	2.601	2.752	85.00	2.010	2.140	0.379	0.722
50.00	2.564	2.713	90.00	1.984	2.104	0.416	0.758
50.00	2.522	2.688	90.00	1.923	2.049	0.458	0.813
50.00	2.521	2.679	95.00	1.845	1.978	0.459	0.884
50.00	2.518	2.675	95.00	1.816	1.937	0.462	0.925
50.00	2.512	2.670	100.00	1.753	1.876	0.468	0.986
50.00	2.507	2.667	100.00	1.691	1.816	0.473	1.046
50.00	2.503	2.661	80.000	1.744	1.865	0.477	0.997
40.00	2.521	2.681	80.000	1.753	1.871	0.459	0.991
40.00	2.521	2.682	60.000	1.811	1.931	0.459	0.931
30.00	2.552	2.715	60.000	1.815	1.935	0.428	0.927
30.00	2.552	2.718	40.000	1.845	2.026	0.428	0.836
20.00	2.592	2.761	40.000	1.861	2.017	0.388	0.845
10.00	2.651	2.821	20.000	2.006	2.163	0.329	0.699
10.00	2.653	2.825	20.000	2.018	2.165	0.327	0.697
2.50	2.703	2.876	2.500	2.221	2.368	0.277	0.494
0.00	2.706	2.879	0.000		2.370	0.274	0.492



Installation Information

Depth of pile (ft) = 70'-0"
Installation Torque (ft-lb) = 4,200 ft-lb
Torque measuring device : Pressure Gage

Specimen Specification:

Helix Diameter (in) = 14"
Helix Thickness (in) = 3/8"
Shaft = 2.5 Round Shaft

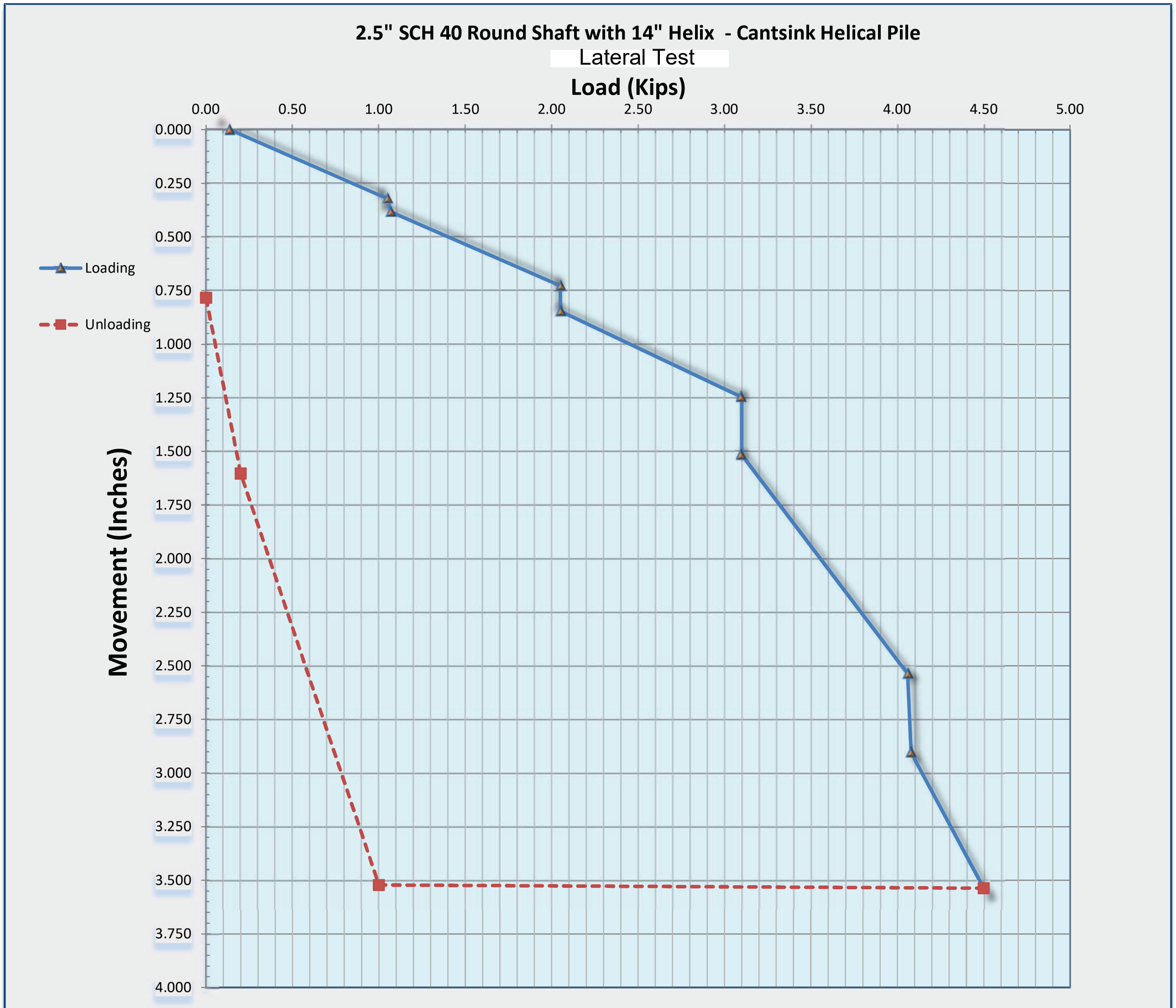
Exhibit II

2.5" Round Shaft Cantsink Manufacturing Helical Pile Lateral Load Test
Charleston - South Carolina

Project Name: 8 Franklin Street
Customer: CNT Foundations
Test Site: 8 Franklin Street
Date Tested: 08/06/2019

Coordenates:	N	32.776701
	W	-79.937434

Load (kips)	Gage No. 1 (Inches)	Net Deflection (Inches)
0.00		
0.14	3.540	0.000
1.05	3.218	0.322
1.07	3.157	0.383
2.05	2.812	0.728
2.05	2.694	0.846
3.10	2.293	1.247
3.10	2.024	1.516
4.06	1.003	2.537
4.08	0.636	2.904
4.50	0.003	3.537
1.00	0.018	3.522
0.20	1.937	1.603
0.00	2.757	0.783



Installation Information

Depth of pile (ft) = 70'-0"
Installation Torque (ft-lb) = 4,200 ft-lb
Torque measuring device : Pressure Gage

Specimen Specification:

Helix Diameter (in) = 14"
Helix Thickness (in) = 3/8"
Shaft = 2.5 Round Shaft

COMPRESSION TEST SETUP

GAGES SETUP

LATERAL LOAD TEST SETUP

READOUT AT DESIGN LOAD

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Acct. No:	**CN001** **Project No: 191096**	**Date Sampled:**	**02/18/2019**
Report Date:	**02/18/2019**	**Sampled By:**	**Henderson, III, P.E., A. T**
Project:	**Palmetto Plains Solar Farm 115-34.5kV Substation, Mohawk Street, State Road S-38-258, Bowman, SC**	**By Order Of:**	**Client**
Location:		**Order Number:**	
Client:	**CNT FOUNDATIONS**		
REPORT:	**Pile Load Test Report**	**LAB NO:**	**54592-1**
		Test Method:	**See Below**

TEST RESULTS **Report No: 54592-1**

The following items are included as attachments:

-

Orig: CNT FOUNDATIONS Attn: Mr. Travis Bedson
E-Mail: travis@yourfoundationexperts.com
(1-ec copy)

Respectfully Submitted,
SOIL CONSULTANTS, INC.

A. Talbot Henderson III

A. Talbot Henderson, III, P.E.
Project Engineer

SCI
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February 20, 2019

CNT Foundations
3362 Navajo Street
North Charleston, SC 29405

Attention: Mr. Travis Bedson

Reference: Helical Pier Load Tests
Palmetto Plains Solar Farm 115-34.5kV Substation
Mohawk Street, Bowman, SC

SCI Project 191096

Dear Mr. Bedson:

As requested, a representative of Soil Consultants, Inc. (SCI) visited the project site on February 13, 2019, to observe static load testing of two helical piers for the referenced project. This report includes a summary of our observations and test results. Our services were performed in general accordance with SCI Proposal No. 12-19-066, dated February 12, 2019.

Project Information

SCI is not the design engineer for the helical piers, and SCI representatives were not present during installation of the test piers. Project information was provided by CNT Foundations (CNT) and representatives of Terracon, who we understand is providing geotechnical engineering and quality assurance testing services for this project. The approximate test location is shown on the attached Test Pier Location Plan drawing. The two test piers, designated TP-5 and TP-8, were located adjacent to each other at the general location shown on the drawing. We were not informed if the test pier locations were production pier locations.

We understand the test piers have design (allowable) compressive loads of 43 kips. Test Pier TP-5 was installed to a penetration depth of 47 feet below the ground surface, and Test Pier TP-8 was installed to a penetration depth of 60 feet. We understand both piers were installed on a previous date, but the installation date was not provided. We understand both piers have 1½-inch (nominal) solid square galvanized steel shafts with 8-inch (nominal) diameter grout columns. In addition, we understand the piers have a triple helix configuration with helix diameters of 10 inches, 12 inches, and 14 inches. The installation torque measurements for the test piers were not provided.

We were informed that the reaction piers for both tests included 2⅞-inch diameter pipe shafts. We were not informed of the helix configurations for the reaction piers. The reaction piers were not grouted. We understand the reaction piers for Test Pier TP-5 were installed to depths of approximately 42 feet and reaction piers for Test Pier TP-8 were installed to depths of approximately 35 feet.

Load Test Observations

The reaction system and the load application system including a hydraulic jack and a digital load cell was setup by CNT. Calibration information for the load cell, Model 3000-200-2 manufactured by Geokon, was provided, and the calibration date was August 13, 2018. A photograph of the provided calibration sheet is attached to this report. The deflection measurement system, consisting of a reference beam, two dial gauges, and a wire and scale system, as described in the ASTM D1143 test procedure, was set up cooperatively by CNT, SCI, and the Terracon representative.

Both piers were tested to a compressive load of 50 kips in 2.5-kip load increments. Then, the piers were unloaded in decrements of approximately 10 kips. Load increments and decrements were maintained for 4 minutes each, except for the 50-kip maximum load which was maintained for 20 minutes for Test Pier TP-5 and for 15 minutes for Test Pier TP-8.

Table 1 includes a summary of the measured pier top deflections with compressive loads of 42.5 kips (near the design load of 43 kips) and the maximum test load of 50 kips. The individual deflection measurements along with charts showing the load-deflection “curves” are attached. We note that the load was fully removed for the final rebound point for Test Pier TP-5. When the jack lost contact with the reaction beam, the jack/ram and steel bearing plate tilted considerably. Because the ram and bearing plate were the deflection measurement references, we believe the deflection data may not be representative for the final rebound point, and we do not recommend using this point for evaluation of the load test data. Similarly, the wire & scale measurement for the final rebound point for Test Pier TP-8 is unusual and possibly erroneous, so we would recommend using the dial gauge measurements for evaluation of that test data.

Table 1 – Summary of Pier Top Load-Deflection Response

Pier Designation	Pier Depth	Pier Top Deflection at 42.5 kips*	Pier Top Deflection at 50 kips*
TP-5	47 feet	0.19 inch	0.26 inch
TP-8	60 feet	0.36 inch	0.39 inch

*Average of two dial gauges

The tests were performed in general accordance with the ASTM D1143 Procedure A: quick test, with the exception of the following item. The ASTM test procedure states that spacing of reaction piles should be at least 8 feet from the test pile to avoid interaction between the piles, and the reaction piers for this load test were spaced at approximately 5 feet. However, because the depths of the helices of the reaction piers were shallower than the test piers and because the reaction piers were not grouted, we believe there would be no significant interaction between the piers for these load tests, and we recommend that this discrepancy be accepted for this project.

Conclusion

SCI is not aware of the load-deflection performance requirements for this project. Therefore, we recommend that the project structural engineer evaluate the load test results for acceptance.

We appreciate the opportunity to be of service to you on this project. If we may be of further assistance, please call.

Sincerely:

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 29039 ALVIN TALBOT HENDERSON III

2/20/2019

A. Talbot Henderson, III, P.E.

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 17234 RONALD R. AUSTIN

2/20/2019

Ronald R. Austin, P.E.

SOUTH CAROLINA SOIL CONSULTANTS, INC. No. 124 CERTIFICATE OF AUTHORIZATION

ath

Enclosures

Reference: Base map obtained from GoogleMaps.com
Note: Locations are approximate.

Legend: Test Pier Location

TEST PIER LOCATION PLAN

Palmetto Plains Solar Farm 115-34.5kV Substation
Mohawk Street
Bowman, SC

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Scale	Not to scale	Project No. 191096	
Date	2/19/2019	Figure 1	

SOIL CONSULTANTS, INC.

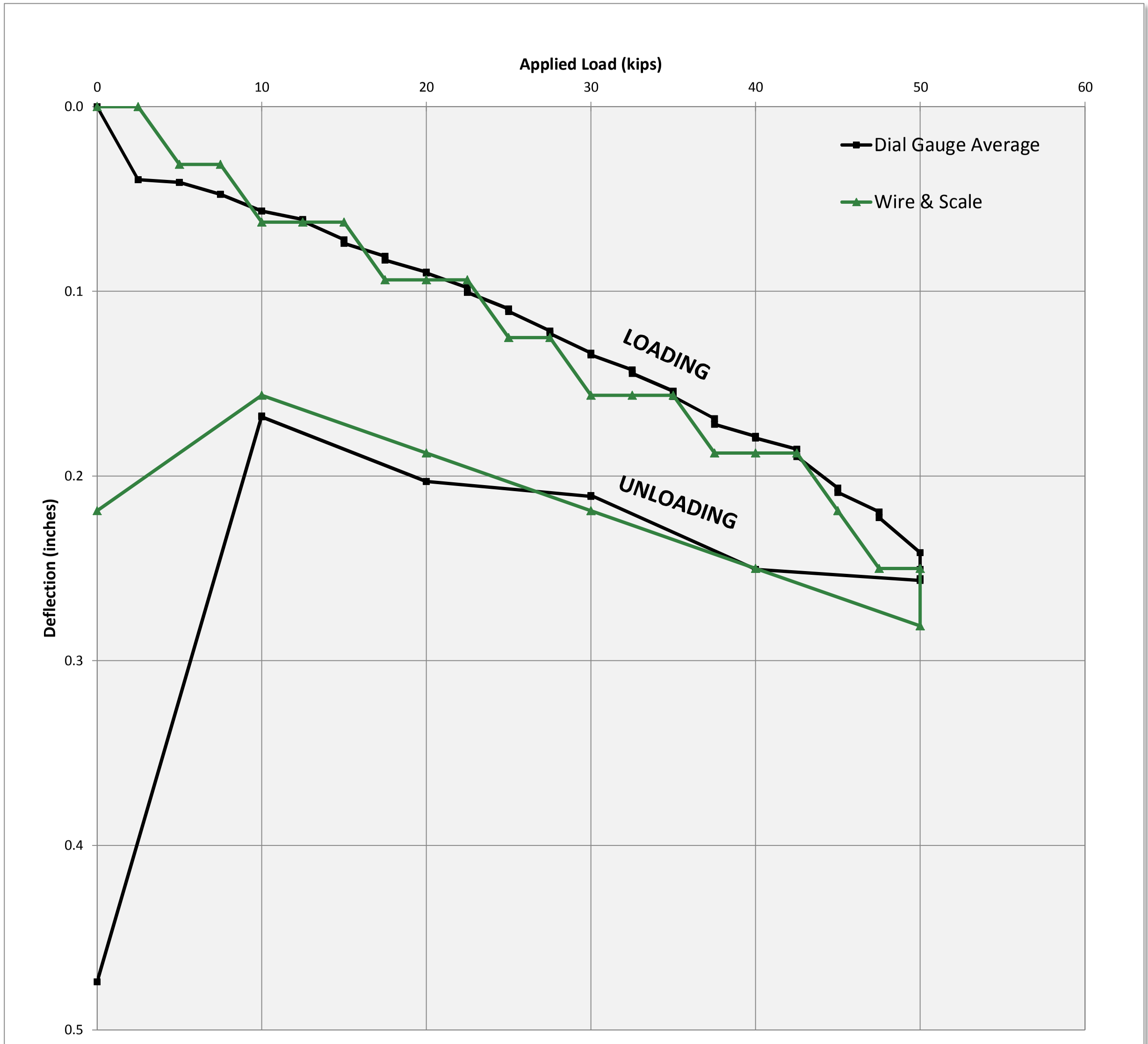
Helical Pier Static Load Test Results

Project: Palmetto Plains Solar Farm 115-34.5 kV Substation
Location: Bowman, SC
SCI Project No.: 191096
Client: CNT Foundations
Test Method: ASTM D1143 (Compression)
Test Date: February 13, 2019

Pier Details

Pier Designation: Test Pier TP-5
Installation Depth: 47 feet
Helix Diameter Configuration: 10", 12", & 14"
Shaft: 1.5-inch solid square (grouted)
Installation Torque: Not provided

Load (kips)	Deflection (inches)			
	Left Dial	Right Dial	Dial Avg.	Wire & Scale
0	0.000	0.000	0.000	0.000
2.5	0.004	0.075	0.040	0.000
2.5	0.004	0.075	0.040	0.000
5	0.012	0.070	0.041	0.031
5	0.012	0.070	0.041	0.031
7.5	0.028	0.067	0.048	0.031
7.5	0.028	0.067	0.048	0.031
10	0.037	0.076	0.057	0.063
10	0.037	0.076	0.057	0.063
12.5	0.038	0.084	0.061	0.063
12.5	0.038	0.086	0.062	0.063
15	0.046	0.098	0.072	0.063
15	0.047	0.101	0.074	0.063
17.5	0.051	0.111	0.081	0.094
17.5	0.051	0.115	0.083	0.094
20	0.051	0.128	0.089	0.094
20	0.051	0.129	0.090	0.094
22.5	0.053	0.143	0.098	0.094
22.5	0.055	0.146	0.101	0.094
25	0.061	0.158	0.110	0.125
25	0.061	0.161	0.111	0.125
27.5	0.066	0.177	0.122	0.125
27.5	0.067	0.179	0.123	0.125
30	0.072	0.195	0.134	0.156
30	0.072	0.197	0.135	0.156
32.5	0.076	0.209	0.143	0.156
32.5	0.077	0.212	0.145	0.156
35	0.084	0.224	0.154	0.156
35	0.085	0.229	0.157	0.156
37.5	0.093	0.245	0.169	0.188
37.5	0.095	0.249	0.172	0.188
40	0.097	0.260	0.179	0.188
40	0.097	0.262	0.180	0.188
42.5	0.097	0.274	0.186	0.188
42.5	0.098	0.281	0.190	0.188
45	0.110	0.303	0.207	0.219
45	0.110	0.308	0.209	0.219
47.5	0.113	0.326	0.220	0.250
47.5	0.115	0.330	0.223	0.250
50	0.125	0.358	0.242	0.250
50	0.133	0.368	0.251	0.250
50	0.139	0.372	0.256	0.281
50	0.140	0.373	0.257	0.281
40	0.140	0.361	0.251	0.250
40	0.140	0.361	0.251	0.250
30	0.106	0.315	0.211	0.219
30	0.107	0.315	0.211	0.219
20	0.110	0.296	0.203	0.188
20	0.110	0.296	0.203	0.188
10	0.084	0.252	0.168	0.156
10	0.083	0.252	0.168	0.156
0	0.383	0.565	0.474	0.219



SOIL CONSULTANTS, INC.

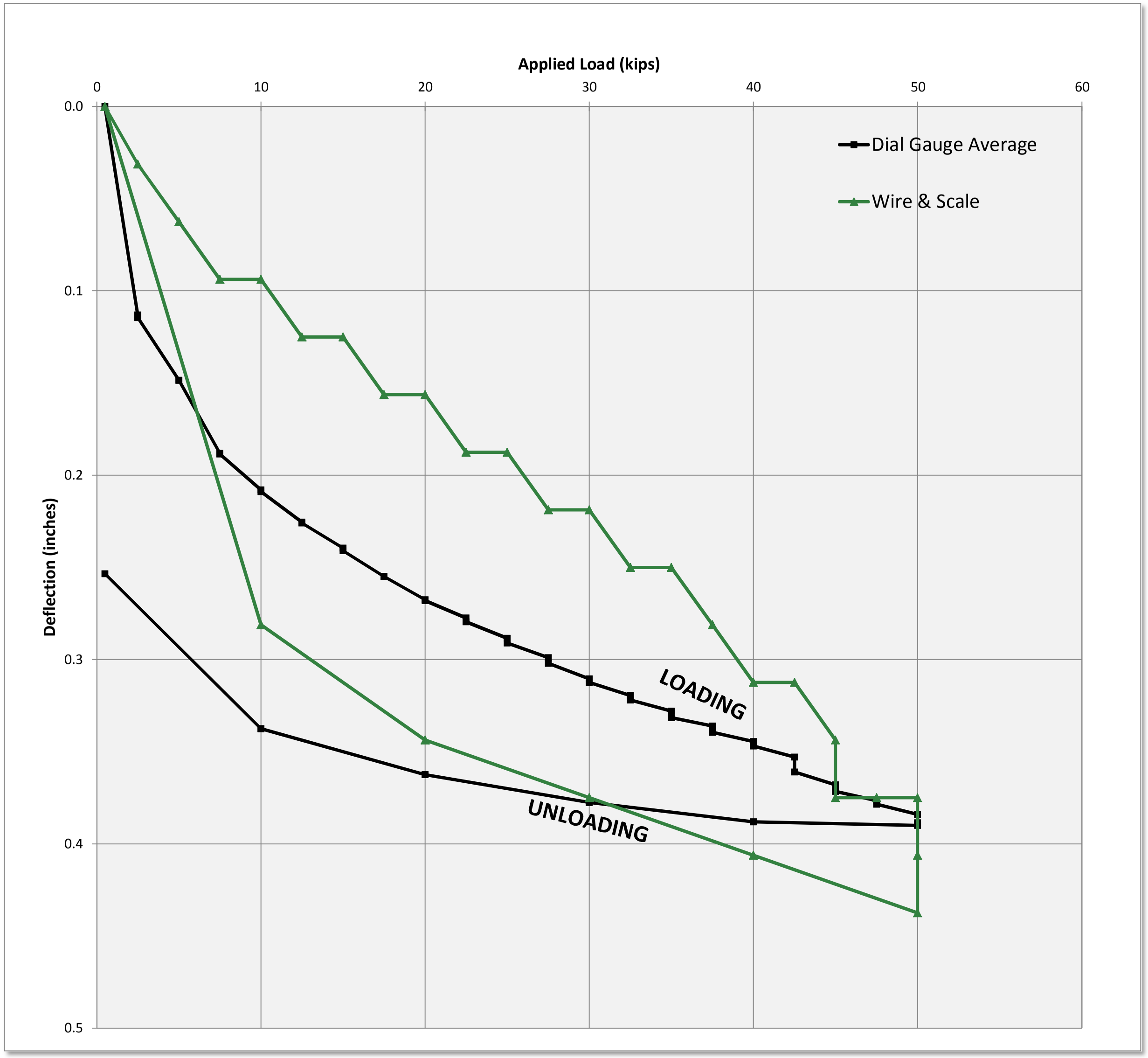
Helical Pier Static Load Test Results

Project: Palmetto Plains Solar Farm 115-34.5 kV Substation
Location: Bowman, SC
SCI Project No.: 191096
Client: CNT Foundations
Test Method: ASTM D1143 (Compression)
Test Date: February 13, 2019

Pier Details

Pier Designation:	Test Pier TP-8
Installation Depth:	60 feet
Helix Diameter Configuration:	10", 12", & 14"
Shaft:	1.5-inch solid square (grouted)
Installation Torque:	Not provided

Load (kips)	Deflection (inches)			
	Left Dial	Right Dial	Dial Avg.	Wire & Scale
0.5	0.000	0.000	0.000	0.000
2.5	0.128	0.098	0.113	0.031
2.5	0.129	0.100	0.115	0.031
5	0.170	0.127	0.149	0.063
5	0.170	0.127	0.149	0.063
7.5	0.214	0.162	0.188	0.094
7.5	0.215	0.162	0.189	0.094
10	0.241	0.175	0.208	0.094
10	0.242	0.176	0.209	0.094
12.5	0.263	0.188	0.226	0.125
12.5	0.263	0.189	0.226	0.125
15	0.282	0.197	0.240	0.125
15	0.285	0.197	0.241	0.125
17.5	0.303	0.207	0.255	0.156
17.5	0.303	0.207	0.255	0.156
20	0.324	0.211	0.268	0.156
20	0.325	0.211	0.268	0.156
22.5	0.343	0.212	0.278	0.188
22.5	0.347	0.212	0.280	0.188
25	0.363	0.214	0.289	0.188
25	0.366	0.216	0.291	0.188
27.5	0.381	0.217	0.299	0.219
27.5	0.387	0.217	0.302	0.219
30	0.404	0.217	0.311	0.219
30	0.408	0.217	0.313	0.219
32.5	0.422	0.217	0.320	0.250
32.5	0.427	0.217	0.322	0.250
35	0.440	0.216	0.328	0.250
35	0.447	0.216	0.332	0.250
37.5	0.458	0.214	0.336	0.281
37.5	0.465	0.214	0.340	0.281
40	0.477	0.212	0.345	0.313
40	0.482	0.212	0.347	0.313
42.5	0.496	0.210	0.353	0.313
42.5	0.515	0.207	0.361	0.313
45	0.533	0.203	0.368	0.344
45	0.542	0.201	0.372	0.375
47.5	0.554	0.199	0.377	0.375
47.5	0.560	0.197	0.379	0.375
50	0.575	0.193	0.384	0.375
50	0.587	0.190	0.389	0.406
50	0.591	0.188	0.390	0.406
50	0.593	0.187	0.390	0.438
40	0.585	0.191	0.388	0.406
40	0.585	0.191	0.388	0.406
30	0.568	0.187	0.378	0.375
30	0.568	0.187	0.378	0.375
20	0.543	0.182	0.363	0.344
20	0.543	0.182	0.363	0.344
10	0.497	0.178	0.338	0.281
10	0.497	0.178	0.338	0.281
0.5	0.345	0.162	0.254	0.000



GEOKON 48 Spencer St. Lebanon, NH 03766 USA

Load Cell Calibration Report

Model Number: 3000-200-2

Calibration Date: August 13, 2018

Serial Number: 1532440

This calibration has been verified/validated as of 08/13/2018

Max. Range (lbs): 200000

Calibration Instruction: CI-3000 Tinius

Cable Length: 15 feet

Technician: [signature]

Initial Cycling Data

Load (lbs):	0	0	300000	0
Reading:	2539	2532	22034	2534

Calibration

Applied Load in lbs	Readings from GK-501 or GK-502 readout box				Linearity % Max Load	Polynomial Error (%FS)
	Cycle 1	Cycle 2	Average	Change		
0	2534	2534	2534		-0.65	-0.56
20000	3910	3902	3906	1372	-0.07	-0.01
40000	5210	5220	5215	1309	0.02	0.04
60000	6497	6507	6502	1287	-0.06	-0.07
80000	7815	7810	7813	1311	0.04	0.01
100000	9115	9120	9118	1305	0.10	0.06
120000	10401	10407	10404	1286	0.02	-0.02
140000	11699	11704	11702	1298	0.02	-0.01
160000	12988	12999	12994	1292	-0.02	-0.03
180000	14288	14301	14295	1301	0.01	0.03
200000	15577	15588	15583	1288	-0.07	0.00
0	2529	2529	2529			

GK-501 or GK-502 Readout

Linear Gauge Factor (G): 15.42 lbs/digit

Regression Zero (R_0):* 2618

Polynomial Gauge Factors: A: 0.000006026 B: 15.30 C: -39870

Polynomial, $L = AR_1^2 + BR_1 + C$

Full Scale mV/ V: 3.262 mV/ V

Calculate C by setting L=0 and R_1 = initial field zero reading in the polynomial equation

* Note: The above calibration uses a linear regression method. The Regression Zero Reading shown is ideal for straight line computation and does not usually agree with the actual no-load reading.

The above named instrument has been calibrated by comparison with standards traceable to the NIST, in compliance with ANSI Z540-1.

May 01, 2019

Mr. Travis Bedson
CNT Foundations, LLC
3362 Navajo Street
North Charleston SC 29405
Tel: (843)-577-7268
Cell: (843)-343-9068

Subject: Static Axial Compressive Test Report
Charleston Steel Apartments
Brigade Street, Charleston, South Carolina 29403

Mr. Bedson,

At your request The Foundation Firm, LLC provided instrumentation and witnessed the compression test of two Cantsink Manufacturing helical piles on Friday April 26th, 2019. One pile is a double helix round shaft, the second pile was a grouted 1.5″ solid square shaft helical pile. These two piles were installed on Monday April 22nd and we not present during the installation of these piles.

One of the tested helical piles consisted of a six-inch diameter grouted pile (we were not present during the installation of the pile and placement of the grout). The helical pile has a single 14″ helix and was installed to a maximum depth of 42 feet. The second pile consisted of a round shaft of 2.875″ O.D. and was installed with a lead section with two helices, 14″ and 12″ in diameter. The final embedment depth for this pile was 35 feet to the tip of the pile. It is our understanding according to the contractor that the final installation torques were 4,200 ft-lb and 5,000 ft-lb respectively. The purpose for this test was to evaluate the capacity of these particular piles at this particular site and in these particular soil conditions.
For these compression tests a set of four reaction piles were installed to a maximum depth of 42 feet. Steel beams were set up to create a frame to be able to apply load to the test pile. The test helical pile was monitored for deflection (pile movement) under axial compressive load. Pile deflection was measured with two dial gauges.

Static Axial Compressive Load Test Monitoring

Procedure for the testing was based on guidelines of ASTM D1143-07 quick load test procedure section 8.1.2. A target test load of 50 kips was used for the test pile. The test load was determined using a safety factor of 2.0 of the design service load capacity of 25 kips. The pile was preloaded to 1 kip in the case of the round shaft and 2 kips in the case of the grouted 1.5″ solid square shaft to provide proper alignment to the reaction beams and take the all the gaps out of the reaction frame (alignment load) a start reading was taken at this load and was used as the

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Lilburn, GA 30047
Office: (770) 564-6949
Fax: (678) 280-3476

initial reading on the dial gauges. The test pile was then loaded in load increments of 2.5 kips at a time (5% of the target load). Each load interval was kept constant for a time interval of 4 minutes, with readings being recorded at beginning and the end of the holding time.

Per guidelines of ASTM D1143-07 the intention was to hold the ultimate load for a period of time of twenty minutes but due excessive deflections this was not accomplished.

A digital load cell was used to determine the applied loads. See Exhibit IV for calibration sheet of this equipment. Two four-inch continuous reading dial indicators, AGD 4, dial gauge with 0.001-inch accuracy were used to measure pile head deflection under the applied loads.
The comprehensive pile load test results are presented graphically and tabulated as attachments on Exhibit II), while a summary of the test results including pile embedment are presented below.

PILE NUMBER	NET PILE HEAD DEFLECTION DL (IN.)*	NET PILE HEAD DEFLECTION UL (IN.)	PILE EMBEDMENT (FT)
Round Shaft	0.545	1.342	35′-0″
Square Shaft	0.342	1.387	42′-0″

* As identified on the load vs. deflection curve.
**DL = Design Load; UL = Ultimate Load

The test for the round shaft was performed up to 31 kips and the test was stopped as the dial gauges ran out-of-stroke. At that point the pile had experienced an excessive amount of deflection in comparison to what is considered acceptable, based on the guidelines of the Acceptance Criteria for helical piles.
The test for the grouted 1.5″ square shaft was tested up to 50 kips load but immediately after the pile plunged and was not able to take any additional load nor was able to maintain the 50 kips load. Failure started to develop right after the 47.5 kips load as can be seen on the load versus deflection curve.

THE FOUNDATION FIRM

71 First Avenue NW
Lilburn, GA 30047
Office: (770) 564-6949
Fax: (678) 280-3476

Evaluation and Recommendations

Marked up plans from Geotrack show that expected design load for the helical piles was 25 kips. There was not mentioned within the plans what was the expected allowable pile movement. In absence of this we defer to the Acceptance Criteria for Helical Piles Systems and Devices where the allowable deflection of helical piles is accepted as ten percent of the diameter of the helix or the average diameter of the helices.

Based on this criterion we can infer that the grouted pile achieves an ultimate load bearing capacity of 45 kips which with a safety factor of two yields a working load of 22.5 kips. For the round shaft the ultimate capacity was reached at 12.50 kips and with a safety of factor of two the working load is 6.25 kips. With this information the design engineer will need to consider an appropriate pile spacing should he or she decides to use these types of pile configuration or pretest an additional pile with greater embedment depth in order to gain additional bearing capacity.

The Foundation Firm, LLC appreciates the opportunity to be of service on this phase of the project. Please contact us if you have any questions concerning this letter or if we may be of additional service on this or other projects.

Respectfully Submitted,

THE FOUNDATION FIRM

Philip E Slemons

Philip E Slemons, P.E.
Senior Engineer

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 27217 PHILIP E SLEMONS

5/1/19

Lester Salinas
Project Engineer

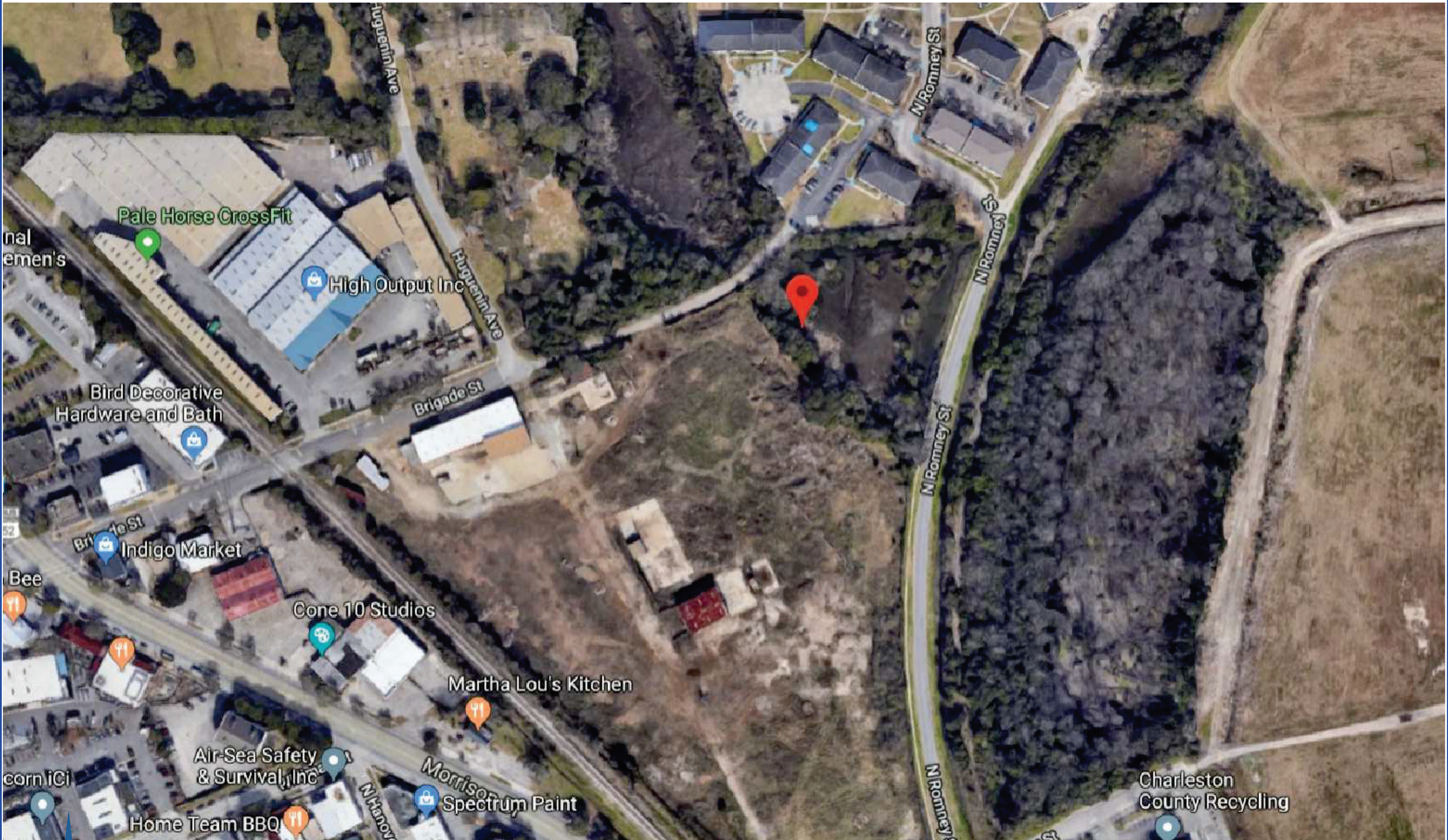


Image Courtesy of
Google Maps

No claim is made as to the accuracy of the indicated beam locations other than for conceptual purposes to illustrate the test locations relative to existing site features. In consideration of the method used in their determination, as well as the base map's accuracy, the test locations shown should be considered approximate.

THE FOUNDATION FIRM

71 First Avenue, NW
Lilburn, GA 30047
TEL 770-564-6949
FAX:678-280-3476

Description:	TEST PILE LOCATION PLAN
Customer:	CNT Foundations
Location:	Charleston Steel Apartments

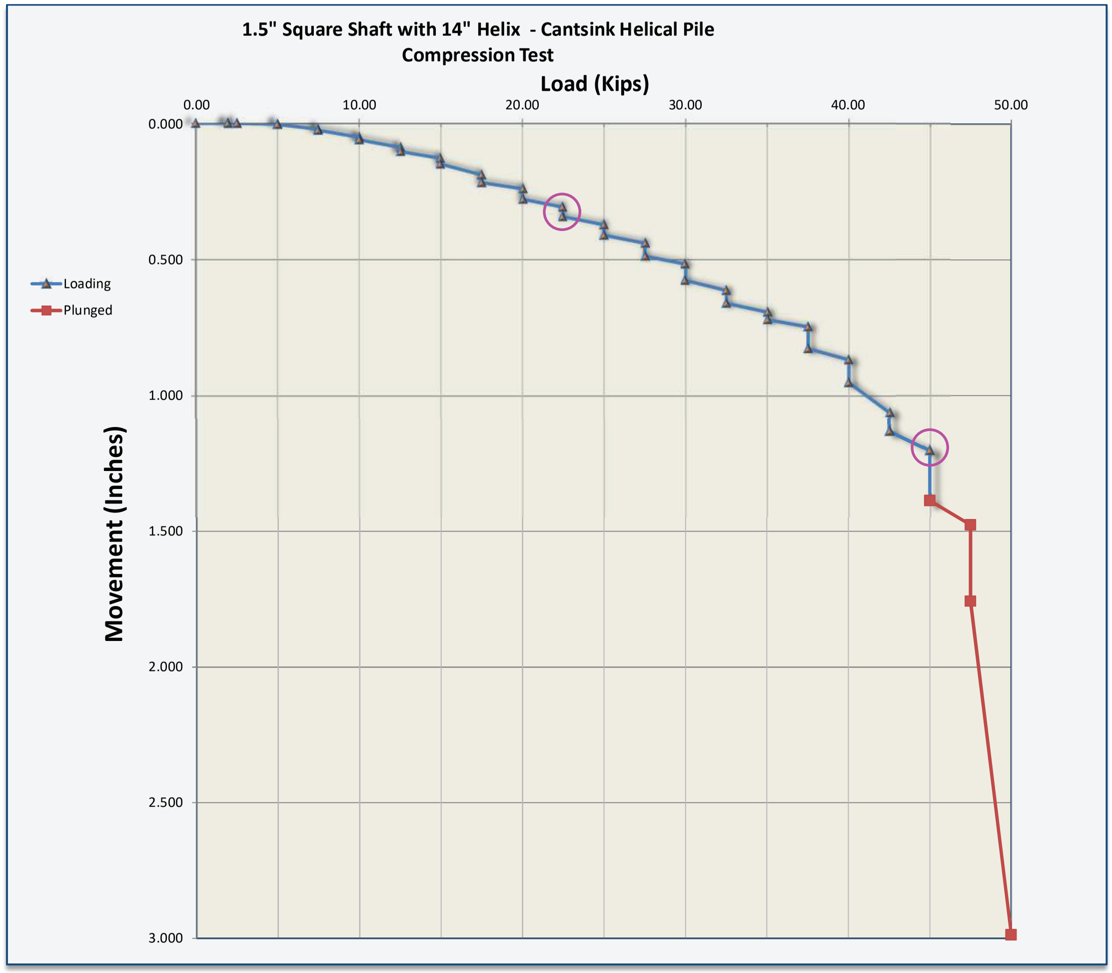
Check by:	Date:	Scale:	Job No. :	Sheet No. :
LESTER SALINAS	05/01/2019	N.T.S.	FF-19-041	EXHIBIT A-1

1.5" Square Shaft Cantsink Manufacturing Helical Pile Compression Load Test
Apartment Complex - Brigade St, Charleston, SC 29403

Project Name: Charleston Steel Apartments - Phase II
Customer: CNT Foundations
Test Site: Brigade Street, Charleston, SC 29403
Date Tested: 02/26/2019

Coordenates:		
	N	32.811487
	W	-79.943655

Load (kips)	Reading 1 (Inches)	Reading 2 (Inches)	Net Deflection (Inches)	
0.00			0.000	
2.00	3.681	3.839	0.000	0.000
2.00	3.681	3.839	0.000	0.000
2.50	3.681	3.839	0.000	0.000
2.50	3.681	3.839	0.000	0.000
5.00	3.678	3.838	0.003	0.001
5.00	3.677	3.838	0.004	0.001
7.50	3.660	3.824	0.021	0.015
7.50	3.656	3.823	0.025	0.016
10.00	3.630	3.808	0.051	0.031
10.00	3.621	3.795	0.060	0.044
12.50	3.593	3.768	0.088	0.071
12.50	3.578	3.755	0.103	0.084
15.00	3.553	3.730	0.128	0.109
15.00	3.532	3.714	0.149	0.125
17.50	3.492	3.664	0.189	0.175
17.50	3.464	3.643	0.217	0.196
20.00	3.441	3.611	0.240	0.228
20.00	3.403	3.583	0.278	0.256
22.50	3.374	3.548	0.307	0.291
22.50	3.339	3.521	0.342	0.318
25.00	3.309	3.484	0.372	0.355
25.00	3.271	3.452	0.410	0.387
27.50	3.242	3.418	0.439	0.421
27.50	3.194	3.374	0.487	0.465
30.00	3.165	3.335	0.516	0.504
30.00	3.105	3.278	0.576	0.561
32.50	3.068	3.231	0.613	0.608
32.50	3.021	3.186	0.660	0.653
35.00	2.989	3.138	0.692	0.701
35.00	2.961	3.095	0.720	0.744
37.50	2.935	3.026	0.746	0.813
37.50	2.855		0.826	
40.00	2.814		0.867	
40.00	2.730		0.951	
42.50	2.620		1.061	
42.50	2.550		1.131	
45.00	2.478		1.203	
45.00	2.294		1.387	
47.50	2.205		1.476	
47.50	1.924		1.757	
50.00	0.694		2.987	



Installation Information

Depth of pile (ft) = 42'-0"
Installation Torque (ft-lb) = 4,200
Torque measuring device : Pressure Gauge

Specimen Specification:

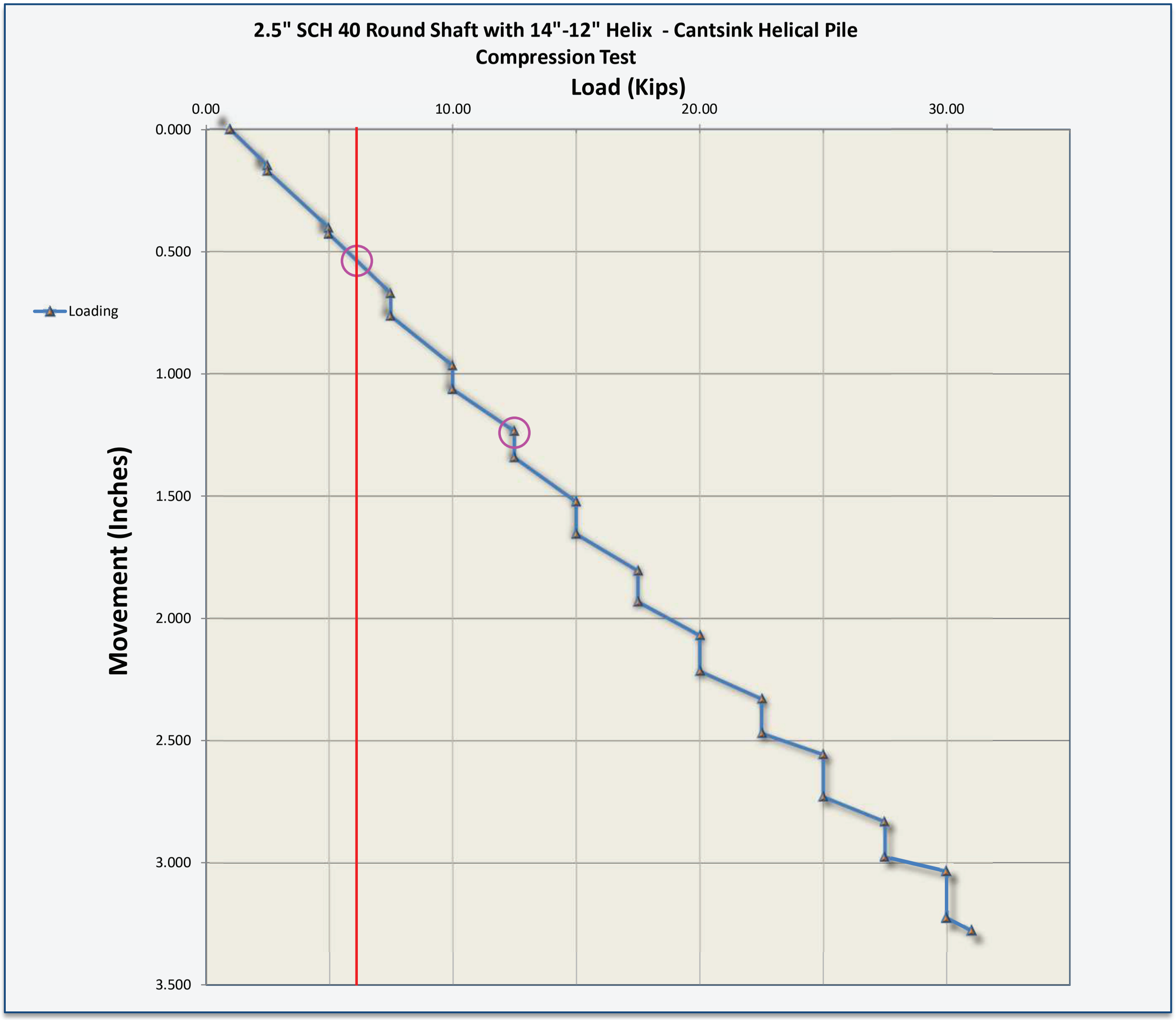
Helix Diameter (in) = 14"
Helix Thickness (in) = 3/8"
Shaft = 1.5 Inch - Square Shaft

2.5" SCH 40 Round Shaft Cantsink Manufacturing Helical Pile Compression Load Test
Apartment Complex - Brigade St, Charleston, SC 29403

Project Name: Charleston Steel Apartments - Phase II
Customer: CNT Foundations
Test Site: Brigade Street, Charleston, SC 29403
Date Tested: 02/26/2019

Coordenates:		
	N	32.811484
	W	-79.943691

Load (kips)	Reading 1 (Inches)	Reading 2 (Inches)	Net Deflection (Inches)	
0.00			0.000	
1.00	2.649	3.288	0.000	0.000
1.00	2.649	3.288	0.000	0.000
2.50	2.557	3.140	0.092	0.148
2.50	2.532	3.118	0.117	0.170
5.00	2.337	2.885	0.312	0.403
5.00	2.272	2.860	0.377	0.428
7.50	2.090	2.617	0.559	0.671
7.50	1.980	2.524	0.669	0.764
10.00	1.809	2.322	0.840	0.966
10.00	1.698	2.225	0.951	1.063
12.50	1.564	2.054	1.085	1.234
12.50	1.432	1.946	1.217	1.342
15.00	1.291	1.765	1.358	1.523
15.00	1.132	1.633	1.517	1.655
17.50	0.995	1.482	1.654	1.806
17.50	0.864	1.356	1.785	1.932
20.00	0.754	1.218	1.895	2.070
20.00	0.583	1.072	2.066	2.216
22.50	0.491	0.959	2.158	2.329
22.50	0.332	0.818	2.317	2.470
25.00	0.076	0.731	2.573	2.557
25.00		0.558	2.649	2.730
27.50		0.456	2.649	2.832
27.50		0.312	2.649	2.976
30.00		0.253	2.649	3.035
30.00		0.062	2.649	3.226
31.00		0.010	2.649	3.278



Installation Information

Depth of pile (ft) = 42'-0"
Installation Torque (ft-lb) = 5,000 ft-lb
Torque measuring device : Pressure Gauge

Specimen Specification:

Helix Diameter (in) = 12" & 14"
Helix Thickness (in) = 3/8"
Shaft = 2.5 Inch - Round Shaft

COMPRESSION TEST SETUP

INSTRUMENTS SETUP

LOAD CELL READING

CNT Foundations, LLC
3362 Navajo Street
North Charleston SC 29405
Tel.: (843)-577-7268
Cell: (843)-343-9068
travis@yourfoundationexperts.com

May 18, 2017

RE: Static Axial Compressive Test Report
Citadel War Memorial, Charleston, SC

Dear Sir:

Dear Mr. Bedson,

At your request The Foundation Firm, LLC witnessed the compression test of one Cantsink helical piles on Tuesday May 09, 2017 at the above referenced project.

The tested helical pile consisted of a 1.5" square bar with a 6" grout collar and a single 14" helix with a final embedment depth of 63 feet at the tip of the pile. This firm was not present to monitor the installation of the piles as this was done prior to our arrival to the site. We were informed by the contractor that this helical pile was installed with an approximate torque of 5,300 ft-lb (3,000 psi on a pressure gauge). The purpose for this test was to evaluate the capacity of this particular pile at this particular site and in these particular soil conditions.
For this compression test a set of four reaction piles were installed 37 feet deep. Steel beams were set up to create a frame to be able to apply load to the test pile. The test helical pile was monitored for deflection (pile movement) under axial compressive load. Pile deflection was measured with a new dial gauge. The results of the axial compression test are illustrated at the bottom of this letter.

Static Axial Compressive Load Test Monitoring

One static axial compression test was performed on May 09, 2017. Procedure for the testing was based on guidelines of AC358, Acceptance Criteria for Helical Pile Systems and Devices section 4.4.1.1 and guidelines of ASTM D1143. A test load of 120 kips was used for the test pile. The test load was determined using a safety factor of 2.0 of the design service load capacity of 60 kips. The test pile was preloaded to 7,000 lbs. prior to taking the initial reading. The test pile was then loaded in load increments of 6,000 lbs. (5% of the targeted final load). Each load interval was kept constant for a time interval of approximately 5 minutes, with readings being recorded at beginning and the end of the holding time.

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A reaction pile bolt broke at 102,000 pounds as one reaction pile was pulling out, so all were drilled four feet deeper and the test reset. Testing continued until a second reaction pile broke at 132,000 pounds where the test was stopped. Static Compressive Load Test records are presented at the end of this letter.

A digital load cell was used to determine the applied loads. A three inch continuous reading dial indicator, AGD 4, dial gauge with 0.001-inch accuracy was used to measure pile head deflection under the applied loads. See Appendix II for calibration sheet of the dial gauge.
The comprehensive pile load test results are presented graphically and tabulated as attachments for compression (Appendix I), while a summary of the test results including pile embedment are presented below.

PILE NUMBER	PILE HEAD DEFLECTION (IN.) AT 60 kips*	PILE HEAD DEFLECTION (IN.) AT 126 kips	PILE EMBEDMENT (FT)
Test Pile	0.096	0.623	63

* As identified on the load vs. deflection curve.

Evaluation and Recommendations

The helical pile shall be capable of resisting service level forces of 60 kips in compression. Per guidelines of AC358 based on the 10 percent maximum deflection of the helix the pile tested under axial compression load meets this criteria.

We appreciate the opportunity to be of service to you on this project and trust that you will call this office with any questions that you may have.

Respectfully Submitted,

THE FOUNDATION FIRM, LLC

Philip E. Slemons

Philip E. Slemons, P.E.

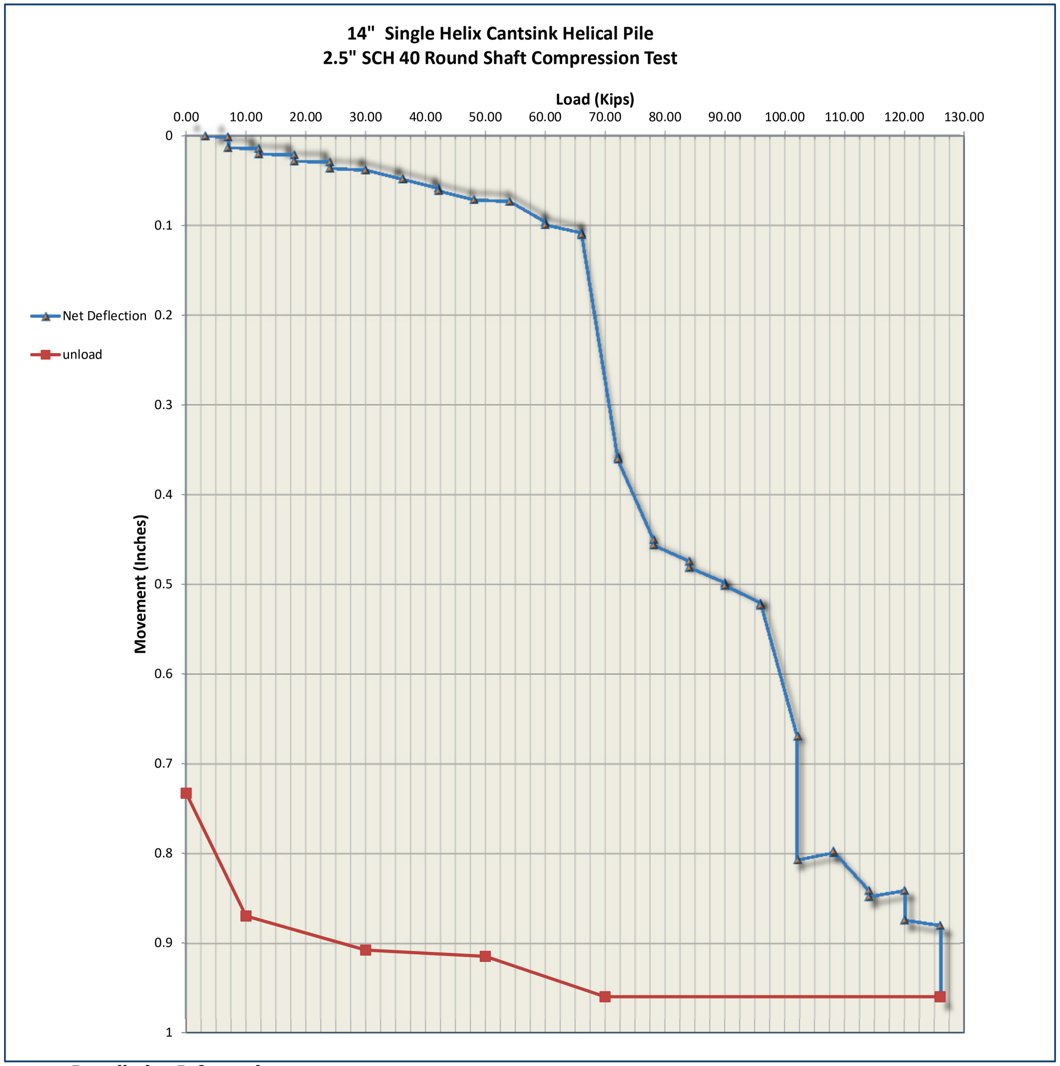
SOUTH CAROLINA
LICENSED PROFESSIONAL ENGINEER
No. 27217
PHILIP E SLEMONS

Philip E Slemons
5/18/17

1.5" Square Shaft Cantsink Helical Pile Compression Load Test
CITADEL WAR MEMORIAL

Project Name: CITADEL WAR MEMORIAL
Customer: CNT Foundations
Test Site: 171 Moultrie Street, Charleston, SC 29409
Date Tested: 5/9/2017

Load (kips)	Reading (Inches)		Reading Delta
0.00			
3.00	3.100		0.000
7.00	3.099		0.001
7.00	3.087		0.013
12.00	3.086		0.014
12.00	3.080		0.020
18.00	3.079		0.021
18.00	3.072		0.028
24.00	3.071		0.029
24.00	3.064		0.036
30.00	3.062		0.038
30.00	3.062		0.038
36.00	3.052		0.048
36.00	3.052		0.048
42.00	3.042		0.058
42.00	3.039		0.061
48.00	3.029		0.071
54.00	3.027		0.073
60.00	3.004		0.096
60.00	3.001		0.099
66.00	2.992		0.108
66.00	2.990		0.110
72.00	2.742		0.358
72.00	2.740		0.360
78.00	2.650		0.450
78.00	2.644		0.456
84.00	2.626		0.474
84.00	2.619		0.481
90.00	2.602		0.498
90.00	2.599		0.501
96.00	2.579		0.521
96.00	2.577		0.523
102.00	2.561	2.431	0.669
102.00		2.423	0.807
108.00		2.431	0.799
108.00		2.433	0.797
114.00		2.389	0.841
114.00		2.382	0.848
120.00		2.389	0.841
120.00		2.382	0.874
126.00		2.356	0.880
126.00		2.350	0.960
70.00		2.270	0.960
50.00		2.315	0.915
30.00		2.322	0.908
10.00		2.360	0.870
0.00		2.497	0.733



Specimen Specification:

Helix Diameter (in) = 14.0
Helix Thickness (in) = 0.375

Installation Information
Depth of pile (ft) = 63
Installation pressure (psi) = 3000
Torque measuring device = Pressure Gauge

THE FOUNDATION FIRM

October 30, 2020

Mr. Travis Bedson
CNT Foundations, LLC
3362 Navajo Street
North Charleston, SC 29405
Tel: 843-343-9068

Subject: Static Axial Compressive Test Report
300 Steel Mill Road, Darlington, SC 29540
Nucor - Degasser Addition
Project No. FFIRM20-133

Mr. Bedson,

At your request The Foundation Firm, LLC witnessed the compression test of one Cantsink Manufacturing 4.50-inch OD round shaft helical pile on Wednesday October 28, 2020 at the above referenced project.

The tested helical pile consisted of a 4.0" SCH 40 round shaft (4.50" O.D.) with triple helix configuration (12", 14" and 14 - 3/8" thick plates) on a seven-foot lead with a final embedment depth of approximately 52 feet at the tip of the pile. The first eleven feet from grade have a colum grout with an average diameter of 16" (predrilled with an auger) and the subsequent thirty-four feet thereafter have a column grout with an average diameter of 10". The helical pile was installed with an Prodig X12K5 motor with a final installation torque of approximately 12,000 ft-lb. We did not witness the installation of the pile as this was done on Thursday October 22nd to allow for cement grout curing.

The purpose for this test was to evaluate the capacity of this particular pile configuration at this particular site and in these particular soil conditions.

For the compression test a set of four reaction piles were installed to a maximum depth that varied from 50 to 52 feet (see Exhibit A-4 for compression test setup). Steel beams were set up to create a frame to be able to apply load to the test pile (W27x94 beams). The tested helical pile was monitored for deflection (pile movement) under axial compressive load. Pile deflections were measured with two calibrated dial gauges. The magnitude of the loads were monitored with a digital 200 tons Geokon load cell (Model 3000-400-4). The results of the axial compression tests are illustrated in Exhibit A-2 and Exhibit A-3.

Pile was loaded in increments of approximate five percent of the ultimate load (280 kips) with a holding time of five minutes per increment. An alignment load of four kips was utilized to align the reaction beams. Once the ultimate load was reached a holding time of thirty minutes was used to verify creep. A summary of the test results including pile embedment are presented below.

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Lilburn, GA 30047

Office: (770) 564-6949
Fax: (678) 280-3476

PILE NUMBER	INSTALLATION TORQUE (FT-LB)	HEAD AND PILE TIP DEFLECTION (IN.) AT 140 kips	HEAD AND PILE TIP DEFLECTION (IN.) AT 280 kips	PILE EMBEDMENT (FT)
LT-1	12,000	0.154	0.317	52′-0″

* As identified on the load vs. deflection curve.

Evaluation

The allowable deflection of helical piles is typically accepted as ten percent of the diameter of the helix or the average diameter of helix when the shaft is a multi-helix pile per guidelines of AC358 "Acceptance Criteria for Helical Piles Systems and Devices".

Engineer of record, will need to observe and analyzed the presented data and determine what is acceptable for the intended structure.

We appreciate the opportunity to be of service to you on this project and trust that you will call this office with any questions that you may have.
And the subsequent

Respectfully Submitted,

THE FOUNDATION FIRM

Philip E. Slemons, P.E.
Senior Engineer

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 27217 PHILIP E SLEMONS

Lester Salinas
Project Engineer

TEST LOCATION

NUCOR - DARLINGTON

Image courtesy of Google Maps

No claim is made as to the accuracy of the indicated beam locations other than for conceptual purposes to illustrate the test locations relative to existing site features. In consideration of the method used in their determination, as well as the base map's accuracy, the test locations shown should be considered approximate.

EXHIBIT A-1

Project Name: Nucor - Degrasser
Customer: CNT Foundations, LLC
Test Site: 300 Steel Mill Road, Darlington, SC 29540
Date Tested: October 28, 2020

Load (kips)	Reading I (Inches)	Reading II (Inches)	Zero Out I	Zero Out2 II	Average (Inches)
0.00	3.526	3.312	0	0	0
14.00	3.496	3.295	0.030	0.017	0.023
14.00	3.496	3.261	0.030	0.051	0.040
28.00	3.492	3.250	0.034	0.062	0.048
28.00	3.488	3.245	0.038	0.067	0.052
42.00	3.477	3.231	0.049	0.081	0.065
42.00	3.474	3.229	0.052	0.083	0.067
56.00	3.465	3.219	0.061	0.093	0.077
56.00	3.464	3.216	0.062	0.096	0.079
70.00	3.456	3.210	0.070	0.102	0.086
70.00	3.451	3.204	0.075	0.108	0.091
84.00	3.441	3.195	0.085	0.117	0.101
84.00	3.440	3.195	0.086	0.117	0.102
98.00	3.433	3.185	0.093	0.127	0.110
98.00	3.428	3.180	0.098	0.132	0.115
112.00	3.420	3.175	0.106	0.137	0.122
112.00	3.417	3.170	0.109	0.142	0.126
126.00	3.410	3.162	0.116	0.150	0.133
126.00	3.404	3.158	0.122	0.154	0.138
140.00	3.395	3.150	0.131	0.162	0.147
140.00	3.385	3.145	0.141	0.167	**0.154**
154.00	3.380	3.137	0.146	0.175	0.161
154.00	3.375	3.134	0.151	0.178	0.165
168.00	3.368	3.126	0.158	0.186	0.172
168.00	3.359	3.119	0.167	0.193	0.180
182.00	3.352	3.110	0.174	0.202	0.188
182.00	3.339	3.098	0.187	0.214	0.201
196.00	3.331	3.090	0.195	0.222	0.209
196.00	3.321	3.082	0.205	0.230	0.218
210.00	3.315	3.076	0.211	0.236	0.224
210.00	3.312	3.071	0.214	0.241	0.228
224.00	3.290	3.052	0.236	0.260	0.248
224.00	3.280	3.044	0.246	0.268	0.257

Load (kips)	Reading I (Inches)	Reading II Delta	Reading I Zero	Reading II Zero	Average (inches)
238.00	3.278	3.043	0.248	0.269	0.259
238.00	3.278	3.042	0.248	0.270	0.259
252.00	3.270	3.034	0.256	0.278	0.267
252.00	3.259	3.025	0.267	0.287	0.277
266.00	3.251	3.016	0.275	0.296	0.286
266.00	3.242	3.009	0.284	0.303	0.294
280.00	3.234	3.002	0.292	0.310	**0.301**
280.00	3.220	2.992	0.306	0.320	0.313
280.00	3.220	2.992	0.306	0.320	0.313
280.00	3.217	2.990	0.309	0.322	0.316
280.00	3.217	2.990	0.309	0.322	0.316
280.00	3.216	2.990	0.310	0.322	0.316
280.00	3.215	2.989	0.311	0.323	**0.317**
230.00	3.224	3.002	0.302	0.310	0.306
230.00	3.225	3.004	0.301	0.308	0.305
180.00	3.238	3.018	0.288	0.294	0.291
180.00	3.240	3.019	0.286	0.293	0.290
130.00	3.255	3.035	0.271	0.277	0.274
130.00	3.256	3.036	0.270	0.276	0.273
80.00	3.281	3.063	0.245	0.249	0.247
80.00	3.283	3.065	0.243	0.247	0.245
30.00	3.323	3.105	0.203	0.207	0.205
30.00	3.326	3.106	0.200	0.206	0.203
0.00	3.385	3.186	0.141	0.126	0.134

Specimen Specification:

Test No. : LT-4
Helix Diameter (in) = 12", 14" and 16"
Helix Thickness (in) = 0.375 in
Cross section area = 2.96 in^2

GPS Coordinates:

Lat : 34.375436
Long : -79.893149

Installation Information

Final Depth of pile (ft) = **52**
Installation torque (ft-lb) = 12,000
Torque measuring device =

EXHIBIT A-2

THE FOUNDATION FIRM

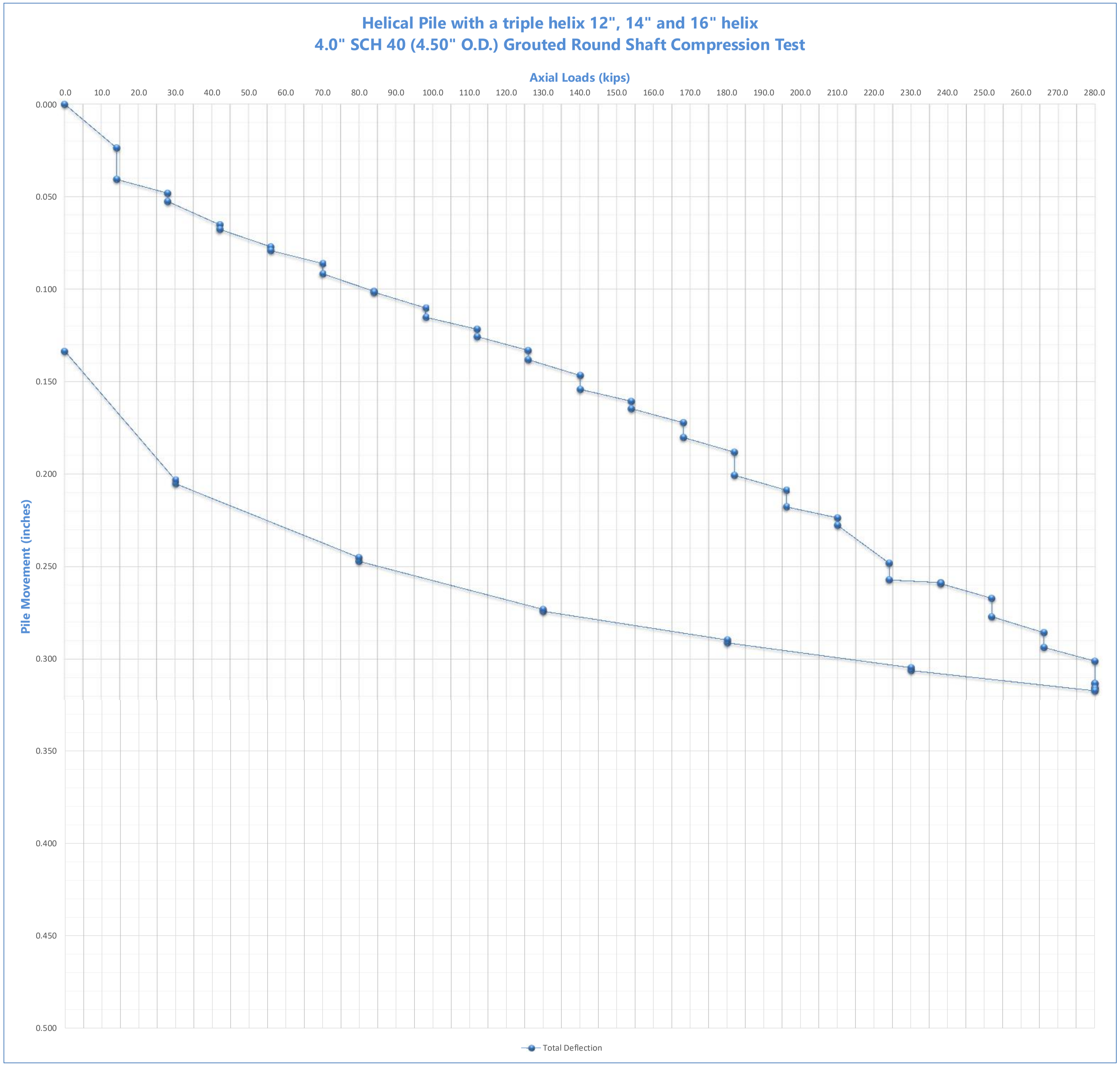
Helical Pile with a triple helix 12", 14" and 16" helix
4.0" SCH 40 (4.50" O.D.) Grouted Round Shaft Compression Test
Axial Loads (kips)
Pile Movement (inches)
Total Deflection

EXHIBIT A-3

TESTING SETUP

NUCOR

COMPRESSION LOAD TEST SETUP

EXHIBIT A-4

TESTING SETUP

NUCOR

LOAD CELL AND DIAL GAUGES SETUP

EXHIBIT A-5

THE FOUNDATION FIRM

Mr. Travis Bedson
CNT Foundations, LLC
3362 Navajo Street
North Charleston, SC 29405
Tel.: 843-577-7268
travis@yourfoundationexperts.com

May 23, 2017

RE: Static Axial Compression Test
HCA Trident Medical Plaza, Charleston, SC 29406

Dear Mr. Bedson:

At your request The Foundation Firm, LLC witnessed the compression test of one 1.5" square shaft grouted Cantsink helical pile and one MaClean Dixie SCH 80 round shaft helical pile on Saturday May 20, 2017 at the above referenced project.

The tested helical piles consisted of a 1.5" solid square shaft with a six inch diameter neat cement grout column with a final embedment depth of 40 feet. The bottom seven feet does not have grout. The second helical pile tested consisted on a 2.5" SCH 80 round shaft (nominal wall thickness of 0.276") with three helices, 14", 12" and 10" with a final embedment depth of 40 feet. This firm was not present to monitor the installation of the helical piles as this was done prior to our arrival to the site. It is our understanding that a representative from S&ME witnessed the installation of both helical piles and recorded the type of pile used, final installation depths and installation torques. The purpose for this test was to evaluate the capacity of these particular piles at this particular site and in these particular soil conditions as part of the requirements set forth on Note 6 of the foundation section of the structural plans.

For this compression test a set of four reaction piles where installed to a depth of 37 ft. Steel beams were set up to create a frame to be able to apply load to the test pile. The test helical pile was monitored for deflection (pile movement) under axial compressive load. Pile deflection was measured with a new calibrated dial gauge (see Picture 1). The results of the axial compression test are illustrated on the Appendix I-a and I-b.

Static Axial Compressive Load Test Monitoring

Two static axial compression tests were performed on May 20, 2017. Procedure for the testing was based on guidelines of AC358, Acceptance Criteria for Helical Pile Systems and Devices section 4.4.1.1 and guidelines of ASTM D1143. A test load of 50 kips was used for the test pile. The test load was determined using a safety factor of 2.0 of the design service load capacity of 25 kips, per helical pile notes on S0.1 by GS & P. The test pile was preloaded to 2,500 lbs. prior to taking the initial reading. The test pile was then loaded in load

71 First Avenue NW
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Fax: (678) 280-3476

increments of 2,500 lbs. (5% of the targeted final load). Each load interval was kept constant for a time interval of approximately 4 minutes, with readings being recorded at beginning and the end of the holding time.

A digital load cell was used to determine the applied loads. See Appendix II for calibration sheet of this equipment. A three inch continuous reading dial indicator, AGD 4, dial gauge with 0.001-inch accuracy was used to measure pile head deflection under the applied loads. See Appendix III for calibration sheet of the dial gauge.
The comprehensive pile load test results are presented graphically and tabulated as attachments for compression (Appendix I), while a summary of the test results including pile embedment are presented below.

PILE NUMBER	PILE HEAD DEFLECTION (IN.) AT 25 kips*	PILE HEAD DEFLECTION (IN.) AT 50.0 kips	PILE EMBEDMENT (FT)
Grouted Square Shaft	0.085	0.152	40
SCH 80 Round Shaft	0.270	0.522	40

* As identified on the load vs. deflection curve.

Evaluation and Recommendations

As indicated by the plans sheet S0.1, provided by GS & P dated September 06, 2016, the helical piles shall be capable of resisting service level forces of 25 kips in compression. The maximum net settlement allowed was established at ¾” at the service load as shown on note 6.4 of the structural plans. Both piles at the maximum deflection at 200% of the service load presented less than ¾” movement. The structural engineer of record should review the provided test report data to determine if this type of piling is acceptable.

We appreciate the opportunity to be of service to you on this project and trust that you will call this office with any questions that you may have.

Respectfully Submitted,
THE FOUNDATION FIRM, LLC

Philip E. Slemons, P.E.
Chief Engineer
SC Registration No. 27217

SOUTH CAROLINA
PROFESSIONAL
ENGINEER
LICENSED
No. 27217
PHILIP E. SLEMONS

5/23/17

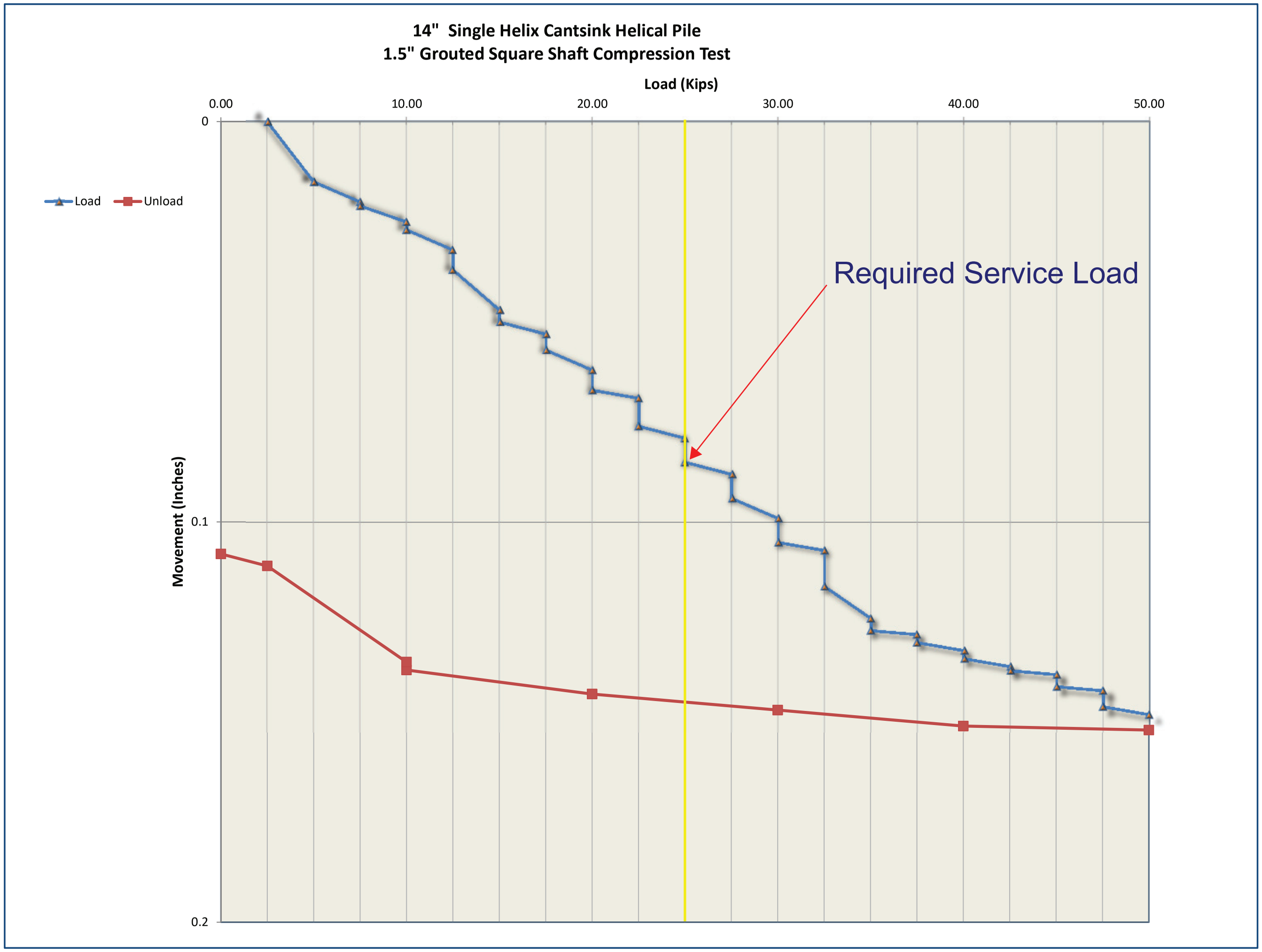
Lester Salinas
Project Engineer

Appendix I-a

1.5" Grouted Square Shaft Cantsink Helical Pile Compression Load Test
9330 Medical Plaza Dr, North Charleston, SC 29406

Project Name: OR Expansion - ED Rennovation / HCA Trident Medical Center
Customer: CNT Foundations
Test Site: 9330 Medical Plaza Dr, North Charleston, SC 29406
Date Tested: 05/20/2017

	Load (kips)	Reading (Inches)		Reading Delta
	0.00			
	2.50	2.241		0.000
	2.50	2.241		0.000
	5.00	2.226		0.015
	5.00	2.226		0.015
	7.50	2.221		0.020
	7.50	2.220		0.021
	10.00	2.216		0.025
	10.00	2.214		0.027
	12.50	2.209		0.032
	12.50	2.204		0.037
	15.00	2.194		0.047
	15.00	2.191		0.050
	17.50	2.188		0.053
	17.50	2.184		0.057
	20.00	2.179		0.062
	20.00	2.174		0.067
	22.50	2.172		0.069
	22.50	2.165		0.076
	25.00	2.162		0.079
	25.00	2.156		0.085
	27.50	2.153		0.088
	27.50	2.147		0.094
	30.00	2.142		0.099
	30.00	2.136		0.105
Reset	32.50	2.134	2.143	0.107
	32.50	2.125	2.138	0.116
	35.00		2.135	0.124
	35.00		2.132	0.127
	37.50		2.131	0.128
	37.50		2.129	0.130
	40.00		2.127	0.132
	40.00		2.125	0.134
	42.50		2.123	0.136
	42.50		2.122	0.137
	45.00		2.121	0.138
	45.00		2.118	0.141
	47.50		2.117	0.142
	47.50		2.113	0.146
	50.00		2.111	0.148
	50.00		2.107	0.152
	40.00		2.108	0.151
	40.00		2.108	0.151
	30.00		2.112	0.147
	30.00		2.112	0.147
	20.00		2.116	0.143
	20.00		2.116	0.143
	10.00		2.122	0.137
	10.00		2.124	0.135
	2.50		2.148	0.111
	0.00		2.151	0.108



Specimen Specification:

Helix Diameter (in) = 14.0
Helix Thickness (in) = 0.375

Installation Information

Depth of pile (ft) = 40
Installation pressure (psi) = 3000
Torque measuring device = Pressure Gauge

GPS Coordinates:

Lat : 32.975410
Long : -80.073422

1.5" GROUTED SQUARE SHAFT

COMPRESSION TEST SETUP

SERVICE LOAD READING

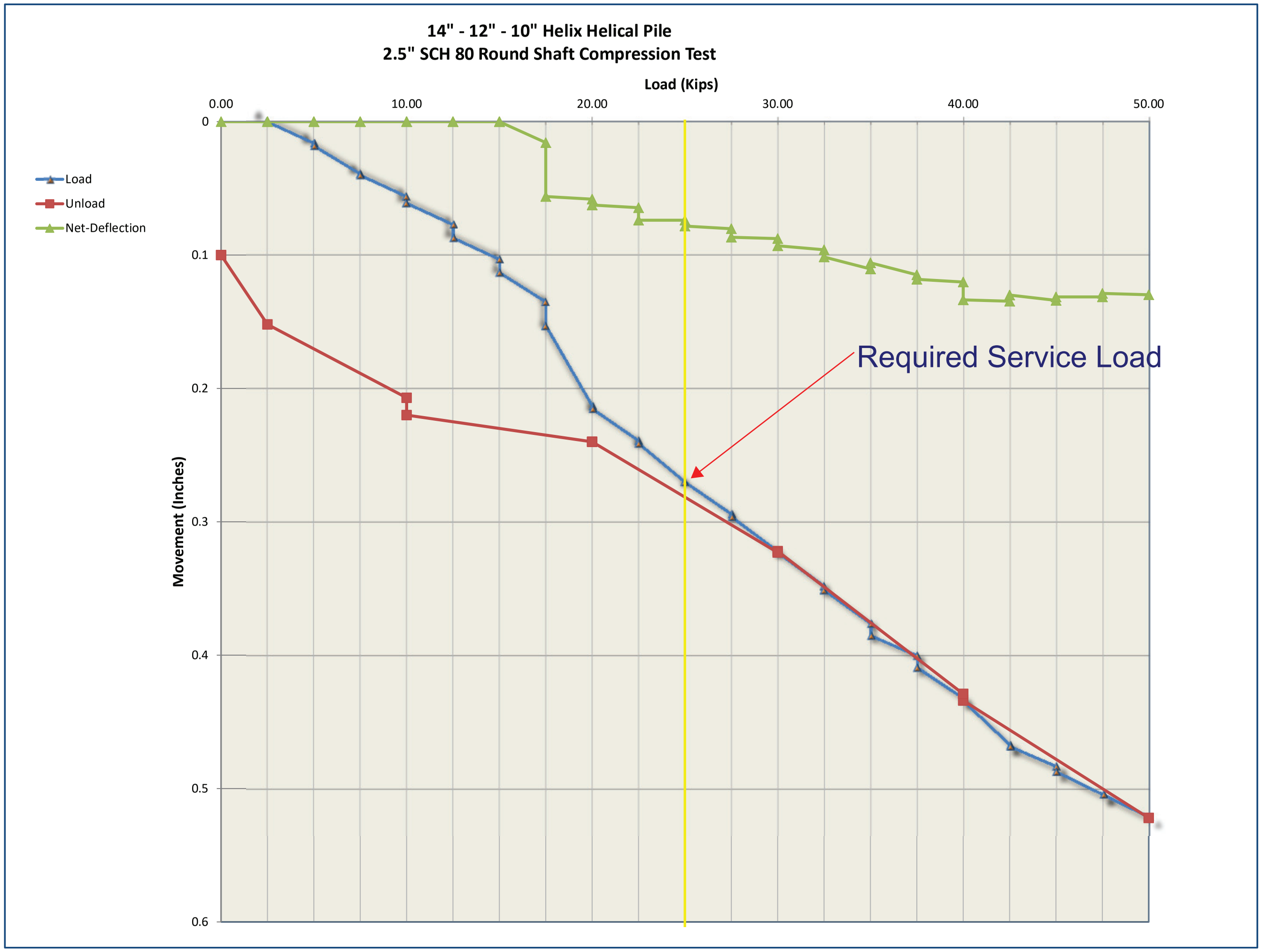
FINAL LOAD CELL READING

Appendix I-b

2.5" SCH 80 Round Shaft Helical Pile Compression Load Test
9330 Medical Plaza Dr, North Charleston, SC 29406

Project Name: OR Expansion - ED Rennovation / HCA Trident Medical Center
Customer: CNT Foundations
Test Site: 9330 Medical Plaza Dr, North Charleston, SC 29406
Date Tested: 05/20/2017

Load (kips)	Reading (Inches)	PL/AE	Reading Delta	PL AE	Net Deflection
0.00					
2.50	2.088		0.000	0.000	0.000
2.50	2.088		0.000	0.020	0.000
5.00	2.072		0.016	0.039	0.000
5.00	2.070		0.018	0.039	0.000
7.50	2.049		0.039	0.059	0.000
7.50	2.048		0.040	0.059	0.000
10.00	2.032		0.056	0.078	0.000
10.00	2.027		0.061	0.078	0.000
12.50	2.011		0.077	0.098	0.000
12.50	2.001		0.087	0.098	0.000
15.00	1.985		0.103	0.118	0.000
15.00	1.975		0.113	0.118	0.000
17.50	1.953		0.135	0.137	0.000
17.50	1.935		0.153	0.137	0.016
20.00	1.875		0.213	0.157	0.056
20.00	1.873		0.215	0.157	0.058
22.50	1.849		0.239	0.176	0.063
22.50	1.847		0.241	0.176	0.065
25.00	1.818		0.270	0.196	0.074
25.00	1.818		0.270	0.196	0.074
27.50	1.794		0.294	0.216	0.078
27.50	1.792		0.296	0.216	0.080
30.00	1.766		0.322	0.235	0.087
30.00	1.765		0.323	0.235	0.088
32.50	1.740		0.348	0.255	0.093
32.50	1.737		0.351	0.255	0.096
35.00	1.712		0.376	0.275	0.101
35.00	1.703		0.385	0.275	0.110
37.50	1.688		0.400	0.294	0.106
37.50	1.679		0.409	0.294	0.115
40.00	1.656		0.432	0.314	0.118
40.00	1.654		0.434	0.314	0.120
42.50	1.621		0.467	0.333	0.134
42.50	1.620		0.468	0.333	0.135
45.00	1.605		0.483	0.353	0.130
45.00	1.601		0.487	0.353	0.134
47.50	1.584		0.504	0.373	0.131
47.50	1.584		0.504	0.373	0.131
50.00	1.567		0.521	0.392	0.129
50.00	1.566		0.522	0.392	0.130
40.00	1.654		0.434	0.314	0.120
40.00	1.659		0.429	0.314	0.115
30.00	1.766		0.322	0.235	0.087
30.00	1.765		0.323	0.235	0.088
20.00	1.848		0.240	0.157	0.083
20.00	1.848		0.240	0.157	0.083
10.00	1.868		0.220	0.078	0.142
10.00	1.881		0.207	0.078	0.129
2.50	1.936		0.152	0.020	0.132
0.00	1.988		0.100	0.000	0.132



Specimen Specification:

Helix Diameter (in) = 14-10-12
Helix Thickness (in) = 0.375

Installation Information

Depth of pile (ft) = 40
Installation pressure (psi) = 2000
Torque measuring device = Pressure Gauge

GPS Coordinates:

Lat : 32.975410
Long : -80.073422

COMPRESSION TEST SETUP

COMPRESSION TEST INSTRUMENT SETUP

FINAL LOAD CELL READING

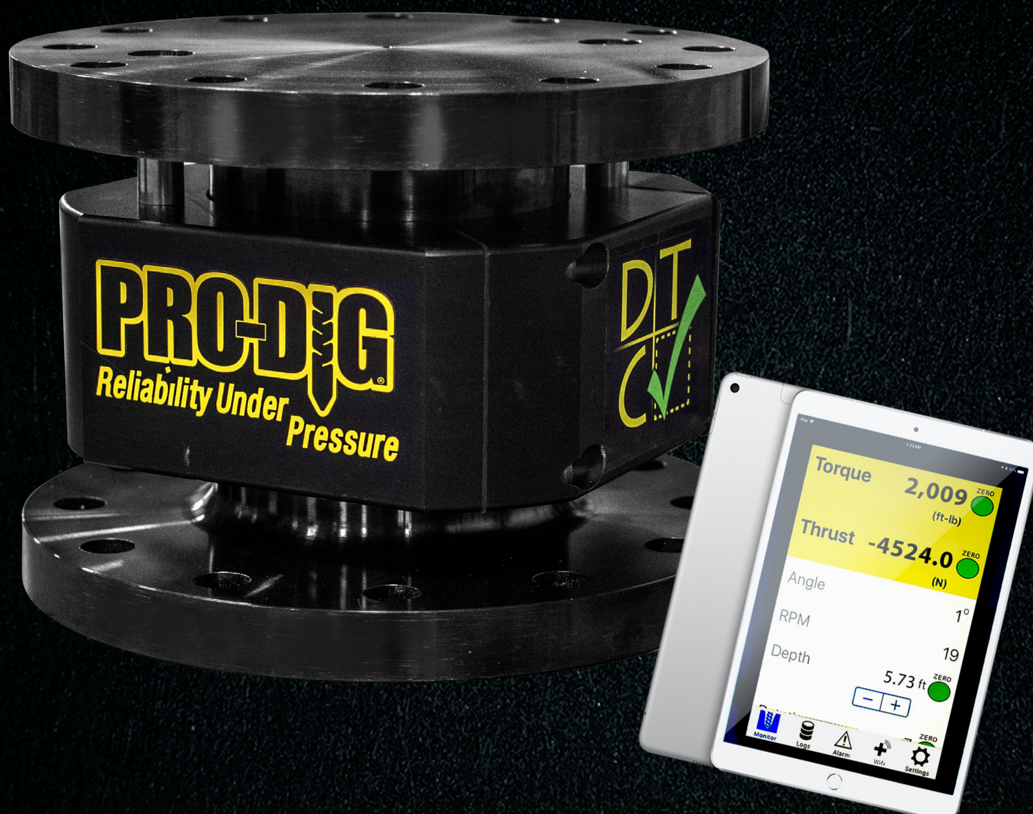
SCH 80 ROUND SHAFT PILE

Section 15
Intelli-Tork

PRO-DIG
Reliability Under Pressure

intelli·Tork
WIRELESS MONITORING SYSTEM

WIRELESS SCREW PIER TORQUE MONITORING SYSTEM



Instruction Manual
Owners Manual
Safety Precautions

Manual number: PROITK
Release Date: September 3, 2019

Operating Instructions
Maintenance
Parts Lists

785.856.2661
WWW.PRO-DIG-USA.COM

TABLE OF CONTENTS

MODEL NO: ______________________________

SERIAL NO: ______________________________

DATE OF PURCHASE: ____________________

INTRODUCTION

The PRO-DIG®, Intelli-Tork is a revolutionary, fully wireless, non-contact torque transducer with fully potted state-of-the-art electronics. The unit is extremely rugged and ideal for field based applications.

The Intelli-Tork measures the torque and axial load applied between two flanges and transmits the readings to the smart device for visual display and data logging. This method of measuring the torque applied is highly accurate (+/- 0.3%). The torque sensor is built into the housing of the Intelli-Tork. The data is transmitted directly to a smart device that has the "Intelli-tork®" App downloaded. The data is captured by the program and recorded as a text file (TXT). The text file can be viewed in the app and emailed to anyone where peripheral software such as Microsoft Excel for further custom analysis can be done.

OPERATION OF THE INTELLI-TORK MONITORING SYSTEM

Powering On the PRO-DIG® Intelli-Tork Hub

LED Status:

1. **Press the on/off button for at LEAST 2 SECONDS: a GREEN slow flashing light will appear when powered on.**
2. **If the battery state is good, the GREEN light will stays slow flashing.**
3. **The battery has an approximate life of 1 week, depending on use.**

Connecting the PRO-DIG® Intelli-Tork to the Intelli-tork App

1.0 COMPATIBILITY

An iPhone, iPad or Android with the operating system iOS 6.0 or later is required. The App is optimized for iPhone.

2.0 DOWNLOADING THE INTELLI-TORK© APP FROM THE APPLE© APP STORE OR GOOGLE PLAY STORE

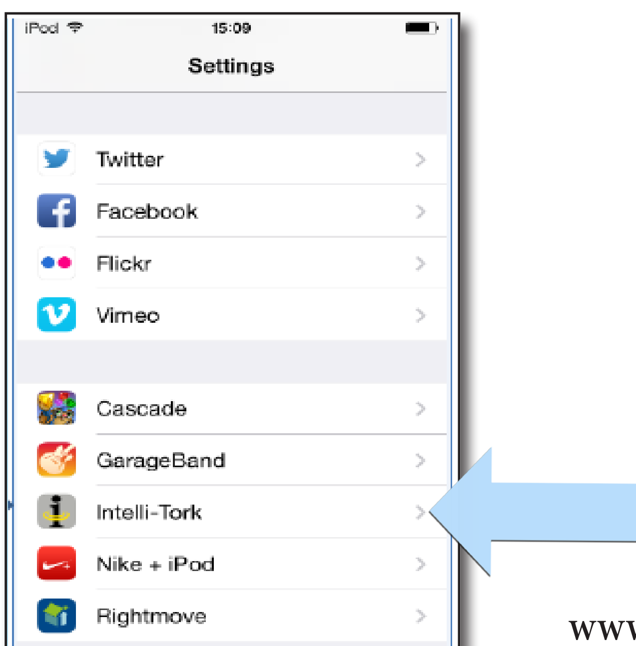
IPad users will need to navigate to the upper left corner of the screen and change the device to iPhone. Then search for "Intelli-Tork"

3.0 SETTINGS PAGE

All settings for your PRO-DIG® Intelli-Tork© are accessable from within the App. Simply select the settings icon from the bottom right.

On this page you can select preferences and make changes, as appropriate to your desired usage.

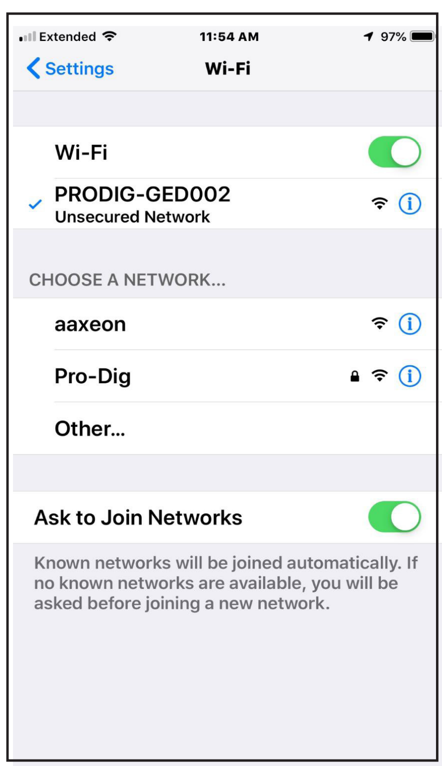
1) Torque units can be selected, options are ***ft.lbs*** or ***Nm***.
2) Thrust units can be selected, options are ***lbf, N, t*** or ***kgf***.
3) When using the logging feature, you can choose your desired logging interval. Options are,
   a) ***Auto*** (approximately 100 samples per second)
   b) On ***Depth Change*** (this option only creates a log entry when the depth is manually changed on the App's Monitor page
   c) A sample taken every ***1 second, 5 seconds, 10 seconds & 30 seconds***
4) The ***Show Voltage*** option allows you to choose whether or not to show the Battery Voltage on the Monitor page
5) Similarly, you can choose to show or hide the Angle here also
6) The default email address, which is the destination email address for all log files, can be entered on this page by simply clicking in the space and typing an email address. A default email address is not required, if one is not entered here then you'll be prompted to enter an email address each time a log file is emailed.
7) The Log All Raw Sensor Data option is a factory option used in fault finding. Under normal working this should be left unselected
8) The ***Network Settings*** are shown for reference. Multicast enabled should normally be left selected as non-multicast mode is for debug only.



4.0 CONNECTION GUIDE

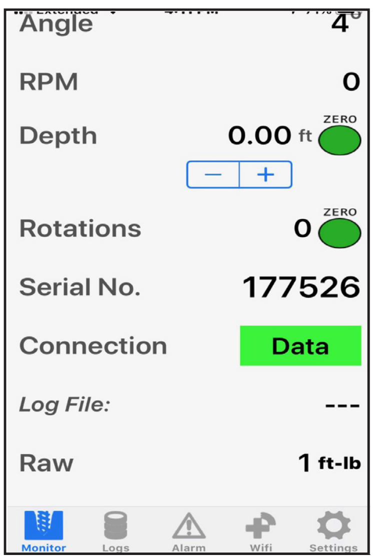
Once you have switched on your PRO-DIG® Intelli-Tork by pressing the on/off switch on the front of the transmitter please wait 10 seconds for the transmitter configuration to take place.
You'll notice a **green** flashing LED, which signifies that the transmitter is on, and also a flashing **orange** LED to signify that the transmitter configuration has taken place successfully.
It is then essential that the PRO-DIG® Intelli-Tork© is held vertically (within 5° or so) and that 3 full and slow rotations are completed. This allows for self-calibration of the rotational count and RPM functions. This must be done each time that the PRO-DIG® Intelli-Tork© is switched on.
To connect to the WiFi enabled PRO-DIG® Intelli-Tork© torque sensor please choose the wireless network, from your device, which corresponds to the serial number of your Intelli-Tork© by accessing the list of wireless networks available in Settings. Please ensure that you are in close proximity to the PRO-DIG® Intelli-Tork© when you do this.
Your PRO-DIG® Intelli-Tork© wireless network name (SSID) will appear in the list of available wireless networks and will be as per follows,

Pro-dig-Intell-XXXXXX

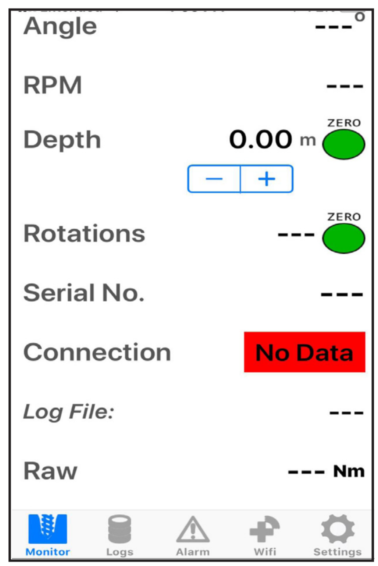


You can now close the settings page and open the PRO-DIG® Intelli-Tork© App which you will have previously downloaded from the Apple App Store or Google Play Store.

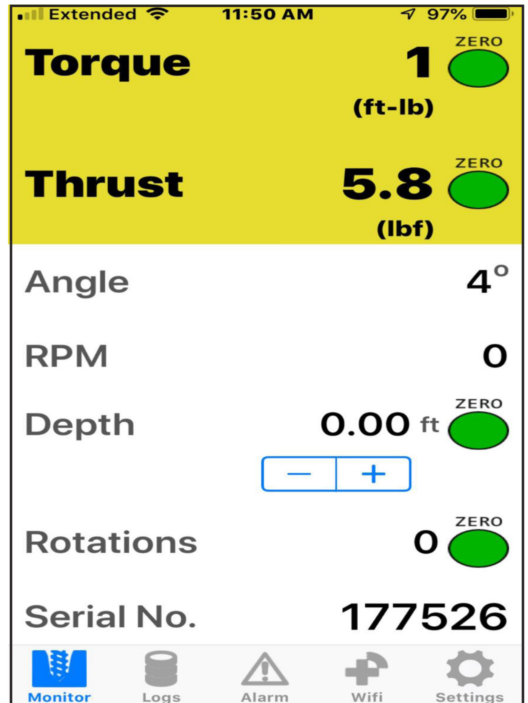
If the App is connected successfully to the PRO-DIG® Intell-Tork© then **Data** will be displayed in the connection information. Otherwise **No Data** will be shown, examples follow,



Connected Successfully



Not Connected



5.0 GENERAL OPERATION

The App has five pages and navigation between pages is achieved by clicking on one of the four icons shown at the bottom of the screen, as below,

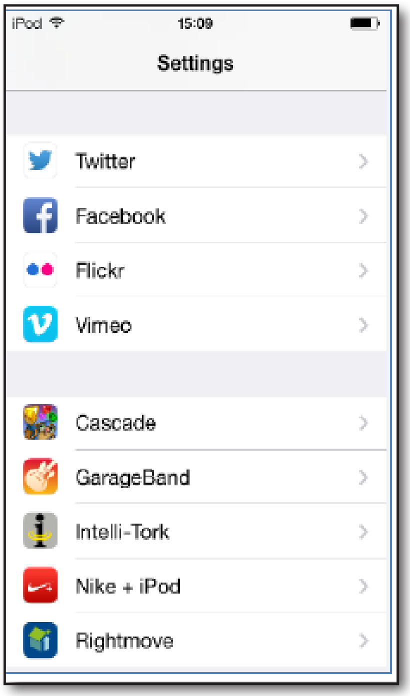
MONITOR: this is the page on which we imagine you will spend the most time. It is where all the parameters are displayed. Connection status is also displayed here.

LOGS: This is where you can create log files and then start, stop, view and email log files.

ALARM: On this page you can set an audible and visual alarm to alert you to when a specific torque value has been reached.

WiFi: Here you can change the WiFi channel, should you experience loss of data associated with a busy WiFi channel.

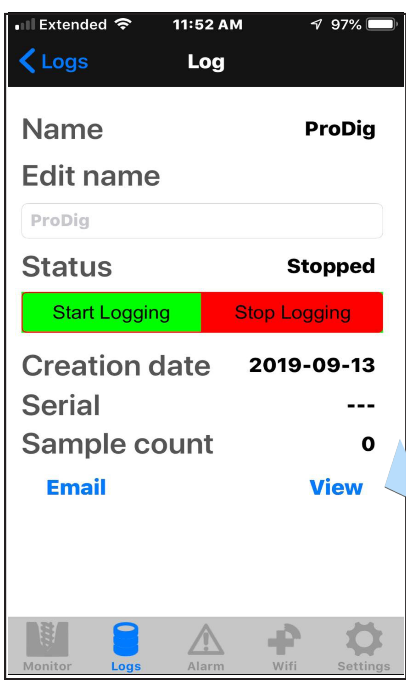
Settings: Here you can change the Intellitork settings,



5.1 MONITOR PAGE IN THE APP

This is the page that you will use to monitor parameters during operation of your equipment.
This page is essentially self-explanatory but there are a few things which should be pointed out.

The Torque and thrust values can be zeroed by pressing the green zero button but should only be done so when no load is on the Intelli-Tork© drilling head. ***Incorrect torque and thrust vales can be indicated if they are zeroed when the drilling head is under load. If in doubt, take all load of the drill head and re-zero. You can do this as many times as you wish.***
The depth can be increased manually here, in increments of 0.5ft or 0.5m dependent on whether you have ft.lbs or Nm selected for the units.
The number of rotations are displayed and can be zeroed from the Monitor page.
The serial number of your Intelli-Tork© is displayed, as is the battery voltage, assuming that you have selected this option in the Settings page, see above.
The Raw figure is referenced here and it is generally a factory parameter used for reference.



5.2 LOGS & LOGGING

5.2.1 Creating a new log file

Your Intelli-Tork© App deals with logging simply and easily. Once you've selected the Logs page follow the instructions below to create a new log,

1) Click in the dialogue box and enter the desired name for your new log file, in the example below we've used Pro-Dig and click Add Log and the new log file will appear in the list of available log files

5.2.2 Logging

To start and stop logging simply click on the desired log file and you'll be taken to the options associated with that log. Starting and stopping the logging is done simply by using the **Start Logging** and **Stop Logging** buttons.
The status of logging is shown as either Logging or Stopped.
A log can be stopped and re-started, as required, and the log file will simply continue logging from where it was stopped.
Once you start logging you are then free to move around the other screens in the app whilst logging takes place. **The *Monitor* screen should be displayed while drilling operations are taking place to ensure safe operation. While logging data do not navigate outside the app**

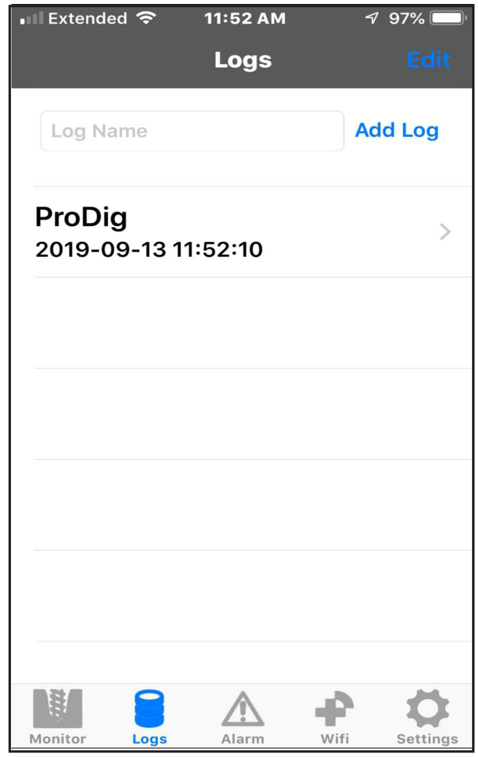
5.2.3 Viewing and Emailing Log Files

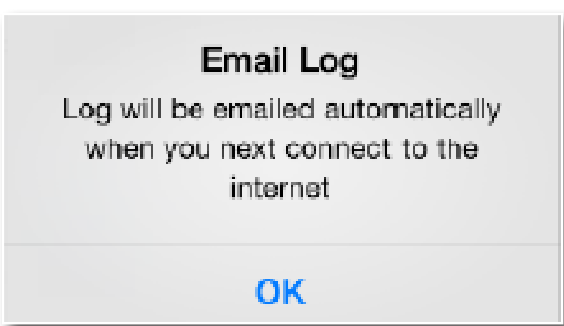
Once you have finished logging you have the option to view the log file before emailing it, should you require. To do this, simply click on the View option, which is located at the bottom right hand corner of the Log screen.

If you wish to email the log file then simply click on Email, as above, and you'll be prompted to review the email before sending. If you have not entered a default email address in the Settings page then you'll need to enter one here in the To: box.

If you are not connected to the internet, through WiFi or 3G, then a message may be shown advising you that the email will be sent next time you connect to the internet, this is normal.

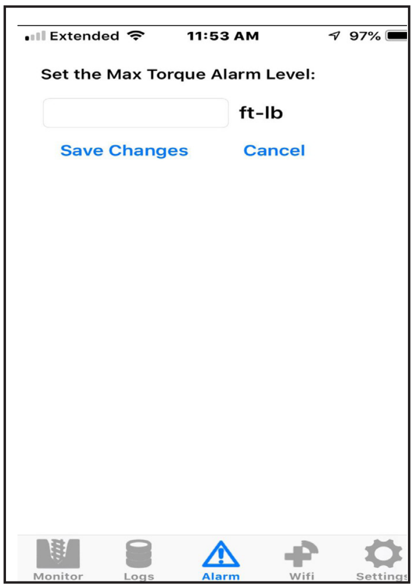
Once an address appears in the To: box your email is ready to send by pressing the send button, at the top right hand of the screen, and the email will be sent automatically next time you have internet connection.





The email will contain a .csv which will contain the log data and can be opened in Microsoft Excel© or most other spreadsheet software packages.

5.3 ALARMS

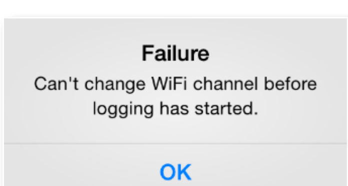


Your PRO-DIG® Intelli-Tork© App allows a torque alarm to be set to aid safe operation of your equipment. This alarm remains active until the maximum torque alarm level is set to zero.
An audible and visual alarm is triggered when the measured torque value exceeds the maximum torque alarm level.
To set the alarm simply go the alarm page and enter your desired maximum torque alarm level and select Save Changes.
Your alarm is now set and will remain set allowing for multiple operations.
To cancel the alarm simply enter '0' as the maximum torque alarm level.
During normal operation the actual measured torque value should be monitored from the Monitor page. The alarm function should not be relied on as the only source of information relating to the measured torque.

5.4 CHANGING THE WIFI CHANNEL

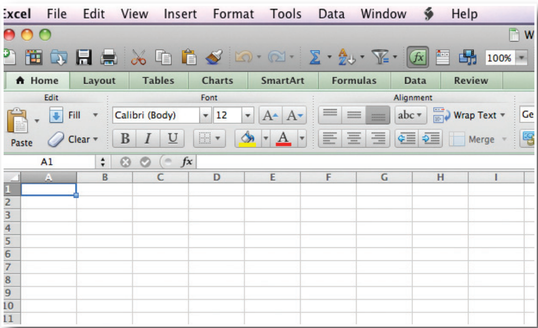
Under certain circumstances you may experience a loss of data when using a WiFi channel which is also used by other devices. Should this occur, and if you wish to change the WiFi channel, this can be done from the WiFi page. Simply click on the '+' or '-' buttons to select the desired new channel. Click on Change Channel to make the change.
If the WiFi channel has been changed successfully you will receive a confirmation message informing you of such.
If the channel was unable to be changed then an error message will be displayed.

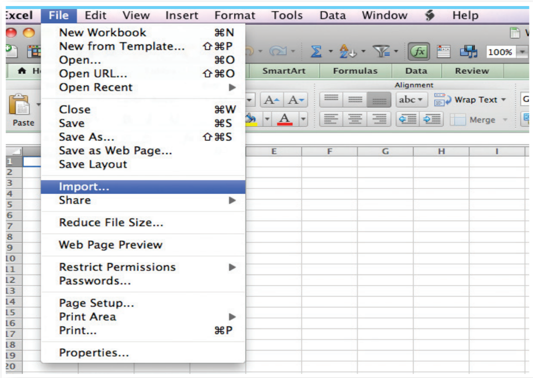
Should you try and change the WiFi channel while not connected to your PRO-DIG® Intelli-Tork© torque sensor then you will receive an error, as below. Just press OK. You should not receive this error message when connected to your PRO-DIG® Intelli-Tork©.

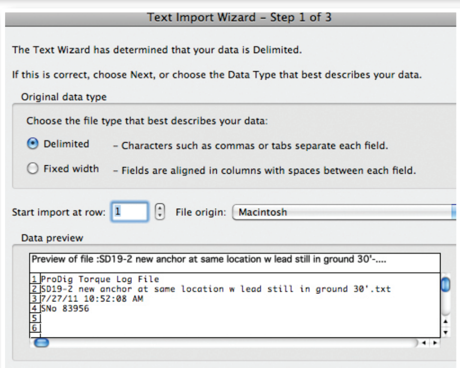


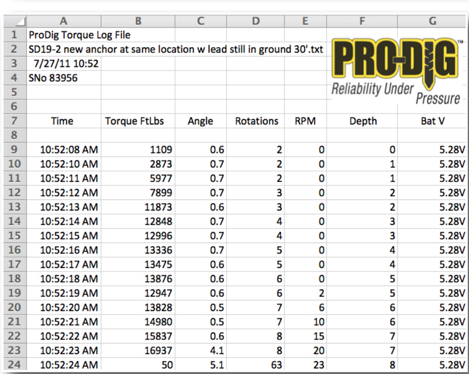
intelli-Tork
WIRELESS MONITORING SYSTEM

PRO-DIG™
Reliability Under Pressure








ProDig Torque Log File
SD19-2 new anchor at same location w lead still in ground 30'.txt
7/27/11 10:52
SNo 83956

Time	Torque FtLbs	Angle	Rotations	RPM	Depth	Bat V
10:52:08 AM	1109	0.6	2	0	0	5.28V
10:52:10 AM	2873	0.7	2	0	1	5.28V
10:52:11 AM	5977	0.7	2	0	1	5.28V
10:52:12 AM	7899	0.7	3	0	2	5.28V
10:52:13 AM	11873	0.6	3	0	2	5.28V
10:52:14 AM	12848	0.7	4	0	3	5.28V
10:52:15 AM	12996	0.7	4	0	3	5.28V
10:52:16 AM	13336	0.7	5	0	4	5.28V
10:52:17 AM	13475	0.6	5	0	4	5.28V
10:52:18 AM	13876	0.6	6	0	5	5.28V
10:52:19 AM	12947	0.6	6	2	5	5.28V
10:52:20 AM	13828	0.5	7	6	6	5.28V
10:52:21 AM	14980	0.5	7	10	6	5.28V
10:52:22 AM	15837	0.6	8	15	7	5.28V
10:52:23 AM	16937	4.1	8	20	7	5.28V
10:52:24 AM	50	5.1	63	23	8	5.28V

EXPORTING TO THIRD PARTY SOFTWARE

1. Select the file you wish to download, and then store it in a location on your computer.
2. Open the Microsoft Excel software on your PC, and click on the “FILE” tab.
3. Click on the “Import” tab (highlighted in blue).
4. Then select the file from your directory that you saved. (4).
5. The “Text Import Wizard” will appear. Follow the wizard instructions on how you wish the data to appear.
6. After completing the text import wizard, your data will display in your Microsoft Excel spread-sheet. The displayed data can be manipulated and graphed to suit your individual preferences.

CALIBRATION

The Intelli-Tork has been precision-factory calibrated and tested. In order to maintain an accurate and reliable reading, periodic calibration and certification is essential. THE INTELLI-TORK NEEDS TO BE RETURNED TO OUR CALIBRATION DEPARTMENT AT LEAST ONCE A YEAR. (For further information, please contact a member of our engineering dept.)

INSTALLING THE TORQUE HUB

1. The Intelli-Tork should be installed between the flange faces of the shaft adaptor and the pile drive tool. Please ensure that the fastening bolts are rated to at least grade 8, and torque the bolts to the appropriate manufacturer's guidelines. The total assembly is now ready to be attached to the drive motor.

   a. Pile Drive Tool

   b. Shaft Adaptor

   c. Drive tool and Kelly Bar secured to the Torque Hub. Use the correct bolts and not screws

See plain shaft above the thread, Correct ✓ If correct diameter and have a long enough shank to fit in the hole.

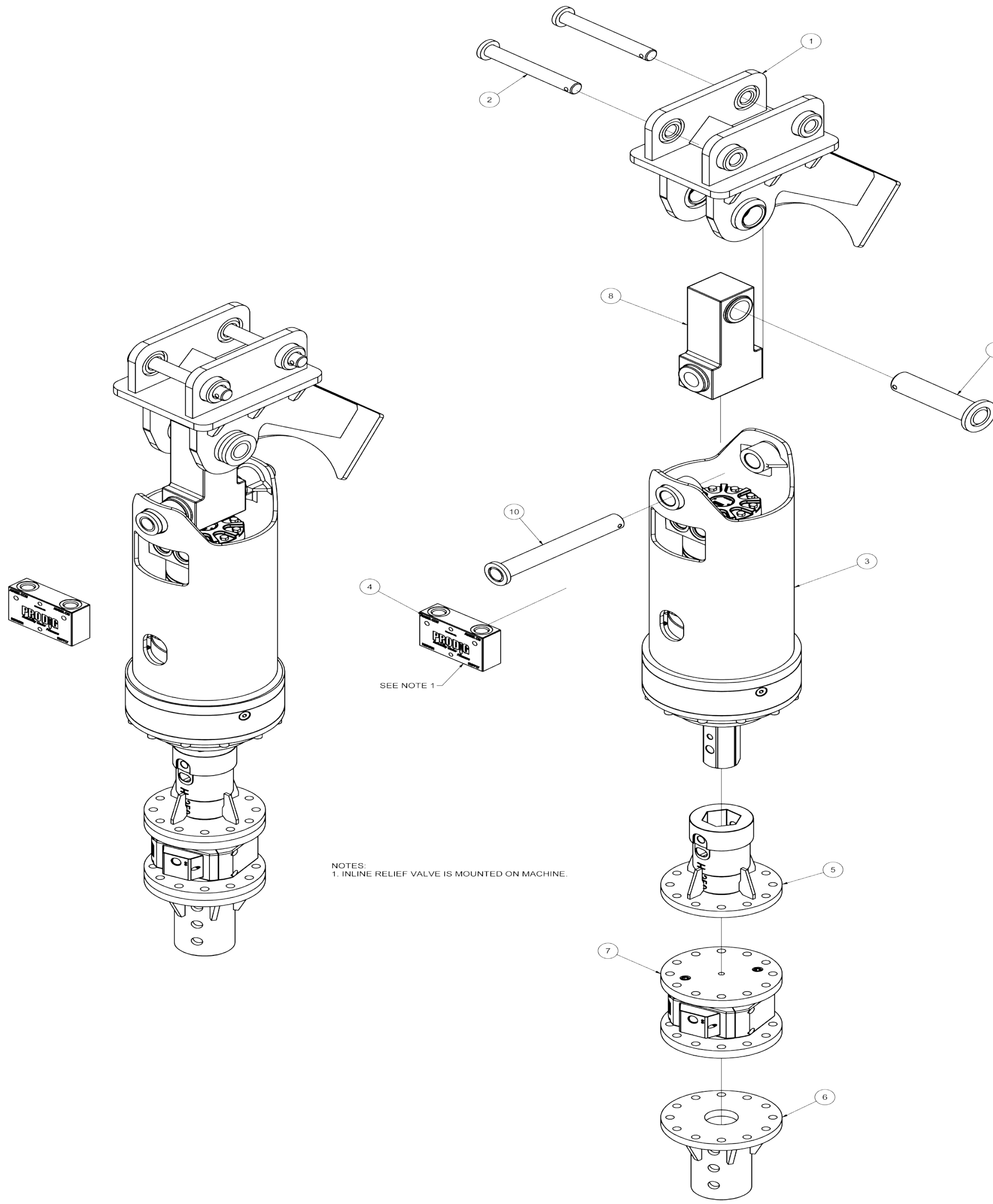
Not correct and threaded section will damage the holes ✗

2. Attach the shaft adaptor to the shaft of the drive motor. Screw Piles can then be attached to the pile drive tool.

3. The unit is now ready to operate.

To power off the Intelli-Tork, hold down the power button for 2 seconds.

*There is an option to use our Handhled Nautiz device vs. using an apple or android device please call for information.


1
2
8
9
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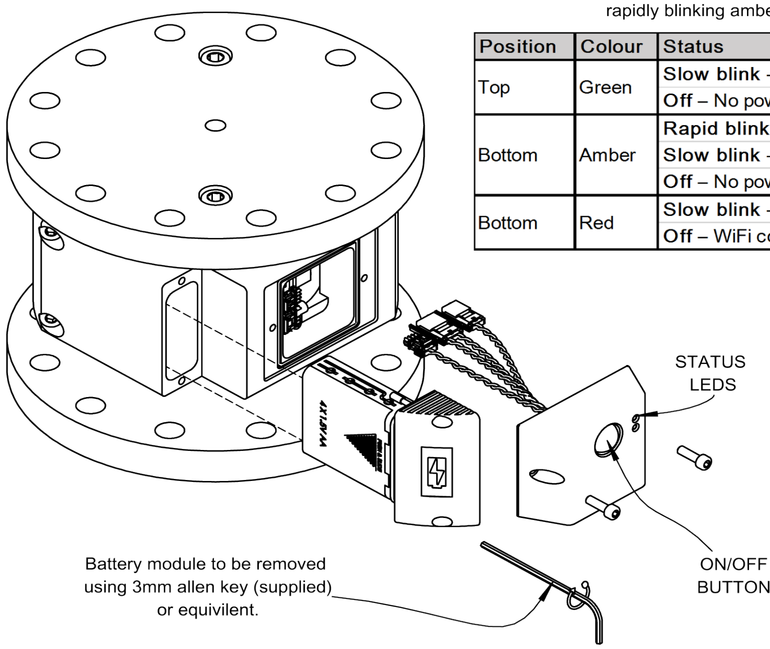
SEE NOTE 1
5
7
6
NOTES:
1. INLINE RELIEF VALVE IS MOUNTED ON MACHINE.

PRO-DIG™
Reliability Under Pressure

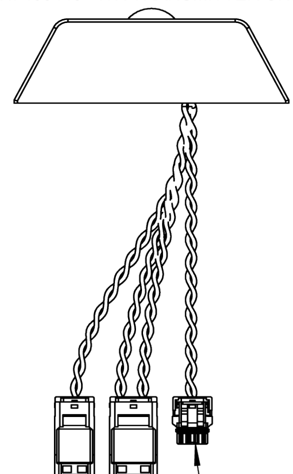
intelli-Tork® WIRELESS MONITORING SYSTEM

Intelli-Tork Transmitter Status LEDs
In normal, powered, transmitting state the top LED will be blinking green and the bottom rapidly blinking amber. See below for more LED status details.

Position	Colour	Status
Top	Green	**Slow blink** – Device powered
		Off – No power
Bottom	Amber	**Rapid blinking** – Data active
		Slow blink – Data paused (transmitter may need restarting)
		Off – No power
Bottom	Red	**Slow blink** – Transmitter WiFi not connected
		Off – WiFi connected (Or no power if all other LEDs are off)

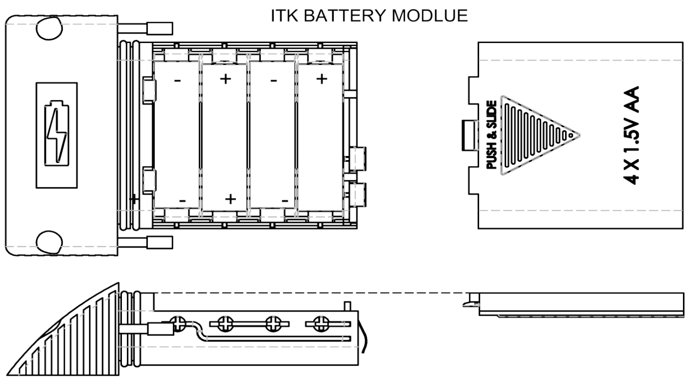


P/N: 400443 - ITK TRANSMITTER UNIT

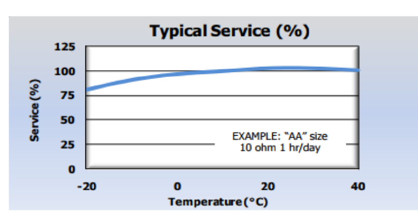


	H200	H300	S400	Units
Calibrated torque rating	10,000	25,000	60,000	lb.ft
Max torque rating	12,000	32,000	67,000	lb.ft
Calibrated thrust rating	33	33	33	klbf
Max thrust rating	330	660	990	klbf
Operating Temperature range	-30 up to 60			C
Operating voltage	6			V
Data output rate	10			sps
Non-linearity Accuracy	0.25			%

ITK BATTERY MODLUE



Battery life vs temperature - Lithium Iron Disulfide



4 x 1.5V Energizer Lithium Iron Disulfide AA Battery 12-2034 recommended. Other standard AA or rechargeable batteries can be used however please note, sensor performance and run time will be affected according to battery quality and chemistry

NOTES:

PRO-DIG

Reliability Under Pressure

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ADSC The International Association of Foundation Drilling

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