TECHNICAL MANUAL

SECOND EDITION

Helical Piles and Anchors
Hydraulically Driven Push Piers
Polyurethane Injection
Supplemental Support Systems

FOUNDATION SUPPORTWORKS®

FSI Technical Manual

Second Edition

July 2014

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Supplemental information for products and systems not published in this manual can be found on the Foundation Supportworks®, Inc. commercial website, www.OnStableGround.com.

CHAPTER 1 INTRODUCTION

1.1 – About Foundation Supportworks®, Inc.

Foundation Supportworks, Inc. (FSI) is a leading manufacturer of helical pile systems, hydraulically-driven push pier systems, wall anchoring and wall bracing systems, and supplemental crawl space support systems. Within the line of wall bracing products, FSI has the exclusive rights to market and distribute PowerBrace™ Foundation Wall Bracing Systems and CarbonArmor® and ArmorLock™ carbon fiber wall reinforcement. Outside of the more traditional offerings of foundation support and repair products, FSI formulates and distributes the PolyLEVEL® product line of polyurethane foams and resins. Each of the products or systems listed above are included in chapters of this Second Edition FSI Technical Manual. FSI was founded on the principles of integrity, quality and service and it is our mission to provide the industry with innovative solutions that are appropriately designed and tested, expertly installed, and dependable to perform as promised.

Omaha, NE

FSI began in April 2008 with the partnership of Greg Thrasher and Larry Janesky, both feeling there was a need for a manufacturer in the foundation support and foundation repair industry to not only bring fully-engineered, quality products to the marketplace, but to also provide a network of installing contractors with the proper tools and training to assist them in building more successful businesses. Greg and Larry each already owned successful basement waterproofing and foundation repair companies. Greg Thrasher started his business in 1975 in Earling, Iowa and then moved to Omaha, Nebraska in 1980. Thrasher Basement Systems, Inc. has since grown to become one of the largest residential foundation stabilization companies in the United States. With continued company growth, Foundation Supportworks® by Thrasher was formed as the commercial division of the business. Larry Janesky started his career as a self-employed carpenter and builder in 1982. In 1987, Larry founded Basement Systems, Inc. in Seymour, Connecticut. Basement Systems, Inc. is now the largest network of waterproofing and crawlspace repair contractors in the world. In addition to being a successful businessman, Larry holds 29 patents and is the author of six books. Please visit the FSI website at *www.OnStableGround.com* for more biographical information about the company founders.

Seymour, CT

FSI's commitment to its network of installing contractors and, ultimately, the end consumer, is apparent by employing a team of customer service and dealer support staff that is unparalleled in the industry. Our staff of full-time professionals include:

- Geotechnical and Structural Engineers,
- Corporate Trainers,
- Dealer Support (Product Line Managers and Product Installation Experts),
- Sales Development Specialists,
- Video Production Specialists,
- Graphic Designers,
- 3D Graphics and 3D Animation Experts,
- Computer Programmers, and
- a team of Website Developers.

As of the printing date of this manual, there are over 100 independent contractors throughout the United States and Canada utilizing Foundation Supportworks' products for their projects/applications. With installing contractors from coast to coast, Foundation Supportworks is focused on training, gathering and sharing the best practices in the industry. Your authorized FSI installing contractor is therefore operating with the resources of literally hundreds of years of combined experience.

With major dealer support facilities in Omaha, Nebraska and Seymour, Connecticut, Foundation Supportworks operates with a long-term vision. The coupled effect of the expertise of the Foundation Supportworks staff and our installing contractor network's years of combined experience and leadership in the structural stabilization industry translates to quality products, dependable service, expert installations and a team you can trust.

Our Mission Statement:

To champion dealer success through genuine relationships and radical support.

It is our core belief that if we provide a level of support that is unheard of and unexpected in this industry, our dealers will be equipped and able to fully focus on their main objective – serving the customers and design professionals in their markets with unparalleled quality and expertise.

1.2 – FSI Engineering Roles

Foundation Supportworks has both geotechnical and structural engineers on staff for product design, quality assurance of products and support to our network of FSI installing contractors. Our in-house engineers are available to assist with preliminary design applications and to provide technical support to engineers, architects, building departments and general contractors local to the projects. Our engineers are experts in the industry and routinely present technical information at industry trade conferences, engineering and architectural meetings and conferences, as well as to contractors and home inspectors. Please visit the FSI website for biographical information about our engineers.

FSI engineers act on behalf of the product manufacturer and therefore are unable to serve as the engineer of record by signing and stamping project-specific drawings and details. Rather, they can provide preliminary designs, consultation, and the technical information and support necessary for the local design professional to feel comfortable specifying and utilizing FSI products for their projects and applications. Preliminary design services and consultation for prospective piering and anchoring projects are free of charge to design professionals and FSI installing contractors. A description of this service and its process are provided in the Preliminary Design Services letter in Appendix 2G.

To simplify the design process for helical piles and tiebacks, Foundation Supportworks created HelixPro® Helical Foundation Design Software for Professionals, a web-based helical foundation design tool available free of charge to design professionals. For more information on HelixPro, refer to Appendix 2C.

1.3 Continuing Education Opportunities

Foundation Supportworks is a leading provider of continuing education for design professionals in North America on the topics of helical foundation systems, push pier systems, and polyurethane injection. We know there are many individuals and companies who claim to offer great learning opportunities, but we find that it is the collaboration of both subject matter experts and curriculum design experts that yields the best results in terms of attendees meeting learning objectives.

Engineering Seminars

Our curriculum and education methods have been evaluated by national institutions such as the American Council of Engineering Companies' (ACEC) Registered Continuing Education Program (RCEP) and the American Institute of Architects (AIA), and FSI has been named an approved provider of continuing education through both of these organizations. Our curriculum has also been evaluated and approved directly by the Florida State Board of Engineers, as Florida was the one state where RCEP approval was not recognized.

FSI offers both distant and in-person learning opportunities. We host regularly scheduled monthly webinars on the following topics at 11:30 am and 1:30 pm Central Time:

1st Wednesday of each month
An Introduction to Helical Foundation Systems

2nd Wednesday of each month
An Introduction to Polyurethane Injection

3rd Wednesday of each month
An Introduction to Push Pier Systems

For more in-depth knowledge about product design, installation and applications for the systems FSI provides, we offer half day in-person presentations several times each year in various locations throughout North America. We also speak regularly for professional organizations throughout the nation during their monthly, quarterly or annual meetings. To see if there is an upcoming presentation near you, please visit our commercial website at *www.OnStableGround.com* and click on the "Events" button.

To register for any continuing education opportunity, please contact your local FSI installing contractor or send us an email at *training@foundationsupportworks.com*.

All learning opportunities provided by Foundation Supportworks are free of charge, and best of all, can be submitted as hours to fulfill your CEU/PDH requirements.

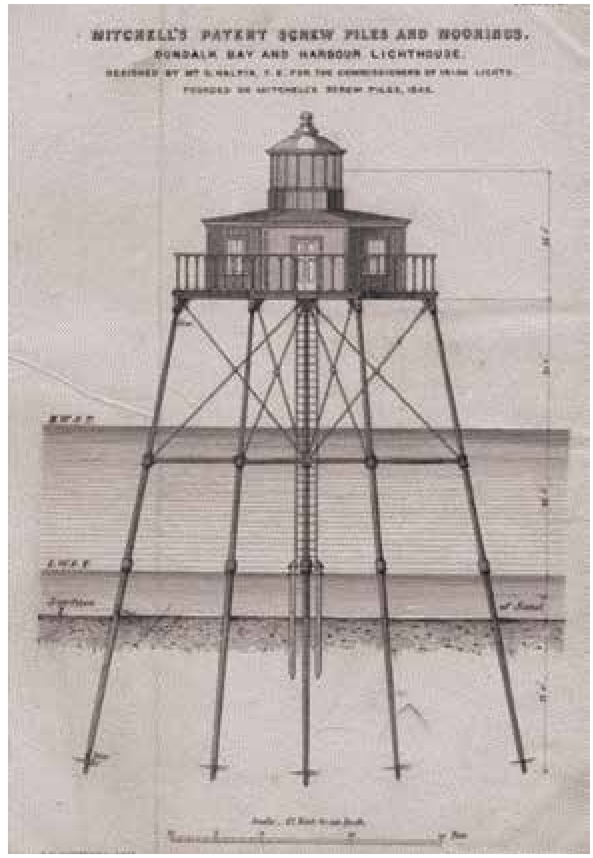
CHAPTER 2 HELICAL FOUNDATION SYSTEMS

2.1 – History

The use of helical piles and anchors in construction dates back nearly 200 years. In the 1830s, the earliest versions of today's helical piles were used in England for moorings and for the foundations of lighthouse structures.

Lighthouse supported by helical pile foundations

Usage spread throughout the world through the later 19th century for similar type applications. Developments and improvements in other deep foundation alternatives then resulted in a general decrease in the use of helical piles in the first half of the 20th century. Following World War II, advancements in hydraulic motors and the explosive expansion of the national utility grid caused a resurgence in the use of helical piles, primarily in tension applications for guying towers and poles. Today, helical piles are used in both tension and compression load applications and are gaining worldwide acceptance throughout the construction industry and engineering community due to the versatility of both the



Early patent of screw (helical) piles

product and the installation equipment. In 2007, the International Code Council Evaluation Service (ICC-ES) approved AC358, Acceptance Criteria for Helical Pile Systems and Devices. AC358 provides helical pile manufacturers with standardized methods for the design and testing of helical piles, resulting in product capacity ratings that are generally considered conservative, yet appropriate. Interested parties may purchase a copy of AC358 from the ICC-ES website: *www.icc-es.org*. Helical piles have also been included of the International Building Code since the 2009 edition.

2.2 – Summary Description

Helical piles are a factory-manufactured steel foundation designed to resist axial compression, axial tension, and/or lateral loads from residential and commercial structures. The system consists of a central shaft, one or more helix-shaped bearing plates, and a bracket that allows attachment to structures. The helix plates are commonly referred to as blades or flights and are welded to the lead section. Extension shafts, with or without additional helix plates, are used to extend the pile to competent load bearing soil and to achieve design depth and

New construction helical pile installation

capacity. Brackets are used at the tops of the piles for attachment to structures, either for new construction or retrofit applications. Helical piles are advanced (screwed) into the ground with the application of torque.

Sheet pile wall stabilization with helical tiebacks

The terms helical piles, screw piles, helical piers, helical anchors, helix piers, and helix anchors are often used interchangeably by specifiers. However, the term “pier” more often refers to a helical pile loaded in axial compression, while the term “anchor” more often refers to a helical pile loaded in axial tension. The term “pile” traditionally describes a deep foundation that can resist both tension and compression loads.

Helical tieback installation with hand-held equipment

Helical tiebacks and helical soil nails are types of helical anchors differentiated by their specific design methodology and/or installation orientation. Helical tiebacks are designed similarly but differ from vertically-installed helical piles in that they are typically installed in a horizontal to 45-degree downward from horizontal orientation to laterally support the tops of earth retaining structures; e.g., retaining walls, foundation walls, sheet pile walls, soldier pile walls with wood lagging, etc. Helix plates are typically limited to the lead section or the lead and first extension of the tieback. Multi-helix leads for piles and tiebacks generally consist of increasing plate sizes from the tip. Helical soil nails are designed with same-sized helix plates, typically 6 or 8 inches in diameter, spaced evenly along the entire length of the nail, including the lead and extensions. Soil nails are typically installed in a closely-spaced grid pattern to reinforce the soil and provide a stable earth mass. Helical tiebacks and helical soil nails are presented in their own sections later in this chapter.

2.3 – Helical Foundation System Components

2.3.1 – Helix Plates

The initial installation of a helical pile is performed by applying downward force (crowd) and rotating the pile into the earth via the helix plates. Once the helix plates penetrate to a depth of about two to three feet, the piles generally require less crowd and installation is accomplished mostly by the downward force generated from the helix plates, similar to the effect of turning a screw into a block of wood. Therefore, the helix plate performs a vital role in providing the downward force or thrust needed to advance the pile to the bearing depth. The helix plate geometry

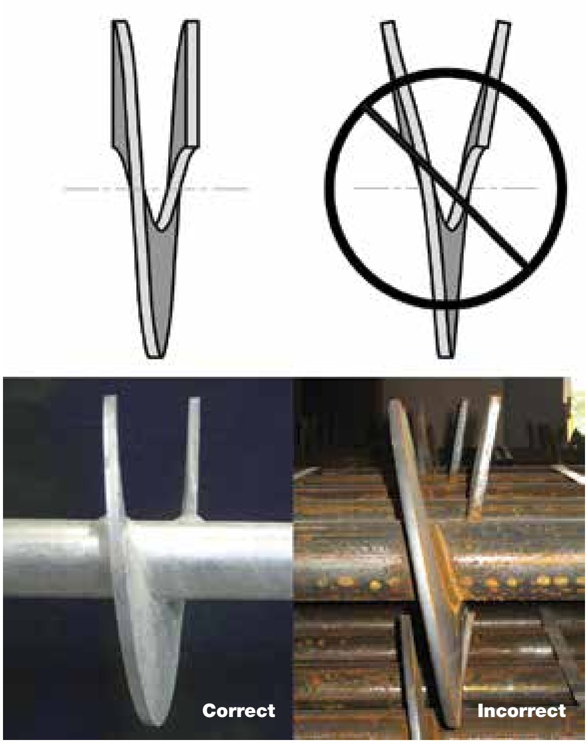


Figure 2.3.1.a

further affects the rate of penetration, soil disturbance and torque to capacity correlation. The consequences of a poorly-formed helix are twofold; (1) the helix plate severely disturbs the soil with an augering effect which (2) directly results in more movement upon loading than a pile with well-formed helices. The differences between a well-formed helix and poorly-formed helix are visually obvious and are shown in *Figure 2.3.1.a*.

A true helix shape can be described as a three-dimensional curve that travels along and sweeps around an axis where any radial line remains perpendicular to that axis.

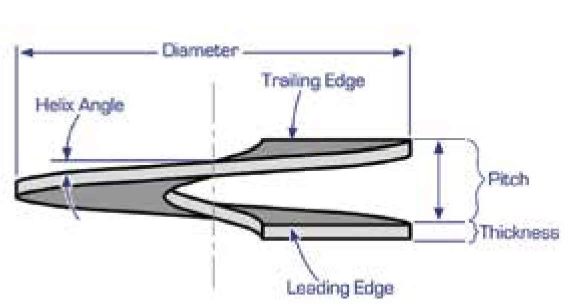


Figure 2.3.1.b
Helix Plate Geometry

A helix plate is further defined by geometric parameters including diameter, thickness, pitch, helix angle and edge geometry *(Figure 2.3.1.b)*. Helix plate diameters can vary from 6 to 16 inches for most commonly used shaft sizes. The majority of helix plates have thicknesses of either ⅜ or ½ inch, however, thicker plates are used for larger diameter piles. The pitch is the distance or separation between the leading and trailing edges and controls the depth of installation per revolution of the helix plate. The helix angle is the blade angle formed relative to the shaft and will vary within the blade for any given radius. The edge geometry refers both to the perimeter geometry of the helix and the shape of the leading and trailing edges. Most helix flights are manufactured with a perimeter geometry that is generally circular. The leading edge can have varying cuts and shapes including blunt (flat), sharpened, standard cut, V-style cut, etc. to provide options for changing soil conditions. The trailing edge is generally a standard cut, blunt or sharpened, and has no effect on installation in varying soils.

A helix plate is formed by cold pressing the steel plate with matching machined dies. Both the shape of the die and the amount of applied force during the press operations are important to ensure parallel leading and trailing edges and the required pitch tolerances. The amount of die press; i.e., the pressed shape and deflection, must also be adjusted for changing plate thicknesses, steel grades and anticipated spring back.

ICC-ES AC358 establishes design and testing criteria for helical piles evaluated in accordance with the International Building Code. AC358 provides the following criteria for helix plates in order to be considered as a "conforming system".

- True helix shaped plates that are normal with the shaft such that the leading and trailing edges are within ¼ inch of parallel.
- Helix plate diameters may be between 8 and 14 inches with thicknesses between ⅜ inch and ½ inch.
- Helix plates and shafts are smooth and absent of irregularities that extend more than 1⁄16 inch from the surface excluding connection hardware and fittings.
- Helix spacing along the shaft shall be between 2.4 to 3.6 times the helix diameter.
- The helix pitch is 3 inches ± ¼ inch.
- All helix plates have the same pitch.
- Helical plates are arranged such that they theoretically track the same path as the leading helix.
- 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.
- Helical foundation shaft advancement equals or exceeds 85% of helix pitch revolution at time of final torque measurement.
- Helix plates have generally circular edge geometry.

Non-conforming systems may also seek an ICC-ES product evaluation, but must undergo additional product testing.

Foundation Supportworks' helical piles feature plates manufactured with a helix shape conforming to the geometry criteria of ICC-ES AC358. Conversely, plates that are not a helix shape are often formed to a "duckbill" appearance. These plates create a great deal of soil disturbance, do not conform to the helix geometry requirements of ICC-ES AC358, and their torque to capacity relationships are not well documented.

The helix plate diameter, thickness and cut are selected based upon the soil and load conditions for the project. FSI currently offers:

- Helix plate diameters ranging from 6 inches to 16 inches
- Helix plate thicknesses of 5⁄16 inch, ⅜ inch, and ½ inch
- Plate steel yield strengths of at least 50 ksi (Grade 50).

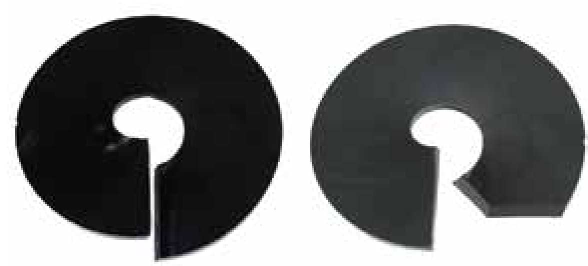


Figure 2.3.1.c
Standard H-style and V-style plates

- Standard H-style cut and V-style cut plates *(Figure 2.3.1.c)*. V-style plates are special order to assist in penetrating dense or rocky soils. The leading edges of all helix plates are sharpened (cut) to a 45-degree angle.

2.3.2 Central Shaft

The central shaft of a helical pile typically consists of either solid square bar or hollow round sections of tube or pipe. The shaft size is selected to (1) resist the torsional forces applied during installation and (2) transfer the axial loads applied by the structure down to the helix plates and surrounding soils. The central shaft of an installed helical pile is comprised of a lead section and extensions. The lead section includes a 45-degree bevel cut tip and one or more helix plates welded along its length. The 45-degree bevel cut tip further assists with pile advancement and penetration through the soil. Lead sections are generally fabricated in 5, 7 and 10-foot lengths. Extensions, which may include additional helix plates to provide increased pile capacity in weaker soil conditions, are used to advance the pile to the design depth, length, and/or until the desired torque is achieved. Extensions are generally fabricated in 3, 5, 7 and 10-foot lengths. Custom lead and extension lengths up to about 20 feet may also be considered to reduce or eliminate coupled connections, thereby minimizing overall product costs and improving installation efficiencies. Generally, a large track excavator would be required to provide the reach necessary to install these longer sections.

2.3.2.1 Coupler Detail

The coupler detail is yet another extremely important feature when considering helical piles and when selecting or specifying a product manufacturer. Manufacturers may advertise that they carry the same or equivalent helical shaft. However, shaft and coupler details are not consistent between manufacturers and these differences may not be readily apparent by simply reviewing marketing brochures and product capacity tables. Some manufacturers rate their products based upon the capacities of the gross section of the shaft, thereby ignoring any limitations caused by the coupled connections. For these "equivalent" products, there can be dramatic differences in material properties, tolerances, spacing of bolt holes, oversize of bolt holes, general fit-up, weld quality, etc.

Some of the more common coupler details for round shaft include external welded, external detached, internal detached, and forged and upset. External couplers utilize tube or pipe sections with an internal diameter slightly larger than the outside diameter of the central shaft material *(Figures 2.3.2.1.a1 and 2.3.2.1.a2)*. These couplers can be sized to provide tight connections that reduce angular deformation and variances from straightness. Such displacements at the couplers introduce eccentricities to the system which can significantly reduce the allowable compressive capacity of the pile, especially considering the slenderness of the more widely used shaft material (typically 3.5-inch outside diameter and smaller).

Figure 2.3.2.1.a1
FSI external welded coupler

Figure 2.3.2.1.a2
FSI external detached coupler

Internal detached couplers are made from solid round stock or tube or pipe material but with an outside diameter smaller than the inside diameter of the central shaft material *(Figure 2.3.2.1.b)*.

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Internal coupler diameters may be significantly undersized to prevent interferences with internal weld beads of the central shaft or due to the variations that are typical in wall thicknesses and inside diameters of pipe sections. Larger gaps between the inside diameter of the shaft and the outside diameter of the coupler can result in a connection with more potential for angular displacements.

Figure 2.3.2.1.b Internal detached coupler detail of FSI HP450

Forged and upset couplers are formed by heating one end of the shaft, placing this end in a form and then enlarging the end with a hammer-like tool or press *(Figure 2.3.2.1.c)*.

Figure 2.3.2.1.c Upset coupler with oversized closely-spaced bolt holes

With this method of manufacturing, it is difficult to create tight connections to strict tolerances. It is not uncommon to have ⅛ inch or more difference between the outside diameter of the shaft and the inside diameter of the upset coupler of the round shaft *(Figure 2.3.2.1.d)*.

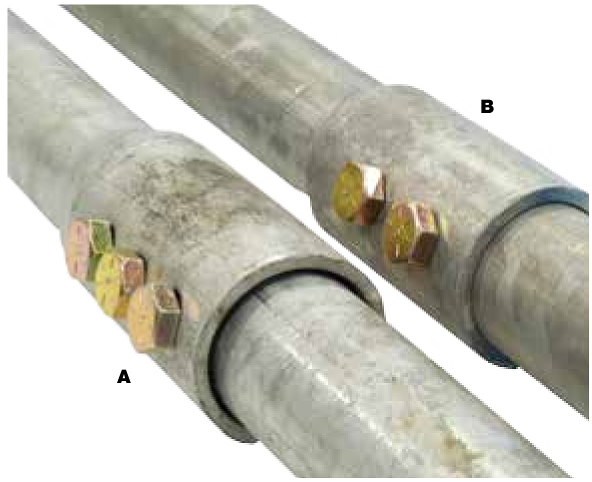

Figure 2.3.2.1.d Coupler tolerances; (A) Competitor upset coupler, (B) FSI external welded coupler

Again, the greater the freedom allowed in the connection, the greater the potential variance from straightness and the higher the potential for bending or buckling of the pile under high compressive loads *(Figure 2.3.2.1.e)*. The risk of pile buckling further increases if the pile extends through soil strata consisting of very soft clay or very loose sand, or with unsupported pile lengths through water, through fluid soils or above the ground surface.

Figure 2.3.2.1.e Competitor upset coupler variance from straightness

FSI round shaft helical piles are manufactured with external welded, external detached or internal detached couplers. Piles with shaft outside diameters (O.D.) of 2.875 inches and smaller have external welded couplers while 3.5-inch O.D. and 4.5-inch O.D. shafts have external detached and internal detached couplers, respectively. FSI offers larger diameter helical piles by special order with shaft sizes up to 12 inches. These larger diameter piles; e.g., 6.625-inch, 7-inch, 10-inch and 12-inch O.D., may be designed with external detached couplers, internal detached couplers or connections with complete joint penetration welds *(Figure 2.3.2.1.f)*.

Figure 2.3.2.1.f FSI HP700 with complete joint penetration welds between sections

All of these systems are designed and manufactured to strict tolerances to allow the pile shafts to be in direct contact when coupled, similar to *Figure 2.3.2.1.g*. Why is this important? Except for product with joint penetration welds at the couplings, the load path for piles under compression is then directly through the shafts of the extensions and lead section without having to pass through welds and bolts at each connection. The annular space between the pile shaft and coupler is also kept as tight as practical to maintain pile rigidity while also providing connections that are easily joined in the field *(Figures 2.3.2.1.h1 and 2.3.2.1.h2)*.

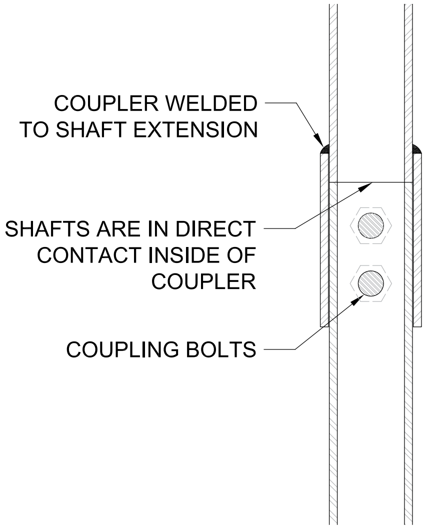

Figure 2.3.2.1.g Coupler detail showing shaft contact within coupler

Figure 2.3.2.1.h1
FSI external welded coupler

Figure 2.3.2.1.h2
FSI external detached coupler

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The most common coupler detail for solid square shaft utilizes a forged and upset end *(Figures 2.3.2.1.i1* and *2.3.2.1.i2)*. Cast detached couplers and weldments have also been used in lieu of the upsetting process. The upset end of square shaft is created in a similar manner as for the round shaft, except for forming a square socket connection. *Figure 2.3.2.1.j* clearly shows a comparison of coupling rigidity between an FSI external welder coupler for round shaft and a typical upset coupler for square shaft. A similar draping effect is typical for round shaft helical piles with upset couplers.

FSI recommends that the design engineer request product drawings and review coupling details, tolerances and general fit-up prior to product selection. As you have read in the preceding paragraphs, seemingly equivalent products may actually turn out to have very different connection details, material properties and capacities.

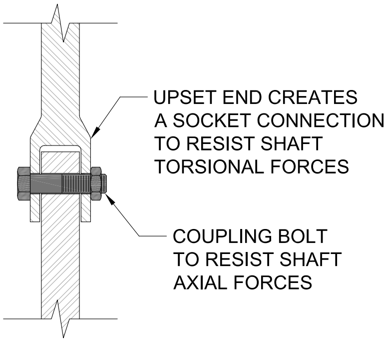

Figure 2.3.2.1.i1 Schematic of square shaft forged and upset coupler

Figure 2.3.2.1.i2
Square shaft forged and upset coupler

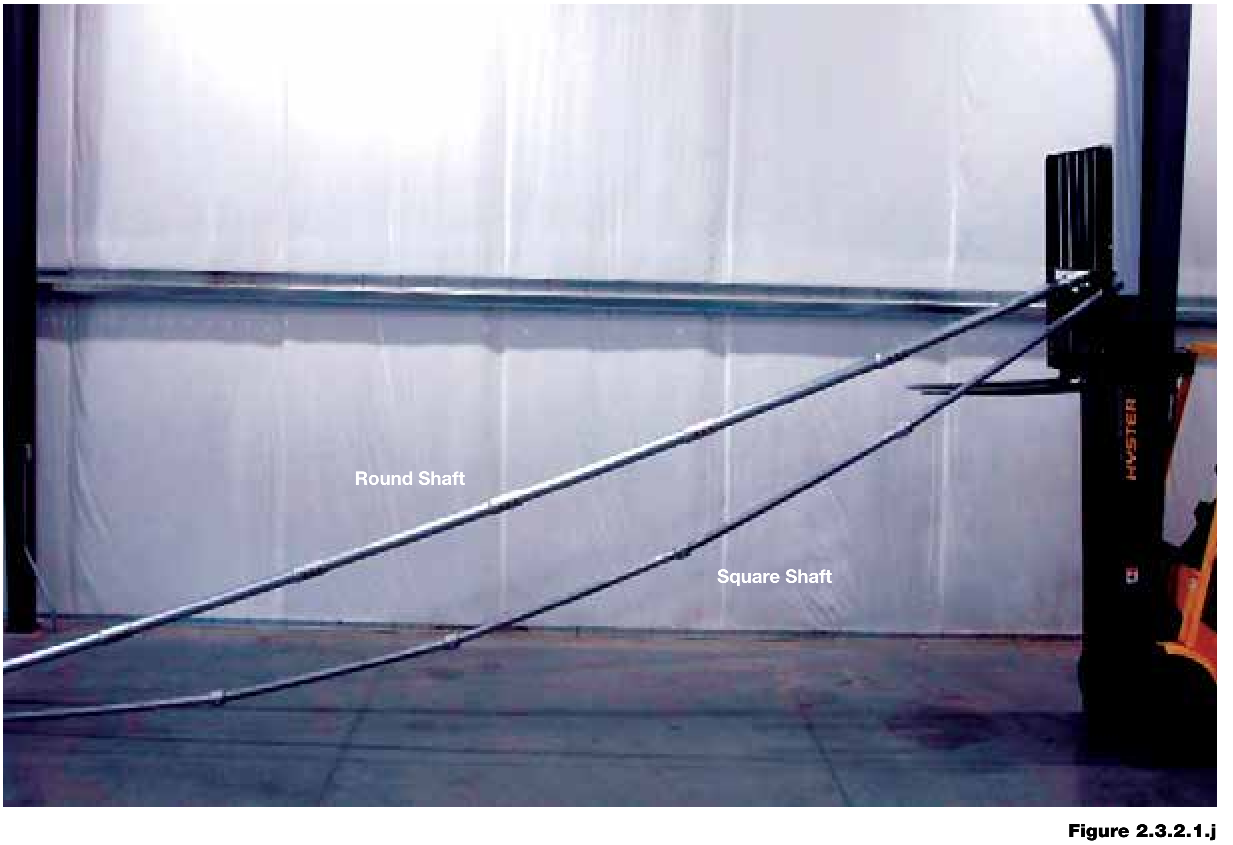

Figure 2.3.2.1.j
Coupler Rigidity Comparison: FSI round shaft external welded coupler vs. typical upset coupler for square shaft

2.3.2.2 Round vs. Square

Solid square shaft helical piles have been used successfully for decades in tension applications; i.e., as anchors, tiebacks and soil nails, and have proven to be a suitable and reliable support alternative for such projects. Not surprisingly, some manufacturers then adapted the use of square shaft helical products to be installed vertically for the support of compression loads. There is much discussion amongst design professionals and even professionals within the helical pile industry about appropriate applications for square and round shaft products. With just a little understanding of the design and manufacturing of these two systems, it quickly becomes apparent for what applications the products are better suited.

Square shaft helical piles have traditionally been used in tension applications whereas hollow round shaft piles have been used in both tension and compression. In general, FSI believes that hollow round shafts are better suited for compression whereas solid square shaft may provide some advantages in certain tension applications. That said, project parameters and site-specific soil conditions vary, which may push the merits and advantages of one system over the other, and the design professional should select the product best suited for the project. Please contact the FSI Engineering Department with any questions regarding product selection.

Hollow round shaft helical piles are particularly suited to compression loading applications and offer the following advantages over comparably-sized square shaft piles.

- Round shaft helical piles, excluding those with upset couplers, generally have more rigid coupling connections. Square shaft helical piles typically have a socket and pin coupling which increases variances from straightness, introduces eccentricity to the system, and increases buckling potential. Refer back to *Figure 2.3.2.1.j*. Square shaft piles may be considered for some light compression load applications in soil profiles that offer sufficient lateral support for these loads; e.g., Standard Penetration Test (SPT) blow count values ≥ 10 blows/foot (ASTM D1586).
- As stated in the Coupler Detail section, The FSI round shaft helical piles are designed so the pile shafts are in direct contact within the coupling connections *(Figure 2.3.2.1.g)*. The load path for round shaft piles in compression is then directly through the shafts without having to pass through the welds or bolts at each coupling. Shaft to shaft contact is more difficult to achieve within forged, upset couplers. For square shaft piles, both compression and tension loads are then transferred through the one or two coupling bolts in double shear.
- The area of steel for a round shaft is located outward from the centroid, thereby providing a greater structural section modulus and a higher moment of inertia. In layman's terms, a round shaft pile is more resistant to bending *(Figure 2.3.2.2.a)*. This is an important consideration for piles with unsupported lengths, piles penetrating loose or soft soils, or for piles that are eccentrically loaded such as in a retrofit application.

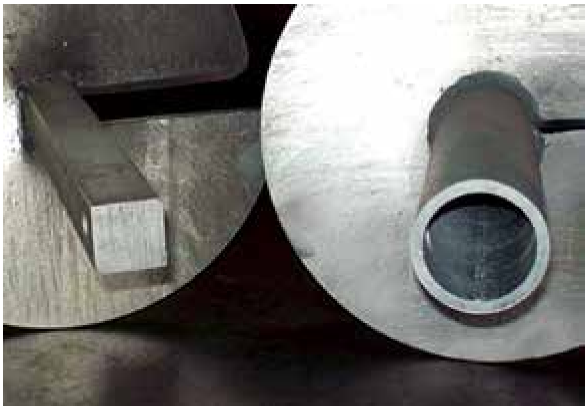

Figure 2.3.2.2.a Section comparison between 2.875" diameter, 0.276" wall round shaft and 1.5" square shaft

- Round shaft typically has a higher installation torque rating than a comparably-sized square shaft. For certain product comparisons, this results in higher pile capacities.

- Round shaft offers a higher lateral resistance with more shaft area exposed to the surrounding soil. If necessary, hollow round shafts can also be grout-filled to further improve the pile stiffness.

Solid square shaft helical piles do offer some advantages over their round shaft counterparts.

- Square shaft is a more compact section than comparably-sized round shafts and will therefore achieve greater soil penetration for a given amount of torque. This benefit is particularly important in tieback applications where the piles must be installed to certain embedment criteria as well as torque/capacity criteria.
- Square shaft, again due to its more compact shape, may penetrate through or into dense soils or soft or weathered bedrock layers more easily.
- Square shaft has less surface area exposed to corrosion and corrosion can only occur from the outside surface inward. Conversely, corrosion is possible for round shaft on both the outside and inside surfaces, although actually limited on the inside surfaces of closed pipe sections due to lack of oxygen. See Appendix 2E for additional information on corrosion.
- The degree of shaft twist may be considered as another rough indication of applied torque since permanent deformation begins within a known narrow range for each product. Contractors know they have past this threshold when the shaft twist is not recovered when the installation torque is released. **Although these observations can be used as a guide or point of reference during installation, FSI does not recommend that shaft twist be used solely as a measure or estimate of applied torque.**
- Square shaft can withstand more deformation/ twist before shaft failure. Square shaft is therefore much more forgiving during installation, allowing less experienced installers to decrease the applied torque before shaft damage may occur.

2.3.3 Brackets

A load transfer device (bracket) is used as a mechanism to transfer the structural load to the pile shaft. In **new construction** applications, a bracket; i.e., cap plate or T-cap, is welded or bolted to the top of the pile and then cast into the structural concrete, into the grade beam or pile cap. New construction brackets often consist of round shaft sleeve material with a flat plate welded to the top *(Figures 2.3.3.a1 and 2.3.3.a2)*. Steel reinforcing bars may also be welded to the sleeve or plate to further engage the concrete. In compression load applications, the new construction bracket could theoretically be set on top of the pile without welding or bolting.

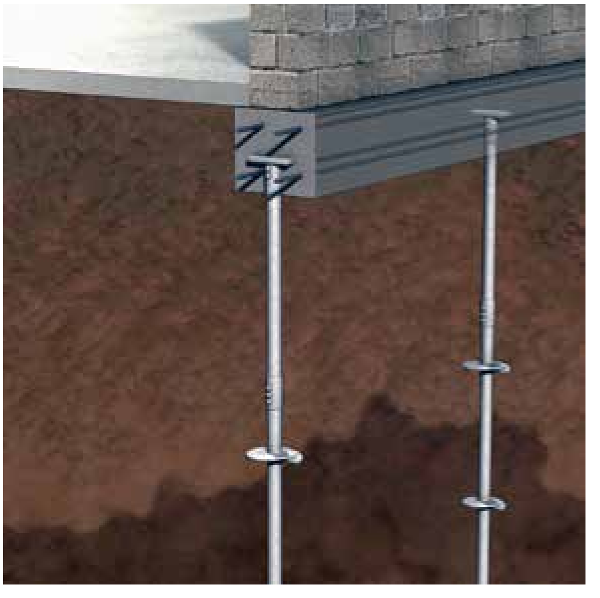

Figure 2.3.3.a1 Rendering of new construction helical piles cast into a structural grade beam

NEW CONSTRUCTION BRACKET

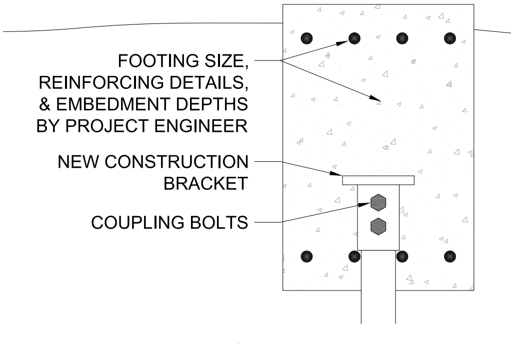

Figure 2.3.3.a2 Schematic of new construction bracket

However, FSI still recommends that a positive connection be made so the bracket is not lifted or floated off the top of the pile during concrete placement operations. Welding or bolting of the bracket to the helical pile is required to resist tension loads.

Figure 2.3.3.b1
Rendering of retrofit helical piers

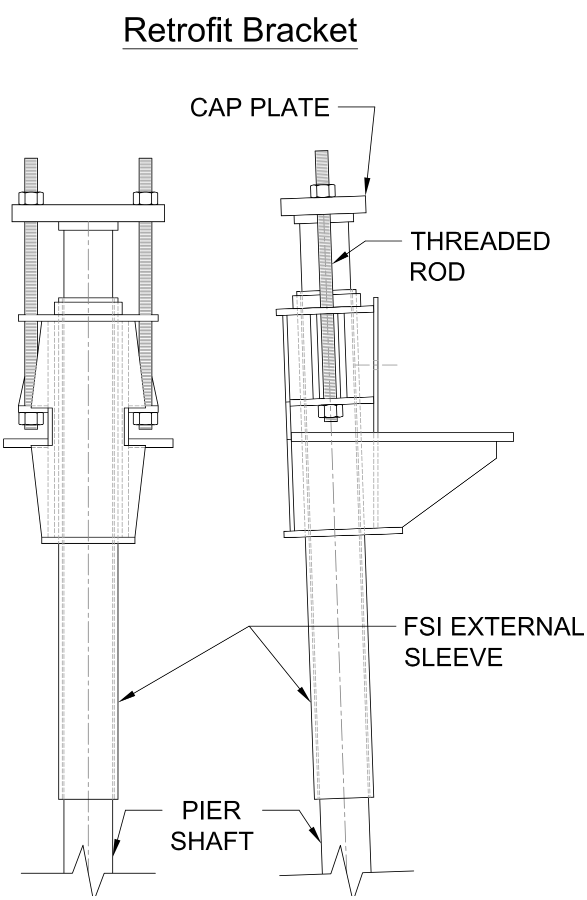

Figure 2.3.3.b2
Retrofit bracket detail

Retrofit brackets are used for **underpinning existing structures**. These brackets are often referred to as side-load or “L” brackets and are typically designed to support the foundation from below *(Figures 2.3.3.b1 and 2.3.3.b2)*. The horizontal leg of the “L” is positioned below the footing or foundation wall while the vertical leg is positioned against the vertical face of the footing or foundation wall. Footings that extend beyond the face of the foundation wall are typically notched-out at the bracket locations to create a smooth, flat surface and so the bracket is positioned as far as practical below the wall. Helical piers with retrofit brackets are often used to re-support existing structures that have undergone settlement. These same retrofit systems can be used to support additional loads transferred to an existing structure due to a building renovation or construction of an adjacent addition.

Figure 2.3.3.c
Rendering of helical tieback installation

Wall stabilization, earth retention, or embankment stabilization projects often utilize helical tiebacks or helical soil nails as system components *(Figure 2.3.3.c)*. Helical tiebacks and helical soil nails may consist of either hollow round shaft or solid square shaft, although square is more common due to its socket-and-pin style coupling (quick connection) and the ability to penetrate further into the soil with a similar amount of installation torque than a comparably-sized round shaft. The end of the shaft is typically fitted with an adaptor to transition the shaft to threaded rod.

Plate brackets can be cast into the concrete of a poured concrete wall or mounted to the face of an existing concrete wall, sheet pile wall, or soldier beam and lagging wall. Waler beams may also be considered to more uniformly spread the tieback or soil nail load to the wall *(Figure 2.3.3.d)*.

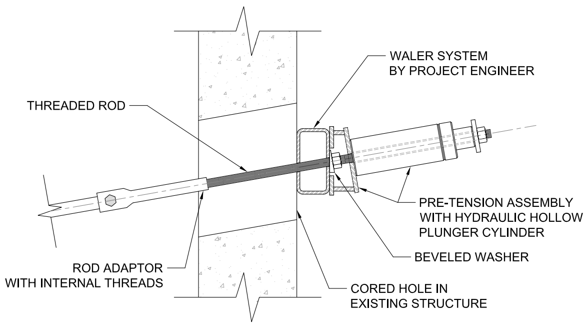

Figure 2.3.3.d Schematic of helical tieback and pre-tension assembly

Foundation Supportworks recommends that all helical anchors and tiebacks (excluding soil nails) be pre-tensioned or proof-tested following installation. Pre-tensioning to 1.0 to 1.33 times the design working load minimizes deflection of the tiebacks and structure as the tiebacks are put into service and the soil strength around the helix plates is mobilized. Tiebacks installed to support existing walls are typically locked off at 0.75 to 1.1 times the design working load after proof-testing. Helical anchors and tiebacks to be cast into new concrete retaining walls may be completely unloaded, or locked off with a modest seating load, after proof-testing. Tiebacks can be "pull tested" or load tested to typically two (2) times the design working load or more to identify the ultimate system capacity, better assess soil conditions and soil/anchor interaction, and validate design assumptions and parameters. Tiebacks that undergo load testing to greater than 1.5 times the design working load, or failure, are generally considered sacrificial and should not be used as production tiebacks.

Specialty brackets may be required for certain projects. Some of the more common specialty brackets are often modified in their dimensions, material properties, material thicknesses, and/or connection details from project to project due to variations in the design loading and/or construction. Specialty brackets are available for deck supports, boardwalk projects *(Figures 2.3.3.e1 and 2.3.3.e2)*, elevated structures in high tide or hurricane-prone areas, pipe buoyancy control, guy wires, tie downs, etc. Contact FSI with any questions regarding bracket details or availability.

Figure 2.3.3.e1 Boardwalk supported on 2.375" O.D. vertical and battered helical piles

Figure 2.3.3.e2 Custom saddle bracket connected with clevis to battered helical pile

2.4 Benefits

The use of helical piles in construction continues to increase due to product and equipment versatility and the various benefits that the systems offer. Some of the benefits/advantages of helical piles include:

- **High capacity deep foundation alternative** – Allowable torque-rated capacities on the order of 60 kips may be achieved with helical shaft sizes up to 3.5 inches in diameter, as noted in ICC-ES AC358 for conforming products. Even higher capacities may be achieved with larger shaft sections.
- **Predictable capacity** – With adequate soil information and designer experience, system capacities may be estimated very closely to capacities determined from full-scale load testing.
- **Lead sections and extensions can be configured to achieve design depth and capacity** – The design professional will choose the helical pile shaft size and helix plate configuration appropriate for the soil conditions. Additional helix plates may be considered on extensions when bearing in weaker soils. Special "V-style" plates are available to assist in penetrating dense soils.
- **Well-established torque to capacity relationship** – Empirical torque factors have been established through years of product testing. Default capacity to torque ratios are listed in ICC-ES AC358 for conforming products.
- **All-weather installation** – Helical piles can be installed through inclement weather and freezing temperatures.
- **Installed in areas of limited or tight access** – Helical piles can be installed with hand-held equipment, mini-excavators, skid steers, backhoes and larger track equipment *(Figures 2.4.a1, 2.4.a2 and 2.4.a3)*. The equipment and drive heads can be sized according to the project design loads as well as site access.

Figure 2.4.a1 Skid steer installing helical piles within limited space at a substation

Figure 2.4.a2 Mini-excavator lowered by crane into excavation to install helical tiebacks

Figure 2.4.a3 Helical piles installed with hand-held equipment to support new elevator within existing school

- **Low mobilization costs** – Helical piles have in part become a popular deep foundation option because of the ability to achieve moderate to high capacities, yet be installed with smaller equipment. Mobilization costs are then much lower than other deep foundation alternatives, which in turn makes helical piles an economical solution for many projects.

- **Vibration-free installation** – Rotary installation of helical piles does not produce ground vibrations, unlike traditional driven piles or rammed aggregate soil improvement options.
- **Install quickly without generating spoils** – Helical piles do not auger soils to the surface. Therefore, there are no hauling or disposal costs for spoils similar to auger-cast piles or drilled shafts. For contaminated sites, disposal and/or treatment of disturbed material can be extremely costly or make the project cost-prohibitive. Helical piles simply pass through contaminated soils and do not bring them to the surface.
- **Support of temporary structures** – Helical piles can be removed from the ground by reversing the installation process.
- **Load tests can be conducted immediately following installation** – Installed steel piles do not require a curing period like drilled shafts or auger-cast piles. It is common to install a helical test pile and then test it later that day or the very next day. However, know that especially on clay sites or clayey sand sites, the soils will “heal” or “set up” around the shaft and helix plates over time. In general, within practical hold periods allowed by construction schedules, the longer the pile sits before testing, the higher the pile capacity for a given amount of deflection.
- **Foundation concrete can be poured immediately following installation** – Installed steel piles do not require a curing period like drilled shafts or auger-cast piles. On schedule-sensitive projects, the contractor may place reinforcing steel and pour foundation concrete directly behind the helical pile installation.
- **Clean installation** – Installation of helical piles, helical tie-backs and helical soil-nails does not include concrete or grout, thereby minimizing equipment, vehicles and mess on the construction site.

2.5 Limitations

Helical piles will not be the best-suited, most economical deep foundation option for every project or soil profile. In the same way, other deep foundation alternatives such as driven piles, auger-cast piles, drilled shafts, and drilled and grouted micropiles, have their own benefits and limitations and may be more or less suited for certain project conditions.

- Helical piles are a steel foundation that will be exposed to the atmosphere or buried within the earth. AC358 defines corrosive soil environments by: (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. In such environments, the steel can be protected with a hot-dip galvanized zinc coating or with other measures such as sacrificial anodes. A site-specific evaluation of the soil can be conducted in order to determine an appropriate level of protection. Refer to Appendix 2E for additional information about corrosion.
- AC358 is currently limited to use of helical foundation systems and devices supporting structures in IBC Seismic Design Categories A, B, or C, or UBC Seismic Zones 0, 1, or 2. Even so, helical piles have been used successfully across North America for decades and in regions considered seismically active.
- Helical piles will not easily penetrate construction debris, wood, dense gravelly soils, or soils containing large, hard fractions such as cobbles and boulders. These materials could hinder installation or cause damage to the helical pile shaft or helix plates. When such conditions exist, a thicker or larger pile shaft may be considered to resist impact loading and torque spikes. Thicker helix plates with a V-style cut could more easily penetrate dense soils and, again, resist impact loading. A solid square bar “stinger” lead section coupled immediately to round shaft extensions

may also be considered to pass through or penetrate into dense soil *(Figure 2.5.a)*. Where large obstructions are encountered, the helical piles may have to be offset from plan locations. The project engineer should first be notified to determine if other piles should be relocated or if additional piles will be required.

Figure 2.5.a Combination pile with HA175 stinger coupled immediately to 3.5" O.D. shaft

- Helical piles will not typically penetrate hard rock, defined by auger refusal by the drill rig or Standard Penetration Test (SPT) blow count values ≥ 50 blows/6 inches of sampler penetration (ASTM D1586). Helical piles may penetrate into hard clay, dense sand and soft or weathered bedrock; however, larger installation equipment is generally recommended to provide "crowd" or axial force on the pile during advancement into these soils. A square bar stinger lead section may again be considered along with the larger installation equipment.

- The slenderness of helical piles and their limited exposed area to the surrounding soil does not allow for generation of high lateral capacities. In competent soils, allowable lateral capacities may range from less than 1 kip to more than 5 kips for 2.875-inch to 3.5-inch O.D. round shafts. Higher capacities may be achieved as the central shaft size of the pile increases. These capacities are typically achieved with lateral deflections of one inch or more. Where higher lateral loads are anticipated, or lower deflection criteria required, lateral loads could be resisted by (1) extending the structural concrete grade beams or pile caps deeper to take advantage of the passive resistance of the soil, (2) incorporating battered helical piles into the foundation design, (3) using structural elements in the current design, such as floor slabs with hairpin bars, or (4) incorporating other structural elements to create fully-braced conditions. Site-specific lateral load tests can be completed to document the lateral capacity to deflection relationship prior to installing production piles.

2.6 Design Considerations

2.6.1 Spacing and Depth

Helical piles are designed such that most of the axial capacity of the pile is generated through bearing of the helix plates against the soil. The helix plates are typically spaced three diameters apart along the pile shaft to prevent one plate from contributing significant stress to the bearing soil of the adjacent plate. Significant stress influence is limited to a "bulb" of soil within about two helix diameters from the bearing surface in the axial direction and one helix diameter from the center of the pile shaft in the lateral direction. Each helix plate therefore acts independently in bearing along the pile shaft *(Figure 2.6.1.a)*. Helical piles designed with helix plate spacing in accordance with AC358 could therefore use either the Individual Bearing or Cylindrical Shear Methods of calculating capacity. Helical piles manufactured with more closely-spaced helix plates should consider the Cylindrical Shear Method only. These design methods are presented in Section 2.7.

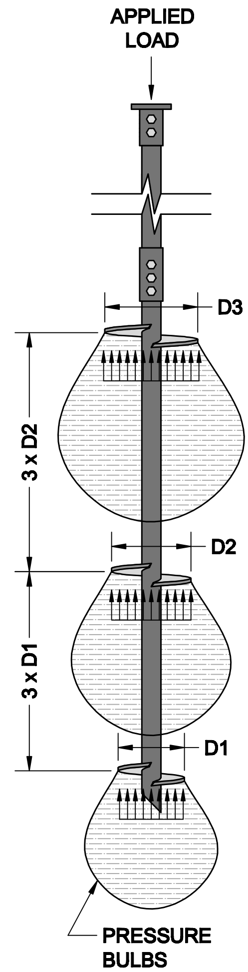

Figure 2.6.1.a Helix plate spacing with bulbs of significant stress influence

Axially loaded helical piles shall have a center to center spacing at the helix depth of at least three (3) times the diameter of the largest helix plate to avoid group efficiency effects (ICC-ES AC358). The tops of the piles may be closer at the ground surface, but the piles be installed at a batter away from each other in order to meet the spacing criteria at the helix depth.

The center to center spacing of laterally loaded piles shall be at least eight (8) times the diameter of the pile shaft at the ground surface and four (4) times the diameter of the largest helix plate measured at the plate depths (ICC-ES AC358). If both of these criteria are not met, an analysis should be completed to determine if there should be a reduction in the lateral capacity per pile.

For tension applications, the uppermost helix plate shall be installed to a depth at least twelve (12) diameters below the ground surface (ICC-ES AC358). Default torque correlation factors (capacity to torque ratios) have been verified for conforming systems tested and evaluated in tension applications at and below these depths. Design professionals may still determine that shallower installations are appropriate for the project given the site-specific soil conditions.

The uppermost helix plate shall be embedded in the ground to a depth of at least five (5) diameters to create a deep foundation bearing condition. The upper helix plate shall also be located below the depth of seasonal frost penetration and below the "active zone"; i.e., the depth of soil that undergoes seasonal volume changes with changes in moisture content. The depth of the uppermost helix plate would therefore be determined from the greatest of these values.

2.6.2 New Construction vs. Retrofit

New construction helical piles are generally designed to be concentrically loaded; i.e., the load is transferred axially down the pile shaft without inducing bending. These piles are commonly installed longitudinally along a grade beam and directly below the wall load, or multiple piles may be incorporated into a rigid pile cap to support and balance a column load. New construction piles that are concentrically loaded will behave purely as columns and will be capable of supporting loads up to the maximum allowable mechanical capacity per AISC design methods. The maximum allowable mechanical capacity should consider the bracket capacity, the shaft and coupling capacity, and the helix plate capacity. The connection to the structure must also be designed appropriately with proper pile head embedment in the concrete, concrete strength, reinforcing steel, etc. Consideration of the maximum allowable mechanical capacity assumes that the soil is also capable of supporting the load and that the shaft is laterally supported or braced along its entire length. In practice, the maximum allowable mechanical capacity of the pile is seldom achieved as the pile capacity is typically limited by soil strength.

Helical piles used in retrofit applications utilize side-load brackets that introduce eccentricity to the system. The pile shaft is not located directly under the footing or structural load. Therefore, retrofit piering systems are eccentrically loaded and must be designed to resist the bending forces generated by this loading condition *(Figure 2.6.2.a)*.

Most helical piles, especially in retrofit applications, have outer dimensions of 3.5 inches or less. These sections are therefore very sensitive to the bending moments introduced by this eccentricity, thereby reducing the capacity of the pier to carry axial load. The retrofit pier does not act as a pure column as in a new construction application, but rather as a beam-column that must resist both axial load and bending. Herein lies the problem. The pier shaft has quantifiable axial and bending capacities, and independent of the other, may be significant. However, when both of these forces are applied concurrently to the same section, both the allowable compressive capacity and allowable bending capacity are reduced. In fact, according to AISC design methods, the allowable compressive capacity may be reduced by one-half or more for certain pile sections when applying a bending moment generated by an eccentricity of only two inches, which is less than what would be considered typical for most retrofit piering systems.

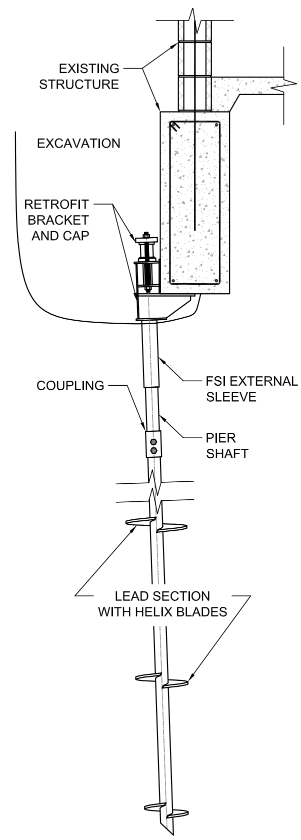

Figure 2.6.2.a
Schematic of retrofit helical pier installation

Foundation Supportworks addresses the issue of retrofit helical pier eccentricities either of two ways. The first is to increase the stiffness of the pier system and then allow more of the resulting bending forces to be transferred through the pier system itself. This is accomplished by incorporating an external sleeve to resist the bending forces. The external sleeve extends through and below the foundation bracket to essentially create a bracket that is 30 inches tall. Since the external sleeve and the pier shaft are confined by the earth, the bending moment dissipates quickly into the surrounding soils and generally within the first few feet. The depth at which the bending moment dissipates is a function of the soil strength and is greater in soft soils and less in stiff soils. With the external sleeve present to resist most of the bending forces, the capacity of the pier section is preserved to resist the axial compressive forces.

The second way to address retrofit helical pier eccentricities is to increase rigidity of the bracket connection to the foundation. With an adequately designed rigid connection, much of the eccentricity is transferred back to the foundation and less to the pier section. This connection detail typically consists of several strategically-located, deeply embedded adhesive anchors. For example, the FSI HP350B helical bracket is attached to a foundation with six (6) ⅝-inch adhesive anchors embedded 7.5 inches.

2.7 Helical Bearing Capacity Design Overview

There are three common methods for predicting helical pile capacity; the **individual bearing method**, the **cylindrical shear method** and the **torque correlation method**. The first two methods are rooted in traditional geotechnical methodology, slightly modified with empirical data. The individual bearing and cylindrical shear methods are generally used to calculate or estimate the pile capacity during the design phase. The individual bearing method relies on each helix plate to act independently in bearing with no overlap of significant stress influence between adjacent helices. The cylindrical shear method is applicable for multi-helix piles and assumes that the top or bottom helix plate acts in bearing (depending upon direction of loading) and a cylindrical shear surface develops between the top and bottom helix. The helical pile designer must have adequate subsurface information or a thorough knowledge of the local soil conditions in order to select the geotechnical parameters for use in these design equations.

The torque correlation method is fully empirical and generally used to confirm or verify capacity during field installation. The torque correlation method uses the linear relationship between installation torque and capacity; i.e., the capacity is calculated as the product of the installation torque and an empirical torque factor established through decades of full scale load testing. The torque correlation method has even been used on projects with insufficient soil information as the sole determination of pile capacity. However, there are increased risks with relying on this method alone due to potential weak soil layers that may be present below the bottom of pile elevation.

Foundation Supportworks recommends that subsurface information be determined to a depth of at least 5 to 10 feet below the anticipated helical pile depth. Soil borings should be extended into competent bearing soils capable of supporting the design working

loads with an adequate factor of safety. Helical test probes may also be considered to back-calculate the soil shear strength from the pile installation torque determined from calibrated equipment. Helical test probes should be extended to depths at least 10 feet below the anticipated depths of the helical production piles. Refer to "Geotechnical Investigation Guidelines for Helical Pile, Helical Anchor and Push Pier Design" in Appendix 2G for additional information.

The helix plate spacing along the pile shaft can control whether a helical pile acts in individual bearing or cylindrical shear. Closely spaced helix plates will exhibit cylindrical shear behavior while well-spaced helix plates will typically fail the soil in individual bearing. Research has shown that the transition between cylindrical shear and individual bearing generally occurs at helix spacings of 2.5D to 3.5D, where D is the diameter of the lead helix plate. Within that range, either method may be considered applicable. Foundation Supportworks' helical piles, tiebacks and soil nails are generally manufactured with helix plate spacings of 3.0D.

The individual bearing method essentially utilizes the traditional bearing capacity equation introduced by Carl Terzaghi in 1943 to determine the bearing capacity of shallow spread footings. This method is also used to determine the end bearing capacity of deep foundations. The other two capacity prediction methods (cylindrical shear and torque correlation) were developed specifically for helical piles used in tension load applications. These methods were then later considered to predict compression capacity as well. The use of the cylindrical shear method and torque correlation method for compression capacity determination may then be considered conservative since at least one helix plate (bottom plate) is bearing against undisturbed soil, while in tension applications, all helix plates are bearing against partially disturbed soil.

A factor of safety of 2.0 is typically used to calculate the allowable soil bearing capacity of a helical pile if torque is monitored during the helical pile installation. Higher or lower factors of safety may also be considered at the discretion of the helical pile designer or as dictated by local code requirements. Lower factors of safety may be considered for non-critical structures or temporary applications. Higher factors of safety may be considered for critical structures, structures sensitive to movement, or where soil conditions suggest that creep movement may be a concern. Total stress parameters should be used for short-term and transient load applications and effective stress parameters should be used for long-term, permanent load applications.

Like other deep foundation alternatives, there are many factors to be considered in designing a helical pile foundation. Foundation Supportworks recommends that helical pile design be completed by an experienced geotechnical engineer or other qualified design professional.

HELICAL FOUNDATION SYSTEMS **CHAPTER 2**

2.7.1 Individual Bearing Method

The individual bearing method (Adams and Klym 1972; Hoyt and Clemence 1989) states that the ultimate pile capacity is equal to the sum of the individual helix plate capacities. Spacing of the helix plates along the shaft is generally 3 times the diameter of the leading plate, the uppermost helix plate is embedded to a depth of at least 5 diameters, and skin friction along the shaft is generally ignored for shaft sizes less than 6 inches in outside diameter. *Figure 2.7.1.a* illustrates the load transfer mechanism for the individual bearing method in compression loading.

Helical pile capacity by the individual bearing method can be calculated from:

$$\mathbf{Q_u = \sum A_h(cN_c + q'N_q + 0.5\gamma BN_\gamma)}$$

Where,

$\mathbf{Q_u}$	= Ultimate Pile Capacity (lb)
c	= Cohesion at Helix Depth (lb/ft^2)
q'	= Effective Vertical Overburden Stress at Helix Depth (lb/ft^2)
$\boldsymbol{\gamma}$	= Soil Unit Weight (lb/ft^3)
B	= Diameter of Helix Plate (ft)
$\mathbf{A_h}$	= Area of Helix Plate (ft^2)
$\mathbf{N_c, N_q, N_\gamma}$	= Dimensionless Bearing Capacity Factors

The last part of the equation that includes the helix diameter (B) is often ignored in the calculation of end-bearing capacity of deep foundations. The diameter or width of the pile is relatively small and therefore this portion of the equation contributes little to the overall pile capacity. With that portion of the equation conservatively ignored, the equation further simplifies to:

$$\mathbf{Q_u = \sum A_h(cN_c + q'N_q)}$$

For purely cohesive soils with $\Phi = 0$ and $c = s_u$ (soil undrained shear strength), $N_c \approx 9$ and $N_q = 1$. The equation can conservatively be rewritten again as:

$$\mathbf{Q_u = \sum A_h(9c)}$$

For purely granular (frictional) soils with c = 0, the equation can be rewritten as:

$$\mathbf{Q_u = \sum A_h(q'N_q)}$$

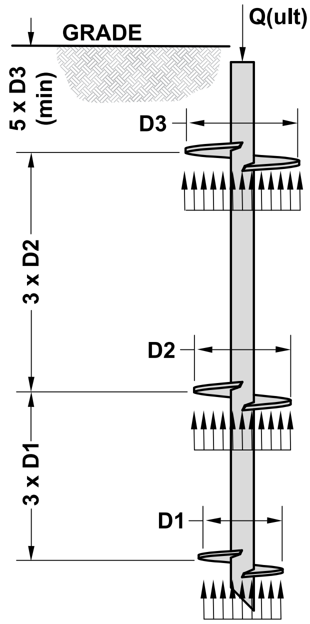

Figure 2.7.1.a
Individual Bearing Method

Bearing capacity factors N_c and N_q are typically provided in foundation design textbooks and these values may not be appropriate for use in helical pile design. Research has shown that N_q may not only be a function of the soil friction angle, but also pile embedment depth, pile type and installation method (drilled, driven, etc.). Unfortunately, there has been little research to investigate how N_q might vary for helical piles. Since helical piles are generally considered low-displacement to displacement piles due to the helix plates and shaft, one could theorize similar N_q values as determined by Meyerhof (1976) for driven piles, with a reduction to account for soil disturbance created by the helix plates. Foundation Supportworks recommends N_c and N_q bearing capacity factors calculated by the following equations and shown graphically in *Figure 2.7.1.b*:

$$\mathbf{N_c = (N_q - 1)\cot\Phi \geq 9}$$

$$\mathbf{N_q = 1 + 0.56(12\Phi)^{\Phi/54}}$$

These values of N_c and N_q are slightly lower and therefore more conservative than the values typically provided in textbooks.

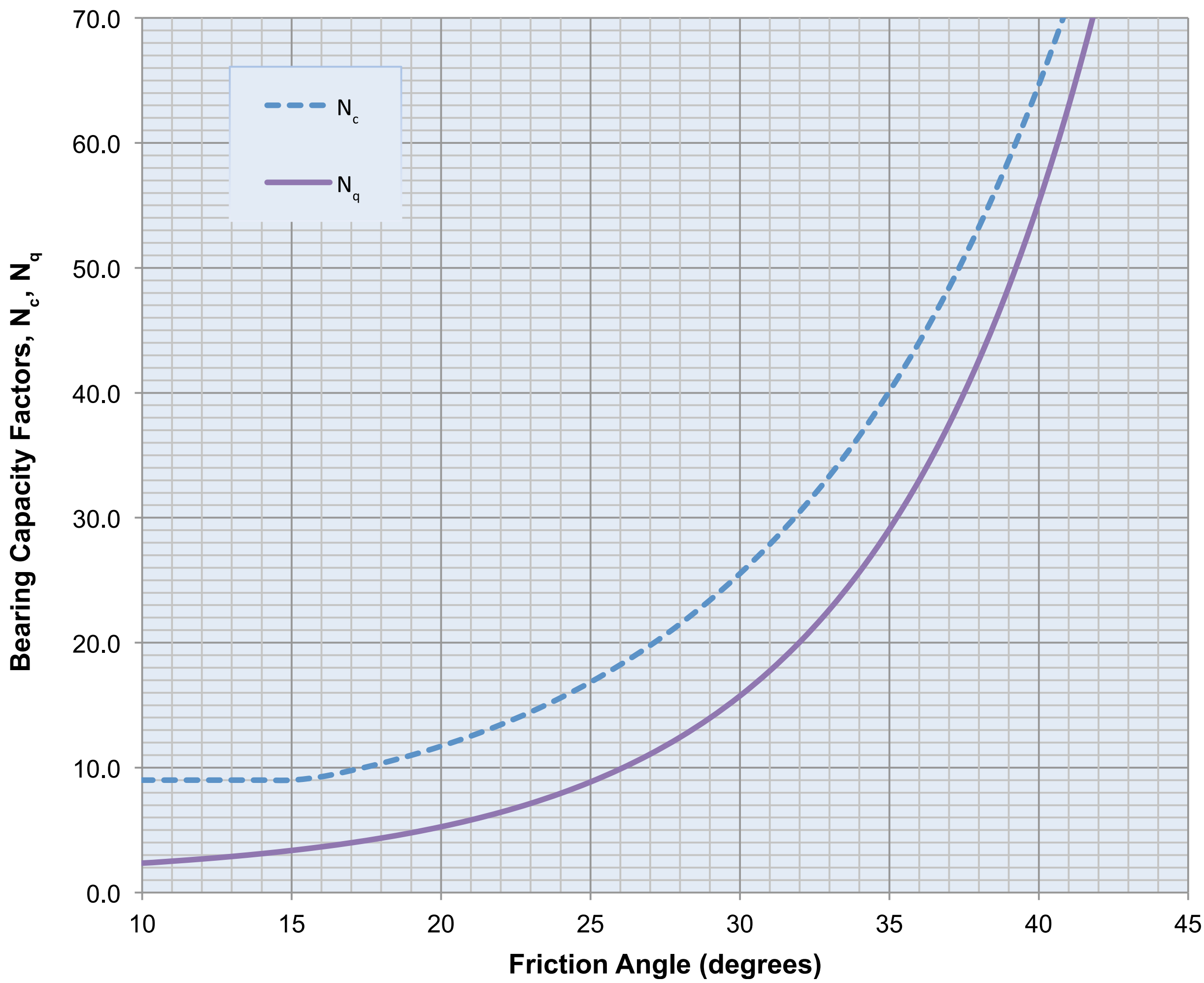

Figure 2.7.1.b Recommended Bearing Capacity Factors N_c and N_q versus Soil Friction Angle

2.7.1.1 Critical Depth

In granular soils the helical pile capacity is largely a function of the vertical effective overburden stress at the helix plate depth. Therefore, one may expect that the pile capacity would increase without bound as the effective stress increases with increasing pile depth. According to the Individual Bearing and Cylindrical Shear Method equations, the helical pile capacity should increase by simply extending the pile deeper into granular soils. In reality, there is a critical depth within uniform granular soils where a further increase in vertical effective stress results in little to no increase in the end bearing capacity of the pile. Certainly, if the strength of the granular soil increases with depth, you would expect an increase in pile capacity, but not due to an increase in the overburden stress. This concept is well-documented in many foundation design textbooks and design manuals.

Critical depth may range from 10D to 40D (where D is the largest helix plate diameter), depending upon the relative density and position of the water table. FSI recommends critical depths of 20D to 30D be considered for design purposes. For example, if the helix plate depth is greater than an assumed critical depth of 20D, limit the vertical effective stress at the helix plate to that value corresponding to the critical depth of 20D.

2.7.2 Cylindrical Shear Method

The design equation for determining helical pile capacity by the cylindrical shear method was originally developed by Mitsch and Clemence (1985) and later modified for simplicity. The cylindrical shear method assumes the development of a soil friction column (cylinder) between the upper and lower helix plates along with individual bearing of either the upper or lower helix, depending upon loading direction. The ultimate bearing capacity is then determined by the summation of shear strength of the soil cylinder, shaft adhesion/friction and end bearing of either the upper or lower helix. For deep cylindrical shear failure to occur, spacing of the helix plates along the shaft is generally less than or equal to 3 times the diameter of the leading plate and the uppermost helix plate is embedded to a depth of at least 5 diameters. Skin friction along the shaft is generally ignored for shaft sizes less than 6 inches in outside diameter. *Figure 2.7.2.a* illustrates the load transfer mechanism for the cylindrical shear method in compression loading.

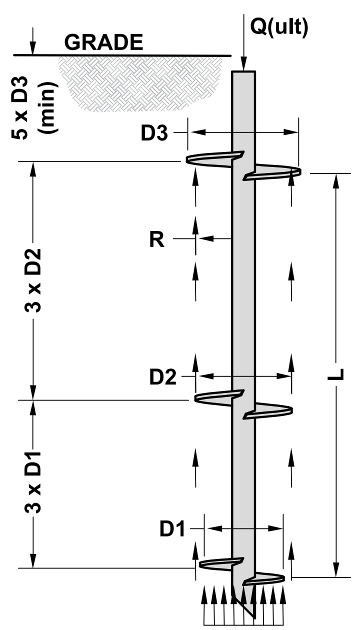

Figure 2.7.2.a
Cylindrical Shear Method

The helical pile capacity by the cylindrical shear method can be calculated as:

$$Q_u = 2\pi RL(c+K_o q'\tan\Phi)+A_h(cN_c+q'N_q)$$

Where,

- Q_u = Ultimate Pile Capacity (lb)
- R = Average Helix Radius (ft)
- L = Total Spacing Between All Helix Plates (ft)
- c = Cohesion at the Helix Depth (lb/ft^2)
- K_o = Dimensionless At-Rest Earth Pressure Coefficient
- Φ = Soil Friction Angle (degrees)
- A_h = Area of the Top or Bottom Helix Plate (ft^2)
- q' = Effective Vertical Overburden Stress at A_h (lb/ft^2)
- N_c, N_q = Dimensionless Bearing Capacity Factors

Refer to Section 2.7.1 for discussions regarding Bearing Capacity Factors and Critical Depth.

Based upon previous research, the individual bearing method and cylindrical shear method should provide similar results at helix spacings of 2.5D to 3.5D. FSI promotes the use of the individual bearing method for determination of pile capacity due to its relative simplicity and since the original form from which this method is derived is already widely accepted by the geotechnical engineering community.

2.7.3 Torque Correlation Method

The torque correlation method has become a well-documented and accepted method for estimating or verifying helical pile capacity during installation. In simple terms, the torsional resistance generated during helical pile installation is a measure of soil undrained shear strength and can be related to the bearing capacity of the pile with the following equation:

$$Q_u = K_t \times T$$

Where,

Q_u = Ultimate Pile Capacity (lb)

K_t = Empirical Torque Correlation Factor (ft^{-1})

T = Final Installation Torque (ft-lb)

The relationship between installation torque and helical pile capacity was generally considered proprietary information by helical foundation manufacturers until the results of an extensive study performed by Hoyt and Clemence were released in the late 1980s (Hoyt and Clemence 1989). The Hoyt and Clemence study included tension load test results for 91 multi-helix piles at 24 different sites with varying soil conditions, embedment depths, shaft sizes, helix spacings and number of helices. The helix plate spacing along the pile shafts varied from 1.5D to 4.5D and the number of helices varied from two to 14 with the diameters ranging from 6 to 20 inches. Shaft sizes consisted of 1.5, 1.75 and 2.0-inch square and 3.5 and 8.625-inch round. The load test results were compared with capacity predictions using the torque correlation method, the individual bearing method and the cylindrical shear method (Mitsch and Clemence 1985). The statistical results of this study show that the torque correlation method is the more precise predictor of capacity of the three methods. The researchers recommended torque correlation factors (K_t) of 10 ft^{-1} for all size square bar shafts and round shafts less than 3.5 inches in diameter, K_t of 7 ft^{-1} for 3.5-inch diameter round shafts and K_t of 3 ft^{-1} for 8.625-inch diameter round shafts. It must be recognized that the recommended K_t values in the Hoyt and Clemence paper were based on a wide range of soil conditions and pile configurations (configurations that may not be considered as conforming products per ICC-ES AC358) and should only be used with confirmation from site-specific, full-scale load testing. Some of the recommended Hoyt and Clemence K_t values differ from the default values provided in ICC-ES AC358.

ICC-ES AC358 recognizes the following helical pile shaft sizes and default K_t factors for conforming systems, since the installation torque to capacity ratios have been established with documented research:

- 1.5 and 1.75-inch solid square — $K_t = 10\ ft^{-1}$
- 2.875-inch O.D. round — $K_t = 9\ ft^{-1}$
- 3.0-inch O.D. round — $K_t = 8\ ft^{-1}$
- 3.5-inch O.D. round — $K_t = 7\ ft^{-1}$

The K_t factors above may be considered conservative for most applications, and even though they are often presented as constants, K_t can vary depending upon the soil conditions. K_t factors are generally higher in sands, gravels and overconsolidated clays, and lower in underconsolidated clays, normally consolidated clays and sensitive clays and silts. K_t is also inversely proportional to the shaft dimension/diameter as shown above.

Factors that affect installation torque may also have an effect on the resultant K_t determined from a field load test. In addition to soil type and shaft dimension, studies have indicated that other factors such as the number of helix plates, helix thickness, helix pitch, helix spacing along the shaft, helix diameter, depth of pile embedment, applied downward force during installation (crowd), and test load direction may have an effect on installation torque and/or the resultant K_t. Other studies have discounted some or most of these factors as inconsequential.

The use of uncalibrated torque monitoring equipment or uncertified gear motors will likely affect K_t determined from field load testing. The helical pile industry has long used the differential pressure across the gear motor for correlation to installation torque. The installation torque is then correlated to pile capacity. In other words, the differential pressure across the gear motor is commonly used to determine the pile capacity. The current state-of-practice involves using a gear motor multiplier (GMM) to convert from differential pressure to torque. The GMMs are provided by the gear motor manufacturers based on theoretical equations and will vary with the planetary gear ratio, motor displacement and motor efficiency. Gear motor manufacturers typically show a linear fit between the differential pressure and output torque with no scaling effect. Research has shown that the gear motor differential pressure to torque relationship is generally linear, however, there is a scaling adjustment needed (Deardorff 2007). This results in a range of GMMs from low to high differential pressure. The discrepancy between actual installation torque and torque determined by correlation to differential pressure is highest at low differential pressures. This difference often decreases steadily as the differential pressure increases up to the point of maximum motor efficiency. Therefore, it is highly recommended that gear motors be certified on an annual basis, or whenever changes occur to alter their performance, in order to establish their true differential pressure to torque relationship. Calibrated in-line torque monitoring devices may also be used as an alternative to having the drive motors certified.

Finally, the installation practices of the specialty contractor and the quality control of the helical pile manufacturer will affect K_t. Helical piles should ideally be installed at a rate equal to the pitch of the helix plate (3 inches per revolution) with no more than 25 revolutions per minute (rpm). The installation rate should be reduced to about 10 rpm during final seating of the helical plates. The rate of advancement can be controlled by the installing contractor by adjusting the speed and downward force (crowd) as different soil layers are encountered and penetrated. The helical pile manufacturer should provide a helix plate geometry that is a true ramped-spiral with uniform pitch. The geometry of the helix plate is instrumental in providing the downward thrust or pull into the ground and should be controlled to increase the installation efficiency and subsequent K_t. Refer to Section 2.3.1 for an in-depth discussion about helix plate geometry. Proper installation procedures and well-formed helix plates are critical to minimize soil disturbance.

2.8 Helical Tiebacks

Helical anchors/tiebacks are commonly used in tension applications to provide either temporary or permanent lateral or tie-down support for applications including:

- Earth retention systems such as concrete retaining walls, soldier pile and timber lagging, and sheet piling *(Figures 2.8.a1 and 2.8.a2)*

Figure 2.8.a1 Rendering of helical tieback installation for soldier pile and timber lagging wall

Figure 2.8.a2 Multi-tier helical tieback installation to support sheet pile wall

- Seismic loading restraint for foundation uplift and lateral support systems
- Guy anchor support for power line and communication towers
- Seawalls and marine bulkhead support *(Figure 2.8.b)*

Figure 2.8.b Helical tiebacks stabilize marina seawall

Helical tiebacks are manufactured with similar helix plate sizes and helix spacing as helical piles installed vertically to support foundation loads. Tiebacks differ from helical piles in that they are typically installed in a horizontal to 45-degree downward from horizontal orientation to laterally support the tops of earth retaining structures. Helix plates are typically limited to the lead section or the lead and first extension of the tieback. The helix plate design depends on the soil strength parameters and the required working capacity. Multi-helix leads generally consist of increasing plate sizes from the tip. Helical tiebacks may consist of either hollow round shaft or solid square shaft, although square is more common due to its socket-and-pin style coupling (quicker and easier to connect) and the ability to penetrate further into the soil with a similar installation torque than a comparably-sized round shaft. The end of the shaft is typically coupled to an adaptor that transitions the shaft to threaded rod. Refer back to *Figure 2.3.3.d*.

Both the individual bearing method and the cylindrical shear method are appropriate for determining helical tieback capacity. The torque correlation method is commonly used to verify capacity during tieback installation. These methods are discussed in Section 2.7.

Helical tiebacks are often used to stabilize existing earth retaining structures that have experienced excessive movement; i.e., walls that are cracked, leaning and/or bowing *(Figures 2.8.c1 and 2.8.c2)*. The wall distress may be a result of changes in soil moisture conditions, rise in groundwater levels, plugging of the wall drainage system over time, plumbing leaks, expansive clay soils, frost-jacking, or surcharge loads above the wall.

Figure 2.8.c1 Helical tiebacks stabilize sheet pile wall below historic home

Figure 2.8.c2 Helical tiebacks and tube steel walers stabilize concrete retaining wall

2.8.1 Design Considerations

The helix plates along the tieback shaft must be located beyond the active wedge or failure plane to provide proper anchorage. The last helix plate from the tip (closest to the wall) shall be at least five (5) times its diameter beyond the failure plane *(Figure 2.8.1.a)*. The helix plates should also be located at least five (5) diameters below the ground surface of the retained soils to model deep foundation behavior. Multiple tiebacks shall have a center to center spacing at the helices of at least three (3) times the diameter of the largest helix plate to avoid significant stress overlap within the bearing soils.

Helical tiebacks are often installed at a downward angle from horizontal, typically on the order of 5 to 15 degrees. This downward angle is often considered in order achieve the 5D depth criteria below the surface of the retained soils, to increase the vertical effective overburden stress at the helix depths (in granular soils), or to extend the helix plates to a deeper, more competent soil layer. A slight downward angle may also be considered to simply minimize the potential for groundwater to follow the shaft and seep through the wall penetration.

Tiebacks designed with a downward angle should be installed to a capacity higher than the calculated/required horizontal tieback capacity *(Figure 2.8.1.b)*. The calculated horizontal tieback capacity (T_{CH}) is determined from analysis considering the various loads on the wall. If the tieback is designed for an installation angle, alpha (α), then the tieback should be installed to a capacity T_R and its corresponding value of torque if the torque correlation method is used for capacity verification. ***Remember that the torque-correlated ultimate capacity should exceed the design working load by an appropriate factor of safety.***

$$T_R = T_{CH} / \cos\alpha$$

The vertical component of the tieback force should also be considered so as not to overstress the wall or the wall bearing soils. The vertical

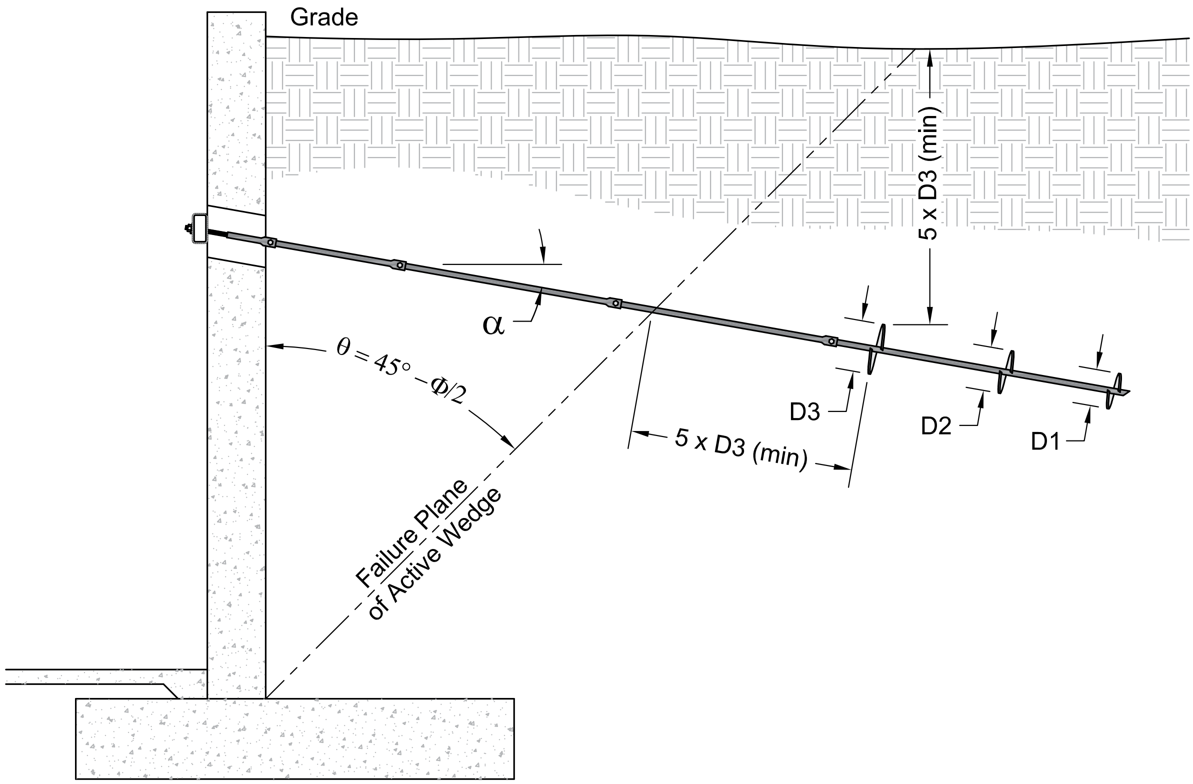

Figure 2.8.1.a Helical tieback design considerations

component of the tieback force will increase with increasing installation angle, provided the installed tieback force, T_R, is held constant. The vertical force on the wall generated by the tieback may be calculated by:

$$T_{CV} = T_{CH} \tan\alpha$$

or

$$T_{CV} = T_R \sin\alpha$$

Where,

T_R = Installed capacity of tieback at angle **α**

α = Angle of tieback installation measured downward from horizontal

T_{CH} = Calculated horizontal tieback capacity determined from wall analysis

T_{CV} = Calculated vertical load on the wall due to tieback installation

Angled or beveled washers are recommended at the tieback-to-bearing plate interface to more uniformly transfer the tension loads. Beveled washers are especially critical when the tiebacks are pre-tensioned with a torque wrench. Uniform bearing of the nut to the beveled washer to the bearing plate provides a more accurate reading of resistance and torque, which in turn is a more accurate determination of axial force on the tieback.

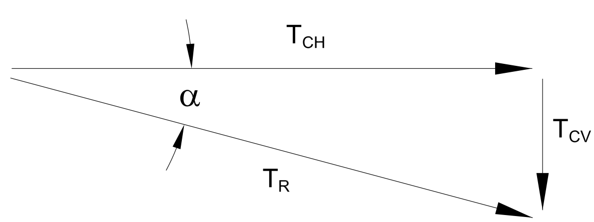

Figure 2.8.1.b
Vector mechanics of tieback forces

The design of the tieback system should consider the structural details of the wall, the wall reinforcing, present wall condition, and the effect of any penetrations necessary to install the tiebacks. Cantilevered concrete retaining walls, for example, are generally designed with significant reinforcing steel on the backfilled side of the wall where tension and bending are greatest. Reinforcing steel within the compression side of the wall is generally the minimum required by code. Tieback installation induces a negative bending in the cantilevered wall for which the wall was not originally designed. Walls to be stabilized with tiebacks, or walls that will be designed with tieback support, should be reviewed by a design professional.

The assumed failure plane behind an earth retaining wall is dependent upon soil conditions and wall type. As a general rule of thumb, a failure plane can be projected from the bottom back face of the wall upward at an angle of 45-Φ/2 (degrees) from vertical for both active and at-rest conditions. For retaining walls and basement walls, this failure plane is usually assumed to begin at the bottom of wall. For sheet pile walls, the failure plane is usually assumed to begin at the mud line.

Failure modes for restrained walls should be evaluated for internal stability, external stability, bearing capacity and global stability. It is the responsibility of the design professional of record to perform these evaluations. Typical factors of safety for helical tiebacks used in conjunction with earth retention systems are generally on the order of 1.3 to 1.5 for temporary applications and 2.0 for permanent applications.

Foundation Supportworks recommends that all helical anchors and tiebacks (excluding soil nails) be pre-tensioned or proof-tested following installation (Figure 2.8.1.c). Pre-tensioning to 1.0 to 1.33 times the design working load minimizes deflection of the tiebacks and structure as the tiebacks are put into service and the soil strength around the helix plates is mobilized. Tiebacks installed to support existing walls are typically locked off at 0.75 to 1.1 times the design working load after proof-testing. Helical anchors and tiebacks to be cast into new concrete retaining walls may be completely unloaded, or locked off with a modest seating load, after proof-testing. Tiebacks can be "pull tested" or load tested to typically two (2) times the design working load or more to identify the ultimate system capacity, better assess soil conditions and soil/anchor interaction, and validate design assumptions and parameters. Tiebacks that undergo load testing to greater than 1.5 times the design working load, or failure, are generally considered sacrificial and should not be used as production tiebacks.

Figure 2.8.1.c
Pre-tensioning helical tieback

2.9 Helical Soil Nails

Soil nailing is a method of earth retention that relies on a grid of individual reinforcing strands or members installed within a soil mass to create an internally stable gravity wall/retaining system. Soil nail wall technology began in Europe with use of the New Austrian Tunneling Method in rock formations in 1961. The technology then carried over to applications involving unconsolidated soil retention, primarily in France and Germany. Soil nail walls were first used in North America for temporary excavation support in the late 1960s and continued to gain recognition and acceptance during the 1970s and 1980s for higher profile projects including highway applications. Much of the soil nail wall research performed in North America was funded by the Federal Highway Administration (FHWA) and other state highway agencies during the 1990s. Although helical piles have been used as tiebacks since the early 1950s, helical soil nails are a relatively new alternative to their grouted counterparts.

Soil nail walls offer the following advantages over tieback walls as well as other top down construction techniques:

- Soil nail walls are more economical than conventional concrete gravity walls and are often more economical than tieback walls due to reduced wall facing requirements. There would likely be more soil nails than tiebacks for a given project, but this additional cost for the nails is outweighed by the difference in cost of a shotcrete facing versus a more substantial soldier pile, sheet pile, or reinforced concrete wall detail.
- Soil nails are typically shorter than tiebacks for similar wall heights so there will be reduced right-of-way (ROW) requirements.
- There is less impact to adjacent structures since soil nail walls are not installed with vibratory energy like soldier piles or sheet piles.
- Overhead clearance requirements are less than driven soldier pile or sheet-pile wall construction. Soil nail walls can therefore be installed easily below bridges and even within existing buildings.

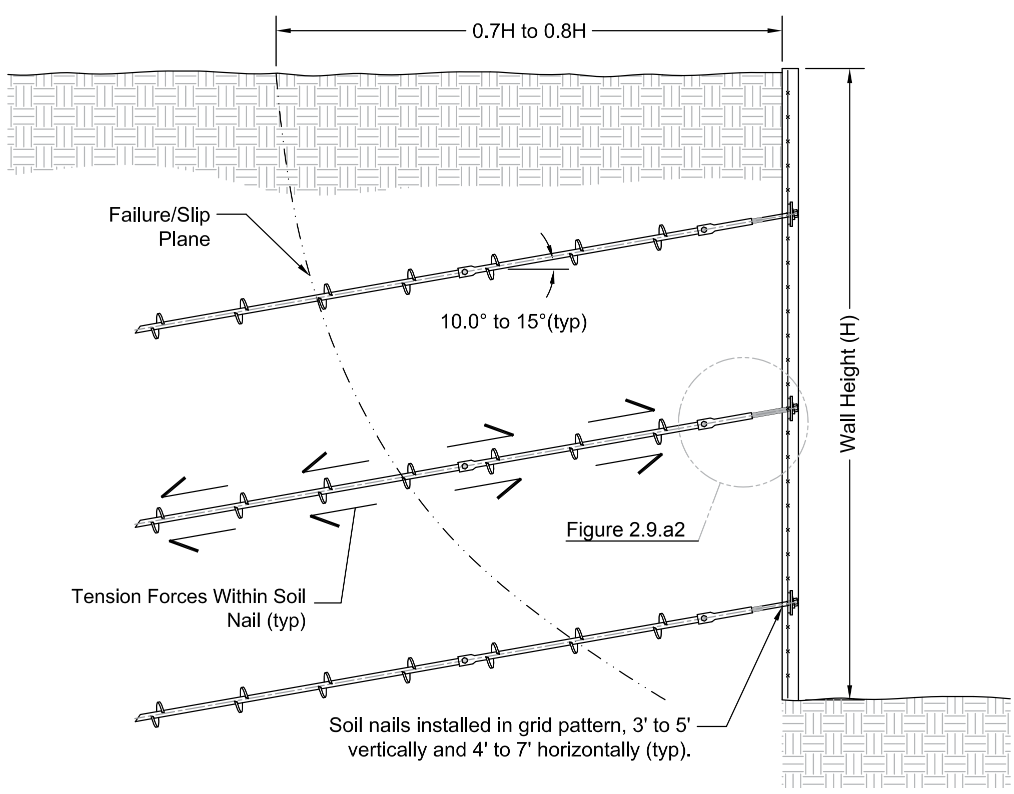

Figure 2.9.a1
Helical soil nail wall installation

- There is no need to embed structural elements below the proposed ground surface elevation on the low side of the soil nail wall. Soldier pile and sheet-pile walls require minimum embedment depths for wall stability.
- Soil nail wall construction is typically quicker than other earth retention methods.
- Soil nail walls can be constructed in remote areas with smaller equipment.
- Soil nail walls have performed well during seismic loading events due to the overall system flexibility.

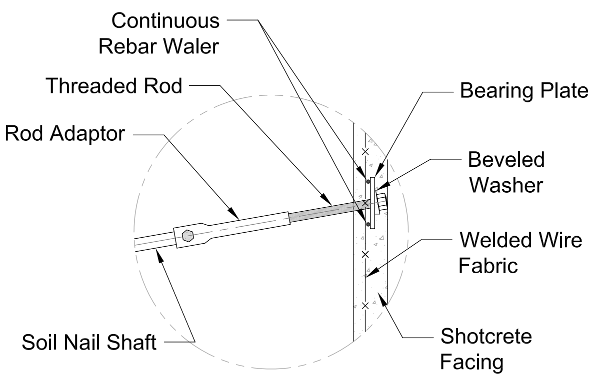

Figure 2.9.a2
Nail head to wall detail

A helical soil nail typically consists of square shaft lead and extension sections with small diameter (6 to 8 inches) helix plates spaced evenly along the entire shaft length *(Figures 2.9.a1* and *2.9.a2)*. The helical soil nail is installed by application of torque, similar to the installation of a helical tieback. The helical soil nail is a passive bearing element, which relies on movement of the soil mass and active earth pressures to mobilize the soil shear strength along the nail. In contrast, a tieback is pre-tensioned to mobilize the soil shear strength around the helix plates. Excavation, soil nail installation and application of wall facing is completed in steps from the top of the wall downward.

2.9.1 Construction Methodology

Soil nail walls are constructed from the top down where the excavation proceeds as shown in *Figure 2.9.1.a*. The construction sequence for a typical helical soil nail wall includes:

- Initial excavation about 3 to 5 feet deep depending upon design parameters and soil conditions
- Installation of the first row of helical soil nails to the required inclination angle, torque and embedment length
- Placement of drainage medium (if required)
- Placement of wall reinforcement and bearing plates
- Placement of shotcrete to the required design wall thickness
- After shotcrete has cured, repeat sequence for successive rows of soil nails. Continue process to the final design depth (wall height).

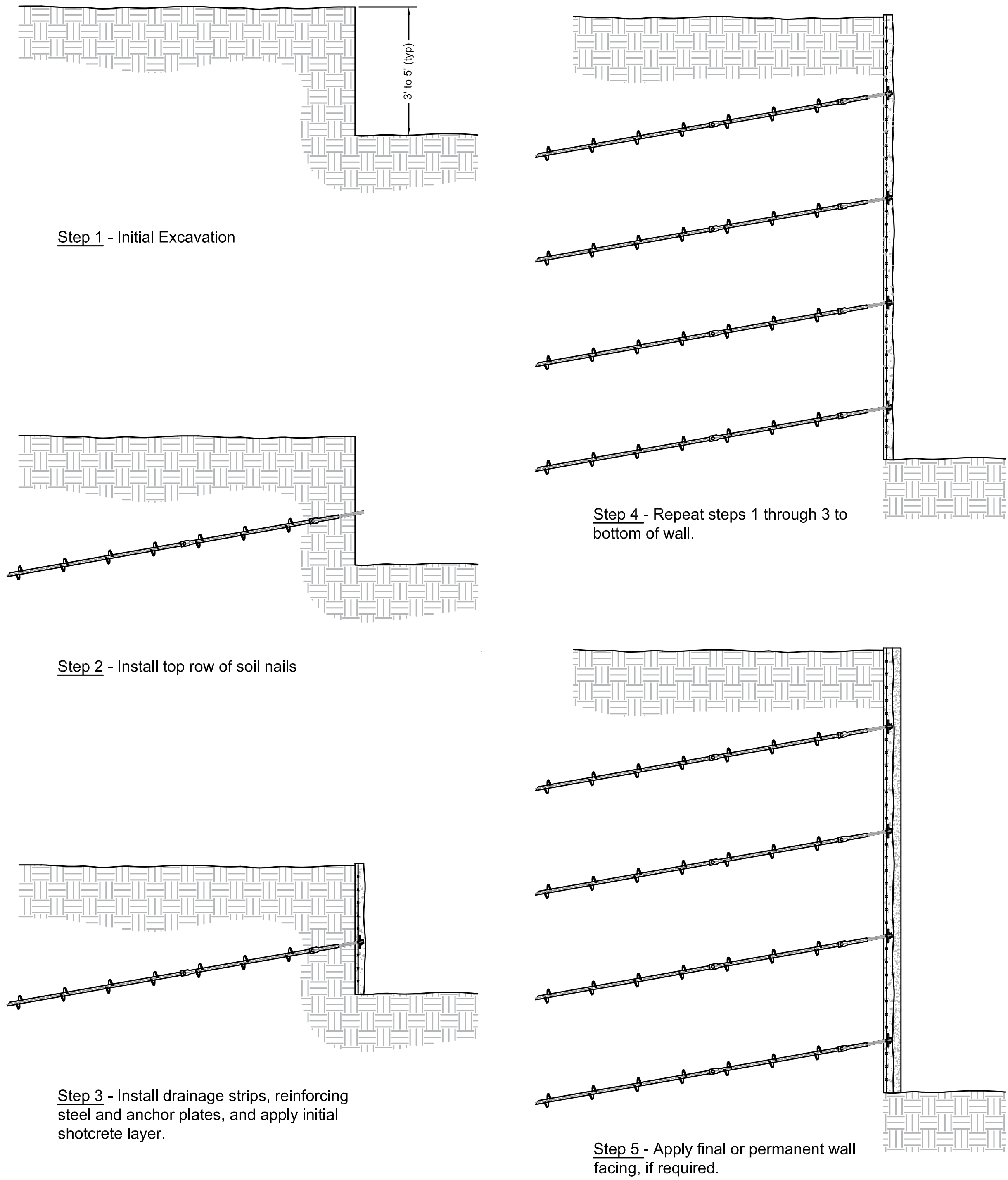

Figure 2.9.1.a Typical soil nail wall installation sequence

2.9.2 Design Considerations

Helical soil nails are passive bearing elements which rely on movement of the soil mass to mobilize the soil shear strength along the nail. As a result, soil nail walls typically experience more lateral movement than tieback walls of similar height. By allowing this movement, the highest stress in the soil nail is near the failure plane, centered between the opposing tensile forces. Conversely, the highest stress in a tieback is at the wall face. Therefore, soil nails have less nail head force than tiebacks for a similar size wall, which results in potential cost savings by using soil nails due to reduced wall thickness requirements.

The following should be considered when designing soil nail walls.

- Not all soil conditions are suitable for construction of helical soil nail walls. Excavations are generally made in 3 to 5-foot steps, depending upon soil type and strength. The soil should be able to stand unsupported for a period of at least one day after the vertical cut is made. Soil conditions that may not be favorable for helical soil nail wall construction include:
  - o Dry, poorly-graded cohesionless soils; e.g., clean sands or sands with SPT N-values less than 5 blows/foot
  - o Highly plastic clays, expansive soils, organic soils, or soils with a liquidity index of 0.2 or greater
  - o Clays with SPT N-values less than 4 blows/foot
  - o Soil profiles with high groundwater levels - dewatering may be required to facilitate installation
  - o Soil with cobbles, boulders or weathered rock lenses
  - o Highly corrosive soils
  - o Collapsible soils
  - o Very dense sands and hard clays - may be difficult to penetrate without pre-drilling a pilot hole
- A failure plane generally develops at the top of the wall at a horizontal distance of about 0.7 to 0.8 times the height of the wall away from the wall face (Lazarte, Elias et al. 2003). This distance may be reduced by battering the wall face. Any structure, utility, roadway, etc. that would be impacted by the wall movement and/or failure plane should be considered during the design phase.
- Top of wall lateral movements on the order of 0.2% to 0.3% of the wall height should be expected with soil nail lengths to wall height ratios between 0.7 to 1.0, negligible surcharge loading and a design including a global factor of safety of at least 1.5. As a general guide, the soil mass located between the failure plane and the wall facing may slump approximately ⅛-inch laterally and ⅛-inch vertically for each 5-foot depth of excavation.
- Soil nail walls may be designed with a slight batter to account for anticipated lateral wall movement.
- There may be restrictions to the design soil nail lengths, including property lines, right-of-way (ROW), underground utility corridors, bridge abutments or existing structures.
- Consider temporary and/or permanent surcharge loads from adjacent structures, roadways, construction equipment, fill placement, etc.
- Maximum wall heights for helical soil nail walls are practically limited to 20 to 30 feet. Increased heights may be considered with a stepped wall design.
- Helical soil nails are typically installed in a grid pattern, spaced 3 to 5 feet vertically and 4 to 7 feet horizontally.
- Helical soil nails are typically installed at an angle of 10 to 15 degrees downward from horizontal, although a batter in not required. The downward installation angle is a carryover from grouted nail design where an angle is required to prevent wet grout from flowing out the hole.
- Soil nails may be installed with consistent lengths for all rows, or be longer at the top of the wall, becoming shorter with successive rows toward the bottom. Nail length determination depends upon soil strength parameters, location of the failure plane, and design for critical limit states as discussed in Section 2.9.2.2.

The design procedure for helical soil nails is similar to that for grouted nails. For a helical soil nail, the bond stress with the soil is assumed to act along a cylindrical surface area defined by the outside edge of the helix plates. Bearing capacity of the soil nail is determined using the Individual Bearing Method described in Section 2.7 and is correlated to bond stress by:

$$\mathbf{q_u = Q_u / L\pi D_h FS}$$

Where,

- **q_u** = Ultimate Bond Stress (psi)
- **Q_u** = Ultimate Capacity of the Helical Soil Nail by Individual Bearing Method (lb)
- **L** = Soil Nail Length (in)
- **D_h** = Helix Diameter (in)
- **FS** = Factor of Safety for Uncertainties in Soil Conditions (Typically 1.5 to 2.0 Based on Quality of Soil Information)

As the construction of the wall progresses, the upper soil nails become less important for the stabilization of the soil mass, and depending upon wall height, may not contribute to the global stability at the final excavation phase. However, the upper soil nails are instrumental in providing stability during the early phases of excavation and contribute to limiting wall deflections. *Figure 2.9.2.a* shows the distribution of tensile force in Nail 1, cumulative wall movement and the critical failure surfaces as the soil nail wall construction progresses. The upper schematic of *Figure 2.9.2.a* illustrates the tensile force distribution along the top soil nail as construction continues through the various excavation phases. Phase N in the upper schematic does not reflect the maximum soil nail tensile force since additional loading occurs after construction to reach long term equilibrium of soil nail forces.

The design of helical soil nail walls should be performed in general accordance with

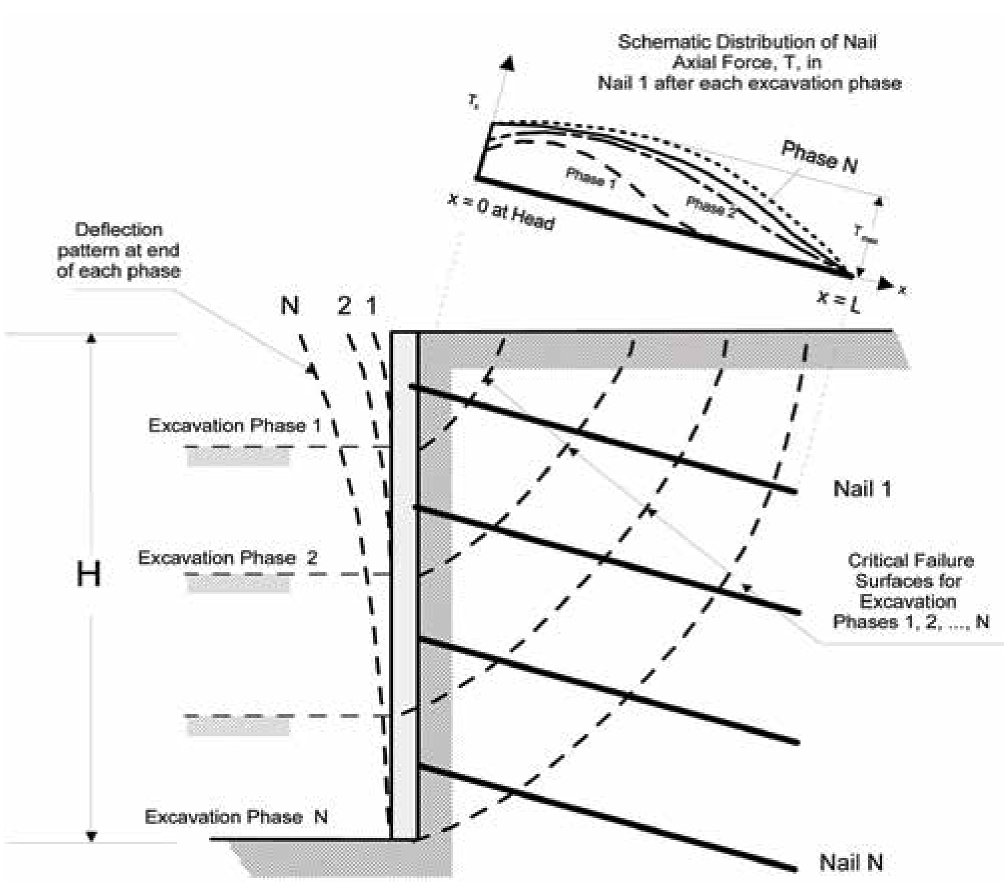

Figure 2.9.2.a Potential failure surfaces and soil nail tensile forces *(Lazarte, Elias et al. 2003)*

requirements detailed in FHWA Geotechnical Engineering Circular No. 7 (Lazarte, Elias et al. 2003). Several computer programs are available for design of soil nail walls, with the more common programs being SNAIL (CALTRANS 1999) and GoldNail (Golder 1996).

SNAIL is a Windows-based program developed by the California Department of Transportation (CALTRANS) and is available free to the public. SNAIL is a two dimensional limit equilibrium program that uses force equilibrium exclusively. Either metric or English units can be used during the design process. Soil nail reinforcement inputs include location, diameter, inclination angle, vertical and horizontal spacing, cross sectional area, yield strength and tensile strength. The soil parameter inputs include soil unit weight, cohesion, friction angle, bond strength and the bond strength reduction factor. The soil strength parameters are modeled with the conventional linear Mohr-Coulomb envelope. The only data entered for the wall design is the face punching shear, therefore an initial wall facing design must be used for the trial runs. The program allows for consideration of up to seven soil layers and provides inputs for two uniform vertical surcharge loads and an internal or external point load. The program output provides the global factor of safety, an estimated location of the failure plane and the tensile forces for each nail for each of the 10 most critical failure surfaces analyzed.

GoldNail is a Windows-based proprietary program developed by Golder Associates which satisfies both moment and force equilibrium. The program can work in one of three modes; design, factor of safety and nail service load. The program allows factored strengths for Load and Resistance Factor Design (LRFD). The soil strength parameters can be modeled with the conventional linear Mohr-Coulomb envelope or using a bi-linear strength envelope. Up to 13 soil layers can be modeled with more complex geometry capability than SNAIL. The program can only model a circular failure surface which must pass at the toe of the wall or above the toe. This limits the ability to evaluate sliding and bearing capacity failure modes. Data input variables and output reports are generally similar to SNAIL.

FHWA Geotechnical Engineering Circular No. 7 (Lazarte, Elias et al. 2003) provides design tables and charts that can be used for preliminary estimation of the wall design. The tables and charts were developed using SNAIL simulations and include the following assumptions:

- The soil is homogenous (only one soil type and strength parameter)
- There are no surcharge loads or sloped backfill conditions
- There are no seismic forces/loads
- The soil nails are of uniform length, spacing and inclination for each row
- There is no groundwater present

There should always be a final design prior to construction activities which take into consideration any deviations from the assumptions listed above and determination of the Limit States described in Section 2.9.2.2.

The design of helical soil nails should be completed by experienced design professionals. Installation of FSI helical soil nails shall be by certified FSI Installing Contractors trained specifically for helical soil nail installations. FSI recommends that the wall design follow the general guidelines detailed in the FHWA Geotechnical Engineering Circular No. 7 (Lazarte, Elias et al. 2003).

Preliminary design recommendations are available to FSI Installing Contractors to assist with costing of helical soil nail wall projects. However, the final design must be completed and/or approved by the engineer of record.

2.9.2.1 Temporary and Permanent Wall Facing

Helical soil nail walls can be categorized as temporary or permanent, and the design of the wall facing and nail head connection details will vary based upon this determination. ***Whether the soil nail wall is temporary or permanent, the wall facing and helical soil nail connection detail must be completed and/or approved by the engineer of record.***

Helical soil nail walls are used most often in temporary shoring applications, with reinforced shotcrete the most common temporary wall facing material. Shotcrete is concrete conveyed through a hose and projected through a nozzle at high velocity onto a working surface. The shotcrete is applied/sprayed in thin lifts until the design thickness requirement is met for the wall. For temporary wall applications, the shotcrete is typically applied to a thickness of 3 to 4 inches. Internal reinforcement of the shotcrete may consist of welded wire fabric (WWF), steel reinforcing bars (rebar), or fiber reinforcement. WWF with rebar walers at the nail heads is typically favored due to ease of installation.

Permanent helical soil nail walls may either have an additional thickness of shotcrete applied or another facing attached to the temporary shotcrete layer. For permanent soil nail walls with shotcrete facing, the typical wall thickness varies from 6 to 12 inches, not including the thickness of the temporary facing. Cast in place and precast concrete facings can also be used in conjunction with the temporary shotcrete wall facing. Facings can be attached to the shotcrete wall to form decorative facades.

2.9.2.2 Limit States

The design of the helical soil nail wall must consider two distinct limiting conditions; Strength Limit States and Service Limit States. The Strength Limit States refer to failure of the system due to loading forces greater than the strength of the system or its individual components. Specifically, the following potential failure modes must be evaluated:

- External failure modes
- Internal failure modes
- Facing failure modes

External failure modes include global stability, sliding and bearing failure. Internal failure modes include soil nail pullout failure, soil nail tensile failure and soil nail shear failure along the failure plane. Facing failure modes include flexure failure, punching shear failure and head stud failure.

The service limit states do not include failure of the structure, but rather consider serviceability issues such as wall deformation, wall settlements or cracking of the facing.

For further information related to designing for these potential failure modes, please refer to FHWA Geotechnical Engineering Circular No. 7 (Lazarte, Elias et al. 2003).

2.10 Load Tests

Load tests are routinely completed on helical piles to establish ultimate and allowable pile capacities, determine pile head movement under load, verify design assumptions and capacities, and establish site-specific torque correlation factors (K_t). ICC-ES AC358 states that full-scale load tests on helical piles shall be conducted in general accordance with the following standards:

- ASTM D1143, Standard Test Methods for Deep Foundations Under Static Axial Compressive Load *(Figures 2.10.a1 and 2.10.a2)*

Figure 2.10.a1
Compression load test

Figure 2.10.a2 Close up of test pile, hydraulic cylinder, dial gauges and hemispherical bearing plate

- ASTM D3689, Standard Test Methods for Deep Foundations Under Static Axial Tensile Load *(Figures 2.10.b1 and 2.10.b2)*

Figure 2.10.b1
Tension load test

Figure 2.10.b2
Tension load test within basement

- ASTM D3966, Standard Test Methods for Deep Foundations Under Lateral Load

AC358 further states that the Quick Test method of ASTM D1143 shall be used for compression tests. Additional discussion and guidance regarding the test procedures are provided in AC358 and within the respective standards.

For axial compression and tension tests, AC358 defines the ultimate pile capacity as the load achieved when plunging of the helix plate occurs or when the net deflection exceeds 10 percent of the average helix diameter, whichever occurs first. Net deflection is defined as the total pile head deflection minus the elastic shortening or lengthening of the shaft.

2.11 Design Examples

Three common methods for determining helical pile capacity are presented in Section 2.7. The individual bearing and cylindrical shear methods are used during the design phase to calculate or estimate the pile capacity. The torque correlation method is generally used to confirm or verify pile capacity during field installation. FSI promotes the use of the individual bearing method for design calculations, therefore, that method will be used in the following examples. Helical pile product ratings, properties and details are provided in Appendix 2A.

HelixPro® Helical Foundation Design Software for Professionals was created by Foundation Supportworks to simplify the design process for helical piles and tiebacks. HelixPro is a web-based helical foundation design tool available free of charge to design professionals. For more information on HelixPro, please refer to Appendix 2C.

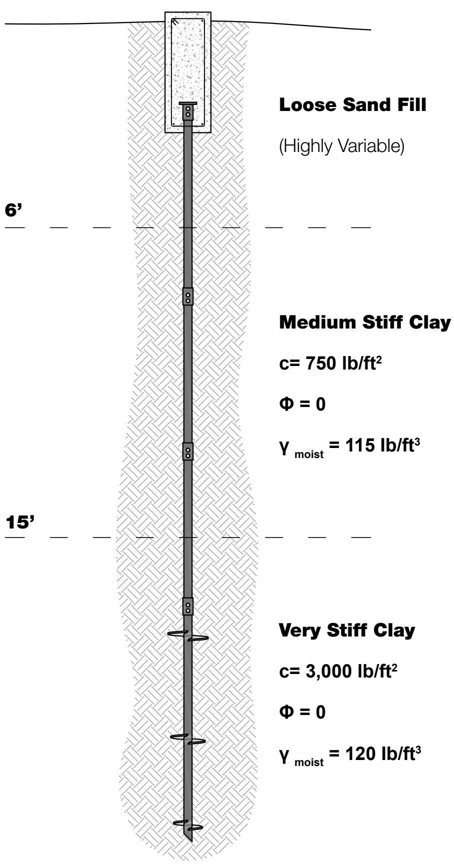

Figure 2.11.1.a
Example 1. Helical Pile Capacity

2.11.1 Helical Piles

Example 1

Helical piles are proposed to support a new structure. The proposed pile layout is shown on the foundation plan along with a design working load of 30 kips per pile with a factor of safety (FOS) = 2. Preliminary product selection suggests that the HP288 helical pile is the best fit for this load condition with an ultimate torque rated capacity of 71.1 kips. The allowable torque rated capacity would then be 35.5 kips with a FOS = 2. A geotechnical investigation was completed for the project and the soil profile is shown in *Figure 2.11.1.a*.

The helical piles will penetrate the upper fill and medium stiff clay to bear within the deeper very stiff clay. With the helix plates bearing entirely within the very stiff clay soil below a depth of 15 feet, we can use the equation from Section 2.7.1 for purely cohesive soils with $\Phi = 0$:

$$\mathbf{Q_u = \sum A_h(9c)}$$

Solve for the required helix plate area:

$\mathbf{A_h}$ = $Q_u/9c$

$\mathbf{Q_u}$ = Design Working Load (30,000 lb) x FOS (2) = 60,000 lb

c = 3,000 lb/ft²

$\mathbf{A_h}$ = 60,000 / (9)(3,000)

$\mathbf{A_h}$ = 2.22 ft²

Helix plate areas for the various shaft sizes can be found in Appendix 2A. For the HP288 shaft (2.875-inch O.D.), a total helix plate area of 2.22 ft² can be most efficiently achieved with a 10/12/14 triple-helix plate configuration.

$\mathbf{A_{10"}}$ = 0.50 ft²

$\mathbf{A_{12"}}$ = 0.74 ft²

$\mathbf{A_{14"}}$ = 1.02 ft²

$\mathbf{\sum A_h}$ = 2.26 ft²

Solve for the ultimate and allowable pile capacities:

$\mathbf{Q_u}$ = (2.26)(9)(3,000) = 61,000 lb = 61 kips

The allowable pile capacity,

$$\mathbf{Q_a = Q_u / FOS}$$

$\mathbf{Q_a}$ = 61,000 / 2 = 30,500 lb = 30.5 kips…OK

Determine the required final installation torque in accordance with the equations and procedures of Section 2.7.3:

$$\mathbf{Q_u = K_tT}$$

The equation can be rewritten to solve for torque:

$$\mathbf{T = Q_u / K_t}$$

Without site-specific load testing and determination of K_t, we use the default value from ICC-ES AC358 for a 2.875-inch O.D. shaft, $K_t = 9\ ft^{-1}$:

T = 60,000 / 9 = 6,667 ft-lb

Install the helical piles to a final installation torque of at least 6,700 ft-lb.

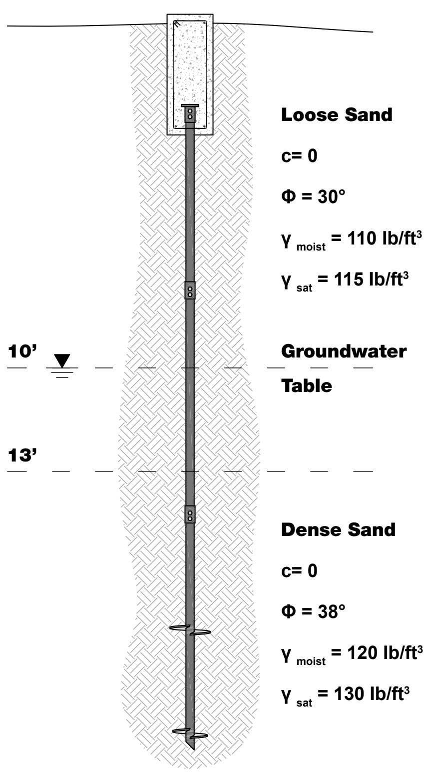

Figure 2.11.1.b
Example 2. Helical Pile Capacity

Example 2

Grain conveyor towers will be constructed at an ethanol facility. The towers will be designed with four support legs, each leg designed for working loads of 40 kips in compression and 15 kips in tension/uplift. A FOS = 2 is required for both the compression and uplift pile capacities. A geotechnical exploration was completed for the project and the soil profile is shown in *Figure 2.11.1.b*. Groundwater was encountered at a depth of 10 feet below the surface. Preliminary product selection suggests that the HP350 helical pile is best suited to support the proposed loads. The HP350 has an ultimate torque rated capacity of 122.5 kips and an allowable capacity of 60 kips (FOS = 2.04; see footnote 7 on page 61). Allowable mechanical compression and tension capacities are well above the service loads to be resisted. The helical piles will be embedded into the dense sand as shown in *Figure 2.11.1.b*.

For purely granular (frictional) soils with c = 0, the ultimate pile capacity can be determined from equation:

$$\mathbf{Q_u = \sum A_h(q'N_q)}$$

Solve for the required helix plate area:

$$\mathbf{A_h = Q_u/q'N_q}$$

The helix plates should be embedded several plate diameters into the dense sand to provide uplift resistance. This depth depends upon the pile load. We can fine tune the embedment depth at a later point, but for an uplift load of 15 kips, we'll consider a minimum helix plate embedment of three diameters. A pile with an ultimate capacity of 80 kips often has three helix plates on the lead section. A 10/12/14 lead has a distance of 5.5 feet between the uppermost and bottommost plates. With these parameters in mind, we'll choose a trial depth of:

13 feet + 3.5 feet (depth of 14-inch plate into dense sand) + 2.75 feet (half the distance between bearing plates) = 19.5 feet.

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The vertical effective overburden stress, q', at 19.5 feet:

q' = $(110\ lb/ft^3)(10\ ft) + ((115\text{-}62.4)\ lb/ft^3)(3\ ft) + ((130\text{-}62.4)\ lb/ft^3)(6.5\ ft) = 1{,}697\ lb/ft^2$

Q_u = Design Working Load (40,000 lb) x FOS (2) = 80,000 lb

N_q = $1+0.56(12\Phi)^{\Phi/54} = 42.6$ (for $\Phi = 38°$)

A_h = 80,000 / (1,697)(42.6)

A_h = 1.11 ft^2

For the HP350 shaft (3.5-inch O.D.), a total helix plate area of at least 1.11 ft^2 can be achieved with a 10/12 double-helix plate configuration.

$A_{10"}$ = 0.47 ft^2

$A_{12"}$ = 0.71 ft^2

$\sum A_h$ = 1.18 ft^2

Solve for the ultimate and allowable pile capacities:

Q_u = (1.18)(1,697)(42.6) = 85,000 lb = 85 kips

$Q_{a,\ compression}$ = 85,000 / 2 = 42,500 lb = 42.5 kips…OK

To maintain the average vertical effective overburden stress at a depth of 19.5 feet, the 12-inch blade would be installed to a depth of 18.25 feet and the 10-inch blade would be installed to a depth of 20.75 feet. The upper helix plate is now 5.25 feet below the loose sand to dense sand interface. With this depth of embedment, we would expect the allowable uplift capacity to be similar to the allowable compressive capacity.

To be very conservative and consider that the loose sand above the 12-inch plate could have some effect on the uplift capacity, we could model the soil strength (friction angle) above the 12-inch plate to represent the loose sand.

$$Q_u = \sum A_h(q'N_q)$$

$q'_{12"}$ = (110)(10) + (115-62.4)(3) + (130-62.4)(5.25) = 1612 lb/ft^2

$q'_{10"}$ = (110)(10) + (115-62.4)(3) + (130-62.4)(7.75) = 1781 lb/ft^2

$N_{q,\ 12"}$ = 15.7 (for $\Phi = 30°$)

$N_{q,\ 10"}$ = 42.6 (for $\Phi = 38°$)

Q_u = (0.71)(1612)(15.7) + (0.47)(1781)(42.6) = 53,600 lb = 53.6 kips

$Q_{a,uplift}$ = 53,600 / 2 = 26,800 lb or 26.8 kips…OK

Determine the required final installation torque in accordance with the equations and procedures of Section 2.7.3:

$$Q_u = K_tT$$

The equation can be rewritten to solve for torque:

$$T = Q_u / K_t$$

Without site-specific load testing and determination of K_t, we use the default value from ICC-ES AC358 for a 3.5-inch O.D. shaft, $K_t = 7\ ft^{-1}$:

T = 80,000 / 7 = 11,428 ft-lb

Install the helical piles to a final installation torque of at least 11,500 ft-lb.

2.11.2 Helical Tiebacks

Example 3

Helical tiebacks are being considered to stabilize an existing reinforced concrete retaining wall. The tiebacks can extend no further than 20 feet from the front face of the wall due to property line issues. A geotechnical investigation found the retained soils to consist of silty sand. The design engineer proposed an HA150 shaft (1.5-inch solid square) with a 10/12/14 helix plate configuration. The soil parameters and preliminary tieback design are shown on *Figure 2.11.2.a*. The engineer must determine the allowable tieback capacity so tieback spacing can be established.

$$Q_u = \sum A_h(q'N_q)$$

$A_{14"}$ = 1.05 ft²

$A_{12"}$ = 0.77 ft²

$A_{10"}$ = 0.53 ft²

$q'_{14"}$ = (120)(7.17) = 860 lb/ft²

$q'_{12"}$ = (120)(7.67) = 920 lb/ft²

$q'_{10"}$ = (120)(8.08) = 969 lb/ft²

$N_q = 1+0.56(12\Phi)^{\Phi/54} = 15.7$

Q_u = (1.05)(860)(15.7) + (0.77)(920)(15.7) + (0.53)(969)(15.7) = 33,300 lb = 33.3 kips

Q_a = 33,300 / 2 (FOS) = 16,650 lb = 16.6 kips

The horizontal and vertical components of the tieback force can be calculated in accordance with Section 2.8.1.

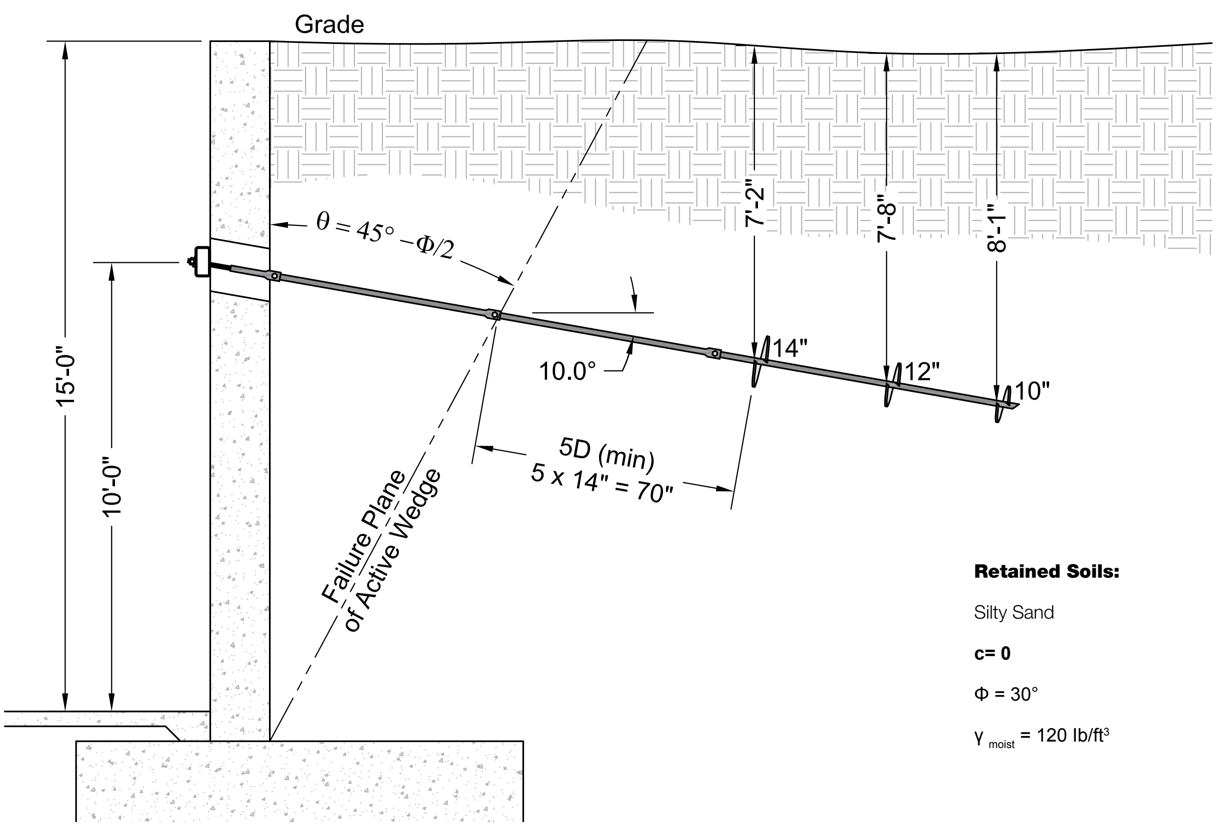


Figure 2.11.2.a
Example 3. Helical Tieback Capacity

2.12 Installation

2.12.1 General Information

2.12.1.1 Preparation

Prior to any excavation or installation of helical piles or tiebacks, all utilities, pipelines, cables, or any other service lines or structures shall be identified and marked. The appropriate utility locating agency should be contacted in advance of the project, allowing adequate notification time frames mandated by the agency.

Call number "811" is a federally mandated FCC designated N-11 number. The 811 number is a national "Call Before You Dig" phone number designated by the FCC to eliminate the confusion of multiple "Call Before You Dig" numbers, minimize damages to underground utilities and help save lives. One easy phone call to 811 quickly begins the process of getting underground utility lines marked. Local One Call Center personnel will then notify affected utility companies, who will send crews to mark underground lines for free.

Foremen and installers should be mindful of potential hazards and understand the meanings and definitions of common tags provided by the American National Standards Institute (ANSI) and the Occupational Safety and Health Administration (OSHA) *(Figure 2.12.1.1.a)*.

ANSI Z535.5 Definitions:

- **Danger:** Indicate[s] a hazardous situation which, if not avoided, will result in death or serious injury. The signal word "DANGER" is to be limited to the most extreme situations. DANGER [signs] should not be used for property damage hazards unless personal injury risk appropriate to these levels is also involved.

- **Warning:** Indicate[s] a hazardous situation which, if not avoided, could result in death or serious injury. WARNING [signs] should not be used for property damage hazards unless personal injury risk appropriate to this level is also involved.

- **Caution:** Indicate[s] a hazardous situation which, if not avoided, could result in minor or moderate injury. CAUTION [signs] without a safety alert symbol may be used to alert against unsafe practices that can result in property damage only.

- **Notice:** [this header is] preferred to address practices not related to personal injury. The safety alert symbol shall not be used with this signal word. As an alternative to "NOTICE" the word "CAUTION" without the safety alert symbol may be used to indicate a message not related to personal injury.

The OSHA 1910.145 definitions for tags are as follows:

- **Danger:** "shall be used in major hazard situations where an immediate hazard presents a threat of death or serious injury to employees. Danger tags shall be used only in these situations."

- **Warning:** "may be used to represent a hazard level between "Caution" and "Danger," instead of the required "Caution" tag, provided that they have a signal word of "Warning," an appropriate major message, and otherwise meet the general tag criteria of paragraph (f)(4) of this section."

- **Caution:** "shall be used in minor hazard situations where a non-immediate or potential hazard or unsafe practice presents a lesser threat of employee injury."

Figure 2.12.1.1.a

2.12.1.2 Crowd

During the installation of a helical pile or tieback, axial force or "crowd" is applied to the pile/tieback shaft to advance the helix plates into the soil. The density or stiffness of the soil dictates the amount of crowd necessary to advance the pile to a depth where the helix plates can then provide the downward thrust. Multi-helix pile configurations often install easier than single-helix configurations due to the thrust provided by the additional helix plates. At a depth typically just a few feet below the surface, little to no external force is necessary unless deeper, dense soil layers or obstructions are encountered. Additional crowd may be required to either penetrate the dense layers or fully embed the helix plates into dense bearing soil. In soft soil conditions, it is important not to over-crowd or restrain the advancement of the pile. Applying the proper crowd is critical to maintain the penetration rate and minimize disturbance or mixing of the soils, especially within the final 3 to 5 feet of installation prior to pile termination.

Installation equipment not only needs to be sized correctly to provide the proper hydraulic flow and hydraulic fluid pressure for the drive head, but also to provide the proper crowd for pile advancement. The lack of appropriate machine weight during installation into dense soils or weathered bedrock may limit pile penetration, resulting in less than anticipated tensile or compressive capacities.

2.12.1.3 Penetration Rate

Helical piles or tiebacks should ideally be advanced into the soil at a rate equal to the pitch of a properly formed, conforming helix plate; i.e., 3 inches per revolution. ICC-ES AC358 further states that the pile advancement shall equal or exceed 85 percent of the helix pitch per revolution at the time of final torque measurement. Crowd may be required to maintain adequate pile penetration or advancement. Installation speeds should be limited to less than 25 revolutions per minute (rpm) within the last several feet of pile installation to minimize soil disturbance. It is good practice to further reduce installation speeds to 10 rpm or less within the final 3 to 5 feet so the operator can concentrate on pile alignment, crowd and rate of advancement. Installation speeds may be further restricted by soil conditions or operating equipment.

2.12.2 Equipment

2.12.2.1 Drive Heads

FSI currently offers drive heads that range in capacity from 5,000 to 20,000 ft-lb of torque. This range allows for installation of the most common helical products to their maximum torsional ratings. Larger capacity drive heads are available upon request.

Proper selection of the drive head should consider the torsional rating of the helical shaft, project installation torque requirements, and the output pressure and flow rate of the hydraulic system to be used. All drive heads have optimum operating specifications that should be partnered with an appropriate hydraulic system to achieve maximum performance in the field. FSI recommends that drive heads have a rated torque output capacity at least 15 percent higher than what is required by project specifications.

Hydraulic hoses and fittings should be rated for the operating pressures required and specified by the drive head manufacturer. Hoses and fittings should be checked periodically for damage and replaced when in question. Failure to follow manufacturer's specifications may result in equipment failure and/or personal injury.

Drive heads are generally designed with bail assemblies for mounting to machinery such as skid steers *(Figure 2.12.2.1.a)*, mini-excavators *(Figure 2.12.2.1.b)*, backhoes *(Figure 2.12.2.1.c)*, and full-size excavators *(Figure 2.12.2.1.d)*. Smaller, lighter-weight drive heads may also be used with hand-held equipment for interior or limited access installations. Machinery used to power and operate drive heads should have sufficient weight and structural capacity to handle the output torque. A basic rule of thumb has been a pound of machine weight for each ft-lb of torque from the drive head. Although a somewhat conservative rule, machine weight and structural capacity become increasingly more important with greater output torque.

Figure 2.12.2.1.a
Skid steer

Figure 2.12.2.1.b
Mini-excavator

Figure 2.12.2.1.c
Backhoe

Figure 2.12.2.1.d
Excavator

The following machine specifications are required:

- The machine should have a bi-directional auxiliary circuit to power the drive head.
- Hydraulic fluid pressure output from the circuit used to power the drive head should meet the drive head specifications. On some machines, it may be necessary to adjust the relief valve on the machine's auxiliary system hydraulic pump to provide the appropriate pressure specified by the drive head manufacturer.
- The flow rate of hydraulic fluid to the drive head should meet the drive head specifications for optimum performance during installation.
- The machine should have adequate weight to resist torsional forces from the drive head and to allow for proper crowd during installation.

FSI offers portable hand-held equipment for operating smaller, lighter-weight drive heads when access with machinery is not feasible. The drive heads can be powered by auxiliary hydraulic circuits of machinery or by portable hydraulic power packs. The power source should meet the operating specifications of the drive head. A portable, remote valve assembly allows for safe operation of the drive head when used with the hand-held equipment.

Figure 2.12.2.1.e
Hand-held equipment

The drive head is mounted to the frame of the hand-held equipment *(Figure 2.12.2.1.e)* so that it can be supported and operated by at least two technicians. To provide the reaction for the output torque, a telescoping torque arm is attached to the frame of the hand-held equipment. The torque arm *(Figure 2.12.2.1.f)* is secured against the ground, a wall or other suitable structure or device capable of resisting the torsional forces transferred to the end of the torque arm by the drive head. Hand-held equipment is typically limited to a maximum installation torque of 6,000 ft-lb, or less. Consult FSI with any questions regarding the rated capacities of FSI hand-held equipment.

Figure 2.12.2.1.f
Torque arm

Installers and personnel in the immediate work area should be properly trained in the safe operation and use of hand-held equipment. The torque arm shall be properly restrained for the direction of arm rotation. Reversing the rotation also requires restraining of the torque arm in the opposite direction. Personnel in the work area should understand the direction that the torque arm will tend to swing and position themselves in a safe location (considering any possibility that the torque arm could break free from its restraint). Appropriate installation geometry should be maintained during pile installation. The ideal position for the torque arm is as follows:

1) ***Arm is fully extended and is restrained at its maximum radius from the pile shaft.***
2) ***Arm is at an angle which is perpendicular to the pile shaft.***

Actual installation geometry is adjustable and will vary, but in no circumstance shall the torque arm be placed at an angle in excess of 35 degrees from perpendicular and in no circumstance shall the torque arm restraint

be placed at a distance less than 7 feet from the axis of the pile shaft. The capacity of the hand-held equipment decreases significantly when used outside of these parameters. The force that will be required to restrain the torque arm will also vary, but even within the operation parameters just described, restraint forces can approach 1,000 lb. The torque arm restraint is therefore recommended to be capable of resisting a force of at least 1,500 lb. Lateral restraint is not only required at the end of the torque arm. The drive head shall also be restrained from lateral movement.

Drive heads used with hand-held installation equipment should not be operated at speeds exceeding 10 rpm. Operators shall be ready at the controls and prepared to shut down the equipment at any moment.

Failure to properly restrain and operate the hand held installation equipment per these guidelines can result in serious injury or death.

2.12.2.2 Installation Tooling

Installation tooling consists of the components that are attached in-line between the drive head and the helical pile, generally a hex adaptor and a product adaptor. The drive head output shaft is typically a hexagonal shape with measurements ranging from 2 inches to 3 inches across flats. The hex adaptor slides over and is pinned to the output shaft of the drive head *(Figure 2.12.2.2.a)*. The flange plate of the

Figure 2.12.2.2.a
Hex adaptor

hex adaptor has a bolt hole pattern with hole spacing and diameters to allow bolting to the appropriate product adaptor for the dimensions of helical pile shaft to be installed. Product adaptors are available for the various sizes of hollow round shaft as well as for solid square shaft. The ends of round shaft helical piles generally slide into the product adaptors and are connected with temporary hitch pins or bolts for installation *(Figure 2.12.2.2.b)*. Solid-stock internal product adaptors may also be used for certain sizes of the round shaft helical product line. The ends of square shaft helical piles and tiebacks slide into a square, socket-like product adaptor *(Figure 2.12.2.2.c)*.

Figure 2.12.2.2.b
Round shaft product adaptor

Installation tooling may also include an in-line torque monitoring device as discussed in the following section.

Figure 2.12.2.2.c
Square shaft adaptor

CHAPTER 2
HELICAL FOUNDATION SYSTEMS

2.12.2.3 Monitoring Torque

Monitoring torque is a key process during the installation of helical piles since the installation torque directly correlates to pile capacity in accordance with the torque correlation method described in Section 2.7.3. A number of devices are available to assist in determining torque and, ultimately, the calculation of pile capacity. These devices range from simple pressure gauges to shear pin indicators to more sophisticated electronic data acquisition systems.

Dual hydraulic pressure gauges *(Figures 2.12.2.3.a1 and 2.12.2.3.a2)* can be used to measure the "pressure drop" across a hydraulic torque motor. This method is based on the principle that the work output of the torque motor is directly related to measurement of the pressure drop across the motor as force is applied. To measure the pressure drop, one gauge is placed in line with the feed from the hydraulic pump or machine to the drive head. A second gauge is placed in line with the return from the drive head back to the pump.

Figure 2.12.2.3.a1
Dual pressure gauges

The return line pressure is subtracted from the feed line pressure resulting in the determination of "differential" pressure. The installation torque can be calculated relative to the differential pressure by applying the gear motor multiplier (GMM) provided by the drive head manufacturer. Most drive head manufacturers provide correlation charts for quick conversion of differential pressure to torque.

Figure 2.12.2.3.a2 Monitoring pressure gauges while installing square shaft helical anchor

The return line gauge is an indicator of the hydraulic system "back pressure", which is variable with each machine and may average from 50 psi to over 800 psi. Systems with high return line pressures may damage a hydraulic torque motor. The installation of a "case drain" on the hydraulic torque motor can prevent damage to the motor seal. A case drain line is simply directed back to the hydraulic fluid reservoir.

Some operators choose to use a single gauge on the feed line side only, rather than to use a second gauge to measure back pressure. This can result in decreased accuracy and over-estimation of applied torque if back pressure is under-estimated or ignored all together.

Differential pressure cylinders such as the DP-1, are hydraulic cylinders with opposing pistons within the cylinder body that measure the differential pressure on a single gauge. Similar to the use of dual pressure gauges, the determination of torque is based on the pressure drop across the motor and, subsequently, the differential pressure to torque correlation.

Differential pressure gauge technology is based on similar principals as the DP-1. Differential pressure gauges still measure the feed and return line pressures to determine the pressure drop across the motor, but with ports for the lines within a single-gauge body. This differential pressure, as in the case of the Tru Torque model *(Figure 2.12.2.3.b)*, is related to torque by the GMM for a specific drive head. The

Figure 2.12.2.3.b
Tru Torque gauge

dial face then provides a reading of torque rather than pressure. A different differential pressure gauge is therefore needed for each drive head.

Electronic pressure indicators measure the feed and return line pressures with electronic pressure transducers. Low voltage power is supplied to the unit by either a portable battery pack or a direct connection to an appropriate low voltage source generated by the installation equipment. Instead of analog gauges, electronic indicators such as the PT Tracker by Marian Technologies *(Figure 2.12.2.3.c)* typically have a digital screen output to provide a direct reading of torque, which is generated by a pre-programmed relationship of the pressure drop across the motor and the GMM for the drive head being used. Some units have a selector switch that allows for torque readings with various motors. Some models also allow for data acquisition and/or blue tooth technology.

Figure 2.12.2.3.c
PT tracker

Shear pin torque limiters are mechanical in-line devices consisting of two independent plate assemblies mounted to a central shaft, but allowed to rotate independently. Each plate has a series of holes around the perimeter that allow for insertion of steel pins with a given shear strength. The pins are placed in the holes of the top plate to extend past the interface between plates and into the holes of the bottom plate. The pins bridge across the interface and restrict the independent rotation of the plates until sufficient torque is applied. The pins will theoretically shear simultaneously when the torque applied exceeds the summed capacity rating of the pins. For example, if 3,000 ft-lb of torque is required for a helical pile installation, six pins rated at 500 ft-lb each would be inserted into the housing. The pins should shear simultaneously when 3,000 ft-lb of torque is reached.

Mechanical dial indicators are in-line mechanical devices consisting of a torsion bar mounted between two separate, bolted flange plates. An in-line dial indicator measures the twist of the torsion bar and reads torque in units of ft-lb on the dial gauge. This device can be used to establish a torque correlation between pressure gauges and a specific drive head through dynamic testing.

Figure 2.12.2.3.d
Shaft twist

The **shaft twist method** is simply a visual observation of the shaft deformation or twist that occurs with square bar helical products *(Figure 2.12.2.3.d)* during installation. With this method, the installer must know the range of torque required to initiate plastic deformation in the shaft for the given product. This method does not provide an accurate or reliable indication of torque and should not be used solely as a measure or estimate of applied torque.

Electronic torque transducers such as the Pro-Dig intelli-Tork® are placed in line with the tool string. Torque is a true real time measurement and is generated continually during the installation of a helical pile or tieback. The intelli-Tork *(Figures 2.12.2.3.e1 and 2.12.2.3.e2)* measures the torque applied between two flanges and transmits the torque reading to a hand held unit for display and logging. A built in torque sensor within the housing of the flanged instrument transfers data via blue tooth wireless technology to the PDA system. The PDA based system and software

Figure 2.12.2.3.e1
Electronic torque transducer

provide a remote visual indication of the torque during the installation. Software provided with the instrument has the ability to log the torque, depth and installation angle. Torque transducers can be re-calibrated as needed to ensure accuracy. In turn, a properly calibrated torque transducer can be used to calibrate analog gauge systems relative to differential pressure.

Figure 2.12.2.3.e2 Torque transducer providing direct reading of torque for helical pile project at Reagan National Airport

2.12.3 Installation Guidelines

2.12.3.1 New Construction

Installing the Lead Section:

- Align the lead section with the product adaptor and install the temporary hitch pins or bolts.
- Position the installation equipment and pile directly over the marked location.
- Apply a small amount of crowd to seat the pile shaft tip into the soil.
- Use a level or digital gauge to plumb or set the installation angle (batter) of the pile shaft.
- Advance the pile in a continuous even manner, making periodic adjustments to maintain alignment throughout the installation. Record torque as required by project specifications or as dictated by changing soil conditions. Although the final installation torque is arguably the most critical, it is good practice to record pressure or torque during the entire installation. This allows for development of a relative soil strength profile with depth. The interval of readings is often dictated by the soil variability; i.e., more readings should be taken in heterogeneous soils and fewer readings are required in uniform, homogeneous soils. At a minimum, record torque for every lead section and extension.
- Terminate installation of the lead section before the product adaptor penetrates the soil.
- Remove the hitch pins or bolts and carefully remove and raise the drive head.

Installing Extension Sections:

- Place the first extension on top of the buried lead. Align the coupler bolt holes with the bolt holes of the lead section. Use a spud wrench if necessary.
- Install coupler bolts taking care not to damage threads. Tighten nuts to snug condition *(Figure 2.12.3.1.a)*.

Figure 2.12.3.1.a
Install coupler bolts

Figure 2.12.3.1.b
Install new construction bracket

- Align the drive head and product adaptor over the extension shaft to allow for installation of hitch pins or bolts.

- Advance the extension and any additional extensions following the alignment adjustment and coupling procedures described above.

Termination of Installation:

- Over the final 3 to 5 feet of installation, assuming depth and minimum torque requirements are being met, reduce rotational speed to approximately 10 rpm. Provide proper alignment and crowd. **Refer to the Model Specifications for Helical Pile Foundations in Appendix 2F for termination criteria when the minimum overall length or minimum torsional resistance is not met.**

- Remove equipment from pile, establish top of pile elevation and cut pile shaft to specified elevation (if necessary).

- Install new construction bracket as specified *(Figure 2.12.3.1.b)*. For compression applications, the new construction bracket could technically be set on top of the pile without bolting or welding. However, FSI feels that it is good practice to provide positive attachment of the bracket to the top of the pile to prevent the bracket from being lifted off the pile during concrete placement. Tack welds, a single bolt, or use of compression-only plate bracket assemblies are generally adequate for this purpose. Where the top of pile has been cut to achieve design elevation and tension loads will be applied, bolt holes should be drilled using drills and drill fixtures as recommended by FSI to maintain bolt hole size, location and spacing tolerances.

- Complete field installation logs.

Should field conditions present unanticipated obstacles that require relocating of piles or tiebacks, consult the engineer of record for approval before proceeding.

2.12.3.2 Retro-Fit Helical Piers

Excavation:

- Hand or machine-excavate to expose the bottom of the footing. Individual holes should be approximately 3 feet square while a continuous trench excavation should extend at least 3 feet away from the structure.

- Depth of excavation should locally extend 13 inches below bottom of footing and 9 inches back under bottom of footing where brackets will be placed *(Figure 2.12.3.2.a)*.

Figure 2.12.3.2.a
Excavation at bracket

Follow OSHA Trench Safety Procedures. Failure to follow trench safety procedures could result in death or serious injury.

Preparation of Footing:

- Notch spread footings 16 to 22 inches wide (depending upon width of retrofit bracket) and approximately flush with the face of the foundation wall *(Figure 2.12.3.2.b)*. Thick column footings or trench footings often do not require notching of the concrete, but should still be prepared so the face of the footing has full contact with the back, vertical plate of the bracket.

Figure 2.12.3.2.b
Footing preparation

- Clean and prepare bottom of footing to allow full contact and seating of the bracket.

Installation of Helical Pier:

- Attach lead section to drive head and product adaptor.

- Place lead section's first blade under the footing and the shaft of the lead 1½ inches away from bottom edge of footing *(Figure 2.12.3.2.c)*. This will allow alignment of the lead to the required angle of inclination of 3.0 ± 1.0 degrees from vertical. As additional plates on the lead section advance down the face of the footing and pass the bottom of the footing, forward crowd will be required to realign the shaft to the appropriate inclination. In the case of a vertical pile installation, a pilot hole is required to set all plates of the lead below the bottom of the footing before advancement of the pile. The pile shaft is still set 1½ inches from the face of the footing.

Figure 2.12.3.2.c
Alignment of lead

- Advance lead section and extensions to design depth and minimum torque requirements.
- Record pressure or torque readings. Although the final installation torque is arguably the most critical, it is good practice to record pressure or torque during the entire installation. This allows for development of a relative soil strength profile with depth. The interval of readings is often dictated by the soil variability; i.e., more readings should be taken in heterogeneous soils and fewer readings are required in uniform, homogeneous soils. At a minimum, record torque for every lead section and extension.
- If necessary, cut the last extension shaft to an elevation approx. 13 inches above bottom of footing.
- Ideally, the last coupler on the helical pile shall be at least 23 inches below the bottom of the footing to allow installation of the 30-inch external sleeve.

Installation of Underpinning/Retrofit Bracket:

- Place external sleeve through bracket.
- Lower bracket and external sleeve assembly over the pier shaft with bracket bearing plate facing away from the footing *(Figure 2.12.3.2.d)*.

Figure 2.12.3.2.d
Lower assembly over pile shaft

- Rotate the bracket body 180 degrees toward the footing.
- Raise bracket to the footing and hold the bracket in place while attaching the thread rods and cap plate. A bracket RAYser™ is a great tool to hold the bracket in place during this operation *(Figure 2.12.3.2.e)*.

Figure 2.12.3.2.e
Bracket RAYser™

- Tap the external sleeve down until the top flange rests on the bracket.
- Install the cap plate and all thread rods and tighten nuts to snug the bracket to the bottom of the footing *(Figure 2.12.3.2.f)*.

Figure 2.12.3.2.f
Install cap plate and rods

- Remove bracket RAYser™, backfill and compact soil up to the bottom of the bracket.

Load transfer and Lift:

- Set lift cylinders and apply load to project specifications. Discontinue if structure begins to lift prior to achieving the design working load. Alternatively, load can be increased until the structure lifts and the desired elevation is met *(Figure 2.12.3.2.g)*.

Figure 2.12.3.2.g
Set hydraulic lift cylinders

- Lock off and transfer load to piers by tightening nuts down to cap plate.
- Remove lifting hardware and hydraulics.
- Complete field installation logs.
- Establish bench marks (if required).

Backfill and cleanup:

- Backfill holes or trenches with the excavated, on-site material or imported soil.
- Follow proper backfill/compaction procedures and tamp in maximum 6 to 12-inch lifts depending upon type and weight of compaction equipment *(Figure 2.12.3.2.h)*.

Figure 2.12.3.2.h
Backfill and compact soil

- When possible, establish grades to allow positive surface drainage away from the structure.
- Clean up and haul away construction debris from the piering operation.

Should field conditions present unanticipated obstacles that require relocating any of the proposed piers, consult the engineer of record for approval before proceeding.

2.12.3.3 Helical Tiebacks

Helical tiebacks can be installed using either machine-mounted or hand-held drive heads. Basic installation procedures consist of the following:

- Attach lead section to product adaptor and install hitch pin or bolt.
- Elevate lead section along with drive head assembly and place tip of tieback at marked location.
- Establish angle of inclination and align lead section per design specifications.
- Provide minimal crowd to seat lead shaft tip.
- Install lead section while maintaining installation angle.
- Connect extension section to the lead and tighten bolted connection at coupler.
- Continue adding extensions until embedment length and minimal torque requirements are met. **Refer to the Model Specifications for Helical Anchor Foundations in Appendix 2F for termination criteria when the minimum overall length or minimum torsional resistance is not met.**
- Remove installation equipment and install threaded transition assembly on end of extension. Tighten coupling bolt.
- Install threaded rod into transition so threads are fully engaged.
- Place wall plate, bracket or waler system over threaded rod. Place nut on threaded rod and tighten to bearing plate.
- Tighten nut to a predetermined torque to correlate to an axial tensile force. For larger projects, a calibrated hydraulic cylinder may be used to pre-tension the tieback to the design working load or to run a load test.
- Remove equipment and cut threaded rod.

Appendix 2A

Helical Product Ratings, Properties and Details

Helical Pile Capacities Summary

		Installation Torque		Maximum Soil Capacity[7]		Shaft Max Allowable Capacity[4] P_n/Ω					
						Compression[3]			Tension		
		Correlation Factor	Max Rated Torque	Ultimate	Allowable	Plain	Plain Corroded[1]	Galvanized Corroded[1,2]	Plain	Plain Corroded[1]	Galvanized Corroded[1,2]
		K_t (ft^{-1})	T (ft-lb)	Q_u (kips)	Q_a (kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)
Shaft	HA150	10[5]	6,500	65.0	32.5	29.8[8]	27.0[8]	27.1[8]	29.8	27.0	27.1
	HA175	10[5]	10,000	100.0	50.0	59.6[8]	54.1[8]	54.2[8]	59.6	54.1	54.2
	HP237	10[6]	2,500	25.0	12.5	35.1	26.3	32.6	19.3	13.6	16.9
	HP287	9[5]	5,600	50.4	25.2	55.8	45.2	52.9	30.6	23.6	27.6
	HP288	9[5]	7,900	71.1	35.5	74.0	63.6	71.1	41.6	34.1	38.1
	HP350	7[5]	17,500	122.5	60.0	118.5	105.0	114.8	73.0	62.8	69.1
	HP450	6[6]	22,000	132.0	66.0	123.3	109.1	119.3	59.1	50.8	56.7
	HP662[9]	N/A	35,000	N/A	N/A	182.6	157.4	175.6	98.7	83.3	93.0
	HP700[9]	N/A	50,000	N/A	N/A	246.9	220.5	239.6	135.0	118.8	130.5

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123. Coatings on fasteners vary by product line. See individual shaft specification sheets for more details.

(3) Allowable mechanical compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(4) Listed mechanical capacities are for the shaft and coupled connections only. System capacity should also not exceed the installed allowable torque-correlated soil capacity or the allowable capacity of the respective bracket (see additional bracket tables).

(5) Listed default K_t factors are consistent with those listed in ICC-ES AC358. These values are generally conservative. Site-specific K_t factors can be determined for a given project with full-scale load testing.

(6) Listed K_t factors are those recommended by FSI. Site-specific K_t factors can be determined for a given project with full-scale load testing.

(7) Maximum ultimate soil capacity is the product of the torque correlation factor and the shaft maximum torque rating per the equation $Q_u = K_t \times T$. The maximum allowable soil capacity for the HP350 shaft is limited to 60 kips per AC358. The maximum allowable soil capacity for all other shaft sizes is obtained by dividing the maximum ultimate soil capacity by a factor of safety of 2.0. Although a factor of safety of 2.0 is commonly used, a higher or lower factor of safety may be considered at the discretion of the helical pile designer or as dictated by local code requirements. System capacity should also not exceed the mechanical capacity of the shaft or those listed in the respective bracket capacity tables.

(8) Square shaft piles may be considered for compression applications in soil profiles that offer sufficient continuous lateral support; e.g., in soils with SPT blow counts ≥ 10. In profiles or conditions that limit pile stability, buckling analyses should be considered by the project engineer, taking into account discontinuities and potential eccentricities created by the couplers.

(9) FSI's larger diameter product lines are fully customized on a project specific basis. All values provided for these products are for general informational purposes only. Actual capacities (including any related to installation torque) will vary based on several project specific variables such as coupler details, end termination details, site specific soil profiles, and even material availability. Full scale load tests are recommended to confirm soil capacities determined in the design phase of the project.

Rev. 10/14/16

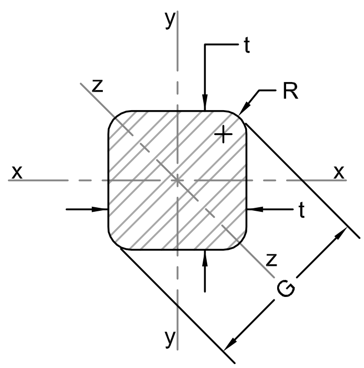
HA150 Shaft Specifications and Capacities

Shaft Material:
1.50” round corner square bar
ASTM A29
Yield strength = 90 ksi (min)
Tensile strength = 115 ksi (min)

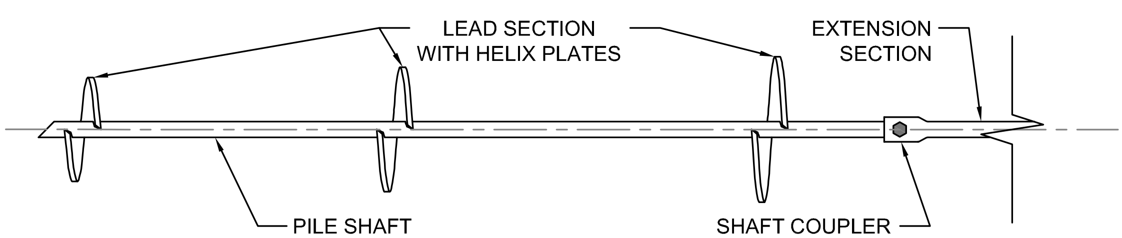
Helix Plates:
ASTM A572 Grade 50 material
⅜” thick (standard)
½” thick (available)
Helix plate geometry conforming to ICC-ES AC358

Shaft Coupling Hardware:
(1) - Ø¾” Grade 8 bolt with nut
Mechanically galvanized per ASTM B695

Surface Finish of Shaft Segments:
Available plain or hot-dip galvanized[2]



	Plain	Plain Corroded[1]	Galvanized Corroded[1,2]
t (in)	1.500	1.464	1.490
G (in)	1.914	1.878	1.904
R (in)	0.250	0.232	0.245
A (in^2)	2.20	2.10	2.17
I_x, I_y (in^4)	0.40	0.36	0.39
I_z (in^4)	0.40	0.36	0.39
S_x, S_y (in^3)	0.53	0.49	0.52
S_z (in^3)	0.41	0.38	0.41
Z_x, Z_y (in^3)	0.81	0.75	0.79
Z_z (in^3)	0.77	0.72	0.75
r_x, r_y (in)	0.42	0.42	0.42
r_z (in)	0.42	0.42	0.42
Shaft Max Allowable Compression Capacity[3,4] P_n/Ω (kips)	**29.8**	**27.0**	**27.1**
Shaft Max Allowable Tension Capacity[4] P_n/Ω (kips)	**29.8**	**27.0**	**27.1**



Default Torque Correlation Factor[5] K_t = 10 (ft^{-1})	**Maximum Ultimate Soil Capacity[6] Q_u = 65.0 (kips)**
Maximum Installation Torque T = 6,500 (ft-lb)	**Maximum Allowable Soil Capacity[6] Q_a = 32.5 (kips) FOS = 2.0**

(1) Corroded properties and capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Square shaft piles may be considered for compression applications in soil profiles that offer sufficient continuous lateral support; e.g., in soils with SPT blow counts ≥ 10. In profiles or conditions that limit pile stability, buckling analyses should be considered by the project engineer, taking into account discontinuities and potential eccentricities created by the couplers.

(4) Listed mechanical capacities are for the shaft and coupled connections only. System capacity should also not exceed the installed allowable torque-correlated soil capacity or the allowable capacity of the respective bracket (see additional bracket tables).

(5) Listed default K_t factor is consistent with that listed in ICC-ES AC358. This value is generally conservative. Site-specific K_t factors can be determined for a given project with full-scale load testing.

(6) Soil capacities listed are at maximum installation torque. Ultimate soil capacity is based on the equation $Q_u = K_t \times T$. Allowable soil capacity is obtained by dividing the ultimate value by the appropriate factor of safety ($Q_a = Q_u$ / FOS). FOS is most commonly taken as 2.0, although a higher or lower FOS may be considered at the discretion of the helical pile designer or as dictated by local code or project requirements. System capacity should also not exceed the mechanical capacity of the shaft or those listed in the respective bracket capacity tables.

HA150NCB Bracket Specifications and Capacities
when used with the HA150 Helical Pile System

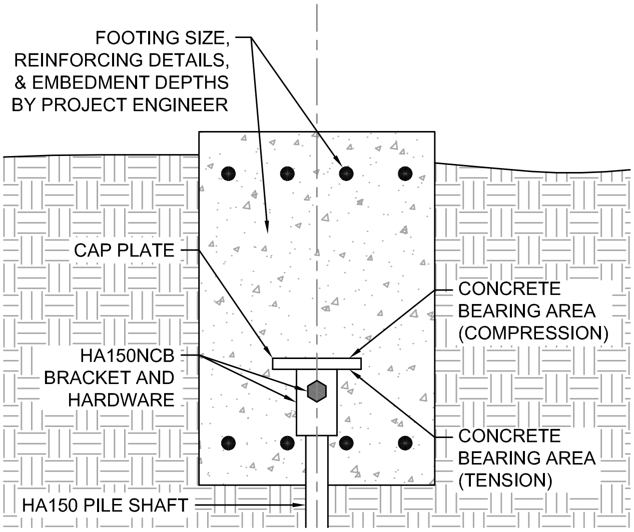
Bracket Sleeve Material:
Ø2.750” x 0.313” wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Cap Plate Material:
¾” x 6.00” square
ASTM A36

Bracket Hardware:
(1) - Ø¾” Grade 8 bolt with nut
Mechanically galvanized per ASTM B695

Bracket Finish:
Available plain or hot-dip galvanized(2)

Concrete Bearing Area(5) (Compression) = 36.0 in^2
Concrete Bearing Area(5) (Tension) = 30.1 in^2



	Allowable Bracket Capacity(4) R_n/Ω			
	Compression(3) (kips)	**Concrete Bearing(5) (ksi)**	**Tension (kips)**	**Concrete Bearing(5) (ksi)**
Plain	29.6	0.82	29.6	0.98
Plain Corroded(1)	24.9	0.70	24.9	0.84
Galvanized Corroded(1,2)	27.1	0.76	27.1	0.91

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Square shaft piles may be considered for compression applications in soil profiles that offer sufficient continuous lateral support; e.g., in soils with SPT blow counts ≥ 10. In profiles or conditions that limit pile stability, buckling analyses should be considered by the project engineer, taking into account discontinuities and potential eccentricities created by the couplers.

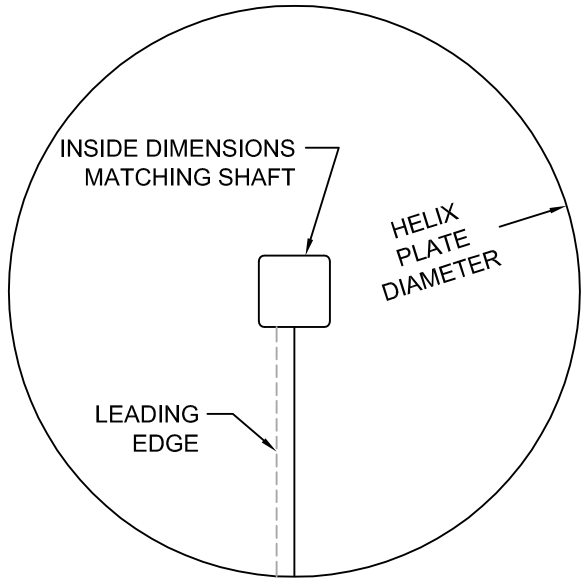
(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

(5) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f’c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

HA150 Helix Plate Net Bearing Areas

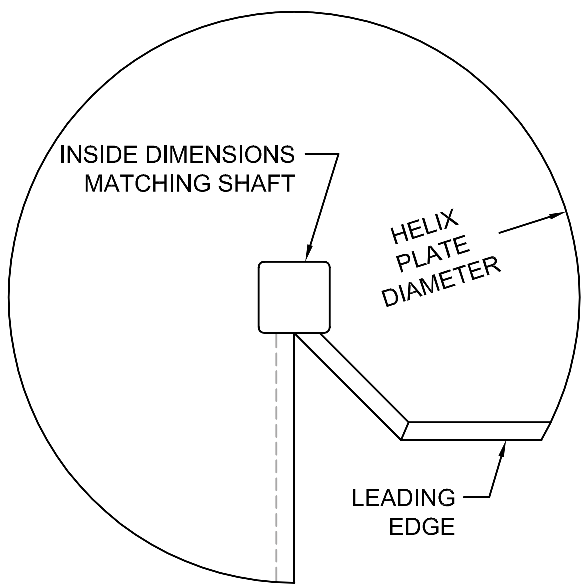
H-Style Plates

Plate Diameter (inches)	Area (ft^2)
6	0.18
8	0.33
10	0.53
12	0.77
14	1.05
16	1.38



V-Style Plates(1)

Plate Diameter (inches)	Area (ft^2)
6	0.16
8	0.30
10	0.47
12	0.68
14	0.92
16	1.20



(1) V-Style plates feature a special cut on the leading edge (or cutting edge). This edge is cut at two successive 45° angles to roughly simulate a spiral. This is in addition to the 45° bevel on the leading edge which is a standard feature for helix plates of both styles. V-Style plates are appropriate for use in applications where rocky or rubble-filled soils are anticipated, or where very dense layers need to be penetrated.

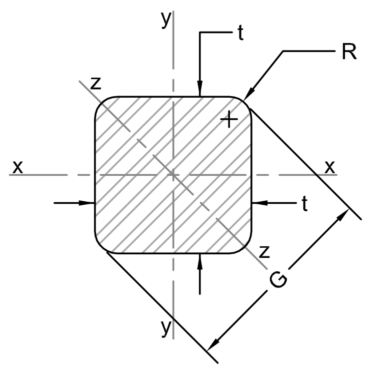
HA175 Shaft Specifications and Capacities

Shaft Material:
1.75” round corner square bar
ASTM A29
Yield strength = 90 ksi (min)
Tensile strength = 115 ksi (min)

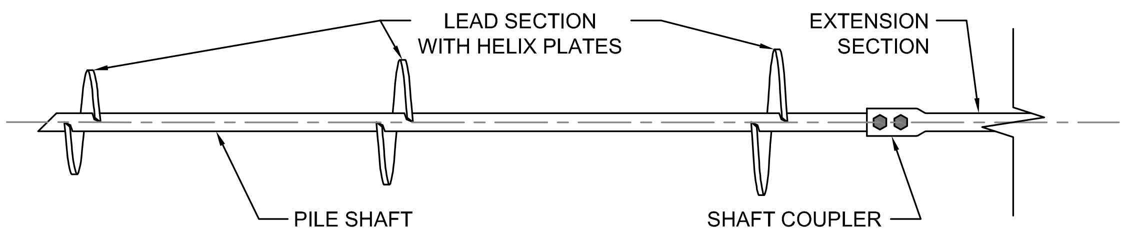
Helix Plates:
ASTM A572 Grade 50 material
⅜” thick (standard)
½” thick (available)
Helix plate geometry conforming to ICC-ES AC358

Shaft Coupling Hardware:
(2) - Ø¾” Grade 8 bolts with nuts
Mechanically galvanized per ASTM B695

Surface Finish of Shaft Segments:
Available plain or hot-dip galvanized(2)



	Plain	Plain Corroded(1)	Galvanized Corroded(1,2)
t (in)	1.750	1.714	1.740
G (in)	2.268	2.232	2.258
R (in)	0.250	0.232	0.245
A (in^2)	3.01	2.89	2.98
I_x, I_y (in^4)	0.75	0.69	0.73
I_z (in^4)	0.75	0.69	0.73
S_x, S_y (in^3)	0.85	0.80	0.84
S_z (in^3)	0.66	0.62	0.65
Z_x, Z_y (in^3)	1.30	1.22	1.27
Z_z (in^3)	1.23	1.16	1.21
r_x, r_y (in)	0.50	0.49	0.50
r_z (in)	0.50	0.49	0.50
Shaft Max Allowable Compression Capacity(3,4) P_n/Ω (kips)	**59.6**	**54.1**	**54.2**
Shaft Max Allowable Tension Capacity(4) P_n/Ω (kips)	**59.6**	**54.1**	**54.2**



Default Torque Correlation Factor(5) K_t = 10 (ft^{-1})	**Maximum Ultimate Soil Capacity(6) Q_u = 100.0 (kips)**
Maximum Installation Torque T = 10,000 (ft-lb)	**Maximum Allowable Soil Capacity(6) Q_a = 50.0 (kips) FOS = 2.0**

(1) Corroded properties and capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Square shaft piles may be considered for compression applications in soil profiles that offer sufficient continuous lateral support; e.g., in soils with SPT blow counts ≥ 10. In profiles or conditions that limit pile stability, buckling analyses should be considered by the project engineer, taking into account discontinuities and potential eccentricities created by the couplers.

(4) Listed mechanical capacities are for the shaft and coupled connections only. System capacity should also not exceed the installed allowable torque-correlated soil capacity or the allowable capacity of the respective bracket (see additional bracket tables).

(5) Listed default K_t factor is consistent with that listed in ICC-ES AC358. This value is generally conservative. Site-specific K_t factors can be determined for a given project with full-scale load testing.

(6) Soil capacities listed are at maximum installation torque. Ultimate soil capacity is based on the equation $Q_u = K_t \times T$. Allowable soil capacity is obtained by dividing the ultimate value by the appropriate factor of safety ($Q_a = Q_u$ / FOS). FOS is most commonly taken as 2.0, although a higher or lower FOS may be considered at the discretion of the helical pile designer or as dictated by local code or project requirements. System capacity should also not exceed the mechanical capacity of the shaft or those listed in the respective bracket capacity tables.

HA175NCB Bracket Specifications and Capacities
when used with the HA175 Helical Pile System

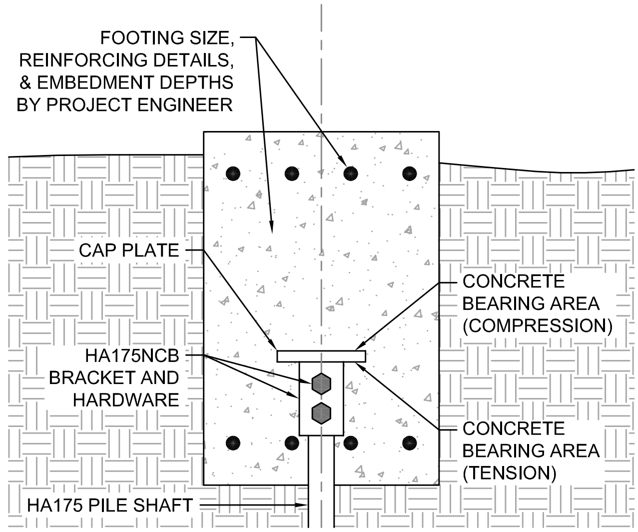
Bracket Sleeve Material:
Ø3.000” x 0.313” wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Cap Plate Material:
¾” x 6.00” square
ASTM A36

Bracket Hardware:
(2) - Ø¾” Grade 8 bolts with nuts
Mechanically galvanized per ASTM B695

Bracket Finish:
Available plain or hot-dip galvanized[2]

Concrete Bearing Area[5] (Compression) = 36.0 in^2
Concrete Bearing Area[5] (Tension) = 28.9 in^2



		Allowable Bracket Capacity[4] R_n/Ω			
		Compression[3] (kips)	Concrete Bearing[5] (ksi)	Tension (kips)	Concrete Bearing[5] (ksi)
2 Bolt	Plain	59.2	1.64	54.3	1.88
2 Bolt	Plain Corroded[1]	49.8	1.40	48.7	1.71
2 Bolt	Galvanized Corroded[1,2]	54.2	1.51	52.7	1.83
1 Bolts	Plain	29.6	0.82	29.6	1.02
1 Bolts	Plain Corroded[1]	24.9	0.70	24.9	0.87
1 Bolts	Galvanized Corroded[1,2]	27.1	0.76	27.1	0.94

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Square shaft piles may be considered for compression applications in soil profiles that offer sufficient continuous lateral support; e.g., in soils with SPT blow counts ≥ 10. In profiles or conditions that limit pile stability, buckling analyses should be considered by the project engineer, taking into account discontinuities and potential eccentricities created by the couplers.

(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

(5) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f’c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

HA175NCB8 Bracket Specifications and Capacities

when used with the HA175 Helical Pile System

Bracket Sleeve Material:

Ø3.000” x 0.313” wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Cap Plate Material:

¾” x 8.00” square
ASTM A36

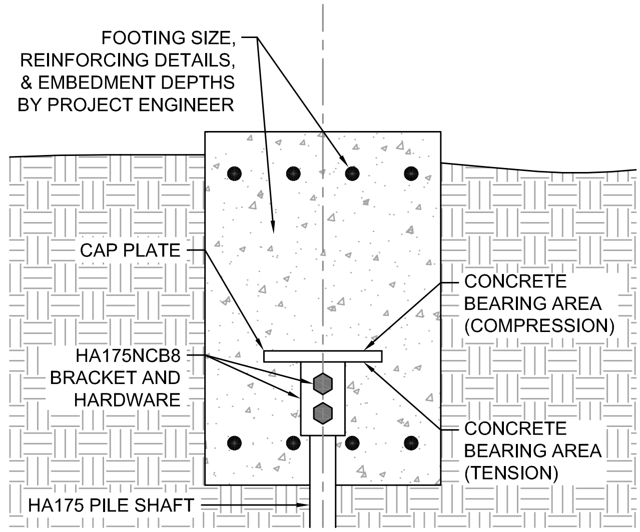
Bracket Hardware:

(2) - Ø¾” Grade 8 bolts with nuts
Mechanically galvanized per ASTM B695

Bracket Finish:

Available plain or hot-dip galvanized[2]

Concrete Bearing Area[5] (Compression) = 64.0 in^2
Concrete Bearing Area[5] (Tension) = 56.9 in^2



		Allowable Bracket Capacity[4] R_n/Ω			
		Compression[3] (kips)	Concrete Bearing[5] (ksi)	Tension (kips)	Concrete Bearing[5] (ksi)
2 Bolt	Plain	49.6	0.78	44.2	0.78
	Plain Corroded[1]	44.7	0.70	39.7	0.70
	Galvanized Corroded[1,2]	48.2	0.76	42.9	0.76
1 Bolts	Plain	29.6	0.46	29.6	0.52
	Plain Corroded[1]	24.9	0.39	24.9	0.44
	Galvanized Corroded[1,2]	27.1	0.42	27.1	0.48

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Square shaft piles may be considered for compression applications in soil profiles that offer sufficient continuous lateral support; e.g., in soils with SPT blow counts ≥ 10. In profiles or conditions that limit pile stability, buckling analyses should be considered by the project engineer, taking into account discontinuities and potential eccentricities created by the couplers.

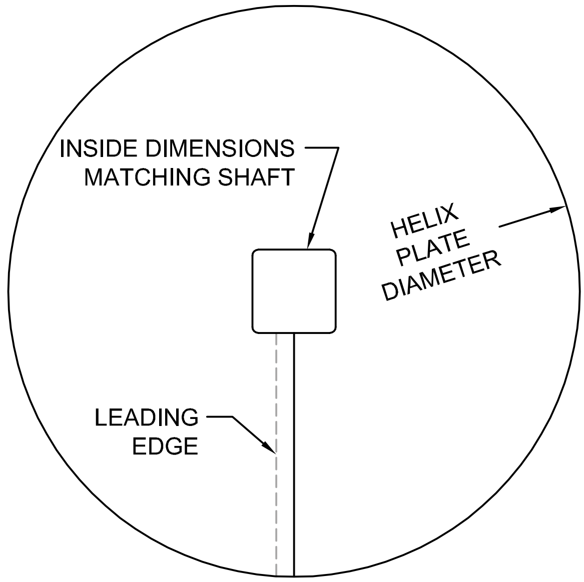
(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

(5) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f’c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

HA175 Helix Plate Net Bearing Areas

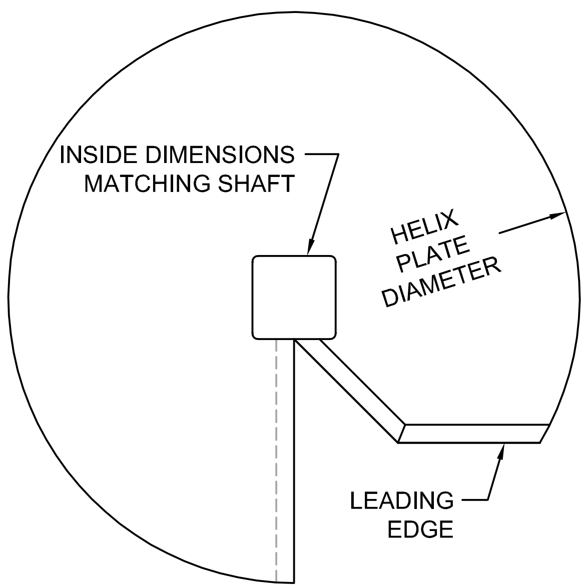
H-Style Plates

Plate Diameter (inches)	Area (ft^2)
6	0.18
8	0.33
10	0.52
12	0.76
14	1.05
16	1.38



V-Style Plates(1)

Plate Diameter (inches)	Area (ft^2)
6	0.16
8	0.29
10	0.46
12	0.67
14	0.92
16	1.20



(1) V-Style plates feature a special cut on the leading edge (or cutting edge). This edge is cut at two successive 45° angles to roughly simulate a spiral. This is in addition to the 45° bevel on the leading edge which is a standard feature for helix plates of both styles. V-Style plates are appropriate for use in applications where rocky or rubble-filled soils are anticipated, or where very dense layers need to be penetrated.

HP237 Shaft Specifications and Capacities

Shaft Material:
Ø2.375” x 0.154” wall
ASTM A500 Grade B or C
Yield strength = 60 ksi (min)
Tensile strength = 70 ksi (min)

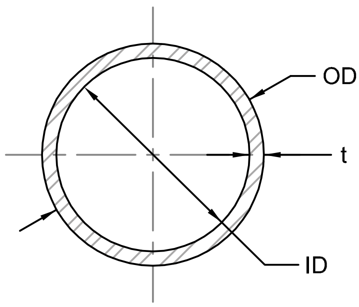
Shaft Coupler Material:
Ø2.750” x 0.156” wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Helix Plates:
ASTM A572 Grade 50 material
5⁄16” thick (standard)
3⁄8” thick (available)
Helix plate geometry conforming to ICC-ES AC358

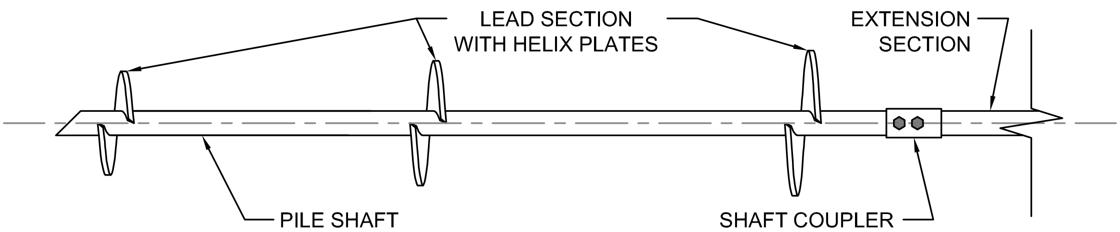
Shaft Coupling Hardware:
(2) - Ø5⁄8” ASTM A325 bolts with nuts
Hot-dip galvanized per ASTM A153

Surface Finish of Shaft Segments:
Available plain or hot-dip galvanized(2)

Nominal Thickness	0.154 (in)
Design Thickness(3)	0.143 (in)



	Plain	Plain Corroded(1)	Galvanized Corroded(1,2)
OD (in)	2.375	2.339	2.365
t (in)	0.143	0.107	0.133
ID (in)	2.089	2.125	2.099
A (in^2)	1.00	0.75	0.93
I (in^4)	0.63	0.47	0.58
S (in^3)	0.53	0.40	0.49
Z (in^3)	0.71	0.53	0.66
r (in)	0.79	0.79	0.79
Shaft Max Allowable Compression Capacity(4,5) P_n/Ω (kips)	**35.1**	**26.3**	**32.6**
Shaft Max Allowable Tension Capacity(5) P_n/Ω (kips)	**19.3**	**13.6**	**16.9**



Torque Correlation Factor(6) K_t = 10 (ft^{-1})	**Maximum Ultimate Soil Capacity(7) Q_u = 25.0 (kips)**
Maximum Installation Torque T = 2,500 (ft-lb)	**Maximum Allowable Soil Capacity(7) Q_a = 12.5 (kips) FOS = 2.0**

(1) Corroded properties and capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Design thickness for HSS and Pipe based on 93% of nominal thickness per AISC.

(4) Allowable mechanical compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(5) Listed mechanical capacities are for the shaft and coupled connections only. System capacity should also not exceed the installed allowable torque-correlated soil capacity or the allowable capacity of the respective bracket (see additional bracket tables).

(6) Listed K_t factor is that recommended by FSI. Site-specific K_t factors can be determined for a given project with full-scale load testing.

(7) Soil capacities listed are at maximum installation torque. Ultimate soil capacity is based on the equation $Q_u = K_t \times T$. Allowable soil capacity is obtained by dividing the ultimate value by the appropriate factor of safety ($Q_a = Q_u$ / FOS). FOS is most commonly taken as 2.0, although a higher or lower FOS may be considered at the discretion of the helical pile designer or as dictated by local code or project requirements. System capacity should also not exceed the mechanical capacity of the shaft or those listed in the respective bracket capacity tables.

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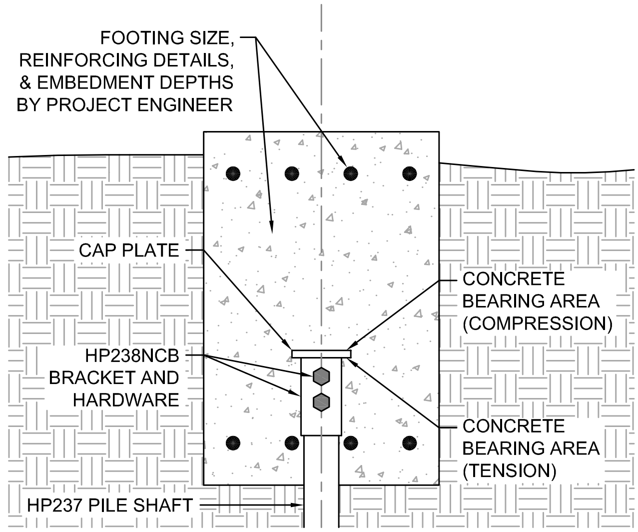
HP238NCB Bracket Specifications and Capacities
when used with the HP237 Helical Pile System



Bracket Sleeve Material:
Ø2.750” x 0.156” wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Cap Plate Material:
½” x 4.00” square
ASTM A36

Bracket Hardware:
(2) - Ø⅝” ASTM A325 bolts with nuts
Hot-dip galvanized per ASTM A153

Bracket Finish:
Available plain or hot-dip galvanized(2)

Concrete Bearing Area(6) (Compression) = 16.0 in^2
Concrete Bearing Area(6) (Tension) = 10.1 in^2

		Allowable Bracket Capacity(4) R_n/Ω			
		Compression(3) (kips)	Concrete Bearing(6) (ksi)	Tension (kips)	Concrete Bearing(6) (ksi)
2 Bolts	Plain	35.1	2.19	19.3	1.92
2 Bolts	Plain Corroded(1)	26.3	1.67	13.6	1.39
2 Bolts	Galvanized Corroded(1,2)	32.6	2.05	16.9	1.70
1 Bolt	Plain	35.1	2.19	9.7	0.96
1 Bolt	Plain Corroded(1)	26.3	1.67	6.8	0.70
1 Bolt	Galvanized Corroded(1,2)	32.6	2.05	8.5	0.85
0 Bolts(5)	Plain	35.1	2.19	0	0
0 Bolts(5)	Plain Corroded(1)	26.3	1.67	0	0
0 Bolts(5)	Galvanized Corroded(1,2)	32.6	2.05	0	0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

(5) Applications utilizing no bolts should either be tack welded or utilize some other mechanism to immobilize the bracket and maintain firm contact between the cap plate and pile shaft throughout construction and concrete placement.

(6) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f’c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

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HP238B2 Bracket Specifications and Capacities

when used with the HP237 Helical Pile System

Bracket:
Weldment manufactured from ⅜”
ASTM A36 plate.

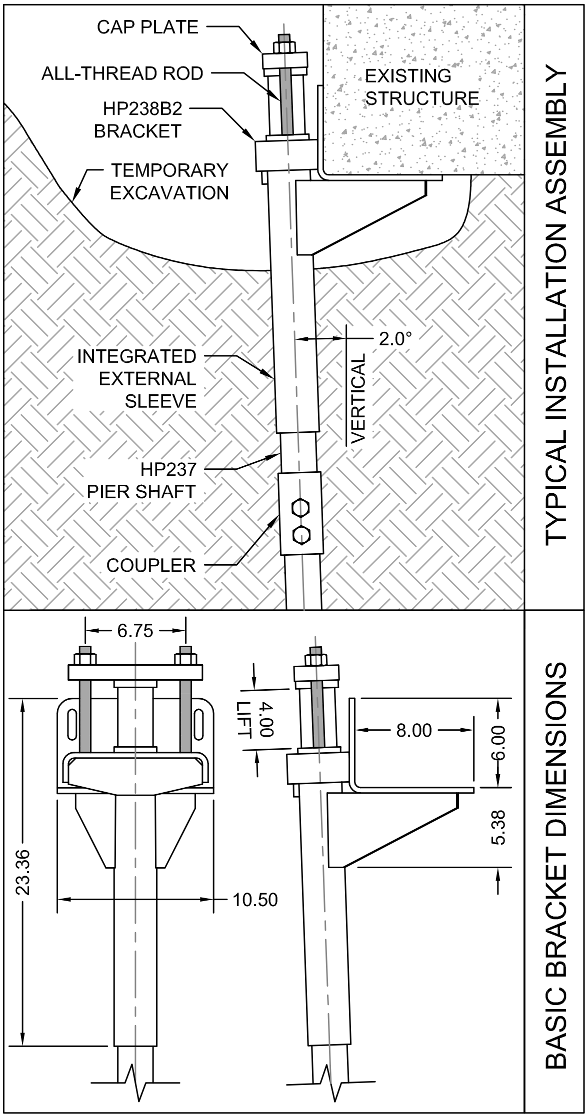
Integrated External Sleeve:
Ø2.875” x 0.203” wall x 20” long
ASTM A500 Grade B or C
Yield strength = 60 ksi (min)
Tensile strength = 70 ksi (min)

Cap Plate:
1” x 3.00” x 9.00” ASTM A572 Grade 50
with confining ring welded to one side.

Bracket Hardware(3):
(2) - Ø¾” x 16” long all-thread rod
Grade B7, tensile strength = 125 ksi (min)
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized(2)

Allowable Bracket Capacity(4,5,6,7) R_n/Ω	
	(kips)
Plain	10.9
Plain Corroded (1)	8.3
Galvanized Corroded (1,2)	10.2



(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Optional hardware utilizes similar sized contour (coil) thread made from AISI 1045, tensile strength = 120 ksi. Slightly lower tensile strength material does not govern the listed capacities.

(4) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(5) Allowable capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(6) Concrete bearing assumes a minimum compressive strength (f’c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

(7) Listed allowable capacities are for the specific shaft/bracket combination shown. System capacity should also not exceed the installed torque-correlated soil capacity (See Shaft Specifications and Capacities).

HP238BML Bracket Specifications and Capacities

when used with the HP237 Helical Pile System

Bracket:

Weldment manufactured from ⅜” and 1” ASTM A36 plate.

Integrated External Sleeve:

Ø2.875” x 0.203” wall x 15” long
ASTM A500 Grade B or C
Yield strength = 60 ksi (min)
Tensile strength = 70 ksi (min)

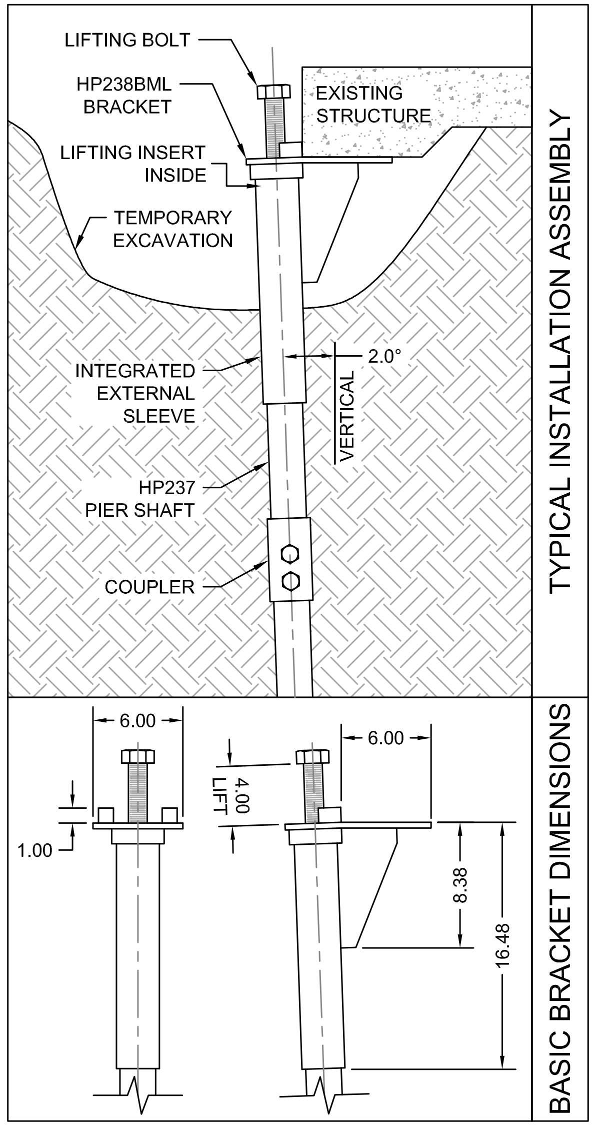
Bracket Hardware:

(1) - Ø1¼” x 6” long lifting bolt
Grade 5, tensile strength = 120 ksi (min)
Available electrozinc plated per ASTM B633
or hot-dip galvanized per ASTM A153

Bracket Finish:

Available plain or hot-dip galvanized(2)

Allowable Bracket Capacity(3,4,5,6) R_n/Ω	
	(kips)
Plain	6.0
Plain Corroded (1)	4.6
Galvanized Corroded (1,2)	5.6



(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(4) Allowable capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

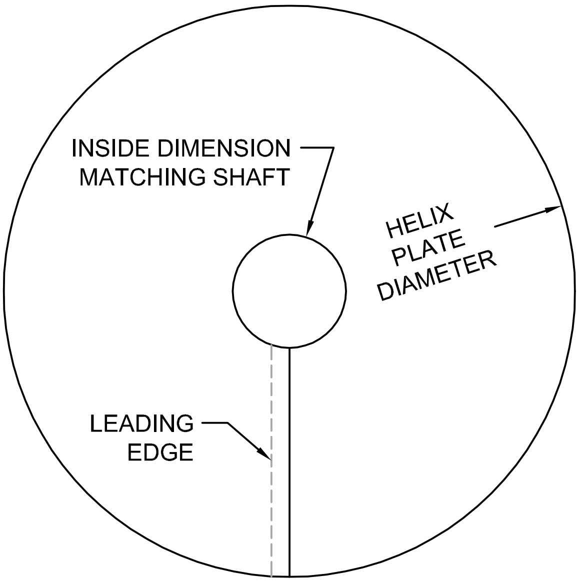
(5) Concrete bearing assumes a minimum compressive strength (f’c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

(6) Listed allowable capacities are for the specific shaft/bracket combination shown. System capacity should also not exceed the installed torque-correlated soil capacity (See Shaft Specifications and Capacities).

HP237 Helix Plate Net Bearing Areas

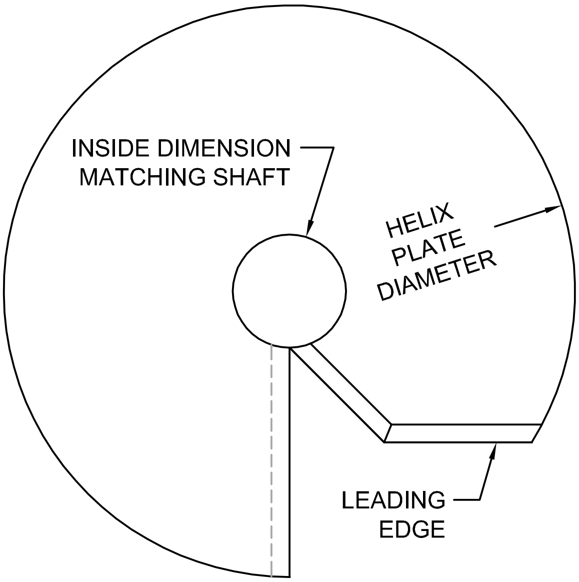
H-Style Plates

Plate Diameter (inches)	Area (ft^2)
6	0.17
8	0.32
10	0.51
12	0.75
14	1.04
16	1.37



V-Style Plates[1]

Plate Diameter (inches)	Area (ft^2)
6	0.15
8	0.29
10	0.46
12	0.67
14	0.92
16	1.20



(1) V-Style plates feature a special cut on the leading edge (or cutting edge). This edge is cut at two successive 45° angles to roughly simulate a spiral. This is in addition to the 45° bevel on the leading edge which is a standard feature for helix plates of both styles. V-Style plates are appropriate for use in applications where rocky or rubble-filled soils are anticipated, or where very dense layers need to be penetrated.

HP287 Shaft Specifications and Capacities

Shaft Material:
Ø2.875" x 0.203" wall
ASTM A500 Grade B or C
Yield strength = 60 ksi (min)
Tensile strength = 70 ksi (min)

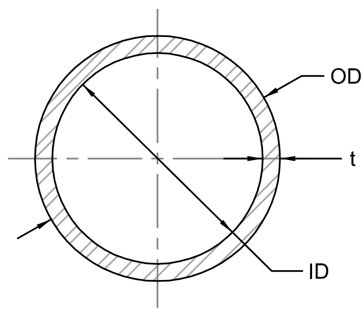
Shaft Coupler Material:
Ø3.500" x 0.281" wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Helix Plates:
ASTM A572 Grade 50 material
⅜" thick (standard)
½" thick (available)
Helix plate geometry conforming to ICC-ES AC358

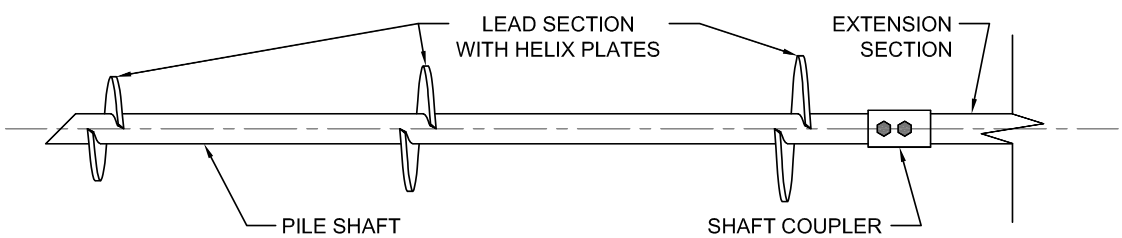
Shaft Coupling Hardware:
(2) - Ø¾" Grade 8 bolts with nuts
Electrozinc plated per ASTM B633

Surface Finish of Shaft Segments:
Available plain or hot-dip galvanized[2]

Nominal Thickness	0.203 (in)
Design Thickness[3]	0.189 (in)



	Plain	Plain Corroded[1]	Galvanized Corroded[1,2]
OD (in)	2.875	2.839	2.865
t (in)	0.189	0.153	0.179
ID (in)	2.497	2.533	2.507
A (in²)	1.59	1.29	1.51
I (in⁴)	1.45	1.17	1.37
S (in³)	1.01	0.82	0.96
Z (in³)	1.37	1.11	1.29
r (in)	0.95	0.95	0.95
Shaft Max Allowable Compression Capacity[4,5] P_n/Ω (kips)	**55.8**	**45.2**	**52.9**
Shaft Max Allowable Tension Capacity[5] P_n/Ω (kips)	**30.6**	**23.6**	**27.6**



Default Torque Correlation Factor[6] K_t = 9 (ft⁻¹)	**Maximum Ultimate Soil Capacity[7] Q_u = 50.4 (kips)**
Maximum Installation Torque T = 5,600 (ft-lb)	**Maximum Allowable Soil Capacity[7] Q_a = 25.2 (kips) FOS = 2.0**

(1) Corroded properties and capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Design thickness for HSS and Pipe based on 93% of nominal thickness per AISC.

(4) Allowable mechanical compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(5) Listed mechanical capacities are for the shaft and coupled connections only. System capacity should also not exceed the installed allowable torque-correlated soil capacity or the allowable capacity of the respective bracket (see additional bracket tables).

(6) Listed default K_t factor is consistent with that listed in ICC-ES AC358. This value is generally conservative. Site-specific K_t factors can be determined for a given project with full-scale load testing.

(7) Soil capacities listed are at maximum installation torque. Ultimate soil capacity is based on the equation $Q_u = K_t \times T$. Allowable soil capacity is obtained by dividing the ultimate value by the appropriate factor of safety ($Q_a = Q_u$ / FOS). FOS is most commonly taken as 2.0, although a higher or lower FOS may be considered at the discretion of the helical pile designer or as dictated by local code or project requirements. System capacity should also not exceed the mechanical capacity of the shaft or those listed in the respective bracket capacity tables.

HP288NCB Bracket Specifications and Capacities
when used with the HP287 Helical Pile System

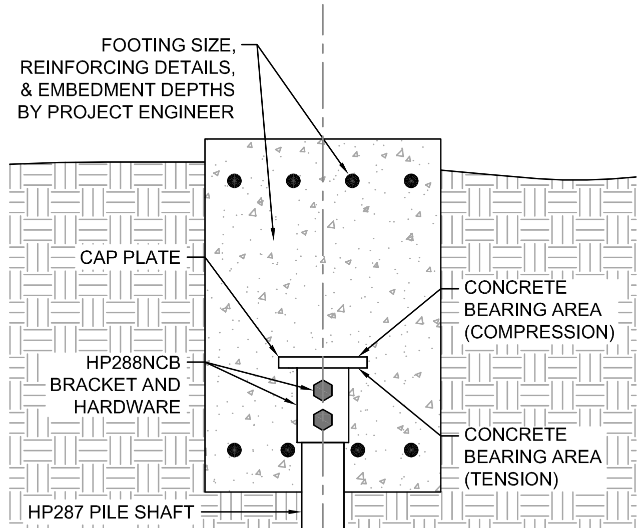
Bracket Sleeve Material:
Ø3.500" x 0.250" wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Cap Plate Material:
¾" x 6.00" square
ASTM A36

Bracket Hardware:
(2) - Ø¾" Grade 8 bolts with nuts
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized(2)

Concrete Bearing Area(6) (Compression) = 36.0 in²
Concrete Bearing Area(6) (Tension) = 26.4 in²



		Allowable Bracket Capacity(4) R_n/Ω			
		Compression(3) (kips)	Concrete Bearing(6) (ksi)	Tension (kips)	Concrete Bearing(6) (ksi)
2 Bolts	Plain	55.8	1.55	30.6	1.16
2 Bolts	Plain Corroded(1)	45.2	1.27	23.6	0.91
2 Bolts	Galvanized Corroded(1,2)	52.9	1.47	27.6	1.05
1 Bolt	Plain	55.8	1.55	15.3	0.58
1 Bolt	Plain Corroded(1)	45.2	1.27	11.8	0.45
1 Bolt	Galvanized Corroded(1,2)	52.9	1.47	13.8	0.53
0 Bolts(5)	Plain	55.8	1.55	0	0
0 Bolts(5)	Plain Corroded(1)	45.2	1.27	0	0
0 Bolts(5)	Galvanized Corroded(1,2)	52.9	1.47	0	0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

(5) Applications utilizing no bolts should either be tack welded or utilize some other mechanism to immobilize the bracket and maintain firm contact between the cap plate and pile shaft throughout construction and concrete placement.

(6) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f'c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

HP288NCB8 Bracket Specifications and Capacities
when used with the HP287 Helical Pile System

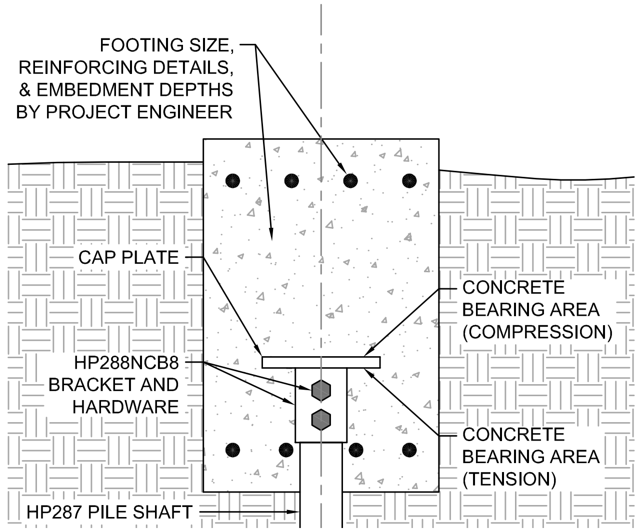
Bracket Sleeve Material:
Ø3.500” x 0.250” wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Cap Plate Material:
¾” x 8.00” square
ASTM A36

Bracket Hardware:
(2) - Ø¾” Grade 8 bolts with nuts
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized(2)

Concrete Bearing Area(6) (Compression) = 64.0 in²
Concrete Bearing Area(6) (Tension) = 54.4 in²



		Allowable Bracket Capacity(4) R_n/Ω			
		Compression(3) (kips)	Concrete Bearing(6) (ksi)	Tension (kips)	Concrete Bearing(6) (ksi)
2 Bolts	Plain	47.9	0.75	30.6	0.56
2 Bolts	Plain Corroded(1)	43.1	0.68	23.6	0.44
2 Bolts	Galvanized Corroded(1,2)	46.6	0.73	27.6	0.51
1 Bolt	Plain	47.9	0.75	15.3	0.28
1 Bolt	Plain Corroded(1)	43.1	0.68	11.8	0.22
1 Bolt	Galvanized Corroded(1,2)	46.6	0.73	13.8	0.25
0 Bolts(5)	Plain	47.9	0.75	0	0
0 Bolts(5)	Plain Corroded(1)	43.1	0.68	0	0
0 Bolts(5)	Galvanized Corroded(1,2)	46.6	0.73	0	0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

(5) Applications utilizing no bolts should either be tack welded or utilize some other mechanism to immobilize the bracket and maintain firm contact between the cap plate and pile shaft throughout construction and concrete placement.

(6) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f’c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

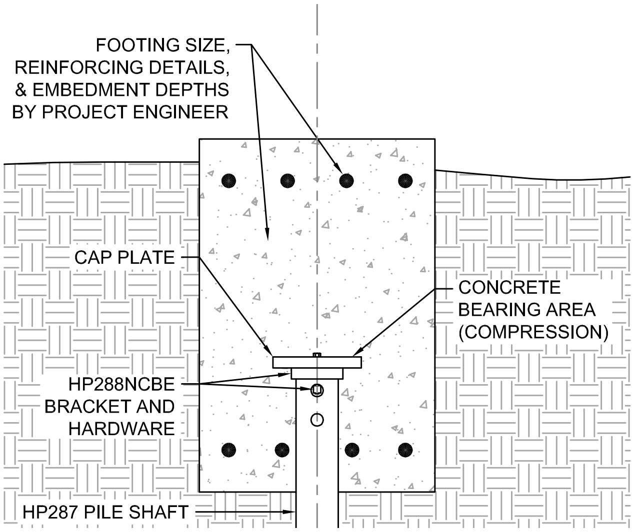
HP288NCBE Bracket Specifications and Capacities
when used with the HP287 Helical Pile System

Cap Plate Material:
¾" x 6.00" square
ASTM A36

Bracket Hardware[5]:
½" square bar stock with tapped hole and Ø¼" retention bolt

Bracket Finish:
Available plain or hot-dip galvanized[2]

Concrete Bearing Area[6] (Compression) = 36.0 in²
Concrete Bearing Area[6] (Tension) = N/A



	Allowable Bracket Capacity[4] R_n/Ω			
	Compression[3] (kips)	Concrete Bearing[6] (ksi)	Tension (kips)	Concrete Bearing[6] (ksi)
Plain	54.0	1.10	0	0
Plain Corroded[1]	45.2	0.93	0	0
Galvanized Corroded[1,2]	52.4	1.07	0	0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

(5) Supplied bracket hardware does not contribute to the system strength and is only intended to immobilize the bracket. Other methods, such as tack welds, may be substituted to maintain firm contact between the cap plate and pile shaft throughout construction and concrete placement at the discretion of the project engineer.

(6) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f'c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

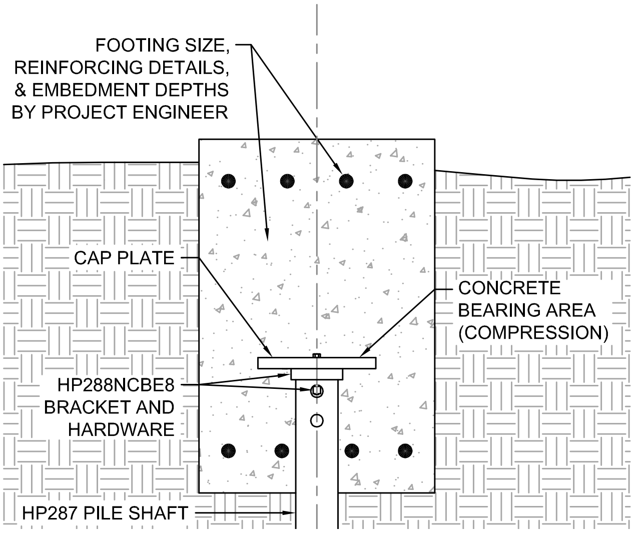
HP288NCBE8 Bracket Specifications and Capacities
when used with the HP287 Helical Pile System

Cap Plate Material:
¾” x 8.00” square
ASTM A36

Bracket Hardware(5):
½” square bar stock with tapped hole and Ø¼” retention bolt

Bracket Finish:
Available plain or hot-dip galvanized(2)

Concrete Bearing Area(6) (Compression) = 64.0 in²
Concrete Bearing Area(6) (Tension) = N/A



	Allowable Bracket Capacity(4) R_n/Ω			
	Compression(3) (kips)	Concrete Bearing(6) (ksi)	Tension (kips)	Concrete Bearing(6) (ksi)
Plain	47.9	0.75	0	0
Plain Corroded(1)	43.1	0.68	0	0
Galvanized Corroded(1,2)	46.6	0.73	0	0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

(5) Supplied bracket hardware does not contribute to the system strength and is only intended to immobilize the bracket. Other methods, such as tack welds, may be substituted to maintain firm contact between the cap plate and pile shaft throughout construction and concrete placement at the discretion of the project engineer.

(6) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f’c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

FS288B Bracket Specifications and Capacities
when used with the HP287 Helical Pile System

Bracket:
Weldment manufactured from ¼”, ⅜”, and ½” ASTM A36 plate.

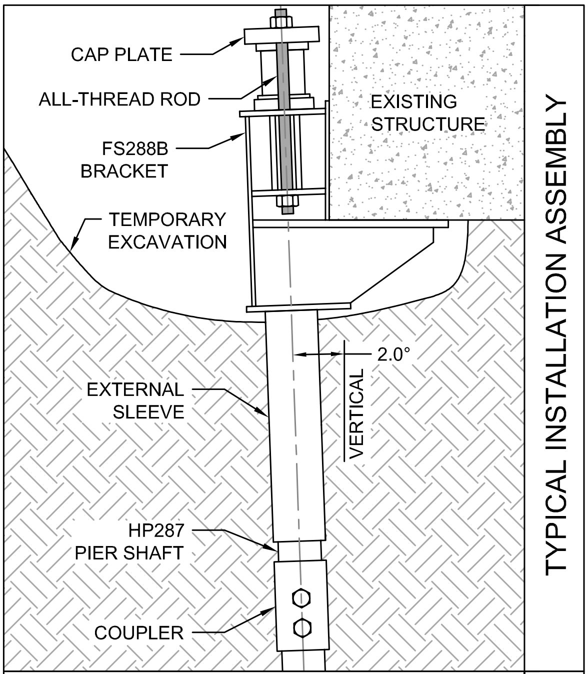
External Sleeve:
Ø3.500” x 0.216” wall x 30” long
with welded collar or trumpet flare at one end.
ASTM A500 Grade B or C
Yield strength = 50 ksi (min)
Tensile strength = 62 ksi (min)

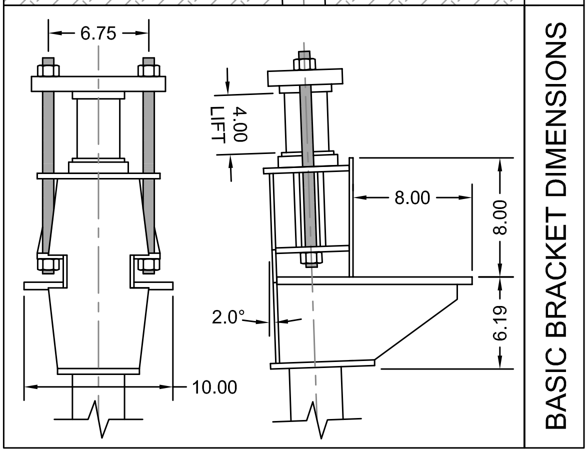
Cap Plate:
1” x 5.00” x 9.00” ASTM A572 Grade 50
with confining ring welded to one side.

Bracket Hardware(3):
(2) - Ø¾” x 16” long all-thread rod
Grade B7, tensile strength = 125 ksi (min)
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized(2)

Allowable Bracket Capacity(4,5,6,7) R_n/Ω	
	(kips)
Plain	25.5
Plain Corroded (1)	20.9
Galvanized Corroded (1,2)	24.2





(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Optional hardware utilizes similar sized contour (coil) thread made from AISI 1045, tensile strength = 120 ksi. Slightly lower tensile strength material does not govern the listed capacities.

(4) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(5) Allowable capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(6) Concrete bearing assumes a minimum compressive strength (f’c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

(7) Listed allowable capacities are for the specific shaft/bracket combination shown. System capacity should also not exceed the installed torque-correlated soil capacity (See Shaft Specifications and Capacities).

FS288BL Bracket Specifications and Capacities
when used with the HP287 Helical Pile System

Bracket:
Weldment manufactured from ¼”, ⅜”, and ½” ASTM A36 plate.

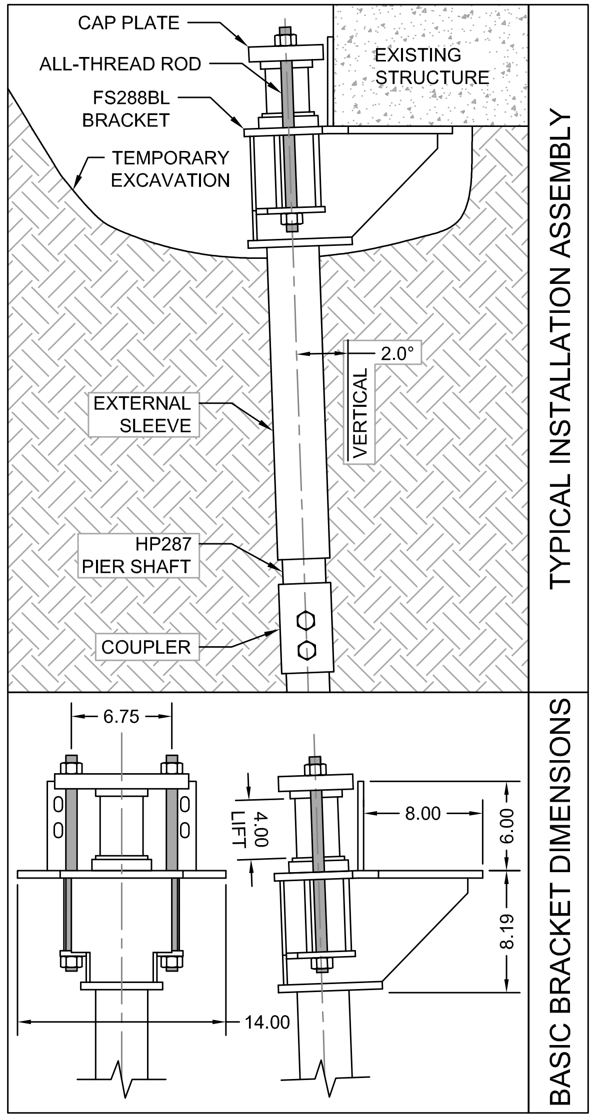
External Sleeve:
Ø3.500” x 0.216” wall x 30” long
with welded collar or trumpet flare at one end.
ASTM A500 Grade B or C
Yield strength = 50 ksi (min)
Tensile strength = 62 ksi (min)

Cap Plate:
1” x 5.00” x 9.00” ASTM A572 Grade 50
with confining ring welded to one side.

Bracket Hardware(3):
(2) - Ø¾” x 16” long all-thread rod
Grade B7, tensile strength = 125 ksi (min)
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized(2)

Allowable Bracket Capacity(4,5,6,7) R_n/Ω	
	(kips)
Plain	25.5
Plain Corroded (1)	20.9
Galvanized Corroded (1,2)	24.2



(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Optional hardware utilizes similar sized contour (coil) thread made from AISI 1045, tensile strength = 120 ksi. Slightly lower tensile strength material does not govern the listed capacities.

(4) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(5) Allowable capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(6) Concrete bearing assumes a minimum compressive strength (f’c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

(7) Listed allowable capacities are for the specific shaft/bracket combination shown. System capacity should also not exceed the installed torque-correlated soil capacity (See Shaft Specifications and Capacities).

HP288B2 Bracket Specifications and Capacities
when used with the HP287 Helical Pile System

Bracket:
Weldment manufactured from ⅜”
ASTM A36 plate.

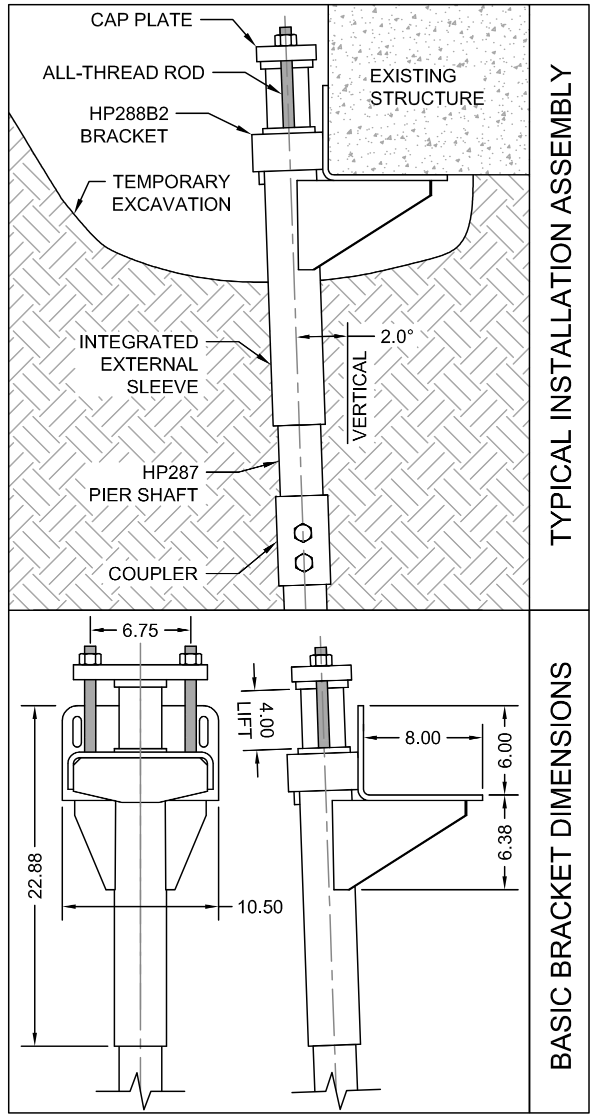
Integrated External Sleeve:
Ø3.500” x 0.216” wall x 20” long
ASTM A500 Grade B or C
Yield strength = 50 ksi (min)
Tensile strength = 62 ksi (min)

Cap Plate:
1” x 4.00” x 9.00” ASTM A572 Grade 50
with confining ring welded to one side.

Bracket Hardware(3):
(2) - Ø¾” x 16” long all-thread rod
Grade B7, tensile strength = 125 ksi (min)
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized(2)

Allowable Bracket Capacity(4,5,6,7) R_n/Ω	
	(kips)
Plain	21.0
Plain Corroded (1)	17.2
Galvanized Corroded (1,2)	19.9



(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Optional hardware utilizes similar sized contour (coil) thread made from AISI 1045, tensile strength = 120 ksi. Slightly lower tensile strength material does not govern the listed capacities.

(4) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(5) Allowable capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(6) Concrete bearing assumes a minimum compressive strength (f’c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

(7) Listed allowable capacities are for the specific shaft/bracket combination shown. System capacity should also not exceed the installed torque-correlated soil capacity (See Shaft Specifications and Capacities).

HP288BML Bracket Specifications and Capacities

when used with the HP287 Helical Pile System

Bracket:

Weldment manufactured from ⅜” ASTM A36 plate.

Integrated External Sleeve:

Ø3.500” x 0.216” wall x 20” long
ASTM A500 Grade B or C
Yield strength = 50 ksi (min)
Tensile strength = 62 ksi (min)

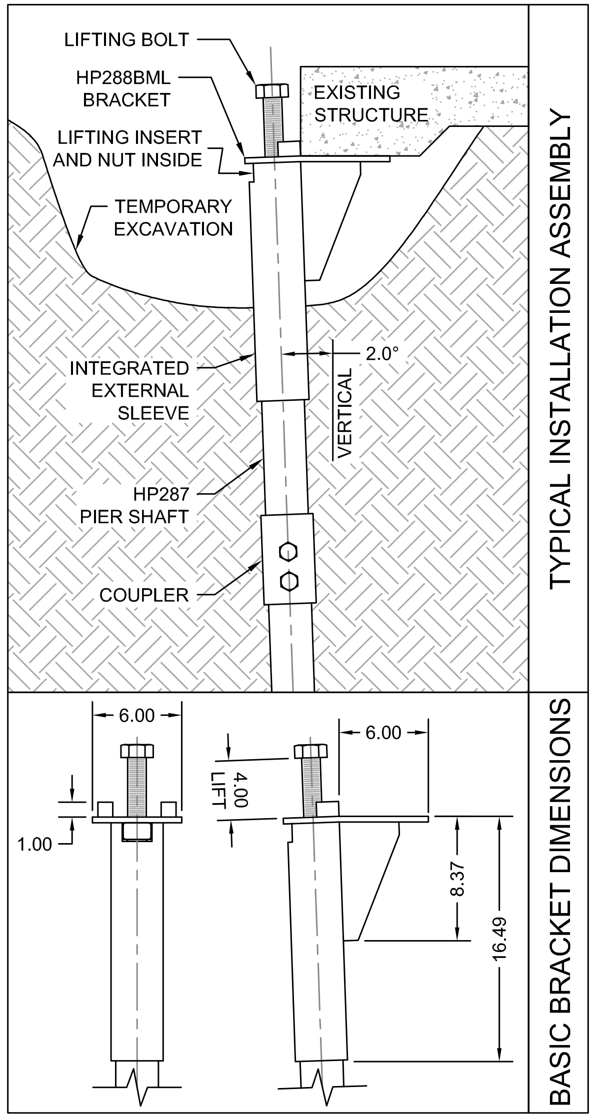
Bracket Hardware:

(1) - Ø1¼” x 6” long lifting bolt with nut
Grade 5, tensile strength = 120 ksi (min)
Available electrozinc plated per ASTM B633
or hot-dip galvanized per ASTM A153

Bracket Finish:

Available plain or hot-dip galvanized[2]

Allowable Bracket Capacity[3,4,5,6] R_n/Ω	
	(kips)
Plain	12.0
Plain Corroded [1]	9.8
Galvanized Corroded [1,2]	11.4



(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(4) Allowable capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

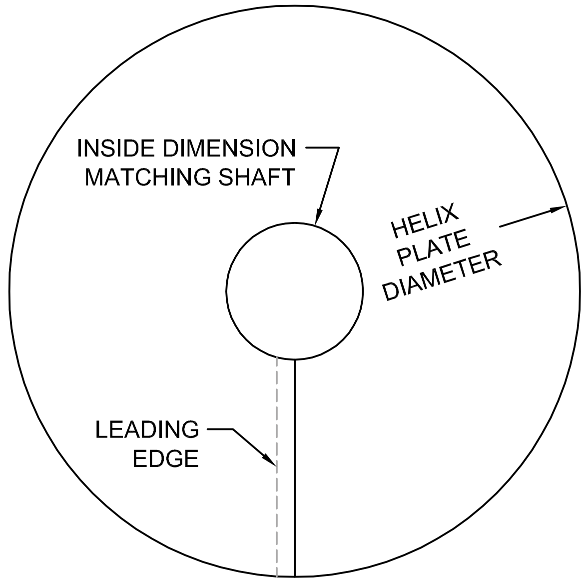
(5) Concrete bearing assumes a minimum compressive strength (f’c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

(6) Listed allowable capacities are for the specific shaft/bracket combination shown. System capacity should also not exceed the installed torque-correlated soil capacity (See Shaft Specifications and Capacities).

HP287 Helix Plate Net Bearing Areas

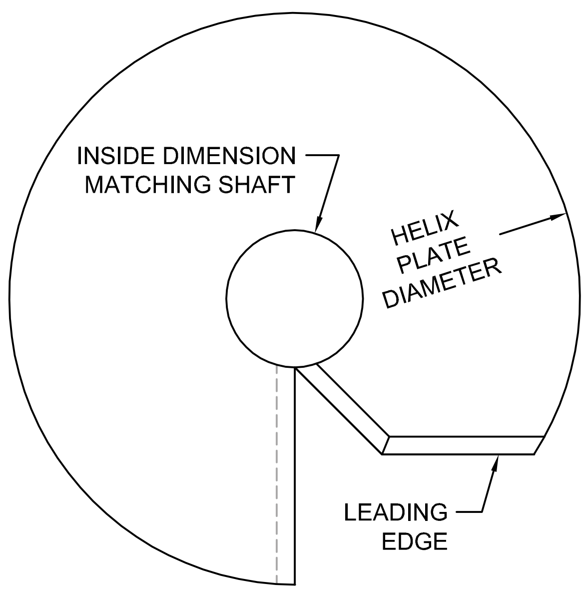
H-Style Plates

Plate Diameter (inches)	Area (ft^2)
6	0.15
8	0.30
10	0.50
12	0.74
14	1.02
16	1.35



V-Style Plates(1)

Plate Diameter (inches)	Area (ft^2)
6	0.14
8	0.28
10	0.45
12	0.66
14	0.91
16	1.19



(1) V-Style plates feature a special cut on the leading edge (or cutting edge). This edge is cut at two successive 45° angles to roughly simulate a spiral. This is in addition to the 45° bevel on the leading edge which is a standard feature for helix plates of both styles. V-Style plates are appropriate for use in applications where rocky or rubble-filled soils are anticipated, or where very dense layers need to be penetrated.

HP288 Shaft Specifications and Capacities

Shaft Material:
Ø2.875” x 0.276” wall
ASTM A500 Grade B or C
Yield strength = 60 ksi (min)
Tensile strength = 70 ksi (min)

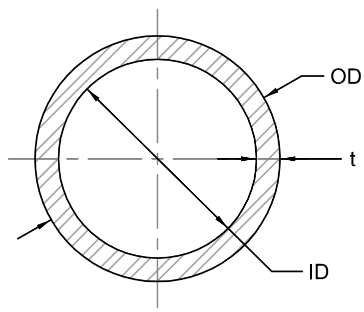
Shaft Coupler Material:
Ø3.500” x 0.281” wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Helix Plates:
ASTM A572 Grade 50 material
⅜” thick (standard)
½” thick (available)
Helix plate geometry conforming to ICC-ES AC358

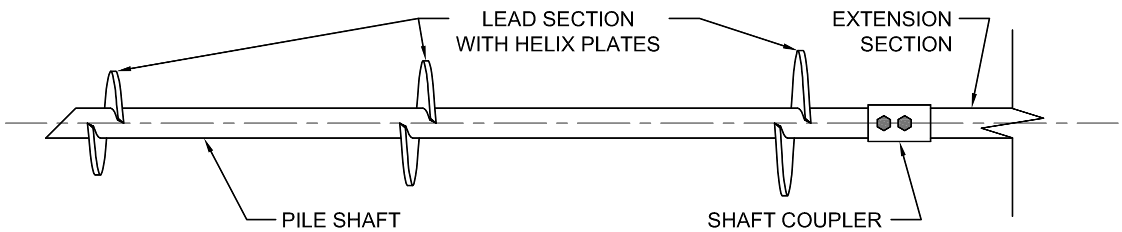
Shaft Coupling Hardware:
(2) - Ø¾” Grade 8 bolts with nuts
Electrozinc plated per ASTM B633

Surface Finish of Shaft Segments:
Available plain or hot-dip galvanized[2]

Nominal Thickness	0.276 (in)
Design Thickness[3]	0.257 (in)



	Plain	Plain Corroded[1]	Galvanized Corroded[1,2]
OD (in)	2.875	2.839	2.865
t (in)	0.257	0.221	0.247
ID (in)	2.361	2.397	2.371
A (in²)	2.11	1.82	2.03
I (in⁴)	1.83	1.57	1.76
S (in³)	1.27	1.10	1.23
Z (in³)	1.77	1.52	1.70
r (in)	0.93	0.93	0.93
Shaft Max Allowable Compression Capacity[4,5] P_n/Ω (kips)	**74.0**	**63.6**	**71.1**
Shaft Max Allowable Tension Capacity[5] P_n/Ω (kips)	**41.6**	**34.1**	**38.1**



Default Torque Correlation Factor[6] K_t = 9 (ft⁻¹)	**Maximum Ultimate Soil Capacity[7] Q_u = 71.1 (kips)**
Maximum Installation Torque T = 7,900 (ft-lb)	**Maximum Allowable Soil Capacity[7] Q_a = 35.5 (kips) FOS = 2.0**

(1) Corroded properties and capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Design thickness for HSS and Pipe based on 93% of nominal thickness per AISC.

(4) Allowable mechanical compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(5) Listed mechanical capacities are for the shaft and coupled connections only. System capacity should also not exceed the installed allowable torque-correlated soil capacity or the allowable capacity of the respective bracket (see additional bracket tables).

(6) Listed default K_t factor is consistent with that listed in ICC-ES AC358. This value is generally conservative. Site-specific K_t factors can be determined for a given project with full-scale load testing.

(7) Soil capacities listed are at maximum installation torque. Ultimate soil capacity is based on the equation $Q_u = K_t \times T$. Allowable soil capacity is obtained by dividing the ultimate value by the appropriate factor of safety ($Q_a = Q_u$ / FOS). FOS is most commonly taken as 2.0, although a higher or lower FOS may be considered at the discretion of the helical pile designer or as dictated by local code or project requirements. System capacity should also not exceed the mechanical capacity of the shaft or those listed in the respective bracket capacity tables.

HP288NCB Bracket Specifications and Capacities

when used with the HP288 Helical Pile System

Bracket Sleeve Material:

Ø3.500" x 0.250" wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Cap Plate Material:

¾" x 6.00" square
ASTM A36

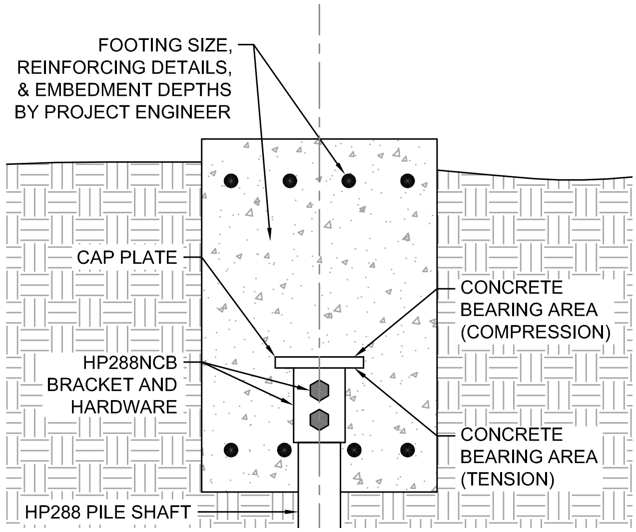
Bracket Hardware:

(2) - Ø¾" Grade 8 bolts with nuts
Electrozinc plated per ASTM B633

Bracket Finish:

Available plain or hot-dip galvanized(2)

Concrete Bearing Area(6) (Compression) = 36.0 in^2
Concrete Bearing Area(6) (Tension) = 26.4 in^2



		Allowable Bracket Capacity(4) R_n/Ω			
		Compression(3) (kips)	Concrete Bearing(6) (ksi)	Tension (kips)	Concrete Bearing(6) (ksi)
2 Bolts	Plain	64.0	1.78	41.6	1.58
2 Bolts	Plain Corroded(1)	57.5	1.62	34.1	1.31
2 Bolts	Galvanized Corroded(1,2)	62.1	1.73	38.1	1.45
1 Bolt	Plain	64.0	1.78	20.8	0.79
1 Bolt	Plain Corroded(1)	57.5	1.62	17.0	0.66
1 Bolt	Galvanized Corroded(1,2)	62.1	1.73	19.1	0.73
0 Bolts(5)	Plain	64.0	1.78	0	0
0 Bolts(5)	Plain Corroded(1)	57.5	1.62	0	0
0 Bolts(5)	Galvanized Corroded(1,2)	62.1	1.73	0	0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

(5) Applications utilizing no bolts should either be tack welded or utilize some other mechanism to immobilize the bracket and maintain firm contact between the cap plate and pile shaft throughout construction and concrete placement.

(6) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f'c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

HP288NCB8 Bracket Specifications and Capacities

when used with the HP288 Helical Pile System

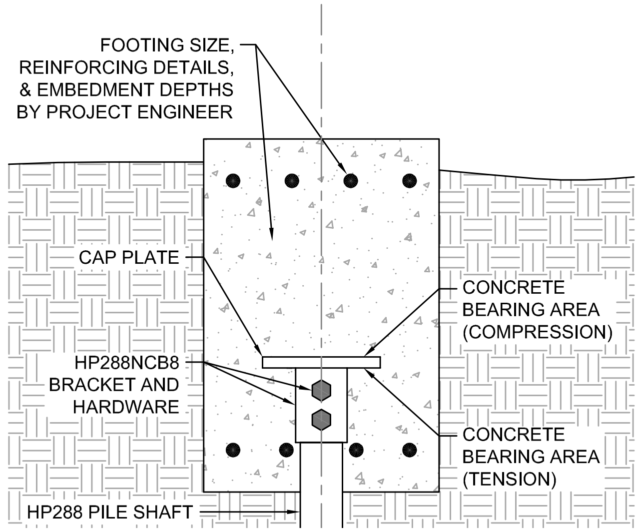
Bracket Sleeve Material:
Ø3.500" x 0.250" wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Cap Plate Material:
¾" x 8.00" square
ASTM A36

Bracket Hardware:
(2) - Ø¾" Grade 8 bolts with nuts
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized[2]

Concrete Bearing Area[6] (Compression) = 64.0 in²
Concrete Bearing Area[6] (Tension) = 54.4 in²



		Allowable Bracket Capacity[4] R_n/Ω			
		Compression[3] (kips)	Concrete Bearing[6] (ksi)	Tension (kips)	Concrete Bearing[6] (ksi)
2 Bolts	Plain	47.9	0.75	41.6	0.77
2 Bolts	Plain Corroded[1]	43.1	0.68	34.1	0.63
2 Bolts	Galvanized Corroded[1,2]	46.6	0.73	38.1	0.70
1 Bolt	Plain	47.9	0.75	20.8	0.38
1 Bolt	Plain Corroded[1]	43.1	0.68	17.0	0.32
1 Bolt	Galvanized Corroded[1,2]	46.6	0.73	19.1	0.35
0 Bolts[5]	Plain	47.9	0.75	0	0
0 Bolts[5]	Plain Corroded[1]	43.1	0.68	0	0
0 Bolts[5]	Galvanized Corroded[1,2]	46.6	0.73	0	0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

(5) Applications utilizing no bolts should either be tack welded or utilize some other mechanism to immobilize the bracket and maintain firm contact between the cap plate and pile shaft throughout construction and concrete placement.

(6) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f'c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

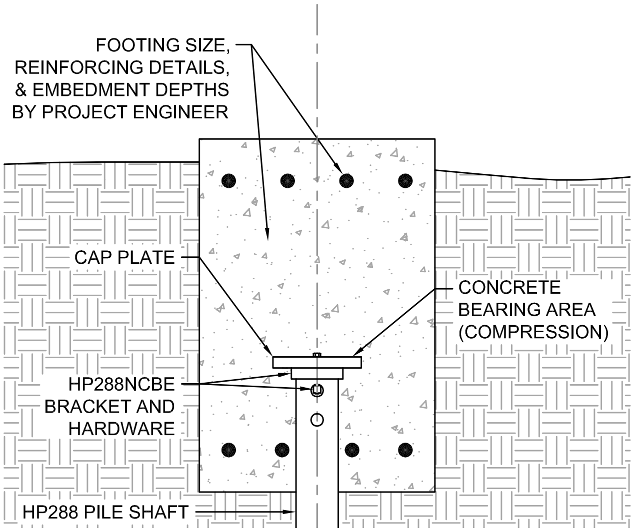
HP288NCBE Bracket Specifications and Capacities
when used with the HP288 Helical Pile System

Cap Plate Material:
¾" x 6.00" square
ASTM A36

Bracket Hardware[5]:
½" square bar stock with tapped
hole and Ø¼" retention bolt

Bracket Finish:
Available plain or hot-dip galvanized[2]

Concrete Bearing Area[6] (Compression) = 36.0 in^2
Concrete Bearing Area[6] (Tension) = N/A



	Allowable Bracket Capacity[4] R_n/Ω			
	Compression[3] (kips)	Concrete Bearing[6] (ksi)	Tension (kips)	Concrete Bearing[6] (ksi)
Plain	64.0	1.78	0	0
Plain Corroded[1]	57.5	1.62	0	0
Galvanized Corroded[1,2]	62.1	1.73	0	0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

(5) Supplied bracket hardware does not contribute to the system strength and is only intended to immobilize the bracket. Other methods, such as tack welds, may be substituted to maintain firm contact between the cap plate and pile shaft throughout construction and concrete placement at the discretion of the project engineer.

(6) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f'c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

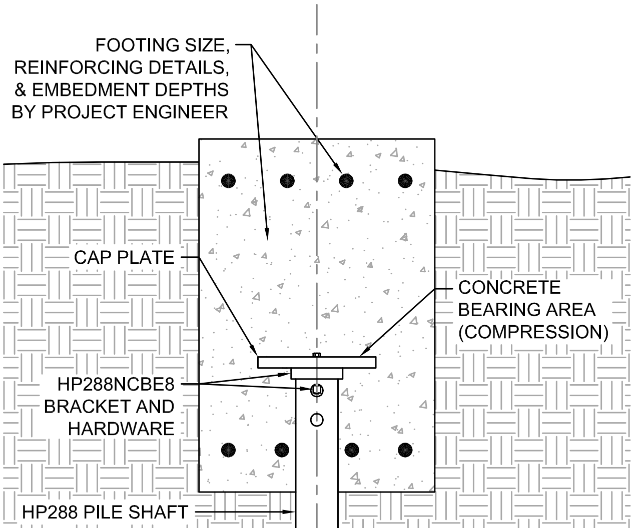
HP288NCBE8 Bracket Specifications and Capacities
when used with the HP288 Helical Pile System

Cap Plate Material:
¾" x 8.00" square
ASTM A36

Bracket Hardware(5):
½" square bar stock with tapped hole and Ø¼" retention bolt

Bracket Finish:
Available plain or hot-dip galvanized(2)

Concrete Bearing Area(6) (Compression) = 64.0 in²
Concrete Bearing Area(6) (Tension) = N/A



	Allowable Bracket Capacity(4) R_n/Ω			
	Compression(3) (kips)	Concrete Bearing(6) (ksi)	Tension (kips)	Concrete Bearing(6) (ksi)
Plain	47.9	0.75	0	0
Plain Corroded(1)	43.1	0.68	0	0
Galvanized Corroded(1,2)	46.6	0.73	0	0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

(5) Supplied bracket hardware does not contribute to the system strength and is only intended to immobilize the bracket. Other methods, such as tack welds, may be substituted to maintain firm contact between the cap plate and pile shaft throughout construction and concrete placement at the discretion of the project engineer.

(6) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f'c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

FS288B Bracket Specifications and Capacities
when used with the HP288 Helical Pile System

Bracket:
Weldment manufactured from ¼", ⅜", and ½" ASTM A36 plate.

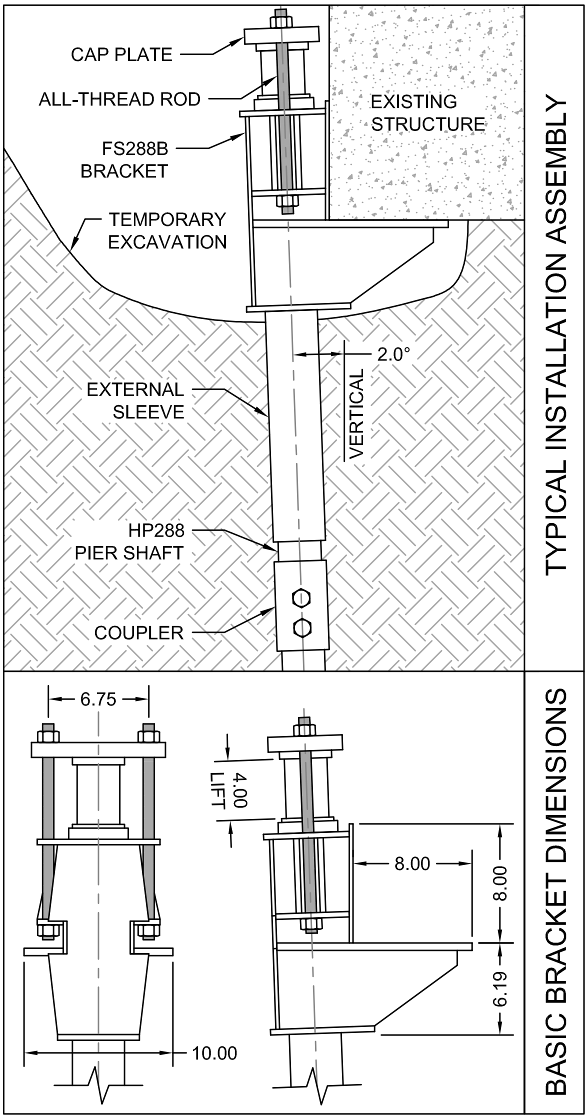
External Sleeve:
Ø3.500" x 0.216" wall x 30" long
with welded collar or trumpet flare at one end.
ASTM A500 Grade B or C
Yield strength = 50 ksi (min)
Tensile strength = 62 ksi (min)

Cap Plate:
1" x 5.00" x 9.00" ASTM A572 Grade 50
with confining ring welded to one side.

Bracket Hardware[3]:
(2) - Ø¾" x 16" long all-thread rod
Grade B7, tensile strength = 125 ksi (min)
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized[2]

Allowable Bracket Capacity[4,5,6,7] R_n/Ω	
	(kips)
Plain	28.8
Plain Corroded [1]	24.9
Galvanized Corroded [1,2]	27.9



(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Optional hardware utilizes similar sized contour (coil) thread made from AISI 1045, tensile strength = 120 ksi. Slightly lower tensile strength material does not govern the listed capacities.

(4) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(5) Allowable capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(6) Concrete bearing assumes a minimum compressive strength (f'c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

(7) Listed allowable capacities are for the specific shaft/bracket combination shown. System capacity should also not exceed the installed torque-correlated soil capacity (See Shaft Specifications and Capacities).

FS288BL Bracket Specifications and Capacities
when used with the HP288 Helical Pile System

Bracket:
Weldment manufactured from ¼", ⅜", and ½"
ASTM A36 plate.

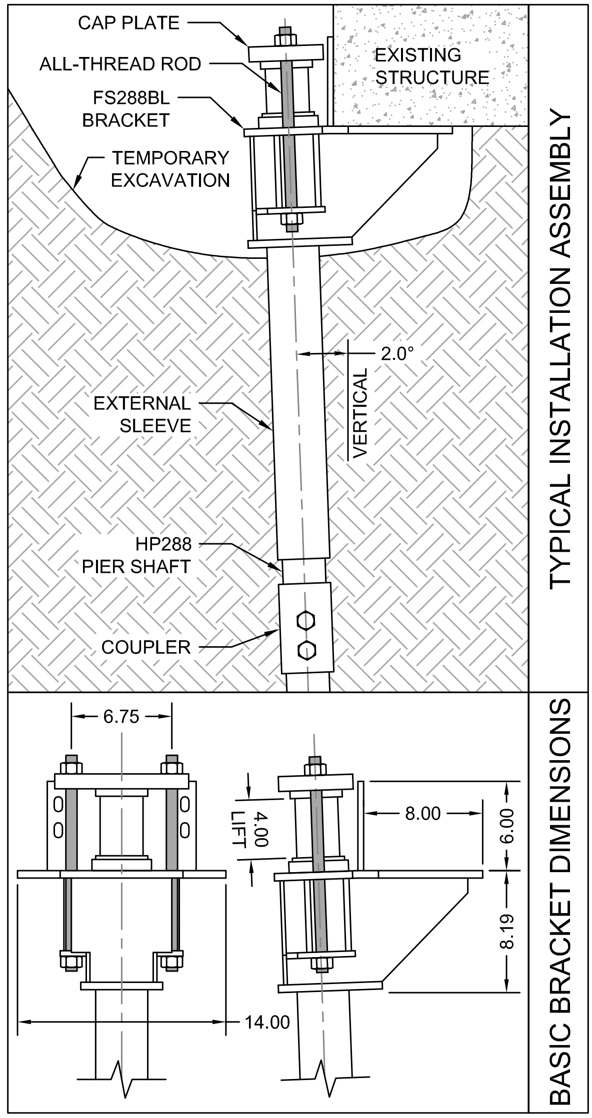
External Sleeve:
Ø3.500" x 0.216" wall x 30" long
with welded collar or trumpet flare at one end.
ASTM A500 Grade B or C
Yield strength = 50 ksi (min)
Tensile strength = 62 ksi (min)

Cap Plate:
1" x 5.00" x 9.00" ASTM A572 Grade 50
with confining ring welded to one side.

Bracket Hardware(3):
(2) - Ø¾" x 16" long all-thread rod
Grade B7, tensile strength = 125 ksi (min)
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized(2)

Allowable Bracket Capacity(4,5,6,7) R_n/Ω	
	(kips)
Plain	29.2
Plain Corroded (1)	25.3
Galvanized Corroded (1,2)	28.2



(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Optional hardware utilizes similar sized contour (coil) thread made from AISI 1045, tensile strength = 120 ksi. Slightly lower tensile strength material does not govern the listed capacities.

(4) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(5) Allowable capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(6) Concrete bearing assumes a minimum compressive strength (f'c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

(7) Listed allowable capacities are for the specific shaft/bracket combination shown. System capacity should also not exceed the installed torque-correlated soil capacity (See Shaft Specifications and Capacities).

HP288B2 Bracket Specifications and Capacities
when used with the HP288 Helical Pile System

Bracket:
Weldment manufactured from ⅜”
ASTM A36 plate.

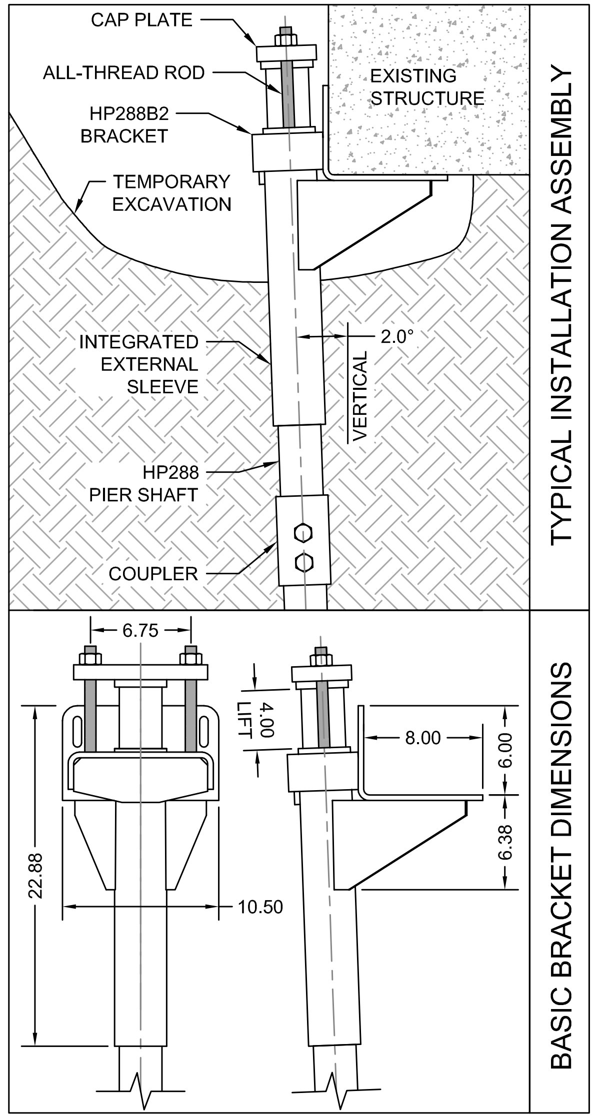
Integrated External Sleeve:
Ø3.500” x 0.216” wall x 20” long
ASTM A500 Grade B or C
Yield strength = 50 ksi (min)
Tensile strength = 62 ksi (min)

Cap Plate:
1” x 4.00” x 9.00” ASTM A572 Grade 50
with confining ring welded to one side.

Bracket Hardware(3):
(2) - Ø¾” x 16” long all-thread rod
Grade B7, tensile strength = 125 ksi (min)
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized(2)

Allowable Bracket Capacity(4,5,6,7) R_n/Ω	
	(kips)
Plain	28.8
Plain Corroded (1)	24.9
Galvanized Corroded (1,2)	27.9



APPENDIX 2A
HELICAL PRODUCT RATINGS, PROPERTIES AND DETAILS

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Optional hardware utilizes similar sized contour (coil) thread made from AISI 1045, tensile strength = 120 ksi. Slightly lower tensile strength material does not govern the listed capacities.

(4) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(5) Allowable capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

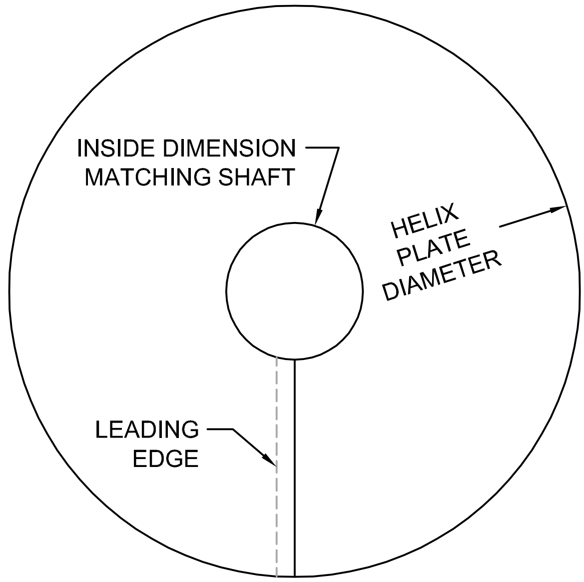
(6) Concrete bearing assumes a minimum compressive strength (f’c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

(7) Listed allowable capacities are for the specific shaft/bracket combination shown. System capacity should also not exceed the installed torque-correlated soil capacity (See Shaft Specifications and Capacities).

HP288 Helix Plate Net Bearing Areas

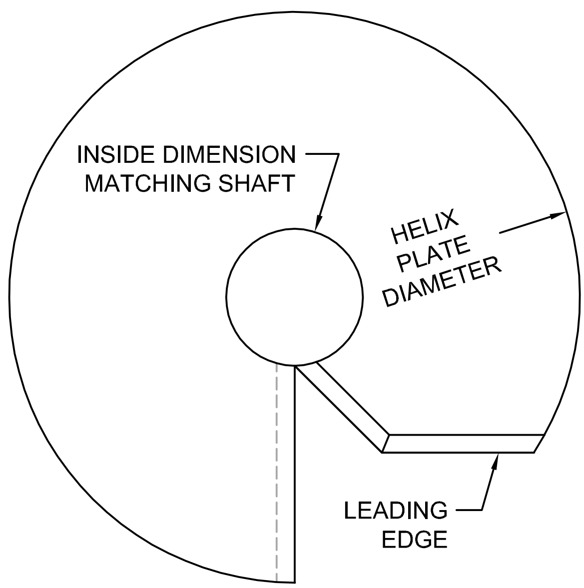
H-Style Plates

Plate Diameter (inches)	Area (ft²)
6	0.15
8	0.30
10	0.50
12	0.74
14	1.02
16	1.35



V-Style Plates(1)

Plate Diameter (inches)	Area (ft²)
6	0.14
8	0.28
10	0.45
12	0.66
14	0.91
16	1.19



(1) V-Style plates feature a special cut on the leading edge (or cutting edge). This edge is cut at two successive 45° angles to roughly simulate a spiral. This is in addition to the 45° bevel on the leading edge which is a standard feature for helix plates of both styles. V-Style plates are appropriate for use in applications where rocky or rubble-filled soils are anticipated, or where very dense layers need to be penetrated.

HP350 Shaft Specifications and Capacities

Shaft Material:
Ø3.500" x 0.340" wall
ASTM A500 Grade B or C
Yield strength = 65 ksi (min)
Tensile strength = 75 ksi (min)

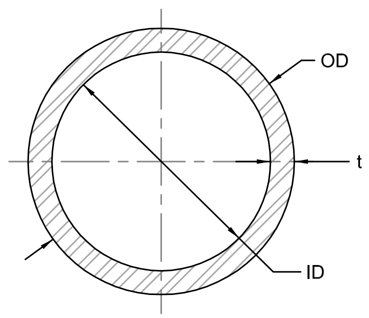
Shaft Coupler Material:
Ø4.250" x 0.344" wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Helix Plates:
ASTM A572 Grade 50 material
⅜" thick (standard)
½" thick (available)
Helix plate geometry conforming to ICC-ES AC358

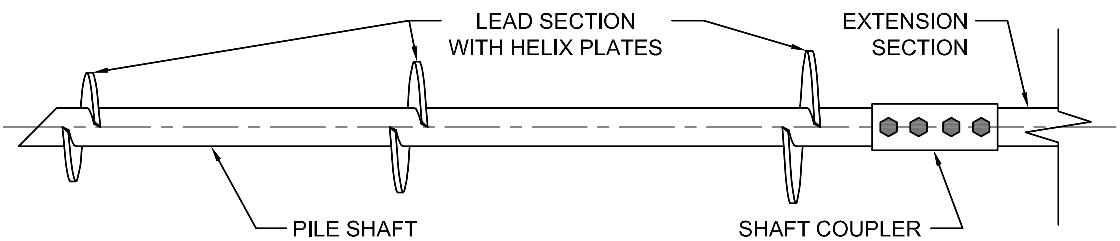
Shaft Coupling Hardware:
(4) - Ø1" Grade 5 bolts with nuts
Electrozinc plated per ASTM B633

Surface Finish of Shaft Segments:
Available plain or hot-dip galvanized(2)

Nominal Thickness	0.340 (in)
Design Thickness(3)	0.316 (in)



	Plain	Plain Corroded(1)	Galvanized Corroded(1,2)
OD (in)	3.500	3.464	3.490
t (in)	0.316	0.28	0.306
ID (in)	2.868	2.904	2.876
A (in^2)	3.16	2.80	3.06
I (in^4)	4.05	3.58	3.91
S (in^3)	2.31	2.07	2.24
Z (in^3)	3.21	2.85	3.11
r (in)	1.13	1.13	1.13
Shaft Max Allowable Compression Capacity(4,5) P_n/Ω (kips)	**118.5**	**105.0**	**114.8**
Shaft Max Allowable Tension Capacity(5) P_n/Ω (kips)	**73.0**	**62.8**	**69.1**



Default Torque Correlation Factor(6) K_t = 7 (ft^{-1})	**Maximum Ultimate Soil Capacity(7) Q_u = 122.5 (kips)**
Maximum Installation Torque T = 17,500 (ft-lb)	**Maximum Allowable Soil Capacity(7) Q_a = 60.0 (kips) FOS = 2.04**

(1) Corroded properties and capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Design thickness for HSS and Pipe based on 93% of nominal thickness per AISC.

(4) Allowable mechanical compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(5) Listed mechanical capacities are for the shaft and coupled connections only. System capacity should also not exceed the installed allowable torque-correlated soil capacity or the allowable capacity of the respective bracket (see additional bracket tables).

(6) Listed default K_t factor is consistent with that listed in ICC-ES AC358. This value is generally conservative. Site-specific K_t factors can be determined for a given project with full-scale load testing.

(7) Soil capacities listed are at maximum installation torque. Ultimate soil capacity is based on the equation $Q_u = K_t \times T$. Allowable soil capacity is obtained by dividing the ultimate capacity by an appropriate factor of safety, but should not exceed 60 kips per AC358. Although a factor of safety of 2.0 is commonly used, a higher or lower factor of safety may be considered at the discretion of the helical pile designer or as dictated by local code requirements. System capacity should also not exceed the mechanical capacity of the shaft or those listed in the respective bracket capacity tables.

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HP350NCB Bracket Specifications and Capacities
when used with the HP350 Helical Pile System

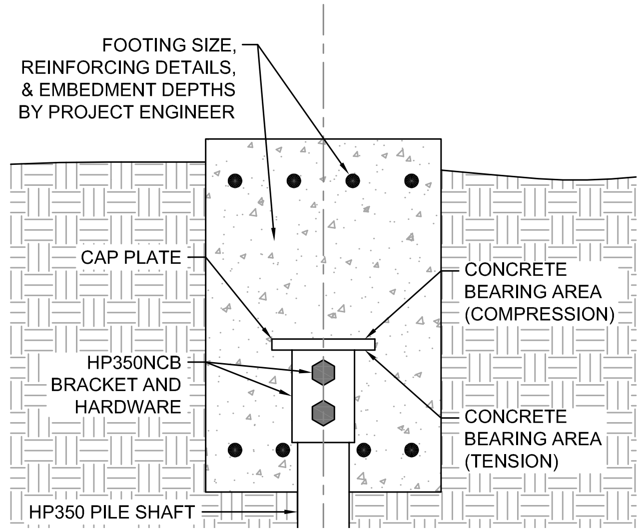
Bracket Sleeve Material:
Ø4.250" x 0.313" wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Cap Plate Material:
¾" x 7.00" square
ASTM A36

Bracket Hardware:
(2) - Ø1" Grade 5 bolts with nuts
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized(2)

Concrete Bearing Area(6) (Compression) = 49.0 in²
Concrete Bearing Area(6) (Tension) = 34.8 in²



		Allowable Bracket Capacity(4) R_n/Ω			
		Compression(3) (kips)	Concrete Bearing(6) (ksi)	Tension (kips)	Concrete Bearing(6) (ksi)
2 Bolts	Plain	67.6	1.38	64.2	1.85
	Plain Corroded(1)	60.8	1.25	58.6	1.71
	Galvanized Corroded(1,2)	65.7	1.34	62.7	1.81
1 Bolt	Plain	67.6	1.38	26.7	0.77
	Plain Corroded(1)	60.8	1.25	22.7	0.66
	Galvanized Corroded(1,2)	65.7	1.34	25.6	0.74
0 Bolts(5)	Plain	67.6	1.38	0	0
	Plain Corroded(1)	60.8	1.25	0	0
	Galvanized Corroded(1,2)	65.7	1.34	0	0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

(5) Applications utilizing no bolts should either be tack welded or utilize some other mechanism to immobilize the bracket and maintain firm contact between the cap plate and pile shaft throughout construction and concrete placement.

(6) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f'c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

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HP350NCB8 Bracket Specifications and Capacities

when used with the HP350 Helical Pile System

Bracket Sleeve Material:

Ø4.250” x 0.313” wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Cap Plate Material:

¾” x 8.00” square
ASTM A36

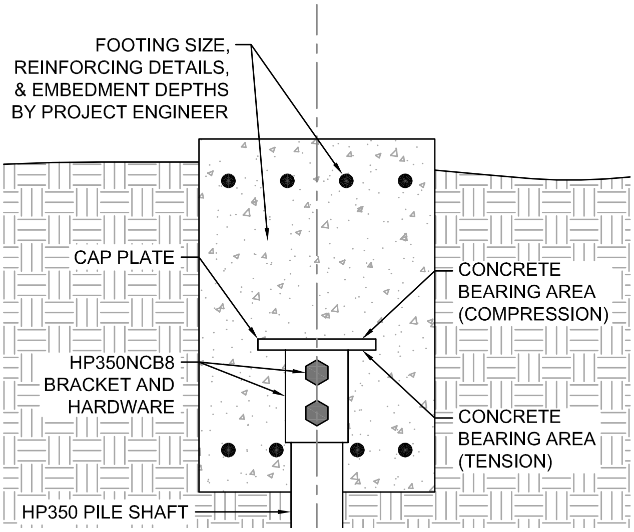
Bracket Hardware:

(2) - Ø1” Grade 5 bolts with nuts
Electrozinc plated per ASTM B633

Bracket Finish:

Available plain or hot-dip galvanized(2)

Concrete Bearing Area(6) (Compression) = 64.0 in^2
Concrete Bearing Area(6) (Tension) = 49.8 in^2



		Allowable Bracket Capacity(4) R_n/Ω			
		Compression(3) (kips)	Concrete Bearing(6) (ksi)	Tension (kips)	Concrete Bearing(6) (ksi)
2 Bolts	Plain	57.6	0.90	57.3	1.15
	Plain Corroded(1)	51.9	0.82	51.5	1.05
	Galvanized Corroded(1,2)	56.0	0.88	55.6	1.12
1 Bolt	Plain	57.6	0.90	26.7	0.54
	Plain Corroded(1)	51.9	0.82	22.7	0.46
	Galvanized Corroded(1,2)	56.0	0.88	25.6	0.51
0 Bolts(5)	Plain	57.6	0.90	0	0
	Plain Corroded(1)	51.9	0.82	0	0
	Galvanized Corroded(1,2)	56.0	0.88	0	0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

(5) Applications utilizing no bolts should either be tack welded or utilize some other mechanism to immobilize the bracket and maintain firm contact between the cap plate and pile shaft throughout construction and concrete placement.

(6) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f’c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

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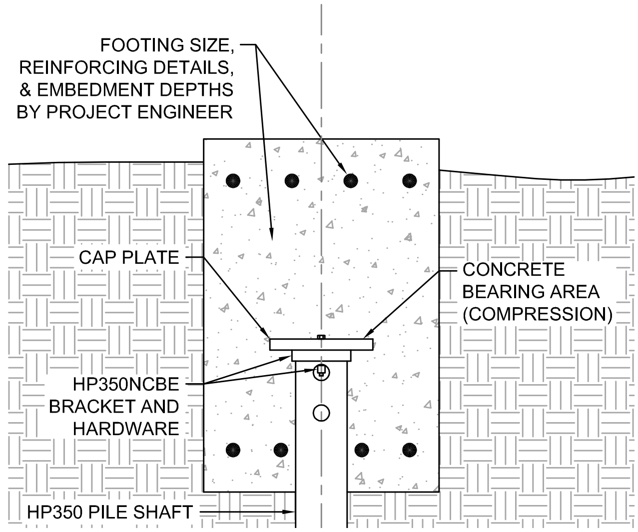
HP350NCBE Bracket Specifications and Capacities
when used with the HP350 Helical Pile System

Cap Plate Material:
¾" x 7.00" square
ASTM A36

Bracket Hardware(5):
½" square bar stock with tapped hole and Ø¼" retention bolt

Bracket Finish:
Available plain or hot-dip galvanized(2)

Concrete Bearing Area(6) (Compression) = 49.0 in^2
Concrete Bearing Area(6) (Tension) = N/A



	Allowable Bracket Capacity(4) R_n/Ω			
	Compression(3) (kips)	Concrete Bearing(6) (ksi)	Tension (kips)	Concrete Bearing(6) (ksi)
Plain	67.6	1.38	0	0
Plain Corroded(1)	60.8	1.25	0	0
Galvanized Corroded(1,2)	65.7	1.34	0	0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

(5) Supplied bracket hardware does not contribute to the system strength and is only intended to immobilize the bracket. Other methods, such as tack welds, may be substituted to maintain firm contact between the cap plate and pile shaft throughout construction and concrete placement at the discretion of the project engineer.

(6) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f'c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

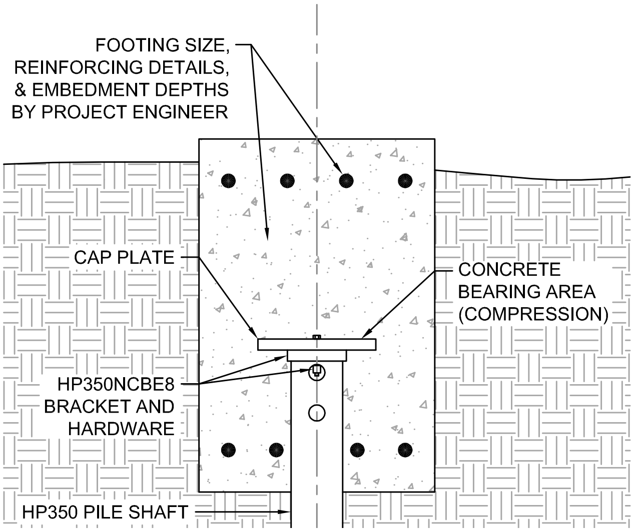
HP350NCBE8 Bracket Specifications and Capacities
when used with the HP350 Helical Pile System

Cap Plate Material:
¾” x 8.00” square
ASTM A36

Bracket Hardware[5]:
½” square bar stock with tapped hole and Ø¼” retention bolt

Bracket Finish:
Available plain or hot-dip galvanized[2]

Concrete Bearing Area[6] (Compression) = 64.0 in^2
Concrete Bearing Area[6] (Tension) = N/A



	Allowable Bracket Capacity[4] R_n/Ω			
	Compression[3] (kips)	Concrete Bearing[6] (ksi)	Tension (kips)	Concrete Bearing[6] (ksi)
Plain	57.6	0.90	0	0
Plain Corroded[1]	51.9	0.82	0	0
Galvanized Corroded[1,2]	56.0	0.88	0	0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

(5) Supplied bracket hardware does not contribute to the system strength and is only intended to immobilize the bracket. Other methods, such as tack welds, may be substituted to maintain firm contact between the cap plate and pile shaft throughout construction and concrete placement at the discretion of the project engineer.

(6) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f’c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

HP350BS Bracket Specifications and Capacities
when used with the HP350 Helical Pile System

Bracket:
Weldment manufactured from ⅜" and ½" ASTM A36 plate.

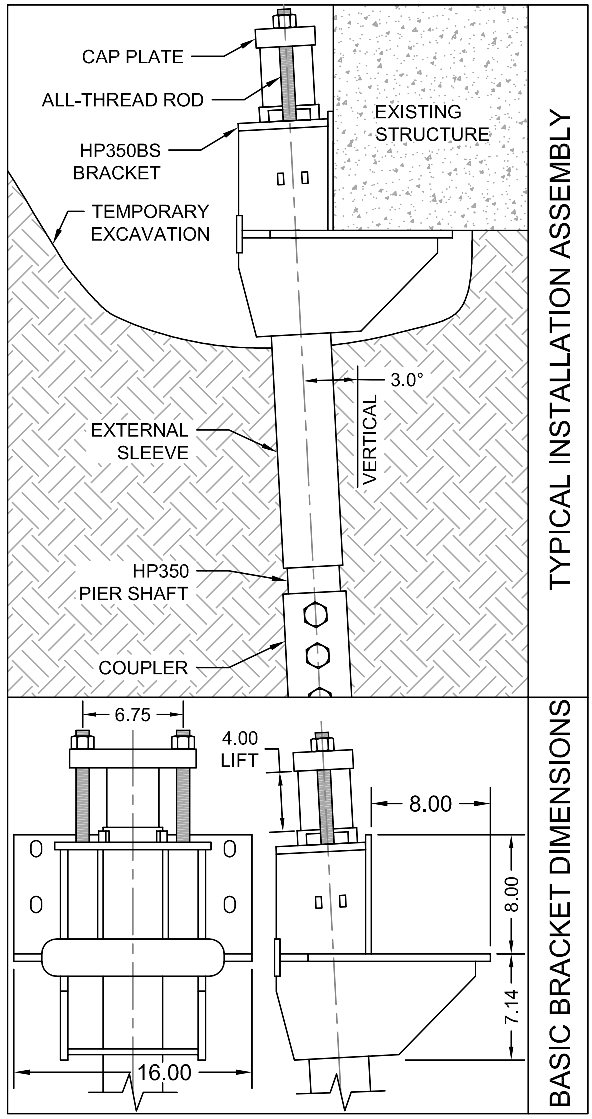
External Sleeve:
Ø4.000" x 0.226" wall x 30" long
with welded collar at one end.
ASTM A500 Grade B or C
Yield strength = 50 ksi (min)
Tensile strength = 62 ksi (min)

Cap Plate:
1-¼" x 4.00" x 8.50" ASTM A572 Grade 50
with capture plate welded to one side.

Bracket Hardware:
(2) - Ø⅞" x 18" long all-thread rod
Grade B7, tensile strength = 125 ksi (min)
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized(2)

Allowable Bracket Capacity(3,4,5,6) R_n/Ω	
	(kips)
Plain	50.7
Plain Corroded (1)	45.4
Galvanized Corroded (1,2)	49.2



(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(4) Allowable capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(5) Concrete bearing assumes a minimum compressive strength (f'c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

(6) Listed allowable capacities are for the specific shaft/bracket combination shown. System capacity should also not exceed the installed torque-correlated soil capacity (See Shaft Specifications and Capacities).

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HP350B Bracket Specifications and Capacities

when used with the HP350 Helical Pile System

Bracket:

Weldment manufactured from ⅜" and ½" ASTM A36 plate.

Concrete Anchorage(7) (Optional):

(6) - Ø⅝" x 9" long all-thread rod
Grade B7, tensile strength = 125 ksi (min)
Electrozinc plated per ASTM B633

Cap Plate:

1-¼" x 4.00" x 8.50" ASTM A572 Grade 50 with capture plate welded to one side.

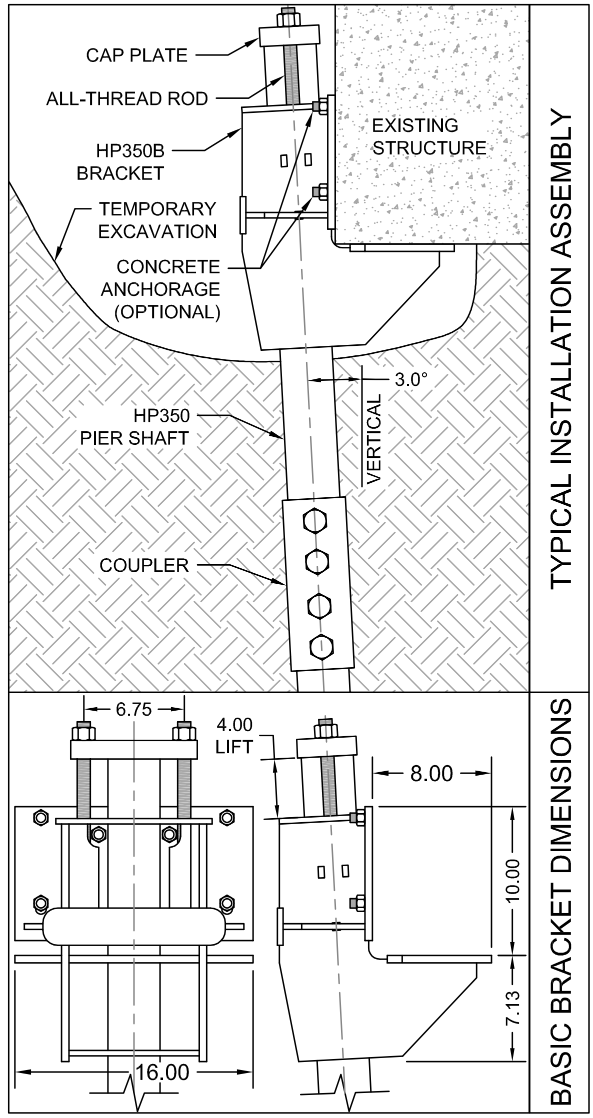
Bracket Hardware:

(2) - Ø⅞" x 18" long all-thread rod
Grade B7, tensile strength = 125 ksi (min)
Electrozinc plated per ASTM B633

Bracket Finish:

Available plain or hot-dip galvanized(2)

Allowable Bracket Capacity(3,4,5,6) R_n/Ω	NO Adhesive Anchors (kips)	WITH Adhesive Anchors(7) (kips)
Plain	36.5	45.0
Plain Corroded (1)	32.6	40.2
Galvanized Corroded (1,2)	35.4	43.7



(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(4) Allowable capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(5) Concrete bearing assumes a minimum compressive strength (f'c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

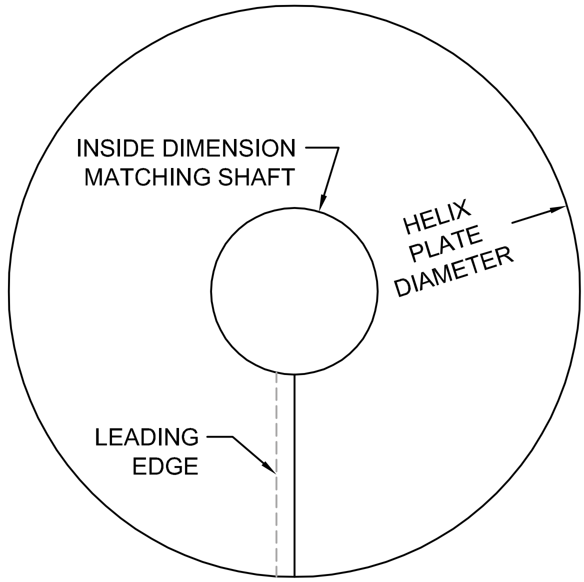
(6) Listed allowable capacities are for the specific shaft/bracket combination shown. System capacity should also not exceed the installed torque-correlated soil capacity (See Shaft Specifications and Capacities).

(7) Specified anchors installed to a minimum embedment of 7.5" into concrete with a minimum compressive strength f'c = 2,500 psi utilizing Simpson AT adhesive.

HP350 Helix Plate Net Bearing Areas

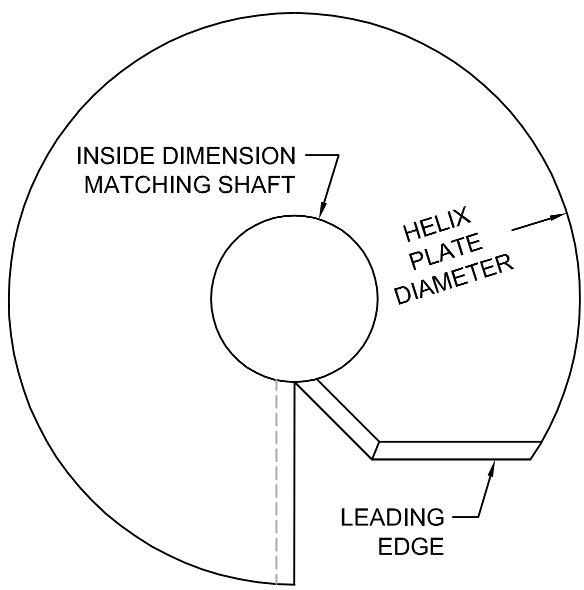
H-Style Plates

Plate Diameter (inches)	Area (ft^2)
6	0.13
8	0.28
10	0.48
12	0.72
14	1.00
16	1.33



V-Style Plates(1)

Plate Diameter (inches)	Area (ft^2)
6*	0.12
8	0.26
10	0.43
12	0.65
14	0.89
16	1.18



(1) V-Style plates feature a special cut on the leading edge (or cutting edge). This edge is cut at two successive 45° angles to roughly simulate a spiral. This is in addition to the 45° bevel on the leading edge which is a standard feature for helix plates of both styles. V-Style plates are appropriate for use in applications where rocky or rubble-filled soils are anticipated, or where very dense layers need to be penetrated. Some smaller plate diameters indicated by an asterisk (*), are not typically available in a V-Style.

HP450 Shaft Specifications and Capacities

Shaft Material:
Ø4.500" x 0.337" wall
ASTM A500 Grade B or C
Yield strength = 50 ksi (min)
Tensile strength = 60 ksi (min)

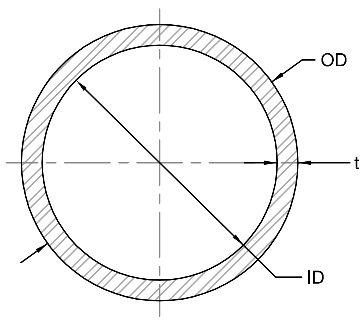
Shaft Coupler Material:
Ø3.750" x 0.500" wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Helix Plates:
ASTM A572 Grade 50 material
⅜" thick (standard)
½" thick (available)
Helix plate geometry conforming to ICC-ES AC358

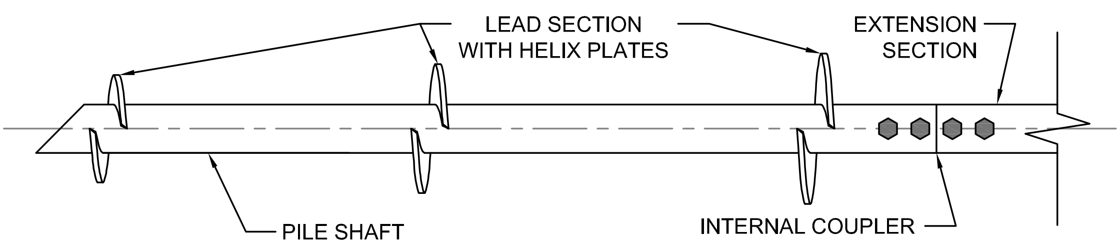
Shaft Coupling Hardware:
(4) - Ø1-⅛" Grade 5 bolts with nuts
Electrozinc plated per ASTM B633

Surface Finish of Shaft Segments:
Available plain or hot-dip galvanized(2)

Nominal Thickness	0.337 (in)
Design Thickness(3)	0.313 (in)



	Plain	Plain Corroded(1)	Galvanized Corroded(1,2)
OD (in)	4.500	4.464	4.490
t (in)	0.313	0.277	0.303
ID (in)	3.874	3.910	3.884
A (in^2)	4.12	3.64	3.99
I (in^4)	9.07	8.02	8.78
S (in^3)	4.03	3.59	3.91
Z (in^3)	5.50	4.86	5.32
r (in)	1.48	1.48	1.48
Shaft Max Allowable Compression Capacity(4,5) P_n/Ω (kips)	**123.3**	**109.1**	**119.3**
Shaft Max Allowable Tension Capacity(5) P_n/Ω (kips)	**59.1**	**50.8**	**56.7**



Torque Correlation Factor(6) K_t = 6 (ft^{-1})	**Maximum Ultimate Soil Capacity(7) Q_u = 132.0 (kips)**
Maximum Installation Torque T = 22,000 (ft-lb)	**Maximum Allowable Soil Capacity(7) Q_a = 66.0 (kips) FOS = 2.0**

(1) Corroded properties and capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Design thickness for HSS and Pipe based on 93% of nominal thickness per AISC.

(4) Allowable mechanical compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(5) Listed mechanical capacities are for the shaft and coupled connections only. System capacity should also not exceed the installed allowable torque-correlated soil capacity or the allowable capacity of the respective bracket (see additional bracket tables).

(6) Listed K_t factor is that recommended by FSI. Site-specific K_t factors can be determined for a given project with full-scale load testing.

(7) Soil capacities listed are at maximum installation torque. Ultimate soil capacity is based on the equation $Q_u = K_t \times T$. Allowable soil capacity is obtained by dividing the ultimate value by the appropriate factor of safety ($Q_a = Q_u$ / FOS). FOS is most commonly taken as 2.0, although a higher or lower FOS may be considered at the discretion of the helical pile designer or as dictated by local code or project requirements. System capacity should also not exceed the mechanical capacity of the shaft or those listed in the respective bracket capacity tables.

HP450NCB8 Bracket Specifications and Capacities
when used with the HP450 Helical Pile System

APPENDIX 2A
HELICAL PRODUCT RATINGS, PROPERTIES AND DETAILS

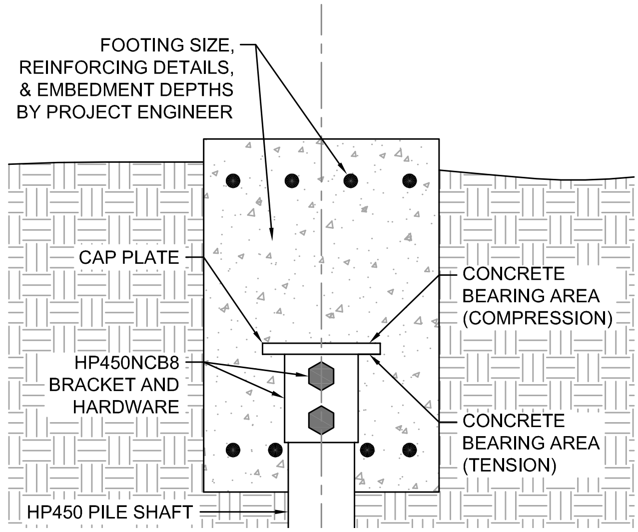
Bracket Sleeve Material:
Ø5.000" x 0.188" wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Cap Plate Material:
¾" x 8.00" square
ASTM A36

Bracket Hardware:
(2) - Ø1-⅛" Grade 5 bolts with nuts
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized(2)

Concrete Bearing Area(6) (Compression) = 64.0 in²
Concrete Bearing Area(6) (Tension) = 44.4 in²



		Allowable Bracket Capacity(4) R_n/Ω			
		Compression(3) (kips)	Concrete Bearing(6) (ksi)	Tension (kips)	Concrete Bearing(6) (ksi)
2 Bolts	Plain	80.4	1.26	47.0	1.06
	Plain Corroded(1)	72.5	1.14	37.0	0.84
	Galvanized Corroded(1,2)	78.2	1.22	44.2	1.00
1 Bolt	Plain	80.4	1.26	16.0	0.36
	Plain Corroded(1)	72.5	1.14	12.4	0.28
	Galvanized Corroded(1,2)	78.2	1.22	15.0	0.34
0 Bolts(5)	Plain	80.4	1.26	0	0
	Plain Corroded(1)	72.5	1.14	0	0
	Galvanized Corroded(1,2)	78.2	1.22	0	0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

(5) Applications utilizing no bolts should either be tack welded or utilize some other mechanism to immobilize the bracket and maintain firm contact between the cap plate and pile shaft throughout construction and concrete placement.

(6) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f'c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

Rev. 8/21/14

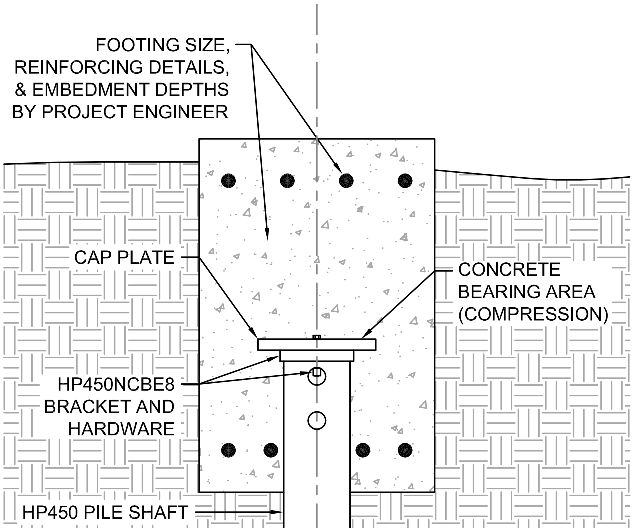
HP450NCBE8 Bracket Specifications and Capacities
when used with the HP450 Helical Pile System

Cap Plate Material:
¾" x 8.00" square
ASTM A36

Bracket Hardware[5]:
½" square bar stock with tapped hole and Ø¼" retention bolt

Bracket Finish:
Available plain or hot-dip galvanized[2]

Concrete Bearing Area[6] (Compression) = 64.0 in^2
Concrete Bearing Area[6] (Tension) = N/A



	Allowable Bracket Capacity[4] R_n/Ω			
	Compression[3] (kips)	Concrete Bearing[6] (ksi)	Tension (kips)	Concrete Bearing[6] (ksi)
Plain	80.4	1.26	0	0
Plain Corroded[1]	72.5	1.14	0	0
Galvanized Corroded[1,2]	78.2	1.22	0	0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(4) Listed capacities include limiting mechanical capacities of the shaft when the shaft and bracket are combined as a system. System capacity should also not exceed the installed allowable torque-correlated soil capacity (See Shaft Specifications and Capacities).

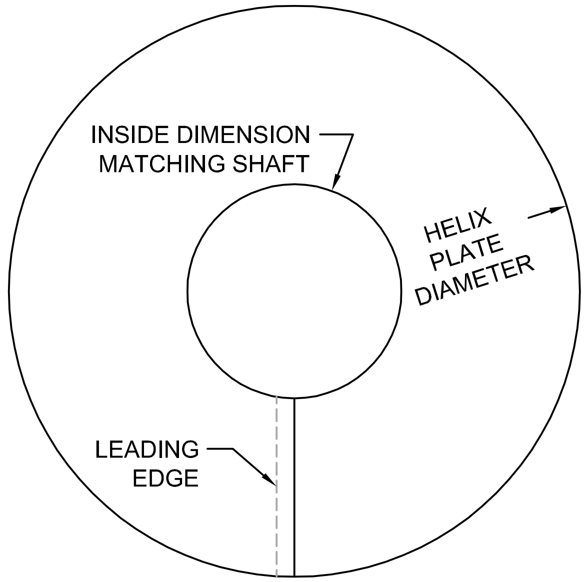
(5) Supplied bracket hardware does not contribute to the system strength and is only intended to immobilize the bracket. Other methods, such as tack welds, may be substituted to maintain firm contact between the cap plate and pile shaft throughout construction and concrete placement at the discretion of the project engineer.

(6) Concrete bearing values provided are the uniform bearing stresses required to achieve the full corresponding bracket capacity. Allowable concrete bearing is a function of several project specific variables including depth of embedment, edge distance, and concrete compressive strength (f'c). When allowable concrete bearing stresses are lower than these values, corresponding bracket capacities can be obtained by multiplying the actual allowable concrete bearing stress by the respective bearing areas provided, but should not exceed the capacities listed in this table. Other concrete design checks including shear, bending, and punching of the supported structure are also project specific and shall be the responsibility of the project engineer.

HP450 Helix Plate Net Bearing Areas

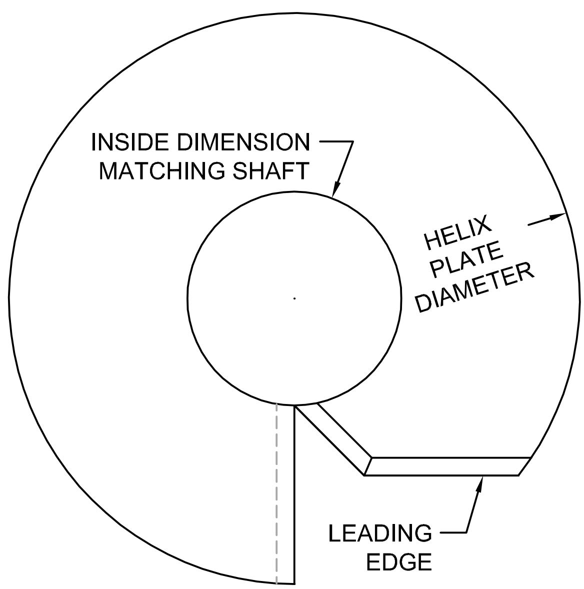
H-Style Plates

Plate Diameter (inches)	Area (ft^2)
8	0.24
10	0.43
12	0.67
14	0.96
16	1.29



V-Style Plates(1)

Plate Diameter (inches)	Area (ft^2)
8*	0.22
10	0.40
12	0.62
14	0.87
16	1.15



(1) V-Style plates feature a special cut on the leading edge (or cutting edge). This edge is cut at two successive 45° angles to roughly simulate a spiral. This is in addition to the 45° bevel on the leading edge which is a standard feature for helix plates of both styles. V-Style plates are appropriate for use in applications where rocky or rubble-filled soils are anticipated, or where very dense layers need to be penetrated. Some smaller plate diameters indicated by an asterisk (*), are not typically available in a V-Style.

HP662 Shaft Specifications and Capacities

Shaft Material[(6)]:
Ø6.625” x 0.280” wall
ASTM A500 Grade B or C
Yield strength = 60 ksi (min)
Tensile strength = 70 ksi (min)

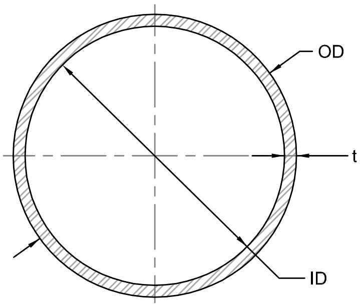
Shaft Coupler Material[(6)]:
Ø6.000” x 0.375” wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Helix Plates[(6)]:
ASTM A572 Grade 50 material
½” thick (standard)
Helix plate geometry conforming to
ICC-ES AC358

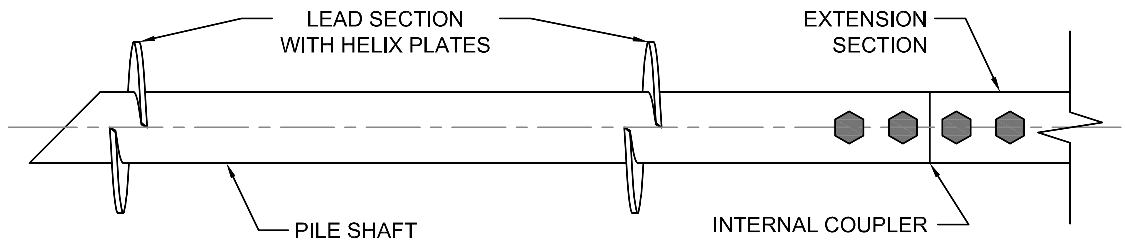
Shaft Coupling Hardware[(6)]:
(4) - Ø1-¾” ASTM A307 bolts with nuts
Electrozinc plated per ASTM B633

Surface Finish of Shaft Segments:
Available plain or hot-dip galvanized[(2)]

Nominal Thickness	0.280 (in)
Design Thickness[(3)]	0.261 (in)



	Plain	Plain Corroded[(1)]	Galvanized Corroded[(1,2)]
OD (in)	6.625	6.589	6.615
t (in)	0.261	0.225	0.251
ID (in)	6.103	6.139	6.113
A (in^2)	5.22	4.50	5.02
I (in^4)	26.46	22.80	25.44
S (in^3)	7.99	6.92	7.69
Z (in^3)	10.58	9.12	10.17
r (in)	2.25	2.25	2.25
Shaft Max Allowable Compression Capacity[(4,5)] P_n/Ω (kips)	**182.6**	**157.4**	**175.6**
Shaft Max Allowable Tension Capacity[(5)] P_n/Ω (kips)	**98.7**	**83.3**	**93.0**



Torque Correlation Factor[(6)] K_t = N/A (ft^{-1})	**Maximum Ultimate Soil Capacity[(6)] Q_u = N/A (kips)**
Maximum Installation Torque[(6)] T = 35,000 (ft-lb)	**Maximum Allowable Soil Capacity[(6)] Q_a = N/A (kips)**

(1) Corroded properties and capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Design thickness for HSS and Pipe based on 93% of nominal thickness per AISC.

(4) Allowable mechanical compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(5) Listed mechanical capacities are for the shaft and coupled connections only. System capacity should also not exceed the installed allowable soil capacity or the allowable capacity of the respective bracket (see additional bracket tables).

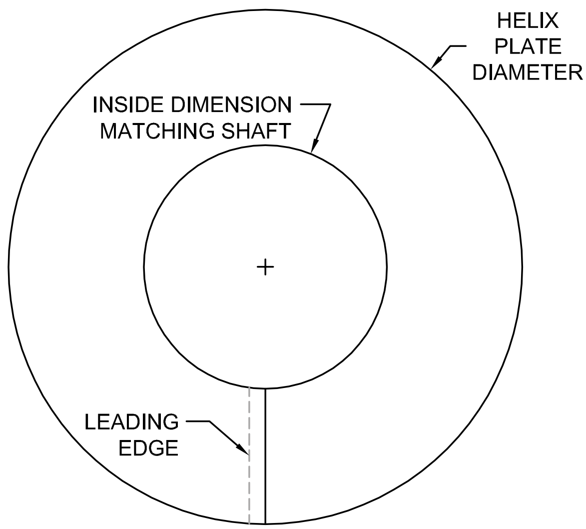
(6) FSI’s larger diameter product lines are fully customized on a project specific basis. All values provided for these products are for general informational purposes only. Actual capacities (including any related to installation torque) will vary based on several project specific variables such as coupler details, end termination details, site specific soil profiles, and even material availability. Full scale load tests are recommended to confirm soil capacities determined in the design phase of the project.

Rev. 5/26/15

HP662 Helix Plate Net Bearing Areas

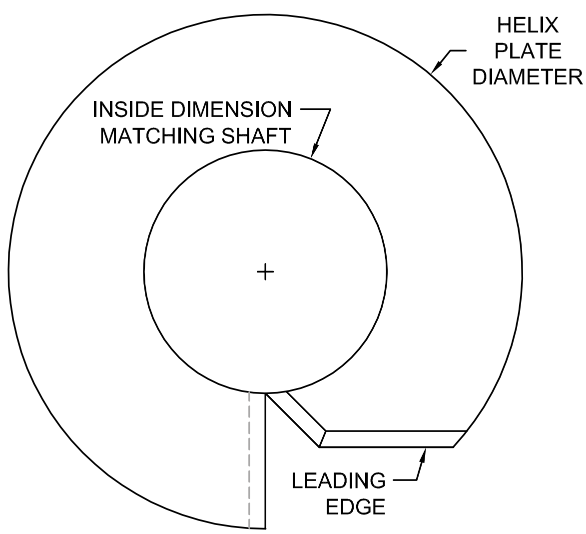
H-Style Plates

Plate Diameter (inches)	Area (ft^2)
10	0.31
12	0.55
14	0.83
16	1.16



V-Style Plates(1)

Plate Diameter (inches)	Area (ft^2)
10*	0.29
12	0.51
14	0.77
16	1.06



(1) V-Style plates feature a special cut on the leading edge (or cutting edge). This edge is cut at two successive 45° angles to roughly simulate a spiral. This is in addition to the 45° bevel on the leading edge which is a standard feature for helix plates of both styles. V-Style plates are appropriate for use in applications where rocky or rubble-filled soils are anticipated, or where very dense layers need to be penetrated. Some smaller plate diameters indicated by an asterisk (*), are not typically available in a V-Style.

HP700 Shaft Specifications and Capacities

Shaft Material[6]:
Ø7.000" x 0.362" wall
ASTM A252 Grade 3
Yield strength = 60 ksi (min)
Tensile strength = 70 ksi (min)

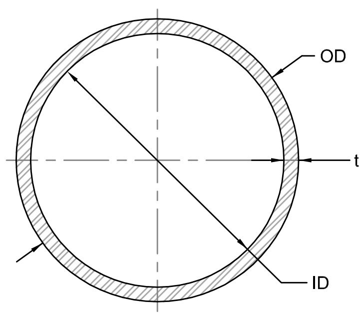
Shaft Coupler Material[6]:
Ø7.750" x 0.313" wall
ASTM A513 Type 5 Grade 1026
Yield strength = 70 ksi (min)
Tensile strength = 80 ksi (min)

Helix Plates[6]:
ASTM A572 Grade 50 material
½" thick (standard)
Helix plate geometry conforming to ICC-ES AC358

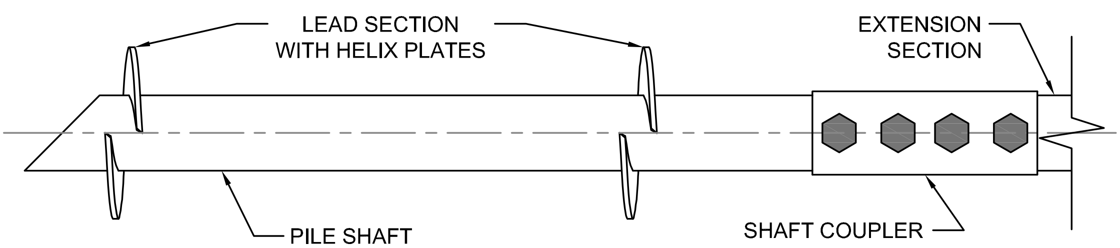
Shaft Coupling Hardware[6]:
(4) - Ø2" ASTM A307 bolts with nuts
Electrozinc plated per ASTM B633

Surface Finish of Shaft Segments:
Available plain or hot-dip galvanized[2]

Nominal Thickness	0.362 (in)
Design Thickness[3]	0.337 (in)



	Plain	Plain Corroded[1]	Galvanized Corroded[1,2]
OD (in)	7.000	6.964	6.990
t (in)	0.337	0.301	0.327
ID (in)	6.326	6.362	6.336
A (in^2)	7.05	6.30	6.84
I (in^4)	39.25	35.04	38.08
S (in^3)	11.21	10.06	10.89
Z (in^3)	14.97	13.37	14.53
r (in)	2.36	2.36	2.36
Shaft Max Allowable Compression Capacity[4,5] P_n/Ω (kips)	**246.9**	**220.5**	**239.6**
Shaft Max Allowable Tension Capacity[5] P_n/Ω (kips)	**135.0**	**118.8**	**130.5**



Torque Correlation Factor[6] K_t = N/A (ft^{-1})	**Maximum Ultimate Soil Capacity[6] Q_u = N/A (kips)**
Maximum Installation Torque[6] T = 50,000 (ft-lb)	**Maximum Allowable Soil Capacity[6] Q_a = N/A (kips)**

(1) Corroded properties and capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC358.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Design thickness for HSS and Pipe based on 93% of nominal thickness per AISC.

(4) Allowable mechanical compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piles with exposed unbraced lengths or piles placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(5) Listed mechanical capacities are for the shaft and coupled connections only. System capacity should also not exceed the installed allowable soil capacity or the allowable capacity of the respective bracket (see additional bracket tables).

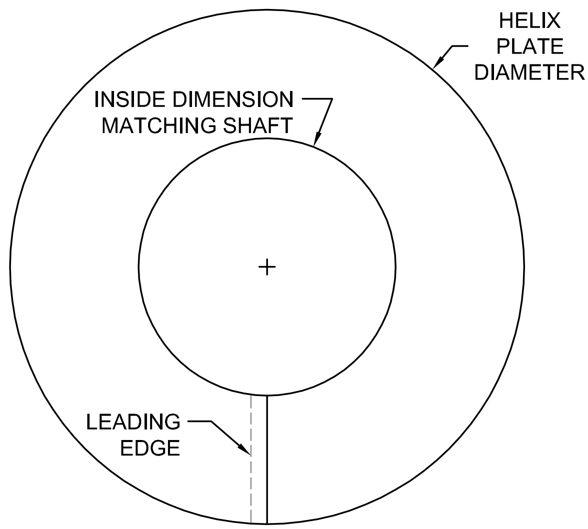
(6) FSI's larger diameter product lines are fully customized on a project specific basis. All values provided for these products are for general informational purposes only. Actual capacities (including any related to installation torque) will vary based on several project specific variables such as coupler details, end termination details, site specific soil profiles, and even material availability. Full scale load tests are recommended to confirm soil capacities determined in the design phase of the project.

Rev. 8/21/14

HP700 Helix Plate Net Bearing Areas

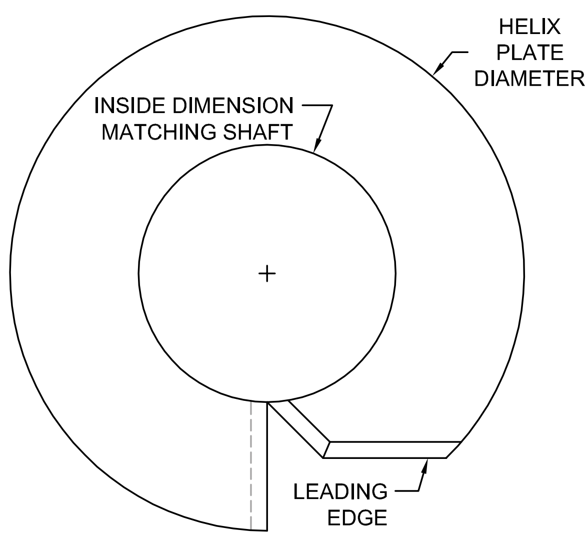
H-Style Plates

Plate Diameter (inches)	Area (ft^2)
10	0.28
12	0.52
14	0.80
16	1.13



V-Style Plates(1)

Plate Diameter (inches)	Area (ft^2)
10*	0.27
12	0.49
14	0.75
16	1.04



(1) V-Style plates feature a special cut on the leading edge (or cutting edge). This edge is cut at two successive 45° angles to roughly simulate a spiral. This is in addition to the 45° bevel on the leading edge which is a standard feature for helix plates of both styles. V-Style plates are appropriate for use in applications where rocky or rubble-filled soils are anticipated, or where very dense layers need to be penetrated. Some smaller plate diameters indicated by an asterisk (*), are not typically available in a V-Style.

Helix Plate Capacities

The capacity of an individual helix plate is determined through laboratory testing in accordance with Section 4.3 of ICC-ES AC358. This test is completed by placing a short section of shaft with a single helix plate in a laboratory load frame or universal machine. The helix plate bears on a helix-shaped fixture or on an adjustable mandrill with five or more pins. The line of bearing varies and is pre-determined for each helix plate and pile shaft combination. Load is applied to be coaxial with the longitudinal axis of the pile shaft and normal to the bearing surface of the helix plate.

Foundation Supportworks® completed helix capacity testing for several pile shaft/helix plate configurations in accordance with AC358 *(See Figure 2A.1)*. HP288 and HP350 shafts were tested with 8, 10, 12 and 14-inch diameter helix plates. The helix plates were ⅜-inch thick Grade 50 (50 ksi min. yield) steel. Load was applied until (1) the test sample refused any additional load by reaching a failure mechanism such as weld shear, plate bending, or shaft buckling, or (2) a practical, usable resistance was exceeded. Test results (ultimate helix plate capacities) ranged from 101 kips to 200 kips, with most tests being terminated at loads of 140 kips (HP288) or 200 kips (HP350) since these values greatly exceed a practical usable pile capacity. Considering even the lowest atypical test result, the allowable individual helix plate capacity for both shafts and all plate diameters would exceed 41 kips with a factor of safety of 2.0 and a scaling factor to normalize for 50 years of plain steel corrosion.

An allowable individual helix plate capacity of 41 kips will rarely be approached in practice since most heavily-loaded applications will include pile designs with multiple helix plates. Exceptions could be installations in hard clay, dense granular soils or bedrock, where ½-inch thick helix plates would then likely be considered to increase individual plate capacities and minimize plate deflections under load. In most soil conditions, the torque-correlated allowable soil capacity will limit the working load distributed to each helix plate to much less than 41 kips. Even in light load applications, multi-helix pile configurations are commonly utilized to increase bearing area, lower contact pressures within the soil, and minimize pile deflections due to soil deformation. Multi-helix pile configurations are also typically easier to install due to the downward thrust provided by the additional helix plates. For additional discussion about helix plates see Section 2.3.1 of this manual.

Figure 2A.1 Helix capacity testing of HP288 with 14-inch diameter helix plate

Appendix 2B

Lift Assembly Specifications

Model 238 Lift Assembly Specifications

Compatible Brackets[3]:

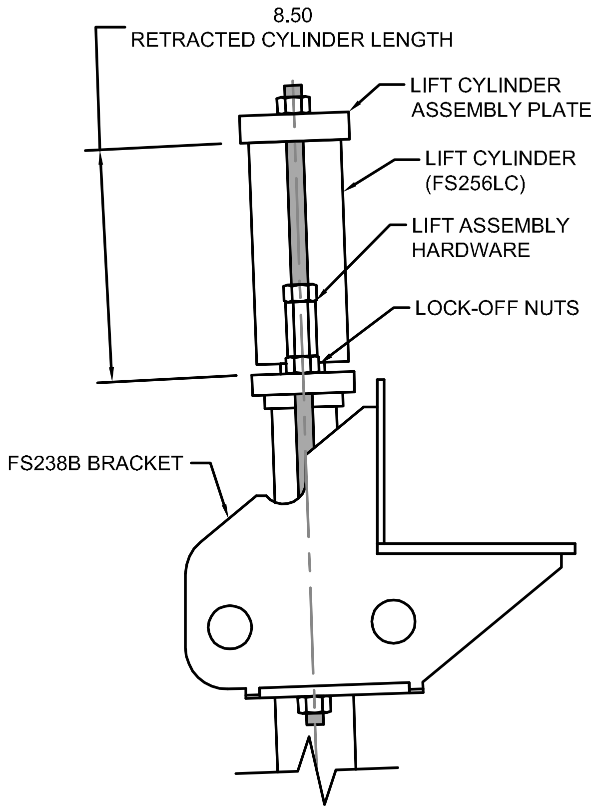
FS238B

Lift Cylinder (FS256LC):

Stroke = 4”
Cylinder action = single
Bore = Ø2.56”
Hydraulic area = 5.15 in^2
Max operating pressure[2] = 8,000 psi

Lift Assembly Hardware[1]:

(2) - Ø⅝” x 16” long coil rod
with nuts and hex couplers, or
(2) - Ø⅝” x 14” long all-thread rod
with nuts and hex couplers



Lift Assembly Rated Lifting Load[2,3] 27.6 kips			
Hydraulic Pressure (psi)	Lift Force[2,3] (kips)	Hydraulic Pressure (psi)	Lift Force[2,3] (kips)
400	2.1	3,200	16.5
800	4.1	3,400	17.5
1,200	6.2	3,600	18.6
1,400	7.2	3,800	19.6
1,600	8.3	4,000	20.6
1,800	9.3	4,200	21.7
2,000	10.3	4,400	22.7
2,200	11.3	4,600	23.7
2,400	12.4	4,800	24.8
2,600	13.4	5,000	25.8
2,800	14.4	5,200	26.8
3,000	15.5	5,350	27.6

(1) Hardware used in the lift assembly must be selected to match the hardware used with the installed bracket assembly.

(2) Do not operate at pressures that produce lift forces in excess of the lift assembly’s rated lifting load. Max operating pressure of the lift cylinder produces forces that exceed this value and is given for informational purposes only.

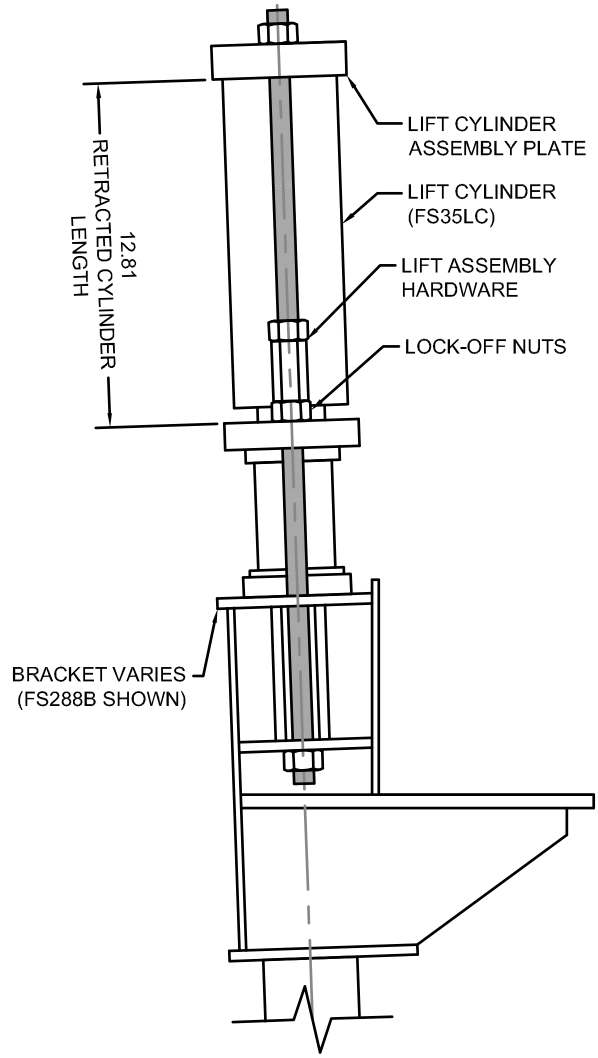
(3) Rated lifting load is given for the lift assembly only. Do not operate at pressures that exceed the allowable capacities of the system which are governed by the allowable capacities of the bracket and other system components, as well as the torque correlated soil capacity, or installed driving force divided by an appropriate factor of safety. All of these governing limits are outlined in places elsewhere in this appendix.

Model 288 Lift Assembly[2] Specifications

Compatible Brackets[4]:
HP238B2, HP288B2, FS288B
FS288BV, FS288BL, FS288BFM

Lift Cylinder (FS35LC):
Stroke = 4”
Cylinder action = double
Bore = Ø3.50”
Hydraulic area = 9.62 in^2
Max operating pressure[3] = 8,000 psi

Lift Assembly Hardware[1,2]:
(2) - Ø¾” x 16” long all-thread rod
with nuts and hex couplers, or
(2) - Ø¾” x 16” long coil rod
with nuts and hex couplers



Lift Assembly Rated Lifting Load[3,4] 39.7 kips			
Hydraulic Pressure (psi)	Lift Force[3,4] (kips)	Hydraulic Pressure (psi)	Lift Force[3,4] (kips)
200	1.9	2,600	25.0
400	3.8	2,800	26.9
600	5.8	3,000	28.9
800	7.7	3,200	30.8
1,000	9.6	3,400	32.7
1,200	11.5	3,600	34.6
1,400	13.5	3,700	35.6
1,600	15.4	3,800	36.6
1,800	17.3	3,900	37.5
2,000	19.2	4,000	38.5
2,200	21.2	4,100	39.4
2,400	23.1	4,130	39.7

(1) Hardware used in the lift assembly must be selected to match the hardware used with the installed bracket assembly.

(2) Note that the only difference between the model 288 and model 350 lift assemblies is the diameter of the threaded rod hardware. All other components of the two assemblies are identical.

(3) Do not operate at pressures that produce lift forces in excess of the lift assembly’s rated lifting load. Max operating pressure of the lift cylinder produces forces that exceed this value and is given for informational purposes only.

(4) Rated lifting load is given for the lift assembly only. Do not operate at pressures that exceed the allowable capacities of the system which are governed by the allowable capacities of the bracket and other system components, as well as the torque correlated soil capacity, or installed driving force divided by an appropriate factor of safety. All of these governing limits are outlined in places elsewhere in this appendix.

Model 350 Lift Assembly[1] Specifications

Compatible Brackets[3]:

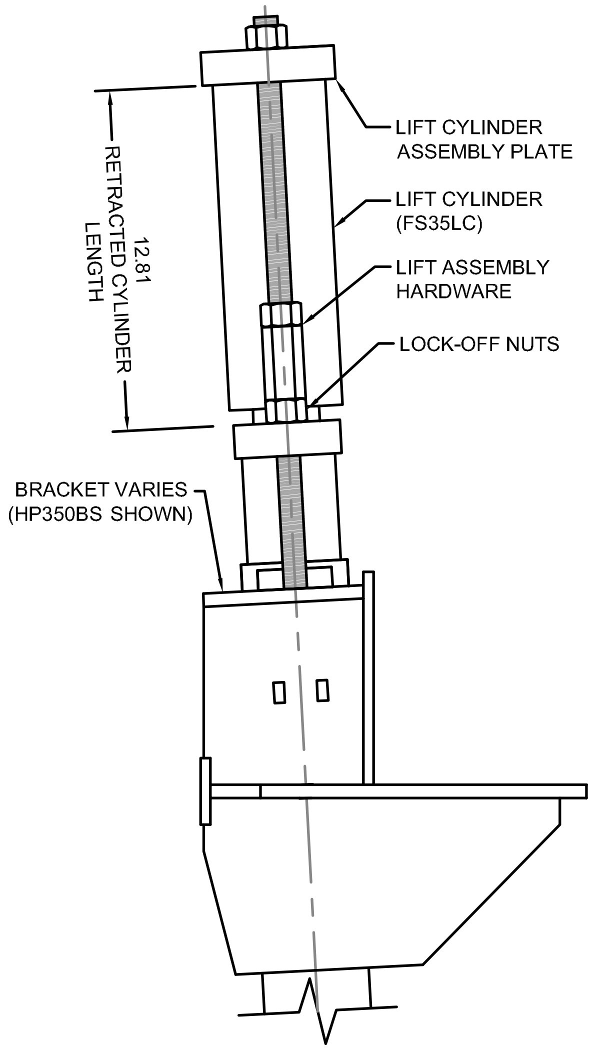
HP350BS, HP350B,
FS350BV, FS400BV

Lift Cylinder (FS35LC):

Stroke = 4”
Cylinder action = double
Bore = Ø3.50”
Hydraulic area = 9.62 in^2
Max operating pressure[2] = 8,000 psi

Lift Assembly Hardware[1]:

(2) - Ø⅞” x 18” long all-thread rod
with nuts and hex couplers



Lift Assembly Rated Lifting Load[2,3] 56.3 kips

Hydraulic Pressure (psi)	Lift Force[2,3] (kips)	Hydraulic Pressure (psi)	Lift Force[2,3] (kips)
400	3.8	3,800	36.6
800	7.7	4,000	38.5
1,200	11.5	4,200	40.4
1,600	15.4	4,400	42.3
2,000	19.2	4,600	44.3
2,400	23.1	4,800	46.2
2,600	25.0	5,000	48.1
2,800	26.9	5,200	50.0
3,000	28.9	5,400	52.0
3,200	30.8	5,600	53.9
3,400	32.7	5,800	55.8
3,600	34.6	5,850	56.3

(1) Note that the only difference between the model 288 and model 350 lift assemblies is the diameter of the threaded rod hardware. All other components of the two assemblies are identical.

(2) Do not operate at pressures that produce lift forces in excess of the lift assembly’s rated lifting load. Max operating pressure of the lift cylinder produces forces that exceed this value and is given for informational purposes only.

(3) Rated lifting load is given for the lift assembly only. Do not operate at pressures that exceed the allowable capacities of the system which are governed by the allowable capacities of the bracket and other system components, as well as the torque correlated soil capacity, or installed driving force divided by an appropriate factor of safety. All of these governing limits are outlined in places elsewhere in this appendix.

Appendix 2C

HelixPro® Helical Foundation Design Software for Professionals

HELIXPRO® HELICAL FOUNDATION DESIGN SOFTWARE FOR PROFESSIONALS

HelixPro is a state-of-the-art web-based program that allows the user to calculate bearing and uplift capacities of helical piles as well as tension capacities of helical tiebacks as they pertain to project specific site conditions and soil profiles. The program is ideal for analyzing both vertical and battered piles for deep foundations of new structures, seismic retrofitting applications, tension/uplift elements of guyed structures, tiebacks for earth retention systems, tiedowns, and more. HelixPro calculates capacities of helical piles and tiebacks using the Individual Bearing Method, which is referenced in Chapter 2 of the FSI Technical Manual.

HelixPro allows the user to quickly perform multiple trials with varying soil profiles and helix configurations to select the most economical and practical solution for the project. The program provides a step-by-step "wizard" approach through the design process, making the program intuitive and easy to navigate. Some of the many features of the software include:

- Video tutorials available on the FSI website: www.OnStableGround.com
- Help menus and buttons along the way to further assist the user through the design process
- Pop-up warnings to alert the user when the torsional rating of the shaft is exceeded, when non-standard helix plate configurations are selected, when minimum depth or embedment criteria are violated, etc.
- Graphical representation of soil layers and helix plate depths
- Graphical representation of installation torque with depth, along with boundary lines to represent the torsional rating of the shaft
- Generation of a summary report with a graphical representation of the proposed installation
- Ability to save and manage projects and sort these projects by date, application and project status
- Links to case studies, current and previous issues of the Foundation Nation for Design Professionals (FNDP) newsletter, and technical content on the FSI website

The software's layout and functionality are illustrated in the following guide and design example. The example utilizes real soil and project information for a guyed tower project completed in St. Louis, Missouri. The tower supports were retrofitted with helical piles and anchors to provide additional support and stability. The example is for the design of the southwest guy support where two Model 150 square shaft helical anchors were installed at a 40 degree batter.

Software Guide and Design Example

Step 1

Following log-in, the program opens to the **Home** page where the user can create a new project or view saved projects. The top menu bar also allows access to **My Profile** where user information is input. User name and company name are automatically incorporated into the final report. The **Home** page also has links to FSI case studies, newsletters and other technical information.

For this example the "Create A New Project" button is selected.

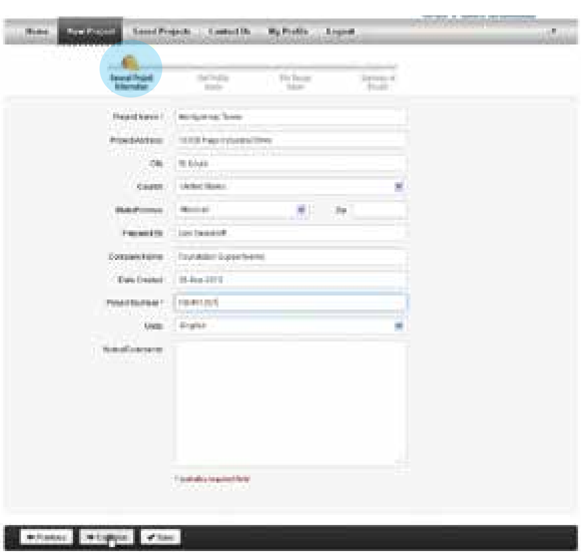
Step 2

The **New Project** page allows the user to choose between the "Helical Piles" or "Helical Tiebacks" modules. The helical piles module is selected to determine capacities of vertical and battered piles in both tension and compression. The helical tiebacks module allows the user to create multiple wall configurations, define the active zone (failure plane) for each wall and determine capacities for multiple rows of tiebacks.

The "Helical Piles" button is selected.

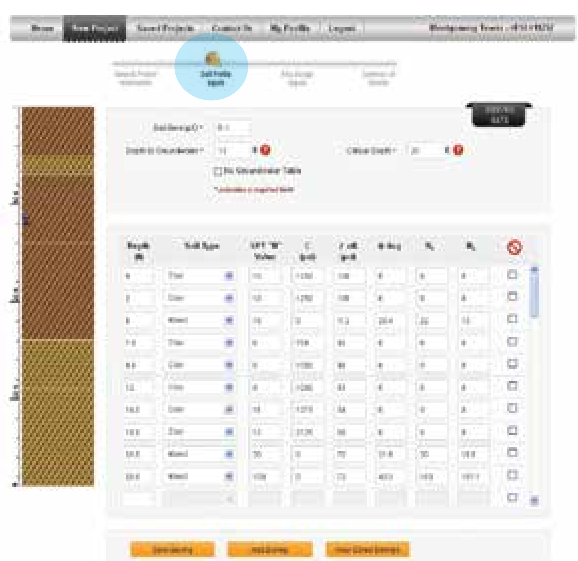
Step 3

The **General Project Information** page allows the user to enter project information and select either English or Metric units. The project name and project number are required fields for this page. The buttons at the bottom allow the user to save the information on this page, go back to the previous page or continue to the next page. Information has been entered for the Montgomery Tower project located in St. Louis, MO. Select "Continue" to navigate to the next screen of data entry.



Step 4

Required fields on the **Soil Profile Inputs** page include soil boring ID, depth to groundwater, critical depth and at least one soil layer. Seven soil types are available including sand, clay, mixed, organics, sand fill, clay fill and mixed fill. Soil strength parameter fields for clay, sand, clay fill and sand fill are populated automatically by correlation to SPT N-values; however, the user can manually override these values by entering new data. Selection of organic or mixed soils requires manual entry of the soil strength parameters. The soil profile is graphically displayed as the data is entered. Multiple borings can be entered and saved. The help menu is accessed by clicking on the question mark icons next to various entry fields. In this example, the user has entered and saved the information from Boring B-1 and is ready to continue to the next page.



Step 5

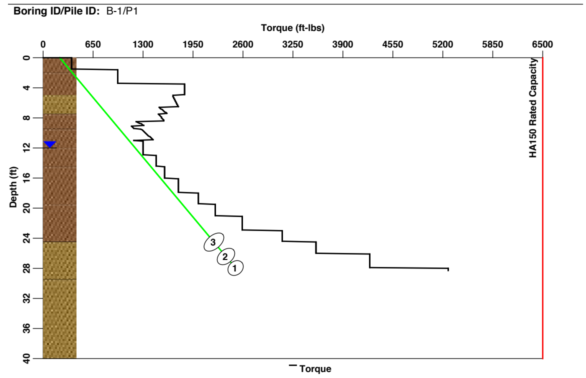
The **Pile Design Inputs** page requires input for boring ID, pile ID, pile shaft type, helix plate configuration and geometry, pile length, batter angle and pile head depth. After required fields have been filled, pile capacity is determined by the program by clicking the “Calculate” button. The ultimate tension and compression capacity, maximum installation torque, final installation torque and depth to maximum installation torque are calculated and displayed. Installation torque versus depth is displayed graphically next to the soil profile. Multiple pile types can be entered and saved.

An ultimate tension capacity of 45.9 kips is determined for the HA150 (1.5-inch round corner square bar) with an 8”-10”-12” helix plate configuration, a 40 degree batter, 37 feet of installed length and the soil conditions represented by Boring B-1.

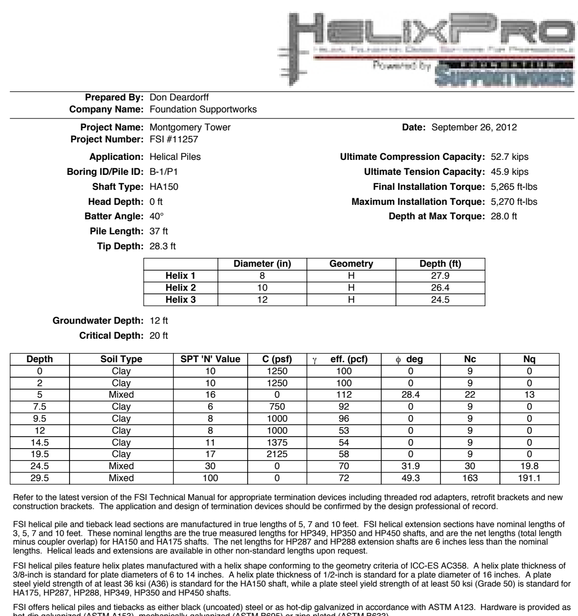
Step 6

Continuing to the **Summary of Results** page allows the user to select the boring/pile combinations to include in the final report, and also the order in which to present the results. Output reports are generated in PDF format for each boring/pile (pile module) or each boring/wall/tieback row (tieback module). The reports are formatted to include all of the input data, the calculated results and other design information needed for project submittals. A graphical representation of the soil profile, helical pile batter and depth, and installation torque with depth is created. For the tieback module, the graphical representation also includes the wall and failure plane geometry.

If you have any questions regarding the software, please feel free to contact FSI through the Contact Us page of the software.



The engineers at Foundation Supportworks®, Inc. utilize HelixPro every day to prepare preliminary design recommendations for design professionals and our contractor network. We are confident that you will also find HelixPro to be a valuable tool for your design of helical foundations.



Prepared By: Don Deardorff
Company Name: Foundation Supportworks

Project Name: Montgomery Tower
Project Number: FSI #11257
Date: September 26, 2012

Application: Helical Piles
Boring ID/Pile ID: B-1/P1
Shaft Type: HA150
Head Depth: 0 ft
Batter Angle: 40°
Pile Length: 37 ft
Tip Depth: 28.3 ft

Ultimate Compression Capacity: 52.7 kips
Ultimate Tension Capacity: 45.9 kips
Final Installation Torque: 5,265 ft-lbs
Maximum Installation Torque: 5,270 ft-lbs
Depth at Max Torque: 28.0 ft

	Diameter (in)	Geometry	Depth (ft)
Helix 1	8	H	27.9
Helix 2	10	H	26.4
Helix 3	12	H	24.5

Groundwater Depth: 12 ft
Critical Depth: 20 ft

Depth	Soil Type	SPT 'N' Value	C (psf)	γ eff. (pcf)	φ deg	Nc	Nq
0	Clay	10	1250	100	0	9	0
2	Clay	10	1250	100	0	9	0
5	Mixed	16	0	112	28.4	22	13
7.5	Clay	6	750	92	0	9	0
9.5	Clay	8	1000	96	0	9	0
12	Clay	8	1000	53	0	9	0
14.5	Clay	11	1375	54	0	9	0
19.5	Clay	17	2125	58	0	9	0
24.5	Mixed	30	0	70	31.9	30	19.8
29.5	Mixed	100	0	72	49.3	163	191.1

Refer to the latest version of the FSI Technical Manual for appropriate termination devices including threaded rod adapters, retrofit brackets and new construction brackets. The application and design of termination devices should be confirmed by the design professional of record.

FSI helical pile and tieback lead sections are manufactured in true lengths of 5, 7 and 10 feet. FSI helical extension sections have nominal lengths of 3, 5, 7 and 10 feet. These nominal lengths are the true measured lengths for HP349, HP350 and HP450 shafts, and are the net lengths (total length minus coupler overlap) for HA150 and HA175 shafts. The net lengths for HP287 and HP288 extension shafts are 6 inches less than the nominal lengths. Helical leads and extensions are available in other non-standard lengths upon request.

FSI helical piles feature helix plates manufactured with a helix shape conforming to the geometry criteria of ICC-ES AC358. A helix plate thickness of 3/8-inch is standard for plate diameters of 6 to 14 inches. A helix plate thickness of 1/2-inch is standard for a plate diameter of 16 inches. A plate steel yield strength of at least 36 ksi (A36) is standard for the HA150 shaft, while a plate steel yield strength of at least 50 ksi (Grade 50) is standard for HA175, HP287, HP288, HP349, HP350 and HP450 shafts.

FSI offers helical piles and tiebacks as either black (uncoated) steel or as hot-dip galvanized in accordance with ASTM A123. Hardware is provided as hot-dip galvanized (ASTM A153), mechanically-galvanized (ASTM B695) or zinc-plated (ASTM B633).

Register now to use this FREE state-of-the-art software program by going to *www.helixpro.foundationsupportworks.com*. Within two working days, you should receive an email stating that your account has been activated. No gimmicks. No strings attached.

Appendix 2D

Pile Bucking Considerations

PILE BUCKLING CONSIDERATIONS

Buckling of helical piles is generally only considered when soil conditions consist of very soft clays or very loose sands with SPT N-values less than 4 blows per foot (bpf). Research has shown that soils with SPT N-values greater than or equal to 4 bpf provide sufficient lateral support to prevent buckling, however, determination of pile buckling is a complex problem that is affected by coupling strength/stiffness, pile batter, shaft section and elastic properties, load type and eccentricity, length of exposed pile shaft and soil strength.

The methods described in this section for buckling evaluation may not account for dynamic loading, partial embedment (exposed pile), pile geometry changes, and stiffness variations due to pile shaft couplings. The methods may be applicable for cases where fully-embedded grout filled pipe piles (with couplings) are used, or for fully-embedded piles without couplings. The design professional should be aware of the buckling design method assumptions as they apply to the helical pile design.

After the critical buckling load is calculated, a factor of safety (FOS) is applied to determine the allowable pile capacity to prevent buckling. A FOS of 1.67 would be consistent with AISC design methods, although helical pile designers routinely use factors of safety in the range of 1.5 to 2.0.

Euler Method

The Euler equation shown below provides an estimation of the elastic critical buckling load for a long, slender, ideal column:

$$P_e = \frac{\pi^2 EI}{(KL)^2}$$

Where,

P_e = Elastic Critical Buckling Load

E = Modulus of Elasticity of the Pile Shaft Cross Section

I = Moment of Inertia of the Pile Shaft Cross Section

K = Effective Length Factor

L = Unsupported Length

An ideal column is one that is perfectly straight, homogeneous, and free from any initial residual stresses. Since an ideal column can only exist in theory, AISC utilizes an adjustment coefficient to normalize the theoretical elastic buckling with the results observed in testing research. The elastic critical buckling load then becomes:

$$P_{crit} = 0.877P_e$$

Where,

P_{crit} = Critical Buckling Load

It should be noted that the Euler Method is only suitable for intermediate length to longer columns that produce values of P_e less than $0.44F_yA$. When the Euler load (P_e) is greater than this value, then inelastic buckling will govern and P_{crit} becomes:

$$P_{crit} = \left[0.658^{\frac{F_yA}{P_e}} \right] F_yA$$

Where,

F_y = Yield Stress

A = Cross Sectional Area

These equations for elastic and inelastic buckling would be applicable to helical piles installed without lateral soil support; e. g., piles with exposed lengths above the ground surface or piles penetrating fluid soils (SPT N-values = 0). In most other conditions, the critical buckling load determined using these equations may be overly conservative.

Davisson Method

The Davisson Method (1963) considers lateral support from the surrounding soil and variable boundary conditions for the pile. This method is based on manipulation of the governing differential equation which assumes the subgrade modulus of the soil is constant with depth along the pile:

$$EI \frac{d^4y}{dx^4} + P \frac{d^2y}{dx^2} + ky = 0$$

Where,

EI = Flexural Stiffness of the Pile

P = Axial Load

k = Subgrade Modulus

The differential equation was solved for various boundary conditions using non-dimensional variables. The boundary conditions are free (f), pinned (p) and fixed-translating (ft). For initial conditions where the pile is fully-embedded, initially straight and the axial load is assumed constant (no skin friction), the dimensionless solutions are shown in *Figure 2D.1*. For further discussion of the derivation of these solutions the reader is advised to see the paper by Davisson (1963).

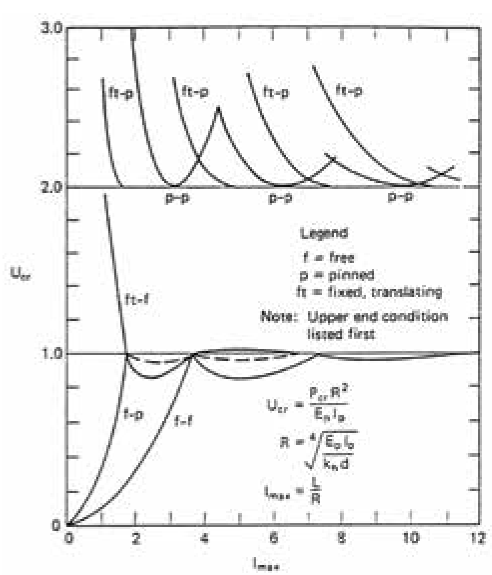


Figure 2D.1 Buckling load (load coefficient) vs. length (depth coefficient) for k_h = constant (Davisson 1963)

The dimensionless variables are the critical axial load coefficient (U_{cr}) and the maximum value of the depth coefficient (I_{max}) and are defined as:

$$U_{cr} = \frac{P_{cr}R^2}{E_pI_p}$$

$$R = \sqrt[4]{\frac{E_pI_p}{k_hd}}$$

$$I_{max} = \frac{L}{R}$$

Where,

P_{cr} = Critical Axial Load

R = Relative Stiffness Factor

E_pI_p = Flexural Stiffness of the Pile

k_h = Horizontal Subgrade Modulus

d = Pile Diameter

L = Shaft Length over which k_h is constant

Typical values of k_h for design purposes are shown in *Figure 2D.2*.

Soil Type-Consistency	Cohesion (psf)	SPT N-value (bpf)	Design kh (pci)
Clay-Very Soft	<250	0-1	<30
Clay-Soft	250-500	2-4	30
Clay-Medium Stiff	500-1000	5-8	100
Sand-Very Loose (above GWT)	NA	0-4	<25
Sand-Very Loose (below GWT)	NA	0-4	<20
Sand-Loose (above GWT)	NA	5-10	25
Sand-Loose (below GWT)	NA	5-10	20

Figure 2D.2 Typical design values for horizontal subgrade modulus (Reese, Wang et al. 2004b)

Example

Consider a fully embedded pile installed to a depth of 25 feet within a soil profile consisting of 15 feet of very soft clay with an average SPT N-value = 1 bpf and an average cohesion value = 200 psf. The very soft clay layer is underlain by a dense sand which extends beyond the pile tip. Based on the pile design, a pinned-pinned boundary condition is selected. A HP287 pile is considered with the following parameters:

E_p = $29(10^6)$ psi

I_p = 1.445 in^4 for plain steel (plain corroded or galvanized corroded could also be considered)

d = 2.875 in. for plain steel (plain corroded or galvanized corroded could also be considered)

L = 15 ft. = 180 in.

Based on the cohesion of 200 psf, a design value of k_h = 20 pci is selected from *Figure 2D.2*.

$$R = \sqrt[4]{\frac{29(10^6)(1.445)}{(20)(2.875)}} = 29.22 \text{ in.}$$

$$I_{max} = \frac{180}{29.22} = 6.2$$

From *Figure 2D.1*, with I_{max} = 6.2 and assuming pinned-pinned (p-p) boundary conditions, U_{cr} = 2.0. The critical buckling load (P_{cr}) can then be calculated from the design equation:

$$P_{cr} = \frac{U_{cr}E_pI_p}{R^2} = \frac{(2)(29)(10^6)(1.445)}{29.22^2}$$

$$= 98{,}160 \text{ lb}$$

Divide the critical buckling load by an appropriate FOS to determine the allowable pile capacity to prevent buckling.

Appendix 2E

Corrosion Considerations

CORROSION CONSIDERATIONS

The term "corrosion" is used to describe the degradation of a material or its properties due to reaction with its environment. Although most materials are known to corrode over time, corrosion is typically considered as the destructive attack of a metal by chemical or electrochemical reaction. During this process, ions from the base metal migrate from the surface, resulting in material loss. As the corrosion process and metal loss continues, there can be a reduction in material thickness and area, which could result in loss of structural capacity of a given member.

Romanoff (1957): "For electrochemical corrosion to occur there must be a potential difference between two points that are electrically connected and immersed in an electrolyte. Whenever these conditions are fulfilled, a small current flows from the anode area through the electrolyte to the cathode area and then through the metal to complete the circuit, and the anode area is the one that has the most negative potential, and is the area that becomes corroded through loss of metal ions to the electrolyte. The cathode area, to which the current flows through the electrolyte, is protected from corrosion because of the deposition of hydrogen or other ions that carry the current."

The following conditions must be met in order for corrosion to occur:

1) There must be two points (anode and cathode) on a metal structure with different electrical potential and these two points must be electrically connected to complete the circuit. The difference in electrical potential could be caused by inconsistencies in the metal, varying stress/strain points, contact with dissimilar metals or materials, etc.

2) There must be an electrolyte to carry current, and for below ground pile applications, soil moisture serves this purpose. The presence of soluble salts increases the electrical conductivity (or lowers resistivity) of the electrolyte, thereby increasing corrosion potential.

There is still much discussion and debate about how much corrosion actually occurs for buried metal, with the central argument typically being the amount of available oxygen. The amount of oxygen within soil decreases significantly just a few feet from the surface, unless the material is loosely-placed fill or an open-graded, granular soil. Relatively speaking, we would then expect these materials to present a higher potential for corrosion than undisturbed clayey soils. The presence of a water table further complicates the discussion as you'd expect less oxygen below the water table than above. It is also important to note that although oxygen-starved environments inhibit rusting, which is a specific type of corrosion, other types of galvanic or bacterial corrosion are still possible.

ICC Evaluation Service, LLC (ICC-ES) Acceptance Criteria 358 (AC358) and ICC-ES AC406 define corrosive soil environments by: (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. In such environments, the steel can be protected with a hot-dip galvanized zinc coating or with other means such as sacrificial anodes. A site-specific evaluation of the soil can be conducted in order to determine an appropriate level of protection. Foundation Supportworks®, Inc. (FSI) recommends that a corrosion engineer be consulted when site or project conditions warrant further evaluation.

FSI helical products and hardware may be ordered as plain (black, uncoated) steel or with a protective coating to further prolong the anticipated service life. Helical pile capacity ratings are therefore provided for plain, plain corroded, and galvanized corroded pile sections. Scheduled corrosion losses are for a period of 50 years and are in accordance with ICC-ES AC358. Helical products (leads, extensions and bracket assemblies) are available hot-dip galvanized in accordance with:

- ASTM A123, Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products

Hardware and fasteners may be hot-dip galvanized, electro-plated, or mechanically galvanized in accordance with:

- ASTM A153, Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware
- ASTM B633, Standard Specification for Electrodeposited Coatings of Zinc on Iron and Steel
- ASTM B695, Standard Specification for Coatings of Zinc Mechanically Deposited on Iron and Steel

A Common Sense Discussion

Potential corrosion may be an objection for specifiers considering helical piles. These specifiers may feel that helical piles may not be an appropriate option because of their concerns about the steel corroding away and leaving the supported structure on a compromised foundation. While it's true that steel does corrode over time, it is actually quite rare that corrosion will govern the design of new construction helicals. This is because of the nature of how helical piles are designed and installed. To state it simply, the amount of steel which is required to develop the necessary torque during installation far exceeds the amount of steel that is required to resist the design axial compressive forces. This can be demonstrated in the following example.

A helical pile is required to resist an allowable compressive load of 35 kips. The FSI Model HP288 helical pile is selected for the project (see Appendix 2A, Helical Product Ratings, Properties and Details). The pile is installed to a torque of 7,800 ft-lb to provide an ultimate torque-correlated soil capacity of 70 kips (FOS = 2.0). The pile has an uncorroded cross-sectional area of the shaft of 2.11 in^2 and an allowable mechanical axial capacity of 74.0 kips on the day the pile is installed. However, the overall allowable pile capacity would remain at 35 kips, limited by the installation torque and the correlated allowable soil capacity, even though the steel section in the ground is capable of a great deal more.

Following installation, we can now consider the effects of corrosion. ICC-ES AC358 provides scheduled losses or "sacrificial thicknesses" for black steel or steel with protective coatings, and these sacrificial thicknesses must be considered for design purposes. These sacrificial thicknesses are based on moderately corrosive soils over a period of 50 years. This is a design criteria only and should not be confused with service life. In our example, after 50 years in the ground, a black, uncoated steel pile would have lost a steel thickness of 0.036 inch due to corrosion. The pile would have a remaining cross-sectional area of the shaft of 1.82 in^2 and an allowable (mechanical) axial capacity of 63.6 kips. This is the value that Foundation Supportworks lists as the "plain corroded" allowable mechanical axial capacity in compression for the HP288. The overall allowable pile capacity still remains 35 kips, limited by the installation torque that was applied 50 years earlier.

So how much steel would have to be lost before corrosion would begin to govern the design? See *Figure 2E.1*. The remaining allowable mechanical capacity does not fall below the allowable pile capacity of 35 kips from our example until the sacrificial thickness reaches 0.135 inch. This is nearly four times greater than the scheduled 50-year corrosion loss rate for black steel and over eight times greater than the scheduled 50-year corrosion loss rate for hot-dip galvanized steel.

Corrosion is a very complex subject involving many factors which can affect loss rates. With some understanding, it quickly becomes apparent that even if the corrosive properties of the soil at a particular site are especially aggressive, it is still quite rare for corrosion to govern the design of a helical pile solution.

	Sacrificial Thickness (in)	Steel Area (in^2)	Allowable Mechanical Capacity (k)	Sacrificial Thickness (in)	Steel Area (in^2)	Allowable Mechanical Capacity (k)
Day of installation	**0.000**	**2.11**	**74.0**	0.090	1.37	48.1
	0.005	2.07	72.5	0.095	1.33	46.6
	0.100	1.29	45.2	0.100	1.29	45.2
	0.015	1.99	69.7	0.105	1.25	43.8
Scheduled 50 year corrosion loss for zinc coated steel per AC358	**0.016**	**1.98**	**69.4**	0.110	1.21	42.3
	0.020	1.95	68.2	0.115	1.17	40.9
	0.025	1.91	66.8	0.120	1.13	39.4
	0.030	1.87	65.3	0.125	1.09	38.0
	0.035	1.83	63.9	0.130	1.04	36.6
Scheduled 50 year corrosion loss for plain black steel per AC358	**0.036**	**1.82**	**63.6**	0.135	1.00	35.1
	0.040	1.78	62.5	0.140	0.96	33.7
	0.045	1.74	61.0	0.145	0.92	32.2
	0.050	1.70	59.6	0.150	0.88	30.8
	0.055	1.66	58.1	0.155	0.84	29.4
	0.060	1.62	56.7	0.160	0.80	27.9
	0.065	1.58	55.3	0.165	0.76	26.5
	0.070	1.54	53.8	0.170	0.72	25.0
	0.075	1.50	52.4	0.175	0.67	23.6
	0.080	1.46	51.0	0.180	0.63	22.2
	0.085	1.41	49.5	0.185	0.59	20.7

Figure 2E.1 HP288 steel areas and allowable mechanical capacities for increasing sacrificial thicknesses

Appendix 2F

Model Specifications

MODEL SPECIFICATION FOR HELICAL PILE FOUNDATIONS COMPRESSION APPLICATIONS

1 SCOPE

1.1 The work consists of designing, furnishing and installing helical piles and load transfer devices used to support compressive loads according to the project Plans and these specifications.

1.2 The parties and contract terms referred to in this specification are as follows:

1.2.1 The Owner is the person or entity that owns the facility or will own the facility once it is completed. The Owner may have contractual agreements with, and be represented by, other parties such as engineers, architects or contractors that perform services under the direction of the Owner. Where Owner is used in this specification, it refers to the Owner or the Owner's contracted representatives separate from the Installing Contractor.

1.2.2 The Pile Designer is the individual or firm generally hired by the Installing Contractor to design the helical piles.

1.2.3 The Installing Contractor installs and tests (if necessary) the helical piles, and possibly performs other tasks associated with the project.

1.2.4 The Plans refer to the contract documents; including but not limited to the drawings and specifications for the project.

1.3 The work may include helical pile load testing.

1.4 The Owner will be responsible for obtaining any right-of-way or easement access permits necessary for the helical pile installation.

1.5 Unless otherwise noted, the Installing Contractor shall provide all labor, tools, equipment and materials necessary to accomplish the work.

1.6 The Owner will provide suitable access to the construction site for the Installing Contractor's personnel and equipment.

1.7 Unless specifically noted otherwise in the contract documents, the Owner will remove and replace any structures, utilities, pavements, landscaping or other surficial improvements in the work area as necessary to facilitate the work.

1.8 The Owner will be responsible for overall construction oversight to preclude the development of unsafe conditions.

1.9 The Owner will be responsible for a horizontal field survey of the helical pile locations prior to helical pile installation and an elevation survey to determine pile shaft cutoff height subsequent to helical pile installation.

1.10 The work does not include any post-construction monitoring of pile performance unless specifically noted otherwise in the contract documents.

2 REFERENCES

2.1 American Institute of Steel Construction (AISC)

2.1.1 AISC 360: Specification for Structural Steel Buildings

2.2 American Society for Testing and Materials (ASTM)

2.2.1 ASTM A36: Carbon Structural Steel

2.2.2 ASTM A123: Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products

2.2.3 ASTM A153: Zinc Coating (Hot-Dip) on Iron and Steel Hardware

2.2.4 ASTM A307: Carbon Steel Bolts, Studs, and Threaded Rod 60 000 PSI Tensile Strength

2.2.5 ASTM A325: Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength

2.2.6 ASTM A500: Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

2.2.7 ASTM A513: Electric-Resistance Welded Carbon and Alloy Steel Mechanical Tubing

2.2.8 ASTM A572: High-Strength Low-Alloy Columbian-Vanadium Structural Steel

2.2.9 ASTM B633: Electrodeposited Coatings of Zinc on Iron and Steel

2.2.10 ASTM D1143: Deep Foundations Under Static Axial Compressive Load

2.3 International Code Council Evaluation Services (ICC-ES)

2.3.1 Acceptance Criteria 358 (AC358): Acceptance Criteria for Helical Pile Systems and Devices

2.4 Society of Automotive Engineers (SAE)

2.4.1 SAE J429: Mechanical and Material Requirements for Externally Threaded Fasteners

3 DEFINITIONS

3.1 The following terms apply to helical piles used to support compressive loads:

3.1.1 Allowable Stress Design: A structural and geotechnical design methodology that states that the summation of the actual estimated loads (nominal loads) must be less than or equal to the allowable design load (required strength). Allowable loads are obtained by dividing a nominal resistance (strength) by an appropriate factor of safety.

3.1.2 Bearing Stratum: The soil layer (or layers) that provide the helical pile end-bearing capacity through load transfer from the helical plates.

3.1.3 Crowd: Axial compressive force applied to the helical pile shaft as needed during installation to ensure the pile advances at a rate approximately equal to the helix pitch for each revolution.

3.1.4 Design Loads: A generic and ambiguous term used to describe any load used in design. It is not specific to factored or unfactored loads or any particular design methodology. It is a term; therefore, that should be avoided when specifying load requirements. FSI recommends using the term service load, nominal load or factored load, as described herein, where applicable.

3.1.5 Design Strength: A term used in structural design which is defined as the product of the nominal strength and the applicable resistance factor. An equivalent term typically used in geotechnical design is, also sometimes referred to as factored resistance (Load and Resistance Factor Design).

3.1.6 Extension Section: Helical pile shaft sections connected to the lead section or other extension sections to advance the helix plates to the required bearing depth. Plain extensions (without helix plates) or helical extensions (with one or more helix plates) may be used depending upon soil conditions or project requirements.

3.1.7 Factor of Safety: The ratio of the ultimate pile capacity or nominal resistance (strength) to the nominal or service load used in the design of any helical pile component or interface (Allowable Stress Design).

3.1.8 Factored Load: The product of a nominal load and an applicable load factor (Load and Resistance Factor Design).

3.1.9 Factored Resistance: The product of a nominal resistance and an applicable resistance factor (Load and Resistance and Factor Design).

3.1.10 Geotechnical Capacity: The maximum load or the load at a specified limit state, that can be resisted through the piles interaction with the bearing soils (see also Ultimate Pile Capacity).

3.1.11 Helical Pile: Consists of a central steel shaft with one or more helix-shaped bearing plates and a load transfer device (bracket) that allows attachment to structures. Helical piles are installed into the ground by application of torque and axial compressive force (“crowd”).

3.1.12 Helix (Helical) Plate: Generally round steel plate formed into a helical spiral and welded to the central steel shaft. When rotated in the ground, the helix shape provides thrust along the pile’s longitudinal axis thus aiding in pile installation. The plate transfers axial load to the soil through bearing.

3.1.13 Helix Pitch: The distance measured along the axis of the shaft between the leading and trailing edges of the helix plate.

3.1.14 Lead Section: The first helical pile shaft component installed into the soil. It consists of one or more helical plates welded to a central steel shaft.

3.1.15 Limit State: A condition beyond which a helical pile component or interface becomes unfit for service and is judged to no longer be useful for its intended function (serviceability limit state) or to be unsafe (ultimate limit state (strength)).

3.1.16 Load and Resistance Factor Design: A structural and geotechnical design methodology that states that the Factored Resistance (Design Strength) must be greater than or equal to the summation of the applied factored loads.

3.1.17 Load Factor: A factor that accounts for the probability of deviation of the actual load from the predicted nominal load due to variability of material properties, workmanship, type of failure and uncertainty in the prediction of the load (Load and Resistance Factor Design).

3.1.18 Load Test: A process to test the ultimate pile capacity and relation of applied load to pile head settlement by application of a known load on the helical pile head and monitoring movement over a specific time period.

3.1.19 Loads: Forces that result from the weight of all building materials, occupants and their possessions, environmental effects, differential movement, and restrained dimensional changes. Permanent loads are those loads in which variations over time are rare or of small magnitude. All other loads are variable loads (see also Nominal Loads).

3.1.20 Mechanical Strength: The maximum load or the load at a specified limit state that can be resisted by the structural elements of a helical pile.

3.1.21 Net Deflection: The total settlement at the pile head minus the theoretical elastic deformation of the pile shaft during a load test.

3.1.22 Nominal Loads: The magnitude of the loads specified, which include dead, live, soil, wind, snow, rain, flood and earthquakes (also referred to as service loads or working loads).

3.1.23 Nominal Resistance: The pile capacity at a specified ultimate limit state (Load and Resistance Factor Design). See Ultimate Pile Capacity.

3.1.24 Nominal Strength: A term used in structural design which is defined as the structure or member capacity at a specified strength limit state. See Ultimate Pile Capacity.

3.1.25 Resistance Factor: A factor that accounts for the probability of deviation of the actual resistance (strength) from the predicted nominal resistance (strength) due to variability of material properties, workmanship, type of failure and uncertainties in the analysis (Load and Resistance Factor Design).

3.1.26 Service Loads: See "Nominal Loads" above.

3.1.27 Ultimate Pile Capacity: The helical pile capacity based on the least capacity determined from applicable ultimate limit states for mechanical and geotechnical capacity.

4 APPROVED HELICAL PILE MANUFACTURERS

4.1 Foundation Supportworks®, Inc., 12330 Cary Circle, Omaha, NE 68128; Phone: (800) 281-8545; Fax: (402) 393-4002.

4.2 Due to the special requirements for design and manufacturing of helical piles, the piles shall be obtained from Foundation Supportworks®, Inc., or other qualified manufacturer with an approved equivalent product. A request to substitute any other manufactured helical product must be submitted to the Owner for review not less than seven (7) calendar days prior to the bid date. The request must include:

4.2.1 Documentation of at least five years of production experience manufacturing helical piles, 4.2.2 Documentation that the manufacturer's helical piles have been used successfully in at least five engineered construction projects within the last three years,

4.2.3 Product acceptance by the local building code official(s) having jurisdiction over the project, and/or

4.2.4 Current ICC-ES product evaluation report or complete description of product testing and manufacturing quality assurance programs used to assess and maintain product quality and determine product mechanical strength and geotechnical capacity.

5 ACCEPTABLE PRODUCTS

5.1 Hollow Round Shaft Helical Pile Models HP237, HP287, HP288, HP350, HP450, HP662 and HP700 manufactured in accordance with the requirements of Sections 5 and 6 of this specification.

5.1.1 Hollow round shaft helical piles shall be used to resist compression loads. Round shaft helical piles are generally more resistant to bending or buckling over solid square shaft counterparts due to superior cross-sectional properties and coupling details.

5.1.2 Pile shaft sections shall be in full, direct contact within couplings so as to remove coupling bolts and coupling welds from the "in-service" axial load path.

5.1.3 Pile shafts and couplings shall have a fit-up tolerance of 1/16 inch or less.

5.1.4 Helix plates shall meet the following geometry and spacing criteria to minimize soil disturbance:

5.1.4.1 True helix-shaped plates that are normal to the shaft such that the leading and trailing edges are within ¼ inch of parallel.

5.1.4.2 Helix pitch is 3 inches ± ¼ inch.

5.1.4.3 All helix plates have the same pitch.

5.1.4.4 Helix plates have generally circular edge geometry.

5.1.4.5 Helix spacing along the shaft shall be between 2.4 and 3.6 times the helix diameter.

5.1.4.6 Helix plates are arranged along the shaft such that they all theoretically track the same path as the proceeding plate.

6 MATERIALS

6.1 Model HP237 Helical Pile System

6.1.1 Central Steel Shaft: The central steel shaft of the lead and extension sections are 2.375-inch outer diameter by 0.154-inch nominal wall thickness, hollow structural section in conformance with ASTM A500 Grade B or C with a minimum yield strength of 60 ksi and a minimum tensile strength of 70 ksi. The shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.1.2 Shaft Coupling Material: The shaft coupling material is factory welded to the extension shaft and consists of 2.750-inch outer diameter by 0.156-inch nominal wall thickness, hollow structural section in conformance with ASTM A513 Type 5, Grade 1026 with a minimum yield strength of 70 ksi and a minimum tensile strength of 80 ksi. The shaft coupling finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.1.3 Helix Plate Material: The helix plates are factory welded to the lead or extension shaft sections and consist of either 0.313 or 0.375-inch thick ASTM A572 Grade 50 steel with a minimum yield strength of 50 ksi and a minimum tensile strength of 65 ksi. Helix plate outer diameters are 6, 8, 10, 12 or 14 inches. The helix plate finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.1.4 Shaft Coupling Hardware: The lead and extension shaft sections are coupled with two (2) bolts and nuts per coupled shaft section. The coupling hardware consists of 0.625-inch standard hex bolts conforming to ASTM A325 and heavy hex jam nuts. The bolts and nuts are hot-dip galvanized in accordance with ASTM A153.

6.1.5 Brackets: New construction bracket HP238NCB and retrofit brackets HP238B2 and HP238BML are designed for use with the HP237 shaft. Bracket finishes are either plain steel or hot-dip galvanized in accordance with ASTM A123. Bracket hardware finishes for the new construction brackets are hot-dip galvanized in accordance with ASTM A153. Bracket hardware finishes for the retrofit brackets are zinc coated in accordance with ASTM B633.

6.2 Model HP287 and Model HP288 Helical Pile Systems

6.2.1 Central Steel Shaft: The central steel shaft of the lead and extension sections are 2.875-inch outer diameter by 0.203-inch nominal wall thickness (HP287) or 0.276-inch nominal wall thickness (HP288), hollow structural section in conformance with ASTM A500 Grade B or C with a minimum yield strength of 60 ksi and a minimum tensile strength of 70 ksi. The shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.2.2 Shaft Coupling Material: The shaft coupling material is factory welded to the extension shaft and consists of 3.500-inch outer diameter by 0.281-inch nominal wall thickness, hollow structural section in conformance with ASTM A513 Type 5, Grade 1026 with a minimum yield strength of 70 ksi and a minimum tensile strength of 80 ksi. The shaft coupling finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.2.3 Helix Plate Material: The helix plates are factory welded to the lead or extension shaft sections. Helix plates with outer diameters of 6, 8, 10, 12 or 14 inches are either 0.375 or 0.500-inch thick and 16-inch diameter helix plates are 0.500-inch thick. The helix plates are manufactured with ASTM A572 Grade 50 steel with a minimum yield strength of 50 ksi and a minimum tensile strength of 65 ksi. The helix plate finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.2.4 Shaft Coupling Hardware: The lead and extension shaft sections are coupled with two (2) bolts and nuts per coupled shaft section. The coupling hardware consists of 0.750-inch standard hex bolts conforming to SAE J429 Grade 8 and standard jam nuts. The bolts and nuts are zinc coated in accordance with ASTM B633.

6.2.5 Brackets: New construction brackets HP288NCB or HP288NCB8 shall be used for tension and compression applications and HP288NCBE or HP288NCBE8 shall be used for compression only applications with the HP287 or HP288 shafts. Retrofit brackets FS288B, FS288BL, HP288B2, and HP288BML are designed for use with the HP287 or HP288 shafts. Bracket finishes are either plain steel or hot-dip galvanized in accordance with ASTM A123. Bracket hardware finishes are zinc coated in accordance with ASTM B633.

6.3 Model HP350 Helical Pile System

6.3.1 Central Steel Shaft: The central steel shaft of the lead and extension sections are 3.500-inch outer diameter by 0.340-inch nominal wall thickness, hollow structural section in conformance with ASTM A500 Grade B or C with a minimum tensile strength of 75 ksi. The shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.3.2 Shaft Coupling Material: The shaft coupling material consists of 4.250-inch outer diameter by 0.344-inch nominal wall thickness, hollow structural section in conformance with ASTM A513 Type 5, Grade 1026 with a minimum yield strength of 70 ksi and a minimum tensile strength of 80 ksi. The shaft coupling finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.3.3 Helix Plate Material: The helix plates are factory welded to the lead or extension shaft sections. Helix plates with outer diameters of 6, 8, 10, 12 or 14 inches are either 0.375 or 0.500-inch thick and 16-inch diameter helix plates are 0.500-inch thick. The helix plates are manufactured with ASTM A572 Grade 50 steel with a minimum yield strength of 50 ksi and a minimum tensile strength of 65 ksi. The helix plate finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.3.4 Shaft Coupling Hardware: The lead and extension shaft sections are coupled with four (4) bolts and nuts per coupled shaft section. The coupling hardware consists of 1.000-inch standard hex bolts conforming to SAE J429 Grade 5 and standard hex jam nuts. The bolts and nuts are zinc coated in accordance with ASTM B633.

6.3.5 Brackets: New construction brackets HP350NCB or HP350NCB8 shall be used for tension and compression applications and HP350NCBE or HP350NCBE8 shall be used for compression only applications with the HP350 shaft. Retrofit brackets HP350B and HP350BS are designed to use with the HP350 shaft. Bracket finishes are either plain steel or hot-dip galvanized in accordance with ASTM A123. Bracket hardware finishes are zinc coated in accordance with ASTM B633.

6.4 Model HP450 Helical Pile System

6.4.1 Central Steel Shaft: The central steel shaft of the lead and extension sections are 4.500-inch outer diameter by 0.337-inch nominal wall thickness, hollow structural section in conformance with ASTM A500 Grade B or C with a minimum yield strength of 50 ksi and a minimum tensile strength of 60 ksi. The shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.4.2 Shaft Coupling Material: The shaft coupling material consists of 3.750-inch outer diameter by 0.500-inch nominal wall thickness, hollow structural section in conformance with ASTM A513 Type 5, Grade 1026 with a minimum yield strength of 70 ksi and a minimum tensile strength of 80 ksi. The shaft coupling finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.4.3 Helix Plate Material: The helix plates are factory welded to the lead or extension shaft sections. Helix plates with outer diameters of 8, 10, 12 or 14 inches are either 0.375 or 0.500-inch thick and 16-inch diameter helix plates are 0.500-inch thick. The helix plates are manufactured with ASTM A572 Grade 50 steel with a minimum yield strength of 50 ksi and a minimum tensile strength of 65 ksi. The helix plate finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.4.4 Shaft Coupling Hardware: The lead and extension shaft sections are coupled with four (4) bolts and nuts per coupled shaft section. The coupling hardware consists of 1.125-inch standard hex bolts conforming to SAE J429 Grade 5 and standard hex jam nuts. The bolts and nuts are zinc coated in accordance with ASTM B633.

6.4.5 Brackets: New construction bracket HP450NCB8 shall be used for tension and compression applications and HP450NCBE8 shall be used for compression only applications with the HP450 shaft. Bracket finishes are either plain steel or hot-dip galvanized in accordance with ASTM A123. Bracket hardware finishes are zinc coated in accordance with ASTM B633.

6.5 Model HP662 Helical Pile System

6.5.1 This section is for general information purposes only. Larger diameter product lines, such as Model HP662, are typically customized on a project specific basis.

6.5.1.1 Central Steel Shaft: The central steel shaft of the lead and extension sections are 6.625-inch outer diameter by 0.280-inch nominal wall thickness, hollow structural section in conformance with ASTM A500 Grade B or C with a minimum yield strength of 60 ksi and a minimum tensile strength of 70 ksi. The shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.5.1.2 Shaft Coupling Material: The shaft coupling material consists of 6.000-inch outer diameter by 0.375-inch nominal wall thickness, hollow structural section in conformance with ASTM A513 Type 5, Grade 1026 with a minimum yield strength of 70 ksi and a minimum tensile strength of 80 ksi. The shaft coupling finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.5.1.3 Helix Plate Material: The helix plates are factory welded to the lead or extension shaft sections. Helix plates with outer diameters of 10, 12 or 14 inches are either 0.375 or 0.500-inch thick; 16-inch diameter helices are 0.500-inch thick. Helix plates are manufactured with ASTM A572 Grade 50 steel with a minimum yield strength of 50 ksi and a minimum tensile strength of 65 ksi. The helix plate finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.5.1.4 Shaft Coupling Hardware: The lead and extension shaft sections are coupled with four (4) bolts and nuts per coupled shaft section. The coupling hardware consists of 1.750-inch standard hex bolts conforming to SAE J429 Grade 5 and standard hex nuts. The bolts and nuts are zinc coated in accordance with ASTM B633.

6.6 Model HP700 Helical Pile System

6.6.1 This section is for general information purposes only. Larger diameter product lines, such as Model HP700, are typically customized on a project specific basis.

6.6.1.1 Central Steel Shaft: The central steel shaft of the lead and extension sections are 7.000-inch outer diameter by 0.362-inch nominal wall thickness, hollow structural section in conformance with ASTM A252 Grade 3 with a minimum yield strength of 60 ksi and a minimum tensile strength of 70 ksi. The shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.6.1.2 Shaft Coupling Material: The shaft coupling material consists of 7.750-inch outer diameter by 0.313-inch nominal wall thickness, hollow structural section in conformance with ASTM A513 Type 5, Grade 1026 with a minimum yield strength of 70 ksi and a minimum tensile strength of 80 ksi. The shaft coupling finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.6.1.3 Helix Plate Material: The helix plates are factory welded to the lead or extension shaft sections. Helix plates with outer diameters of 10, 12 or 14 inches are either 0.375 or 0.500-inch thick; 16-inch diameter helices are 0.500-inch thick. Helix plates are manufactured with ASTM A572 Grade 50 steel with a minimum yield strength of 50 ksi and a minimum tensile strength of 65 ksi. The helix plate finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.6.1.4 Shaft Coupling Hardware: The lead and extension shaft sections are coupled with four (4) bolts and nuts per coupled shaft section. The coupling hardware consists of 2.000-inch heavy hex bolts conforming to ASTM A307 and heavy hex nuts. The bolts and nuts are zinc coated in accordance with ASTM B633.

7 DESIGN AND PERFORMANCE REQUIREMENTS

7.1 Helical piles shall be designed to support the specified compressive load(s) as shown on the project Plans. The overall length, helix configuration and minimum torsional resistance of a helical pile shall be such that the required capacity is developed by the helix plate(s) in an appropriate bearing stratum.

7.2 All structural steel pile components shall be designed within the limits provided by the American Institute of Steel Construction (AISC) Specification for Structural Steel Buildings (AISC-360). Either Allowable Stress Design (ASD) or Load and Resistance Factor Design (LRFD) are acceptable methods of analysis. Product testing in accordance with ICC-ES Acceptance Criteria 358 may also be considered as an acceptable means of establishing system capacities.

7.3 Except where noted otherwise on the project Plans, all piles shall be installed to provide an ultimate torque-correlated capacity based on an ASD or LRFD analysis. For ASD, a minimum factor of safety of 2 applied to the service or nominal loading shall be required. When an LRFD analysis is required, the Owner shall provide applicable pile design information including but not limited to; factored loads, resistance factors and/or the required ultimate pile capacity. Factors of safety (ASD) or resistance factors (LRFD) may require modification to meet specific deflection criteria stated on the Plans or drawings.

7.4 The required ultimate torque-correlated capacity shall be verified at each pile location by monitoring and recording the final installation torque and applying default torque correlations per ICC-ES AC358. Site specific torque correlation factors may be determined by field compression load testing as specified in Section 14.

7.5 Except where noted otherwise on the project Plans, each pile shall be designed to meet a corrosion service life of 50 years in accordance with ICC-ES AC358.

7.6 The pile design shall take into account group efficiency from pile spacing, pile buckling potential, soil stratification, and strain compatibility issues.

8 QUALIFICATIONS OF INSTALLING CONTRACTOR AND DESIGNER

8.1 The Installing Contractor and/or Pile Designer shall submit to the Owner, a proposal including the documentation required in this Section. Work shall not begin until all the submittals have been received and approved by the Owner. All costs associated with incomplete or unacceptable submittals shall be the responsibility of the Installing Contractor.

8.2 Evidence of Installing Contractor's competence in the installation of helical piles shall be provided to the Owner's satisfaction and may include any or all of the following:

8.2.1 Pile manufacturer's certificate of competency for the installation of helical piles,

8.2.2 A list of at least three projects completed within the previous three years wherein the Installing Contractor installed helical piles similar to those shown in the project Plans. Such list to include names and phone numbers of those project representatives who can verify the Installing Contractor's participation in those projects, and/or

8.2.3 A letter from the pile manufacturer or manufacturer's representative expressing ability and intent to provide on-site supervision of the pile installation.

8.3 A listing of all safety violations lodged against the Installing Contractor within the previous three years and the current status or final resolutions thereof. Descriptions of safety improvements instituted within the previous three years may also be submitted, at the Installing Contractor's discretion.

8.4 Evidence of Pile Designer's competence shall be provided to the Owner's satisfaction and may include any or all of the following:

8.4.1 Registration as a Professional Engineer or recognition by the local jurisdictional authority, 8.4.2 A list of at least three projects completed within the previous three years wherein the Pile Designer designed helical piles similar to those shown in the project Plans. The list shall include names and phone numbers of those project representatives who can verify the Pile Designer's participation in those projects, and/or

8.4.3 Recommendation from the pile manufacturer or manufacturer's representative.

9 PRE-CONSTRUCTION SUBMITTALS

9.1 Within 2 weeks of receiving the contract award, the Installing Contractor and/or Pile Designer shall submit the following helical pile design documentation:

9.1.1 Certification from the Pile Designer that the proposed piles meet the requirements of this specification.

9.1.2 Qualifications of the Installing Contractor and Pile Designer per Section 8.

9.1.3 Product designations for helical lead and extension sections and all ancillary products to be supplied at each helical pile location.

9.1.4 Individual pile nominal loads, factors of safety, LRFD load and resistance factors and required ultimate torque correlated capacities, where applicable.

9.1.5 Individual pile loading requirements (if any).

9.1.6 Manufacturer's published allowable system capacities for the proposed pile assemblies, including load transfer devices.

9.1.7 Calculated mechanical and theoretical geotechnical capacities of the proposed piles.

9.1.8 Minimum pile termination torque requirements.

9.1.9 Maximum estimated installation torque and allowable installation torque rating of the proposed piles.

9.1.10 Minimum and/or maximum embedment lengths or other site specific embedment depth requirements as may be appropriate for the site soil profiles.

9.1.11 Inclination angle and location tolerance requirements.

9.1.12 Load test procedures and failure criteria, if applicable.

9.1.13 Copies of certified calibration reports for torque measuring equipment and load test measuring equipment to be used on the project. The calibrations shall have been performed within one year of the proposed helical pile installation starting date or as recommended by the equipment manufacturer.

9.1.14 Provide proof of insurance coverage as stated in the general specifications and/or contract.

10 PLACEMENT REQUIREMENTS

10.1 Helical piles shall be installed within 3 inches of the indicated plan location.

10.2 Helical pile shaft alignment shall be within 2 degrees of the inclination angle shown on the Plans.

10.3 Top elevation of the helical piles shall be within 2 inches of the design vertical elevation.

11 PILE INSTALLATION

11.1 Installing Contractor shall furnish and install all helical piles per the project Plans and approved pile design documentation. In the event of conflict between the project Plans and the approved pile design documentation, the Installing Contractor shall not begin construction on any affected items until such conflict has been resolved.

11.2 The Installing Contractor shall conduct their construction operations in a manner to insure the safety of persons and property in the vicinity of the work. The Installing Contractor's personnel shall comply with safety procedures in accordance with OSHA standards and any established project safety plan.

11.3 The Owner shall request marking of underground utilities by an underground utility location service as required by law, and the Installing Contractor shall avoid contact with all marked underground facilities.

11.4 The portion of the construction site occupied by the Installing Contractor, including equipment and material stockpiles shall be kept reasonably clean and orderly.

11.5 Installation of helical piles may be observed by representatives of the Owner for quality assurance purposes. The Installing Contractor shall give the Owner at least 24 hours' notice prior to the pile installation operations.

11.6 The helical pile installation technique shall be such that it is consistent with the geotechnical, logistical, environmental, and load carrying conditions of the project. The lead section shall be positioned at the appropriate site survey stake location as determined from the plan drawings.

11.7 The helical pile sections shall be advanced into the soil in a continuous manner at a rate of rotation less than 25 revolutions per minute (rpm). Sufficient crowd shall be applied to advance the helical pile sections at a rate approximately equal to the pitch of the helix plate per revolution. The rate of rotation and magnitude of down pressure shall be adjusted for different soil conditions and depths. Extension sections shall be provided to obtain the required minimum overall length and minimum torsional resistance as shown on the project Plans.

12 TERMINATION CRITERIA

12.1 The minimum final torsional resistance and/or any required pile length and embedment depth criteria, as specified in the Pre-Construction Submittals, must be satisfied prior to terminating the pile installation. In the event any helical pile fails to meet these production quality control termination criteria, the following remedies may be suitable if authorized by the Owner:

12.1.1 If the installation fails to meet the minimum torsional resistance criterion at the minimum embedment length:

12.1.1.1 Continue the installation to greater depths until the torsional resistance criterion is met, provided that, if a maximum length constraint is applicable, continued installation does not exceed said maximum length constraint, or

12.1.1.2 Demonstrate acceptable pile performance through pile load testing, or

12.1.1.3 Replace the pile with one having a different helix plate configuration. The replacement pile must not exceed any applicable maximum embedment length criteria and either: (A) be embedded to a length that places the last helix plate at least equal to its own diameter beyond the depth of the first helix plate of the replaced pile and meet the minimum torsional resistance criterion; or (B) pass pile load testing criteria.

12.1.2 If the torsional resistance during installation reaches the helical pile's allowable torque rating prior to satisfaction of the minimum embedment length criterion:

12.1.2.1 Terminate the installation at the depth obtained, or

12.1.2.2 Replace the pile with one having a shaft with a higher torsional strength rating. The replacement pile must be installed to satisfy the minimum embedment length criterion. It must also be embedded to a length that places the last helix plate at least equal to its own diameter beyond the depth of the first helix plate of the replaced pile without exceeding any applicable maximum embedment length requirements and it must meet the minimum final torsional resistance criterion, or

12.1.2.3 Replace the pile with one having a different helix plate configuration. The replacement pile must be installed to satisfy the minimum embedment length criterion. It must also be embedded to a length that places the last helix plate at least equal to its own diameter beyond the depth of the first helix plate of the replaced pile without exceeding any applicable maximum embedment length requirements, and it must meet the minimum final torsional resistance criterion.

12.1.3 If the installation reaches a specified maximum embedment length without achieving the minimum torsional resistance criterion:

12.1.3.1 If allowed, remove and reinstall the pile at a position at least three times the diameter of the largest helix plate away from the initial location. Original embedment length and torsional resistance criteria must be met. The pile repositioning may require the installation of additional helical piles with nominal loads adjusted for these spacing changes, or

12.1.3.2 Demonstrate acceptable pile performance through pile load testing, or

12.1.3.3 De-rate the load capacity of the helical pile based on default or site specific torque correlation factors and install additional piles as necessary, or

12.1.3.4 Replace the pile with one having a different helix plate configuration. The replacement pile must be installed to satisfy the minimum and/or maximum embedment length criterion and it must meet the minimum final torsional resistance criterion.

12.1.4 If a helical pile fails to meet the acceptance criteria in a pile load test:

12.1.4.1 Install the pile to a greater depth and installation torque and re-test; provided that, if a maximum embedment length constraint is applicable, continued installation will not exceed said maximum length constraint, or

12.1.4.2 Replace the pile with one having more and/or larger helix plates. The replacement pile must be embedded to a length that places the last helix plate at equal to its own diameter beyond the depth of the first helix plate of the replaced pile without exceeding any applicable maximum embedment length requirements. The replacement pile must be re-tested, or,

12.1.4.3 De-rate the load capacity of the helical pile based on the results of the load test and install additional piles. Additional piles must be installed at positions that are at least three times the diameter of the largest helix plate away from any other pile locations.

12.1.5 If a helical pile fails a production quality control criterion as described in this Section or for any reason other than described in this Section, any proposed remedy must be approved by the Owner prior to initiating its implementation at the project site.

13 INSTALLATION RECORD SUBMITTALS

13.1 The Installing Contractor shall provide the Owner copies of the individual helical pile installation records within 24 hours after each installation is completed. Formal copies shall be submitted within 30 days following the completion of the helical pile installation. These installation records shall include, but are not limited to, the following information:

13.1.1 Date and time of installation

13.1.2 Location of helical pile and pile identification number

13.1.3 Installed helical pile model and configuration

13.1.4 Termination depth, pile head depth, and length of installed pile

13.1.5 Actual inclination of the pile

13.1.6 Final torsional resistance

13.1.7 Calculated geotechnical capacity based on final torsional resistance

13.1.8 Comments pertaining to interruptions, obstructions, or other relevant information

14 FIELD COMPRESSION LOAD TESTING

14.1 If field compression load testing is required, the Installing Contractor shall furnish all labor, equipment and pre-production helical piles necessary to accomplish the testing as shown in the approved pile design documentation. Installing Contractor shall apply the specified loads for the specified durations and record the specified data, for the specified number of piles. No deviations from the test plan(s) will be allowed without explicit approval in writing from the Owner. Pile testing shall be in general accordance with the ASTM D1143 quick test method and the following criteria:

14.1.1 Failure criteria shall be in accordance with AC358 and is when plunging occurs or when the net deflection exceeds 10% of the average helix plate diameter, whichever occurs first.

14.1.2 An alignment load equal to 5% of the anticipated failure load or maximum anticipated test load may be applied prior to the start of the test to take out slack in the load test frame.

14.1.3 Loading increments shall be performed at 5% of the anticipated failure load or maximum anticipated test load with a minimum hold time of 4 minutes at each increment.

14.1.4 Upon completion of the maximum test load hold increment, the pile shall be unloaded in 5 to 10 even increments with minimum hold times of 4 minutes at each increment.

14.2 Installing Contractor shall provide the Owner copies of raw field test data within 24 hours after the completion of each load test. Formal test reports shall be submitted within 30 days following test completion. Formal test reports shall include the following information:

14.2.1 Name of project and Installing Contractor's representative(s) present during load testing.

14.2.2 Name of manufacturer's representative(s) present during load testing, if any.

14.2.3 Name of third party test agency and personnel present during load testing, if any.

14.2.4 Date, time, duration and type of the load test.

14.2.5 Unique test identifier and map showing the test pile location.

14.2.6 Pile model and installation information including shaft type, helix configuration, lead and extension section quantities and lengths, final pile tip depth, installation date, total test pile length and final termination torque.

14.2.7 Calibration records for applicable pile installation and test equipment.

14.2.8 Tabulated test results including cumulative pile head movement, loading increments and hold times.

14.2.9 Plots showing load versus deflection for each loading/unloading interval.

15 CLEANUP

15.1 Within one week of completion of the work, the Installing Contractor shall remove any and all material, equipment, tools, debris or other items belonging to the Installing Contractor or used under the Installing Contractor's direction.

MODEL SPECIFICATION FOR HELICAL ANCHOR FOUNDATIONS TENSION APPLICATIONS

1 SCOPE

1.1 The work consists of designing, furnishing and installing helical anchors and load transfer devices used to support tension loads according to the project Plans and these specifications.

1.2 The parties and contract terms referred to in this specification are as follows:

1.2.1 The Owner is the person or entity that owns the facility or will own the facility once it is completed. The Owner may have contractual agreements with, and be represented by, other parties such as engineers, architects or contractors that perform services under the direction of the Owner. Where Owner is used in this specification, it refers to the Owner or the Owner's contracted representatives separate from the Installing Contractor.

1.2.2 The Anchor Designer is the individual or firm generally hired by the Installing Contractor to design the helical anchors.

1.2.3 The Installing Contractor installs and tests (if necessary) the helical anchors, and possibly performs other tasks associated with the project.

1.2.4 The Plans refer to the contract documents; including but not limited to the drawings and specifications for the project.

1.3 The work may include helical anchor load testing.

1.4 The Owner will be responsible for obtaining any right-of-way or easement access permits necessary for the helical anchor installation.

1.5 Unless otherwise noted, the Installing Contractor shall provide all labor, tools, equipment and materials necessary to accomplish the work.

1.6 The Owner will provide suitable access to the construction site for the Installing Contractor's personnel and equipment.

1.7 Unless specifically noted otherwise in the contract documents, the Owner will remove and replace any structures, utilities, pavements, landscaping or other surficial improvements in the work area as necessary to facilitate the work.

1.8 The Owner will be responsible for overall construction oversight to preclude the development of unsafe conditions.

1.9 The Owner will be responsible for a horizontal field survey of the helical anchor locations prior to helical anchor installation and a post installation survey to determine anchor shaft cutoff lengths (if necessary).

1.10 The work does not include any post-construction monitoring of anchor performance unless specifically noted otherwise in the contract documents.

2 REFERENCES

2.1 American Institute of Steel Construction (AISC)

2.1.1 AISC 360: Specification for Structural Steel Buildings

2.2 American Society for Testing and Materials (ASTM)

2.2.1 ASTM A29: Steel Bars, Carbon and Alloy, Hot-Wrought

2.2.2 ASTM A36: Carbon Structural Steel

2.2.3 ASTM A123: Zinc Coating (Hot-Dip) Coatings on Iron and Steel Products

2.2.4 ASTM A153: Zinc Coating (Hot-Dip) on Iron and Steel Hardware

2.2.5 ASTM A307: Carbon Steel Bolts, Studs, and Threaded Rod 60,000 PSI Tensile Strength

2.2.6 ASTM A325: Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength

2.2.7 ASTM A500: Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

2.2.8 ASTM A513: Electric-Resistance Welded Carbon and Alloy Steel Mechanical Tubing

2.2.9 ASTM A572: High-Strength Low-Alloy Columbian-Vanadium Structural Steel

2.2.10 ASTM B633: Electrodeposited Coatings of Zinc on Iron and Steel

2.2.11 ASTM B695: Coatings of Zinc Mechanically Deposited on Iron and Steel

2.2.12 ASTM D3689: Deep Foundations Under Static Axial Tensile Load

2.3 International Code Council Evaluation Services (ICC-ES)

2.3.1 Acceptance Criteria 358 (AC358): Acceptance Criteria for Helical Pile Systems and Devices

2.4 Society of Automotive Engineers (SAE)

2.4.1 SAE J429: Mechanical and Material Requirements for Externally Threaded Fasteners

3 DEFINITIONS

3.1 The following terms apply to helical anchors used to support tension loads:

3.1.1 Allowable Stress Design: A structural and geotechnical design methodology that states that the summation of the actual estimated loads (nominal loads) must be less than or equal to the allowable design load (required strength). Allowable loads are obtained by dividing a nominal resistance (strength) by an appropriate factor of safety.

3.1.2 Bearing Stratum: The soil layer (or layers) that provides the helical anchor end-bearing capacity through load transfer from the helical plates.

3.1.3 Crowd: Axial compressive force applied to the helical anchor shaft as needed during installation to ensure the anchor advances at a rate approximately equal to the helix pitch for each revolution.

3.1.4 Design Loads: A generic and ambiguous term used to describe any load used in design. It is not specific to factored or unfactored loads or any particular design methodology. It is a term; therefore, that should be avoided when specifying load requirements. FSI recommends using the term service load, nominal load or factored load, as described herein, where applicable.

3.1.5 Design Strength: A term used in structural design which is defined as the product of the nominal strength and the applicable resistance factor. An equivalent term typically used in geotechnical design is, also sometimes referred to as factored resistance (Load and Resistance Factor Design).

3.1.6 Extension Section: Helical anchor shaft sections connected to the lead section or other extension sections to advance the helix plates to the required bearing depth. Plain extensions (without helix plates) or helical extensions (with one or more helix plates) may be used depending upon soil conditions or project requirements.

3.1.7 Factor of Safety: The ratio of the ultimate anchor capacity or nominal resistance (strength) to the nominal or service load used in the design of any helical anchor component or interface (Allowable Stress Design).

3.1.8 Factored Load: The product of a nominal load and an applicable load factor (Load and Resistance Factor Design).

3.1.9 Factored Resistance: The product of a nominal resistance and an applicable resistance factor (Load and Resistance and Factor Design).

3.1.10 Geotechnical Capacity: The maximum load or the load at a specified limit state, that can be resisted through the anchors interaction with the bearing soils (see also Ultimate Anchor Capacity).

3.1.11 Helical Anchor: Consists of a central steel shaft with one or more helix-shaped bearing plates and a load transfer device (bracket) that allows attachment to structures. Helical anchors are installed into the ground by application of torque and axial compressive force (“crowd”).

3.1.12 Helix (Helical) Plate: Generally round steel plate formed into a helical spiral and welded to the central steel shaft. When rotated in the ground, the helix shape provides thrust along the anchor’s longitudinal axis thus aiding in anchor installation. The plate transfers axial load to the soil through bearing.

3.1.13 Helix Pitch: The distance measured along the axis of the shaft between the leading and trailing edges of the helix plate.

3.1.14 Lead Section: The first helical anchor shaft component installed into the soil. It consists of one or more helical plates welded to a central steel shaft.

3.1.15 Limit State: A condition beyond which a helical anchor component or interface becomes unfit for service and is judged to no longer be useful for its intended function (serviceability limit state) or to be unsafe (ultimate limit state (strength)).

3.1.16 Load and Resistance Factor Design: A structural and geotechnical design methodology that states that the Factored Resistance (Design Strength) must be greater than or equal to the summation of the applied factored loads.

3.1.17 Load Factor: A factor that accounts for the probability of deviation of the actual load from the predicted nominal load due to variability of material properties, workmanship, type of failure and uncertainty in the prediction of the load (Load and Resistance Factor Design).

3.1.18 Load Test: A process to test the ultimate anchor capacity and relation of applied load to anchor head movement by application of a known load on the helical anchor head and monitoring movement over a specific time period.

3.1.19 Loads: Forces that result from the weight of all building materials, occupants and their possessions, environmental effects, differential movement, and restrained dimensional changes. Permanent loads are those loads in which variations over time are rare or of small magnitude. All other loads are variable loads (see also Nominal Loads).

3.1.20 Mechanical Strength: The maximum load or the load at a specified limit state that can be resisted by the structural elements of a helical anchor.

3.1.21 Net Deflection: The total movement at the anchor head minus the theoretical elastic deformation of the anchor shaft during a load test.

3.1.22 Nominal Loads: The magnitude of the loads specified, which include dead, live, soil, wind, snow, rain, flood and earthquakes (also referred to as service loads or working loads).

3.1.23 Nominal Resistance: The anchor capacity at a specified ultimate limit state (Load and Resistance Factor Design). See Ultimate Anchor Capacity.

3.1.24 Nominal Strength: A term used in structural design which is defined as the structure or member capacity at a specified strength limit state. See Ultimate Anchor Capacity.

3.1.25 Resistance Factor: A factor that accounts for the probability of deviation of the actual resistance (strength) from the predicted nominal resistance (strength) due to variability of material properties, workmanship, type of failure and uncertainties in the analysis (Load and Resistance Factor Design).

3.1.26 Service Loads: See “Nominal Loads” above.

3.1.27 Ultimate Anchor Capacity: The helical anchor capacity based on the least capacity determined from applicable ultimate limit states for mechanical and geotechnical capacity.

4 APPROVED HELICAL ANCHOR MANUFACTURERS

4.1 Foundation Supportworks®, Inc., 12330 Cary Circle, Omaha, NE 68128; Phone: (800) 281-8545; Fax: (402) 393-4002.

4.2 Due to the special requirements for design and manufacturing of helical anchors, the anchors shall be obtained from Foundation Supportworks®, Inc., or other qualified manufacturer with an approved equivalent product. A request to substitute any other manufactured helical product must be submitted to the Owner for review not less than seven (7) calendar days prior to the bid date. The request must include:

4.2.1 Documentation of at least five years of production experience manufacturing helical anchors,

4.2.2 Documentation that the manufacturer's helical anchors have been used successfully in at least five engineered construction projects within the last three years,

4.2.3 Product acceptance by the local building code official(s) having jurisdiction over the project, and/or

4.2.4 Current ICC-ES product evaluation report or complete description of product testing and manufacturing quality assurance programs used to assess and maintain product quality and determine product mechanical strength and geotechnical capacity.

5 ACCEPTABLE PRODUCTS FOR TENSION ONLY APPLICATIONS

5.1 Solid Square Shaft Helical Anchor Models HA150 and HA175 manufactured in accordance with the requirements of Sections 5 and 7 of this specification.

5.1.1 Solid round corner square shaft helical anchors may be used for tension only applications.

5.1.2 Helix plates shall meet the following geometry and spacing criteria to minimize soil disturbance:

5.1.2.1 True helix-shaped plates that are normal to the shaft such that the leading and trailing edges are within ¼ inch of parallel.

5.1.2.2 Helix pitch is 3 inches ± ¼ inch.

5.1.2.3 All helix plates have the same pitch.

5.1.2.4 Helix plates have generally circular edge geometry.

5.1.2.5 Helix spacing along the shaft shall be between 2.4 and 3.6 times the helix diameter.

5.1.2.6 Helix plates are arranged along the shaft such that they all theoretically track the same path as the proceeding plate.

6 ACCEPTABLE PRODUCTS FOR COMBINED COMPRESSION AND TENSION APPLICATIONS

6.1 Hollow Round Shaft Helical Pile/Anchor Models HP237, HP287, HP288, HP350, HP450, HP662 and HP700 manufactured in accordance with the requirements of Sections 6 and 7 of this specification.

6.1.1 Hollow round shaft helical piles/anchors shall be used in applications of alternating compression and tension loads. During compression loading, round shaft helical piles/anchors are generally more resistant to bending or buckling over solid square shaft counterparts due to superior cross-sectional properties and coupling details.

6.1.2 During compression loading, pile/anchor shaft sections shall be in full, direct contact within couplings so as to remove coupling bolts and coupling welds from the "in-service" axial load path.

6.1.3 Pile/anchor shafts and couplings shall have a fit-up tolerance of 1⁄16 inch or less.

6.1.4 Helix plates shall meet the following geometry and spacing criteria to minimize soil disturbance:

6.1.4.1 True helix-shaped plates that are normal to the shaft such that the leading and trailing edges are within ¼ inch of parallel.

6.1.4.2 Helix pitch is 3 inches ± ¼ inch.

6.1.4.3 All helix plates have the same pitch.

6.1.4.4 Helix plates have generally circular edge geometry.

6.1.4.5 Helix spacing along the shaft shall be between 2.4 and 3.6 times the helix diameter.

6.1.4.6 Helix plates are arranged along the shaft such that they all theoretically track the same path as their proceeding plate.

7 MATERIALS

7.1 Model HA150 Helical Anchor System

7.1.1 Central Steel Shaft: The central steel shaft of the lead and extension sections are 1.50-inch, solid, round-corner square (RCS) hot-rolled steel bars conforming to ASTM A29 with a minimum yield strength of 90 ksi and a minimum tensile strength of 115 ksi. The shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.1.2 Shaft Coupling Material: The extension shaft sections have an internally forged upset socket coupling at one end. Since the socket coupling is internally forged from the parent shaft material, the material properties of the coupling are similar to the central steel shaft. The shaft coupling finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.1.3 Helix Plate Material: The helix plates are factory welded to the shaft lead or extension shaft sections. Helix plates with outer diameters of 6, 8, 10, 12 or 14 inches are either 0.375 or 0.500-inch thick and 16-inch diameter helix plates are 0.500-inch thick. The helix plates are manufactured with ASTM A572 Grade 50 steel with a minimum yield strength of 50 ksi and a minimum tensile strength of 65 ksi. The helix plate finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.1.4 Shaft Coupling Hardware: The lead and extension shaft sections are coupled with one (1) bolt and nut per coupled shaft section. The coupling hardware consists of 0.750-inch standard hex bolts conforming to SAE J429 Grade 8 and jam nuts. The bolts and nuts are mechanically galvanized in accordance with ASTM B695.

7.1.5 Brackets: New construction bracket HA150NCB and thread rod adaptor HA150TRA are suitable for tension applications with the HA150 shaft. Bracket finishes are either plain steel or hot-dip galvanized in accordance with ASTM A123. Bracket shaft coupling hardware finishes are mechanically galvanized in accordance with ASTM B695.

7.2 Model HA175 Helical Anchor System

7.2.1 Central Steel Shaft: The central steel shaft of the lead and extension sections are 1.75-inch, solid, round-corner square (RCS) hot-rolled steel bars conforming to ASTM A29 with a minimum yield strength of 90 ksi and a minimum tensile strength of 115 ksi. The shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.2.2 Shaft Coupling Material: The extension shaft sections have an internally forged upset socket coupling at one end. Since the socket coupling is internally forged from the parent shaft material, the material properties of the coupling are similar to the central steel shaft. The shaft coupling finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.2.3 Helix Plate Material: The helix plates are factory welded to the lead or extension shaft sections. Helix plates with outer diameters of 6, 8, 10, 12 or 14 inches are either 0.375 or 0.500-inch thick and 16-inch diameter helix plates are 0.500-inch thick. The helix plates are manufactured with ASTM A572 Grade 50 steel with a minimum yield strength of 50 ksi and a minimum tensile strength of 65 ksi. The helix plate finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.2.4 Shaft Coupling Hardware: The lead and extension shaft sections are coupled with two (2) bolts and nuts per coupled shaft section. The coupling hardware consists of 0.750-inch standard hex bolts conforming to SAE J429 Grade 8 and standard hex jam nuts. The bolts and nuts are mechanically galvanized in accordance with ASTM B695.

7.2.5 Brackets: New construction bracket HA175NCB and thread rod adaptor HA175TRA are suitable for tension applications with the HA175 shaft. Bracket finishes are either plain steel or hot-dip galvanized in accordance with ASTM A123. Bracket shaft coupling hardware finishes are mechanically galvanized in accordance with ASTM B695.

7.3 Model HP237 Helical Pile/Anchor System

7.3.1 Central Steel Shaft: The central steel shaft of the lead and extension sections are 2.375-inch outer diameter by 0.154-inch nominal wall thickness, hollow structural section in conformance with ASTM A500 Grade B or C with a minimum yield strength of 60 ksi and a minimum tensile strength of 70 ksi. The shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.3.2 Shaft Coupling Material: The shaft coupling material is factory welded to the extension shaft and consists of 2.750-inch outer diameter by 0.156-inch nominal wall thickness, hollow structural section in conformance with ASTM A513 Type 5, Grade 1026 with a minimum yield strength of 70 ksi and a minimum tensile strength of 80 ksi. The shaft coupling finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.3.3 Helix Plate Material: The helix plates are factory welded to the lead or extension shaft sections and consist of either 0.313 or 0.375-inch thick ASTM A572 Grade 50 steel with a minimum yield strength of 50 ksi and a minimum tensile strength of 65 ksi. Helix plate outer diameters are 6, 8, 10, 12 or 14 inches. The helix plate finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.3.4 Shaft Coupling Hardware: The lead and extension shaft sections are coupled with two (2) bolts and nuts per coupled shaft section. The coupling hardware consists of 0.625-inch standard hex bolts conforming to ASTM A325 and heavy hex jam nuts. The bolts and nuts are hot-dip galvanized in accordance with ASTM A153.

7.3.5 Brackets: New construction bracket HP238NCB shall be used for both tension and compression applications with the HP237 shaft. Bracket finishes are either plain steel or hot-dip galvanized in accordance with ASTM A123. Bracket hardware finishes are hot-dip galvanized in accordance with ASTM A153.

7.4 Model HP287 and Model HP288 Helical Pile/Anchor Systems

7.4.1 Central Steel Shaft: The central steel shaft of the lead and extension sections are 2.875-inch outer diameter by 0.203-inch nominal wall thickness (HP287) or 0.276-inch nominal wall thickness (HP288), hollow structural section in conformance with ASTM A500 Grade B or C with a minimum yield strength of 60 ksi and a minimum tensile strength of 70 ksi. The shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.4.2 Shaft Coupling Material: The shaft coupling material is factory welded to the extension shaft and consists of 3.500-inch outer diameter by 0.281-inch nominal wall thickness, hollow structural section in conformance with ASTM A513 Type 5, Grade 1026 with a minimum yield strength of 70 ksi and a minimum tensile strength of 80 ksi. The shaft coupling finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.4.3 Helix Plate Material: The helix plates are factory welded to the lead or extension shaft sections. Helix plates with outer diameters of 6, 8, 10, 12 or 14 inches are either 0.375 or 0.500-inch thick and 16-inch diameter helix plates are 0.500-inch thick. The helix plates are manufactured with ASTM A572 Grade 50 steel with a minimum yield strength of 50 ksi and a minimum tensile strength of 65 ksi. The helix plate finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.4.4 Shaft Coupling Hardware: The lead and extension shaft sections are coupled with two (2) bolts and nuts per coupled shaft section. The coupling hardware consists of 0.750-inch standard hex bolts conforming to SAE J429 Grade 8 and standard jam nuts. The bolts and nuts are zinc coated in accordance with ASTM B633.

7.4.5 Brackets: New construction brackets HP288NCB or HP288NCB8 shall be used for both tension and compression applications with the HP287 or HP288 shafts. Bracket finishes are either plain steel or hot-dip galvanized in accordance with ASTM A123. Bracket hardware finishes are zinc coated in accordance with ASTM B633.

7.5 Model HP350 Helical Pile/Anchor System

7.5.1 Central Steel Shaft: The central steel shaft of the lead and extension sections are 3.500-inch outer diameter by 0.340-inch nominal wall thickness, hollow structural section in conformance with ASTM A500 Grade B or C with a minimum yield strength of 65 ksi and a minimum tensile strength of 75 ksi. The shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.5.2 Shaft Coupling Material: The shaft coupling material consists of 4.250-inch outer diameter by 0.344-inch nominal wall thickness, hollow structural section in conformance with ASTM A513 Type 5, Grade 1026 with a minimum yield strength of 70 ksi and a minimum tensile strength of 80 ksi. The shaft coupling finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.5.3 Helix Plate Material: The helix plates are factory welded to the lead or extension shaft sections. Helix plates with outer diameters of 6, 8, 10, 12 or 14 inches are either 0.375 or 0.500-inch thick and 16-inch diameter helix plates are 0.500-inch thick. The helix plates are manufactured with ASTM A572 Grade 50 steel with a minimum yield strength of 50 ksi and a minimum tensile strength of 65 ksi. The helix plate finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.5.4 Shaft Coupling Hardware: The lead and extension shaft sections are coupled with four (4) bolts and nuts per coupled shaft section. The coupling hardware consists of 1.000-inch standard hex bolts conforming to SAE J429 Grade 5 and standard hex jam nuts. The bolts and nuts are zinc coated in accordance with ASTM B633.

7.5.5 Brackets: New construction brackets HP350NCB or HP350NCB8 shall be used for both tension and compression applications. Bracket finishes are either plain steel or hot-dip galvanized in accordance with ASTM A123. Bracket hardware finishes are zinc coated in accordance with ASTM B633.

7.6 Model HP450 Helical Pile/Anchor System

7.6.1 Central Steel Shaft: The central steel shaft of the lead and extension sections are 4.500-inch outer diameter by 0.337-inch nominal wall thickness, hollow structural section in conformance with ASTM A500 Grade B or C with a minimum yield strength of 50 ksi and a minimum tensile strength of 60 ksi. The shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.6.2 Shaft Coupling Material: The shaft coupling material consists of 3.750-inch outer diameter by 0.500-inch nominal wall thickness, hollow structural section in conformance with ASTM A513 Type 5, Grade 1026 with a minimum yield strength of 70 ksi and a minimum tensile strength of 80 ksi. The shaft coupling finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.6.3 Helix Plate Material: The helix plates are factory welded to the lead or extension shaft sections. Helix plates with outer diameters of 8, 10, 12 or 14 inches are either 0.375 or 0.500-inch thick and 16-inch diameter helix plates are 0.500-inch thick. The helix plates are manufactured with ASTM A572 Grade 50 steel with a minimum yield strength of 50 ksi and a minimum tensile strength of 65 ksi. The helix plate finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.6.4 Shaft Coupling Hardware: The lead and extension shaft sections are coupled with four (4) bolts and nuts per coupled shaft section. The coupling hardware consists of 1.125-inch standard hex bolts conforming to SAE J429 Grade 5 and standard hex jam nuts. The bolts and nuts are zinc coated in accordance with ASTM B633.

7.6.5 Brackets: New construction bracket HP450NCB8 shall be used for both tension and compression applications with the HP450 shaft. Bracket finishes are either plain steel or hot-dip galvanized in accordance with ASTM A123. Bracket hardware finishes are zinc coated in accordance with ASTM B633.

7.7 Model HP662 Helical Pile/Anchor System

7.7.1 This section is for general information purposes only. Larger diameter product lines, such as Model HP662, are typically customized on a project specific basis.

7.7.1.1 Central Steel Shaft: The central steel shaft of the lead and extension sections are 6.625-inch outer diameter by 0.280-inch nominal wall thickness, hollow structural section in conformance with ASTM A500 Grade B or C with a minimum yield strength of 60 ksi and a minimum tensile strength of 70 ksi. The shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.7.1.2 Shaft Coupling Material: The shaft coupling material consists of 6.000-inch outer diameter by 0.375-inch nominal wall thickness, hollow structural section in conformance with ASTM A513 Type 5, Grade 1026 with a minimum yield strength of 70 ksi and a minimum tensile strength of 80 ksi. The shaft coupling finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.7.1.3 Helix Plate Material: The helix plates are factory welded to the lead or extension shaft sections. Helix plates with outer diameters of 10, 12 or 14 inches are either 0.375 or 0.500-inch thick; 16-inch diameter helices are 0.500-inch thick. Helix plates are manufactured with ASTM A572 Grade 50 steel with a minimum yield strength of 50 ksi and a minimum tensile strength of 65 ksi. The helix plate finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.7.1.4 Shaft Coupling Hardware: The lead and extension shaft sections are coupled with four (4) bolts and nuts per coupled shaft section. The coupling hardware consists of 1.750-inch standard hex bolts conforming to SAE J429 Grade 5 and standard hex nuts. The bolts and nuts are zinc coated in accordance with ASTM B633.

7.8 Model HP700 Helical Pile/Anchor System

7.8.1 This section is for general information purposes only. Larger diameter product lines, such as Model HP700, are typically customized on a project specific basis.

7.8.1.1 Central Steel Shaft: The central steel shaft of the lead and extension sections are 7.000-inch outer diameter by 0.362-inch nominal wall thickness, hollow structural section in conformance with ASTM A252 Grade 3 with a minimum yield strength of 60 ksi and a minimum tensile strength of 70 ksi. The shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.8.1.2 Shaft Coupling Material: The shaft coupling material consists of 7.750-inch outer diameter by 0.313-inch nominal wall thickness, hollow structural section in conformance with ASTM A513 Type 5, Grade 1026 with a minimum yield strength of 70 ksi and a minimum tensile strength of 80 ksi. The shaft coupling finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.8.1.3 Helix Plate Material: The helix plates are factory welded to the lead or extension shaft sections. Helix plates with outer diameters of 10, 12 or 14 inches are either 0.375 or 0.500-inch thick; 16-inch diameter helices are 0.500-inch thick. Helix plates are manufactured with ASTM A572 Grade 50 steel with a minimum yield strength of 50 ksi and a minimum tensile strength of 65 ksi. The helix plate finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

7.8.1.4 Shaft Coupling Hardware: The lead and extension shaft sections are coupled with four (4) bolts and nuts per coupled shaft section. The coupling hardware consists of 2.000-inch heavy hex bolts conforming to ASTM A307 and heavy hex nuts. The bolts and nuts are zinc coated in accordance with ASTM B633.

8 DESIGN AND PERFORMANCE REQUIREMENTS

8.1 Helical anchors shall be designed to support the specified tension load(s) as shown on the project Plans. The overall length, helix configuration and minimum torsional resistance of a helical anchor shall be such that the required geotechnical capacity is developed by the helix plate(s) in an appropriate bearing stratum.

8.2 All structural steel anchor components shall be designed within the limits provided by the American Institute of Steel Construction (AISC) Specification for Structural Steel Buildings (AISC-360). Either Allowable Stress Design (ASD) or Load and Resistance Factor Design (LRFD) are acceptable methods of analysis. Product testing in accordance with ICC-ES Acceptance Criteria 358 may also be considered as an acceptable means of establishing system capacities.

8.3 Except where noted otherwise on the project Plans, all anchors shall be installed to provide an ultimate torque-correlated capacity based on an ASD or LRFD analysis. For ASD, a minimum factor of safety of 2 applied to the service or nominal loading shall be required. When an LRFD analysis is required, the Owner shall provide applicable anchor design information including but not limited to; factored loads, resistance factors and/or the required ultimate anchor capacity. Factors of safety (ASD) or resistance factors (LRFD) may require modification to meet specific deflection criteria stated on the Plans or drawings.

8.4 The required ultimate torque-correlated capacity shall be verified at each anchor location by monitoring and recording the final installation torque and applying default torque correlations per ICC-ES AC358. Site specific torque correlation factors may be determined by field tension load testing as specified in Section 15.

8.5 Except where noted otherwise on the project Plans, each anchor shall be designed to meet a corrosion service life of 50 years in accordance with ICC-ES AC358.

8.6 The anchor design shall take into account group efficiency from anchor spacing, soil stratification, and strain compatibility issues.

9 QUALIFICATIONS OF INSTALLING CONTRACTOR AND ANCHOR DESIGNER

9.1 The Installing Contractor and/or Anchor Designer shall submit to the Owner, a proposal including the documentation required in this Section. Work shall not begin until all the submittals have been received and approved by the Owner. All costs associated with incomplete or unacceptable submittals shall be the responsibility of the Installing Contractor.

9.2 Evidence of Installing Contractor's competence in the installation of helical anchors shall be provided to the Owner's satisfaction and may include any or all of the following:

9.2.1 Anchor manufacturer's certificate of competency in installation of helical anchors,

9.2.2 A list of at least three projects completed within the previous three years wherein the Installing Contractor installed helical anchors similar to those shown in the project Plans. Such list to include names and phone numbers of those project representatives who can verify the Installing Contractor's participation in those projects, and/or

9.2.3 A letter from the anchor manufacturer or manufacturer's representative expressing ability and intent to provide on-site supervision of the anchor installation.

9.3 A listing of all safety violations lodged against the Installing Contractor within the previous three years and the current status or final resolutions thereof. Descriptions of safety improvements instituted within the previous three years may also be submitted, at the Installing Contractor's discretion.

9.4 Evidence of Anchor Designer's competence shall be provided to the Owner's satisfaction and may include any or all of the following:

9.4.1 Registration as a Professional Engineer or recognition by the local jurisdictional authority,

9.4.2 A list of at least three projects completed within the previous three years wherein the Anchor Designer designed helical anchors similar to those shown in the project Plans. The list shall include names and phone numbers of those project representatives who can verify the Anchor Designer's participation in those projects, and/or

9.4.3 Recommendation from the anchor manufacturer or manufacturer's representative.

10 PRE-CONSTRUCTION SUBMITTALS

10.1 Within 2 weeks of receiving the contract award, the Installing Contractor and/or Anchor Designer shall submit the following helical anchor design documentation:

10.1.1 Certification from the Anchor Designer that the proposed anchors meet the requirements of this specification.

10.1.2 Qualifications of the Installing Contractor and Anchor Designer per Section 9.

10.1.3 Product designations for helical lead and extension sections and all ancillary products to be supplied at each helical anchor location.

10.1.4 Individual anchor nominal loads, factors of safety, LRFD load and resistance factors and required ultimate torque-correlated capacities, where applicable.

10.1.5 Individual anchor loading requirements (if any).

10.1.6 Manufacturer's published allowable system capacities for the proposed anchor assemblies, including load transfer devices.

10.1.7 Calculated mechanical and theoretical geotechnical capacity of anchors.

10.1.8 Minimum anchor termination torque requirements.

10.1.9 Maximum estimated installation torque and allowable installation torque rating of anchor.

10.1.10 Minimum and/or maximum embedment lengths or other site specific embedment depth requirements as may be appropriate for the site soil profiles.

10.1.11 Inclination angle and location tolerance requirements.

10.1.12 Load test procedures and failure criteria, if applicable.

10.1.13 Copies of certified calibration reports for torque measuring equipment and load test measuring equipment to be used on the project. The calibrations shall have been performed within one year of the proposed helical anchor installation starting date or as recommended by the equipment manufacturer.

10.1.14 Provide proof of insurance coverage as stated in the general specifications and/or contract.

11 PLACEMENT REQUIREMENTS

11.1 Helical anchors shall be installed within 3 inches of the indicated plan location.

11.2 Helical anchor shaft alignment shall be within 2 degrees of the inclination angle shown on the Plans.

11.3 Top elevation of helical anchors shall be within 2 inches of the design vertical elevation.

12 ANCHOR INSTALLATION

12.1 Installing Contractor shall furnish and install all helical anchors per the project Plans and approved anchor design documentation. In the event of conflict between the project Plans and the approved anchor design documentation, the Installing Contractor shall not begin construction on any affected items until such conflict has been resolved.

12.2 The Installing Contractor shall conduct their construction operations in a manner to insure the safety of persons and property in the vicinity of the work. The Installing Contractor's personnel shall comply with safety procedures in accordance with OSHA standards and any established project safety plan.

12.3 The Owner shall request marking of underground utilities by an underground utility location service as required by law, and the Installing Contractor shall avoid contact with all marked underground facilities.

12.4 The portion of the construction site occupied by the Installing Contractor, including equipment and material stockpiles shall be kept reasonably clean and orderly.

12.5 Installation of helical anchors may be observed by representatives of the Owner for quality assurance purposes. The Installing Contractor shall give the Owner at least 24 hours' notice prior to the anchor installation operations.

12.6 The helical anchor installation technique shall be such that it is consistent with the geotechnical, logistical, environmental, and load carrying conditions of the project. The lead section shall be positioned at the appropriate site survey stake location as determined from the plan drawings.

12.7 The helical anchor sections shall be advanced into the soil in a continuous manner at a rate of rotation less than 25 revolutions per minute (rpm). Sufficient crowd shall be applied to advance the helical anchor sections at a rate approximately equal to the pitch of the helix plate per revolution. The rate of rotation and magnitude of down pressure shall be adjusted for different soil conditions and depths. Extension sections shall be provided to obtain the required minimum overall length and minimum torsional resistance as shown on the project Plans.

13 TERMINATION CRITERIA

13.1 The minimum final torsional resistance and/or any required anchor length and embedment depth criteria, as specified in the Pre-Construction Submittals, must be satisfied prior to terminating the anchor installation. In the event any helical anchor fails to meet these production quality control termination criteria, the following remedies may be suitable if authorized by the Owner:

13.1.1 If the installation fails to meet the minimum torsional resistance criterion at the minimum embedment length:

13.1.1.1 Continue the installation to greater depths until the torsional resistance criterion is met, provided that, if a maximum length constraint is applicable, continued installation does not exceed said maximum length constraint, or

13.1.1.2 Demonstrate acceptable anchor performance through anchor load testing, or

13.1.1.3 Replace the anchor with one having a different helix plate configuration. The replacement anchor must not exceed any applicable maximum embedment length criteria and either: (A) be embedded to a length that places the last helix plate at least three times its own diameter beyond the depth of the first helix plate of the replaced anchor and meet the minimum torsional resistance criterion; or (B) pass anchor load testing criteria.

13.1.2 If the torsional resistance during installation reaches the helical anchor's allowable torque rating prior to satisfaction of the minimum embedment length criterion:

13.1.2.1 Terminate the installation at the depth obtained, or

13.1.2.2 Replace the anchor with one having a shaft with a higher torsional strength rating. The replacement anchor must be installed to satisfy the minimum embedment length criterion. It must also be embedded to a length that places the last helix plate at least three times its own diameter beyond the depth of the first helix plate of the replaced anchor without exceeding any applicable maximum embedment length requirements and it must meet the minimum final torsional resistance criterion, or

13.1.2.3 Replace the anchor with one having a different helix plate configuration. The replacement anchor must be installed to satisfy the minimum embedment length criterion. It must also be embedded to a length that places the last helix plate at least three times its own diameter beyond the depth of the first helix plate of the replaced anchor without exceeding any applicable maximum embedment length requirements, and it must meet the minimum final torsional resistance criterion.

13.1.3 If the installation reaches a specified maximum embedment length without achieving the minimum torsional resistance criterion:

13.1.3.1 If allowed, remove and reinstall the anchor at a position at least three times the diameter of the largest helix plate away from the initial location. Original embedment length and torsional resistance criteria must be met. The anchor repositioning may require the installation of additional helical anchors with nominal loads adjusted for these spacing changes, or

13.1.3.2 Demonstrate acceptable anchor performance through anchor load testing, or

13.1.3.3 De-rate the load capacity of the helical anchor based on default or site specific torque correlation factors and install additional anchors as necessary, or

13.1.3.4 Replace the anchor with one having a different helix plate configuration. The replacement anchor must be installed to satisfy the minimum and/or maximum embedment length criterion and it must meet the minimum final torsional resistance criterion.

13.1.4 If a helical anchor fails to meet the acceptance criteria in an anchor load test:

13.1.4.1 Install the anchor to a greater depth and installation torque and re-test; provided that, if a maximum embedment length constraint is applicable, continued installation will not exceed said maximum length constraint, or

13.1.4.2 Replace the anchor with one having more and/or larger helix plates. The replacement anchor must be embedded to a length that places the last helix plate at least three times its own diameter beyond the depth of the first helix plate of the replaced anchor without exceeding any applicable maximum embedment length requirements. The replacement anchor must be re-tested, or,

13.1.4.3 De-rate the load capacity of the helical anchor based on the results of the load test and install additional anchors. Additional anchors must be installed at positions that are at least three times the diameter of the largest helix plate away from any other anchor locations.

13.1.5 If a helical anchor fails a production quality control criterion as described in this Section or for any reason other than described in this Section, any proposed remedy must be approved by the Owner prior to initiating its implementation at the project site.

14 INSTALLATION RECORD SUBMITTALS

14.1 The Installing Contractor shall provide the Owner copies of the individual helical anchor installation records within 24 hours after each installation is completed. Formal copies shall be submitted within 30 days following the completion of the helical anchor installation. These installation records shall include, but are not limited to, the following information:

14.1.1 Date and time of installation

14.1.2 Location of helical anchor and anchor identification number

14.1.3 Installed helical anchor model and configuration

14.1.4 Termination depth, anchor head depth, and length of installed anchor

14.1.5 Actual inclination of the anchor

14.1.6 Final torsional resistance

14.1.7 Calculated geotechnical capacity based on final torsional resistance

14.1.8 Comments pertaining to interruptions, obstructions, or other relevant information

15 FIELD TENSION LOAD TESTING

15.1 If field tension load testing is required, the Installing Contractor shall furnish all labor, equipment and pre-production helical anchors necessary to accomplish the testing as shown in the approved anchor design documentation. Installing Contractor shall apply the specified loads for the specified durations and record the specified data, for the specified number of anchors. No deviations from the test plan(s) will be allowed without explicit approval in writing from the Owner. Anchor testing shall be in general accordance with the ASTM D3689 quick test method and the following criteria:

15.1.1 Failure criteria shall be in accordance with AC358 and is when plunging occurs or when the net deflection exceeds 10% of the average helix plate diameter, whichever occurs first.

15.1.2 An alignment load equal to 5% of the anticipated failure load or maximum anticipated test load may be applied prior to the start of the test to take out slack in the load test frame.

15.1.3 Loading increments shall be performed at 5% of the anticipated failure load or maximum anticipated test load with a minimum hold time of 4 minutes at each increment.

15.1.4 Upon completion of the maximum test load hold increment, the anchor shall be unloaded in 5 to 10 even increments with minimum hold times of 4 minutes at each increment.

15.2 Installing Contractor shall provide the Owner copies of raw field test data within 24 hours after the completion of each load test. Formal test reports shall be submitted within 30 days following test completion. Formal test reports shall include the following information:

15.2.1 Name of project and Installing Contractor's representative(s) present during load testing.

15.2.2 Name of manufacturer's representative(s) present during load testing, if any.

15.2.3 Name of third party test agency and personnel present during load testing, if any.

15.2.4 Date, time, duration and type of the load test.

15.2.5 Unique test identifier and map showing the test anchor location.

15.2.6 Anchor model and installation information including shaft type, helix configuration, lead and extension section quantities and lengths, final anchor tip depth, installation date, total test anchor length and final termination torque.

15.2.7 Calibration records for applicable anchor installation and test equipment.

15.2.8 Tabulated test results including cumulative anchor head movement, loading increments and hold times.

15.2.9 Plots showing load versus anchor head movement for each loading/unloading interval.

16 CLEANUP

16.1 Within one week of completion of the work, the Installing Contractor shall remove any and all material, equipment, tools, debris or other items belonging to the Installing Contractor or used under the Installing Contractor's direction.

MODEL SPECIFICATION FOR HELICAL SOIL NAILS EARTH RETENTION APPLICATIONS

1 SCOPE

1.1 The work consists of designing, furnishing and installing helical soil nails and load transfer devices for helical soil nail walls or slopes according to the project Plans and these specifications.

1.2 The parties and contract terms referred to in this specification are as follows:

1.2.1 The Owner is the person or entity that owns the facility or will own the facility once it is completed. The Owner may have contractual agreements with, and be represented by, other parties such as engineers, architects or contractors that perform services under the direction of the Owner. Where Owner is used in this specification, it refers to the Owner or the Owner's contracted representatives separate from the Installing Contractor.

1.2.2 The Soil Nail Designer is the individual or firm generally hired by the Installing Contractor to design the helical soil nails.

1.2.3 The Installing Contractor installs and tests (if necessary) the helical soil nails, and possibly performs other tasks associated with the project.

1.2.4 The Plans refer to the contract documents; including but not limited to the drawings and specifications for the project.

1.3 Helical soil nail walls or slopes are built from the top down in existing ground. The work consists of the following items as shown on the Plans:

1.3.1 Excavating in staged lifts in accordance with federal, state or local safety guidelines.

1.3.2 Installing helical soil nails to the specified minimum length, orientation and minimum final termination torque.

1.3.3 Providing and placing drainage elements at the specified locations.

1.3.4 Placing wall or slope face steel reinforcement, attaching bearing plates and connection devices and applying shotcrete or other specified facing over the reinforcement.

1.4 The work may include helical soil nail load testing.

1.5 The Owner will be responsible for obtaining right-of-way or easement access permits necessary for the helical soil nail installation.

1.6 Unless otherwise noted, the Installing Contractor shall provide all labor, tools, equipment and materials necessary to accomplish the work.

1.7 The Owner will provide suitable access to the construction site for the Installing Contractor's personnel and equipment.

1.8 Unless specifically noted otherwise in the contract documents, the Owner will remove and replace any structures, utilities, pavements, landscaping or other surficial improvements in the work area as necessary to facilitate the work.

1.9 The Owner will be responsible for overall construction oversight to preclude the development of unsafe conditions.

1.10 The Owner will be responsible for a horizontal field survey of the helical soil nail locations prior to helical soil nail installation and an elevation survey to determine soil nail shaft cutoff height subsequent to helical soil nail installation.

1.11 The work does not include any post-construction monitoring of soil nail performance unless specifically noted otherwise in the contract documents.

2 REFERENCES

2.1 American Institute of Steel Construction (AISC)

2.1.1 AISC 360: Specification for Structural Steel Buildings

2.2 American Society for Testing and Materials (ASTM)

2.2.1 ASTM A29: Steel Bars, Carbon and Alloy, Hot-Wrought

2.2.2 ASTM A36: Carbon Structural Steel

2.2.3 ASTM A123: Zinc Coating (Hot-Dip) on Iron and Steel Products

2.2.4 ASTM A153: Zinc Coating (Hot-Dip) on Iron and Steel Hardware

2.2.5 ASTM A185: Standard Specification for Steel Welded Wire Reinforcement, Plain, for Concrete

2.2.6 ASTM A572: High-Strength Low-Alloy Columbian-Vanadium Structural Steel

2.2.7 ASTM A615: Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement

2.2.8 ASTM B633: Electrodeposited Coatings of Zinc on Iron and Steel

2.2.9 ASTM B695: Coatings of Zinc Mechanically Deposited on Iron and Steel

2.2.10 ASTM D1785: Poly (Vinyl Chloride) (PVC) Plastic Pipe, Schedules 40, 80, and 120

2.3 Federal Highway Administration (FHWA)

2.3.1 FHWA Geotechnical Engineering Circular No. 7, “Soil Nail Walls”

2.4 International Code Council Evaluation Services (ICC-ES)

2.4.1 Acceptance Criteria 358 (AC358): Acceptance Criteria for Helical Pile Systems and Devices

2.5 Society of Automotive Engineers (SAE)

2.5.1 SAE J429: Mechanical and Material Requirements for Externally Threaded Fasteners

3 DEFINITIONS

3.1 The following terms apply to helical soil nails:

3.1.1 Allowable Stress Design: A structural and geotechnical design methodology that states that the summation of the actual estimated loads (nominal loads) must be less than or equal to the allowable design load (required strength). Allowable loads are obtained by dividing a nominal resistance (strength) by an appropriate factor of safety.

3.1.2 Bearing Stratum: The soil layer (or layers) that provides helical soil nail end bearing capacity through load transfer from the helical plates.

3.1.3 Crowd: Axial compressive force applied to the helical soil nail shaft as needed during installation to ensure the soil nail advances at a rate approximately equal to the helix pitch for each revolution.

3.1.4 Design Loads: A generic and ambiguous term used to describe any load used in design. It is not specific to factored or unfactored loads or any particular design methodology. It is a term; therefore, that should be avoided when specifying load requirements. FSI recommends using the term service load, nominal load or factored load, as described herein, where applicable.

3.1.5 Design Strength: A term used in structural design which is defined as the product of the nominal strength and the applicable resistance factor. An equivalent term typically used in geotechnical design is, also sometimes referred to as factored resistance (Load and Resistance Factor Design).

3.1.6 Extension Section: Helical soil nail shaft sections connected to the lead section or other extension sections to advance the helix plates to the required bearing strata and nail length. Helical soil nail extension have helix plates.

3.1.7 Factor of Safety: The ratio of the ultimate soil nail capacity or nominal resistance (strength) to the nominal or service load used in the design of any helical soil nail component or interface (Allowable Stress Design).

3.1.8 Factored Load: The product of a nominal load and an applicable load factor (Load and Resistance Factor Design).

3.1.9 Factored Resistance: The product of a nominal resistance and an applicable resistance factor (Load and Resistance and Factor Design).

3.1.10 Geotechnical Capacity: The maximum load or the load at a specified limit state, that can be resisted through the soil nails interaction with the bearing soils (see also Ultimate Soil Nail Capacity).

3.1.11 Helical Soil Nail: Consists of a central steel shaft with multiple helix-shaped bearing plates and a load transfer device that allows attachment to wall or slope facing components. Helical soil nails are installed into the ground by application of torque and axial compressive force (“crowd”).

3.1.12 Helix (Helical) Plate: Generally round steel plate formed into a helical spiral and welded to the central steel shaft. When rotated in the ground, the helix shape provides thrust along the soil nail's longitudinal axis thus aiding in soil nail installation. The plate transfers axial load to the soil through bearing.

3.1.13 Helix Pitch: The distance measured along the axis of the shaft between the leading and trailing edges of the helix plate.

3.1.14 Lead Section: The first helical soil nail shaft component installed into the soil. It consists of multiple helical plates welded to a central steel shaft.

3.1.15 Limit State: A condition beyond which a helical soil nail component or interface becomes unfit for service and is judged to no longer be useful for its intended function (serviceability limit state) or to be unsafe (ultimate limit state (strength)).

3.1.16 Load and Resistance Factor Design: A structural and geotechnical design methodology that states that the Factored Resistance (Design Strength) must be greater than or equal to the summation of the applied factored loads.

3.1.17 Load Factor: A factor that accounts for the probability of deviation of the actual load from the predicted nominal load due to variability of material properties, workmanship, type of failure and uncertainty in the prediction of the load (Load and Resistance Factor Design).

3.1.18 Load Test: A process to test the ultimate soil nail capacity and relation of applied load to soil nail head movement by application of a known load on the helical soil nail head and monitoring movement over a specific time period.

3.1.19 Loads: Forces that result from the weight of all building materials, occupants and their possessions, environmental effects, differential movement, and restrained dimensional changes. Permanent loads are those loads in which variations over time are rare or of small magnitude. All other loads are variable loads (see also Nominal Load).

3.1.20 Mechanical Strength: The maximum load or the load at a specified limit state that can be resisted by the structural elements of a helical soil nail.

3.1.21 Net Deflection: The total deflection at the helical soil nail head minus the theoretical elastic deformation of the soil nail shaft during a load test.

3.1.22 Nominal Loads: The magnitude of the loads specified, which include dead, live, soil, wind, snow, rain, flood and earthquakes (also referred to as service loads or working loads).

3.1.23 Nominal Resistance: The soil nail capacity at a specified ultimate limit state (Load and Resistance Factor Design). See Ultimate Soil Nail Capacity.

3.1.24 Nominal Strength: A term used in structural design which is defined as the structure or member capacity at a specified strength limit state. See Ultimate Soil Nail Capacity.

3.1.25 Resistance Factor: A factor that accounts for the probability of deviation of the actual resistance (strength) from the predicted nominal resistance (strength) due to variability of material properties, workmanship, type of failure and uncertainties in the analysis (Load and Resistance Factor Design).

3.1.26 Service Loads: See "Nominal Loads" above.

3.1.27 Ultimate Soil Nail Capacity: The helical soil nail capacity based on the least capacity determined from applicable ultimate limit states for mechanical and geotechnical capacity.

4 APPROVED HELICAL SOIL NAIL MANUFACTURERS

4.1 Foundation Supportworks®, Inc., 12330 Cary Circle, Omaha, NE 68128; Phone: (800) 281-8545; Fax: (402) 393-4002.

4.2 Due to the special requirements for design and manufacturing of helical soil nails, the soil nails shall be obtained from Foundation Supportworks®, Inc., or other qualified manufacturer with an approved equivalent product. A request to substitute any other manufactured helical product must be submitted to the Owner for review not less than seven (7) calendar days prior to the bid date. The request must include:

4.2.1 Documentation of at least five years of production experience manufacturing helical soil nails,

4.2.2 Documentation that the manufacturer's helical soil nails have been used successfully in at least three engineered construction projects within the last three years,

4.2.3 Product acceptance by the local building code official(s) having jurisdiction over the project, and/or

4.2.4 Current ICC-ES product evaluation report or complete description of product testing and manufacturing quality assurance programs used to assess and maintain product quality and determine product mechanical strength and geotechnical capacity.

5 ACCEPTABLE PRODUCTS

5.1 Solid Square Shaft Helical Soil Nail Models HS150 and HS175 manufactured in accordance with the requirements of Sections 5 and 6 of this specification.

5.1.1 Helix plates shall meet the following geometry and spacing criteria to minimize soil disturbance:

5.1.1.1 True helix-shaped plates that are normal to the shaft such that the leading and trailing edges are within ¼ inch of parallel.

5.1.1.2 Helix pitch is 3 inches ± ¼ inch.

5.1.1.3 All helix plates have the same pitch.

5.1.1.4 Helix plates have generally circular edge geometry.

5.1.1.5 Helix spacing along the shaft shall be between 2.4 and 3.6 times the helix diameter.

5.1.1.6 Helix plates are arranged along the shaft such that they all theoretically track the proceeding plate.

5.2 Wall Reinforcement, Anchorage, Bearing Plates, Nuts and Washers.

5.2.1 The wall anchorage shall consist of a bearing plate, or other fabricated bearing device connected to the helical soil nail with threaded rod adaptors.

5.2.2 A spherical seat nut or beveled washer and nut may be required at the connection of the helical soil nail to the bearing device to accommodate soil nail inclination per manufacturer recommendations.

5.2.3 Bearing devices shall be fabricated from steel conforming to ASTM A36 or A572 specifications, or equivalent.

5.2.4 Welded wire fabric shall conform to ASTM A185 or equivalent.

5.2.5 Reinforcing steel shall conform to ASTM A615, Grade 420 deformed.

5.3 Drainage Material

5.3.1 Vertical Wall Drains: Provide prefabricated, fully wrapped preformed geocomposite drains as required and shown on the Plans. The drainage core shall be either a preformed grid of embossed plastic or a system of plastic pillars and interconnections forming a semi-rigid mat, not less than 0.25-inch or more than 0.50-inch thick. The core material, when covered with filter fabric, shall be capable of maintaining a drainage void for the entire length of the permeable liner. Preformed drains shall be no wider than 12 inches unless special methods are used to ensure adherence of the shotcrete to the fabric and to preclude the fabric from sagging under the weight of the shotcrete. They shall be suitably outletted or connected to a longitudinal drain at the base of the wall. When splicing of drains is required, full flow through the splice shall be maintained and splices shall be suitably protected from damage and contamination during subsequent shotcreting. The shotcrete shall be the full design thickness over the drain.

5.3.2 Horizontal Drains: Provide as required and shown on the Plans, slotted and unslotted PVC pipe conforming to ASTM D1785 or equal. When horizontal drains are installed in bored holes, the Installing Contractor shall make provisions to assure that the drain-hole annulus does not collapse prior to the insertion of the slotted drain. Only the front 12 inches of drain pipe shall be unslotted.

5.4 Shotcrete Wall Facing

5.4.1 The Installing Contractor shall submit for approval by the Owner, materials, methods and control procedures for this work. Shotcrete design shall be in accordance with the shotcrete specifications in FHWA Geotechnical Engineering Circular No. 7, “Soil Nail Walls”, except as otherwise specified on the Plans. If facing material other than shotcrete is specified, the Installing Contractor shall submit for approval by the Owner, materials, methods and control procedures for this work.

5.5 Materials Handling and Storage

5.5.1 Store cement to prevent moisture degradation and partial hydration. Do not use cement that has become caked or lumpy. Store aggregates so that segregation and inclusion of foreign materials are prevented. Store un-galvanized helical soil nails on supports to keep the steel from contacting the ground. Light rust that has not resulted in pitting is acceptable for temporary applications.

6 HELICAL SOIL NAIL MATERIALS

6.1 Model HS150 Helical Soil Nail System

6.1.1 Central Steel Shaft: The central steel shaft of the lead and extension sections are 1.50-inch, solid, round-corner square (RCS) hot-rolled steel bars conforming to ASTM A29 with a minimum yield strength of 90 ksi and a minimum tensile strength of 115 ksi. The shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.1.2 Shaft Coupling Material: The extension shaft sections have an internally forged upset socket coupling at one end. Since the socket coupling is internally forged from the parent shaft material, the material properties of the coupling are similar to the central steel shaft. The shaft coupling finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.1.3 Helix Plate Material: The helix plates are factory welded to the lead or extension shaft sections. The helix plates have outer diameters of either 6 or 8 inches and are manufactured from 0.375-inch thick, ASTM A572 Grade 50 steel with a minimum yield strength of 50 ksi and a minimum tensile strength of 65 ksi. The helix plate finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.1.4 Shaft Coupling Hardware: The lead and extension shaft sections are coupled with one (1) bolt and nut per coupled shaft section. The coupling hardware consists of 0.750-inch standard hex bolts conforming to SAE J429 Grade 8 and jam nuts. The bolts and nuts are mechanically galvanized in accordance with ASTM B695.

6.1.5 Thread Rod Adapter: Thread rod adaptor HA150TRA is suitable for use with the HS150 shaft. Thread rod adapter finishes are either plain steel or hot-dip galvanized in accordance with ASTM A123. Adapter shaft coupling hardware is mechanically galvanized in accordance with ASTM B695.

6.2 Model HS175 Helical Soil Nail System

6.2.1 Central Steel Shaft: The central steel shaft of the lead and extension sections are 1.75-inch, solid, round-corner square (RCS) hot-rolled steel bars conforming to ASTM A29 with a minimum yield strength of 90 ksi and a minimum tensile strength of 115 ksi. The shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.2.2 Shaft Coupling Material: The extension shaft sections have an internally forged upset socket coupling at one end. Since the socket coupling is internally forged from the parent shaft material, the material properties of the coupling are similar to the central steel shaft. The shaft coupling finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.2.3 Helix Plate Material: The helix plates are factory welded to the lead or extension shaft sections. The helix plates have outer diameters of either 6 or 8 inches and are manufactured from 0.375-inch thick, ASTM A572 Grade 50 steel with a minimum yield strength of 50 ksi and a minimum tensile strength of 65 ksi. The helix plate finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

6.2.4 Shaft Coupling Hardware: The lead and extension shaft sections are coupled with two (2) bolts and nuts per coupled shaft section. The coupling hardware consists of 0.750-inch standard hex bolts conforming to SAE J429 Grade 8 and jam nuts. The bolts and nuts are mechanically galvanized in accordance with ASTM B695.

6.2.5 Thread Rod Adapter: Thread rod adaptor HA175TRA is suitable for use with the HS175 shaft. Thread rod adapter finishes are either plain steel or hot-dip galvanized in accordance with ASTM A123. Adapter shaft coupling hardware is mechanically galvanized in accordance with ASTM B695.

7 DESIGN AND PERFORMANCE REQUIREMENTS

7.1 Helical soil nails shall be designed to support the specified load(s) as shown on the project Plans. The overall length, helix configuration and minimum torsional resistance of a helical soil nail shall be such that the required geotechnical capacity is developed by the helix plate(s) in an appropriate bearing stratum.

7.2 All structural steel soil nail components shall be designed within the limits provided by the American Institute of Steel Construction (AISC) Specification for Structural Steel Buildings (AISC-360). Either Allowable Stress Design (ASD) or Load and Resistance Factor Design (LRFD) are acceptable methods of analysis. Product testing in accordance with ICC-ES Acceptance Criteria 358 may also be considered as an acceptable means of establishing system capacities.

7.3 Design of helical soil nail structures for excavation support, earth retention, slope stabilization, or other applications shall consider the following (at a minimum):

7.3.1 Global and internal stability of the system

7.3.2 Surcharge loads from adjacent structures or other loading that will be present during and/or after construction.

7.3.3 Helical soil nail torque correlated bearing capacity (bond strength) & load transfer

7.3.4 Structural capacity of individual helical soil nail components

7.3.5 Construction sequencing

7.3.6 Drainage conditions

7.3.7 Serviceability

7.4 The design of helical soil nail walls shall be in accordance with the FHWA Geotechnical Engineering Circular No. 7, “Soil Nail Walls”.

7.5 Except where noted otherwise on the project Plans, all helical soil nails shall be installed to provide an ultimate torque-correlated capacity based on an ASD or LRFD analysis. For ASD, a minimum factor of safety of 2 for internal stability of the wall and slope stability at the elevation of the toe of the wall shall be used for temporary or permanent applications. Lower factors of safety may be considered if approved by the Owner. When an LRFD analysis is required, the Owner shall provide applicable soil nail design information including but not limited to; factored loads, resistance factors and/or the required ultimate soil nail capacity. Factors of safety (ASD) or resistance factors (LRFD) may require modification to meet specific deflection criteria stated on the Plans or drawings.

7.6 The required ultimate torque-correlated capacity shall be verified at each soil nail location by monitoring and recording final installation torque and applying default torque correlations per ICC-ES AC358. Site specific torque correlation factors may be determined by field load testing as specified in Section 14.

7.7 Except where noted otherwise on the project Plans, each soil nail shall be designed to meet a corrosion service life of 50 years in accordance with ICC-ES AC358.

7.8 The soil nail design shall take into account group efficiency from soil nail spacing, soil stratification, and strain compatibility issues.

8 QUALIFICATIONS OF INSTALLING CONTRACTOR AND DESIGNER

8.1 The Installing Contractor and/or Soil Nail Designer shall submit to the Owner, a proposal including the documentation required in this Section. Work shall not begin until all the submittals have been received and approved by the Owner. All costs associated with incomplete or unacceptable submittals shall be the responsibility of the Installing Contractor.

8.2 Evidence of Installing Contractor's competence in the installation of helical soil nails shall be provided to the Owner's satisfaction and may include any or all of the following:

8.2.1 Helical soil nail manufacturer's certificate of competency in installation of helical soil nails,

8.2.2 A list of at least three projects completed within the previous three years wherein the Installing Contractor installed helical soil nails and/or helical tieback anchors in comparable soil conditions for the nominal loads similar to those shown on the project Plans. Such list to include names and phone numbers of those project representatives who can verify the Installing Contractor's participation in those projects, and/or

8.2.3 A letter from the helical soil nail manufacturer or manufacturer's representative expressing ability and intent to provide on-site supervision of the soil nail installation.

8.3 A listing of all safety violations lodged against the Installing Contractor within the previous three years and the current status or final resolutions thereof. Descriptions of safety improvements instituted within the previous three years may also be submitted, at the Installing Contractor's discretion.

8.4 Evidence of Soil Nail Designer's competence shall be provided to the Owner's satisfaction and may include any or all of the following:

8.4.1 Registration as a Professional Engineer or recognition by the local jurisdictional authority,

8.4.2 A list of at least three projects completed within the previous three years wherein the Soil Nail Designer designed helical soil nails similar to those shown in the project Plans. Such list to include names and phone numbers of those project representatives who can verify the Soil Nail Designer's participation in those projects, and/or

8.4.3 Recommendation from the soil nail manufacturer or manufacturer's representative.

9 PRE-CONSTRUCTION SUBMITTALS

9.1 Within 2 weeks of receiving the contract award, the Installing Contractor and/or helical Soil Nail Designer shall submit the following helical soil nail design documentation:

9.1.1 Certification from the Soil Nail Designer that the proposed soil nails meet the requirements of this specification.

9.1.2 Qualifications of the Installing Contractor and Soil Nail Designer per Section 8.

9.1.3 Product designations for helical soil nail lead and extension sections and all ancillary products to be supplied at each helical soil nail location.

9.1.4 Individual soil nail nominal loads, factors of safety, LRFD load and resistance factors and required ultimate torque-correlated capacities, where applicable.

9.1.5 Individual soil nail loading or post-tensioning requirements (if any).

9.1.6 Manufacturer's published allowable system capacities for the proposed soil nail assemblies, including load transfer devices, if any.

9.1.7 Calculated mechanical and theoretical geotechnical capacities including unit bond stress of the proposed soil nails.

9.1.8 Minimum helical soil nail termination torque requirements.

9.1.9 Maximum estimated installation torque and allowable installation torque rating of soil nail.

9.1.10 Minimum and/or maximum embedment lengths or other site specific embedment length or depth requirements as may be appropriate for the site soil profiles.

9.1.11 Inclination angle and location tolerance requirements.

9.1.12 Load test procedures and failure criteria, if applicable.

9.1.13 Copies of certified calibration reports for torque measuring equipment and load test measuring equipment to be used on the project. The calibrations shall have been performed within one year of the proposed helical soil nail installation starting date or as recommended by the equipment manufacturer.

9.1.14 Provide proof of insurance coverage as stated in the general specifications and/or contract.

10 PLACEMENT REQUIREMENTS

10.1 Helical soil nails shall be installed within 3 inches of the indicated plan location.

10.2 Helical soil nail shaft alignment shall be within 2 degrees of the inclination angle shown on the Plans.

10.3 Soil nail wall bearing plate edge distance from face of wall shall be within 1 inch of the design placement as shown on the Plans.

11 EXECUTION

11.1 Installing Contractor shall furnish and install all helical soil nails per the project Plans and approved soil nail design documentation. In the event of conflict between the project Plans and the approved soil nail design documentation, the Installing Contractor shall not begin construction on any affected items until such conflict has been resolved.

11.2 The Installing Contractor shall conduct their construction operations in a manner to insure the safety of persons and property in the vicinity of the work. The Installing Contractor's personnel shall comply with safety procedures in accordance with OSHA standards and any established project safety plan.

11.3 The Owner shall request marking of underground utilities by an underground utility location service as required by law, and the Installing Contractor shall avoid contact with all marked underground facilities.

11.4 The portion of the construction site occupied by the Installing Contractor, his equipment and his material stockpiles shall be kept reasonably clean and orderly.

11.5 Installation of helical soil nails may be observed by the Owner for quality assurance purposes. The installing contractor shall give the Owner at least 24 hours' notice prior to the soil nail installation operations.

11.6 Construction Procedure

11.6.1 The wall or slope is to be constructed from the top down as the soil in front of the wall is removed to specified depths and the helical soil nails are installed at each level.

11.6.2 The exposed soil face shall be retained with a construction facing consisting of mesh reinforced shotcrete or other appropriate facing material. Drainage systems, when required, shall be installed prior to applying shotcrete or other facing material.

11.7 Site Drainage Control

11.7.1 Provide positive control and discharge of all surface water that will affect construction of the helical soil nail retaining wall or slope. Maintain all pipes and conduits used to control surface water during construction. Surface water drainage network shall be independent of the wall drainage network.

11.7.2 Contact the Owner if unanticipated subsurface drainage structures are discovered during excavation.

11.8 Excavation

11.8.1 Excavation shall proceed in stages or lifts according to the Plans, exposing a minimum amount of soil face which will still allow for the installation of the helical soil nails and wall facing system while assuring stability of the excavated face.

11.8.2 The Owner shall be responsible for providing the necessary survey and alignment control during excavation of each lift and for performing the excavation in a manner which will allow for construction of the wall facing to the specified minimum thickness and to the line and grade indicated on the Plans. The Installing Contractor shall be responsible for locating and installing the helical soil nails within the allowable tolerances in this specification or on the Plans, and for performing the excavation and nail installation in a manner which will allow for constructing the wall facing to the specified minimum thickness and to the line and grade indicated on the Plans.

11.8.3 The Owner shall perform the excavation for the soil nail wall under the direction of the Installing Contractor. Care shall be taken to excavate to the final wall face using procedures that: (1) prevent over-excavation; (2) prevent ground loss, swelling, air slaking, or loosening; (3) prevent loss of support for completed portions of the wall; (4) prevent loss of soil moisture at the face; and (5) prevent ground freezing.

11.8.4 The exposed unsupported final excavation face cut height shall not exceed the vertical nail spacing plus the required reinforcing lap or the short-term stand-up height of the ground, whichever is less. Excavation to the next level shall not proceed until helical soil nail installation, reinforced shotcrete or other wall or slope facing placement, attachment of bearing plates and connection means and nail testing has been completed and accepted in the current lift. Shotcrete shall have cured for at least 72 hours or attained at least its specified 3-day compressive strength before excavating the next lift.

11.9 Helical Soil Nail Installation

11.9.1 Installing Contractor shall determine the installation method necessary to achieve the helical soil nail pullout resistance specified herein or on the Plans, in accordance with the helical soil nail acceptance criteria in the Helical Soil Nail Testing section. Install sacrificial helical soil nails for verification testing using the same equipment, methods, nail inclination and soil nail models as planned for the production soil nails.

11.9.2 No installation of production helical soil nails will be permitted until successful pre-production verification testing of nails is completed and approved by the Owner. The number and location of the verification tests will be as indicated on the Plans or specified herein.

11.9.3 During installation of the helical soil nails, the torque required to install each soil nail shall be monitored and recorded. The installation records shall include the following information:

11.9.3.1 Date and time of installation

11.9.3.2 Gear motor make and model

11.9.3.3 Location of helical soil nail and nail identification number

11.9.3.4 Actual helical soil nail model and configuration

11.9.3.5 Quantity and length of lead and extension sections installed

11.9.3.6 Installation torque log taken in minimum 5-foot increments of the total soil nail length and the final installation torque

11.9.3.7 Actual inclination of the soil nail

11.9.3.8 Comments pertaining to interruptions, obstructions, or other relevant information

11.9.4 When the final installation torque readings at the planned helical soil nail lengths do not achieve the minimum final torque requirements per the design or Plans, the Owner shall be notified to make recommendations for adding soil nails or extending the non-conforming soil nail to greater lengths.

11.9.5 Helical soil nail installation shall be terminated when the planned length is achieved as long as the final installation torque requirements per the design are met. When the maximum torque rating of the helical soil nail shaft is achieved prior to reaching the design length, the Soil Nail Designer shall be notified to make design recommendations to the Owner.

11.9.6 The installation of the helical soil nails shall be made at the locations, inclinations, and lengths shown on the Plans or as directed by the Owner. The installation techniques and equipment used shall be such that it is consistent with the geotechnical, logistical, environmental, and load carrying conditions of the project.

11.9.7 The helical soil nail sections shall be engaged and advanced into the soil in a smooth, continuous manner at a rate of rotation less than 25 rpm. Sufficient crowd shall be applied to advance the helical soil nail sections at a rate approximately equal to the pitch of the helix plate per revolution. The rate of rotation and magnitude of crowd shall be adjusted for different soil conditions and depths. Extension sections shall be provided to obtain the required minimum overall nail length.

11.9.8 Helical soil nails that encounter unanticipated obstructions during installation shall be relocated as approved by the Owner's representative.

12 INSTALLATION RECORD SUBMITTALS

12.1 The Installing Contractor shall provide the Owner, or his authorized representative, copies of individual helical soil nail installation records within 24 hours after each soil nail installation is completed. Formal copies shall be submitted within 30 days following the completion of the helical soil nail installation. These installation records shall include the information listed in Section 11.9.3.

13 HELICAL SOIL NAIL TESTING

13.1 If helical soil nail testing is required, the Installing Contractor shall furnish all labor, equipment and pre-production helical soil nails necessary to accomplish the testing as shown in the approved soil nail design documentation. Installing Contractor shall apply the specified loads for the specified durations and record the specified data, for the specified number of soil nails. No deviations from the test plan(s) will be allowed without explicit approval in writing from the Owner. Testing of any helical soil nail shall not be performed until the shotcrete facing has cured for at least 72 hours or attained at least the specified 3-day compressive strength. Helical soil nails may be tested immediately after installation without a shotcrete facing as long as precautions to maintain face stability are made (i.e., temporary lagging, etc.).

13.2 The test equipment shall consist of:

13.2.1 A calibrated dial gauge capable of measuring to 0.001 inch shall be used to measure movement.

13.2.2 A hydraulic cylinder and gauge calibrated as a unit shall be used to apply the test load. The pressure gauge shall be graduated in 100 psi increments or less and used to measure the applied load.

13.2.3 A reaction frame shall be used to distribute the load to the wall or slope facing without causing excessive deformation to the testing equipment or cracking in the facing.

13.3 Apply and measure the test load with the hydraulic cylinder and pressure gauge. The stroke of the hydraulic cylinder shall have sufficient travel to allow the test to be done without resetting the equipment. Measure the soil nail head movement with the dial gauge. The dial gauge shall have sufficient travel to allow the test to be done without having to reset the gauge. Support the gauge independently from the hydraulic cylinder, wall, or reaction frame.

13.4 Pre-production verification testing of sacrificial test helical soil nails shall be performed prior to installation of production helical nails to verify the Installing Contractor's installation methods and nail pullout resistance. The verification testing shall consist of:

13.4.1 Install sacrificial test helical nails as per Helical Soil Nail Installation Section 11.9.

13.4.2 Perform a minimum of two verification tests for each significantly different soil strata within the bearing zones of the proposed helical soil nails.

13.4.3 Sacrificial helical test nails shall have lengths of shaft without helix plates (smooth shaft) in addition to the test nail with helix plates. The length of smooth shaft is to be located nearest to the wall or slope face and shall be at least three feet long. The quantity of helix plates along the test nail shaft shall be determined based on the diameter of the helix plates used and the soil nail shaft torsional rating such that the torque rating of the shaft and the helical nail structural capacity is not exceeded during installation and testing.

13.4.4 Isolate the test nail from the shotcrete facing and/or reaction frame used during testing.

13.4.5 Submit the proposed location and length of shaft sections with and without helix plates prior to testing to the Owner for review and approval in accordance with the Pre-Construction Submittals Section 9.

13.4.6 The Design Test Load (DTL) shall be determined by the Owner taking into consideration the required ultimate soil nail capacity in the test region, or the helical soil nail with the highest load as determined by the internal and external stability analysis.

13.4.7 Verification test nails shall be incrementally loaded to a maximum test load of 200% of the DTL in accordance with the following loading schedule. The helical soil nail head movements shall be recorded at each load increment.

VERIFICATION TEST LOADING SCHEDULE

Load	**Hold Time**
AL (0.05 DTL max.)	1 minute
0.25DL	10 minutes
0.50DL	10 minutes
0.75DL	10 minutes
1.00DL	10 minutes
1.25DL	10 minutes
1.50DL (Creep Test)	60 minutes
1.75DL	10 minutes
2.00DL	10 minutes

The alignment load (AL) should be the minimum load required to align the testing apparatus and should not exceed 5% of the DTL. Dial gages should be set to “zero” after the alignment load has been applied. Each load increment shall be held for at least 10 minutes. The verification test helical soil nail shall be monitored for creep at 1.50 DTL. Nail movements during the creep portion of the test shall be measured and recorded at 1 minute, 2, 3, 5, 6, 10, 20, 30, 50, and 60 minutes.

13.5 Proof testing of production helical soil nails shall be performed on three percent (1 in 33) or more of the production helical soil nails in each horizontal row or a minimum of 1 nail per row. The quantity and size of helix plates along the soil nail shaft shall be determined such that the torque strength rating and allowable helical nail structural capacity is not exceeded during installation and testing. The minimum length of the proof tested production helical soil nail is 7 feet. The following requirements for proof testing shall be followed:

13.5.1 The DTL shall be determined as shown in Section 14.4.6

13.5.2 Proof tests shall be performed by incrementally loading the helical soil nail to a maximum test load of 150% of the DTL in accordance with the following loading schedule. At load increments other than the maximum test load, the load shall be held long enough to obtain a stable reading. A stable reading is defined as less than 0.010 inches of movement between readings taken two minutes apart. The helical soil nail head movements shall be recorded at each load increment.

PROOF TEST LOADING SCHEDULE

Load	**Hold Time**
AL (0.05 DTL max.)	Until Stable
0.25DL	Until Stable
0.50DL	Until Stable
0.75DL	Until Stable
1.00DL	Until Stable
1.25DL	Until Stable
1.50DL (Max. Test Load)	See Below

The alignment load (AL) should be the minimum load required to align the testing apparatus and should not exceed 5% of the Design Test Load (DTL). The verification test helical soil nail shall be monitored for creep at 1.50 DTL. At the discretion of the Owner, either 10 minute or 60 minutes creep tests shall be performed at the maximum test load (1.5 DTL). The creep period shall start as soon as the maximum test load is applied, and the helical nail movement shall be measured and recorded at 1 minute, 2, 3, 5, 6, and 10 minutes. Where the helical nail movement between 1 minute and 10 minutes exceeds 0.08 inch, the maximum test load shall be maintained until the helical nail movement is less than 0.08 inch for any one log cycle increment (i.e., 2 minutes and 20 minutes, etc.).

13.6 A helical soil nail shall be considered acceptable when the following criteria are met:

13.6.1 For verification tests, a total creep movement of less than 0.08 inch per log cycle of time between the 6 and 60 minute readings, and the creep rate is linear or decreasing throughout the creep test load hold period.

13.6.2 For proof tests, a total creep movement of less than 0.08 inch is measured between the 1 and 10 minute readings, or a total creep movement of less than 0.08 inch is measured between the subsequent log cycles, and the creep rate is linear or decreasing throughout the creep test load hold period.

13.6.3 For verification and proof tests, a pullout failure does not occur at the maximum test load. Pullout failure is defined as the load which results in continued pullout (creep) movement of the test helical soil nail.

13.7 If the pre-production verification test nail does not meet the criterion in Section 13.6, it shall be rejected and the Installing Contractor shall propose alternative methods and install replacement verification test helical nails. Successful proof tested helical soil nails meeting the above test acceptance criteria may be used as production nails, provided the number of helix plates on the test nail is such that the helical nail length is equal to or greater than the specified length. If the production proof test nail does not meet this criterion, it shall become sacrificial and shall be replaced with an additional production helical nail installed.

13.8 The Owner may require the Installing Contractor to replace some or all of the installed production soil nails between a failed proof test soil nail and the adjacent passing proof test nail. Or, the Owner may require the installation and testing of additional proof test helical nails to verify that adjacent previously installed production nails have sufficient load carrying capacity.

14 CLEANUP

14.1 Within one week of completion of the work, the Installing Contractor shall remove any and all material, equipment, tools, debris or other items belonging to the Installing Contractor or used under the Installing Contractor's direction.

Appendix 2G

Guidelines and Recommendations

GEOTECHNICAL INVESTIGATION GUIDELINES FOR HELICAL PILE, HELICAL ANCHOR AND PUSH PIER DESIGN

Design professionals rely on site-specific geotechnical investigations to provide soil strength parameters for use in foundation design. However, when these investigations do not properly identify a suitable bearing stratum, the project may be impacted with additional costs or delays until such adequate information is obtained. At the very least, contractors left to bid on a project with little to no usable soil information will do so conservatively. These bid proposals are then often filled with clauses outlining potential change order items; e.g., additional footage, revisions to the shaft section, revisions to the helix plate configuration, costs and responsibility due to failed load tests, etc. These extras often add up to many times the cost to simply complete deep soil borings and obtain the necessary soil information.

This document is not intended for incorporation into bid specifications, proposals or requests for proposals. Ultimately, it is the role of the Owner and his/her team of Design Professionals and Contractors to determine the scope of geotechnical investigation necessary based on considerations that may include structure size and type, structure design category, anticipated design loads, acceptable levels of risk and/or available funding.

This document is provided as a guide to assist Design Professionals, Contractors and Owners when helical piles, helical anchors, or push piers are planned. Due to differences in design, fabrication, and performance of seemingly equivalent products between manufacturers, these guidelines are intended exclusively for products designed and manufactured by Foundation Supportworks®, Inc. (FSI).

Hydraulically-Driven Push Piers

1. Push piers are advanced into the ground with hydraulic equipment exerting a steady but high downward force at the top of the pier. With most push pier systems using 4-inch O.D. shafts or smaller, this translates to high contact pressures with the soil. In order to provide competent bearing and to also minimize the potential for the push pier to punch through a seemingly competent but thin layer of soil, a minimum thickness of 10 to 15 feet of hard/dense material should be identified or be known to exist. This material may consist of hard clays, dense to very dense sands, or competent bedrock. The required thickness and strength of this layer would increase with an increase in the required pier capacity.
2. The Standard Penetration Test (SPT) completed in accordance with ASTM D1586 is a common method of retrieving disturbed soil samples in the field while also providing correlations to several soil strength parameters. The SPT is performed by driving a 2-inch O.D. split barrel sampler 18 inches with a 140 pound hammer falling a distance of 30 inches. The number of blows required to drive the sampler the final 12 inches is recorded as the standard penetration number, or N-value. Typically, N-values of 35 to 40 blows per foot for clay soils and 30 to 35 blows per foot for sand are required to provide end-bearing resistance for push piers. If the loading is extremely light or the piers are long enough to develop significant skin friction, lower end-bearing resistance may be required. However, these guidelines are good rules of thumb for most installations.
3. The standard penetration test provides a reasonable indication of strength and density of granular soils with correlations available to relate SPT N-values to relative density, unit weight and internal

friction angle. Laboratory direct shear tests or triaxial tests provide even more accurate estimates of soil strength which may be warranted for large projects. The additional cost of performing these tests could be offset by a more economical pier design that would not have been considered using SPT results alone.

4. SPT N-values may be inconsistent for fine-grained, cohesive soils and may not accurately reflect the soil shear strength. Tests may also be conducted on intact cohesive soil samples with pocket penetrometers. These results can vary widely between technicians depending upon the accuracy of the instrument and how closely the test procedure is followed. Laboratory testing of cohesive samples collected using undisturbed sampling methods, such as Shelby tube sampling (ASTM D1587), provides more reliable results. The more common methods for laboratory testing of undisturbed samples of cohesive soils are the unconfined compression test, the triaxial shear test and the direct shear test. Undrained shear strengths in excess of 4,000 pounds per square foot (psf) are typically required to provide end-bearing resistance for push piers.

5. The presence of very loose granular soil and very soft to soft cohesive soil should be identified. Column buckling may be a concern when very loose or soft soil is present in layer thicknesses of more than just a few feet, especially for a deep foundation element that derives its capacity primarily from end-bearing. Column buckling should be considered when SPT N-values are less than 4 blows/foot. When SPT sampling indicates weight-of-hammer (WOH) or weight-of-rod (WOR) values, additional laboratory testing for soil strength is recommended to document the shear strength in the WOH/WOR zone. Cone penetrometer tests (CPT) completed in accordance with ASTM D3441 may also be considered to measure in situ soil strengths. The CPT is widely used in lieu of, or as a supplement to, the SPT. The CPT is particularly suited for soft clays, soft silts and fine to medium sand deposits.

6. The presence of rubble fill, construction debris, or fill soils containing cobbles or boulders should be identified. Large, hard fractions within fill soils would likely stop advancement of the push pier. Pre-drilling may be required to allow the push pier to penetrate these soils and reach a suitable bearing stratum.

7. When project characteristics or site conditions warrant, the Owner and his/her team of Design Professionals may elect to determine the corrosive characteristics of the soils. Geotechnical or environmental consultants may classify soils as corrosive based on visual review of soil samples, from soil survey maps of the area, or from the results of additional testing. At a minimum, pH and resistivity testing are required for a corrosion analysis. Multi-directional field resistivity testing is preferred over laboratory resistivity testing. For a more complete corrosion analysis, chemical analyses may be completed to determine specific concentration levels. Corrosive characteristics of the soil should be determined from the ground surface to the bearing elevation of the deep foundation, if practical.

Helical Piles/Anchors

1. Helical piles and anchors are advanced into the ground by the application of torque and crowd. Crowd (or force) is applied longitudinally with the shaft to initiate penetration into the soil with the helix bearing plates. Less crowd is typically needed after the helix plates advance a few feet below the surface. Additional crowd may be needed in order to maintain the proper penetration rate of about 3 inches per revolution (for a helix plate with a 3-inch pitch) through stiffer/denser soil layers. Helical piles and anchors are best suited for medium dense sands and stiff to very

stiff clay soils, although they can be effectively designed for bearing in very dense sands and hard clay. With proper design and installation techniques, helical piles may also be considered for bearing on or within soft or weathered bedrock. A competent bearing stratum should be identified by the geotechnical investigation. Additional helix plates are often needed along the shaft as the required pile/anchor capacity increases or when bearing in lower-strength soils. Therefore, with higher pile/anchor capacities, the known or investigated thickness of the competent soil layer must also increase to fully embed all of the helix plates along the shaft. For compression piles, the geotechnical investigation should extend at least 5 to 10 feet below the anticipated termination depth of the pile. Soil strength parameters for the soils within that 5 to 10 feet should be equivalent to or greater than those strength parameters for the soils at the helix bearing depths.

2. The Standard Penetration Test (SPT) completed in accordance with ASTM D1586 is a common method of retrieving disturbed soil samples in the field while also providing correlations to several soil strength parameters. The SPT is performed by driving a 2-inch O.D. split barrel sampler 18 inches with a 140 pound hammer falling a distance of 30 inches. The number of blows required to drive the sampler the final 12 inches is recorded as the standard penetration number, or N-value. Typically, N-values of 15 to 30 blows per foot for clay soils and 10 to 25 blows per foot for sand are preferred for providing the necessary end-bearing resistance for helical piles or helical anchors. N-values higher or lower than those ranges may also be considered.

3. The standard penetration test provides a reasonable indication of strength and density of granular soils with correlations available to relate SPT N-values to relative density, unit weight and internal friction angle. Laboratory direct shear tests or triaxial tests provide even more accurate estimates of soil strength which may be warranted for large projects. The additional cost of performing these tests could be offset by a more economical pier design that would not have been considered using SPT results alone.

4. SPT N-values may be inconsistent for fine-grained, cohesive soils and may not accurately reflect the soil shear strength. Tests may also be conducted on intact cohesive soil samples with pocket penetrometers. These results can vary widely between technicians depending upon the accuracy of the instrument and how closely the test procedure is followed. Laboratory testing of cohesive samples collected using undisturbed sampling methods, such as Shelby tube sampling (ASTM D1587), provides more reliable results. The more common methods for laboratory testing of undisturbed samples of cohesive soils are the unconfined compression test, the triaxial shear test and the direct shear test. Undrained shear strengths ranging from 1,500 psf to 4,000 psf are preferred for use of helical piles or anchors, although higher or lower values may also be considered.

5. The presence of very loose granular soil and very soft to soft cohesive soil should be identified. Column buckling may be a concern for compression piles when very loose or soft soil is present in layer thicknesses of more than just a few feet, especially for a deep foundation element that derives its capacity primarily from end-bearing. Column buckling should be considered when SPT N-values are less than 4 blows/foot. When SPT sampling indicates weight-of-hammer (WOH) or weight-of-rod (WOR) values, additional laboratory testing for soil strength is recommended to document the shear strength in the WOH/WOR zone. Cone penetrometer tests (CPT) completed in accordance with ASTM D3441 may also be considered to measure in situ soil strengths. The CPT is widely used in lieu of, or as a supplement to, the SPT. The CPT is particularly suited for soft clays, soft silts and fine to medium sand deposits.

6. Helical bearing within soft or sensitive clays should be avoided due to long term settlement or creep effects. Pile/anchor capacities should be limited when helix plates bear within medium stiff clay or loose sands, and a higher factor of safety (FOS ≥ 3) may also be considered to reduce potential long term settlement or creep.

7. Groundwater levels should be accurately identified during the geotechnical investigation, particularly for sites with granular soils. The presence of groundwater above the anticipated bearing depths of the helix plates may significantly reduce the effective overburden stresses, thereby reducing pile capacities in granular soils. Groundwater table fluctuations should be considered for the pile/anchor design and installation. The highest potential groundwater elevation should be used for design of the helix plate configuration and the current groundwater elevation should be used for the determination of installation torque requirements with that given plate configuration.

8. The presence of rubble fill, construction debris, or fill soils containing cobbles or boulders should be identified. Large, hard fractions within fill soils would likely stop advancement of the helical pile or anchor. Pre-drilling or removal of the hard fractions may be required to allow helical piles/anchors to penetrate these soils and reach a suitable bearing stratum.

9. When project characteristics or site conditions warrant, the Owner and his/her team of Design Professionals may elect to determine the corrosive characteristics of the soils. Geotechnical or environmental consultants may classify soils as corrosive based on visual review of soil samples, from soil survey maps of the area, or from the results of additional testing. At a minimum, pH and resistivity testing are required for a corrosion analysis. Multi-directional field resistivity testing is preferred over laboratory resistivity testing. For a more complete corrosion analysis, chemical analyses may be completed to determine specific concentration levels. Corrosive characteristics of the soil should be determined from the ground surface to the bearing elevation of the deep foundation, if practical.

10. Soil strength parameters should be determined for the top 10 feet of the soil profile when helical piles are subjected to lateral loading. Continuous sampling or continuous in situ testing methods should be used within this zone in order to estimate the lateral load capacity of the pile. For critical projects, consideration could be given to special sampling and testing techniques such as pressuremeter testing; otherwise typical sampling and testing methods as described above may be suitable.

Alternative Methods of Developing Geotechnical Design Information

For many projects, an appropriate level of geotechnical information has not been obtained prior to initiating a preliminary design for helical piles and push piers. New construction helical piles are often an afterthought following a shallow geotechnical investigation and discovery of weak, near-surface soils. Helical piles are then listed in the geotechnical report or on the project plans as a deep foundation alternative, but additional or deeper test borings are rarely completed to adequately identify the soil strength parameters for the deeper bearing soils. In additional to a geotechnical investigation, there are other potential sources or methods for obtaining geotechnical information for a preliminary design.

1. Establish a relationship with a local geotechnical engineering firm. An established, local geotechnical firm may have performed a previous investigation within the area of the proposed project, or have knowledge of the general soil profile. A discussion with the geotechnical firm would allow the installing contractor to, at a minimum, determine if soil conditions are suitable for use of helical piles or push piers.
2. Contact local well drilling firms for any information regarding local ground/geologic conditions.
3. Contact the Geologic Survey for the applicable State jurisdiction. This agency often maintains records of test borings and wells throughout a given state, and this information may be used to support a preliminary design effort.
4. Conduct a test installation of a helical pile/anchor, also called a "helical test probe". A typical helical test probe consists of a Model HA150 or Model HA175 lead section with a single 10 or 12-inch helix plate and multiple extensions to reach the required depth. For some soil profiles, a double helix may be necessary to provide the thrust needed to penetrate stiff or dense strata or advance through particularly soft zones. For these situations, a double 8/10 or 10/12 helix configuration generally works well. Torque must be monitored in one foot intervals during installation of the test probe(s) from the ground surface to the termination depth. The torque readings must be taken with calibrated equipment such as a certified gear motor and calibrated pressure gages or by using a calibrated torque transducer in line with the drive tooling. The depth of investigation would be similar to that described above; i.e., 5 to 10 feet below the anticipated depth of the production piles. A general understanding of the soil profile and depth to groundwater may be required to back-calculate soil strength from a helical test probe. The proposed number of helical test probes for a given project is dependent upon the project characteristics and the variability of site soil conditions. Factors of safety greater than 2.0 must be considered when the helical pile/anchor design is based solely or in large part on the results of helical test probes. The following information is required when evaluating the results of helical test probes: make and model of the gear motor, calibrated torque readings, test probe shaft size, and test probe helix plate configuration. For push pier projects, the installation of a test push pier typically provides sufficient information for a push pier preliminary design.
5. Perform a load test. If site access allows the setup of a load test frame, the results from compression or tension load tests can determine helical pile or anchor capacity without soil information. The required number of load tests would again be dependent upon project characteristics and should be determined by design professionals. The helical piles or anchors used for the load test(s) shall be installed with calibrated equipment so the true installation torque is known. The load tests should be performed in general accordance with applicable ASTM standards. A site-specific torque correlation factor can be determined from the results of the load test(s), which would then allow the final installation torque readings to be used for pile capacity verification.

Helical piles, helical anchors and hydraulically-driven push piers are installed routinely on residential projects without adequate site-specific soil information available. The homeowner is often unwilling to pay for deep soil borings, laboratory tests and recommendations from a geotechnical consultant. In these cases, the installing contractor typically proposes a specific pile or anchor system with depth/length and product spacing based upon experience from doing previous work in the area. The contractor then follows rules-of-thumb guidelines for installation.

FSI offers a software tool called "Foundation View" to assist FSI installing contractors in estimating structural residential loads and spacing of retrofit piers. This program assumes that the soils are capable of providing adequate support to the piering system in order to achieve its full rated allowable capacity with a suitable factor of safety. The contractor should request that adequate soil information be provided, or have at least some general knowledge of the local and/or site-specific soil conditions to reduce potential risks associated with unknowns or unforeseen conditions.

DISCLAIMER

The information contained in this document is provided for Design Professionals, Contractors, Owners and Foundation Supportworks®, Inc. (FSI) Installing Contractors to assist in the application of FSI products. Copying or distributing all or part of this document for any other purpose, without the prior written consent of FSI, is expressly forbidden.

The guidelines provided in this document are not intended to become part of bid specifications, proposals, requests for proposals, or to override the requirements of appropriate national, state or local regulatory agencies or the recommendations of qualified design professionals. Due to the wide variation in building codes, regulations and rules that apply to construction between regions and countries, FSI should not be relied upon for and shall not be responsible for the development or approval of final design documents. Development and submittal of final design documents shall be completed by a Professional Engineer licensed in the state of the project.

PRELIMINARY DESIGN SERVICES

Foundation Supportworks®, Inc. (FSI) offers Preliminary Design Services for new construction and retrofit piering and anchoring projects at no cost to design professionals and FSI installing contractors. This service includes a review of available soil information, plans and specifications, and development of preliminary design recommendations for the load and deflection criteria specified. FSI commits time and resources for each project submitted. In order to make efficient use of time and potentially expedite a response, the Preliminary Design Request Form should be completely filled out each time a request is made for project assistance. The Preliminary Design Request Form can also be provided in an editable PDF format where the information can be typed into the appropriate fields and emailed to FSI along with the pertinent project documents. Contact the FSI engineering department for the electronic version of the request form.

The preliminary design recommendations provided by FSI are intended for use by design professionals and FSI installing contractors for estimating and bidding purposes only and should not be incorporated into project bid documents or specifications. Please see disclaimer language below which will in part or whole be included with preliminary design recommendations provided by FSI. Project specifications typically require that formal pile/pier design submittals be completed by an engineer licensed in the project's state or jurisdiction. FSI engineers cannot serve as the engineer of record by signing and stamping project-specific documents and details.

Preliminary Design Request Process:

1. Assemble all project information (all three items must be provided)
   a. Plans
   b. Specifications
   c. Geotechnical Report
2. Completely fill out the Preliminary Design Request Form
   a. Step 1: Select request recipient
   b. Step 2: Fill out your contact information
   c. Step 3: Fill out the required project information
   d. Step 4: Identify which documents are being provided and how they are being sent
3. Send the Preliminary Design Request Form and all required documents to FSI by
   a. Mail
   b. Fax
   c. Email (preferred)

All requests shall be addressed or copied to James Malone for project tracking purposes. Please allow up to five working days for a preliminary design or up to two working days for bid preparation assistance. A response may be delayed if critical information is missing from the request.

DISCLAIMER

(To Accompany FSI Preliminary Design Recommendations)

The information and preliminary design recommendations contained in this document are provided to design professionals and/or Foundation Supportworks®, Inc. ("FSI") dealers to assist in the application of FSI products. Copying or distributing all or part of this document for any other purpose, without the prior written consent of FSI, is expressly forbidden.

The preliminary design recommendations provided in this document are based on information collected from/by outside agencies or parties. FSI is not responsible for the completeness, accuracy or applicability of any such information. FSI DOES NOT WARRANT THE INFORMATION OR PRELIMINARY DESIGN

RECOMMENDATIONS CONTAINED IN THIS DOCUMENT AND SPECIFICALLY DISCLAIMS ALL WARRANTIES, INCLUDING WARRANTIES OF EXPRESS OR IMPLIED MERCHANTABILITY AND WARRANTIES OF EXPRESS OR IMPLIED FITNESS FOR A PARTICULAR PURPOSE. The preliminary design recommendations provided in this document are not intended to become part of contract bid documents or specifications or to override the requirements of appropriate national, state or local regulatory agencies or the recommendations of qualified design professionals. Due to the wide variation in building codes, regulations and rules that apply to construction between regions and countries, FSI should not be relied upon for and shall not be responsible for the development or approval of final design documents. Development and submittal of final design documents shall be completed by a Professional Engineer licensed in the state of the project.

The design and application of foundation support products (including helical and push pier systems manufactured by FSI) are dependent upon site conditions, including without limitation geotechnical properties of soil which can vary significantly between sampling points. If there is an identified change in site conditions or revision or addition to the information used for the basis of this preliminary design, FSI should be contacted to provide alternative recommendations based on such changing site conditions or information.

PRELIMINARY DESIGN REQUEST FORM

Step 1: Attention

☐ James Malone james.malone@foundationsupportworks.com 402.861.4773

☐ Don Deardorff don.deardorff@foundationsupportworks.com 402.861.4756

☐ Jake Blessen jake.blessen@foundationsupportworks.com 402.885.6608

☐ Jordan Larsen jordan.larsen@foundationsupportworks.com 402.916.4222

Step 2: Contact Information

Date of request: ______________________________

Dealer: ____________________ Contact name: ____________________

Phone: ______________ Cell: ______________ Email: ______________

Step 3: Project Information

Project Name: __________ City: __________ State: __________

Bid Date: ____________________ Date Needed By (allow up to 5 working days): __________

Project Engineer: ____________________ Phone: ____________________

Geotechnical Engineer: ____________________ Phone: ____________________

Product			Allowable/Working Loads (kips)			Factor of Safety
Type	Model #	Quantity	Compression	Tension	Lateral	
New Construction Helical Piles						
Retro Helical Piers						
Helical Tiebacks						
Soil Nails						
Push Piers						
Other						

Define scope of the project: ______________________________

____

____

____

Step 4: Attachments:

Plans or Design Drawings ☐ None ☐ Mail ☐ Email ☐ Fax

Soil Borings or Geotechnical Reports ☐ None ☐ Mail ☐ Email ☐ Fax

Project Specifications ☐ None ☐ Mail ☐ Email ☐ Fax

Photos (Provide drawing to show photo location and direction.) ☐ None ☐ Mail ☐ Email ☐ Fax

POLICY FOR AXIAL COMPRESSION OR AXIAL TENSION LOAD TESTS ON HELICAL PILES AND ANCHORS

Foundation Supportworks®, Inc. (FSI) is equipped and staffed to provide the FSI contractor network with the necessary equipment, support and assistance for completing axial compression and axial tension field load tests on helical piles and anchors. Given FSI's investment in time and resources to provide this level of support, we ask that all parties communicate, understand and be in agreement with the statements outlined in this document. This document is intended to set realistic expectations of FSI's involvement in such projects and to guide the FSI installing contractor, the general contractor, the design team, and the owner through the load test process. The design team member in charge of the load test is hereafter referred to as the "engineer of record". Ultimately, FSI's goal is to bring all possible positive results to fruition in the various steps of the load test process, while maintaining reasonable costs and minimizing frustrations. Contact an FSI representative should any questions arise or special circumstances require consideration.

FSI Field Staff Responsibility

1. FSI field staff may act in an advisory capacity for the installation of test piles, test anchors and reaction piles and also oversee setup of the load test frame and test equipment.
2. Upon request by the FSI installing contractor or engineer of record, FSI field staff will advise or train personnel in recording load and deflection readings during the load test, or otherwise assist in data collection.
3. FSI field staff does not interpret load test results nor do they offer opinions concerning pass/fail criteria. The load test information must be forwarded to FSI engineering for evaluation if interpretation has been requested.
4. FSI assumes no responsibility, economically or otherwise, should the load tests fail to achieve the desired results.
5. FSI is not responsible for site safety issues and shall not be held liable for any breach in site safety protocol. If FSI representatives feel site safety procedures are not followed or are inadequate, said representatives have the authority to immediately terminate the field load test oversight services.

Qualifications for FSI Engineering Involvement with Field Load Tests

1. FSI engineering may provide a preliminary design recommendation and/or a document review prior to the test pile/anchor installation. Project specifications, test loads, applicable failure criteria, and adequate subsurface information must be provided in order to develop the preliminary design recommendation.
2. FSI shall be provided an open line of communication with the project design team, including owners, architects, engineers, and contractors during the preliminary design phase and throughout the load test process.

3. FSI engineering shall have access to all documents and information relating to the load test and the load test results, including project specifications, test loads, failure criteria, subsurface information, installation field logs for all test piles/anchors and reaction piles, calibrations of test equipment, and field logs of load tests.
4. The engineer of record understands and considers opinions and recommendations by FSI engineering regarding options to proceed in the event of unsatisfactory load test results.

FSI Engineering Role and Limitations

1. FSI engineering acts in an advisory capacity only, whether in preliminary design, on site issues and adjustments, use of FSI products and equipment, or interpretation of load test data.
2. FSI engineering assumes no liability in the event that capacity is not achieved, piles/anchors require increased depth or length, or a revised helix plate configuration is needed.
3. FSI and FSI engineering reserve the right to refuse participation in any preliminary design, project review, or field load test operation at its discretion.

Review of Load Test Data

1. Upon request of the FSI installing contractor or the engineer of record, FSI engineering may offer opinions and recommendations regarding the results of the load test. The field installation logs for the test pile/anchor and reaction piles, the test pile/anchor deflection versus load measurements, and all other applicable information should be scanned and emailed to FSI engineering at the earliest opportunity to allow adequate time for review.
2. Please allow at least 24 hours per test from the time of submittal of the field logs for FSI engineering to evaluate the information and offer an interpretation and opinion of the load test results. If the information submitted is incomplete or illegible, additional time may be required for interpretation of data.

Suggested Installation Procedure for Test Piles/Anchors, Reaction Piles and Load Frame

1. If FSI engineering will be involved with interpretation of load test data, the following information shall be provided prior to installation of the test pile/anchor and reaction piles: make and model of installation equipment with ratings for hydraulic fluid pressure and flow rate; make and model of the drive head; method of torque monitoring with any back-up information and/or calibrations, expected ultimate pile capacity, and the load test failure criteria. Provide same information to engineer of record for review and approval.
2. In the event that no soil information or inadequate soil information is provided, it is recommended that helical test probes be installed on the proposed site to a depth at least 10 feet beyond the anticipated pile tip depth using calibrated torque monitoring equipment. See Geotechnical Investigation Guidelines in Appendix 2G for helical test probe installation procedures.
3. It is highly recommended that field installation torque/differential pressure recordings be logged in one-foot increments for the test pile/anchor and reaction piles. At a minimum, torque/differential pressure readings must be recorded on five-foot intervals up to the last ten feet of installation, where then one foot interval readings are required to the termination depth. Field installation logs

shall further note depths and magnitudes of high torque/pressure spikes, apparent encounters with obstructions, or any other difficulties in advancing the helical piles.

4. Test piles/anchors and reaction piles shall be installed as close to vertical and plumb as possible, unless battered or angled elements will be tested. Maintain installation speed under 25 rpm during installation of the test and reaction piles. Installation speed should be further reduced to 10 rpm or less for the final three to five feet of installation or when there is a risk of abruptly exceeding the torque rating of the helical shaft.

5. The test frame shall have an allowable structural capacity greater than the maximum test load. The test frame reaction piles shall have a combined ultimate soil capacity at least two times the anticipated maximum test load. The maximum test load shall be determined by multiplying the design working load by the appropriate factor of safety. Ultimate soil capacities for reaction piles are determined by multiplying the installation torque by the applicable default torque correlation.

6. The test frame shall be installed with the main beam centered over the test pile. The main beam and the reaction beams shall be as close to horizontal (level) as possible and the reaction piles shall be as close to vertical as possible.

7. Tests procedures should generally follow ASTM D1143 (Compression) and ASTM D3689 (Tension). Criteria for evaluating the ultimate capacity of the pile is provided in AC358, Acceptance Criteria for Helical Pile Systems and Devices, by the International Code Council Evaluation Service, Inc. (ICC-ES). Alternative failure criteria may be proposed by the engineer of record. Such criteria must be provided to FSI prior to installation of the test piles/anchors and reaction piles.

Load Test Communication and Responsibilities

1. A representative of the Engineer of Record or the Owner should be on site prior to initiation of the load test and should remain through completion of the test cycles.

2. When an extended or long term test is required, procedures and data recording should be detailed by the Engineer of Record.

3. The Engineer of Record, the Owner, or the Owner's Representative shall have the opportunity to physically read dial gauges and record measurements.

4. Engineer of record and FSI engineering shall be provided with a copy of the proposed test pile/anchor installation procedures and load test procedures for review and comment prior to the start of any site work.

FSI Travel and Expenses

1. FSI has a vested interest in the success of all FSI installing contractor load test operations and offers field assistance when necessary to support its contractor network. FSI typically requires the FSI installing contractor to pay travel costs and expenses for FSI representatives to travel to the project site to provide support. In special cases, FSI may choose to assume some or all of the travel costs and expenses for the FSI representative to be on site for the load test. In the event that the FSI installing contractor fails to be prepared to a reasonable degree and this contributes to extensive delays and extended or repeat trips, FSI reserves the right to pass unnecessary costs on to the FSI installing contractor. Proper communication and organization should prevent additional costs that are within control.

Appendix 2H

Documentation

ICC-ES Evaluation Report

ESR-3074

Reissued May 2014

This report is subject to renewal July 1, 2015.

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:

FOUNDATION SUPPORTWORKS, INC.
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OMAHA, NEBRASKA 68128
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www.foundationsupportworks.com
jeff.kortan@foundationsupportworks.com

EVALUATION SUBJECT:

FOUNDATION SUPPORTWORKS HELICAL FOUNDATION SYSTEMS

1.0 EVALUATION SCOPE

Compliance with the following codes:

2012, 2009 and 2006 *International Building Code®* (IBC)

Properties evaluated:

- Structural
- Geotechnical

2.0 USES

Foundation Supportworks, Inc. (FSI), Model HP288 Helical Foundation Systems are used either to underpin foundations of existing structures or to form deep foundations for new structures, and are designed to transfer axial compression and axial tension loads from the supported structures to suitable soil bearing strata.

3.0 DESCRIPTION

3.1 General:

The FSI Model HP288 helical foundation systems consist of a central lead shaft with one or more helical-shaped steel bearing plates, extension shafts, which may or may not consist of helical bearing plates, shaft couplings that connect multiple shaft sections, and a bracket that allows for attachment to the supported structure. The shafts with helix bearing plates are screwed into the ground by application of torsion and the shaft is extended until a desired depth and/or a suitable soil or bedrock bearing stratum is reached.

3.2 System Components:

The FSI Model HP288 helical foundation systems include a lead shaft (HP288L), extension shafts (HP288E), Type A side-load brackets (FS288B and FS288BL), and Type B direct-load brackets (HP288NCB and HP288NCB8), for attachment to concrete foundations.

3.2.1 Helical Lead Sections and Extensions: The FSI helical pile lead sections consist of one or more helical-shaped circular steel plates factory-welded to a central steel shaft. The depth of the helical piles in soil is typically extended by adding one or more steel shaft extensions that are mechanically connected together by couplings, to form one, continuous steel pile.

The central steel shaft of the lead and extension sections is a round, $2^7/_8$-inch-outside-diameter (73 mm), 0.276-inch-nominal-wall-thickness, hollow structural section. The various shaft lead and extension configurations are listed in Table 5.

Each helical steel bearing plate (helix) is 0.375 inch (9.5 mm) thick, and has a 3-inch (76 mm) pitch and spiral edge geometry with an outer diameter of 8, 10, 12 or 14 inches (203, 254, 305 or 356 mm). The helices are welded to the helical shaft. The lead helix is located about 4 inches from the tip of the shaft lead section. The extensions may consist of the shaft only or include helix plates.

The extension section couplings consist of a round, 6-inch-long (152.4 mm), $3^1/_2$-inch-outside-diameter (89 mm), 0.281-inch-nominal-wall-thickness, hollow structural section outer sleeve, and two $^3/_4$-inch-diameter (19.1 mm) standard hex threaded bolts and matching standard hex jam nuts. The pipe sleeve is factory-welded to the end of the extension section. (See Figure 3.)

3.2.2 Brackets: Brackets are constructed with factory-welded steel plate and steel pipe components. The different brackets are described in Sections 3.2.2.1 through 3.2.2.2.

3.2.2.1 Retrofit Bracket Assemblies FS288B and FS288BL: The FS288B and FS288BL bracket assemblies are designed for use with the HP288 helical shaft and are used to transfer axial compressive loading from existing concrete foundations to the HP288 helical piles. The bracket assembly consists of an FS288B or FS288BL bracket, an external pipe sleeve (FS288ES30 or FS288ES48), a cap plate (FS288C), two threaded rods and matching nuts. (See Figure 1.)

The FS288B and FS288BL brackets are constructed from factory-welded, 0.250-inch-, 0.375-inch- and 0.500-inch-thick (6.4 mm, 9.5 mm, and12.7 mm) steel plates.

The external sleeve (FS288ES30) is manufactured from a 30-inch-long (762 mm), $3^1/_2$-inch-outside-diameter (89 mm) and 0.216-inch-nominal-wall-thickness pipe with a factory-welded end ring which consists of a $^3/_4$-inch-long (19.1 mm), 4.0-inch-outside-diameter (102 mm) and 0.226-inch-nominal-wall-thickness pipe. The FS288ES48 external sleeve is identical to the FS288ES30 except that the FS288ES48 is 48 inches (1219 mm) long.

The FS288C cap plate assembly is manufactured from a $^{1}/_{2}$-inch-long (12.7 mm), $3^{1}/_{2}$-inch-outside-diameter (89 mm), 0.216-inch-nominal-wall-thickness steel pipe that is factory-welded to a 1-inch-thick (25.4 mm), 5-inch-wide (127 mm), 9-inch-long (229 mm) steel plate. The cap plate is attached to the retrofit bracket with two $^{3}/_{4}$-inch-diameter-by-16-inch-long (19.1 mm by 406 mm) threaded rods, and matching $^{3}/_{4}$-inch (19.1 mm) heavy hex nuts. (See Figure 1.)

3.2.2.2 New Construction Brackets HP288NCB and HP288NCB8: HP288NCB and HP288NCB8 brackets are designed for embedment in cast-in-place concrete foundations. The brackets are used to support axial tensile and compressive loads that are concentric with the longitudinal axis of the shaft. (See Figure 2.)

The HP288NCB bracket is manufactured from a 5.06-inch-long (128.5 mm), $3^{1}/_{2}$-inch-outside-diameter (89 mm), 0.250-inch-nominal-wall-thickness steel pipe sleeve which is factory-welded to a $^{3}/_{4}$-inch-thick (19.1 mm), 6-inch-square (152 mm) steel cap plate. The bracket is attached to the shaft with two $^{3}/_{4}$-inch-diameter (19.1 mm) standard hex threaded bolts and with matching $^{3}/_{4}$-inch (19.1 mm) standard hex jam nuts. (See Figure 2.)

The HP288NCB8 bracket is identical to the HP288NCB bracket except that the HP288NCB8 cap plate is an 8-inch-square (203 mm) steel plate. (See Figure 2.)

3.3 Material Specifications:

3.3.1 Lead and Extension Shafts: The leads and extensions are carbon steel round structural tubes that conform to ASTM A500, Grade B or C, having a minimum yield strength of 60 ksi (413 MPa) and a minimum tensile strength of 70 ksi (483 MPa). The shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

3.3.2 Shaft Coupling:

3.3.2.1 Pipe Sleeves: The sleeves are carbon steel round structural tubing that conforms to ASTM A513, Type 5, Drawn Over a Mandrel (DOM), Grade 1026, having a minimum yield strength of 70 ksi (483 MPa) and a minimum tensile strength of 80 ksi (552 MPa). The sleeve finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

3.3.2.2 Bolts and Nuts: The steel coupling bolts are $^{3}/_{4}$–10 UNC 2A standard hex bolts conforming to SAE J429, Grade 8, having a minimum yield strength of 130 ksi (896 MPa) and a minimum tensile strength of 150 ksi (1034 MPa). The matching steel nuts are $^{3}/_{4}$–10 UNC 2B standard hex jam nuts, conforming to SAE J995, Grade 5. The bolts and nuts are zinc-coated in accordance with ASTM B633, with coating classification Fe/Zn 8.

3.3.3 Helix Plates: The steel plates conform to ASTM A572, Grade 50, having a minimum yield strength of 50 ksi (345 MPa) and a minimum tensile strength of 65 ksi (448 MPa). The helix finish is the same as that of the shaft to which the helix is factory-welded.

3.3.4 Retrofit Bracket Assemblies FS288B and FS288BL:

3.3.4.1 FS288B and FS288BL Brackets: The steel plates used in the brackets conform to ASTM A36, having a minimum yield strength of 36 ksi (248 MPa) and a minimum tensile strength of 58 ksi (400 MPa). The bracket finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

3.3.4.2 FS288ES30 and FS288ES48 Sleeves: The carbon steel structural round tubing, used for the 30-inch-and 48-inch-long (762 mm and 1219 mm) sleeves, conforms to ASTM A500, Grade B or C, having a minimum yield strength of 50 ksi (345 MPa) and a minimum tensile strength of 62 ksi (427 MPa). The $^{3}/_{4}$-inch-long (19.1 mm) steel ring (collar) conforms to ASTM A53, Types E and S, Grade B, having a minimum yield strength of 35 ksi (241 MPa) and a minimum tensile strength of 60 ksi (413 MPa). The sleeve finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

3.3.4.3 FS288C Cap Plate Assembly: The $^{1}/_{2}$-inch-long (12.7 mm) steel pipe conforms to ASTM A53, Types E and S, Grade B, having a minimum yield strength of 35 ksi (241 MPa) and a minimum tensile strength of 60 ksi (413 MPa). The steel cap plate conforms to ASTM A572, Grade 50, having a minimum yield strength of 50 ksi (345 MPa) and a minimum tensile strength of 65 ksi (448 MPa). The cap plate assembly finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

3.3.4.4 Threaded Rods and Nuts: The $^{3}/_{4}$-inch-diameter steel threaded rods conform to ASTM A193, Grade B7, having a minimum yield strength of 105 ksi (724 MPa) and a minimum tensile strength of 125 ksi (862 MPa). The matching $^{3}/_{4}$-inch-diameter steel heavy hex nuts conform to ASTM A563 Grade DH or DH3, or ASTM A194 Grade 2H. The threaded rods and nuts are zinc-coated in accordance with ASTM B633, with coating classification Fe/Zn 8.

3.3.5 New Construction Brackets HP288NCB and HP288NCB8:

3.3.5.1 Plates: The steel plates conform to ASTM A36, having a minimum yield strength of 36 ksi (248 MPa) and a minimum tensile strength of 58 ksi (400 MPa). The plate finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

3.3.5.2 Pipe Sleeves: The pipe sleeves are steel round structural tubes that conform to ASTM A513, Type 5, Drawn Over a Mandrel (DOM), Grade 1026, having a minimum yield strength of 70 ksi (483 MPa) and a minimum tensile strength of 80 ksi (552 MPa). The sleeve finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

3.3.5.3 Bolts and Nuts: The steel bolts and nuts are those described in Section 3.3.2.2.

4.0 DESIGN AND INSTALLATION

4.1 Design:

4.1.1 General: Structural calculations (analysis and design) and drawings, prepared by a registered design professional, must be approved by the code official for each project, and must be based on accepted engineering principles as described in IBC Section 1604.4, and must conform to Section 1810 of the 2012 and 2009 IBC (Section 1808 of the 2006 IBC). The design method for the steel components is Allowable Strength Design (ASD), described in IBC Section 1602 and AISC 360 Section B3.4. The structural analysis must consider all applicable internal forces due to applied loads, structural eccentricity, and maximum spans between helical foundations. The result of this analysis, and the structural capacities, shall be used to select a helical foundation system.

The ASD capacities of FSI helical foundation system components are indicated in Tables 1, 2, 3, and 5. The geotechnical analysis must address the suitability of the helical foundation system for the specific project. It must also address the center-to-center spacing of the helical piles, considering both effects on the supported foundation and structure and group effects on the pile-soil capacity. The analysis must include estimates of the axial tension and/or compression capacities of the helical piles, whatever is relevant for the project, and the expected total and differential foundation movements due to single pile or pile group, as applicable.

A written report of the geotechnical investigation must be submitted to the code official as one of the required submittal documents, prescribed in Section 107 of the 2012 and 2009 IBC (Section 106 of the 2006 IBC), at the time of the permit application. The geotechnical report must include, but need not be limited to, the following information:

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 properties, including those affecting the design such as support conditions for the piles.
6. Recommendations for design criteria, including but not limited to mitigations of effects of differential settlement and varying soil strength, and effects of adjacent loads.
7. Field inspection and reporting procedures (to include procedures for verification of the installed bearing capacity when required).
8. Load test requirements.
9. Any questionable soil characteristics and special design provisions, as necessary.

4.1.2 Bracket Capacity (P1): Only the localized limit state of concrete bearing strength in compression has been evaluated for this evaluation report. All other limit states related to the concrete foundation, such as those limit states described in ACI 318 Appendix D, punching (two-way) shear, beam (one-way) shear, and flexural (bending) related limit states, have not been evaluated for this evaluation report. The concrete foundation must be designed and justified to the satisfaction of the code official with due consideration to all applicable limit states, and the direction and eccentricity of applied loads, including reactions provided by the brackets acting on the concrete foundation. (See Tables 1, 2 and 3.)

4.1.3 Shaft Capacity (P2): The tops of shafts must be braced as prescribed in Section 1810.2.2 of the 2012 and 2009 IBC (Section 1808.2.5 of the 2006 IBC). In accordance with Section 1810.2.1 of the 2012 and 2009 IBC (Section 1808.2.9 of the 2006 IBC), any soil other than fluid soil must be deemed to afford sufficient lateral support to prevent buckling of systems that are braced. When piles are standing in air, water or fluid soils, the unbraced length is defined as the length of pile that is standing in air, water or fluid soils plus an additional 5 feet (1524 mm) when embedded into firm soil, or an additional 10 feet (3048 mm) when embedded into soft soil. Firm soils are defined as any soil with a Standard Penetration Test (SPT) blow count of five or greater. Soft soil is defined as any soil with an SPT blow count greater than zero and less than five. Fluid soil is defined as any soil with an SPT blow count of zero [weight of hammer (WOH) or weight of rods (WOR)]. The SPT blow counts must be determined in accordance with ASTM D1586. For fully braced conditions where the pile is installed in accordance with Section 1810.2.2 of the 2012 and 2009 IBC (Section 1808.2.5 of the 2006 IBC) and piles do not stand in air, water, or fluid soils, the allowable shaft capacities must not exceed the maximum design loads shown in Tables 1, 2 and 5. Shaft capacities of helical foundation systems in air, water or fluid soils must be determined by a registered design professional. The ASD shaft tension capacities are shown in Tables 3 and 5, the ASD shaft compression capacities are shown in Tables 1, 2 and 5, and the shaft torsional rating is shown in Table 5.

The elastic shortening/lengthening of the pile shaft will be controlled by the applied loads and the mechanical and geometrical properties of the $2^{7}/_{8}$-inch-diameter (73 mm) round structural tubing and the shaft coupling. The shaft elastic shortening or lengthening can be determined from the equation:

$$\Delta_{shaft} = \frac{P \times L}{A \times E} \quad \text{(Eq. 1)}$$

where:

Δ_{shaft} = change in shaft length due to elastic shortening or lengthening (inches)

P = applied axial compression or tension load (lbf)

L = pile shaft length (inches)

A = shaft cross-sectional area (in^2) (see Table 4)

E = shaft steel modulus of elasticity (psi) (see Table 4)

4.1.4 Helix Plate Capacity (P3): The allowable axial compression and tension load capacities (P3) for each individual helical plate diameter (8, 10, 12 or 14 inches) is 55 kips (244.6 kN). (See Tables 1, 2, 3 and 5.) For helical piles with more than one helix, the allowable helix capacity (P3) for the helical foundation system may be taken as the sum of the allowable capacity of each individual helix.

4.1.5 Soil Capacity (P4): The allowable axial compressive or tensile soil capacity (P4) can be estimated 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 loading 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 those predicted by Method 1 or 2, described above.

With the individual helix bearing method, the total nominal axial load capacity of the helical pile is determined as the sum of the individual areas of the helical bearing plates times the ultimate bearing capacities of the soil or rock comprising the respective bearing strata for the plates.

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 2.0.

With the torque correlation method, the total ultimate and allowable axial load capacities are predicted as follows:

$Q_{ult} = K_t T$ (Eq. 2)

$Q_{all} = 0.5\ Q_{ult}$ (Eq. 3)

Where:

Q_{ult} = Ultimate axial tensile or compressive capacity (lbf or N) of the helical piles. For axial tension, pile ultimate axial load capacity must be limited to 55.1 kips (245.0 kN).

Q_{all} = Allowable axial tensile or compressive capacity (P4) (lbf or N) of the helical piles. For axial tension, pile allowable axial load capacity must be limited to 27.6 kips (122.5 kN).

K_t = Torque correlation factor. (See Table 5.)

T = Final installation torque, which is the final torque recorded at the termination (final) depth of the installed pile during the field installations (lbf-ft or N-m).

4.1.6 Foundation System: The ASD allowable capacity of the FSI helical foundation system in tension and compression depends upon the analysis of interaction of brackets, shafts, helical plates and soils; must be the lowest value of P1, P2, P3 and P4; and must be no larger than 60 kips (266.9 kN).

4.1.6.1 Foundation System (2012 and 2009 IBC): Under the 2012 and 2009 IBC, the additional requirements described in this section (Section 4.1.6.1) must be satisfied. For all design methods permitted under Section 4.1.1 of this report, the allowable axial compressive and tensile load of the helical pile system must be based on the least of the following conditions in accordance with 2012 and 2009 IBC Section 1810.3.3.1.9:

- Allowable load predicted by the individual helix bearing method (or Method 1) described in Section 4.1.5 of this report.
- Allowable load predicted by the torque correlation method described in Section 4.1.5 of this report.
- Allowable load predicted by dividing the ultimate capacity determined from load tests (Method 2 described in Section 4.1.5) by a safety factor of at least 2.0. This allowable load will be determined by a registered design professional for each site-specific condition.
- Allowable capacities of the shaft and shaft couplings. See Section 4.1.3 of this report.
- Sum of the allowable axial capacity of helical bearing plates affixed to the pile shaft. See Section 4.1.4 of this report.
- Allowable axial load capacity of the bracket. See Section 4.1.2 of this report.

4.2 Installation:

4.2.1 General: The FSI helical foundation systems must be installed by FSI trained and certified installers. The FSI helical foundation systems must be installed in accordance with Section 4.2, 2012 and 2009 IBC Section 1810.4.11, site-specific approved construction documents (engineering drawings and specifications), and the manufacturer's written installation instructions. In case of conflict, the most stringent requirement governs.

4.2.2 Helical Pile Installation: The helical piles are typically installed using hydraulic rotary motors having forward and reverse capabilities. The foundation piles must be aligned both vertically and horizontally as specified in the approved plans. The helical piles must be installed in a continuous manner with the pile advancing at a rate equal to at least 85 percent of the helix pitch per revolution at the time of final torque measurement. Installation speeds must be limited to less than 25 revolutions per minute (rpm). The lead and extension sections must be attached to the drive head with a product adaptor supplied by FSI. Torque readings must be taken at minimum intervals corresponding to each lead or extension section length and at final termination depth. The lead and extension sections are connected with the coupling bolts and nuts described in Section 3.2.1, and tightened to a snug-tight condition as defined in Section J3 of AISC 360. The final installation torque must equal or exceed that as specified by the torque correlation method, to support the allowable design loads of the structure using a torque correlation factor (K_t) of 9 ft^{-1} (29.5 m^{-1}). The installation torque must not exceed 7,898 ft-lbs (10 708 N-m). See Section 5.0 for further installation conditions of use.

4.2.3 Retrofit Bracket Installation:

1. An area must be excavated to expose the footing with an excavation approximately 3 feet (914 mm) square and with a depth of about 13 inches (330 mm) below the bottom of the footing. The soil is removed below the bottom of the footing to about 9 inches (229 mm) from the footing face in the area where the bracket bearing plate will be placed. The vertical and bottom faces of the footing must, to the extent possible, be smooth and at right angles to each other for the mounting of the support bracket.
2. Notching of footings may be needed to place the retrofit bracket directly under the wall/column. Notching must be performed, however, only with the acceptance of the registered design professional and the approval of the code official.
3. The bearing surfaces of the concrete (bottom and side of footing) must be prepared so that they are smooth and free of all soil, debris and loose concrete so as to provide a full and firm contact of the retrofit bracket plates.
4. The edge of the lead section shaft must be located about $1^1/_2$ inches (38 mm) from the bottom edge of the footing with a required angle of inclination of 3.0 ± 1.0 degrees from the vertical. Installation must be as described in Section 4.2.2.
5. When the final bearing depth is reached, the pile shafts are cut to approximately 13 inches (330 mm) above the bottom of footing.
6. The external sleeve must be placed through the bracket body and over the shaft. Once under the footing, the bracket must be rotated 180 degrees toward the footing. The bracket must be raised up to the footing and held in place while the thread rods and cap plate are attached.
7. The cap plate and all thread rods and tightening nuts must be installed to snug the bracket to the bottom of the footing.
8. Soil must be placed and compacted adequately up to the bottom of the bracket prior to structural lift or load transfer.
9. A lift cylinder can be used to lift the structure to desired elevation and to transfer the designated portion of the foundation load to the helical pile system.

10. Lifting of the existing foundation structure must be verified by the registered design professional and is subject to approval of the code official to ensure that the foundation and superstructure are not overstressed.
11. Field installation logs must be completed and excavation pits or trenches must be backfilled and compacted. Proper compaction procedures must comply with the approved construction documents for any site-specific requirement. When possible or as required by the approved construction document, grades or other means must be constructed to allow proper, positive surface drainage away from the structure.

4.2.4 New Construction Bracket Installation:

1. The helical pile must be installed in accordance with Section 4.2.2 with an allowable angular tolerance of ± 1 degree from the vertical.
2. The top of pile elevation must be established and must be consistent with the specified elevation. If necessary, the pile can be cut off in accordance with the manufacturer's instructions at the required elevation.
3. The new construction bracket must be placed over the top of the pile, with the bracket cap plate in full, direct contact (bearing) with the top of the pile shaft.
4. If the pile is used to resist tension forces, the new construction bracket must be embedded with proper distance into the footing or grade beam as required to resist the tension loads as determined by a registered design professional, and must be through-bolted to the helical pile shaft with two bolts and matching nuts as specified in Sections 3.2.2.2 and 3.3.5.3, and installed to a snug-tight condition in accordance with Section 4.2.2. Refer to Tables 2 and 3 for the proper embedded edge distance requirements for the shaft and bracket.

4.3 Special Inspection:

Continuous special inspection in accordance with Section 1705.9 of the 2012 IBC (Section 1704.10 of the 2009 IBC, and Section 1704.9 of the 2006 IBC) must be provided for the installation of foundation piles and foundation brackets. Where on-site welding is required, special inspection in accordance with Section 1705.2 of the 2012 IBC (Section 1704.3 of the 2009 and 2006 IBC) is also required. Items to be confirmed by the special inspector include, but are not limited to, the manufacturer's certification of installers, verification of the product manufacturer, helical pile and bracket configuration and identification, inclination and position of the helical pies, the installation torque and depth of the foundation piles, compliance of the installation with the approved construction documents and this evaluation report.

5.0 CONDITIONS OF USE

The Foundation Supportworks, Inc. (FSI), Model HP288 Helical Foundation Systems described in this report comply with the 2012 and 2009 IBC, and are suitable alternatives to what is specified in the 2006 IBC, subject to the following conditions:

5.1 The FSI helical foundation systems are manufactured, identified and installed in accordance with this report, approved construction documents (engineering drawings and specifications), and the manufacturer's written installation instructions. In case of conflict, the most stringent requirement governs.

5.2 The FSI helical foundation systems have been evaluated for support of structures assigned to Seismic Design Categories A, B and C in accordance with IBC Section 1613. Helical foundation systems that support structures assigned to Seismic Design Category 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 code official, based upon submission of an engineering design in accordance with the code by a registered design professional.

5.3 Installations of the helical foundation systems are limited to regions of concrete members where analysis indicates no cracking occurs at service load levels.

5.4 Retrofit and new construction brackets must be used only to support structures that are laterally braced as defined in Section 1810.2.2 of the 2012 and 2009 IBC (Section 1808.2.5 of the 2006 IBC).

5.5 The helical foundation systems must not be used in soil conditions that are indicative of a potential pile deterioration or corrosion situation as defined by the following: (1) soil resistivity of less than 1,000 ohm-cm; (2) soil pH of less than 5.5; (3) soils with high organic content; (4) soil sulfate concentrations greater than 1,000 ppm; (5) soils located in a landfill; or (6) soil containing mine waste.

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

5.7 The new construction helical piles (piles with new construction brackets) must be installed vertically plumb into the ground with a maximum allowable angle of inclination tolerance of 0° ± 1°. To comply with requirements found in Section 1810.3.1.3 of the 2012 and 2009 IBC (Section 1808.2.8 of the 2006 IBC), the superstructure must be designed to resist the effects of helical pile mislocation.

5.8 The retrofit helical piles must be installed at a maximum angle of inclination of 3.0 ± 1.0 degrees from the vertical.

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

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

5.11 The adequacy of the concrete structures that are connected to the FSI brackets must be verified by a registered design professional, in accordance with applicable code provisions, such as Chapter 15 of ACI 318 and Chapter 18 of IBC. The adequacy is subject to the approval of the code official.

5.12 A geotechnical investigation report for each project site must be provided to the code official for approval in accordance with Section 4.1.1 of this report.

5.13 When using the alternative basic load combinations prescribed in Section 1605.3.2, the allowable stress increases permitted by material chapters of the IBC (including Chapter 18) or the referenced standards are prohibited.

5.14 The minimum helical pile center-to-center spacing must be three times the largest helical bearing plate diameters. For piles with closer spacing, the pile allowable load reductions due to pile group effects must be included in the geotechnical report described in Section 4.1.1 of this report, and must be considered in the pile design by a registered design professional. The spacing and load reductions, if applicable, are subject to the approval of the code official.

5.15 For piles supporting tension loads, the piles 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. In cases where the installation depth is less than 12D, the minimum embedment depth must be determined by a registered design professional based on site-specific soil conditions, and the determination is subject to the approval of the code official. For tension applications where the helical pile is installed at an embedment depth of less than 12D, the torque-correlation soil capacity, P4, is outside of the scope of this evaluation report.

5.16 Piles supporting compression loads must be installed such that the minimum depth from the bottom of the pile-supported footing to the uppermost helix is no less than 5 feet (1524 mm). For compression piles with a shallower helix depth, the actual helix depth must be considered in the pile design by a registered design professional. The depth is subject to approval of the code official.

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

5.18 Requirements listed in the footnotes to Tables 1, 2, 3, and 5 must be satisfied.

5.19 Settlement of helical piles is beyond the scope of this evaluation report, and must be determined by a registered design professional as required in Section 1810.2.3 of the 2012 and 2009 IBC (Section 1808.2.12 of the 2006 IBC).

5.20 The FSI helical foundation systems are manufactured at the following facilities: Distefano Technology & Manufacturing Company, 3838 South 108th Street, Omaha, Nebraska; Behlen Manufacturing Company, 4025 East 23rd Street, Columbus, Nebraska; and TSA Manufacturing, 14901 Chandler Road, Omaha, Nebraska. Manufacturing is done 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 Pile Systems and Devices (AC358), dated June 2013.

7.0 IDENTIFICATION

The FSI helical foundation system components described in this report are identified by labels that include the report holder's name (Foundation Supportworks, Inc.); the name and address of Distefano Technology & Manufacturing Company, Behlen Manufacturing Company, or TSA Manufacturing; the product name; the model number (HP288); the part number; the evaluation report number (ESR-3074).

TABLE 1—HP288 (WITH RETROFIT BRACKETS) ASD COMPRESSION CAPACITIES

Bracket Part No.[1]	Sleeve Part No.[1]	HP288 Bracket Description	Allowable Compression Capacity (kips)				
			Bracket (P1)[2]	Shaft (P2)[3]	Helix (P3)[4]	Soil (P4)[5]	Foundation System[6]
FS288B	FS288ES30	Standard Bracket w/30" Sleeve	24.9	60	55	35.5	24.9
FS288B-G	FS288ES30-G		27.9	60	55	35.5	27.9
FS288B	FS288ES48	Standard Bracket w/48" Sleeve	31.4	60	55	35.5	31.4
FS288B-G	FS288ES48-G		35.1	60	55	35.5	35.1
FS288BL	FS288ES30	Low Profile Bracket w/30" Sleeve	25.3	60	55	35.5	25.3
FS288BL-G	FS288ES30-G		28.2	60	55	35.5	28.2

For **SI:** I inch = 25.4 mm, 1 kip = 1000 lbf = 4.448 kN.
[1]Part numbers with "G" suffix indicate hot-dip galvanized coating. Part numbers without a "G" suffix indicate plain steel.
[2]Bracket capacity is based on full scale load tests per AC358 with an installed 5'-0" unbraced pile length per Section 1810.2.1 of the 2012 and 2009 IBC (Section 1808.2.9.2 of the 2006 IBC), having a maximum of one coupling.
[3]Shaft capacity is applicable only to the foundation systems that are fully braced as described in Section 4.1.3.
[4]Helix capacity is based on a single helix plate with outer diameter of 8, 10, 12 or 14 inches (203, 254, 305 or 356 mm).
[5]Soil capacity is based on torque correlation per Section 4.1.5 of this report, with piles installed at the maximum torsion rating.
[6]Foundation system allowable capacity is based on the lowest of P1, P2, P3 and P4 listed in this table. See Section 4.1.6 for additional requirements.

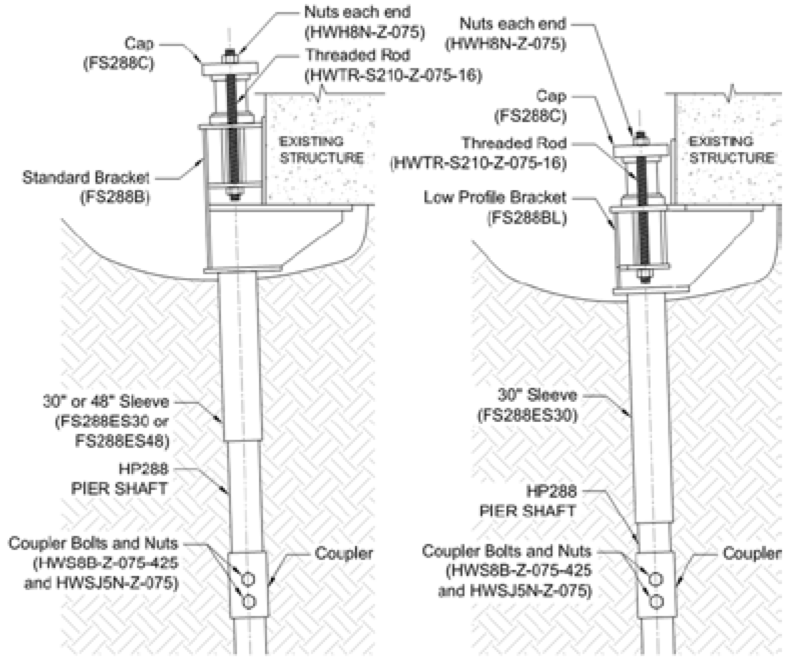

FIGURE 1—HP288 RETROFIT BRACKET AND SHAFT ASSEMBLIES

TABLE 2—HP288 (WITH NEW CONSTRUCTION BRACKETS) ASD COMPRESSION CAPACITIES[7]

Bracket Part No.[1]	Bearing Plate Dimensions (in)	Minimum Concrete Compressive Strength (psi)	Edge Distance "A" (in)	Allowable Compression Capacity (kips)				
				Bracket (P1)[2]	Shaft (P2)[3]	Helix (P3)[4]	Soil (P4)[5]	Foundation System[6]
HP288NCB or HP288NCB-G	6 x 6 x 0.75	2500	3	33.1	60	55	35.5	33.1
			≥ 4	44.1	60	55	35.5	35.5
		3000	≥ 3	39.7	60	55	35.5	35.5
HP288NCB8	8 x 8 x 0.75	2500	≥ 4	43.1	60	55	35.5	35.5
HP288NCB8-G	8 x 8 x 0.75	2500	≥ 4	46.5	60	55	35.5	35.5

For **SI:** l inch = 25.4 mm, 1 kip = 1000 lbf = 4.448 kN.
[1]Part numbers with "G" suffix indicate hot-dip galvanized coating. Part numbers without a "G" suffix indicate plain steel.
[2]Bracket capacity is based on localized limit state of concrete bearing only. All other limit states related to the concrete foundation, such as punching shear, have not been evaluated in this evaluation report.
[3]Shaft capacity is applicable only to the foundation systems that are fully braced as described in Section 4.1.3.
[4]Helix capacity is based on a single helix plate with outer diameter of 8, 10, 12 or 14 inches (203, 254, 305 or 356 mm).
[5]Soil capacity is based on torque correlation per Section 4.1.5 of this report, with piles installed at the maximum torsion rating.
[6]Foundation system allowable capacity is based on the lowest of P1, P2, P3 and P4 listed in this table. See Section 4.1.6 for additional requirements.
[7]Reduction of plain concrete thickness described in Section 22.4.7 of ACI 318-11 for the 2012 IBC (Section 22.4.7 of ACI 318-08 for the 2009 IBC, and 22.4.8 of ACI 318-05 for the 2006 IBC) is assumed not applicable.

TABLE 3—HP288 (WITH NEW CONSTRUCTION BRACKETS) ASD TENSION CAPACITIES[6]

Bracket Part No.[1]	Bearing Plate Dimensions (in)	Minimum Concrete Compressive Strength (psi)	Edge Distance "A" (in)	Allowable Tension Capacity (kips)				
				Bracket (P1)[2,7]	Shaft (P2)	Helix (P3)[3]	Soil (P4)[4]	Foundation System[5]
HP288NCB or HP288NCB-G	6 x 6 x 0.75	2500	3	24.3	34.1	55	27.6	24.3
			≥ 4	32.4	34.1	55	27.6	27.6
		3000	≥ 3	29.1	34.1	55	27.6	27.6
		3500	≥ 3	34.0	34.1	55	27.6	27.6
HP288NCB8	8 x 8 x 0.75	2500	≥ 4	34.1	34.1	55	27.6	27.6
HP288NCB8-G	8 x 8 x 0.75	2500	≥ 4	38.2	38.2	55	27.6	27.6

For **SI:** l inch = 25.4 mm, 1 kip = 1000 lbf = 4.448 kN, 1 psi = 6.895 kPa.
[1]Part numbers with "G" suffix indicate hot-dip galvanized coating. Part numbers without a "G" suffix indicate plain steel.
[2]Bracket capacity is based on localized limit state of concrete bearing only. All other limit states related to the concrete foundation, such as punching shear, have not been evaluated in this evaluation report.
[3]Helix capacity is based on a single helix plate with outer diameter of 8, 10, 12 or 14 inches (203, 254, 305 or 356 mm).
[4]Soil capacity is based on torque correlation per Section 4.1.5 of this report, with piles installed at the maximum torsion rating.
[5]Foundation system allowable capacity is based on the lowest of P1, P2, P3 and P4 listed in this table. See Section 4.1.6 for additional requirements.
[6]Reduction of plain concrete thickness described in Section 22.4.7 of ACI 318-11 for the 2012 IBC (section 22.4.7 of ACI 318-08 for the 2009 IBC, and 22.4.8 of ACI 318-05 for the 2006 IBC) is assumed not applicable.
[7]Bolts must be installed in accordance with Sections 3.2.2.2 and 4.2.4 of this report.

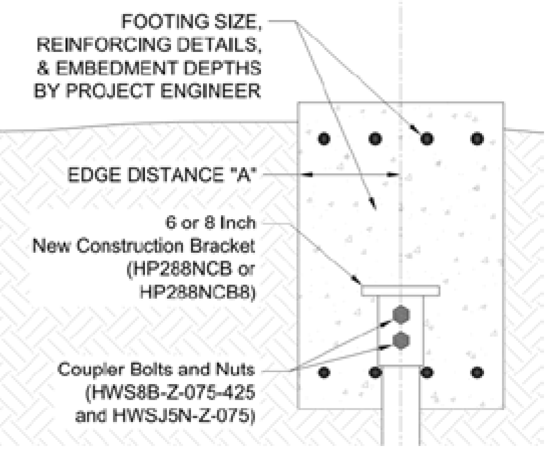

FIGURE 2—HP288 NEW CONSTRUCTION BRACKET ASSEMBLY

TABLE 4—MECHANICAL PROPERTIES OF 2.875-INCH DIAMETER HELICAL SHAFTS

Mechanical Properties	Un-corroded	After 50 Year Corrosion Loss	
	Plain Steel	Plain Steel	Hot-dip Galvanized Steel
Steel Minimum Yield Strength, Fy	60 ksi	60 ksi	60 ksi
Steel Minimum Ultimate Strength, Fu	70 ksi	70 ksi	70 ksi
Modulus of Elasticity, E	29,000 ksi	29,000 ksi	29,000 ksi
Nominal Wall Thickness	0.276 in.	0.276 in.	0.276 in.
Design Wall Thickness	0.257 in.	0.221 in.	0.247 in.
Outside Diameter, OD	2.875 in.	2.839 in.	2.865 in.
Inside Diameter, ID	2.361 in.	2.397 in.	2.371 in.
Cross Sectional Area, A	2.11 in^2	1.82 in^2	2.03 in^2
Moment of Inertia, I	1.83 in^4	1.57 in^4	1.76 in^4
Radius of Gyration, r	0.93 in.	0.93 in.	0.93 in.
Elastic Section Modulus, S	1.27 in^3	1.10 in^3	1.23 in^3
Plastic Section Modulus, Z	1.77 in^3	1.52 in^3	1.70 in^3

For **SI:** I inch = 25.4 mm, 1 ksi = 6.895 MPa, 1lbf-ft = 1.356 N-m, 1 lbf = 4.448 N.

TABLE 5—HP288 LEAD AND EXTENSION ASD TENSION AND COMPRESSION CAPACITIES[1,6]

Lead/Extension Part No.	Net Shaft Length "L" (in)	Helix Diameter (in)				(P2)[2] Shaft Comp. (kips)	(P2) Shaft Ten. (kips)	(P3)[3] Helix (kips)	K_t (ft^{-1})	Shaft Torsion Rating[4] (lbf-ft)	(P4)[5] Torque Correlated Soil Capacity (kips)	
		A	B	C	D						Com.	Ten.
HP288L5H8-3850	60	8	--	--	--	60	34.1	55	9	7898	35.5	27.6
HP288L5H0-3850	60	10	--	--	--	60	34.1	55	9	7898	35.5	27.6
HP288L5H2-3850	60	12	--	--	--	60	34.1	55	9	7898	35.5	27.6
HP288L5H4-3850	60	14	--	--		60	34.1	55	9	7898	35.5	27.6
HP288L5H80-3850	60	8	10	--	--	60	34.1	55	9	7898	35.5	27.6
HP288L5H02-3850	60	10	12	--	--	60	34.1	55	9	7898	35.5	27.6
HP288L5H24-3850	60	12	14	--		60	34.1	55	9	7898	35.5	27.6
HP288L7H8-3850	84	8	--	--	--	60	34.1	55	9	7898	35.5	27.6
HP288L7H0-3850	84	10	--	--	--	60	34.1	55	9	7898	35.5	27.6
HP288L7H2-3850	84	12	--	--	--	60	34.1	55	9	7898	35.5	27.6
HP288L7H4-3850	84	14	--	--		60	34.1	55	9	7898	35.5	27.6
HP288L7H80-3850	84	8	10	--	--	60	34.1	55	9	7898	35.5	27.6
HP288L7H02-3850	84	10	12	--	--	60	34.1	55	9	7898	35.5	27.6
HP288L7H24-3850	84	12	14	--	--	60	34.1	55	9	7898	35.5	27.6
HP288L7H802-3850	84	8	10	12	--	60	34.1	55	9	7898	35.5	27.6
HP288L7H024-3850	84	10	12	14	--	60	34.1	55	9	7898	35.5	27.6
HP288L0H80-3850	120	8	10	--	--	60	34.1	55	9	7898	35.5	27.6
HP288L0H02-3850	120	10	12	--	--	60	34.1	55	9	7898	35.5	27.6
HP288L0H24-3850	120	12	14	--	--	60	34.1	55	9	7898	35.5	27.6
HP288L0H802-3850	120	8	10	12	--	60	34.1	55	9	7898	35.5	27.6
HP288L0H024-3850	120	10	12	14	--	60	34.1	55	9	7898	35.5	27.6
HP288L0H8024-3850	120	8	10	12	14	60	34.1	55	9	7898	35.5	27.6
HP288E3H4-3850	30	14	--	--	--	60	34.1	55	9	7898	35.5	27.6
HP288E4H4-3850	42	14	--	--	--	60	34.1	55	9	7898	35.5	27.6
HP288E5H4-3850	54	14	--	--	--	60	34.1	55	9	7898	35.5	27.6
HP288E7H4-3850	78	14	--	--	--	60	34.1	55	9	7898	35.5	27.6
HP288E0H4-3850	114	14	--	--	--	60	34.1	55	9	7898	35.5	27.6
HP288E7H44-3850	78	14	14	--	--	60	34.1	55	9	7898	35.5	27.6
HP288E0H44-3850	114	14	14	--	--	60	34.1	55	9	7898	35.5	27.6
HP288E3	30	--	--	--	--	60	34.1	55	9	7898	35.5	27.6
HP288E5	54	--	--	--	--	60	34.1	55	9	7898	35.5	27.6
HP288E7	78	--	--	--	--	60	34.1	55	9	7898	35.5	27.6
HP288E0	114	--	--	--	--	60	34.1	55	9	7898	35.5	27.6

For **SI:** I inch = 25.4 mm, 1 kip = 1000 lbf = 4.448 kN, 1lbf-ft = 1.356 N-m.
[1]Part numbers with "G" suffix indicate hot-dip galvanized coating. Part numbers without a "G" suffix indicate plain steel.
[2]Shaft compression capacity (P2) is based on that the foundation system is fully braced as described in Section 4.1.3.
[3]Helix capacity (P3) is applicable to both tension and compression loading and is based on a single helix plate with outer diameter of 8, 10, 12 or 14 inches (203, 254, 305 or 356 mm).
[4]Shaft torsion rating is the maximum torsion that can be applied to the shaft during the helical pile installation.
[5]Torque correlated soil capacity (P4) is applicable to both tension and compression loading and is based on torque correlation per Section 4.1.5, with piles installed at the maximum torsion rating.
[6]For piles with extension(s), shaft coupling(s) must be installed in accordance with Sections 3.2.1 and 4.2.2 of this report.

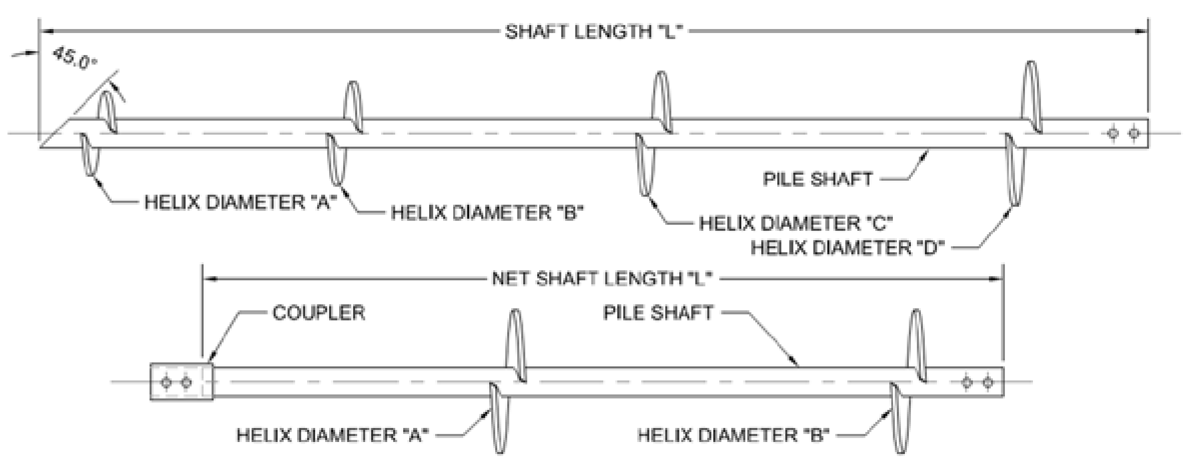

FIGURE 3—TYPICAL HP288 SHAFT LEAD AND EXTENSION SECTIONS AND HELIX PLATES

ICC-ES Evaluation Report

ESR-3074 FBC Supplement

Reissued May 2014

This report is subject to renewal July 1, 2015.

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

REPORT HOLDER:

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EVALUATION SUBJECT:

FOUNDATION SUPPORTWORKS HELICAL FOUNDATION SYSTEMS

1.0 REPORT PURPOSE AND SCOPE

Purpose:

The purpose of this evaluation report supplement is to indicate that the Foundation Supportworks, Inc. (FSI), Model HP288 Helical Foundation Systems, recognized in ICC-ES master report ESR-3074, have also been evaluated for compliance with the codes noted below.

Applicable code editions:

- 2010 *Florida Building Code—Building*
- 2010 *Florida Building Code—Residential*

2.0 CONCLUSIONS

The Foundation Supportworks, Inc. (FSI), Model HP288 Helical Foundation Systems**,** described in Sections 2.0 through 7.0 of the master evaluation report ESR-3074, comply with the 2010 *Florida Building Code—Building* and the 2010 *Florida Building Code—Residential*, provided the design and installation are in accordance with the *International Building Code®* provisions noted in the master report and the following conditions apply:

1. Design wind loads must be based on Section 1609 of the 2010 *Florida Building Code—Building* or Section 301.2.1.1 of the 2010 *Florida Building Code—Residential*, as applicable.
2. Load combinations must be in accordance with Section 1605.2 or Section 1605.3 of the 2010 *Florida Building Code—Building*, as applicable.

Use of the Foundation Supportworks Inc. (FSI) Model HP288 Helical Foundation Systems for compliance with the High-Velocity Hurricane Zone provisions of the 2010 *Florida Building Code—Building* and the 2010 *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 master report, reissued May 2014.

Evaluation Report CCMC 13556-R

MASTERFORMAT: 31 62 16.01
Issued: 2011-11-07
Revised: 2012-05-28
Re-evaluation due: 2014-11-07

Foundation Supportworks® Helical Foundation Systems and Devices

1. Opinion

It is the opinion of the Canadian Construction Materials Centre (CCMC) that "Foundation Supportworks® Helical Foundation Systems and Devices", when used as an auger-installed steel pile in a foundation system in accordance with the conditions and limitations stated in Section 3 of this Report, complies with the National Building Code 2010:

- Clause 1.2.1.1.(1)(a), Division A, using the following acceptable solutions from Division B:
  - Clause 4.2.3.8.(1)(e) Steel Piles
  - Sentence 4.2.3.10.(1) Corrosion of Steel
  - Sentence 4.2.4.1.(1) Design Basis
  - Subclause 9.4.1.1.(1)(c)(i) General (structural requirements)

This opinion is based on CCMC's evaluation of the technical evidence in Section 4.1 provided by the Report Holder.

Ruling No. 12-05-275 (13556-R) authorizing the use of this product in Ontario, subject to the terms and conditions contained in the Ruling, was made by the Minister of Municipal Affairs and Housing on 2012-05-17 pursuant to s.29 of the Building Code Act, 1992 (see Ruling for terms and conditions). This Ruling is subject to periodic revisions and updates.

2. Description

The shaft is manufactured into lead sections with helical plates and extension sections either with helical plates (helical extension) or without helical plates (plain extension). Lead sections are available in lengths of 1524 mm, 2134 mm or 3048 mm and extension sections are available in net lengths of 762 mm, 1372 mm, 1981 mm or 2896 mm. The leads and extensions are connected together with a welded coupling and two bolts. One or more blades, up to a maximum of four blades, can be used for the lead sections. The coupling compression capacity is designed for end-to-end contact of the shaft sections. The helical pile central shaft consists of a pipe with an outside diameter of 73 mm and a 7.0-mm wall. The welded coupling consists of a tube with an outside diameter of 89 mm and a 7.1-mm wall thickness. The pile sections are coupled together with two 19.1-mm diameter bolts and nuts. The bolts and nuts are zinc coated in accordance with ASTM A 153/A 153M-09, "Zinc Coating (Hot-Dip) on Iron and Steel Hardware."

The helical plates are cut into a circular shape from 9.4-mm-thick steel plates and are formed into a true helix shape with outer diameters of 203 mm, 254 mm, 305 mm or 356 mm and are welded to the shaft lead and extension sections. Figure 1 is a diagram of the product.

The steel shaft, blades and accessories for the product conform to CSA G40.21-04(R2009), "Structural Quality Steel," while their galvanic coating meets the requirements of CAN/CSA-G164-M92(R2003), "Hot Dip Galvanizing of Irregularly Shaped Articles."

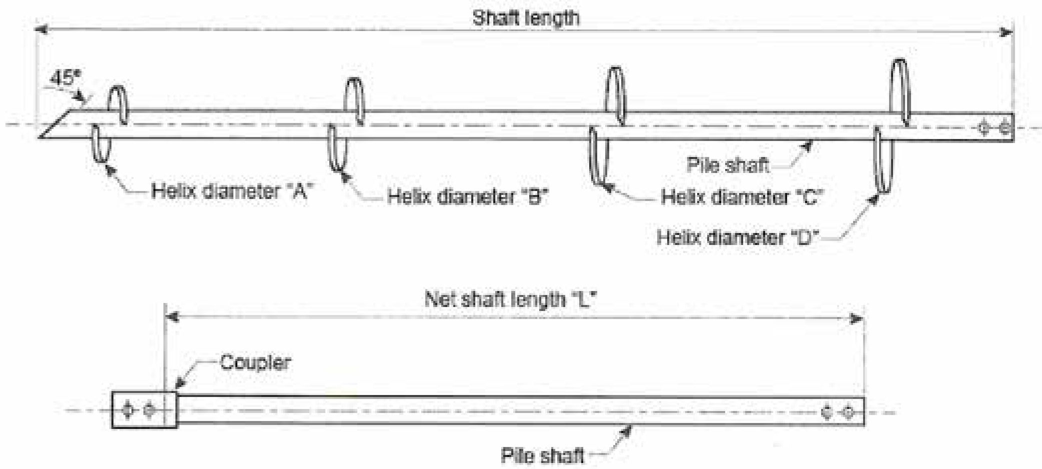

Figure 1. General diagram of the product

3. Conditions and Limitations

CCMC's compliance opinion in Section 1 is bound by the "Foundation Supportworks® Helical Foundation Systems and Devices" being used in accordance with the conditions and limitations set out below.

- The product may be used as a foundation system to support various constructions, provided that it is installed according to the manufacturer's current instructions and within the scope of this Evaluation Report.
- When the product is installed in undisturbed or uniformly placed and well-engineered fill soils there is a direct relationship between the applied torque and the allowable compressive and tensile load. Table 3.1 indicates the allowable compressive and tensile loads as a function of the applied torque. Note: For additional information and system capacity tables refer to Foundation Supportworks® Technical Manual dated November, 2010.
- When the auger-installed steel pile is installed in bedrock, the relationship between the applied torque and the allowable compressive and tensile load is not predictable. As a result, the allowable compressive and tensile loads have to be confirmed by on-site load tests. These load tests are also required if the allowable loads need to be greater than those stated in Table 3.1. The tests must be conducted under the direct supervision of a professional geotechnical engineer skilled in such design and licensed to practice under the appropriate provincial or territorial legislation.
- In all cases, a registered professional engineer skilled in such design and licensed to practice under the appropriate provincial or territorial legislation must determine the number and spacing of the auger-

installed steel piles required to carry the load. A certificate attesting to the conformity of the installation and the allowable loads for the piles must be provided.

- The installation of the auger-installed steel pile must be carried out as per the manufacturer's instructions. The anchors must be screwed into the ground using mechanized equipment. The anchor must be rotated into the ground with sufficient applied downward pressure (crowd) to advance the anchor one pitch distance per revolution. The anchor must be advanced until the applied torque value attains a specified value. Extensions must be added to the central shaft as needed. The applied loads may be tensile (uplift) or compressive (bearing). They are immediately ready for loading after installation.
- When the product is installed in a soil where the conditions are corrosive to steel, adequate protection to the exposed steel must be provided.
- To be permitted to install auger-installed steel piles for the product, the installer must be certified by Foundation Supportworks®. Using approved equipment, the installer must meet the uses and limitations specified in this Report. Each installer must carry a manufacturer approved card with their signature and photograph.
- Each auger-installed steel pile for the product must be identified with a label containing the following information: manufacturer's identification and the phrase "CCMC 13556-R."

Table 3.1 Allowable Compressive and Tensile Loads for the Product[1]

Applied Torque		Torque Correlated Allowable Soil Capacity			
		Compression		Tension	
N-m	(ft-lb)	kN	lb	kN	lb
678	500	10	2250	10	2250
1356	1000	20	4500	20	4500
2034	1500	30	6750	30	6750
2712	2000	40	9000	40	9000
3390	2500	50	11250	50	11250
4067	3000	60	13500	60	13500
4745	3500	70	15750	70	15750
5423	4000	80	18000	80	18000
6101	4500	90	20250	90	20250
6779	5000	100	22500	100	22500
7457	5500	110	24750	110	24750
8135	6000	120	27000	120	27000
8813	6500	130	29250	130	29250
9491	7000	140	31500	140	31500
10169	7500	150	33750	150	33750
10711	7900	158	35550	158	35550

Note to Table 3.1:

1. The allowable loads identified in this Table are only valid when the product is installed in undisturbed or uniformly placed and well-engineered fill soils. Special attention is required when the auger-installed steel piles are installed in recently backfilled sites or in bedrock soils. In these cases, Table 3.1 does not apply and the allowable loads must be determined by on-site confirmatory testing.

4. Technical Evidence

The Report Holder has submitted technical documentation for CCMC's evaluation. Testing was conducted at laboratories recognized by CCMC. The corresponding technical evidence for this product is summarized below.

4.1 General

The product's auger-installed steel piles were tested in accordance with ASTM D 1143/D 1143M-07e1, "Standard Test Methods for Deep Foundations Under Static Axial Compressive Load," and ASTM D 3689-07, "Standard Test Methods for Deep Foundations Under Static Axial Tensile Load." Testing was conducted on two different sites – the first had bedrock and clay soil and the second had only bedrock and sand soil. A series of 16 tests were performed at the two sites, 8 tension and 8 compression. The intent of the testing was to determine a correlation between the torque applied during installation and the allowable loads.

In both cases (compression and tension) the load tests for test piles founded in bedrock did not always provide adequate correlation to the actual maximum capacities for piles installed in bedrock. Based on this result, correlation should only be applied to piles installed in undisturbed soils or uniformly placed and well-engineered fill soils. The correlation may not be applicable in uncontrolled fill situations. In such conditions it will be necessary to perform load tests to determine the capacity of the piles.

The correlation between the torque applied during installation and the allowable loads for compressive and tensile loads is noted in Table 3.1. The factor of safety used was 2.0.

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Plant(s): Columbus, NE, U.S.A.
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CHAPTER 3 HYDRAULICALLY-DRIVEN PUSH PIERS

3.1. History

Commonly referred to as push piers, jacked piles, resistance piers, or hydraulically driven piers, these systems were developed to stabilize buildings against further settlement and/or to provide additional foundation support. Push pier systems have patent history dating back to the late 1800s and had their early beginnings in the populated areas of the northeast United States (US). Several inventors from New York were the pioneers of these systems, and utilized a common methodology of pushing hollow tubular iron columns in sections to a suitable load bearing stratum. They are considered retrofit systems since they require an existing structure to provide the reaction necessary to push or drive the piers to competent soils. These early pier systems were typically installed beneath opposing sides of a building wall (staggered or in pairs), or directly beneath the center of the wall.

The first US patented push pier system was by Jules Breuchard (US Patent No. 563,130) on June 30, 1896, which specified removal of portions of brick foundation walls to allow for placement of structural "headers" (stone or steel) and set up of the drive equipment *(Figure 3.1.a)*. The drive equipment or "ram" would push steel piling sections using the weight of the structure until the desired resistance was achieved. The top of the pier would then be shimmed with brick or other structural elements to another header beam across the bottom of the foundation opening. The space between the structural headers would then be refilled with brick and mortar. The first application of this system was in New York City in 1896. The piers supported a 4-story building during the excavation and construction of the basement level of the new Commercial Cable Building on the adjacent property. Nine piers were installed along a 57-foot long wall to allow excavation to a depth of about 10 feet below the underpinned structure. The piers were manufactured from 10-inch outside diameter (O.D.) pipe with a ⅝-inch wall thickness. Five-foot lengths of pipe were pushed to depths of about 33 feet using a 60-ton jack to its full capacity. External couplings were used to maintain direct bearing of the pier sections. An internal coupling was first patented by Francis Pruyn (US Patent No. 1,188,485) on June 27, 1916. This internal coupling concept allowed for a pre-manufactured connection with the same diameter as the pier pipe *(Figure 3.1.b)*.

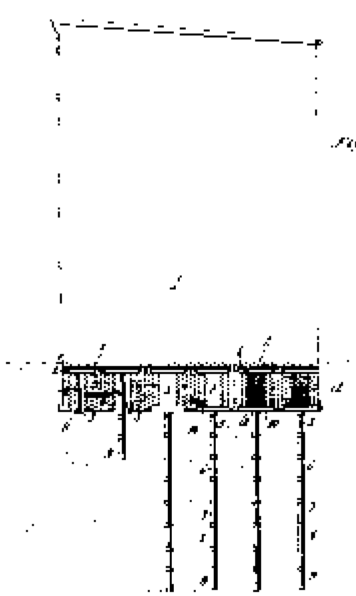


Figure 3.1.a
Breuchard patented system (1896)

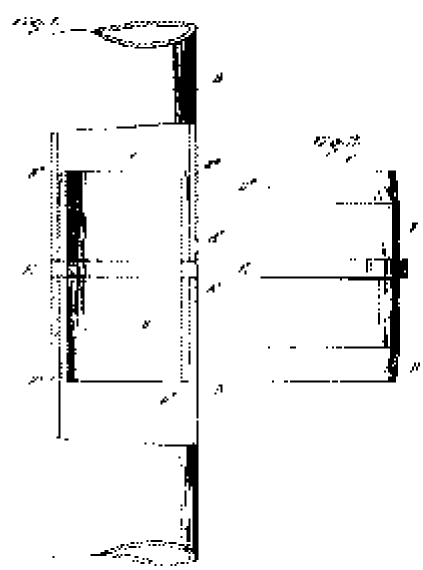


Figure 3.1.b
Pruyn patented internal coupling (1916)

The Breuchard system did not specify a factor of safety to be applied to the service loads and some problems resulted from underperformance of the piers after construction. The first patented push pier system to recommend a factor of safety was registered by Lazarus White (US Patent No. 1,217,128) on February 20, 1917. Specifically, the language in the patent recommends a final drive load about 50 percent greater than the service load applied to the pier, which equates to a factor of safety of 1.5 against pier settlement. This patent also describes using a pressure gauge on the hydraulic jack to monitor final drive and lock-off pressures. The White patent detailed geotechnical considerations, including pier rebound, and provided a drawing of the assumed pressure bulb formed under the pier tip upon loading *(Figure 3.1.c)*. In a later patent registered on October 20, 1931 (US Patent No. 1,827,921), Lazarus White recommended application of the drive force to each pier individually and then simultaneous loading of all piers to evenly distribute structural loads during lock-off procedures. Mr. White also suggested that previous performance problems with push pier systems may have been due to overlap of the assumed stress bulbs for closely spaced piers, which was believed could be mitigated by using simultaneous loading during lock-off.

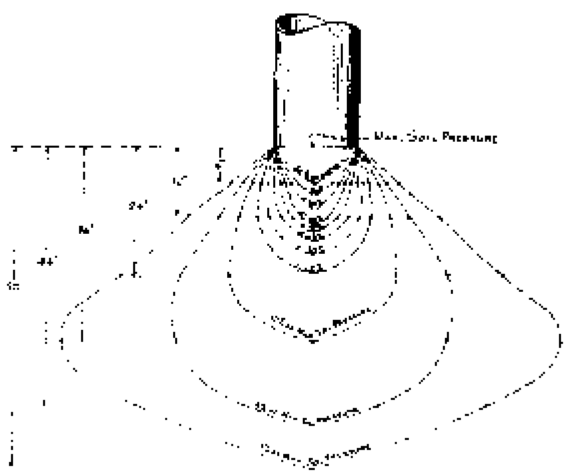


Figure 3.1.c Assumed stress bulb under push pier tip (White, 1917)

An eccentrically-loaded flush-mount bracket system was presented in the Henry Revesz and Jack Steinsberger Patent (US 2,982,103) registered on May 2, 1961. This system had many similarities to current flush-mount pier systems, including attachment of a flush-mount pier bracket to a foundation wall and using a drive stand and hydraulic jack to provide the final drive and lock-off forces. This patent also recommended applying a factor of safety of 1.5 to the service load to determine the required final drive load. The components and setup of this system are shown in *Figure 3.1.d*.

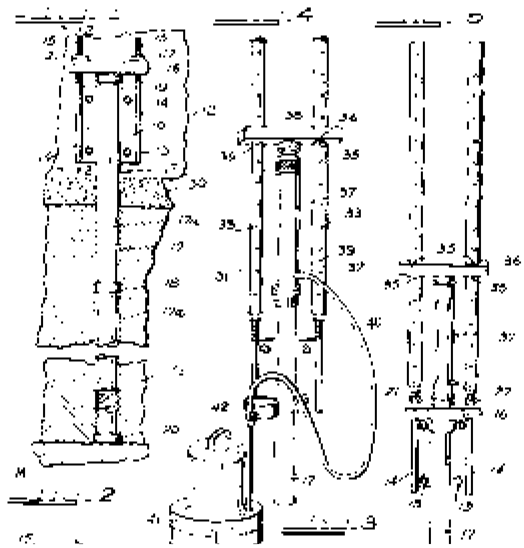


Figure 3.1.d Revesz and Steinsberger flush-mount push pier system (1961)

The first patent for a side-load, under-footing bracket with vertical and horizontal bearing plates positioned against and below a footing was issued on September 2, 1975 to George Langenbach (US Patent No. 3,902,326). The system was further refined in subsequent patents to resemble the eccentric push pier systems common today. The ingenuity of these early inventors paved the way for the development of numerous push pier systems and an industry that has grown dramatically since the 1970s. Manufacturers and installers of underpinning systems continue to provide innovative solutions capable of ever-increasing load capacities and improved system performance. Systems designed by reputable manufacturers, installed by experienced foundation repair contractors, and with a proven record of performance have become widely accepted throughout the engineering community.

3.2. Summary Description

FSI push pier systems utilize high-strength round steel tubes and a load transfer bracket (retrofit foundation repair bracket) to lift and/or stabilize sinking or settling foundations, or to provide additional capacity to existing foundation systems. The foundation bracket is secured against and below the existing footing (under-footing bracket), to the side of the footing or to the foundation wall (flush-mount bracket), or underneath existing slabs (slab pier bracket). Pier sections are then hydraulically driven through the bracket and into the soil below using the combined structural weight and any contributory soil load as drive resistance. Pier sections are added and driven until a suitable load bearing stratum is encountered. At that point, the structure either begins to lift or the target pressure/load is achieved. The weight of the structure is then transferred to the foundation brackets, through the piers, and to firm load bearing soil or bedrock. Typical under-footing, flush-mount and slab pier brackets are shown in *Figure 3.2.a*. A Model PP288 push pier installation is shown in *Figure 3.2.b*.

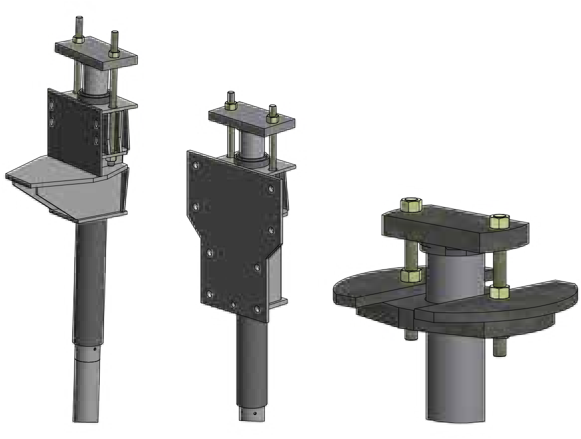


Figure 3.2.a Left to right; typical under-footing,flush-mount and slab pier brackets (no relative scale)

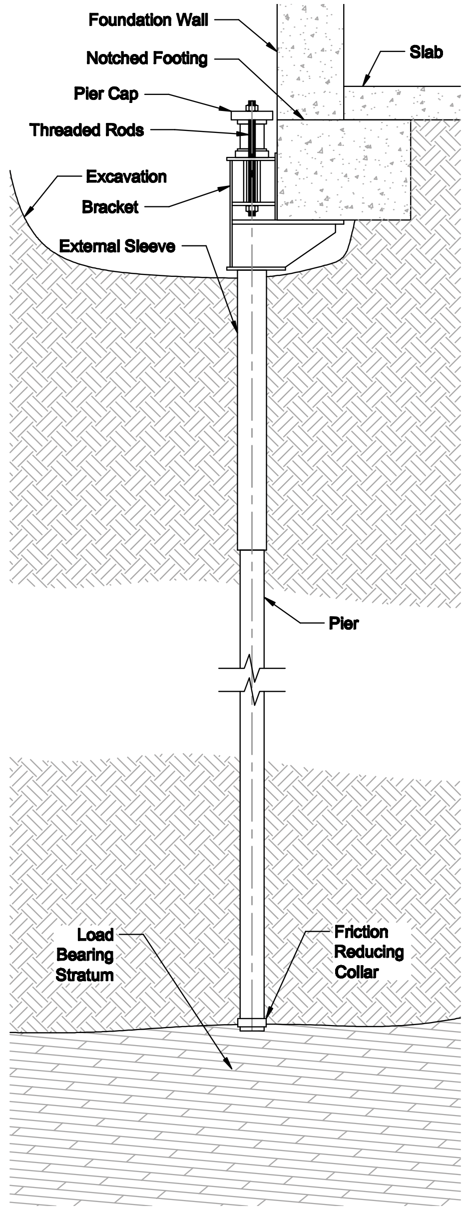


Figure 3.2.b FSI PP288 push pier system with an under-footing bracket

3.2.1. Applications

Push pier systems are typically used for underpinning existing structures in the following applications:

- Stabilization of settled foundations or slabs
- Stabilization and lifting of settled foundations or slabs
- Providing increased capacity (additional support) for existing foundations or slabs
- Providing foundation support while adjacent excavations are made

3.3. Push Pier System Components

FSI push pier system components are manufactured to high quality control standards using ASTM grade steel and certified welding processes. The product line includes Models PP237, PP288, PP350 and PP400, corresponding to shaft sizes of 2.375-inch, 2.875-inch, 3.50-inch and 4.00-inch O.D, respectively. Various exterior sleeve and bracket options are available. Push pier system capacities and specifications are provided Appendix 3A. The FSI push pier system components include the following:

3.3.1 Bracket Assemblies

Bracket assemblies may include an under-footing bracket, flush-mount bracket or a slab bracket. Under-footing brackets are typically placed against and below the footing and have vertical and horizontal bearing plates. Under-footing brackets have been designed to allow piers to be driven vertically or at 2 degrees from vertical orientation. Two-degree brackets are standard for the PP237 and PP288 systems as it allows the brackets to be seated beneath foundation walls as much as practical and provides separation for the drive stand to miss common brick overhangs and window and door trim on residential structures. Independent testing of the PP288 push pier system with both the FS288B (2-degree) and FS288BV (vertical) brackets has shown less than 1 percent difference in the capacities. Flush-mount brackets have a vertical bearing plate anchored to the vertical concrete face of the footing, grade beam or foundation wall. Slab pier brackets are plate assemblies constructed under the concrete floor slab via holes cored in the slab. Pier sections driven through flush-mount and slab pier brackets are in a vertical orientation only. The FSI bracket assemblies generally include the bracket, pier cap, external sleeve and associated hardware. However, the Model PP400 push pier system and the Model PP288 slab pier system are designed without external sleeves. See Appendix 3A for mechanical ratings of the various push pier systems and bracket assemblies.

3.3.1.1 Eccentric Loading

Pier tubes of push pier systems utilizing under-footing and flush-mount brackets are not located directly under the structure's footing. Therefore, these systems are eccentrically loaded and in turn need to resist the bending forces generated by this loading condition *(Figure 3.3.1.1.a)*. The eccentricity generated by under-footing and flush-mount bracket systems is in reality shared by the pier and the structure.

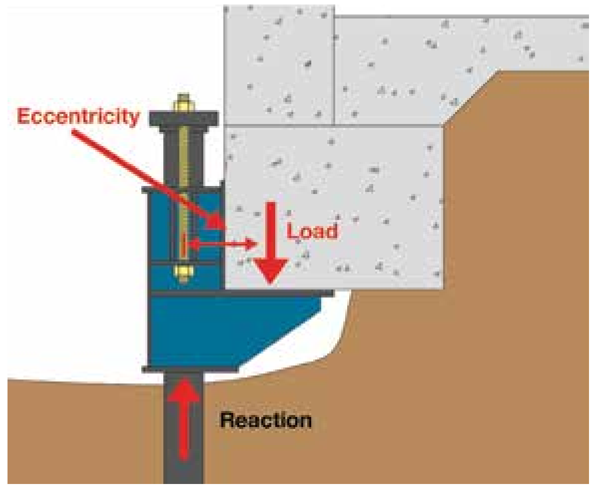


Figure 3.3.1.1.a Eccentric loading condition for under-footing bracket

In general, the more rigid the pier system and its connection to the foundation, the more the system acts as an extension of that foundation and the more eccentricity must be absorbed by the structure. This bending or twisting imparted to the structure can be resisted by the internal strength of the foundations and connections to the superstructure, by passive resistance of the soil along the opposite face of the footing and/or foundation wall, by bracing with internal structural elements such as floor slabs and shear walls, and by support generated at building corners. When the eccentricity cannot be resisted by such conditions, the piers can be staggered or paired on opposing sides of the foundation. Multiple piers are often needed at column locations simply to balance the load and prevent tipping of the footing. Evaluation of the eccentric loading condition on the structure should be completed by a qualified design professional on a case by case basis.

Overall dimensions of a push pier cross section are 4 inches or less in most applications. These sections are therefore very sensitive to the bending moments introduced by eccentric loading. Additionally, as pier bending moments increase, the pier axial capacity will decrease. This loss of axial capacity due to the addition of bending stresses can be demonstrated with the following example. A given pier section with a 3.50-inch O.D., 0.300-inch wall thickness and a yield strength of 35 ksi has a maximum allowable compressive capacity of 59.3 kips according to Allowable Stress Design. When a bending moment of 40 kip-in is applied to the same section, its allowable compressive capacity drops to 24.7 kips. This is a reduction of nearly 60 percent of the section's full axial capacity. What's more, this moment would equate to an equivalent eccentricity of only 1.62 inches, which is a seemingly small eccentricity and is still within the envelope of a typical pier cross section. Since, eccentricities for under-footing bracket systems are generally within the range of 3 to 4 inches, the loss of axial capacity due to the resulting bending moments is a significant design consideration.

The bending moment created by eccentric loading is dissipated by passive resistance of the soil against the pier tube within the first few feet of soil support, therefore, the bending moment only needs to be considered for the pier tubes directly below the bracket. One method of providing the necessary bending resistance could involve using larger diameter and/or thicker pier tube sections for the entire length of the pier. The larger/thicker pier sections would resist bending, yet still have sufficient axial capacity in reserve. Although a seemingly reasonable approach, it is not an economical one since the extra steel is only useful within the region of bending for the first few feet below the bracket where the bending moment is dissipated into the surrounding soil. Another method used by many manufacturers is to utilize internal or external pier reinforcement after the pier has been exposed to the final drive force. Internal reinforcement is simply smaller diameter pipe or tube sections set inside the pier and generally spanning between the internal couplers of the pier shaft. Internal reinforcement can be of inconsistent length and not placed at the optimal location; i.e., extending through and below the bracket, since final coupler location cannot be estimated or predetermined. Also, internal reinforcement is not generally placed until after the final drive load has been applied, when the maximum bending moment may have already caused the pier shaft to deform or buckle. There is little chance for success when trying to insert a straight pipe section through a bent tube. External reinforcement typically consists of larger round or square hollow sections driven or placed around the pier, again after the pier has been exposed to the final drive force. Similar challenges exist with placement of straight external reinforcement elements over a bent pier.

Foundation Supportworks has developed a unique approach to address the issue of eccentric loading on retrofit push pier systems… the external sleeve (see next section).

3.3.1.2 External Sleeve

The FSI PP237, PP288 and PP350 push pier systems incorporate a 48-inch long external sleeve to resist the bending forces generated by the eccentric loading on the under-footing and flush-mount brackets, thereby preserving the axial capacity of the pier sections *(Figure 3.3.1.2.a)*. The external sleeve is hydraulically driven with and around the pier starter tube section to extend through and below the bracket. The effect of the sleeve essentially creates a bracket that is 48 inches tall without any additional excavation. A 30-inch long external sleeve is available for the PP237 and PP288 push pier systems for use in limited headroom and crawl space applications.

The moment or bending force is localized within a relatively short distance below the bracket. Although the bending force is dissipated quickly by the pier bearing against the confining soil, it is significant and cannot be ignored. The

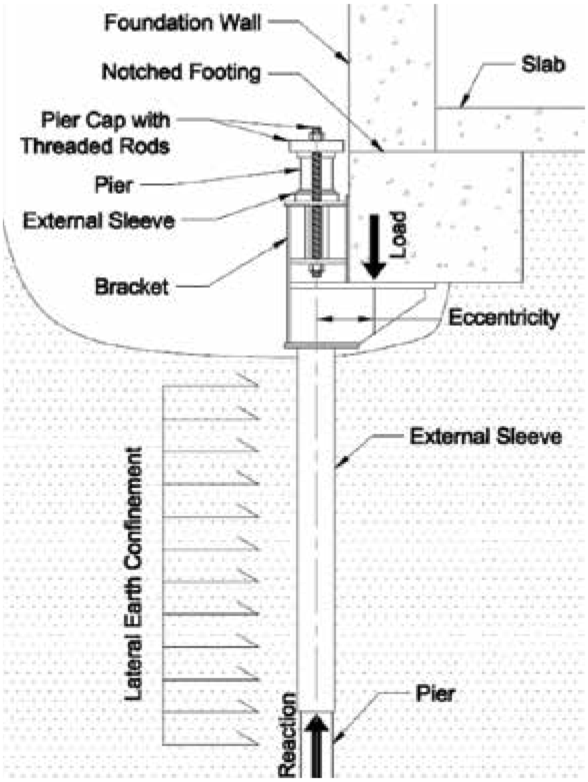


Figure 3.3.1.2.a External sleeve and pier bear against confining soil when system is under load

depth or length of sleeve and pier over which the bending force dissipates is a function of the soil stiffness. The depth is greater in soft clay and loose sand, and less in stiff clay and dense sand. Finite element analysis software was used to analyze how the external sleeve and the pier interact with soils of various properties. Bracket rotation is resisted not only by the rigidity of the pier system, but also by the passive pressure of the soil surrounding the external sleeve and pier. Therefore, the capacity and performance of the pier system is in part governed by the stiffness of the confining soils.

Benefits/advantages of an external sleeve include:

- Sleeved system separates the bending forces from axial compression forces.
- Sleeve resists most of the bending forces and behaves purely as a beam.
- Pier tube is protected from the bending forces preserving its axial capacity so it can behave more like a column.
- Sleeve is easy to install. It's driven at the same time as the starter tube.
- No cumbersome reinforcement to install after driving pier tubes.
- The extra steel is where it needs to be. Much more efficient than using thicker pier tube sections for the entire length. It's a local solution to a local issue.
- Sleeve is in place during the system's maximum load (driving tubes).
- Relieves friction between pier bracket and pier tube. Drive and lift pressures more accurately reflect the load on the pier system.

3.3.2 Starter Tube

The starter tube is the first pier section pushed into the ground and is installed at the same time as the external sleeve, where applicable. Under-footing and flush-mount bracket systems utilize a friction reduction collar at the bearing end of the starter tube. The friction reduction collar will be discussed in more detail in the following section. Models PP237 and PP288 starter tubes come in standard lengths of 32 and 50 inches. The 32-inch starter tubes are used in limited access and low headroom applications, such as within a crawl space, along with a modified (shorter) drive stand, shorter drive cylinder, a 30-inch long external sleeve and 18-inch long pier tubes. The PP350 system uses a 50-inch long starter tube only, to coincide with the standard 48-inch long external sleeve. The PP350 is a higher capacity system and generally not ideal for crawl space applications. The PP400 system uses a starter tube length consistent with the standard 36-inch pier tube length. The PP400 system is unique in the FSI push pier product line in that the under-footing bracket assembly does not include the 48-inch long external sleeve and, therefore, no special design or installation considerations have to be made for starter tube length. The PP288 slab pier system does not utilize a starter tube with a friction reduction collar. The first section advanced consists of a field-modified standard pier section (see Section 3.12).

3.3.2.1 Friction Reduction Collar

A friction reduction collar is included at the bearing end of push pier system starter tubes *(Figure 3.3.2.1.a)*. This collar consists of a 1-inch long slice of a slightly larger round shaft section slid over and welded to the end of the starter tube, or a machined ring with a pressed fit. These friction reduction collars have outside diameters ½-inch larger than their respective pier sections and serve to either create annular space or remold the soil around the pier shaft as it is advanced through the soil. The reduction in frictional resistance on the outside surface of the pier results in a driven pier that generates most of its capacity in end bearing. With reduced skin friction and high bearing pressures generated at the pier tip, push pier systems with friction reduction collars also generally penetrate deeper into the ground (than a frictional pier) and advance through weak or marginal soils to bear on competent material below.

Figure 3.3.2.1.a PP288 starter tubes with pressed fit and welded friction reduction collars

It is common to think of push piers as being advanced through overburden soils to bear on bedrock. This may or may not be the case. For residential and light commercial projects with light to moderate pier loads, adequate resistance may be achieved within very stiff to hard clay soils or medium dense to dense sand and gravel. Allowable pier capacities of 15 to 20 kips, with a factor of safety of 1.5, may be achieved in soils having standard penetration test N-values around 30 blows per foot. Higher strength soils or rock would therefore be required to develop higher pier capacities.

The soils displaced or remolded by the friction reduction collar "heal" back around and against the shaft over time, generating an additional frictional component to the pier's capacity. This effect is often referred to as pile "set up" when driving larger, higher capacity pipe piles or H-piles. Set up can occur within a matter of hours, days or weeks, and is the reason piering contractors generally try to start and finish installation of a push pier the same day and, in some unique conditions, before a work break is taken. Although this frictional capacity can be significant, it is conservatively ignored in most cases in the determination of the pier's factor of safety against settlement. The final drive force is measured and documented prior to development of the soils ultimate frictional resistance. Push pier system factors of safety are further discussed in Section 3.9.1.

3.3.3 Pier Tube

Pier tubes follow the starter tube during installation and have a crimped or plug-welded slip-fit internal coupling at the leading end (see next section). The push pier tubes and couplings are manufactured from hollow round structural steel sections. Models PP237 and PP288 push pier tubes are available in standard lengths of 18 and 36 inches. The 18-inch long pier tubes are again used for limited headroom or crawl space applications. Models PP350 and PP400 pier tubes are available in standard lengths of 36 inches only.

3.3.3.1 Coupling

Pier tube sections are coupled with an internal slip-fit connection *(Figure 3.3.3.1.a)*. A hollow round shaft section with an outside diameter smaller than the inside diameter of the respective pier tube is crimped (button-punched) or plug-welded to the leading end. The internal coupler extends one-half its length inside the pier tube and one-half its length beyond the end to maintain direct bearing of the pier sections. The coupling is not pinned or bolted and is therefore generally considered and utilized for compression applications only.

Figure 3.3.3.1.a
Push pier tube coupling

3.4 FSI Foundation and Slab Push Pier Bracket Assemblies

The FSI push pier product line includes multiple shaft sizes and bracket assemblies to account for most typical system uses and applications. A summary table of the products and system use categories is presented as *Figure 3.4.a*. Bracket systems are categorized by their design or system use and include under-footing brackets, flush-mount brackets and slab pier brackets. The bracket assemblies again include the bracket, pier cap, external sleeve (if applicable), and all associated hardware. Crawl space systems utilize under-footing bracket assemblies with a shorter external sleeve. These systems are designated with part numbers starting with "CS". Part numbers with a "CR" designation identify assemblies with coil rods; otherwise, the assembly comes with threaded rods.

Push Pier System	Bracket Assembly Part No.	Bracket Description	System Use
PP237	PP238CRBA	FS238B - standard 2-degree	Under-footing
	CS238CRBA	FS238B - standard 2-degree	Crawl space
PP288	PP288BA	FS288B - standard 2-degree	Under-footing
	PP288CRBA	FS288B - standard 2-degree	
	PP288BVA	FS288BV – vertical	
	PP288CRBVA	FS288BV – vertical	
	PP288BLA	FS288BL - 2-degree low-profile	
	PP288CRBLA	FS288BL - 2-degree low-profile	
	CS288BA	FS288B – standard 2-degree	Crawl space
	CS288CRBA	FS288B – standard 2-degree	
	FS288BFM22A	FS288BFM - with wedge anchors	Flush-mount
	FS288CRBFM22A	FS288BFM - with wedge anchors	
	FS288BFM31A	FS288BFM - with adhesive anchors	
	FS288CRBFM31A	FS288BFM - with adhesive anchors	
	SPBA	Slab pier bracket (assembled on site)	Slab Pier
PP350	PP350BA	FS350BV - vertical	Under-footing
PP400	PP400BA	FS400BV - vertical	

Figure 3.4.a FSI push pier bracket assemblies

3.5 Benefits

Some of the benefits of hydraulically-driven steel push pier systems versus other underpinning systems may include:

- Pre-manufactured components increase the quality control of the installed system
- Components available with zinc coating for additional corrosion resistance
- Laboratory testing of the push pier system and components documents the system capacity
- Eccentricity between the shaft and bearing area is minimized to reduce the bending moment transferred to the pier system, allowing superior performance when stabilizing or lifting the structure
- Drive and lock-off forces easily determined using hydraulic pumps and cylinders
- Documentation of the final drive force and lock-off force is used to verify a factor of safety at each pier location
- A proof load test is essentially completed for each pier installation
- End bearing pier is driven deep through problem soils
- Additional skin friction develops after installation increasing the factor of safety against pier settlement
- Steel reinforcement and grout added within hollow pier sections improves lateral capacity and pier stiffness
- Installs with portable hydraulic equipment
- Can be installed within areas of limited or difficult access
- Can be installed in areas of low overhead clearance (crawl spaces)
- Easy to install
- No vibration
- Installs quickly from inside or outside the structure
- Cost-effective solution

3.6 Limitations

The use of push pier systems is limited to structures that have sufficient structural load and/or contributory soil load to provide adequate resistance to advance the piers to a competent bearing stratum. Push pier systems are generally considered for compression-only applications and are not considered for lateral capacity. Foundation Supportworks published system capacities are based on the following assumptions:

- The systems should be used on structures that are fixed from translation or braced in some manner to prevent translation of the foundation.

- Concrete bearing assumes a minimum compressive strength (f'c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis.

- The surrounding soils provide continuous lateral support with SPT N-values greater than or equal to 4 blows per foot. Piers with exposed unbraced lengths or piers placed in weaker or fluid soils should be evaluated on a case by case basis.

- The maximum recommended drive load is not exceeded during installation.

3.7 Corrosion Protection

Foundation Supportworks' hydraulically-driven steel push pier systems have been designed following the guidelines of ICC-ES AC358 and ICC-ES AC406 for corrosion loss rates and design period (50 years).

The starter and pier tube sections used for Models PP288 and PP350 push pier systems are manufactured with a triple-layer, in-line galvanized coating. The triple-layer coating process consists of a uniform galvanized zinc coating, an intermediate conversion coating to inhibit the formation of white rust and enhance corrosion resistance, and a clear organic polymer top coating which interacts with the intermediate coating to further enhance the corrosion protection and durability. The insides of the PP288 and PP350 starter and pier tubes also have a zinc-rich coating. Although the triple-layer coating offers significant corrosion resistance, the process is not specifically recognized within AC358 and AC406. The PP288 and PP350 push pier system capacities provided in Appendix 3A are therefore conservatively based on corrosion losses for plain steel.

The starter and pier tube sections for the PP237 and PP400 push pier systems are available in either plain steel or with a hot-dip galvanized coating in accordance with ASTM A123. The FSI push pier system brackets, external sleeves, and pier caps are also available as either plain steel or hot-dip galvanized (ASTM A123). The bracket hardware is electro-plated in accordance with ASTM B633.

Additional corrosion protection may be achieved by filling the pier tubes with a fluid grout or concrete mix following installation. With a dry hole and using a neat or sand mix, the grout may be gravity fed from the top of the pier. In a wet hole, to prevent segregation of aggregate, or to prevent bridging effects, the grout may be placed by tremie tube from the bottom of the pier toward the surface. Pier tubes can also be filled with concrete or grout at intervals during the installation; i.e., piers are gravity filled from the top after advancement of every one or two sections.

3.8 Push Pier Installation Equipment

The equipment needed to first drive the piers individually and then transfer the structural load to the multiple pier locations consists of hydraulic cylinders, a hydraulic pump, a remote valve assembly (or other control device), hoses and fittings, drive stands and lift cylinder assemblies.

Safety precautions must be followed when using high pressure hydraulics. The pressure rating of each system component must be verified prior to use to ensure that all components meet the maximum pressure rating required during the installation. Hoses and fittings should be checked periodically for damage and replaced when in question. Failure to follow manufacturer's specifications may result in equipment failure and/or personal injury.

3.8.1 Drive and Lift Cylinders

Hydraulic drive cylinders (also commonly referred to as “rams”) are used to push (drive) pier sections below the existing footing until the target ultimate pressure or load is achieved or until the structure begins to mobilize (lift response). Hydraulic lift cylinders are then used at each of the multiple pier locations to provide the final lock-off load for stabilization or to lift the structure, if required. FSI offers three drive cylinder (FS425DC, FS35CSDC and FS35DC) and two lift cylinder (FS256LC and FS35LC) options. The FS35CSDC drive cylinder is a shorter version of FS35DC for use in limited headroom and crawl space applications. The FSI drive and lift cylinders are shown in *Figures 3.8.1.a* *and 3.8.1.b*. Drive and lift cylinder specifications are provided in Appendix 3B (Drive Stand Specifications) and Appendix 3C (Lift Assembly Specifications), respectively.

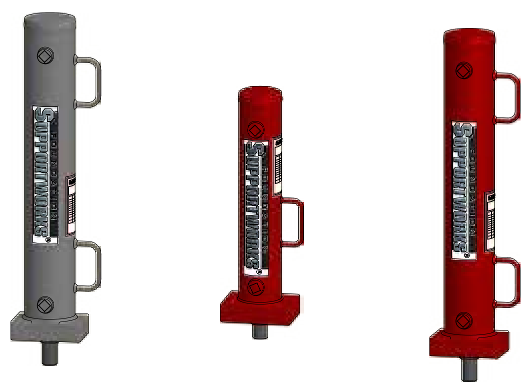


Figure 3.8.1.a FSI drive cylinders. Left to Right; FS425DC, FS35CSDC and FS35DC (no relative scale).

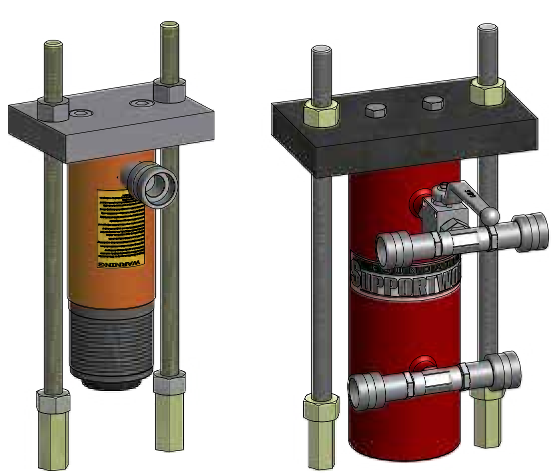


Figure 3.8.1.b FSI lift cylinder assemblies. Left to Right; FS238LCA (with FS256LC) and FS35LCA.

3.8.2 Hydraulic Pumps

Hydraulic pumps used to drive pier tube can be electric or gasoline powered. The selection of the pump unit should take into consideration the maximum drive pressure (ultimate pier capacity) required and the rate of pier installation desired. The flow rate of the hydraulic pump will affect how fast piers can be advanced with higher flow units allowing faster pier installation. That said, gasoline pumps generally provide greater flow than electric pumps and are therefore preferred for deep foundation pier installation. Electric pumps are often recommended for the stabilization/lift operation of foundation pier installation and for both the driving and stabilization/lift operations of slab pier installation. With lower flow rates, electric pumps install piers slower and provide greater control to reduce potential overstressing of the concrete slab or footing should sudden spikes in pressure/load occur. FSI offers two models of electric pumps and two models of gas pumps.

The effective area of the hydraulic drive cylinder used will also have an effect on installation speed. FSI drive cylinders FS35DC and FS35CSDC have an effective area of 9.62 in^2 while operating in extension mode and 6.48 in^2 while operating in retraction mode. FSI drive cylinder FS425DC has effective areas of 14.18 in^2 and 11.04 in^2 for the extension and retraction modes, respectively. With different effective areas, the drive cylinders will have different extension and retraction rates

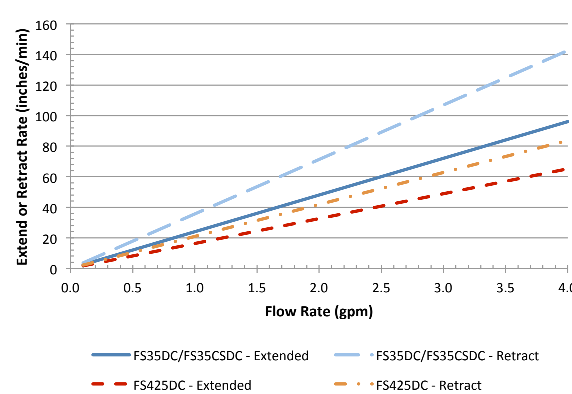


Figure 3.8.2.a FSI drive cylinder extend and retract rates

at similar flow rates, as shown in *Figure 3.8.2.a*. Cylinders with less effective area will have faster extension or retraction rates than cylinders with more effective area. The effective area of the cylinder in retraction mode is less than the effective area in extension mode due to the presence of the internal drive rod.

3.8.2.1 FSI Gasoline Powered Hydraulic Pumps

FSGP5A is a single-stage gear pump driven by a variable speed, 11 HP, 4-cycle gasoline engine. Per manufacturer specifications, the pump unit is capable of a 629 in³/min (2.7 gpm) flow rate at the maximum rated output pressure of 4,000 psi and a speed of 3,600 rpm. FSGPZG6A is a dual-stage piston pump driven by a variable speed, 13 HP, 4-cycle gasoline engine. In the first stage of operation, with an output pressure up to about 1,800 psi, the pump produces hydraulic flow of at least 850 in³/min (3.7 gpm). The flow rate then drops significantly through the second stage of operation to 200 in³/min (0.87 gpm) at the maximum rated output pressure of 10,000 psi and a speed of 3,600 rpm. Pump performance is affected by the output pressure and pump speed. The output pressure may also be limited by the pressure relief valve setting. The pump output flow versus pressure curves for both motors at a speed of 3,600 rpm are illustrated in *Figure 3.8.2.1.a*.

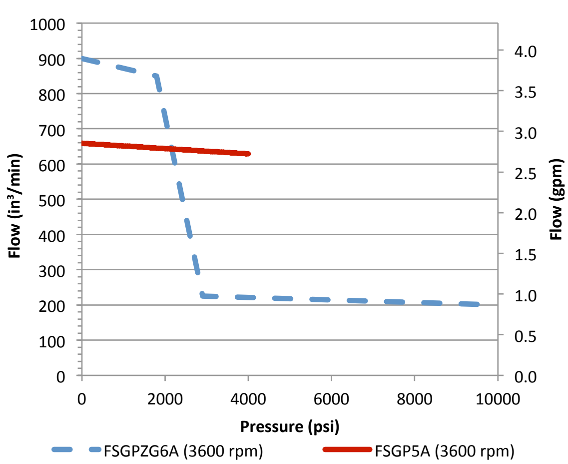


Figure 3.8.2.1.a FSI gasoline powered hydraulic pump curves

Safety precautions must be followed when using gasoline powered equipment. Ignition sources must be kept away from the gasoline tank and any gasoline vapor or fluid leakage must be stopped. Exhaust fumes from operation of gasoline engines require proper ventilation, particularly when used indoors or in confined spaces.

3.8.2.2 FSI Electric Powered Hydraulic Pump Units

The FSEPZU4A hydraulic pump is driven by a single speed, 1.125 HP, 110 volt electrical motor capable of a 60 in³/min (0.26 gpm) flow rate at the maximum rated output pressure of 10,000 psi. The FSEP554A hydraulic pump is driven by a single speed, 1.125 HP, 110 volt electrical motor capable of a 56 in³/min (0.24 gpm) flow rate at the maximum rated output pressure of 10,000 psi. As can be seen by the pump performance curves in *Figure 3.8.2.2.a*, the FSEPZU4A and FSEP554A pump units have similar flow performance above an output pressure of 1,500 psi. Again, pump performance is affected by the output pressure and pump speed and the output pressure may be limited by the pressure relief valve setting.

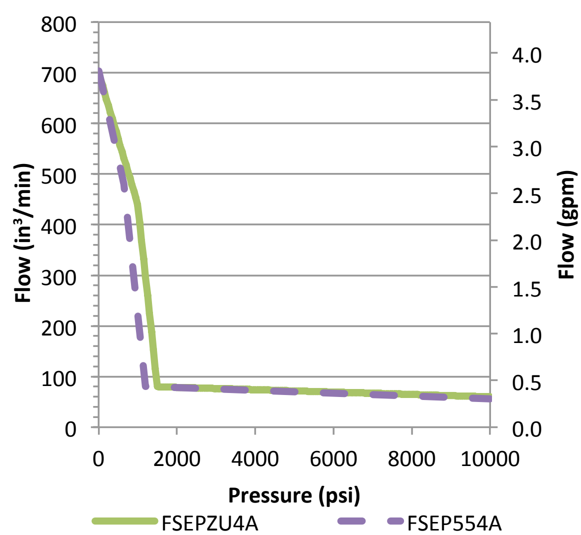


Figure 3.8.2.2.a FSI Electric Powered Hydraulic Pump Curves

3.8.3 Remote Valve Assembly

The remote valve assembly is used to control the hydraulic fluid pressure from a gasoline pump to the drive cylinder during foundation pier and slab pier installation. The remote valve assembly is typically placed near the bracket and drive stand assembly to provide greater control and response during the driving operation. The FSI remote valve assembly is shown in *Figure 3.8.3.a*. The remote valve assembly is not used with an electric pump.

Figure 3.8.3.a
Remote valve assembly

3.8.4 Hoses and Fittings

Hoses and fittings should be designed for the maximum system pressure. Hose lengths of 10, 50 and 100 feet are available to accommodate either sequential connection of lift cylinder assemblies or connection through a manifold system. FSI offers both flush face and threaded fittings for the cylinder and pump connections.

3.8.5 Drive Stands

Drive stands provide the means for advancing pier tubes through foundation and slab brackets. The drive stand is positioned over and then secured to the bracket. Hydraulic drive cylinders or rams are then set within the top fixture of the drive stands. Lengths of the drive stand legs have been specifically designed for the FSI standard pier tube lengths of 36 inches. The drive stand used for PP288 installation within a crawl space has shorter legs designed for 18-inch long pier sections. There are several drive stand assembly options available for the multiple pier sizes and bracket details. *Figure 3.8.5.a* shows crawl space and standard drive stand assemblies for the PP288 system. The FSI drive stand assembly specifications, along with general dimensioning, are included in Appendix 3B.

Figure 3.8.5.a Crawl space and standard drive stand assemblies for the PP288 push pier system

3.8.6 Lift Cylinder Assemblies

Lift cylinder assemblies are connected to the thread rod or coil rod of the bracket assembly after the pier cap has been placed. Refer back to *Figure 3.8.1.b*. The lift cylinder assemblies provide the final application of force to first stabilize and then lift the structure. After the stabilization or lift force has been applied, the top nuts on the thread or coil rods of the bracket assembly are tightened down to the pier cap, thereby locking off the load. See Appendix 3C for lift assembly specifications.

3.9 Push Pier Capacity and Spacing

Foundation Supportworks offers multiple pier sizes and bracket assembly combinations to provide solutions for varying applications and design loads. A main design consideration for eccentrically-loaded retrofit systems is minimizing bracket rotation. This is accomplished not only by designing a stiffer pier system, but also by the system's interaction with the surrounding soil and the structure. Sections 3.3.1.1 and 3.3.1.2 discuss the bending forces that are generated by the eccentric loading condition and how the FSI external sleeve resists the bending force below the bracket. The near-surface soils surrounding the bracket, the external sleeve and the upper sections of pier tube therefore act to resist and dissipate the bending. Finite element analysis software was used to analyze how the external sleeve and pier interact with various soil types and strengths. The standard 48-inch long external sleeve was thereby selected to provide an efficient use of additional steel to resist most, if not all, of the bending force when piers are installed within somewhat typical near-surface soil conditions; i.e., loose sands and medium stiff clays, or stronger. Although laboratory testing cannot exactly duplicate actual installed field conditions with all possible soil types and strengths, the results from the standard test method utilized (ICC-ES AC406) generally confirmed these calculated capacities.

Retrofit bracket testing in accordance with AC406 also considers interaction of the bracket with a concrete block of known compressive strength (2,500 psi). Testing pier systems against concrete is completely logical as it includes concrete failure as a potential failure mechanism of the "system". Bracket testing within a rigid steel frame does little to simulate field behaviors and failure conditions, and these capacities could rarely be duplicated in the field without first buckling the pier and/or breaking the concrete footing. FSI determines push pier capacities by testing in general accordance with AC406 and, as a result, our pier systems may appear to be conservatively rated versus other published system capacities. Even so, AC406 is an appropriate test method for determining push pier system capacities, and the only standard currently available.

Push pier system ultimate capacities may be limited by the ability of the structure and surrounding soil to provide the necessary reaction to drive the piers. Light structures or structures with shallow footings may start to mobilize before the target drive load is achieved. In such cases, it may prove beneficial to excavate small, shallow holes at the pier locations, instead of a full excavation, to allow as much soil load as practical to remain around and beneath the footing. The soil load can contribute significantly to "hold" a light structure down in order to achieve target pressures/loads. When a structure experiences early lift, the project engineer should evaluate if the drive pressure/load is adequate, if adjustments can be made to the proposed piering plan, or if a change to retrofit helical piers should be considered. Helical piers are installed by the application of torque with machines independent of the structure. Helical piers are discussed in Chapter 2.

A structural assessment should be performed prior to installation to determine if the existing footing, stem wall or floor slab can resist the estimated final drive force without structural damage. Overstressing the concrete can be prevented or at least minimized by following proper techniques and best practices for footing preparation and pier installation. The contractor should carefully monitor the installation and release the load at the first sign of foundation or slab distress.

Stone or cobble foundations, brick foundations, or foundations that are severely broken or deteriorated may not be good candidates for retrofit foundation piers. Foundations and slabs must be able to span between pier locations for the system to be effective. Pier locations and pier spacing are often determined by the spanning capability of unreinforced or under-reinforced footings, foundation walls and floor slabs, and not by the pier's capacity. Monolithic footings, footings with short stem walls or footings with masonry stem walls may require closer pier spacing and/or additional support at the bracket locations. Spanning capability of a footing may be improved by using structural steel angles, plate, tube, etc. sandwiched between the bottom of the footing and the horizontal bearing plate of an under-footing bracket. In more severe cases; e.g., stone, cobble, brick and highly deteriorated foundations, the footings can be temporarily undermined in short sections to construct a continuously reinforced concrete grade beam. The grade beam would then provide adequate spanning capability for the installation of the retrofit piers.

With all the discussion above, a push pier system can still only provide support for the structure if competent soils are encountered at the pier tip. Typically, SPT N-values of 35 to 40 blows per foot for clay soils and 30 to 35 blows per foot for granular soils are needed to provide the necessary end bearing resistance for light to moderate push pier loads. See Appendix 2G for additional information regarding geotechnical considerations for push pier systems.

Technical specifications and capacities for FSI push pier systems are provided in Appendix 3A.

3.9.1 Factor of Safety

The push pier system develops a factor of safety against pier settlement by the pier installation methods used and the sequence with which multiple piers are driven and then re-loaded. Piers are first driven individually using the maximum weight of the structure and any contributory soil load as the reaction. The pier gathers load from adjacent sections of the foundation, not just in the immediate area of the pier. The more rigid the structure, the more load can be transferred to the pier during the drive process. It is for this reason, along with consideration of contributory soil load against the foundation, that piers can be driven to loads greater than the calculated service loads.

The drive or installation force on the piers is determined by calculating the structural load (dead plus live) and the soil load on each pier, then multiplying by a factor of safety. Factors of safety of 1.5 to 2.0 are commonly used for push pier systems since the drive and lock-off loads are easily measured and verified using hydraulic cylinders, pumps and gauges. Foundation Supportworks does not recommend the use of bottle jacks for the drive or lift operations of a push pier installation. Loads applied with bottle jacks are unknown and not easily determined. Higher factors of safety may be considered at the discretion of the project engineer or as dictated by local codes.

Piers are driven to the calculated "ultimate" load, or until lift of the structure occurs. After all of the piers are driven, the piers are connected in series with hydraulic lift cylinders and re-loaded to either the design service load to stabilize the structure, or until the desired lift is achieved. The total reaction load is then distributed over the multiple pier locations. The final factor of safety against pier settlement at each pier location is calculated by dividing the drive load by the lock-off load.

As discussed in Section 3.3.2.1, additional skin friction develops over time as the soils heal around the pier shaft. The factor of safety generally increases with an increase in frictional capacity.

3.9.2 Bolting the Under-Footing Bracket to the Foundation

Foundation Supportworks does not require nor recommend bolting of under-footing brackets to a concrete foundation with expansion or adhesive anchors. Experience has shown that bolting to unreinforced or under-reinforced concrete routinely causes concrete to crack and spall during installation of the expansion or adhesive anchors, or during the repeated loading and unloading procedure of driving piers. At best, bolting provides little benefit to the pier capacity and stability while introducing the potential to weaken the system by damaging the footing. Holes are included in the brackets to be used at the discretion of the installer or if a project engineer or building official requires that the piering system be positively attached to the structure.

Actually, the manner in which a push pier system is loaded and supported would tend to cause the bracket to push against the structure, not pull away from it. At the same time, however, while the bracket is pushing against the structure, it also tends to rotate toward the structure. If the pier system does not have adequate stiffness, then the tendency for excessive bracket rotation will be evidenced by the bearing plate being pried away from beneath the structure. This phenomenon does not mean that the overall pier system is translating away from the structure. Instead, it means the pier needs to be much stiffer. The stiffness of FSI push pier systems greatly reduces this rotational tendency and precludes the need to positively attach the bracket to the structure. When such an attachment is made due to preference or local requirements, FSI recommends the expansion or adhesive anchors be installed after completion of the piering operations. Anchors were not used when the pier systems were tested in accordance with AC406.

3.10 Under-footing Push Pier Installation

The steps for under-footing push pier installation include footing preparation, bracket mounting, drive stand and drive cylinder attachment, pier tube driving, application of the final drive force, attachment of the lift cylinder assembly, structural lift (if applicable) and load transfer and lock off.

Safety precautions must be followed prior to and during excavation activities. Locate underground utilities prior to excavation and perform excavations at a distance away from utilities as mandated by the utility owner. Follow OSHA guidelines for trench safety.

Step 1 Footing Preparation

Excavation is required in most cases to expose the concrete footing and prepare it for under-footing bracket placement. The footing may be accessed from either inside or outside the structure using isolated “pocket” or continuous excavations. For structures with basements, it may be advantageous to access the footing from within the basement of the structure by first removing sections of the basement floor slab. General steps for footing preparation include:

- For shallow pocket excavations, soil should be hand or machine-excavated from against the footing and foundation wall within an area approximately 3 feet square and to a depth approximately 9 to 13 inches below the bottom of footing. These excavations are made at each pier location. Alternatively, soil could be removed completely with a trench excavation. Trench excavations are more common when a lift is required.
- The soil under the footing and foundation wall is removed to a distance that allows bracket placement *(Figure 3.10.a)*.
- Notch spread footings 16 to 22 inches wide (depending upon the width of the retrofit bracket) and approximately flush with the face of the foundation wall. Notching of footings may

Figure 3.10.a Removing soil from beneath the footing with a chisel attachment

Figure 3.10.b1 Smoothing the vertical and horizontal bearing surfaces with a chipping hammer

not be necessary depending upon the footing geometry, strength, steel reinforcement and the proposed piering plan. Notching of the footing reduces the eccentricity between the applied load and the pier section. The proposed pier plan may consider pier installation on opposing sides of the footing to better balance support of the loads. Installing piers on opposing sides of the footing in pairs or in a staggered configuration can be an acceptable alternative to notching. The design professional of record should approve the notching, particularly when notching will cut steel reinforcement. The outline of the notch is typically first made by drilling a series of closely-spaced holes with a concrete drill bit. After the notch outline has been made, a chipping hammer or jack hammer can be used to impart energy to the perforated section, causing it to separate from the rest of the footing. Drilling the holes prior to using the chipping hammer or jack hammer also reduces the likelihood of concrete spalling from under the footing.

- Smooth the concrete surfaces with a chipping hammer or other tool to produce similar results. The vertical and horizontal surfaces of the footing and foundation wall must be smooth and clean to allow full contact with the vertical and horizontal bearing plates of the under-footing bracket *(Figures 3.10.b1 and 3.10.b2)*. Footing preparation should be completed to provide proper bracket/pier alignment *(Figure 3.10.b3)*.

Figures 3.10.b2 Checking underside of footing for proper preparation

Figure 3.10.b3 Completed footing excavation and preparation; ready to position bracket

Step 2 Positioning the Bracket

- The bracket is placed under the footing and raised into position with the horizontal and vertical bearing plates in full contact with the concrete surfaces. The bracket is temporarily held in place using wood cribbing *(Figure 3.10.c1)*. Alternatively, a bracket RAYser™ is available from Foundation Supportworks for the more commonly used brackets. The bracket RAYser consists simply of a U-shaped plate with bottle jacks to position the bracket and hold it snug against the concrete during the initial pier installation process *(Figure 3.10.c2)*.

Figure 3.10.c1
Temporary bracket support with wood cribbing

Note: *For the Model PP400 system without an external sleeve, the starter tube with the friction reduction collar needs to slide up through the bottom of the bracket before setting the bracket against the footing.*

- FSI under-footing brackets do not require mechanical anchorage to the concrete foundation. The published capacities are based on testing and analyses without anchors. There are bolt holes that may be used to mount the bracket to the concrete with expansion or adhesive anchors, if needed to meet the project specifications. It should be noted, however, that the use of anchors to mount the under-footing bracket may cause concrete spalling and cracking from the repeated loading and unloading process during pier installation and lock-off. Additionally, drilling the anchor holes could compromise the integrity of steel reinforcement. If mounting of the under-footing bracket with anchors is required, FSI recommends anchoring the bracket after the system has been locked off.

Figure 3.10.c2
Bracket RAYser™ support system

Step 3 Mounting the Drive Stand and Hydraulic Drive Cylinder

- Slide the exterior sleeve over the starter tube and insert the sleeve and starter together through the bracket *(Figures 3.10.d1 and 3.10.d2)*. Care must be taken that the sleeve and starter are properly aligned and extend past both the top and bottom plates of the bracket. The sleeve and starter could also be placed after mounting the drive stand to the bracket. Installers may find it easier, however, to set the sleeve and starter without being restricted by the drive stand legs.

Figure 3.10.d1
Exterior sleeve slid over the starter tube

Figure 3.10.d2 Sleeve and starter inserted together through the bracket

- The drive stand is fitted to the bracket and secured with coil rod (PP237 system), L-pins (PP288 system) or bolts (PP350 and PP400 systems). *Figures 3.10.e1 and 3.10.e2* show setting of the drive stand for the PP288 push pier system and securing it to the bracket with L-pins.

Figure 3.10.e1 Drive stand for PP288 push pier system fitted to the bracket

Figure 3.10.e2 PP288 drive stand secured to the bracket with L-pins

- Slide the hydraulic drive cylinder into the top fixture of the drive stand and lock it in position with the coil rod and nuts *(Figures 3.10.f1 and 3.10.f2)*.

Figure 3.10.f1 Hydraulic drive cylinder placed within top fixture of drive stand

Figure 3.10.f2 Drive cylinder locked in position with coil rod and nuts

- Connect the hydraulic hoses to the inlet and outlet of the drive cylinder and the inlet and outlet of the remote valve assembly *(Figure 3.10.g)*.

Figure 3.10.g
Connect hydraulic hoses

- Align the drive stand by activating the hydraulics and extending the drive cylinder rod to make slight contact with the starter tube section. Use a digital level, protractor or other device to check alignment of the drive stand, sleeve, starter and bracket *(Figure 3.10.h)*. Adjust the alignment as necessary for the bracket system being utilized; i.e., 2-degree or vertical brackets. Proper footing preparation is critical for setting the bracket and system at the correct installation angle. Temporary cribbing may be used between the drive stand and the foundation wall to set the correct installation angle *(Figure 3.10.i)* while advancing the starter tube and external sleeve.

Figure 3.10.h Checking drive stand alignment with digital level

Figure 3.10.i Temporary wood cribbing used to set installation angle

Step 4 Pier Tube Installation

- Drive the external sleeve and starter tube together until the welded collar or trumpeted end of the sleeve is seated at the top of the bracket. Pier tubes are then coupled *(see coupling detail in Figure 3.3.3.1.a)* and pushed through the external sleeve *(Figure 3.10.j)*. The standard length for pier tubes is 36 inches for all of the FSI push pier systems. Drive cylinders FS35DC and FS425DC have 22-inch strokes. The PP237 and PP288 crawl-space pier tubes have lengths of 18 inches and are generally pushed with FSI drive cylinder FS35CSDC, which has a 13-inch stroke. The drive process for the sleeve and starter tube, the standard 36-inch pier tubes, and the 18-inch crawl-space pier tubes therefore requires a two stage process and the use

Figure 3.10.j Installing PP288 push pier tube

of a driving tube tool. When the maximum cylinder stroke has been reached, the cylinder is retracted, the drive tube tool is set in place, and the push is completed to the top of the bracket or external sleeve *(Figure 3.10.k)*.

Figure 3.10.k Driving tube tool set in place for second stage of push

- Record the drive pressure at final stroke of each pier tube section *(Figure 3.10.m)*.

Figure 3.10.m Pressure readings are recorded for each pier tube

Safety precautions must be followed when driving pier tube sections to ensure that body and clothing are away from pinch points. Take caution and avoid over-stroking the cylinder rod which may result in a rapid increase in pressure, possibly resulting in cylinder damage or personal injury.

- Once the pre-determined drive pressure is achieved or the structure starts to lift, the pressure is released from the hydraulic system and the drive stand and drive cylinder are removed from the bracket. The drive process is repeated at each of the proposed pier locations.

Step 5 Assembling the Bracket and Mounting Lift Cylinders

- The final pier tube extending up from the bracket will often have to be cut to the desired elevation. FSI push pier systems with an external sleeve offer a tremendous benefit over systems without external sleeves. The final pier tube is protected from bending and the high pinching forces common between short brackets and push pier tubes. The final pier tube is then easily removed from within the external sleeve and placed in a chop saw to achieve a square cut *(Figure 3.10.n)*. Alternatively, a tube cutting guide can be positioned over the in-place pier section and a cut can be made with a reciprocating saw or portable band saw. The last pier tube section is typically cut to a length to extend above the external sleeve approximately 4½ inches. The cutoff length may vary depending upon the amount of structural lift anticipated. The removed pier section is replaced after the cut is made.

Figure 3.10.n Cutting pier tube to desired length with a chop saw

- The pier cap is set on the pier tube and two threaded rods or coil rods are fed through the holes of the pier cap and bracket. The pier cap is connected to the bracket with nuts on each end of the rods *(Figure 3.10.o1)*. There should be adequate thread left above the top nuts above the pier cap to allow coupling of the lift cylinder assembly to the rods *(Figure 3.10.o2)*.

Figure 3.10.o1 Installation of pier cap plate with threaded rods and nuts

Figure 3.10.o2 PP288 bracket ready for lift cylinder assembly

- Lift cylinder assembly rods are coupled to the bracket assembly rods as shown in *Figure 3.10.p1*. Adjacent lift cylinders (piers) are connected in series to provide uniform application of load *(Figure 3.10.p2)*.

  Note: *The hydraulic system shown in the figures is technically a "parallel" system. However, it is common to say that the piers are connected in "series," which simply means hydraulic lines run between adjacent pier locations and they are often all connected together with one set up.*

Figure 3.10.p1
Connection of lift cylinder assembly

Figure 3.10.p2 Hydraulic system connected to provide uniform application of load

Step 6 Structural Lift and/or Lock Off

- Hydraulic pressure is applied to the system to either lift the structure to the proper elevation or provide the required lock-off pressure/load. The lock-off pressure/load is generally the service load or design working load per pier. It may be necessary to remove the soil from above the footing if pocket excavations were initially made. Removal of as much soil load as possible around the foundation will increase the potential to achieve a desired lift.

- The system is first equalized by opening the valves at each cylinder in sequence and adjusting the system pressure. The system should be equalized to a pressure on the order of 1,000 psi.
- Slowly raise the pump pressure to raise the foundation. Monitor the lift at each pier location and after achieving proper lift, close the valve to the top of the cylinder. If the piers are for stabilization only, close the valves as soon as noticeable movement occurs.
- When the structure has been lifted to the proper elevation or the piers reloaded to the required lock-off load (typically the design working load), the load is locked off to the piers by tightening the two nuts down to the top of the pier cap.
- The pressure is released from the hydraulic system and the lift cylinder assembly is removed from the bracket *(Figure 3.10.q)*. The pier system installation is essentially complete. Permanent benchmarks may be established within and beyond the work area, if approved by the owner or client. These monitoring points would allow relative movements to be measured in the future, if necessary *(Figure 3.10.r)*.

Figure 3.10.q
System locked off; pier installation complete

Figure 3.10.r Benchmark established with a shallow hole drilled into the concrete block

Step 7 Backfill and Clean Up Work Area

- Backfill excavations and properly compact with a mechanical tamper *(Figure 3.10.s)*. This may also be an ideal time to improve the drainage system around structures with below grade living areas or working space. New drain pipe can be installed along with free-draining granular backfill provided the drain pipe is connected to an interior sump system or drained by gravity to daylight discharge points. Interior pier installation may require concrete patching and finishing after soil compaction *(Figure 3.10.t)*.

Figure 3.10.s
Backfill excavations and compact soil

Figure 3.10.t
Concrete placement at interior pier locations

3.11 PP288 Flush-Mount Push Pier Installation

The PP288 flush-mount bracket system may be used for applications where poured concrete elements such as a foundation wall, column, pile cap or grade beam have adequate strength, thickness and vertical dimensions to allow proper attachment of the bracket. PP288 flush-mount system capacities are provided in Appendix 3A for systems using either expansion or adhesive anchors.

Step 1 Concrete Preparation

- Excavation may be necessary to expose the vertical face of the concrete. If the bracket is mounted on a foundation wall or column above a spread footing, the concrete footing would have to be cored through or removed entirely to allow for advancement of the pier tube sections. The vertical face of the concrete to receive the bracket should be smooth of surface irregularities and free of structural cracks. A thin layer of leveling compound could be considered to create a smooth flat surface prior to mounting the bracket.

  Safety precautions must be followed prior to and during excavation. Locate underground utilities prior to excavation activities and perform excavations at a distance away from utilities as mandated by the utility owner. Follow OSHA guidelines for trench safety during excavation and installation activities.

Step 2 Mounting the Bracket

- The flush-mount bracket is secured to the concrete vertical face using eight (8) ¾-inch diameter anchors. Rather than attempting to position and hold the bracket in place, a template of the bracket bolt holes could be considered to mark the anchor locations.

Steps 3 - 7

- The remaining steps for flush-mount push pier installation are similar to those for the under-footing system described in Section 3.10.

3.12 PP288 Slab Push Pier Installation

The PP288 slab push pier system is used to lift and/or stabilize settling concrete floor slabs. Monometer survey equipment, a laser level, a zip level, or other suitable equipment should be used to identify low areas in the slab. Slab piers should be located at these identified low points. Slab piers should also be considered either centered on or on alternating sides of significant floor cracks to ensure an even lift. Voids beneath a stabilized and lifted slab should be filled with suitable material such as a cement grout or PolyLEVEL® polyurethane foam.

Step 1 Slab Preparation

- Mark the slab pier locations with consideration to possible underground utilities, overhead obstructions, maximum pier spacing, existing floor cracks and lift requirements. Small paper plates may be used to mark preliminary slab pier locations since the plates can be easily moved around the slab *(Figure 3.12.a)*. Slab pier

Figure 3.12.a
Marking slab pier locations

CHAPTER 3
HYDRAULICALLY-DRIVEN PUSH PIERS

spacing can be estimated using the Slab Pier Spacing Guide of *Figure 3.12.b*. A grid pattern spacing is provided for various slab thickness and live load combinations. The guide also considers unreinforced concrete slabs having a minimum concrete strength of 2,500 psi.

		Live Load				
		30 psf	40 psf	50 psf	60 psf	80 psf
Slab Thickness	3.5”	5’-0”	4’-6”	4’-3”	4’-0”	3’-9”
	4.0”	5’-6”	5’-0”	4’-9”	4’-6”	4’-3”
	4.5”	6’-0”	5’-6”	5’-3”	5’-0”	4’-6”
	5.0”	6’-6”	6’-0”	5’-9”	5’-6”	5’-0”
	6.0”	7’-3”	7’-0”	6’-6”	6’-3”	5’-9”
	8.0”	8’-9”	8’-6”	8’-3”	7’-9”	7’-3”

Typical for Residential

Figure 3.12.b Slab pier spacing guide

- Core 8-inch diameter holes in the concrete slab *(Figure 3.12.c)*. Adjust slab pier locations and spacings based on the actual concrete thickness determined at the first cored hole. Remove the concrete cores and use a hand probe to check for underground obstructions *(Figure 3.12.d)*. Using a small hand tool, excavate all material beneath the slab to at least 4 inches below the bottom of the slab and extending at least 3 inches beyond the edges of the cored hole. Check with your hand to confirm that the bottom of slab is relatively smooth and free of subgrade material *(Figure 3.12.e)*.

Figure 3.12.c
Concrete coring

Figures 3.12.d
Probing for utilities or obstructions

Figure 3.12.e
Excavating beneath the slab

Safety precautions must be followed during concrete coring to ensure the core drill is securely mounted to the floor slab and proper safety equipment including safety glasses are worn during coring operations. Immediately remove any water from the floor when coring to reduce potential for electrical shock. Keep body parts and other objects away from core bit during operation.

Step 2 Assembling the Bracket Below the Slab

- The PP288 slab pier bracket assembly consists of one (1) main plate, two (2) wing plates, two (2) 14-inch long ⅝-inch diameter threaded rods, four (4) ⅝-inch hex nuts and one (1) pier cap. Set the main plate (first) and the wing plates (second) through the cored hole. Cover the welded nuts on the bottom of the main plate with duct tape prior to placement through the cored hole to ensure clean threads for later insertion of the threaded rods. Locate the wing plates above the main plate so that the wing plate holes line up with the holes in the main plate *(Figure 3.12.f)*. Align the straight edges of the two wing plates to be essentially parallel with each other.

Figure 3.12.f Main plate and wing plates positioned and aligned beneath the slab

- Install hex nuts on one end of the threaded rods leaving about 2 ½ inches of thread below the nuts. Insert the threaded rods through the wing plate holes and thread them into the weld nuts below the main plate. Turn the rods by hand until the nuts on the threaded rods are seated against the top surface of the wing plates. Continue to tighten the nuts with a deep well socket to fasten the wing plates firmly to the main plate *(Figure 3.12.g)*.

Figure 3.12.g
Threaded rods installed

Step 3 Mounting the Drive Stand and Drive Cylinder

- Cut the coupler extension off a standard 36-inch long pier tube to use as your starter tube *(Figure 3.12.h)*. Insert the "coupler" end of the starter tube through the hole of the main plate. Place the slab pier drive adaptor over the pier tube and fasten to the threaded rods using two hex nuts *(Figure 3.12.i)*. Pull up

Figure 3.12.h Starter tube made by cutting coupler end of standard pier tube

Figure 3.12.i
Slab pier drive adaptor installed

on the threaded rods to bring the main plate and wing plates against the bottom of slab. Slide the PP288 drive stand onto the slab pier drive adaptor and secure with L-pins *(Figure 3.12.j)*. Set the hydraulic drive cylinder into the top fixture of the drive stand and lock it in position with the coil rod and nuts *(Figure 3.12.k)*. Connect the hydraulic hoses.

Figure 3.12.j PP288 drive stand mounted to slab pier drive adaptor

Figure 3.12.k Drive cylinder set into top fixture of drive stand

Step 4 Pier Tube Installation

- Electric pumps are preferred for slab pier installations since the application of drive force is more easily controlled and the risk of overstressing the concrete slab during pier driving is reduced.
- Pier tubes are driven using similar procedures as outlined in Section 3.10 (Step 4), including recording of drive pressures at the end of each driven tube. The drive stand should self-align when force is applied by the drive cylinder to the pier tubes; therefore, no cribbing or alignment of the drive stand should be necessary if the floor slab was prepared properly.

  Safety precautions must be followed when driving pier tube sections to ensure that body and clothing are away from pinch points. Take caution and avoid over-stroking the cylinder rod which may result in a rapid increase in pressure, possibly resulting in cylinder damage or personal injury.
- Drive pier tubes until the required termination drive force is achieved or slab movement (flexing) in excess of about ¼ inch occurs. Care should be taken by the installer to slowly release hydraulic pressure at the end of each cylinder stroke. Once the pre-determined termination drive force is achieved or the slab starts to lift, the pressure is released from the hydraulic system and the drive stand and drive cylinder are removed from the slab pier drive adaptor. The drive adaptor is then disconnected from the threaded rods of the slab pier bracket.

Step 5 Mounting the Lift Cylinder

- The last pier tube section is pulled from the hole, cut to desired length in a chop saw and replaced. The desired top of pier elevation relative to the top of slab depends upon the slab thickness and the maximum amount of lift anticipated. If the slab will be stabilized without lifting, the top of pier tube can be approximately two inches below the top of floor slab. It is imperative that the pier tube is cut correctly to ensure that the pier cap, threaded rod and nuts are below the top of the slab after lift and/or lock-off operations.
- Place the pier cap over the threaded rods and lightly tighten it against the top of the pier tube with two ⅝-inch hex nuts *(Figures 3.12.m and 3.12.n)*. Set the lift cylinder assembly onto the pier cap *(Figure 3.12.o)*. Couple the threaded rods of the lift cylinder plate assembly to the threaded rods of the slab bracket to hold the lift cylinder in place *(Figure 3.12.p)*.

  Note: *The threaded rods of the lift cylinder assembly are larger than the ⅝-inch rods of the slab pier bracket and are not used in this application.*

Figure 3.12.m Pier tube cut to length; pier cap placed

Figure 3.12.n Lightly tighten pier cap down onto pier tube

Figure 3.12.o Lift cylinder assembly set on pier cap

Figure 3.12.p Lift plate assembly coupled to threaded rods of slab pier bracket

Step 6 Slab Lift and/or Lock off

- Connect hydraulic hoses to the top and bottom fittings on the lift cylinders *(Figure 3.12.q)*. The lift cylinders are all hydraulically connected as a system *(Figure 3.12.r)* in order to provide simultaneous lift pressure at each cylinder. The system is first equalized by opening the valves at each cylinder in sequence and adjusting the system pressure. The system should be equalized to pressures on the order of 100 to 300 psi.

Figure 3.12.q
Hydraulic connections at lift cylinder

Figure 3.12.r
Lift cylinders in series

- Slowly raise the pump pressure to raise the slab. Monitor the slab for lift at each pier location and after achieving proper lift, close the valve to the top of the cylinder. If the piers are for stabilization only, close the valves as soon as noticeable slab movement occurs. Once all the cylinder valves are closed, the piers are locked off by tightening the ⅝-inch hex nuts to the tops of the pier caps *(Figure 3.12.s)*.

Figure 3.12.s Pier load locked off by tightening nuts on top of pier cap

- The system pressure is released and the lift cylinder assemblies are removed. Cut the threaded rods flush with the tops of the hex nuts with a grinder or saw *(Figure 3.12.t)*. The tops of the nuts must be below the surface elevation of the slab.

Figure 3.12.t
Cutting the threaded rods

Step 7 Void Fill and Finish Surface

- Place concrete and trowel finish at each pier location *(Figures 3.12.u and 3.12.v)*. Voids under the slab should be filled completely with grout or PolyLEVEL® polyurethane foam. Void filling is typically completed before patching the core holes with concrete, but can be done either before or after the concrete is placed. Concrete patches should be allowed to cure before void filling.

Figure 3.12.u
Patching holes at pier locations

Figure 3.12.v
Finishing concrete

3.13 Push Pier Load Testing

The push pier installation process is essentially equivalent to performing a proof load test at each push pier location and, therefore, prescribed "official" load testing of push pier systems is not common. The piers are advanced to a drive or ultimate load, then reloaded to the specified lock-off load (typically at or near the service load) or until the desired lift is achieved. The drive and lock-off loads are easily calculated from the effective area of the hydraulic cylinder and the pressure gauge reading at the hydraulic pump. Pile head movements are not typically monitored during the proof loading process of a typical push pier installation.

The deflection to load response of the push pier system can be determined by field load testing using calibrated equipment either directly at the bracket location or with sacrificial piers installed away from the structure. The loading methodology in ASTM D1143 may be used to verify pier deformation (elastic compression and settlement) and creep effects. The preferred method is to perform the load test at the bracket location with the bracket attached to the structure. If a sacrificial test pier is used away from the structure, a compression load test frame must be constructed.

Appendix 3A

Bracket Specifications and Capacities

FS238B Bracket Specifications and Capacities
when used with the PP237 Push Pier System

Bracket:
Weldment manufactured from ¼" and ⅜" ASTM A36 plate.

Pier Tube:
Ø2.375" x 0.154" wall x 36" long
ASTM A500 Grade B or C
Yield strength = 60 ksi (min)
Tensile strength = 70 ksi (min)
Available plain or hot-dip galvanized(2)

Pier Tube Coupler:
Ø2.000" x 0.187" wall x 5" long

Pier Starter Tube:
Pier tube section with Ø2.875" friction reduction collar welded at leading end.

External Sleeve:
Ø2.875" x 0.203" wall x 30" or x 48" long with welded collar at one end.
ASTM A500 Grade B or C
Yield strength = 60 ksi (min)
Tensile strength = 70 ksi (min)

Cap Plate:
¾" x 3.75" x 5.75" ASTM A572 Grade 50 with confining ring welded to one side.

Bracket Hardware(3):
(2) - Ø⅝"x 16" long contour (coil) rod
AISI 1045, tensile strength = 120 ksi (min)
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized(2)

Allowable Bracket Capacity(4,5,6,7) R_n/Ω		
	with 30" Sleeve (kips)	with 48" Sleeve (kips)
Plain	13.0	17.0
Plain Corroded(1)	9.9	12.9
Galvanized Corroded(1)	12.1	15.9
Maximum Drive Force During Installation(7)	22.1	28.9

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC406.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

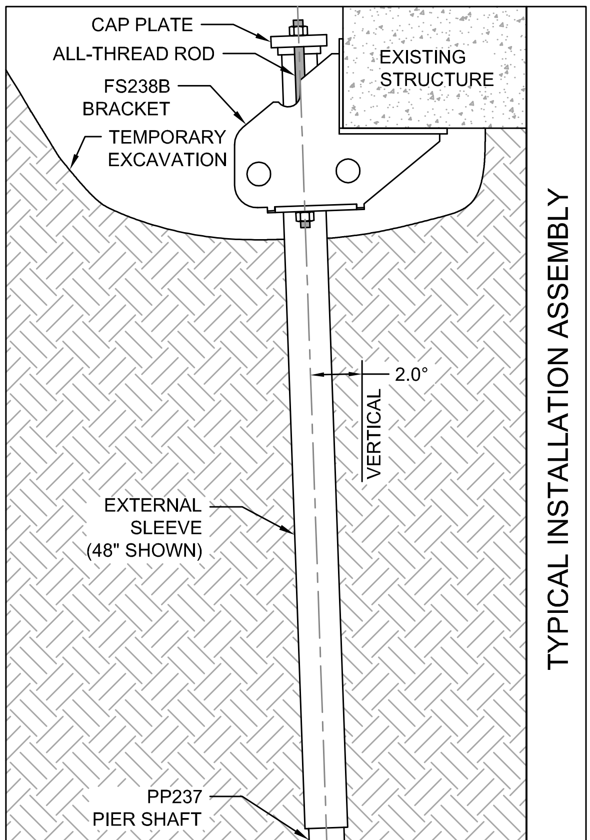
(3) Optional hardware utilizes similar sized all-thread made from Grade B7, tensile strength = 125 ksi.

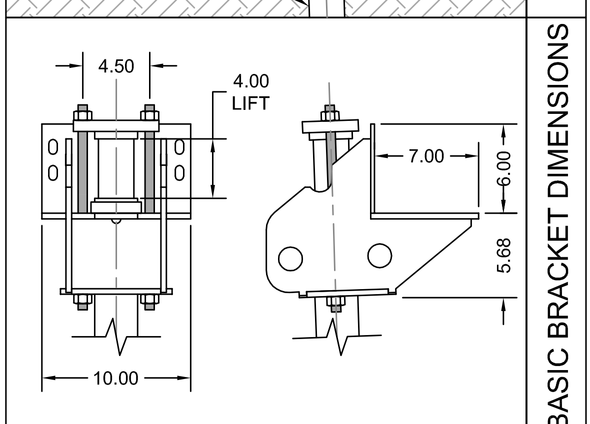
(4) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(5) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piers with exposed unbraced lengths or piers placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(6) Concrete bearing assumes a minimum compressive strength (f'c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

(7) Push Piers shall be installed with a driving force exceeding the required allowable service load by a sufficient factor of safety (FOS). FOS is most commonly between 1.5 and 2.0, although a higher or lower FOS may be considered at the discretion of the pier designer or as dictated by local code or project requirements.





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FS288B Bracket Specifications and Capacities

when used with the PP288 Push Pier System

Bracket:
Weldment manufactured from ¼", ⅜", and ½" ASTM A36 plate.

Pier Tube:
Ø2.875" x 0.165" wall x 36" long
Triple-coated in-line galvanized
ASTM A500 Grade C
Yield strength = 50 ksi (min)
Tensile strength = 55 ksi (min)

Pier Tube Coupler:
Ø2.250" x 0.180" wall x 6" long

Pier Starter Tube:
Pier tube section with machined Ø3.375" friction reduction collar pressed around leading end.

External Sleeve:
Ø3.500" x 0.216" wall x 30" or 48" long with welded collar or trumpet flare at one end.
ASTM A500 Grade B or C
Yield strength = 50 ksi (min)
Tensile strength = 62 ksi (min)

Cap Plate:
1" x 5.00" x 9.00" ASTM A572 Grade 50 with confining ring welded to one side.

Bracket Hardware(3):
(2) - Ø¾" x 16" long all-thread rod
Grade B7, tensile strength = 125 ksi (min)
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized(2)

Allowable Bracket Capacity(4,5,6,7) R_n/Ω

	with 30" Sleeve (kips)	with 48" Sleeve (kips)
Plain	23.9	36.7
Plain Corroded(1)	18.5	28.4
Grout Filled Corroded(1)	20.9	32.1
Maximum Drive Force During Installation(7)	48.1	60.0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC406. Grout filled piers consider a loss in thickness at the outside diameter only.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

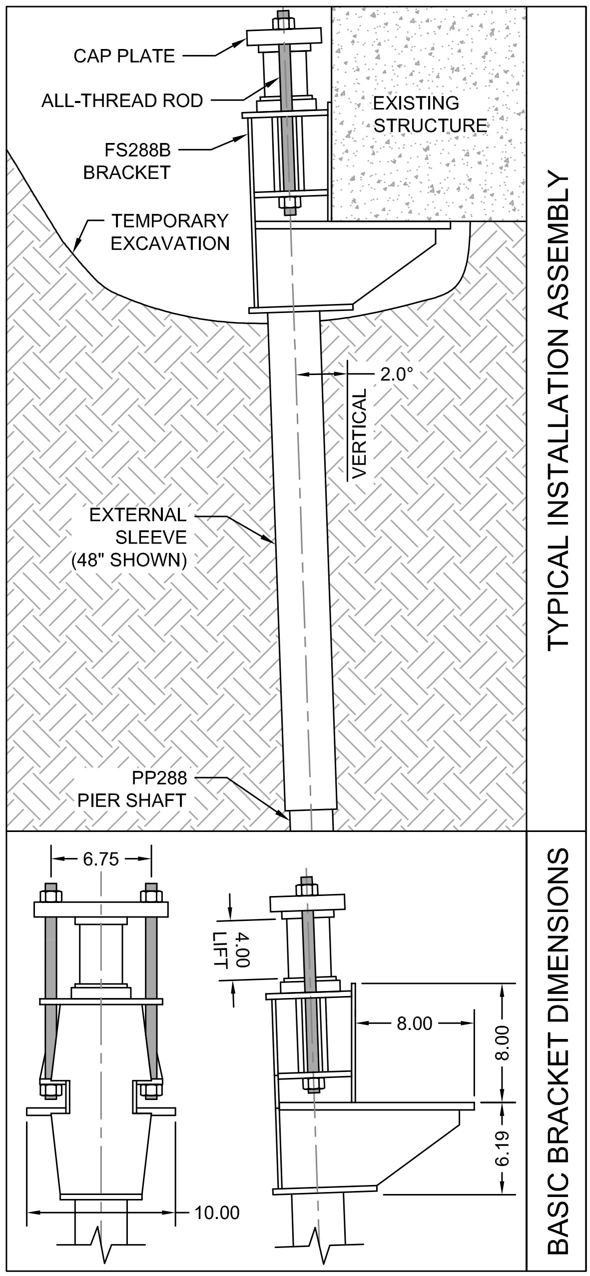
(3) Optional hardware utilizes similar sized contour (coil) thread made from AISI 1045, tensile strength = 120 ksi. Slightly lower tensile strength material does not govern the listed capacities.

(4) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(5) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piers with exposed unbraced lengths or piers placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(6) Concrete bearing assumes a minimum compressive strength (f'c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

(7) Push Piers shall be installed with a driving force exceeding the required allowable service load by a sufficient factor of safety (FOS). FOS is most commonly between 1.5 and 2.0, although a higher or lower FOS may be considered at the discretion of the pier designer or as dictated by local code or project requirements.



APPENDIX 3A
BRACKET SPECIFICATIONS AND CAPACITIES

FS288BV Bracket Specifications and Capacities
when used with the PP288 Push Pier System

Bracket:
Weldment manufactured from ¼", ⅜", and ½" ASTM A36 plate.

Pier Tube:
Ø2.875" x 0.165" wall x 36" long
Triple-coated in-line galvanized
ASTM A500 Grade C
Yield strength = 50 ksi (min)
Tensile strength = 55 ksi (min)

Pier Tube Coupler:
Ø2.250" x 0.180" wall x 6" long

Pier Starter Tube:
Pier tube section with machined Ø3.375" friction reduction collar pressed around leading end.

External Sleeve:
Ø3.500" x 0.216" wall x 30" or 48" long with welded collar or trumpet flare at one end.
ASTM A500 Grade B or C
Yield strength = 50 ksi (min)
Tensile strength = 62 ksi (min)

Cap Plate:
1" x 5.00" x 9.00" ASTM A572 Grade 50 with confining ring welded to one side.

Bracket Hardware(3):
(2) - Ø¾" x 16" long all-thread rod
Grade B7, tensile strength = 125 ksi (min)
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized(2)

Allowable Bracket Capacity(4,5,6,7) R_n/Ω

	with 30" Sleeve (kips)	with 48" Sleeve (kips)
Plain	23.9	36.7
Plain Corroded(1)	18.5	28.4
Grout Filled Corroded(1)	20.9	32.1
Maximum Drive Force During Installation(7)	48.1	60.0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC406. Grout filled piers consider a loss in thickness at the outside diameter only.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

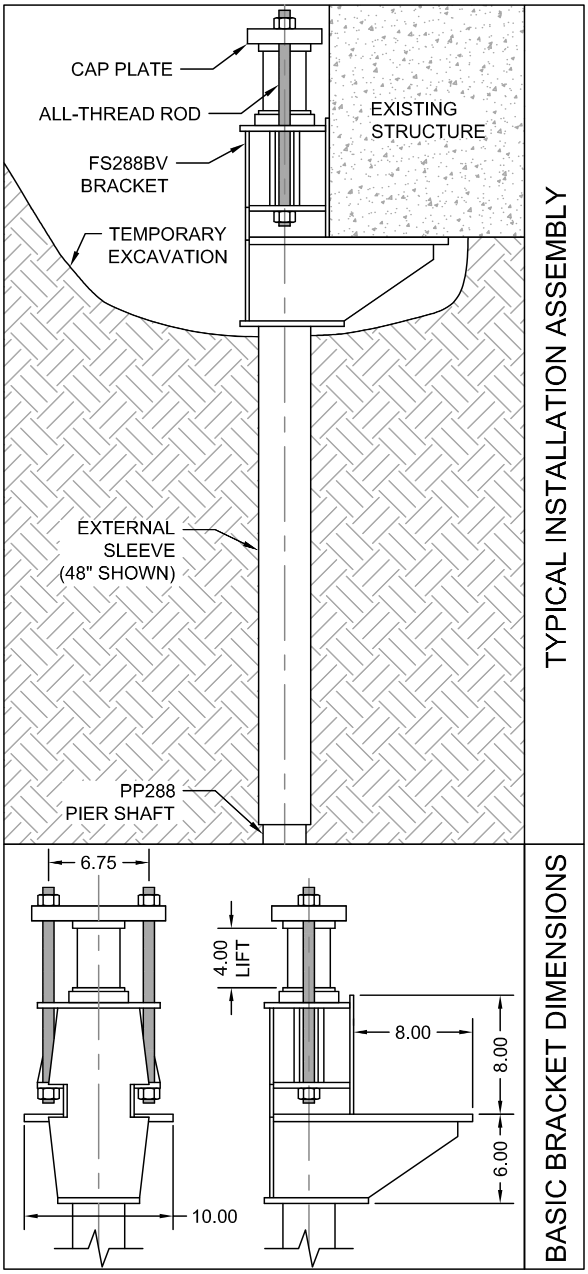
(3) Optional hardware utilizes similar sized contour (coil) thread made from AISI 1045, tensile strength = 120 ksi. Slightly lower tensile strength material does not govern the listed capacities.

(4) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(5) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piers with exposed unbraced lengths or piers placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(6) Concrete bearing assumes a minimum compressive strength (f'c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

(7) Push Piers shall be installed with a driving force exceeding the required allowable service load by a sufficient factor of safety (FOS). FOS is most commonly between 1.5 and 2.0, although a higher or lower FOS may be considered at the discretion of the pier designer or as dictated by local code or project requirements.



APPENDIX 3A
BRACKET SPECIFICATIONS AND CAPACITIES

FS288BL Bracket Specifications and Capacities

when used with the PP288 Push Pier System

Bracket:
Weldment manufactured from ¼”, ⅜”, and ½” ASTM A36 plate.

Pier Tube:
Ø2.875” x 0.165” wall x 36” long
Triple-coated in-line galvanized
ASTM A500 Grade C
Yield strength = 50 ksi (min)
Tensile strength = 55 ksi (min)

Pier Tube Coupler:
Ø2.250” x 0.180” wall x 6” long

Pier Starter Tube:
Pier tube section with machined Ø3.375” friction reduction collar pressed around leading end.

External Sleeve:
Ø3.500” x 0.216” wall x 30” or 48” long with welded collar or trumpet flare at one end.
ASTM A500 Grade B or C
Yield strength = 50 ksi (min)
Tensile strength = 62 ksi (min)

Cap Plate:
1” x 5.00” x 9.00” ASTM A572 Grade 50 with confining ring welded to one side.

Bracket Hardware(3):
(2) - Ø¾” x 16” long all-thread rod
Grade B7, tensile strength = 125 ksi (min)
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized(2)

Allowable Bracket Capacity(4,5,6,7) R_n/Ω	with 30” Sleeve (kips)	with 48” Sleeve (kips)
Plain	21.4	32.9
Plain Corroded(1)	16.6	25.4
Grout Filled Corroded(1)	18.7	28.8
Maximum Drive Force During Installation(7)	48.1	60.0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC406. Grout filled piers consider a loss in thickness at the outside diameter only.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

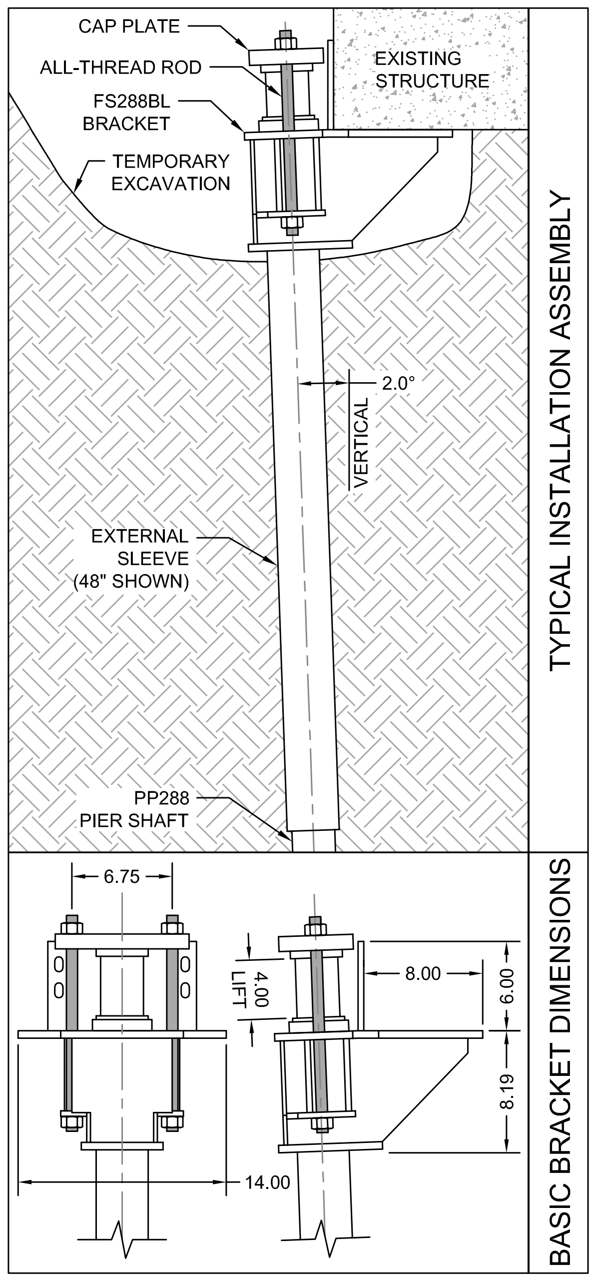
(3) Optional hardware utilizes similar sized contour (coil) thread made from AISI 1045, tensile strength = 120 ksi. Slightly lower tensile strength material does not govern the listed capacities.

(4) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(5) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piers with exposed unbraced lengths or piers placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(6) Concrete bearing assumes a minimum compressive strength (f’c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

(7) Push Piers shall be installed with a driving force exceeding the required allowable service load by a sufficient factor of safety (FOS). FOS is most commonly between 1.5 and 2.0, although a higher or lower FOS may be considered at the discretion of the pier designer or as dictated by local code or project requirements.



FS288BFM Bracket Specifications and Capacities

when used with the PP288 Push Pier System

Bracket:
Weldment manufactured from ¼”, ⅜”, and ½” ASTM A36 plate.

Pier Tube:
Ø2.875” x 0.165” wall x 36” long
Triple-coated in-line galvanized
ASTM A500 Grade C
Yield strength = 50 ksi (min)
Tensile strength = 55 ksi (min)

Pier Tube Coupler:
Ø2.250” x 0.180” wall x 6” long

Pier Starter Tube:
Pier tube section with Ø3.375” friction reduction collar pressed around leading end.

External Sleeve:
Ø3.500” x 0.216” wall x 30” or 48” long with welded collar or trumpet flare at one end.
ASTM A500 Grade B or C
Yield strength = 50 ksi (min)
Tensile strength = 62 ksi (min)

Cap Plate:
1” x 5.00” x 9.00” ASTM A572 Grade 50 with confining ring welded to one side.

Bracket Hardware[3]:
(2) - Ø¾”x 16” long all-thread rod
Grade B7, tensile strength = 125 ksi (min)
Electrozinc plated per ASTM B633

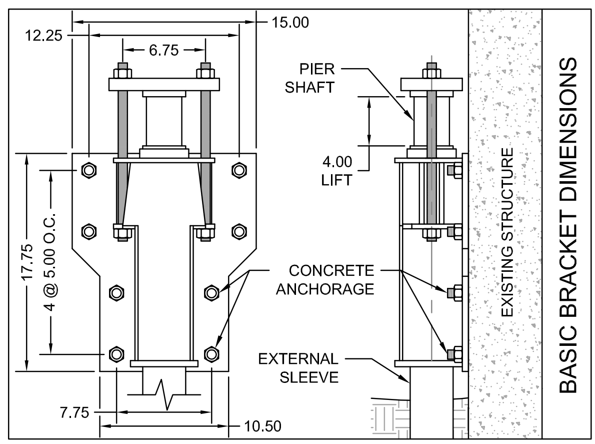
Bracket Finish:
Available plain or hot-dip galvanized[2]

Concrete Anchorage[7] (Option 1):
(8) - Ø¾” x 7” Simpson Wedge-All
Mechanically galvanized per ASTM B695)

Concrete Anchorage[8] (Option 2):
Adhesive = Simpson AT
Quantity = approximately 1.25 oz per hole
(8) - Ø¾” x 7” long all-thread rod
Grade B7, tensile strength = 125 ksi (min)
Electrozinc plated per ASTM B633

Allowable Bracket Capacity[4,5,6,9] R_n/Ω

	with Wedge Anchors[7] (kips)	with Adhesive Anchors[8] (kips)
Plain	22.0	31.0
Plain Corroded[1]	17.0	24.0
Grout Filled Corroded[1]	19.3	27.1
Maximum Drive Force During Installation[9]	44.2	60.0



(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC406. Grout filled piers consider a loss in thickness at the outside diameter only.

(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Optional hardware utilizes similar sized contour (coil) thread made from AISI 1045, tensile strength = 120 ksi. Slightly lower tensile strength material does not govern the listed capacities.

(4) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(5) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piers with exposed unbraced lengths or piers placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(6) Bracket is strictly intended to be mounted to poured concrete structures. Bracket should never be mounted to CMU concrete block. Anchorage assumes a minimum concrete compressive strength (f’c) of 2,500 psi and a minimum concrete thickness of 8”. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

(7) When the bracket is mounted with wedge anchors, the bracket shall be located with a minimum distance of 6” from the edge of the bracket to any concrete edge. Wedge anchors require the use of a Ø¾” drill bit and a minimum embedment depth of 5”. Wedge anchors shall be installed to a torque of 150 ft-lbs.

(8) When the bracket is mounted with adhesive anchors, the bracket shall be located with a minimum distance of 9” from the edge of the bracket to any concrete edge. Wedge anchors require the use of a Ø13/16” drill bit and a minimum embedment depth of 5.50”. Adhesive anchors shall be tightened to a snug tight condition after sufficient curing time.

(9) Push Piers shall be installed with a driving force exceeding the required allowable service load by a sufficient factor of safety (FOS). FOS is most commonly between 1.5 and 2.0, although a higher or lower FOS may be considered at the discretion of the pier designer or as dictated by local code or project requirements.

FS350BV Bracket Specifications and Capacities

when used with the PP350 Push Pier System

Bracket:
Weldment manufactured from ⅜" and ½" ASTM A36 plate.

Pier Tube:
Ø3.500" x 0.165" wall x 36" long
Triple-coated in-line galvanized
ASTM A500 Grade C
Yield strength = 50 ksi (min)
Tensile strength = 55 ksi (min)

Pier Tube Coupler:
Ø3.125" x 0.180" wall x 6" long

Pier Starter Tube:
Pier tube section with Ø4.000" friction reduction collar welded at leading end.

External Sleeve:
Ø4.000" x 0.226" wall x 48" long with welded collar at one end.
ASTM A500 Grade B or C
Yield strength = 50 ksi (min)
Tensile strength = 62 ksi (min)

Cap Plate:
1-¼" x 4.00" x 8.50" ASTM A572 Grade 50 with capture plate welded to one side.

Bracket Hardware:
(2) - Ø⅞"x 18" long all-thread rod
Grade B7, tensile strength = 125 ksi (min)
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized(2)

Allowable Bracket Capacity(3,4,5,6) R_n/Ω	
	(kips)
Plain	48.7
Plain Corroded(1)	37.6
Grout Filled Corroded(1)	42.7
Maximum Drive Force During Installation(6)	77.0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC406. Grout filled piers consider a loss in thickness at the outside diameter only.

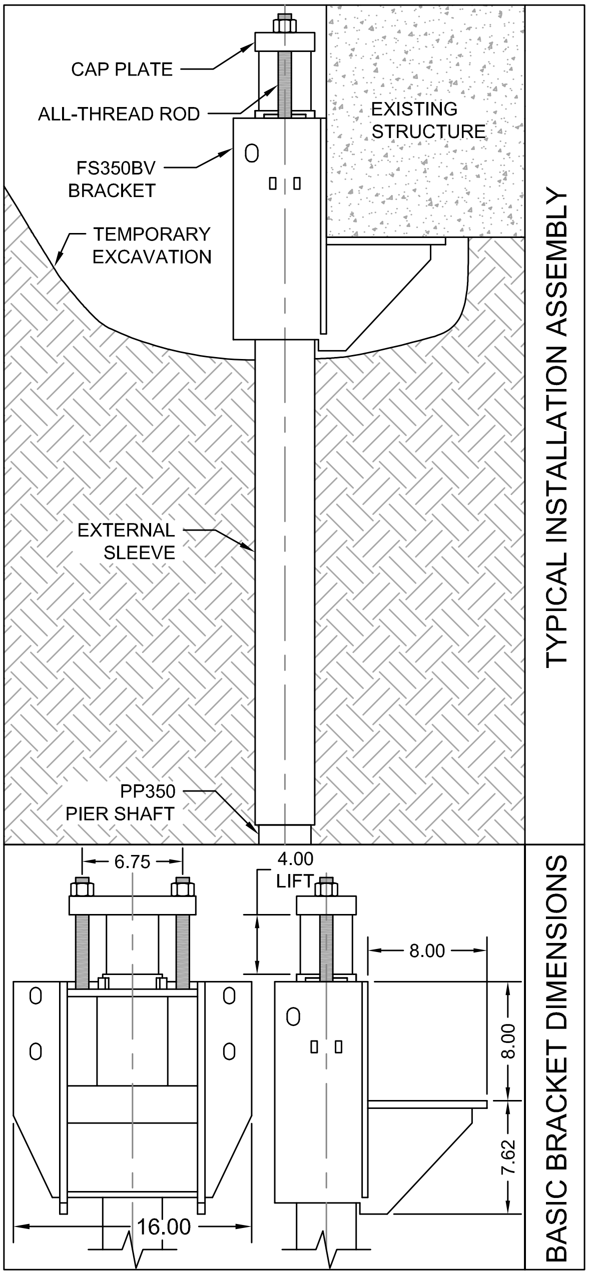
(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(4) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piers with exposed unbraced lengths or piers placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(5) Concrete bearing assumes a minimum compressive strength (f'c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

(6) Push Piers shall be installed with a driving force exceeding the required allowable service load by a sufficient factor of safety (FOS). FOS is most commonly between 1.5 and 2.0, although a higher or lower FOS may be considered at the discretion of the pier designer or as dictated by local code or project requirements.



APPENDIX 3A
BRACKET SPECIFICATIONS AND CAPACITIES

FS400BV Bracket Specifications and Capacities
when used with the PP400 Push Pier System

Bracket:
Weldment manufactured from ⅜" and ½" ASTM A36 plate.

Pier Tube:
Ø4.000" x 0.226" wall x 36" long
ASTM A500 Grade B or C
Yield strength = 50 ksi (min)
Tensile strength = 62 ksi (min)
Available plain or hot-dip galvanized(2)

Pier Tube Coupler:
Ø3.500" x 0.216" wall x 8" long

Pier Starter Tube:
Pier tube section with Ø4.500" friction reduction collar welded at leading end.

External Sleeve:
None

Cap Plate:
1-¼" x 4.00" x 8.50" ASTM A572 Grade 50 with capture plate welded to one side.

Bracket Hardware:
(2) - Ø⅞"x 18" long all-thread rod
Grade B7, tensile strength = 125 ksi (min)
Electrozinc plated per ASTM B633

Bracket Finish:
Available plain or hot-dip galvanized(2)

Allowable Bracket Capacity(3,4,5,6) R_n/Ω	
	(kips)
Plain	43.6
Plain Corroded(1)	36.5
Galvanized Corroded(1)	41.6
Maximum Drive Force During Installation(6)	77.0

(1) Corroded capacities include a 50-year scheduled sacrificial loss in thickness per ICC-ES AC406.

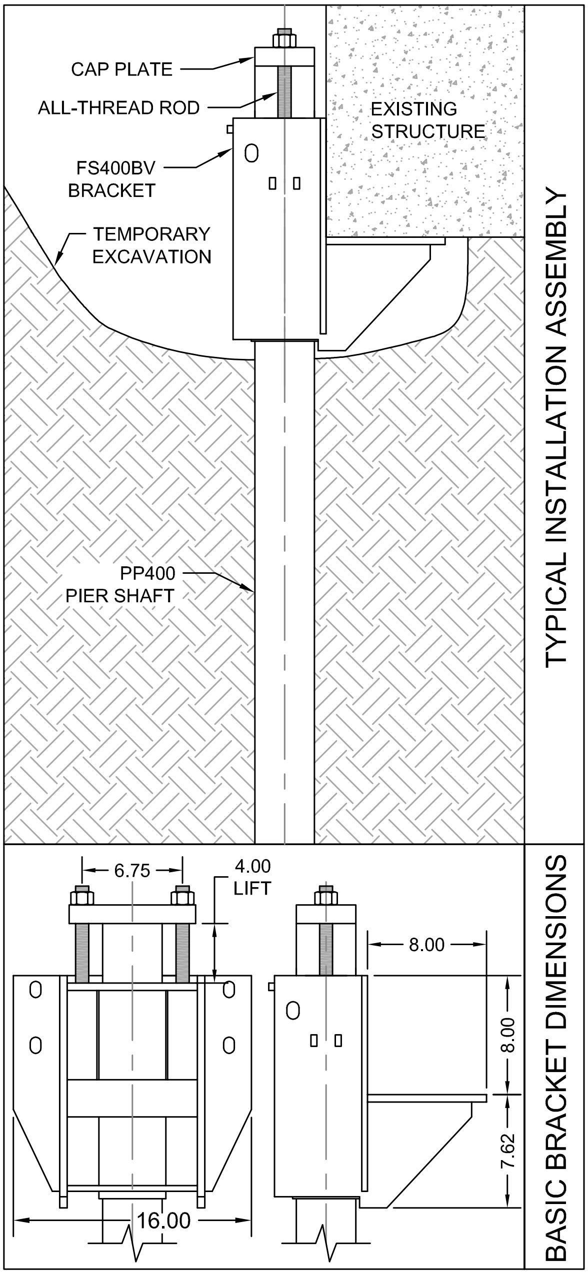
(2) Hot-dip galvanized coating in accordance with ASTM A123.

(3) Brackets shall be used for support of structures that are considered to be fixed from translation. Structures that are not fixed from translation shall be braced in some manner prior to installing retrofit brackets systems.

(4) Allowable compression capacities consider continuous lateral soil confinement in soils with SPT blow counts ≥ 4. Piers with exposed unbraced lengths or piers placed in weaker or fluid soils should be evaluated on a case by case basis by the project engineer.

(5) Concrete bearing assumes a minimum compressive strength (f'c) of 2,500 psi. Local concrete bending and other local design checks should be evaluated on a case by case basis by the project engineer.

(6) Push Piers shall be installed with a driving force exceeding the required allowable service load by a sufficient factor of safety (FOS). FOS is most commonly between 1.5 and 2.0, although a higher or lower FOS may be considered at the discretion of the pier designer or as dictated by local code or project requirements.



Appendix 3B

Drive Stand Specifications

Model 238 Drive Stand Specifications

when used with the RED(4) Drive Cylinder (FS35DC)

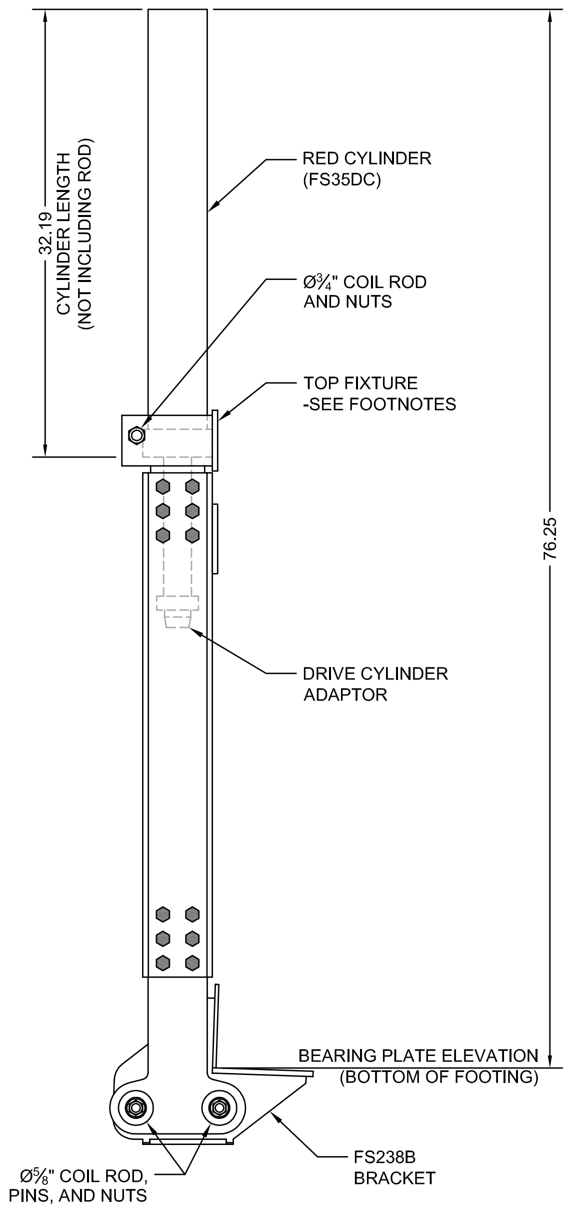
Compatible Brackets(3):
FS238B

Drive Cylinder(4) (FS35DC):
Stroke = 22”
Cylinder action = double
Bore = Ø3.50”
Hydraulic area = 9.62 in^2
Max operating pressure(2) = 8,000 psi

Drive Cylinder Adaptor:
FSDCA238

Drive Stand Hardware(1):
(1) - Ø¾” x 12” long coil rod with nuts
(2) - Ø⅝” x 12” long coil rods with nuts
(4) - 1-½” Flanged Pins

Drive Stand Rated Drive Load(2,3) 30.0 kips			
Hydraulic Pressure (psi)	Drive Force(2,3) (kips)	Hydraulic Pressure (psi)	Drive Force(2,3) (kips)
200	1.9	2,100	20.2
400	3.8	2,200	21.2
600	5.8	2,300	22.1
800	7.7	2,400	23.1
1,000	9.6	2,500	24.1
1,200	11.5	2,600	25.0
1,400	13.5	2,700	26.0
1,600	15.4	2,800	26.9
1,700	16.4	2,900	27.9
1,800	17.3	3,000	28.9
1,900	18.3	3,100	29.8
2,000	19.2	3,120	30.0



(1) Drive stand should never be operated without all hardware components firmly in place.

(2) Do not operate at pressures that produce drive forces in excess of the drive stand’s rated drive load. Max operating pressure of the drive cylinder produces forces that exceed this value and is given for informational purposes only.

(3) Do not operate at pressures that produce drive forces in excess of the “maximum drive force during installation” values specified for the bracket being installed (see Bracket Specifications and Capacities).

(4) Note that the mounting flange dimensions are different between the RED and GRAY cylinders (FS35DC and FS425DC) and thereby require the use of unique top fixture weldments that correspond to the appropriate drive cylinder.

Model 238 Drive Stand Specifications

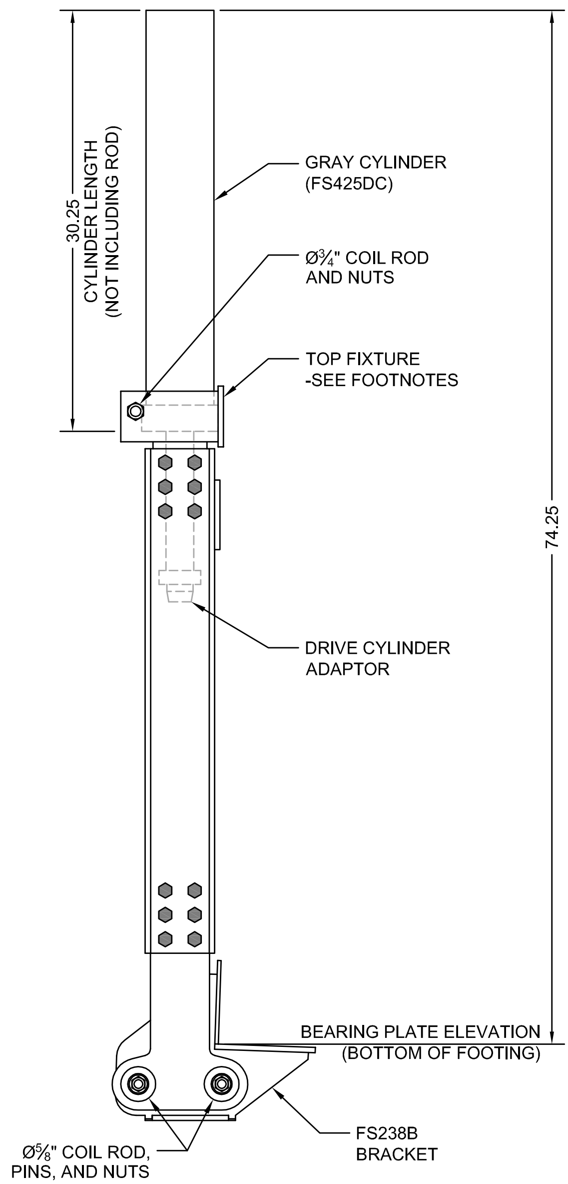
when used with the GRAY[4] Drive Cylinder (FS425DC)

Compatible Brackets[3]:
FS238B

Drive Cylinder[4] (FS425DC):
Stroke = 22”
Cylinder action = double
Bore = Ø4.25”
Hydraulic area = 14.18 in^2
Max operating pressure[2] = 4,000 psi

Drive Cylinder Adaptor:
FSDCA238

Drive Stand Hardware[1]:
(1) - Ø¾” x 12” long coil rod with nuts
(2) - Ø⅝” x 12” long coil rods with nuts
(4) - 1-½” Flanged Pins



Drive Stand Rated Drive Load[2,3]
30.0 kips

Hydraulic Pressure (psi)	Drive Force[2,3] (kips)	Hydraulic Pressure (psi)	Drive Force[2,3] (kips)
100	1.4	1,300	18.4
200	2.8	1,400	19.9
300	4.3	1,500	21.3
400	5.7	1,600	22.7
500	7.1	1,700	24.1
600	8.5	1,800	25.5
700	9.9	1,900	27.0
800	11.3	2,000	28.4
900	12.8	2,100	29.8
1,000	14.2	2,115	30.0
1,100	15.6		
1,200	17.0		

(1) Drive stand should never be operated without all hardware components firmly in place.

(2) Do not operate at pressures that produce drive forces in excess of the drive stand’s rated drive load. Max operating pressure of the drive cylinder produces forces that exceed this value and is given for informational purposes only.

(3) Do not operate at pressures that produce drive forces in excess of the “maximum drive force during installation” values specified for the bracket being installed (see Bracket Specifications and Capacities).

(4) Note that the mounting flange dimensions are different between the RED and GRAY cylinders (FS35DC and FS425DC) and thereby require the use of unique top fixture weldments that correspond to the appropriate drive cylinder.

Model 288 Drive Stand Specifications

when used with the RED[5] Drive Cylinder (FS35DC)

Compatible Brackets[3]:

FS288B, FS288BV,
FS288BL, FS288BFM

Drive Cylinder[5] (FS35DC):

Stroke = 22”
Cylinder action = double
Bore = Ø3.50”
Hydraulic area = 9.62 in^2
Max operating pressure[2] = 8,000 psi

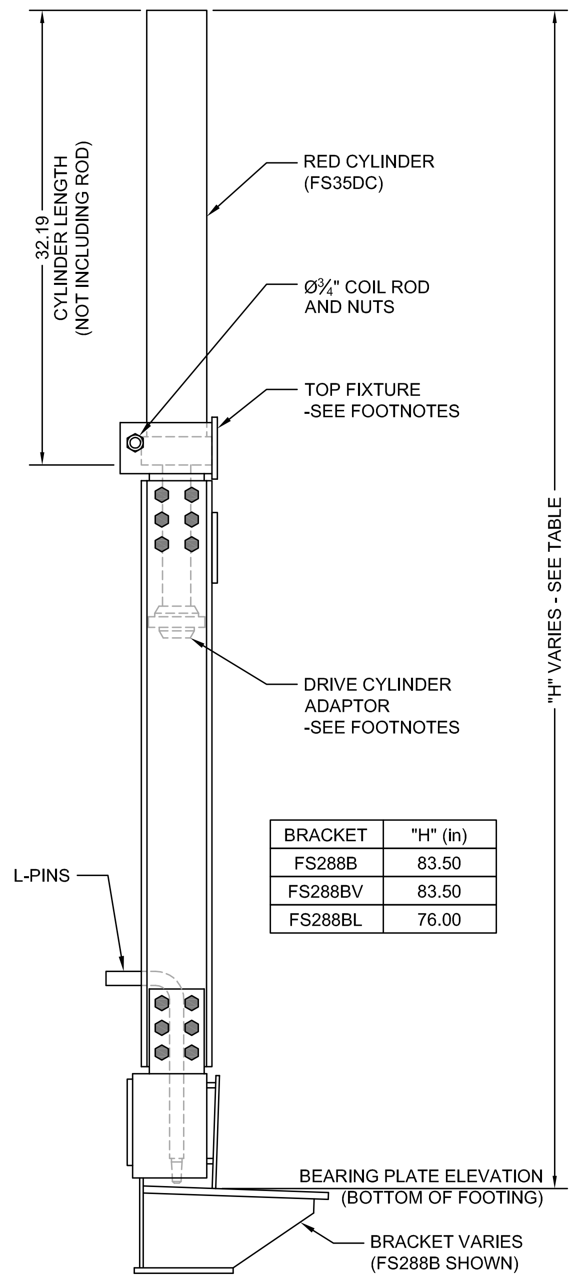
Drive Cylinder Adaptor[4]:

FSDCA (reversible)

Drive Stand Hardware[1]:

(1) - Ø¾” x 12” long coil rod with nuts
(2) - Ø1” x 15” long L-pins

Drive Stand Rated Drive Load[2,3] 60.0 kips			
Hydraulic Pressure (psi)	Drive Force[2,3] (kips)	Hydraulic Pressure (psi)	Drive Force[2,3] (kips)
400	3.8	4,200	40.4
800	7.7	4,400	42.3
1,200	11.5	4,600	44.3
1,600	15.4	4,800	46.2
2,000	19.2	5,000	48.1
2,400	23.1	5,200	50.0
2,800	26.9	5,400	52.0
3,200	30.8	5,600	53.9
3,400	32.7	5,800	55.8
3,600	34.6	6,000	57.7
3,800	36.6	6,200	59.7
4,000	38.5	6,235	60.0



(1) Drive stand should never be operated without all hardware components firmly in place.

(2) Do not operate at pressures that produce drive forces in excess of the drive stand’s rated drive load. Max operating pressure of the drive cylinder produces forces that exceed this value and is given for informational purposes only.

(3) Do not operate at pressures that produce drive forces in excess of the “maximum drive force during installation” values specified for the bracket being installed (see Bracket Specifications and Capacities).

(4) Drive cylinder adaptor FSDCA is a reversible adaptor that is compatible with both PP288 and PP350 push pier systems. Assemble the adaptor to the cylinder rod in the appropriate orientation for the corresponding pier size being installed.

(5) Note that the mounting flange dimensions are different between the RED and GRAY cylinders (FS35DC and FS425DC) and thereby require the use of unique top fixture weldments that correspond to the appropriate drive cylinder.

Model 288 Drive Stand Specifications

when used with the GRAY[4] Drive Cylinder (FS425DC)

Compatible Brackets[2]:
FS288B, FS288BV,
FS288BL, FS288BFM

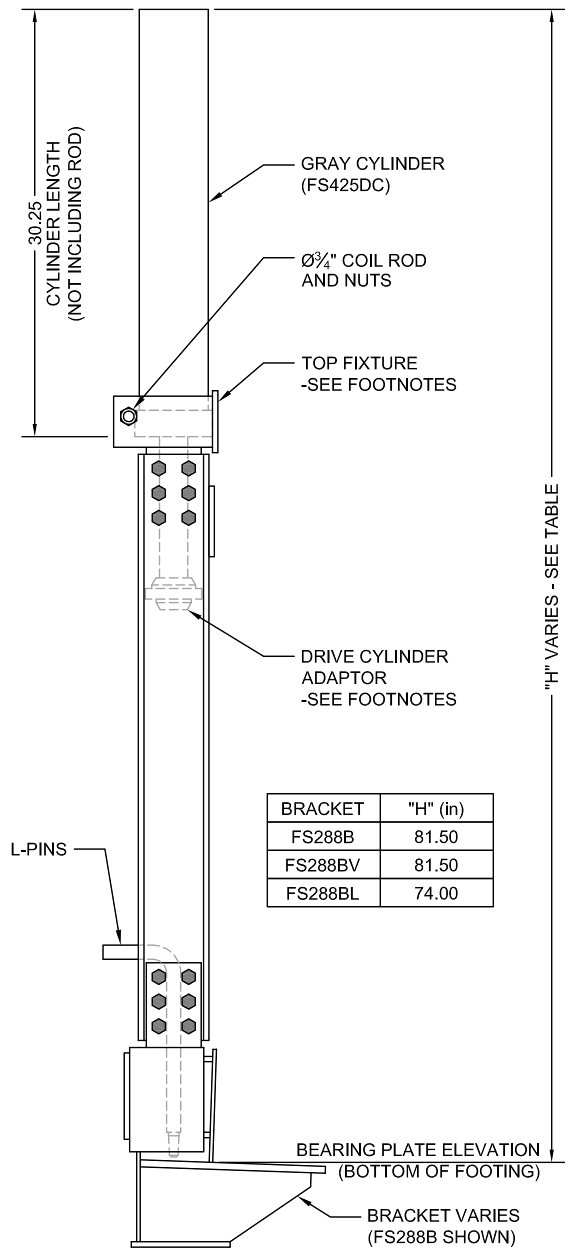
Drive Cylinder[4] (FS425DC):
Stroke = 22”
Cylinder action = double
Bore = Ø4.25”
Hydraulic area = 14.18 in^2
Max operating pressure = 4,000 psi

Drive Cylinder Adaptor[3]:
FSDCA (reversible)

Drive Stand Hardware[1]:
(1) - Ø¾” x 12” long coil rod with nuts
(2) - Ø1” x 15” long L-pins

Drive Stand Rated Drive Load[2]
56.7 kips

Hydraulic Pressure (psi)	Drive Force[2] (kips)	Hydraulic Pressure (psi)	Drive Force[2] (kips)
200	2.8	2,600	36.9
400	5.7	2,800	39.7
600	8.5	3,000	42.6
800	11.3	3,200	45.4
1,000	14.2	3,300	46.8
1,200	17.0	3,400	48.2
1,400	19.9	3,500	49.7
1,600	22.7	3,600	51.1
1,800	25.5	3,700	52.5
2,000	28.4	3,800	53.9
2,200	31.2	3,900	55.3
2,400	34.0	4,000	56.7



BRACKET	"H" (in)
FS288B	81.50
FS288BV	81.50
FS288BL	74.00

(1) Drive stand should never be operated without all hardware components firmly in place.

(2) Do not operate at pressures that produce drive forces in excess of the “maximum drive force during installation” values specified for the bracket being installed (see Bracket Specifications and Capacities).

(3) Drive cylinder adaptor FSDCA is a reversible adaptor that is compatible with both PP288 and PP350 push pier systems. Assemble the adaptor to the cylinder rod in the appropriate orientation for the corresponding pier size being installed.

(4) Note that the mounting flange dimensions are different between the RED and GRAY cylinders (FS35DC and FS425DC) and thereby require the use of unique top fixture weldments that correspond to the appropriate drive cylinder.

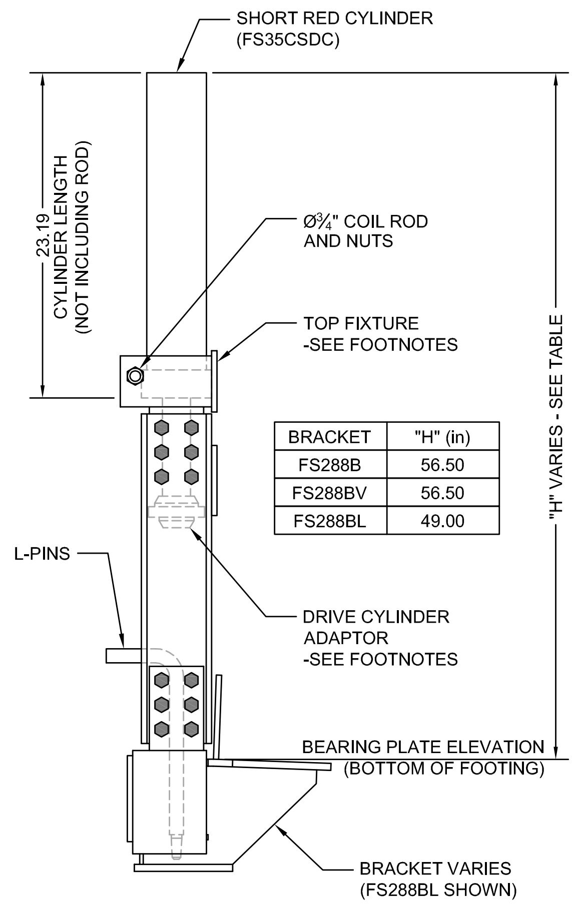
Model 288 Crawl Space Drive Stand Specifications
when used with the SHORT RED Drive Cylinder (FS35CSDC)

Compatible Brackets[3]:
FS288B, FS288BV,
FS288BL, FS288BFM

Drive Cylinder (FS35CSDC):
Stroke = 13”
Cylinder action = double
Bore = Ø3.50”
Hydraulic area = 9.62 in^2
Max operating pressure[2] = 8,000 psi

Drive Cylinder Adaptor[4]:
FSDCA (reversible)

Drive Stand Hardware[1]:
(1) - Ø¾” x 12” long coil rod with nuts
(2) - Ø1” x 15” long L-pins



Drive Stand Rated Drive Load[2,3] 60.0 kips			
Hydraulic Pressure (psi)	Drive Force[2,3] (kips)	Hydraulic Pressure (psi)	Drive Force[2,3] (kips)
400	3.8	4,200	40.4
800	7.7	4,400	42.3
1,200	11.5	4,600	44.3
1,600	15.4	4,800	46.2
2,000	19.2	5,000	48.1
2,400	23.1	5,200	50.0
2,800	26.9	5,400	52.0
3,200	30.8	5,600	53.9
3,400	32.7	5,800	55.8
3,600	34.6	6,000	57.7
3,800	36.6	6,200	59.7
4,000	38.5	6,235	60.0

(1) Drive stand should never be operated without all hardware components firmly in place.

(2) Do not operate at pressures that produce drive forces in excess of the drive stand’s rated drive load. Max operating pressure of the drive cylinder produces forces that exceed this value and is given for informational purposes only.

(3) Do not operate at pressures that produce drive forces in excess of the “maximum drive force during installation” values specified for the bracket being installed (see Bracket Specifications and Capacities).

(4) Drive cylinder adaptor FSDCA is a reversible adaptor that is compatible with both PP288 and PP350 push pier systems. Assemble the adaptor to the cylinder rod in the appropriate orientation for the corresponding pier size being installed.

Model 350 Drive Stand Specifications

when used with the RED Drive Cylinder (FS35DC)

Compatible Brackets(2,3):

FS350BV, FS400BV

Drive Cylinder (FS35DC):

Stroke = 22”
Cylinder action = double
Bore = Ø3.50”
Hydraulic area = 9.62 in^2
Max operating pressure = 8,000 psi

Drive Cylinder Adaptors(3):

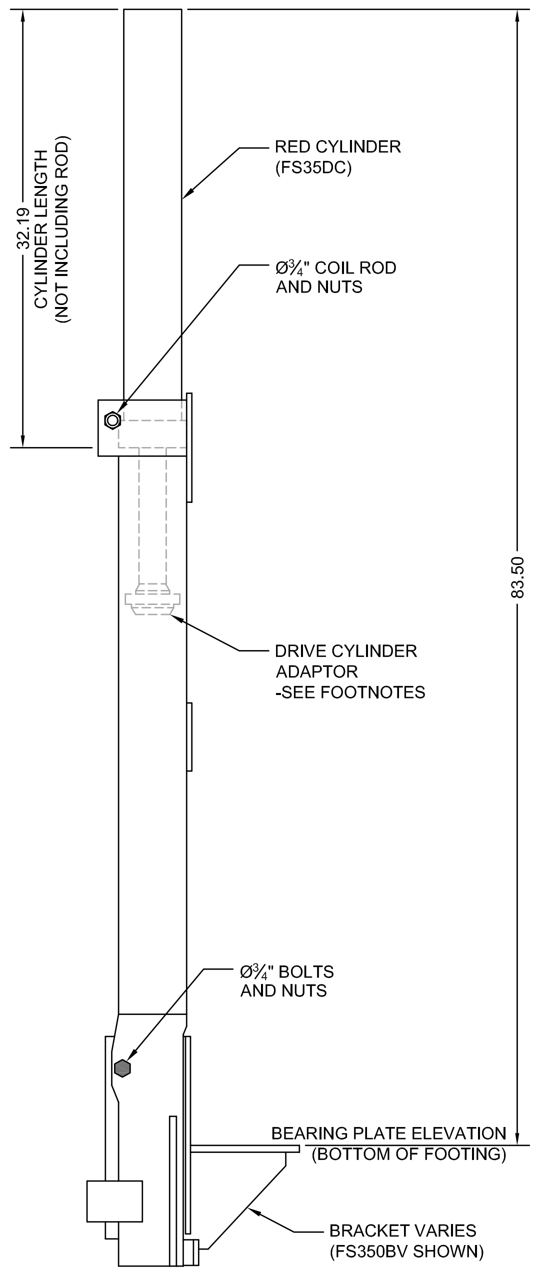
FSDCA (reversible) with FS350BV
FSDCA400 with FS400BV

Drive Stand Hardware(1):

(1) - Ø¾” x 12” long coil rod with nuts
(2) - Ø¾” grade 8 bolts with nuts

Drive Stand Rated Drive Load(2) 77.0 kips

Hydraulic Pressure (psi)	Drive Force(2) (kips)	Hydraulic Pressure (psi)	Drive Force(2) (kips)
400	3.8	5,200	50.0
800	7.7	5,600	53.9
1,200	11.5	6,000	57.7
1,600	15.4	6,400	61.6
2,000	19.2	6,600	63.5
2,400	23.1	6,800	65.4
2,800	26.9	7,000	67.3
3,200	30.8	7,200	69.3
3,600	34.6	7,400	71.2
4,000	38.5	7,600	73.1
4,400	42.3	7,800	75.0
4,800	46.2	8,000	77.0



(1) Drive stand should never be operated without all hardware components firmly in place.

(2) Do not operate at pressures that produce drive forces in excess of the “maximum drive force during installation” values specified for the bracket being installed (see Bracket Specifications and Capacities).

(3) PP350 and PP400 push pier systems require the use of different drive cylinder adaptors. Assemble the appropriate adaptor to the cylinder rod for the corresponding pier size being installed. Also note that drive cylinder adaptor FSDCA is reversible and needs to be assembled in the appropriate orientation when installing PP350 systems.

Appendix 3C

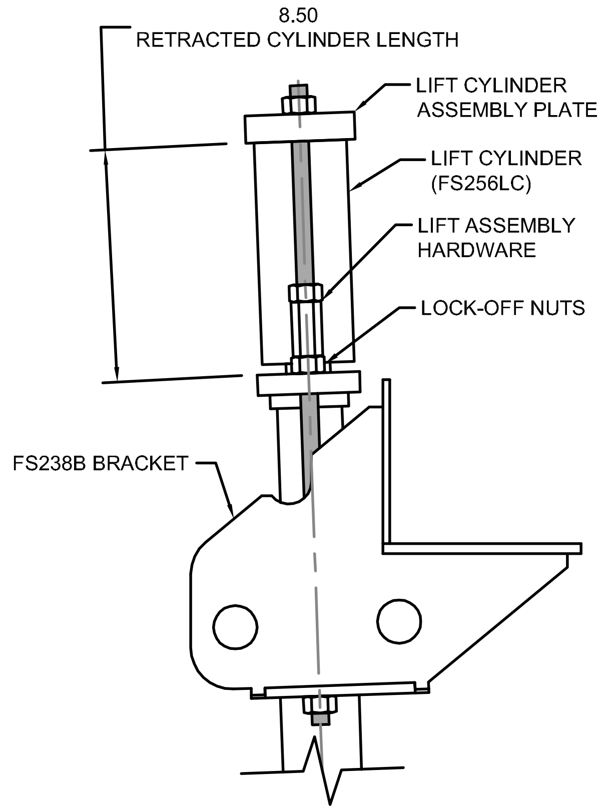
Lift Assembly Specifications

Model 238 Lift Assembly Specifications

Compatible Brackets[3]:
FS238B

Lift Cylinder (FS256LC):
Stroke = 4”
Cylinder action = single
Bore = Ø2.56”
Hydraulic area = 5.15 in^2
Max operating pressure[2] = 8,000 psi

Lift Assembly Hardware[1]:
(2) - Ø⅝” x 16” long coil rod
with nuts and hex couplers, or
(2) - Ø⅝” x 14” long all-thread rod
with nuts and hex couplers



Lift Assembly Rated Lifting Load[2,3] 27.6 kips			
Hydraulic Pressure (psi)	Lift Force[2,3] (kips)	Hydraulic Pressure (psi)	Lift Force[2,3] (kips)
400	2.1	3,200	16.5
800	4.1	3,400	17.5
1,200	6.2	3,600	18.6
1,400	7.2	3,800	19.6
1,600	8.3	4,000	20.6
1,800	9.3	4,200	21.7
2,000	10.3	4,400	22.7
2,200	11.3	4,600	23.7
2,400	12.4	4,800	24.8
2,600	13.4	5,000	25.8
2,800	14.4	5,200	26.8
3,000	15.5	5,350	27.6

(1) Hardware used in the lift assembly must be selected to match the hardware used with the installed bracket assembly.

(2) Do not operate at pressures that produce lift forces in excess of the lift assembly’s rated lifting load. Max operating pressure of the lift cylinder produces forces that exceed this value and is given for informational purposes only.

(3) Rated lifting load is given for the lift assembly only. Do not operate at pressures that exceed the allowable capacities of the system which are governed by the allowable capacities of the bracket and other system components, as well as the torque correlated soil capacity, or installed driving force divided by an appropriate factor of safety. All of these governing limits are outlined in places elsewhere in this appendix.

Model 288 Lift Assembly[2] Specifications

Compatible Brackets[4]:

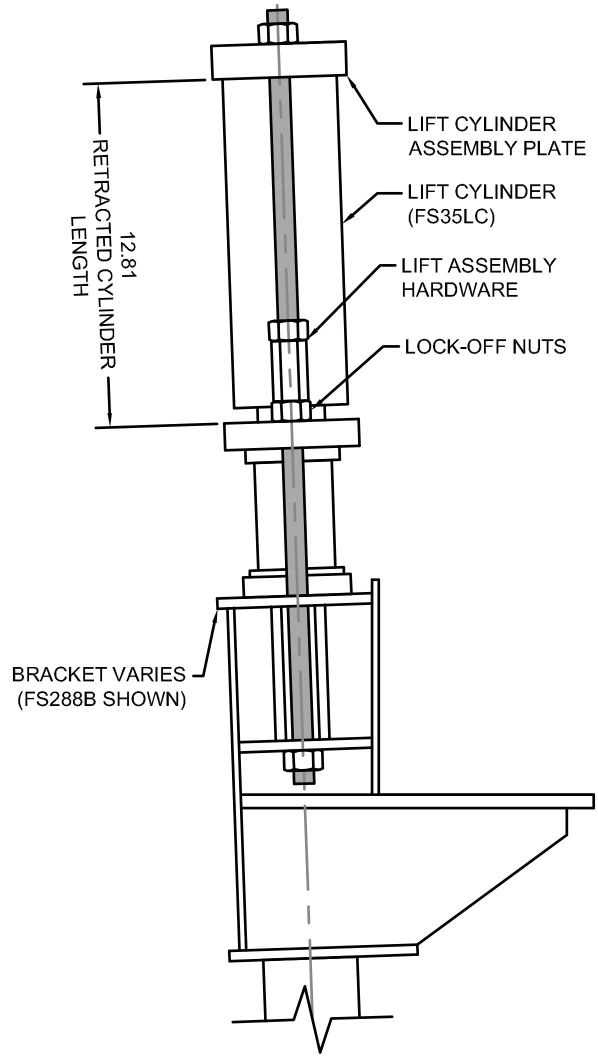
HP238B2, HP288B2, FS288B
FS288BV, FS288BL, FS288BFM

Lift Cylinder (FS35LC):

Stroke = 4”
Cylinder action = double
Bore = Ø3.50”
Hydraulic area = 9.62 in^2
Max operating pressure[3] = 8,000 psi

Lift Assembly Hardware[1,2]:

(2) - Ø¾” x 16” long all-thread rod
with nuts and hex couplers, or
(2) - Ø¾” x 16” long coil rod
with nuts and hex couplers



Lift Assembly Rated Lifting Load[3,4] 39.7 kips			
Hydraulic Pressure (psi)	Lift Force[3,4] (kips)	Hydraulic Pressure (psi)	Lift Force[3,4] (kips)
200	1.9	2,600	25.0
400	3.8	2,800	26.9
600	5.8	3,000	28.9
800	7.7	3,200	30.8
1,000	9.6	3,400	32.7
1,200	11.5	3,600	34.6
1,400	13.5	3,700	35.6
1,600	15.4	3,800	36.6
1,800	17.3	3,900	37.5
2,000	19.2	4,000	38.5
2,200	21.2	4,100	39.4
2,400	23.1	4,130	39.7

(1) Hardware used in the lift assembly must be selected to match the hardware used with the installed bracket assembly.

(2) Note that the only difference between the model 288 and model 350 lift assemblies is the diameter of the threaded rod hardware. All other components of the two assemblies are identical.

(3) Do not operate at pressures that produce lift forces in excess of the lift assembly’s rated lifting load. Max operating pressure of the lift cylinder produces forces that exceed this value and is given for informational purposes only.

(4) Rated lifting load is given for the lift assembly only. Do not operate at pressures that exceed the allowable capacities of the system which are governed by the allowable capacities of the bracket and other system components, as well as the torque correlated soil capacity, or installed driving force divided by an appropriate factor of safety. All of these governing limits are outlined in places elsewhere in this appendix.

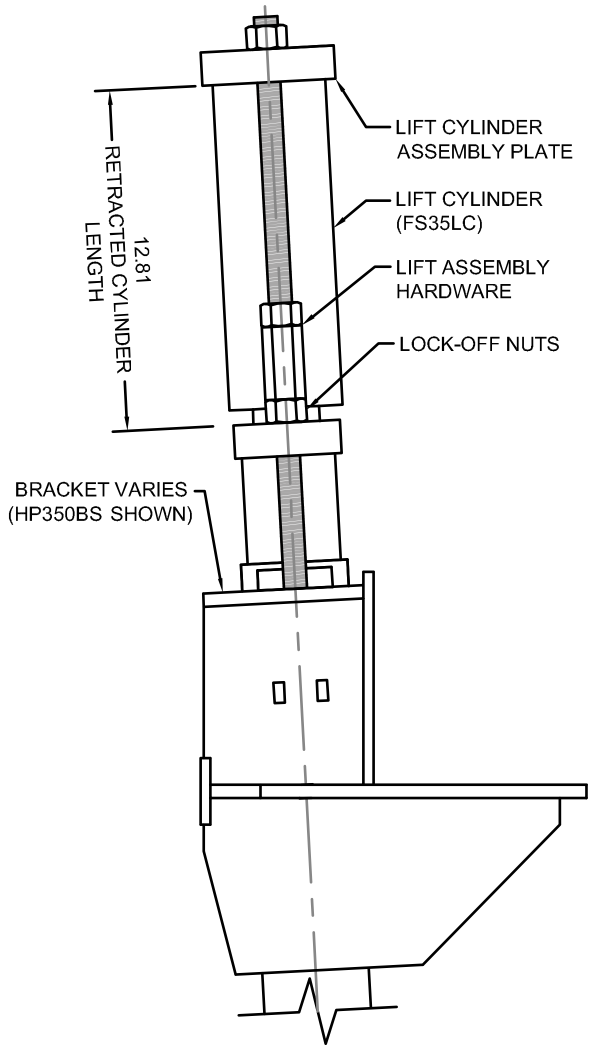
Model 350 Lift Assembly[1] Specifications

Compatible Brackets[3]:
HP350BS, HP350B,
FS350BV, FS400BV

Lift Cylinder (FS35LC):
Stroke = 4”
Cylinder action = double
Bore = Ø3.50”
Hydraulic area = 9.62 in^2
Max operating pressure[2] = 8,000 psi

Lift Assembly Hardware[1]:
(2) - Ø⅞” x 18” long all-thread rod
with nuts and hex couplers

Lift Assembly Rated Lifting Load[2,3] 56.3 kips			
Hydraulic Pressure (psi)	Lift Force[2,3] (kips)	Hydraulic Pressure (psi)	Lift Force[2,3] (kips)
400	3.8	3,800	36.6
800	7.7	4,000	38.5
1,200	11.5	4,200	40.4
1,600	15.4	4,400	42.3
2,000	19.2	4,600	44.3
2,400	23.1	4,800	46.2
2,600	25.0	5,000	48.1
2,800	26.9	5,200	50.0
3,000	28.9	5,400	52.0
3,200	30.8	5,600	53.9
3,400	32.7	5,800	55.8
3,600	34.6	5,850	56.3



(1) Note that the only difference between the model 288 and model 350 lift assemblies is the diameter of the threaded rod hardware. All other components of the two assemblies are identical.

(2) Do not operate at pressures that produce lift forces in excess of the lift assembly’s rated lifting load. Max operating pressure of the lift cylinder produces forces that exceed this value and is given for informational purposes only.

(3) Rated lifting load is given for the lift assembly only. Do not operate at pressures that exceed the allowable capacities of the system which are governed by the allowable capacities of the bracket and other system components, as well as the torque correlated soil capacity, or installed driving force divided by an appropriate factor of safety. All of these governing limits are outlined in places elsewhere in this appendix.

Appendix 3D

Model Specification – Hydraulically-Driven Push Pier Systems

MODEL SPECIFICATION FOR PUSH PIER FOUNDATIONS COMPRESSION APPLICATIONS

1 SCOPE

1.1 The work consists of designing, furnishing and installing push piers and load transfer devices used to support compressive loads according to the project Plans and these specifications.

1.2 The parties and contract terms referred to in this specification are as follows:

1.2.1 The Owner is the person or entity that owns the facility or will own the facility once it is completed. The Owner may have contractual agreements with, and be represented by, other parties such as engineers, architects, or contractors that perform services under the direction of the Owner. Where Owner is used in this specification, it refers to the Owner or the Owner's contracted representatives separate from the Installing Contractor.

1.2.2 The Pier Designer is the individual or firm generally hired by the Installing Contractor to design the push piers.

1.2.3 The Installing Contractor installs and tests (if necessary) the push piers, and possibly performs other tasks associated with the project.

1.2.4 The Plans refer to the contract documents; including but not limited to the drawings and specifications for the project.

1.3 The work may include push pier load testing.

1.4 The Owner will be responsible for obtaining any right-of-way or easement access permits necessary for the push pier installation.

1.5 Unless otherwise noted, the Installing Contractor shall provide all labor, tools, equipment and materials necessary to accomplish the work.

1.6 The Owner will provide suitable access to the construction site for the Installing Contractor's personnel and equipment.

1.7 Unless specifically noted otherwise in the contract documents, the Owner will remove and replace any structures, utilities, pavements, landscaping, or other surficial improvements in the work area as necessary to facilitate the work.

1.8 The Owner will be responsible for overall construction oversight to preclude the development of unsafe conditions.

1.9 Unless specifically noted otherwise in the contract documents, the Owner will be responsible for a horizontal field survey of the push pier locations prior to push pier installation and an elevation survey to determine final structural lift subsequent to push pier installation (if necessary).

1.10 The work does not include any post-construction monitoring of pier performance unless specifically noted otherwise in the contract documents.

2 REFERENCES

2.1 American Institute of Steel Construction (AISC)

2.1.1 AISC 360: Specification for Structural Steel Buildings

2.2 American Society for Testing and Materials (ASTM)

2.2.1 ASTM A36: Carbon Structural Steel

2.2.2 ASTM A123: Zinc Coating (Hot-Dip) on Iron and Steel Hardware

2.2.3 ASTM A500: Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

2.2.4 ASTM A513: Electric-Resistance Welded Carbon and Alloy Steel Mechanical Tubing

2.2.5 ASTM A572: High-Strength Low-Alloy Columbian-Vanadium Structural Steel

2.2.6 ASTM B633: Electrodeposited Coatings of Zinc on Iron and Steel

2.2.7 ASTM D1143: Deep Foundations Under Static Axial Compressive Load

2.3 Council Evaluation Services (ICC-ES)

2.3.1 Acceptance Criteria 358 (AC358): Acceptance Criteria for Helical Pile Systems and Devices

2.3.2 Acceptance Criteria 406 (AC406): Acceptance Criteria for Belled Segmented Pipe Foundation Systems and Devices

3 DEFINITIONS

3.1 The following terms apply to push piers used to support compressive loads.

3.1.1 Allowable Stress Design: A structural and geotechnical design methodology that states that the summation of the actual estimated loads (nominal loads) must be less than or equal to the allowable design load (required strength). Allowable loads are obtained by dividing a nominal resistance (strength) by an appropriate factor of safety.

3.1.2 Bearing Stratum: The soil layer (or layers) that provide the push pier end bearing capacity.

3.1.3 Design Loads: A generic and ambiguous term used to describe any load used in design. It is not specific to factored or unfactored loads or any particular design methodology. It is a term; therefore, that should be avoided when specifying load requirements. FSI recommends using the term service load, nominal load or factored load, as described herein, where applicable.

3.1.4 Design Strength: A term used in structural design which is defined as the product of the nominal strength and the applicable resistance factor. An equivalent term typically used in geotechnical design is, also sometimes referred to as factored resistance (Load and Resistance Factor Design).

3.1.5 External Sleeve: Hollow steel shaft section installed through the bracket assembly and around the pier starter tube to provide additional bending strength at and directly below the bracket.

3.1.6 Factor of Safety: The ratio of the ultimate pier capacity or nominal resistance (strength) to the nominal or service load used in the design of any push pier component or interface (Allowable Stress Design).

3.1.7 Factored Load: The product of a nominal load and an applicable load factor (Load and Resistance Factor Design).

3.1.8 Factored Resistance: The product of a nominal resistance and an applicable resistance factor (Load and Resistance and Factor Design).

3.1.9 Geotechnical Capacity: The maximum load or the load at a specified limit state, that can be resisted through the push piers interaction with the bearing soils (see also Ultimate Pier Capacity).

3.1.10 Limit State: A condition beyond which a push pier component or interface becomes unfit for service and is judged to no longer be useful for its intended function (serviceability limit state) or to be unsafe (ultimate limit state (strength)).

3.1.11 Load and Resistance Factor Design: A structural and geotechnical design methodology that states that the Factored Resistance (Design Strength) must be greater than or equal to the summation of the applied factored loads.

3.1.12 Load Factor: A factor that accounts for the probability of deviation of the actual load from the predicted nominal load due to variability of material properties, workmanship, type of failure, and uncertainty in the prediction of the load (Load and Resistance Factor Design).

3.1.13 Load Test: A process to test the ultimate pier capacity and relation of applied load to pier head settlement by application of a known load on the push pier head and monitoring movement over a specific time period.

3.1.14 Loads: Forces that result from the weight of all building materials, occupants and their possessions, environmental effects, differential movement, and restrained dimensional changes. Permanent loads are those loads in which variations over time are rare or of small magnitude. All other loads are variable loads (see also Nominal Loads).

3.1.15 Mechanical Strength: The maximum load or the load at a specified limit state that can be resisted by the structural elements of a push pier.

3.1.16 Net Deflection: The total settlement at the pier head minus the theoretical elastic deformation of the pier shaft during a load test.

3.1.17 Nominal Loads: The magnitude of the loads specified, which include dead, live, soil, wind, snow, rain, flood, and earthquakes (also referred to as service loads or working loads).

3.1.18 Nominal Resistance: The pier capacity at a specified ultimate limit state (Load and Resistance Factor Design). See Ultimate Pier Capacity.

3.1.19 Nominal Strength: A term used in structural design which is defined as the structure or member capacity at a specified strength limit state. See Ultimate Pier Capacity.

3.1.20 Pier Tube: Hollow steel shaft sections that follow the starter tube section. The pier tubes have slip-fit internal couplings and are hydraulically advanced to the required bearing depth.

3.1.21 Push Pier System: A hydraulically-driven retrofit deep foundation that utilizes high-strength round steel tube and a load-transfer bracket (retrofit bracket) to stabilize and/or lift sinking or settling foundations. The system uses the weight of the structure and any contributory soil load above the footings to create the reaction to hydraulically advance (push) the pier tubes.

3.1.22 Resistance Factor: A factor that accounts for the probability of deviation of the actual resistance (strength) from the predicted nominal resistance (strength) due to variability of material properties, workmanship, type of failure and uncertainties in the analysis (Load and Resistance Factor Design).

3.1.23 Safety Factor: The ratio of the ultimate pier capacity to the nominal or service load used for the design of any push pier component or interface (Allowable Stress Design).

3.1.24 Service Load: See "Nominal Load" above.

3.1.25 Starter Tube: The lead pier tube that is hydraulically driven to the bearing stratum to create end bearing resistance of the push pier system. The starter tube has a friction reduction collar at the pier tip to create a temporary annular space between the shaft and the surrounding soil during installation.

3.1.26 Ultimate Pier Capacity: The push pier capacity based on the least capacity determined from applicable ultimate limit states for mechanical and geotechnical capacity.

4 APPROVED PUSH PIER MANUFACTURERS

4.1 Foundation Supportworks®, Inc., 12330 Cary Circle, Omaha, NE 68128; Phone: (800) 281-8545; Fax: (402) 393-4002.

4.2 Due to the special requirements for design and manufacturing of push pier systems, the systems shall be obtained from Foundation Supportworks®, Inc., or other qualified manufacturer with an approved equivalent product. A request to substitute any other manufactured push pier product must be submitted to the Owner for review not less than seven (7) calendar days prior to the bid date. The request must include:

4.2.1 Documentation of at least five years of production experience manufacturing push piers systems,

4.2.2 Documentation that the manufacturer's push pier systems have been used successfully in at least five engineered construction projects within the last three years,

4.2.3 Product acceptance by the local building code official(s) having jurisdiction over the project, and/or

4.2.4 Current ICC-ES or IAPMO product evaluation report or complete description of product testing and manufacturing quality assurance programs used to assess and maintain product quality and determine product mechanical strength and geotechnical capacity.

5 ACCEPTABLE PRODUCTS

5.1 Push Pier System Models PP237, PP288, PP350 and PP400 manufactured in accordance with the requirements of Sections 5 and 6 of this specification.

5.2 Model PP237 Push Pier System

5.2.1 Starter and Pier Tube Sections: The central steel shaft of the starter and pier tube sections are 2.375-inch outer diameter by 0.154-inch nominal wall thickness hollow structural section in conformance with ASTM A500 Grade B or C with a minimum yield strength of 60 ksi and a minimum tensile strength of 70 ksi. The starter tube includes a 1.00-inch long factory-welded friction reduction collar manufactured from 2.875-inch outer diameter by 0.203-inch nominal wall thickness hollow structural section conforming to ASTM A53 Grade B, Type E & S with a minimum yield strength of 35 ksi and a minimum tensile strength of 60 ksi. The starter tube and pier tube shaft finishes are either plain steel or hot-dip galvanized in accordance with ASTM A123.

5.2.2 Shaft Coupling Material: The shaft coupling material is factory crimped or plug-welded to one end of the tube section and consists of 2.00-inch outer diameter by 0.187-inch nominal wall thickness hollow structural section in conformance with ASTM A500 Grade B with a minimum yield strength of 42 ksi and a minimum tensile strength of 58 ksi. The pier tube shaft coupling finish is plain steel.

5.2.3 External Sleeve: The central steel shaft of the external sleeve is 2.875-inch outer diameter by 0.203-inch wall thickness hollow structural section in conformance with ASTM A500 Grade B or C with a minimum yield strength of 60 ksi and a minimum tensile strength of 70 ksi. A 0.75-inch long collar, welded to one end is manufactured with 3.375-inch by 0.188-inch wall thickness hollow structural section conforming to ASTM A53 Grade B, Type E & S with a minimum yield strength of 35 ksi and a minimum tensile strength of 60 ksi. The external sleeve shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

5.2.4 Bracket: Retrofit bracket PP238B is suitable for use with the PP237 push pier system. Bracket finishes are either plain steel or hot-dip galvanized in accordance with ASTM A123. Bracket hardware finishes are zinc coated in accordance with ASTM B633.

5.3 Model PP288 Push Pier System

5.3.1 Starter and Pier Tube Sections: The central steel shaft of the starter and pier tube sections are 2.875-inch outer diameter by 0.165-inch nominal wall thickness hollow structural section in conformance with ASTM A500 Grade C with a minimum yield strength of 50 ksi and a minimum tensile strength of 55 ksi. The starter tube includes a 1.00-inch long factory-welded friction reduction collar manufactured from 3.375-inch outer diameter by 0.188-inch nominal wall thickness hollow structural section conforming to ASTM A53 Grade B, Type E & S with a minimum yield strength of 35 ksi and a minimum tensile strength of 60 ksi. The starter tube and pier tube shaft finishes are triple coated in-line galvanized.

5.3.2 Shaft Coupling Material: The shaft coupling material is factory crimped or plug-welded to one end of the tube section and consists of 2.50-inch outer diameter by 0.180-inch nominal wall thickness hollow structural section in conformance with ASTM A53 Grade

B, Type E & S with a minimum yield strength of 35 ksi and a minimum tensile strength of 60 ksi. The pier tube shaft coupling finish is plain steel.

5.3.3 External Sleeve: The central steel shaft of the external sleeve is 3.500-inch outer diameter by 0.216-inch wall thickness hollow structural section in conformance with ASTM A500 Grade B or C with a minimum yield strength of 50 ksi and a minimum tensile strength of 62 ksi. A 0.75-inch long collar, welded to one end, is manufactured with 4.000-inch by 0.226 wall thickness hollow structural section conforming to ASTM A53 Grade B, Type E & S with a minimum yield strength of 35 ksi and a minimum tensile strength of 60 ksi or the end of the external sleeve is trumpeted without a collar. The external sleeve shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

5.3.4 Brackets: Retrofit brackets FS288B, FS288BL, FS288BV and FS288BFM are suitable for use with the PP288 push pier system. Bracket finishes are either plain steel or hot-dip galvanized in accordance with ASTM A123. Bracket hardware finishes are zinc coated in accordance with ASTM B633.

5.4 Model PP350 Push Pier System

5.4.1 Starter and Pier Tube Sections: The central steel shaft of the starter and pier tube sections are 3.50-inch outer diameter by 0.165-inch nominal wall thickness hollow structural section in conformance with ASTM A500 Grade C with a minimum yield strength of 50 ksi and a minimum tensile strength of 55 ksi. The starter tube includes a 1.00-inch long factory-welded friction reduction collar manufactured from 4.00-inch outer diameter by 0.226-inch nominal wall thickness hollow structural section conforming to ASTM A53 Grade B, Type E & S with a minimum yield strength of 35 ksi and a minimum tensile strength of 60 ksi. The starter tube and pier tube shaft finishes are triple coated in-line galvanized and the friction reduction collar shaft finish is plain steel.

5.4.2 Shaft Coupling Material: The pier tube shaft coupling material is factory crimped or plug-welded to one end of the pier tube section and consists of 3.125-inch outer diameter by 0.180-inch nominal wall thickness hollow structural section in conformance with ASTM A53 Grade B, Type E & S with a minimum yield strength of 35 ksi and a minimum tensile strength of 60 ksi. The pier tube shaft coupling finish is plain steel.

5.4.3 External Sleeve: The central steel shaft of the external sleeve is 4.00-inch outer diameter by 0.226-inch wall thickness hollow structural section in conformance with ASTM A500 Grade B or C with a minimum yield strength of 50 ksi and a minimum tensile strength of 62 ksi. Two (2) bent steel stop plates are factory welded to one end of the external sleeve. The plates are manufactured from 0.75-inch wide by 2.88-inch long by 0.25-inch thick plate conforming to ASTM A36 with a minimum yield strength of 36 ksi and a minimum tensile strength of 58 ksi. The external sleeve shaft finish is either plain steel or hot-dip galvanized in accordance with ASTM A123.

5.4.4 Bracket: Retrofit bracket FS350BV is suitable for use with the PP350 push pier system. The bracket finish is either plain steel or hot-dip galvanized in accordance with ASTM A123. Bracket hardware finishes are zinc coated in accordance with ASTM B633.

5.5 Model PP400 Push Pier System

5.5.1 Starter and Pier Tube Sections: The central steel shaft of the starter and pier tube sections are 4.00-inch outer diameter by 0.226-inch nominal wall thickness hollow structural section in conformance with ASTM A500 Grade B or C with a minimum yield strength of 50 ksi and a minimum tensile strength of 62 ksi. The starter tube includes a 1.00-inch long factory-welded friction reduction collar welded to one end and is manufactured from 4.50-inch outer diameter by 0.237-inch nominal wall thickness hollow structural section conforming to ASTM A53 Grade B, Type E & S with a minimum yield strength of 35 ksi and a minimum tensile strength of 60 ksi. The starter shaft and pier tube shafts are either plain steel or hot-dip galvanized in accordance with ASTM A123.

5.5.2 Shaft Coupling Material: The pier tube shaft coupling material is factory plug-welded to the pier tube sections and consists of 3.50-inch outer diameter by 0.216-inch nominal wall thickness hollow structural section in conformance with ASTM A500 Grade B with a minimum yield strength of 42 ksi and a minimum tensile strength of 58 ksi. The pier tube shaft coupling finish is plain steel.

5.5.3 Bracket: Retrofit bracket FS400BV is suitable for use with the PP400 push pier system. The bracket finish is either plain steel or hot-dip galvanized in accordance with ASTM A123. Bracket hardware finishes are zinc coated in accordance with ASTM B633.

6 DESIGN AND PERFORMANCE REQUIREMENTS

6.1 Push piers shall be designed to support the nominal compressive load(s) as shown on the project Plans.

6.2 All structural steel pier components shall be designed within the limits provided by the American Institute of Steel Construction (AISC) Specification for Structural Steel Buildings (AISC-360). Either Allowable Stress Design (ASD) or Load and Resistance Factor Design (LRFD) are acceptable methods of analysis. Bracket testing in accordance with ICC-ES Acceptance Criteria 358 and/or Acceptance Criteria 406 may be considered as an acceptable means of establishing system capacities.

6.3 Except where noted otherwise on the project Plans, all piers shall be installed to provide an ultimate pier capacity based on an ASD or LRFD analysis. For ASD, a minimum factor of safety of 1.5 applied to the service or nominal loading shall be required. Higher ASD factors of safety may be required based on the project Plans or at the direction of the Owner. When an LRFD analysis is required, the Owner shall provide applicable pier design information including but not limited to; factored loads, resistance factors and/or the required ultimate pier capacity.

6.4 The required ultimate pier capacity shall be verified at each pier location by monitoring and recording final drive forces using the installation hydraulic pressure and the effective area of the drive cylinder. The maximum drive force shall not exceed the maximum drive force rating of the push pier system and installation tooling.

6.5 Except where noted otherwise on the project Plans, each pier shall be designed to meet a corrosion service life of 50 years in accordance with ICC-ES AC358 and AC406.

6.6 The pier design shall take into account pier buckling potential, soil stratification, and strain compatibility issues.

7 QUALIFICATIONS OF INSTALLING CONTRACTOR AND DESIGNER

7.1 The Installing Contractor and Pier Designer shall submit to the Owner, a proposal including the documentation required in this Section. Work shall not begin until all the submittals have been received and approved by the Owner. All costs associated with incomplete or unacceptable submittals shall be the responsibility of the Installing Contractor.

7.2 Evidence of Installing Contractor's competence in the installation of push piers shall be provided to the Owner's satisfaction and may include any or all of the following:

7.2.1 Pier manufacturer's certificate of competency in installation of push piers,

7.2.2 A list of at least three projects completed within the previous three years wherein the Installing Contractor installed push piers similar to those shown in the project Plans. Such list to include names and phone numbers of those project representatives who can verify the Installing Contractor's participation in those projects, and/or

7.2.3 A letter from the pier manufacturer or manufacturer's representative expressing ability and intent to provide on-site supervision of the pier installation.

7.3 A listing of all safety violations lodged against the Installing Contractor within the previous three years and the current status or final resolutions thereof. Descriptions of safety improvements instituted within the previous three years may also be submitted, at the Installing Contractor's discretion.

7.4 Evidence of Pier Designer's competence shall be provided to the Owner's satisfaction and may include any or all of the following:

7.4.1 Registration as a Professional Engineer or recognition by the local jurisdictional authority,

7.4.2 A list of at least three projects completed within the previous three years wherein the Pier Designer designed push piers similar to those shown in the project Plans. The list shall include names and phone numbers of those project representatives who can verify the Pier Designer's participation in those projects, and/or

7.4.3 Recommendation from the pier manufacturer or manufacturer's representative.

8 PRE-CONSTRUCTION SUBMITTALS

8.1 Within 2 weeks of receiving the contract award, the Installing Contractor and/or Pier Designer shall submit the following push pier design documentation:

8.1.1 Certification from the Pier Designer that the proposed piers meet the requirements of this specification.

8.1.2 Qualifications of the Installing Contractor and Pier Designer per Section 7.

8.1.3 Product designations for system components and ancillary products to be supplied at each push pier location.

8.1.4 Individual pier nominal loads, factors of safety, LRFD load and resistance factors and required ultimate pier capacities, where applicable.

8.1.5 Individual pier loading requirements (if any).

8.1.6 Manufacturer's published allowable system capacities for the pier assemblies, including load transfer devices.

8.1.7 Calculated mechanical and theoretical geotechnical capacity of the proposed piers.

8.1.8 Minimum final drive and lock-off force requirements.

8.1.9 Structural lift requirements, if applicable

8.1.10 Minimum and/or maximum embedment lengths or other site specific embedment depth requirements as may be appropriate for the site soil profiles.

8.1.11 Pier location tolerance requirements.

8.1.12 Load test procedures and failure criteria, if applicable.

8.1.13 Copies of certified calibration reports for load test measuring equipment to be used on the project, if applicable. The calibrations shall have been performed within one year of the proposed starting date for push pier installation or as recommended by the equipment manufacturer.

8.1.14 Provide proof of insurance coverage as stated in the general specifications and/or contract.

9 PIER INSTALLATION

9.1 Installing Contractor shall furnish and install all push piers per the project Plans and approved pier design documentation. In the event of conflict between the project Plans and the approved pier design documentation, the Installing Contractor shall not begin construction on any affected items until such conflict has been resolved.

9.2 The Installing Contractor shall conduct their construction operations in a manner to insure the safety of persons and property in the vicinity of the work. The Installing Contractor's personnel shall comply with safety procedures in accordance with OSHA standards and any established project safety plan.

9.3 The Owner shall request marking of underground utilities by an underground utility location service as required by law, and the Installing Contractor shall avoid contact with all marked underground facilities.

9.4 The portion of the construction site occupied by the Installing Contractor, his equipment and his material stockpiles shall be kept reasonably clean and orderly.

9.5 Installation of push piers may be observed by representatives of the Owner for quality assurance purposes. The Installing Contractor shall give the Owner at least 24 hours' notice prior to the pier installation operations.

9.6 The push pier installation technique shall be such that it is consistent with the geotechnical, logistical, environmental, and load carrying conditions of the project. The push pier shall be positioned at the appropriate site survey location as determined from the plan drawings.

9.7 Push pier installation procedures specified in the manufacturer's technical literature and/or code agency approved evaluation report shall be followed.

10 TERMINATION CRITERIA

10.1 The final drive force and any required pier length and embedment depth criteria as specified in the Pre-Construction Submittals shall be satisfied prior to terminating the pier installation. Push pier installation will be halted if excessive lift of the structure occurs prior to reaching the specified final drive force or pier length/depth criteria and the Owner will be notified of this occurrence prior to continuation of the work. The Owner shall be notified in the event any push pier fails to meet the production quality control termination criteria as specified on the Plans. In the event that the pier does not meet the production quality control termination criteria, the following remedies may be appropriate if approved by the Owner:

10.1.1 If the installation fails to meet the minimum final drive force criterion at the specified embedment length:

10.1.1.1 Continue the installation to greater depths until the minimum final drive force criterion is met, provided that, if a maximum length constraint is applicable, continued installation does not exceed said maximum length constraint

10.1.2 If the maximum drive force rating of the push pier system is achieved prior to satisfaction of a minimum embedment length criterion:

10.1.2.1 Terminate the installation at the depth obtained

10.1.2.2 Pre-drill to a depth that allows termination at or below the minimum embedment length

10.1.3 If the installation reaches a specified maximum embedment length without achieving the minimum final termination force criterion:

10.1.3.1 De-rate the load capacity of the push pier based on the final drive force recorded at termination depth and install additional piers as necessary.

10.1.4 If a push pier fails a production quality control criterion as described in this Section or for any reason other than described in this Section, any proposed remedy must be approved by the Owner prior to initiating its implementation at the project site.

11 INSTALLATION RECORD SUBMITTALS

11.1 The Installing Contractor shall provide the Owner copies of individual push pier installation records within 24 hours after each installation is completed. Formal copies shall be submitted within 30 day from the completion of the push pier installation. These installation records shall include, but are not limited to, the following information:

11.1.1 Date and time of installation

11.1.2 Location of push pier and pier identification number

11.1.3 Installed push pier model and configuration

11.1.4 Total length and tip depth of installed pier

11.1.5 Actual inclination of the pier

11.1.6 Hydraulic pressure reading at the end of each tube section installed

11.1.7 Final hydraulic pressure/force at the termination depth

11.1.8 Lift and/or lock-off pressure/force readings

11.1.9 Amount of lift achieved at each pier location (if applicable)

11.1.10 Calculated geotechnical capacity based on final drive and lock-off force resistance

11.1.11 Comments pertaining to interruptions, obstructions, or other relevant information

12 FIELD COMPRESSION LOAD TESTING

12.1 If field compression load testing is required, the Installing Contractor shall furnish all labor, equipment and pre-production push piers necessary to accomplish the testing as shown in the approved pier design documentation. Installing Contractor shall apply the specified loads for the specified durations and record the specified data, for the specified number of piers. No deviations from the test plan(s) will be allowed without explicit approval in writing from the Owner. Pier testing shall be in general accordance with the ASTM D1143 quick test method and the following criteria:

12.1.1 Failure criteria shall be determined by the Owner prior to load testing

12.1.2 The test pier shall have been installed to the required final drive force and then unloaded prior to start of test

12.1.3 The reaction frame requirements in ASTM D1143 shall not apply. The test setup will include calibrated pressure gages with a calibrated hydraulic ram installed in-line with the bracket and pier shaft to enable using the existing structure's weight as the reaction force during testing.

12.1.4 An alignment load equal to 5% of the maximum anticipated test load may be applied prior to the start of the test to take out slack in the test equipment.

12.1.5 Loading increments shall be in accordance with the ASTM D1143 quick test method with a maximum loading increment of 5% of the maximum anticipated test load and a minimum hold time of 4 minutes at each increment.

12.1.6 The maximum test load shall not exceed the final pier drive force determined in Section 12.1.2.

12.1.7 Upon completion of the maximum test load hold increment, the pier shall be unloaded in 5 to 10 even increments with minimum hold times of 4 minutes at each increment

12.2 Installing Contractor shall provide the Owner, copies of raw field test data within 24 hours after completion of each load test. Formal test reports shall be submitted within 30 days following test completion. Formal test reports shall include the following information:

12.2.1 Name of project and Installing Contractor's representative(s) present during load testing.

12.2.2 Name of manufacturer's representative(s) present during load testing, if any.

12.2.3 Name of third party test agency and personnel present during load testing, if any.

12.2.4 Date, time, duration and type of the load test.

12.2.5 Unique test identifier and map showing the test pier location.

12.2.6 Pier model and installation information including drive pressure/force records of each pier tube, final drive pressure/force, drive tube quantities and lengths, final pier tip depth, installation date, and total test pier length.

12.2.7 Calibration records for applicable pier installation and test equipment

12.2.8 Tabulated test results including cumulative pier head movement, loading increments and hold times

12.2.9 Plots showing load versus deflection for each loading/unloading interval

13 CLEANUP

13.1 Within one week of completion of the work, the Installing Contractor shall remove any and all material, equipment, tools, debris or other items belonging to the Installing Contractor or used under the Installing Contractor's direction.

CHAPTER 4 MISCELLANEOUS STRUCTURAL SUPPORT PRODUCTS

4.1 - Introduction

Foundation Supportworks' helical foundation systems and hydraulically-driven push pier systems are widely-accepted, proven deep foundation support options for both residential and commercial loading conditions. FSI is also the manufacturer or exclusive distributor of a number of other products that are more commonly marketed for and installed on residential projects, yet are occasionally considered for commercial applications as well.

The Geo-Lock® Wall Anchor System, PowerBrace™ System and CarbonArmor® Wall Reinforcing System are used to stabilize foundation walls that are experiencing inward movement and/or distress due to excessive lateral earth or hydrostatic pressures generated by unbalanced soil conditions. The Geo-Lock wall anchor system is also routinely used to stabilize failing retaining walls. These systems may prove to be ideal solutions for certain commercial projects when compared to helical tiebacks (see Section 2.8) or other proposed options because of the smaller equipment/tools needed for installation, the ability to install the systems in areas of limited or difficult access, and smaller penetrations or no penetrations needed through the walls. While the system capacity ratings are generally better-suited to residential loading conditions for which they were originally considered and designed, these products have all found practical uses and applications elsewhere.

The SmartJack® System provides supplemental support within a crawl space. SmartJacks are installed adjacent to existing settling columns or along the span of sagging beams and floor joists. Again, while not used as frequently in commercial stabilization projects, FSI has seen applications for SmartJacks in churches, office buildings, and hospitals where the structure is built on a crawl space foundation.

4.2 Geo-Lock® Wall Anchor System

4.2.1 Summary Description

The Geo-Lock Wall Anchor System designed and manufactured by Foundation Supportworks is a proven method to laterally support bowed, leaning and sheared foundation walls and retaining walls subject to unbalanced earth pressures. Similar-type "plate anchor" systems have been used successfully since the late 1970s to stabilize foundation walls and retaining walls against further appreciable lateral movement. The system consists of an earth anchor buried in the ground an adequate distance from the structure, an interior wall plate set against the wall face being supported, and an anchor rod to connect the two *(Figure 4.2.1.a)*. The passive resistance of the soil in front of the earth anchor resists lateral forces on the wall and further inward movement. Technical specifications for the Geo-Lock Wall Anchor System are included in Appendix 4A.

4.2.1.1 Advantages

Some of the advantages to installing Geo-Lock wall anchors over other wall bracing systems may include:

- Can be installed year-round
- Most jobs completed in one day
- Minimal disturbance to home, lawn and landscaping
- Can straighten walls over time (in many cases)
- Will not damage interior flooring
- Easily hidden within framing of walls

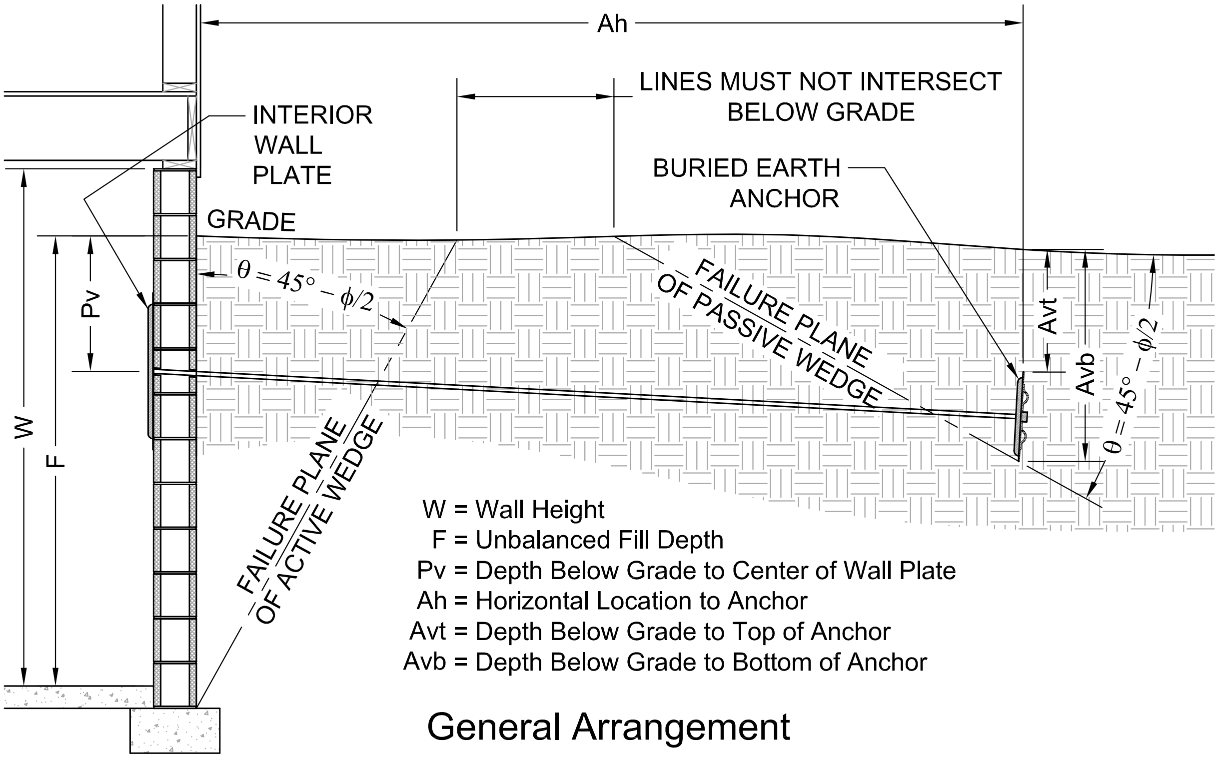


Figure 4.2.1.a General Geo-Lock® Wall Anchor System Arrangement

4.2.2 Design Theory

Wall anchor systems have been proven over the years to be both an effective and economical repair solution. Wall anchors not only arrest further appreciable inward movement of the wall, but in many cases can improve the wall's position and partially or fully straighten it. In fact, in 1992, wall anchor systems were identified by the Chief Appraiser for the U.S. Department of Housing and Urban Development as the most effective means of stabilizing bowing foundation walls. Despite its tremendous track record, it remains, however, a system that in some ways is misunderstood by many design professionals, building departments and code officials.

Why is this? Although the general concepts about how anchor systems work are simple to grasp, detailed evaluations are more elusive than many designers initially suspect.

The first and most significant misstep that is made is in the way the soil loads on the wall are defined and quantified. Most mathematical evaluations of a wall anchor system define soil load as an equivalent fluid pressure gradient. Most often about 60 psf/ft is used and sometimes adjustments are made for the potential existence of hydrostatic pressure from a water table. Soil is obviously not perfectly rigid so it will tend to deform and migrate laterally when subjected to the compressive forces produced by gravity. When designing a wall, soil is placed against a static structure. The soil will exert these lateral forces produced by gravity against the wall which makes the equivalent fluid pressure method a seemingly reasonable approach. However, soil is obviously not a fluid so when evaluating a system being used to actively push back against the soil, the equivalent fluid pressure model quickly breaks down.

This can be demonstrated in the following example: A bowing block wall will typically manifest a long horizontal crack at a mortar bed joint. The soil pressure tends to produce tensile stresses on the inside face of the wall. These stresses have exceeded the capacity of the material, thus resulting in a crack. If the top and bottom of the wall are considered laterally fixed and the soil loads are modeled using equivalent fluid pressure, a designer could calculate how much lateral force needs to be applied from the interior side of the wall at the elevation of the crack to reverse the bending in the wall. This mathematically results in compressive stress on the interior face and tensile stress on the exterior face. With compressive stress on the interior face, the horizontal crack would be expected to close. In practice, however, if this calculated resistance force were to be applied per the described analysis, the crack will likely remain. Even if this applied resistance force exceeded the calculated value by several orders of magnitude, the crack will likely remain. What is actually happening is the anchor forces applied at the interior are tending to impose translational displacement of the wall and the soil. The soil is resisting movement, and the force it exerts on the exterior of the wall (passive resistance) increases to match the forces applied on the interior face of the wall. The equivalent fluid pressure assumption has therefore led us to an erroneous solution.

There are other phenomena that are difficult to explain with an equivalent fluid pressure model. One would expect the aforementioned wall crack to appear at the point of maximum bending. The crack often appears at a higher elevation than the equivalent fluid pressure model would predict. In fact, it can be very close to the elevation of the exterior grade. In this case, frost is often the culprit. Although a true water table is most often not present, there can be significant soil moisture near the surface. When frozen, this can exert very large forces on the wall resulting in a horizontal crack much higher on the wall than an equivalent fluid pressure model would predict. These forces are not only large, but difficult to quantify.

The second item that is often misrepresented in analysis is the load in the anchors. Many designers will attempt to treat the anchors as reaction points in their analysis. They treat the wall as being laterally fixed at the top and bottom and with a third “support” at the anchor location. They then model a gradient load to represent equivalent fluid pressure. This is now a statically indeterminate structure which requires a more complex analysis. Unfortunately, this effort is wasted for a couple of reasons. The first reason is that, as has already been discussed, the equivalent fluid pressure model will yield erroneous results. The second reason is that the load in the anchor should already be known because it is actually an applied force and not a calculated reaction. Anchors are supplied with a threaded rod and nut with wax typically used as a lubricant. The nuts are tightened with a torque wrench to a specific torque value. This installed torque relates directly to tension in the rod. There is no need to calculate the load in the anchor since it is already known with the application of torque.

Some designers recognize the wall anchors as applied forces but still find it necessary to treat them as calculated reactions. They are aware that the forces that are applied at the anchors are far in excess of those that will be calculated as reactions. The designer may then make a comparative rationalization demonstrating that, for example, the force applied at the anchor is twice as much as a middle support would offer and therefore, the anchor system should be more than adequate. Once again, the soil pressure distribution exerted on the wall after the anchors are installed will bear little resemblance to an equivalent fluid pressure gradient so applying twice as much force than a fictional calculated reaction value really serves no purpose.

4.2.3 Why It Works

With all these caveats in the analysis it might appear that providing a plausible explanation for why these systems are so effective would be impossible. Actually, it's quite the opposite. Without the discussion above, one may find it difficult to understand why these systems work so well. After all, the wall often straightens over time when the anchors are tightened to the recommended torque and at the recommended schedule. This is a measurable effect. The wall is contacting a great deal more soil than the earth anchors, so how is it possible to straighten the wall without pulling the earth anchors through the soil?

The answer is actually quite simple. Structures with below ground basements are typically constructed in areas where clay soils are present near the surface. Clay soils have cohesive properties as well as the potential for volume changes with variations in moisture levels. This means that during dry periods, the soils can shrink away from the foundation wall. The cohesive properties in the soil allow it to stand on its own and create thin spaces between the soil and the foundation wall. The tension on installed anchor rods then decreases. This creates an opportunity to tighten the anchors and take advantage of the gaps that have formed between the foundation wall and soil. This moves the wall slightly closer to a straight position. This cycle can be repeated until the desired result is achieved. This also explains why some walls will see better results than others. If the soil is particularly sandy, it will not generally be as cohesive meaning that even during dry periods, the soil will not relieve the pressure on the anchors. Although wall anchor systems are still effective in arresting further appreciable movement in these types of soils, the likelihood that the position of the wall will improve is reduced.

The most important functional consideration for an "active" resistance system such as earth anchors is to provide steady, constant pressure to the wall during the wall straightening process.

4.2.4 Installation Guidelines

Although many of the forces involved are difficult to quantify, the successful performance of the product is undeniable. This therefore leaves us with experience. In the following subsections, we offer general installation guidelines for spacing, tightening, and depth and location of the earth anchor. Deviations to these guidelines may be considered by a qualified design professional based on project-specific variables.

Literally thousands of basement walls have been stabilized with these guidelines and with great results. Non-typical applications, walls that are significantly compromised, walls that have evidence of shear displacement at the bottom, or walls that are more than 2 to 3 inches out of plumb should be given special consideration.

4.2.4.1 Spacing

The designer will consider several factors when providing recommendations for anchor spacing. These factors include the wall height and thickness, the retained height of the backfill, and the general condition and position of the wall. One of the most common situations is for an 8-inch-thick concrete block residential basement wall that is 9 feet tall with 8 feet of unbalanced fill. This scenario most commonly results in a spacing recommendation of 5 feet between anchors and 3 feet from corners. Another common situation would be for an 8-inch-thick poured concrete residential basement wall that is also 9 feet tall with 8 feet of unbalanced fill. This scenario most commonly results in a spacing recommendation of 6 feet between anchors and 3.5 feet from corners.

4.2.4.2 Torque Recommendations

Torque applied to the nuts during the tightening process of the wall anchor system correlates directly to tension in the rod and force applied to the wall. FSI recommends that applied torque not exceed 80 foot-pounds (ft-lb) for block walls and 90 ft-lb for poured concrete walls. These torque values assume that FSI Anchor Wax is applied to the threads on the rod which significantly reduces friction between the rod and nut and results in a higher applied force than nuts tightened to similar torque in a dry condition. The average applied force noted in *Figure 4.2.4.2.a* was generated from dozens of test samples with testing completed at an independent test facility. Due to product variations, these values should only be considered applicable to products supplied by Foundation Supportworks.

Applied Torque (ft-lb)	Average Applied Force (lb)	
	Waxed Condition	Dry Condition
80	11,900	6,100
90	12,900	6,900

Figure 4.2.4.2.a Average applied force on the anchor rod versus applied torque on the anchor rod nut

Installers shall closely monitor the tightening process of the wall anchor installation and reduce the applied torque as necessary for atypical conditions.

4.2.4.3 Depth and Location of Earth Anchor

The Geo-Lock Wall Anchor System is designed with two (2) ¾-inch diameter, 80-inch long all-thread rods coupled together. This total rod length of 13 feet 4 inches allows the earth anchor to be approximately 12 feet from the stabilized wall, far enough to prevent load from the anchor being transferred back to the wall. Considering the backfill height, the vertical placement of the earth anchor, and a range of soil types and strengths, the coupled rod length of 13 feet 4 inches would be adequate for typical applications with backfill heights up to about 8 feet. Additional all-thread rod sections can easily be added as necessary for backfill heights greater than 8 feet.

The following tables provide the horizontal location of the earth anchor from the exterior face of the foundation wall (Ah) and the earth anchor depths (Avt and Avb) considering a variety of soil conditions. Refer to *Figure 4.2.1.a* when using these tables. One quickly observes that soil type has little effect and changes Ah and Avb only slightly. Rather, values of Ah and Avb are driven more by minimum depth criteria and geometry.

Soil Description: Medium Dense Sand and Gravel
Internal angle of friction (Φ) = 34 degrees

“Ah” Minimum Required Horizontal Location to Anchor (ft)											
“Pv” Depth Below Grade to Center of Wall Plate (ft)		1		2		3		4		5	
		Anchor Size		Anchor Size		Anchor Size		Anchor Size		Anchor Size	
		Small	Med/Lrg	Small	Med/Lrg	Small	Med/Lrg	Small	Med/Lrg	Small	Med/Lrg
“F” Unbalanced Fill Depth (ft)	10	12.2	13.0	12.2	13.0	13.0	13.8	14.9	15.7	16.8	17.5
	9	11.7	12.5	11.7	12.5	12.5	13.2	14.4	15.1		
	8	11.2	11.9	11.2	11.9	11.9	12.7	13.8	14.6		
	7	10.6	11.4	10.6	11.4	11.4	12.2				
	6	10.1	10.9	10.1	10.9	10.9	11.7				
	5	9.6	10.3	9.6	10.3						
	4	9.0	9.8	9.0	9.8						
	3	8.5	9.3								

“Avb & Avt” Minimum Required Depths Below Grade to Anchor (ft)										
Bottom - Avb:	3.7	4.1	3.7	4.1	4.1	4.5	5.1	5.5	6.1	6.5
Top - Avt:	Depth as needed to prevent frost effects									

Soil Description: Loose Sand and Gravel

Internal angle of friction (Φ) = 30 degrees

"Ah" Minimum Required Horizontal Location to Anchor (ft)											
"Pv" Depth Below Grade to Center of Wall Plate (ft)		1		2		3		4		5	
		Anchor Size		Anchor Size		Anchor Size		Anchor Size		Anchor Size	
		Small	Med/Lrg	Small	Med/Lrg	Small	Med/Lrg	Small	Med/Lrg	Small	Med/Lrg
"F" Unbalanced Fill Depth (ft)	10	12.1	12.8	12.1	12.8	12.9	13.6	14.6	15.3	16.3	17.0
	9	11.6	12.3	11.6	12.3	12.3	13.0	14.0	14.7		
	8	11.0	11.7	11.0	11.7	11.7	12.4	13.4	14.1		
	7	10.4	11.1	10.4	11.1	11.1	11.8				
	6	9.8	10.5	9.8	10.5	10.5	11.3				
	5	9.2	10.0	9.2	10.0						
	4	8.7	9.4	8.7	9.4						
	3	8.1	8.8								

"Avb & Avt" Minimum Required Depths Below Grade to Anchor (ft)										
Bottom - Avb:	3.7	4.1	3.7	4.1	4.1	4.5	5.1	5.5	6.1	6.5
Top - Avt:	Depth as needed to prevent frost effects									

Soil Description: Silt, Silty/Clayey Sand and Gravel

Internal angle of friction (Φ) = 26 degrees

"Ah" Minimum Required Horizontal Location to Anchor (ft)											
"Pv" Depth Below Grade to Center of Wall Plate (ft)		1		2		3		4		5	
		Anchor Size		Anchor Size		Anchor Size		Anchor Size		Anchor Size	
		Small	Med/Lrg	Small	Med/Lrg	Small	Med/Lrg	Small	Med/Lrg	Small	Med/Lrg
"F" Unbalanced Fill Depth (ft)	10	12.1	12.8	12.1	12.8	12.8	13.5	14.4	15.1	16.0	16.7
	9	11.5	12.2	11.5	12.2	12.2	12.8	13.8	14.4		
	8	10.9	11.5	10.9	11.5	11.5	12.2	13.1	13.8		
	7	10.2	10.9	10.2	10.9	10.9	11.6				
	6	9.6	10.3	9.6	10.3	10.3	11.0				
	5	9.0	9.7	9.0	9.7						
	4	8.4	9.0	8.4	9.0						
	3	7.7	8.4								

"Avb & Avt" Minimum Required Depths Below Grade to Anchor (ft)										
Bottom - Avb:	3.7	4.1	3.7	4.1	4.1	4.5	5.1	5.5	6.1	6.5
Top - Avt:	Depth as needed to prevent frost effects									

Soil Description: Silty Clay, Clay with Sand

Internal angle of friction (Φ) = 18 degrees

"Ah" Minimum Required Horizontal Location to Anchor (ft)											
"Pv" Depth Below Grade to Center of Wall Plate (ft)		1		2		3		4		5	
		Anchor Size		Anchor Size		Anchor Size		Anchor Size		Anchor Size	
		Small	Med/Lrg	Small	Med/Lrg	Small	Med/Lrg	Small	Med/Lrg	Small	Med/Lrg
"F" Unbalanced Fill Depth (ft)	10	12.3	12.9	12.3	12.9	12.9	13.5	14.3	14.8	15.6	16.2
	9	11.6	12.2	11.6	12.2	12.2	12.7	13.5	14.1		
	8	10.9	11.4	10.9	11.4	11.4	12.0	12.8	13.4		
	7	10.1	10.7	10.1	10.7	10.7	11.3				
	6	9.4	10.0	9.4	10.0	10.0	10.6				
	5	8.7	9.2	8.7	9.2						
	4	8.0	8.5	8.0	8.5						
	3	7.2	7.8								

"Avb & Avt" Minimum Required Depths Below Grade to Anchor (ft)										
Bottom - Avb:	3.7	4.1	3.7	4.1	4.1	4.5	5.1	5.5	6.1	6.5
Top - Avt:	Depth as needed to prevent frost effects									

Soil Description: Saturated Clay Soils

Internal angle of friction (Φ) = 0 degrees

"Ah" Minimum Required Horizontal Location to Anchor (ft)											
"Pv" Depth Below Grade to Center of Wall Plate (ft)		1		2		3		4		5	
		Anchor Size		Anchor Size		Anchor Size		Anchor Size		Anchor Size	
		Small	Med/Lrg	Small	Med/Lrg	Small	Med/Lrg	Small	Med/Lrg	Small	Med/Lrg
"F" Unbalanced Fill Depth (ft)	10	13.7	14.1	13.7	14.1	14.1	14.7	15.1	15.5	16.1	16.5
	9	12.7	13.1	12.7	13.1	13.1	13.7	14.1	14.5		
	8	11.7	12.1	11.7	12.1	12.1	12.7	13.1	13.5		
	7	10.7	11.1	10.7	11.1	11.1	11.7				
	6	9.7	10.1	9.7	10.1	10.1	10.7				
	5	8.7	9.1	8.7	9.1						
	4	7.7	8.1	7.7	8.1						
	3	6.7	7.1								

"Avb & Avt" Minimum Required Depths Below Grade to Anchor (ft)										
Bottom - Avb:	3.7	4.1	3.7	4.1	4.1	4.5	5.1	5.5	6.1	6.5
Top - Avt:	Depth as needed to prevent frost effects									

4.2.5 Installation Steps

The following steps provide a broad overview of a typical Geo-Lock Wall Anchor System installation. Intermediate steps, installation equipment and tools used, and considerations for unusual conditions or applications are not addressed.

Step 1 – Sod is carefully removed and a hole is excavated or augered *(Figure 4.2.5.a1)*. The front face of the hole (toward structure) is cut flat to accept the earth anchor.

Figure 4.2.5.a1

Step 2 – A small 1 ⅛-inch hole is drilled through the basement wall and the anchor rod is driven out to penetrate the augered hole *(Figure 4.2.5.a2)*.

Figure 4.2.5.a2

Step 3 – Earth anchor is placed in the augered hole and attached to the anchor rod *(Figure 4.2.5.a3)*.

Figure 4.2.5.a3

Step 4 - Interior wall plate is positioned over the anchor rod and tightened to the specified torque, seating the earth anchor and engaging the passive resistance of the soils *(Figure 4.2.5.a4)*.

Figure 4.2.5.a4

Step 5 - Augered hole is backfilled and compacted and the sod is replaced *(Figure 4.2.5.a5)*.

Figure 4.2.5.a5

Step 6 - Anchors can be tightened at specified intervals to straighten wall over time, if desired *(Figure 4.2.5.a6)*. Tightening of the anchors generally occurs during drier seasons of the year when there is less pressure on the wall and shrinkage gaps may have formed between the wall and the soil.

Figure 4.2.5.a6

4.2.5.1 Solutions for Extreme Wall Problems

An owner may wish to have a basement wall or retaining wall straightened immediately, rather than follow the periodic tightening procedure. The soil behind the wall must then be completely removed down to the footing. With the soil load temporarily removed, the wall can be straightened. The Geo-Lock system then stabilizes the wall against future inward movement. See *Figures 4.2.5.1.a1 and 4.2.5.1.a2*.

Figure 4.2.5.1.a1 Before: Poured concrete basement wall severely leaning in at the top

Figure 4.2.5.1.a2 After: Wall is immediately returned to original straight position after backfill soil is removed and Geo-Lock anchors installed

Block walls in later stages of distress may show inward shear movement at the bottom of the wall. The bottom row of the block wall is restrained by the basement floor slab while the wall section above continues to deflect inward. The movement at the mortar joint between the first and second courses of block is often obvious and measureable. In order to recover this movement and straighten the wall, the backfill soil must again be completely excavated down to the footing. The base of the wall is pushed back and Geo-Lock wall anchors are installed *(Figures 4.2.5.1.b1 and 4.2.5.1.b2)*. In circumstances such as these, the interior wall plate of the Geo-Lock system is often replaced with a steel channel connected to the anchor rod and bolted to the concrete floor *(Figure 4.2.5.1.b3)*.

Figure 4.2.5.1.b1 Before: Shear movement of a block wall evident from exterior

Figure 4.2.5.1.b2 After: Shear movement recovered and wall stabilized with Geo-Lock anchors

Figure 4.2.5.1.b3 Geo-Lock channel anchor system installed to prevent shear movement

CHAPTER 4
MISCELLANEOUS STRUCTURAL SUPPORT PRODUCTS

4.2.5.2 Hide-A-Way®
Wall Anchor Cover

An optional final step of installation is to install Hide-A-Way Wall Anchor Covers. These covers are ideal for those situations where the customer has a finished basement but would like to have continued access to the Geo-Lock system for tightening and wall straightening over time. The cover conceals the anchor, but then allows for easy access *(Figures 4.2.5.2.a1, 4.2.5.2.a2 and 4.2.5.2.a3)*.

Figure 4.2.5.2.a1 Geo-Lock wall anchor installed through drywall penetration and between wood studs

Figure 4.2.5.2.a2 Frame for Hide-A-Way cover installed around drywall opening

Figure 4.2.5.2.a3 Hide-A-Way wall anchor cover installed

4.3 PowerBrace™ System

4.3.1 Summary Description

The PowerBrace™ System is designed and manufactured by PowerBrace, LLC to laterally support bowed, leaning and sheared foundation walls. The system has been used for over a decade with thousands of successful installations throughout the United States and Canada. The PowerBrace™ System is a patented system that, when installed properly, will not only stabilize foundation walls against further appreciable lateral movement, but also in many cases will improve the wall's position over time. A steel beam is positioned against the foundation wall and braced at the top and bottom with brackets. The bottom angle bracket is bolted to the concrete floor. The adjustable top bracket is connected to the joists supporting the floor system above *(See Figure 4.3.1.a)*. Technical specifications and spacing guidelines for the PowerBrace™ System can be found in Appenix 4B.

4.3.2 Installation Steps

The following steps provide a broad overview for a typical PowerBrace™ installation. Intermediate steps, installation equipment and tools used, considerations for obstructions along the wall, and considerations for variable joist details are not addressed. It is critical that adequate blocking be installed along and between floor joists supporting the first floor so loads are adequately and effectively transferred into the floor system without damage. Contact PowerBrace, LLC at (800) 556-5697 with technical questions and for a copy of the manufacturer's installation guidelines.

Step 1 – Measure from the top of the basement slab to 1 inch from the underside of the first floor. Cut the PowerBrace™ beam to this length *(Figure 4.3.2.a1)*.

Figure 4.3.2.a1

Figure 4.3.2.a1
Rendering of PowerBrace™ installation

Step 2 – The top bracket is attached to the floor joist and the top of the beam is positioned within the bracket *(Figure 4.3.2.a2)*. Install blocking along and within floor joists in accordance with the manufacturer's installation guidelines.

Figure 4.3.2.a2

Step 3 – PowerBrace™ beam is plumbed *(Figure 4.3.2.a3)*.

Figure 4.3.2.a3

Step 4 – The bottom bracket is positioned at the bottom of the beam and anchored to the concrete floor *(Figure 4.3.2.a4)*.

Figure 4.3.2.a4

Step 5 – The PowerBrace™ System can be tightened over time for possible wall improvement *(Figure 4.3.2.a5)*.

Figure 4.3.2.a5

4.4 CarbonArmor® Wall Reinforcing System

4.4.1 Summary Description

Foundation Supportworks offers high-strength fiber-reinforced polymer (FRP) composite materials to reinforce concrete block and poured concrete foundation walls. These high-strength composite systems effectively combine the benefits of epoxies and high-strength carbon fibers to create materials that are both extremely reliable and versatile. When used in conjunction with concrete substrates, like basement walls, significant increases to both structural integrity and load capacity can be achieved *(Figure 4.4.1.a)*.

The materials used in the CarbonArmor system achieve the highest strength-per-ply available, and are supported by the most extensive application and durability testing in the industry. The CarbonArmor Wall Reinforcing System is a proven method to laterally support bowing foundation walls. Technical specifications and spacing recommendations are included in Appendix 4C.

Figure 4.4.1.a
Rendering of CarbonArmor® installation with optional ArmorLock™ connection to rim joist

4.4.1.1 Advantages

- Ideal to reinforce the inside face of block walls and prevent further bowing in the middle
- Optional ArmorLock™ system resists movement at the top of the wall
- Flexible material allows the strap to contour tight against the wall
- May be installed around obstructions on the wall such as piping, electric conduit and HVAC ductwork
- Completed installations are suitable for painting
- Low profile system can be concealed by wall framing

4.4.2 Installation Steps

Step 1 – Prepare the wall surface for the CarbonArmor strap. Concrete surfaces must be structurally sound and free from contaminants such as dust, dirt, or oil. Surfaces must be mechanically abraded to remove protrusions from the wall that may trap air behind the installed CarbonArmor strap *(Figure 4.4.2.a1)*.

Figure 4.4.2.a1

Step 2 – Measure the wall and cut strap to proper length *(Figure 4.4.2.a2)*.

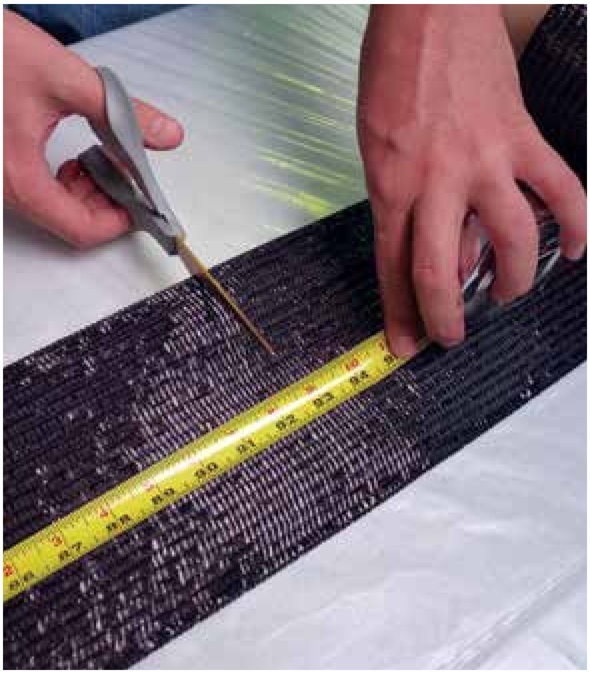


Figure 4.4.2.a2

Step 3 – Thorough and complete mixing of the epoxy is critical for the performance of the product. Mix with a wooden paint stirrer for at least two minutes until a consistent color is achieved. Scrape the mixing container sides and bottom and mix until no stripes, streaks or color variations are visible. Do not mix more material than may be applied within 30 minutes. Apply mixed resin to the prepared surface using the roller provided *(Figure 4.4.2.a3)*.

Figure 4.4.2.a3

Step 4 - Completely saturate the CarbonArmor strap with epoxy resin *(Figure 4.4.2.a4)*.

Figure 4.4.2.a4

Step 5 – Adhere the CarbonArmor strap to the wall *(Figure 4.4.2.a5)*.

Figure 4.4.2.a5

Step 6 – Apply a final topcoat of epoxy. Use the provided roller to apply pressure to the surface of the fabric to ensure good bonding to the wall and to roll out any trapped air before the epoxy sets *(Figure 4.4.2.a6)*.

Figure 4.4.2.a6

4.4.2.1 ArmorLock™ System

The optional ArmorLock™ bracket and FRP system essentially splices with CarbonArmor straps to extend reinforcement to the building's rim joist framing. ArmorLock provides additional reinforcement and stabilization in situations where either movement is observed at the top of the wall, or the owner wishes to prevent such movement.

Step 1 - Bracket is secured to the rim joist with heavy-duty lag screws *(Figure 4.4.2.1.a1)*.

Figure 4.4.2.1.a1

Step 2 – ArmorLock strap is saturated with epoxy resin *(Figure 4.4.2.1.a2)*.

Figure 4.4.2.1.a2

Step 3 – ArmorLock system is connected to the bracket and tensioned into position *(Figure 4.4.2.1.a3)*.

Figure 4.4.2.1.a3

Step 4 – Strap is adhered to the prepared foundation wall *(Figure 4.4.2.1.a4)*. See Step 1 of Section 4.4.2 for wall preparation recommendations.

Figure 4.4.2.1.a4

Step 5 – CarbonArmor strap is installed over and bonded to the ArmorLock by saturating both with epoxy *(Figure 4.4.2.1.a5)*.

Figure 4.4.2.1.a5

Step 6 – Installation of CarbonArmor Wall Reinforcing System with optional ArmorLock system complete *(Figure 4.4.2.1.a6)*.

Figure 4.4.2.1.a6

4.4.2.2 Special Installation Considerations

- Do not apply to concrete less than 30 days old.
- Do not apply to concrete with curing or sealing membranes.
- Do not apply to concrete with a surface temperature less than 45°F.
- Do not thin epoxy with solvent.
- High temperature environments may require a special hardener selection. Consult with a FSI representative for recommendations in these situations.

4.5 SmartJack® System

4.5.1 Summary Description

The Foundation Supportworks SmartJack is a supplemental support system most commonly used for crawl space applications. The SmartJack effectively supports sagging beams and floor joists caused by:

- length of span greater than spanning capability of the members,
- floor load added after construction exceeding design values, and
- weakening of members over time due to high moisture and rot.

The SmartJack may also be used as a supplemental column support where an existing column and pier foundation has settled. The system generally consists of a two-foot square by two-foot deep excavation backfilled with crushed stone, a precast concrete footing, round steel tube, threaded rod and top plate *(Figure 4.5.1.a)*. A concrete footing could be considered in lieu of, or as a partial replacement for, the cube of crushed stone. The crushed stone not only absorbs and dissipates the structural loads to the surrounding soils, but it also allows for SmartJack installation to often be completed in one day. Model 288 (2.875-inch O.D. tube) and Model 350 (3.50-inch O.D. tube) SmartJack technical specifications and design guides are included in Appendix 4D.

4.5.1.1 Advantages

- Installs in tight conditions where height or access is limited
- Crushed stone footing "system" effectively transfers load to existing soils
- Can usually be installed in less than one day
- Immediate stabilization and results
- Can be installed in conjunction with a crawl space liner

Figure 4.5.1.a
Rendering of SmartJacks providing supplemental support for a beam in a crawl space

4.5.2 Installation Steps

The following steps provide a broad overview of a typical SmartJack system installation. Intermediate steps, installation equipment and tools used, and considerations for unusual conditions or applications are not addressed.

Step 1 – Two-foot cube of soil is excavated and backfilled with compacted crushed stone *(Figure 4.5.2.a1)*.

Figure 4.5.2.a1

Step 2 – Pre-cast concrete or cast-in-place footing installed above the crushed stone *(Figure 4.5.2.a2)*.

Figure 4.5.2.a2

Step 3 – Galvanized steel tube is cut to the appropriate length *(Figure 4.5.2.a3)*.

Figure 4.5.2.a3

Step 4 – Steel column and components are assembled and connected to the beam or floor joists *(Figure 4.5.2.a4)*.

Figure 4.5.2.a4

Step 5 – SmartJack System is tightened in place, stabilizing the beams and potentially lifting the above floors and walls back toward level *(Figure 4.5.2.a5)*. The excavation can be made deeper to set the pre-cast footing below grade and then cover it with crushed stone.

Figure 4.5.2.a5

Step 6 – SmartJack System can be installed in conjunction with a crawl space liner *(Figure 4.5.2.a6)*.

Figure 4.5.2.a6

Appendix 4A

Geo-Lock® Wall Anchor System

Geo-Lock® WALL ANCHOR SYSTEM
Technical Specifications

Plate Steel:
ASTM A1011 C1008-C1010, 10 gauge plate embossed with two (2) longitudinal ribs.

Wall Plates:
Two sizes: 12” x 18” and 12” x 28”

Earth Plates:
Fabricated from two wall plates welded in a cross pattern. One inch on each end of wall plates are bent 90 degrees. Three sizes: 16” x 16”, 16” x 26”, and 26” x 26”.

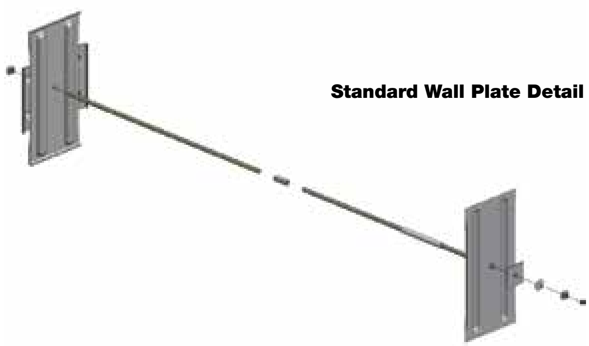
All-Thread Rod:
Medium Carbon Steel. Tensile strength = 85 ksi (min.), ¾ - 10 UNC 2A, 80” long (assembly consists of two rods). Allowable tensile capacity = 14 kips.

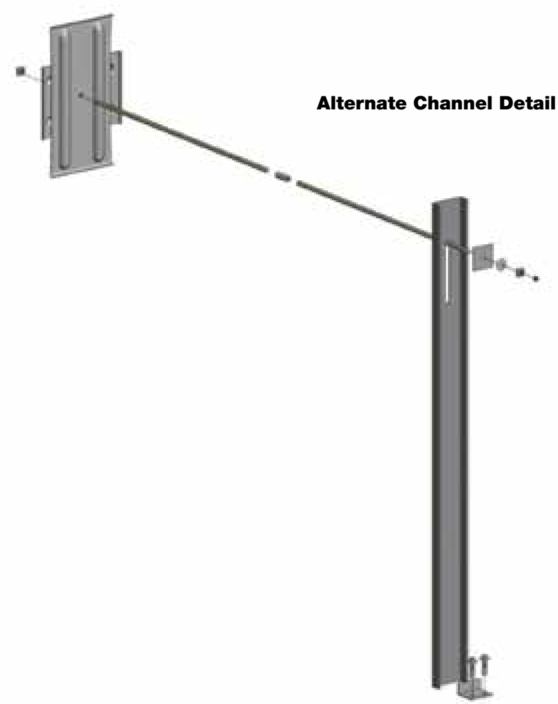
Rod Coupler:
AISI 1144 Yield strength = 100 ksi (min.), Tensile strength = 115 ksi (min.) ¾ - 10 UNC 2B, oversized tap, 3” long x 0.984” diameter.

Termination Hardware:
SAE J995 Grade 2 heavy square nuts ¾ - 10 UNC 2B, oversized tap.

Finish:
All components are hot-dip galvanized in accordance with ASTM A123 or ASTM A153.

Alternate Wall Bracing Detail:
C6 x 8.2 steel channel, ASTM A36. Channel secured at base with steel bracket or cast into concrete. Bracket is 3.75” x 1.75” x 0.25” thick x 4.5” long bent plate, ASTM A36, with (2) Ø0.875 holes. (2) Ø0.75” x 2.5” sleeve anchors.





1. Installation of any tieback or anchor system in potential landslide or hillside creep soils should be designed by a professional engineer.
2. Refer to Section 4.2.4 of the FSI Technical Manual for recommendations on anchor spacing, depth and location of earth plate, and installation torque.

APPENDIX 4A
GEO-LOCK® WALL ANCHOR SYSTEM

Appendix 4B

PowerBrace™ System

PowerBrace™ System
Technical Specifications & Spacing Guidelines

Steel Beam:
S4x7.7 ASTM A572 Grade 50 with length of 8 or 9 feet.

Top Bracket Assembly:
⅜” ASTM A36 L-shaped bent plate 3.50” x 3.50” x 5” long
with holes for bracket hardware field cut to length.
2.50” long ASTM A36 bent plate beam capture
(1) - Ø1” x 5” long ASTM A307 bolt with nut
(2) - Ø¾” x 3” long ASTM A307 bolts with nuts and washers
Top assembly also available as a larger bracket with 4 bolts instead of 2.

Bottom Bracket Assembly:
¼” ASTM A36 L-shaped bent plate 2.50” x 1.75” x 5.50” long
with holes for bracket hardware.
(2) - Ø¾” Red Head Dynabolt sleeve anchor with effective length = 2.50”

Surface Finish:
All components of the bracket assemblies and the steel beam are electrozinc plated per ASTM B633.

		Wall Height (ft.)					
		4	5	6	7	8	9
Unbalanced fill Height (ft.)	4	6.0	6.0	6.0	6.0	6.0	6.0
	5		6.0	6.0	6.0	5.5	5.0
	6			6.0	5.5	5.0	4.5
	7				5.0	4.5	4.0
	8					4.0	3.5
	9						3.0

1. Maximum recommended spacing from corners is 2 feet.
2. Spacing could be less than listed in the above chart based on the condition of the wall and severity of wall displacement.
3. Because variations in building design and construction materials are common, PowerBrace™ applications should be reviewed by a qualified professional.
4. Torque applied to the adjustment bolt at the top bracket should not exceed 45 ft-lb.

Appendix 4C

CarbonArmor® Wall Reinforcing System

CarbonArmor® Wall Reinforcing System

Technical Specifications of Carbon Composite

Tensile Strength:
Design Value = 121.7 ksi, Test method in accordance with ASTM D3039

Modulus of Elasticity:
Design Value = 10,401.7 ksi, Test method in accordance with ASTM D3039

Effective Ply Thickness:
0.062 inches

Strap Width:
7 inches

CarbonArmor® Wall Reinforcing System

Technical Specifications of Saturant Epoxy

Tensile Strength:
9-10 ksi

Tensile Modulus:
441.7 ksi

Elongation at Break Percentage:
4.99%

Flexural Strength:
417.1 ksi

Note: Material Safety Data Sheets (MSDS) for the CarbonArmor™ carbon fabric, saturating resin and saturating hardener are available upon request.

Rendering of the CarbonArmor® system installed within a basement

Spacing Recommendations:
CarbonArmor® spacing tables have been determined with consideration for the recommendations of ACI 440, design calculations, and current industry state of practice. The carbon straps are to be installed per the installation instructions and must be the full height of the wall.

		8" Masonry Block Wall Height (ft.)					
		5	6	7	8	9	10
Unbalanced fill Height (ft.)	5	5.0	5.0	5.0	5.0	5.0	5.0
	6		5.0	5.0	5.0	5.0	5.0
	7			5.0	4.5	4.0	3.5
	8				3.5	3.0	2.5
	9					2.5	2.5
	10						2.5

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10" Masonry Block Wall Height (ft.)							
		5	6	7	8	9	10
Unbalanced fill Height (ft.)	5	5.0	5.0	5.0	5.0	5.0	5.0
	6		5.0	5.0	5.0	5.0	5.0
	7			5.0	5.0	5.0	4.5
	8				4.0	3.5	3.5
	9					3.0	2.5
	10						2.5

8" Concrete Wall Height (ft.)							
		5	6	7	8	9	10
Unbalanced fill Height (ft.)	5	5.0	5.0	5.0	5.0	5.0	5.0
	6		5.0	5.0	5.0	5.0	5.0
	7			5.0	5.0	5.0	5.0
	8				5.0	5.0	5.0
	9					5.0	4.5
	10						3.5

10" Concrete Wall Height (ft.)							
		5	6	7	8	9	10
Unbalanced fill Height (ft.)	5	5.0	5.0	5.0	5.0	5.0	5.0
	6		5.0	5.0	5.0	5.0	5.0
	7			5.0	5.0	5.0	5.0
	8				5.0	5.0	5.0
	9					5.0	5.0
	10						4.5

1. Maximum recommended spacing from corners is 3 feet but should also not exceed the spacing of the interior straps.
2. Spacing could be less than listed in the above chart based on the condition of the wall and the severity of wall displacement.

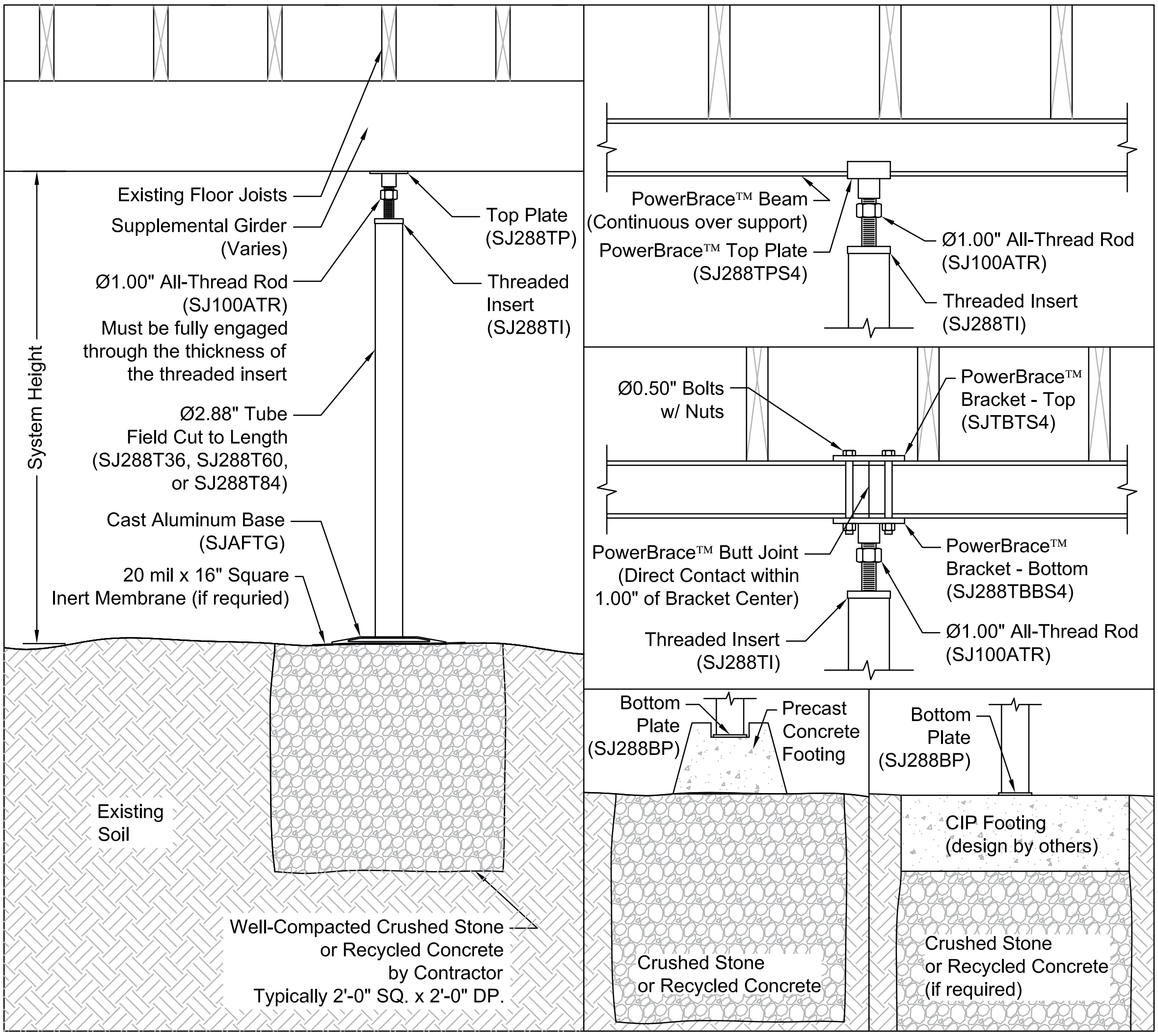
Appendix 4D

SmartJack® Systems

Model 288 SmartJack® System

Typical Crawlspace Detail

Detail Variations



Technical Specifications

SJ288TP (Top Plate):

¼” x 4.00” x 4.00” ASTM A 36 plate with Ø1.500” x 0.250” wall x 1.375” long ASTM A53 Grade B confining ring.
Allowable compression with sawn lumber girder = 4.2 k
Allowable compression with structural steel girder = (will not govern)

SJ288TPS4 (Top Plate):

For use in combination with PowerBrace™ beams
³⁄₁₆” x 3.00” x 6.00” ASTM A36 bent plate with Ø1.500” x 0.250” wall x 1.375” long ASTM A53 Grade B confining ring.
Allowable compression = (will not govern)

SJ288TBBS4, SJTBTS4 (PowerBrace™ Bracket):

For use in combination with PowerBrace™ beams.
(2) - ⅜” x 4.50” x 5.00” ASTM A36 plates, Ø1.500” x 0.250” wall x 1.375” long ASTM A53 Grade B confining ring, (4) - Ø ½” x 5.50” bolts with nuts.
Allowable compression = 8.3 k

SJ100ATR (All-Thread Rod):

Ø1” x 8” long ASTM A 108 Grade 1018 all-thread rod with welded heavy hex nut. Yield strength = 70 ksi (min.), tensile strength = 85 ksi (min.).
Allowable compression for system heights up to 9 feet = 11.0 k

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SJ288TI (Threaded Insert):

Ø3.00” OD x 1.00” thick machined and tapped insert, ASTM A108 Grade 1018. Yield strength = 56 ksi (min.), tensile strength = 90 ksi (min.).
Allowable compression for system heights up to 9 feet = (will not govern)

SJ288T36, SJ288T60, SJ288T84 (Tube):

Ø2.875” OD x 0.165” wall x 36”, 60”, or 84” long field cut to length, ASTM A500 Grade C triple-coated in-line galvanized. Yield strength = 50 ksi (min.), tensile strength = 55 ksi (min.).
Allowable compression for system heights up to 9 feet = 11.0 k

SJAFT6 (Cast Aluminum Base):

0.85” x 12.00” square AISI/AA 356.0-T6 cast aluminum. Yield strength = 20.0 ksi (min), tensile strength = 30.0 ksi (min.).
Allowable compression = 8.0 k

SJ288BP (Bottom Plate):

¼” x 3.50” square ASTM A36 plate and HSS 2.00 x 2.00 x 0.25 tube x 0.75” long ASTM A5 on Grade B. Allowable compression (when used against concrete with minimum f’c = 2,500 psi) = 10.7 k

Precast Concrete Footing:

Typical base dimensions approx. 12” x 12”. Exact dimensions and capacities vary with manufacturer. Item purchased at local building supply center.

Allowable Capacity

The allowable system compression capacity of the assembled 288 SmartJack® system is limited to the least value of the component capacities used in the system assembly. The component capacities are listed in the technical specifications section.

The allowable load applied to the SmartJack® system may be limited by the bearing capacity of the existing soil. The well-compacted crushed stone or recycled concrete base is a proven method to increase support for the higher bearing pressure condition immediately below the cast aluminum base or the concrete footing, and then to absorb and distribute lower pressures to the existing soils. Should settlement of the SmartJack® system occur, adjustments are made easily by extending the all-thread rod.

The SmartJack® is designed to support axial compression loads only. The SmartJack® should not be used in applications where the system is intended to resist lateral loads.

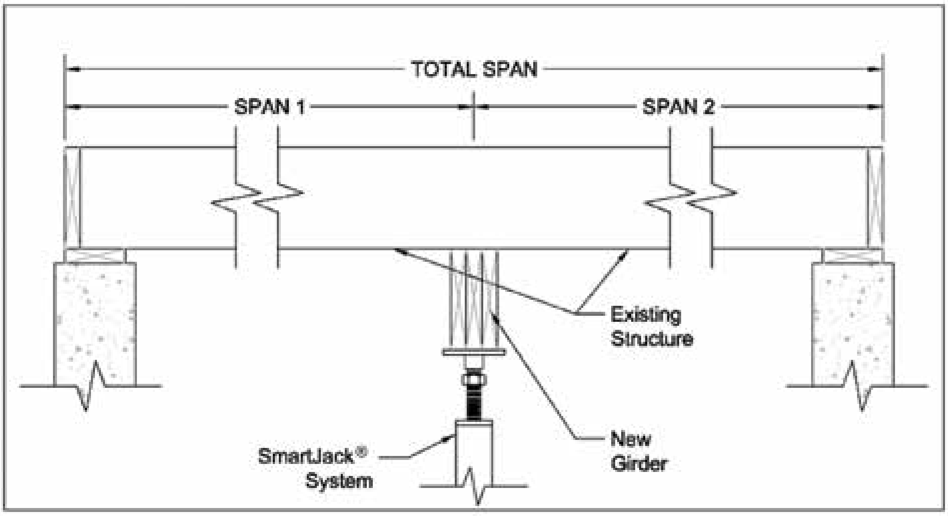
Corrosion Protection

The tube steel used for the SmartJack® is manufactured with a triple-layer, in-line galvanized coating. This coating process consists of: (1) a uniform hot-dip zinc galvanizing layer; (2) an intermediate conversion coating to inhibit the formation of white rust and enhance corrosion resistance; and (3) a clear organic top coating to further enhance appearance and durability. The inside of the pier tube also has a zinc-rich coating.

The cast aluminum base is far less susceptible to corrosion than the steel components that surround it. There are, however, chlorides in some concrete that can be reactive with aluminum. It is recommended that a 20 mil x 16” square inert membrane be placed under the aluminum base to seperate it from recycled or poured concrete. Crushed limestone or other crushed stone alternatives do not have similar considerations.

The remaining steel components of the SmartJack® system come standard as electrozinc plated per ASTM B633.

Design Example



Step 1 - Determine the load which will be supported by the girder in pounds per linear foot:

Girder Load (plf) = (Span 1 (ft) + Span 2 (ft)) x Floor Load (psf) ÷ 2

> **Note 1:** Typical residential wood-framed construction may have an approximate floor load (dead load + live load) = 55 psf

> **Note 2:** This equation assumes a floor system which does not support any load bearing walls or columns.

Step 2 - Determine the load on the SmartJacks® by multiplying the calculated Girder Load (plf) by the spacing of the SmartJacks®:

SmartJack Load (lbs) = Girder Load (plf) x SmartJack® Spacing (ft)

Step 3 - Verify that the calculated SmartJack® load is less than the allowable capacity provided by the various system components as well as the well-compacted crushed stone base and the bearing soils.

> **Note 3:** Without a detailed soil investigation, typical installations should assume no more than 1,500 psf allowable soil bearing pressure. This would equate to an allowable soil capacity of 6,000 lbs for a 2'x2' poured concrete footing or a 2' cube of well-compacted crushed stone. Extremely soft soils may prohibit the use of a crushed stone base or require that a larger poured concrete footing be utilized.

Step 4 - Size the new girder by entering the following table with both the SmartJack® Spacing (ft) and the calculated Girder Load (plf). Choose a girder that has an Allowable Load (plf) greater than the calculated Girder Load (plf).

	Girder Size	Girder Allowable Load (plf)(1,2,3)					
		4 ft	**5 ft**	**6 ft**	**7 ft**	**8 ft**	**SmartJack® Spacing**
Sawn Lumber(4)	(3) – 2 x 8	1,170	750	520	380	290	
	(3) – 2 x 10	1,760	1,120	780	570	440	
	(3) – 2 x 12	2,360	1,510	1,050	770	590	
	(1) – 4 x 6	850	550	380	280	210	
	(1) – 6 x 6	1,030	660	460	330	250	
Engineered Lumber(5)	3.5 x 5.5	1,250	740	420	270	180	
Structural Steel(6)	S4 x 7.7 (PowerBrace™ Beam)	3,780	2,070	1,200	750	500	
	W4 x 13	6,820	3,850	2,240	1,400	940	
	HSS 4 x 4 x ¼	5,190	2,650	1,530	960	640	
	HSS 4 x 2 x ¼ (Lying Flat in Plank Orientation)	990	500	290	180	120	

(1) This table makes no evaluation of the components of the existing structure.

(2) Allowable loads in this table assume the girder is sufficiently restrained against lateral torsional buckling at an interval equal to or less than the SmartJack® spacing.

(3) The new girder may be cantilevered over the end support by a distance of 30 inches or by a distance of approximately 40% of the adjacent SmartJack® spacing, whichever is less.

(4) Sawn lumber is assumed to be Douglas Fir Larch - No. 2 or better.

(5) Engineered lumber is assumed to be iLevel 1.3E TimberStrand LSL or equivalent.

(6) Structural steel is assumed to be ASTM A572 Grade 50 or equivalent for wide flange shapes, and ASTM A500 Grade B or equivalent for HSS tube shapes.

Step 5 - If the required girder size is undesirable, adjust spacing of the SmartJacks® and return to Step 2.

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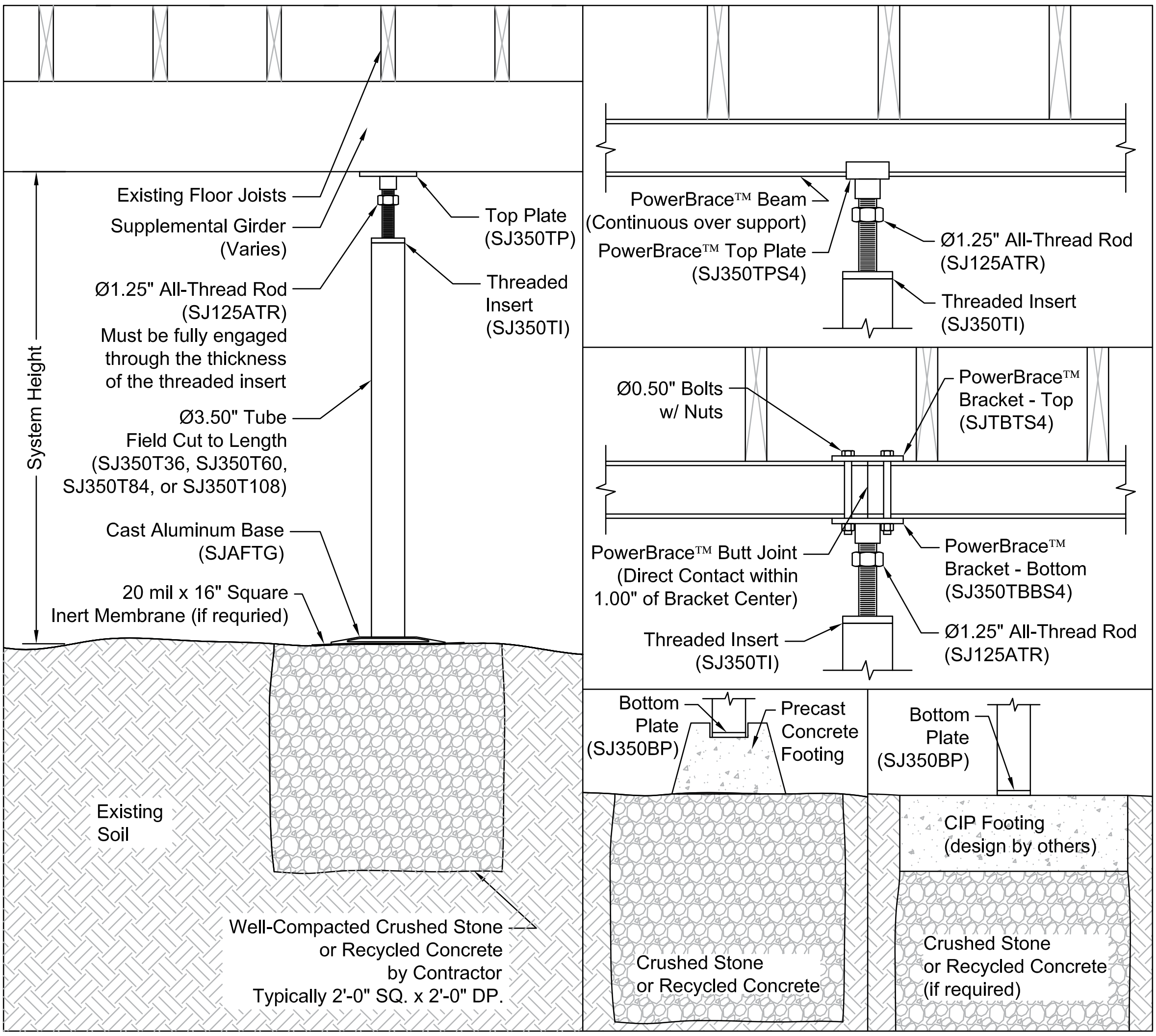
Model 350 SmartJack® System

Typical Crawlspace Detail

Detail Variations



Technical Specifications

SJ350TP (Top Plate):

½" x 5.00" x 6.00" ASTM A36 plate with
Ø1.750" x 0.250" wall x 1.375" long
ASTM A53 Grade B confining ring.
Allowable compression with sawn lumber girder = 10.8 k
Allowable compression with structural steel girder = (will not govern)

SJ350TPS4 (Top Plate):

For use in combination with PowerBrace™ beams
³⁄₁₆" x 3.00" x 6.00" ASTM A36 bent plate with
Ø1.750" x 0.250" wall x 1.375" long
ASTM A53 Grade B confining ring.
Allowable compression = (will not govern)

SJ350TBBS4, SJTBTS4 (PowerBrace™ Bracket):

For use in combination with PowerBrace™ beams.
(2) - ⅜" x 4.50" x 5.00" ASTM A36 plates,
Ø1.750" x 0.250" wall x 1.375" long
ASTM A53 Grade B confining ring,
(4) - Ø ½" x 5.50" bolts with nuts.
Allowable compression = 8.3 k

SJ125ATR (All-Thread Rod):

Ø1-¼" diameter x 10" long ASTM A108
Grade 1018 all-thread rod with welded heavy hex nut.
Yield strength = 70 ksi (min.), tensile strength = 85 ksi (min.).
Allowable compression for system heights up to 9 feet = 20.0 k

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SJ350TI (Threaded Insert):
Ø3.500” x 1.00” thick machined and tapped insert, ASTM A108 Grade 1018.
Yield strength = 56 ksi (min.), tensile strength = 90 ksi (min.).
Allowable compression for system heights up to 9 feet = (will not govern)

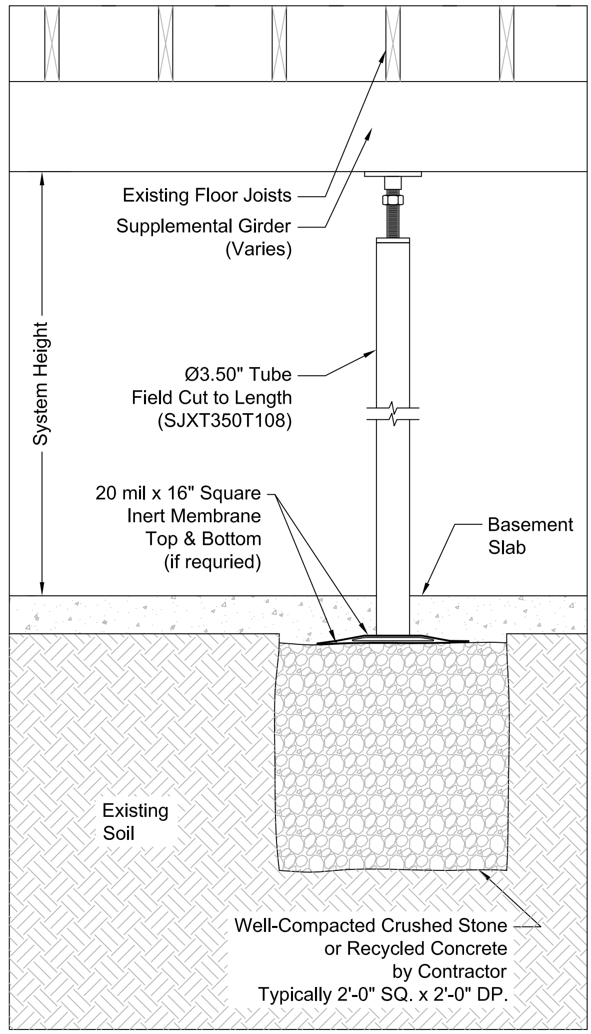
SJ350T36, SJ350T60, SJ350T84, SJ350T108, SJXT350T108 (Tube):
Ø3.500” x 0.165” wall x 36”, 60”, 84”, or 108” long, field cut to length, ASTM A500 Grade C triple-coated in-line galvanized.
Yield strength = 50 ksi (min.), tensile strength = 55 ksi (min.).
Allowable compression for system heights up to 9 feet = 20.0 k

SJAFTG (Cast Aluminum Base):
0.85” x 12.00” square AISI/AA 356.0-T6 cast aluminum. Yield strength = 20.0 ksi (min), tensile strength = 30.0 ksi (min.).
Allowable compression = 8.0 k

SJ350BP (Bottom Plate):
½” x 3.50” square ASTM A36 plate and Ø3.125” x 0.187” wall x 0.75” long ASTM A53 Grade B confining ring.
Allowable compression (when used against concrete with minimum f’c = 2,500 psi) =10.7 k

Precast Concrete Footing
Typical base dimensions approx. 12” x 12”. Exact dimensions and capacities vary with manufacturer. Item purchased at local building supply center.

Typical Basement Detail



Note: Threaded insert is welded to tube for basement applications (SJXT350T108). Remaining details are consistent with those shown for crawlspace applications.

Allowable Capacity

The allowable system compression capacity of the assembled SmartJack® system is limited to the least value of the component capacities used in the system assembly. The component capacities are listed in the technical specifications section.

The allowable load applied to the SmartJack® system may be limited by the bearing capacity of the existing soil. The well-compacted crushed stone or recycled concrete base is a proven method to increase support for the higher bearing pressure condition immediately below the cast aluminum base or the concrete footing, and then to absorb and distribute lower pressures to the existing soils. Should settlement of the SmartJack® system occur, adjustments are made easily by extending the all-thread rod.

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The SmartJack® is designed to support axial compression loads only. The SmartJack® should not be used in applications where the system is intended to resist lateral loads.

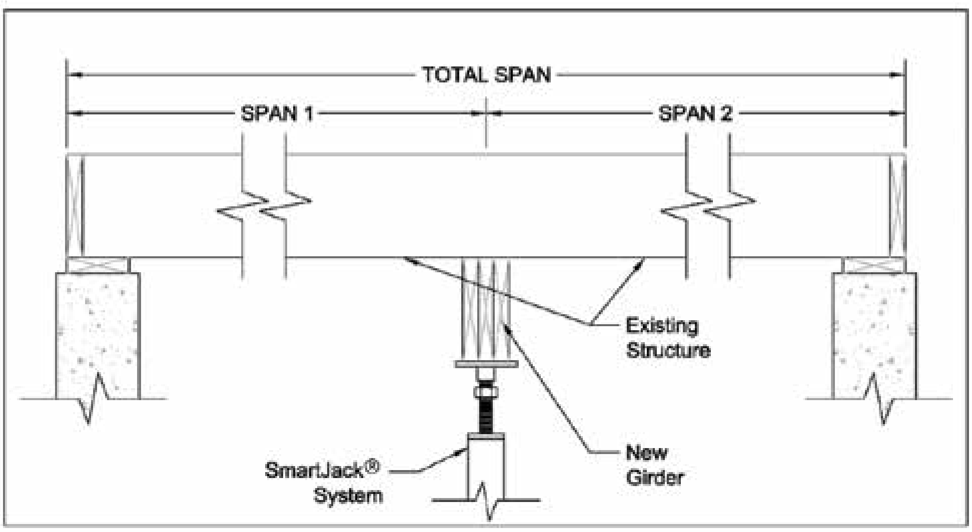
Corrosion Protection

The tube steel used for the SmartJack® is manufactured with a triple-layer, in-line galvanized coating. This coating process consists of: (1) a uniform hot-dip zinc galvanizing layer; (2) an intermediate conversion coating to inhibit the formation of white rust and enhance corrosion resistance; and (3) a clear organic top coating to further enhance appearance and durability. The inside of the pier tube also has a zinc-rich coating.

The cast aluminum base is far less susceptible to corrosion than the steel components that surround it. There are, however, chlorides in some concrete that can be reactive with aluminum. It is recommended that a 20 mil x 16” square inert membrane be placed under or around the aluminum base to seperate it from recycled or poured concrete. Crushed limestone or other crushed stone alternatives do not have similar considerations.

The remaining steel components of the SmartJack® system come standard as electrozinc plated per ASTM B633.

Design Example



Step 1 - Determine the load which will be supported by the girder in pounds per linear foot:

Girder Load (plf) = (Span 1 (ft) + Span 2 (ft)) x Floor Load (psf) ÷ 2

> **Note 1:** Typical residential wood-framed construction may have an approximate floor load (dead load + live load) = 55 psf
>
> **Note 2:** This equation assumes a floor system which does not support any load bearing walls or columns.

Step 2 - Determine the load on the SmartJacks® by multiplying the calculated Girder Load (plf) by the spacing of the SmartJacks:

SmartJack Load (lbs) = Girder Load (plf) x SmartJack® Spacing (ft)

Step 3 - Verify that the calculated SmartJack® load is less than the allowable capacity provided by the various system components as well as the well-compacted crushed stone base and the bearing soils.

> **Note 3:** Without a detailed soil investigation, typical installations should assume no more than 1,500 psf allowable soil bearing pressure. This would equate to an allowable soil capacity of 6,000 lbs for a 2'x2' poured concrete footing or a 2' cube of well-compacted crushed stone. Extremely soft soils may prohibit the use of a crushed stone base or require that a larger poured concrete footing be utilized.

Step 4 - Size the new girder by entering the following table with both the SmartJack® Spacing (ft) and the calculated Girder Load (plf). Choose a girder that has an Allowable Load (plf) greater than the calculated Girder Load (plf).

	Girder Size	Girder Allowable Load (plf)(1,2,3)				
	SmartJack® Spacing	**4 ft**	**5 ft**	**6 ft**	**7 ft**	**8 ft**
Sawn Lumber(4)	(3) – 2 x 8	1,170	750	520	380	290
	(3) – 2 x 10	1,760	1,120	780	570	440
	(3) – 2 x 12	2,360	1,510	1,050	770	590
	(1) – 4 x 6	850	550	380	280	210
	(1) – 6 x 6	1,030	660	460	330	250
Engineered Lumber(5)	3.5 x 5.5	1,250	740	420	270	180
Structural Steel(6)	S4 x 7.7 (PowerBrace™ Beam)	3,780	2,070	1,200	750	500
	W4 x 13	6,820	3,850	2,240	1,400	940
	HSS 4 x 4 x ¼	5,190	2,650	1,530	960	640
	HSS 4 x 2 x ¼ (Lying Flat in Plank Orientation)	990	500	290	180	120

(1) This table makes no evaluation of the components of the existing structure.

(2) Allowable loads in this table assume the girder is sufficiently restrained against lateral torsional buckling at an interval equal to or less than the SmartJack® spacing.

(3) The new girder may be cantilevered over the end support by a distance of 30 inches or by a distance of approximately 40% of the adjacent SmartJack® spacing, whichever is less.

(4) Sawn lumber is assumed to be Douglas Fir Larch - No. 2 or better.

(5) Engineered lumber is assumed to be iLevel 1.3E TimberStrand LSL or equivalent.

(6) Structural steel is assumed to be ASTM A572 Grade 50 or equivalent for wide flange shapes, and ASTM A500 Grade B or equivalent for HSS tube shapes.

Step 5 - If the required girder size is undesirable, adjust spacing of the SmartJacks® and return to Step 2.

CHAPTER 5

POLYLEVEL® POLYURETHANE FOAM AND RESIN

5.1 Summary Description

The PolyLEVEL product line consists of polyurethane foams and resins for use in a wide range of geotechnical and structural applications. The more commonly used products are two-part urethanes that expand into rigid foam to fill voids, stabilize concrete, and lift concrete. The product is injected at the interface between the concrete slab and the subgrade soils *(Figure 5.1.a)*. Variations in the formulas of the two-part foams allow for uses in even more specific applications, as mentioned in Section 5.2. Outside of the general offerings of two-part products, single-part, non-expanding PolyLEVEL resin is injected deep into loose soil as a binder material, thereby stabilizing the soil from further consolidation/densification, or allowing vertical excavations to be made.

Polyurethane is not a new material. The chemistry of polyurethanes was discovered and patented in 1937 and has been used extensively for three quarters of a century in a variety of products and materials. Polyurethanes are best known to the public in the form of flexible foams; i.e., upholstery cushions, mattresses, caulking, weatherstripping, vehicle door and dash moldings, tires for toddler bikes and wheelborrows, etc. Spray foam insulation and foam filler within shells of watercraft are examples most similar to the line of two-part PolyLEVEL products.

Polyurethane foam is created by the reaction of a diisocyanate (the "A" or "Iso" side) with a polyol (the "R" or "Resin" side) to form a urethane linkage. Diisocynates are organic compounds that are specifically manufactured to react with polyols. The A side varies little between polyurethanes of similar product lines or use groups. Polyols are simply alcohols with multiple hydroxyl groups. The polyols and other additives within the R side may vary significantly between products to control characteristics such as color, density and speed of reaction.

Figure 5.1.a Rendering of PolyLEVEL foam injection beneath concrete pavement

5.2 Applications

PolyLEVEL expanding foams are most often used beneath exterior concrete sidewalk and pavement sections for stabilization or re-leveling, but are also utilized for interior work and other specialty applications. PolyLEVEL is often considered as an alternative to mudjacking or removal and replacement.

In **residential** applications, PolyLEVEL foam is used to stabilize, lift and re-level sidewalks *(Figure 5.2.a)*, patios, driveways *(Figures 5.2.b1 and 5.2.b2)*, garages slabs, and slab-on-grade concrete floors. Concrete decks surrounding swimming pools also often settle over time due to poor compaction of fill soils, soil consolidation/densification, and erosion. These repairs can often be completed in a matter of hours with immediate results, less mess, and no "wait time" for typical use of the areas. When the work is done, all the concrete sections will still match and not take on the appearance of a checker board, as is the case with typical removal and replacement.

Figure 5.2.a PolyLEVEL foam injected to re-level faulted sidewalk sections

Figure 5.2.b1 Before: Driveway settled up to 4 inches relative to garage slab

Figure 5.2.b2 After: Driveway lifted back to near original elevation

Commercial/industrial applications include similar types of projects as residential, but also include areas and facilities with heavier use pavements. More common PolyLEVEL applications include re-support of concrete pavement at and around loading docks, and concrete floor slabs within warehouses *(Figure 5.2.c)*, plants and manufacturing facilities. Heavy wheel loads from semi-tractor trailers and from fork trucks within buildings can overstress subgrade soils and lead to pavement/slab cracking, settlement and faulted sections.

Figure 5.2.c PolyLEVEL stabilizes warehouse floor slab

Municipal applications of PolyLEVEL foam include void-filling and lifting of roadway sections, runways and taxi-ways at airports, and floor slabs within schools and other city buildings. Dips and sags along roads can be lifted and releveled *(Figures 5.2.d1 and 5.2.d2)*. Approach slabs to bridges often settle relative to the bridge abutments due to poor compaction of fill soils, consolidation of deep fill within the embankment, or consolidation of the native soils beneath the embankment. The bumps before and after bridges can be removed or softened with the injection of PolyLEVEL. PolyLEVEL is a great option for airports where sections of runways or taxi-ways cannot be shut down for long periods. Maintenance plans including ground penetrating radar can be implemented to locate voids beneath the pavement and fill them with PolyLEVEL before there is a much larger issue. Terrazzo floors within schools and other city buildings create an excellent wearing surface, but are expensive to repair or replace. PolyLEVEL is injected through small ⅝-inch diameter holes drilled in the slabs.

Figure 5.2.d1 PolyLEVEL injection while maintaining an open lane for traffic

Figure 5.2.d2 Monitoring pavement lift with string lines

Specialty applications of PolyLEVEL foam include void-filling after slabs and foundations are lifted with the piering systems described in Chapters 2 and 3, and pipeline/utility projects as a ditch block or trench break material. Ditch blocks prevent water from eroding soil as it migrates through the backfill and along pipes from higher to lower elevations of sloped terrain *(Figure 5.2.e)*. PolyLEVEL is often a faster and more economical alternative to concrete collars and even sand bags. PolyLEVEL Trench-Breaker is a highly specialized product that dissipates heat quickly during application so as not to create a fire hazard as it is placed in wide, thick layers. Another specialty product is the single-part, non-expanding PolyLEVEL resin. As mentioned in Section 5.1, the singe-part resin is injected deep into loose soil as a binder to stabilize the soil from further consolidation/densification, or to allow vertical excavations *(Figure 5.2.f)*.

Figure 5.2.e Placement of polyurethane foam trench breaks

Figure 5.2.f PolyLEVEL resin stabilizes loose sand to prevent undermining of slab during column repair

5.3 Benefits

There are many benefits and advantages to using PolyLEVEL foams over other products, repair options, and stabilization methods.

- **Lightweight** – PolyLEVEL polyurethane foams weigh only 4 to 7 pounds per cubic foot (pcf) when installed and confined beneath slabs and pavements. Mudjacking material and compacted fill soil can weigh 100 to 140 pcf. Very little load is therefore added to the underlying, supporting soils.
- **High Capacity** – Lifting action is the result of the expansion of the polymer, not the pressure at which PolyLEVEL is injected beneath the slab. This generally allows for lifting of higher loads than mudjacking. Mudjacking relies on the hydraulic pressure being contained beneath the slab. When mudjacking slurry "blows out", additional lift is not possible.
- **Accurate Lift** – Installer technique and product knowledge allows for precise lifting *(Figures 5.3.a1 and 5.3.a2)*. Each product has a calculated reaction time.

Figure 5.3.a1 Commuter train track settled up to 2 inches at intersection

Figure 5.3.a2 Lifting precast concrete slabs and embedded rails while monitoring with survey equipment

- **Waterproof –** PolyLEVEL foam will not soften, "dissolve" in water, or wash out. Material can be used to under-seal slabs as well as stop a variety of infrastructure leaks. PolyLEVEL slab support products will not absorb water and are therefore unaffected by freeze-thaw cycles.
- **Non-Invasive** – PolyLEVEL installation equipment allows placement in tight and limited access areas. There is less mess than mudjacking and smaller ⅝-inch holes required for installation. Mudjacking typically requires 2 to 3-inch holes. PolyLEVEL installs quickly and most small projects are completed in one day.
- **Quick Cure Time** – PolyLEVEL foam cures quickly to allow even heavy traffic just 30 minutes after installation.
- **Environmentally Friendly** – The two parts (A and R sides) react fully with each other and create an inert product safe for the environment. Use of PolyLEVEL to stabilize and re-level slabs extends pavement life so less material is removed and hauled off to landfills.

5.4 Product Use Considerations

Special considerations for use of PolyLEVEL expanding foam products include:

- PolyLEVEL expanding foam is intended for use beneath rigid concrete slabs and pavements with some spanning capability. The products should not be used beneath flexible pavements; e.g., asphalt, brick pavers, cobble stone, or highly deteriorated and broken concrete.
- Mixing of the A and R sides creates an exothermic reaction. The heat that is generated generally increases with the thickness of material placed. When a void is deeper than about 6 inches, the material should be placed in thinner lifts to avoid charring or ignition. The PolyLEVEL Trench-Breaker material is designed specifically for placement in thick, successive lifts.
- The chemical reaction generates off-gasses. Fortunately, most applications are outside or within large open and/or well-ventilated areas. The gasses are mostly contained under the slab and dissipate slowly causing no adverse effect. However, in poorly ventilated areas, the gasses can displace or reduce breathing air. Long term exposure can cause an allergic-like sensitivity to the product. It is always best to arrange proper ventilation and to wear a half mask with Organic Vapor/Acid Gasses respirator cartridges when working inside.
- Care should be taken by the installer not to over-lift the slab or pavement sections. Should an over-lift situation occur, slight adjustments may be possible in the elevations of adjacent sections. Otherwise, sections may have to be ground down or removed and replaced.
- Re-leveling slabs and pavements by injecting PolyLEVEL at the interface of the concrete and subgrade soils will not address deep-seated problems such as consolidation of fill or native soils, erosion, or sinkhole development. Experience has shown that deep consolidation of fill or native soils is often the cause of dips and sags in roadway sections and settlement of approach slabs. Deep injection of polyurethane in such applications, especially in clay soils, would provide little to no benefit. Rather, a practical and economical option is periodic re-leveling of the slabs as settlement occurs. Deep consolidation settlement generally slows and decreases in magnitude over time, so these periodic adjustments become less frequent (generally years between applications).

5.5 Products

The PolyLEVEL product line includes four (4), two-part expanding foams and one (1) single-part non-expanding resin. Brief general descriptions and product uses are included below. Additional product descriptions and properties are included on the Technical Information sheets in Appendix 5A.

- **PolyLEVEL® 250 (PL 250)** – Nominal 2.5 pcf density in a free-rise state; typical in-place density of 4 pcf confined beneath slabs and pavements. Used mainly in residential and low load applications.
- **PolyLEVEL® 400 (PL 400)** - Nominal 4 pcf density in a free-rise state, typical in-place density of 6.5 pcf confined beneath slabs or pavements. Used in commercial and moderate to high load applications.
- **PolyLEVEL® 400H (PL 400H)** – Similar properties and uses as PL 400. Hydrophobic material used specifically in applications where water is likely present beneath the slabs or pavements.
- **PolyLEVEL® Trench-Breaker** – Used as a ditch block or trench break material due to its low exothermic reaction temperature and quick dissipation of heat.
- **PolyLEVEL® 100SS (PL 100SS)** - Single-part stabilizer for loose soils.

Model specifications for PL 100SS and PL 400 are included in Appendix 5B. The model specification for PL 400 can be easily modified for the other two-part expanding products.

5.6 Equipment

PolyLEVEL is installed with custom-built installation rigs available as truck-mounted *(Figure 5.6.a1)*, trailer units *(Figure 5.6.a2)*, or ATV buggies *(Figure 5.6.a3)*. The basic components of the system include material storage tanks, a generator, an air compressor, pumps, a reactor, and applicator(s)/gun(s). The reactor includes a proportioner, heater, and insulated and heated hoses. This ensures that the two parts are delivered to the applicator at a consistent pressure and temperature.

Figure 5.6.a1
Truck-mounted installation equipment

Figure 5.6.a2
Enclosed trailer unit

Figure 5.6.a3
ATV buggy

5.7 Installation Steps

The following installation steps provide a broad overview of PL 250, PL 400 and PL 400H injection beneath a concrete slab or pavement. Intermediate steps, installation equipment and tools used, and considerations for unusual conditions or applications are not addressed.

Step 1 – ⅝-inch holes are drilled in strategic locations in the slab *(Figure 5.7.a1)*. In general, the holes are spaced 5 feet apart and 3 feet from edges of the slab. Locations and spacing are often modified in the field to achieve the desired result.

Figure 5.7.a1

Step 2 – Injection ports are placed and tightened with a socket and wrench to seal the hole *(Figure 5.7.a2)*.

Figure 5.7.a2

Step 3 – PolyLEVEL is injected to fill voids and allow for lifting *(Figure 5.7.a3)*.

Figure 5.7.a3

Step 4 – The injection ports are removed and the holes are patched with mortar mix, polyurethane sealant or epoxy sealant *(Figure 5.7.a4)*.

Figure 5.7.a4

Appendix 5A

Technical Information

PolyLEVEL® 100-SS
Technical Information

Description

PolyLEVEL® 100-SS is a low viscosity, polyurethane resin designed to penetrate and stabilize loose or sandy soil when medium or high strength is required. It migrates through loose soil and into voids and reacts with moisture in the soil to form a polymer cement/soil matrix of high strength and durability. PolyLEVEL® 100-SS is resistant to most organic solvents, mild acids and bases, and micro-organisms. The polyurethane encapsulates loose soil, fills voids, and forms a solid, water-tight barrier.

Unique Advantages

- Contains no solvents
- Very low viscosity for good penetration
- Fast cure time controlled by catalyst ratio and moisture in the treated soil
- Encapsulates and strengthens loose soil
- Forms a water-tight barrier to stop water migration
- Good resistance to chemicals
- Excellent compressive strength (over 1,000 psi in sand) and thermal stability

Typical Resin Properties		
	PolyLEVEL® 100-SS A Soil Stabilizer	PolyLEVEL® 100-SS R Activator
Viscosity(1)	25 – 50 cps	100 - 200 cps
Unit Weight	10.25 lb/gal	8.5 lb/gal
Shelf Life	6 months	6 months

Typical Reaction Properties	
% Activator	Gel Time(1)
0.5	12-16 min
1.0	8-11 min
2.0	6-7 min

(1) At 77°F

Storage and Cleanup

Activator MUST be agitated (shaken) before use. Flush equipment with acetone before and after use to clean equipment and remove moisture. If work area temperatures are low, heat the product to 55°F - 85°F to improve product performance. Do not use open flame as a heat source. Store chemicals between 55°F - 85°F in a dry atmosphere. Shelf life is 6 months, in original, unopened factory containers, under normal storage conditions of 55° to 85° F. Do not store in direct sunlight. Keep drums tightly closed when not in use. Cured product may be disposed of without restriction. Excess liquid material should be mixed together with sand or other absorbent material and allowed to cure, then disposed of in the normal manner.

Safe Handling of Liquid Components

Use caution in removing caps and bungs from the container. Loosen caps and bungs first to let any built up gas escape before completely removing. Avoid prolonged breathing of vapors. In case of chemical contact with eyes, flush with water for at least 15 minutes and get medical attention. For further information refer to “MDI-Based Polyurethane Foam Systems: Guidelines for Safe Handling and Disposal” publication AX-119 published by the Center for the Polyurethanes Industry 1300 Wilson Blvd, Suite 800, Arlington, VA 22209.

Caution

Polyurethane products manufactured or produced from this liquid system may present a serious fire hazard if improperly used or allowed to remain exposed or unprotected. The character and magnitude of any such hazard will depend on a broad range of factors which are controlled and influenced by the manufacturing and production process, by the mode of application or installation and by the function and usage of the particular product. ***Any flammability rating or reference contained in this or other PolyLEVEL® 100-SS literature is not intended to reflect hazards presented by this or any other material under actual fire conditions. These ratings are used solely to measure and describe the product's response to heat and flame under controlled laboratory conditions.*** Each person, firm or corporation engaged in the manufacture, production, application, installation, or use of any polyurethane product should carefully determine whether there is a potential fire hazard associated with such product in a specific usage, and utilize all appropriate precautionary and safety measures.

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PolyLEVEL® 250
Technical Information

Description

PolyLEVEL® 250 is a two-component, water blown, nominal 2.5 pcf density, polyurethane foam system designed to fill voids and lift concrete slabs.

Typical Resin Properties		
	PolyLEVEL® 250 R	PolyLEVEL® 250 A
Viscosity(1)	850 cps	200 cps
Unit Weight	9.4 lb/gal	10.25 lb/gal
Shelf Life	6 months	6 months

Mix Ratio		
	PolyLEVEL® 250 R	PolyLEVEL® 250 A
By Weight	100 parts	109 parts
By Volume	100 parts	100 parts

Typical Reaction Properties	
Cream Time(1)	7.5 sec
Rise Time(1)	25 sec
Density(2)	2.5 pcf
Typical Compressive Strength(2)	35 psi

Typical Physical Properties(3)	
Typical In-Place Density (ASTM D1622)	4 pcf
Compressive Strength (ASTM D1621)	70 psi
Closed Cell Content	>90%
Water Absorption (ASTM D2842)	<0.02 lb/sq ft
Resistance to Solvents	Excellent
Resistance to Mold and Mildew	Excellent
Maximum Service Temp	200°F

(1) At 77°F

(2) FRC = Free Rise Condition

(3) Average value from laboratory testing and should serve only as a guideline.

Storage and Cleanup

Shelf life is 6 months, in original, unopened factory containers, under normal storage conditions of 55° to 85° F. Do not store in direct sunlight. Keep drums tightly closed when not in use. Cured product may be disposed of without restriction. Excess liquid “A” and “R” material should be mixed together and allowed to cure, then disposed of in the normal manner.

Safe Handling of Liquid Components

Use caution in removing caps and bungs from the container. Loosen caps and bungs first to let any built up gas escape before completely removing. Avoid prolonged breathing of vapors. In case of chemical contact with eyes, flush with water for at least 15 minutes and get medical attention. For further information refer to “MDI-Based Polyurethane Foam Systems: Guidelines for Safe Handling and Disposal” publication AX-119 published by the Center for the Polyurethanes Industry 1300 Wilson Blvd, Suite 800, Arlington, VA 22209.

Caution

Polyurethane products manufactured or produced from this liquid system may present a serious fire hazard if improperly used or allowed to remain exposed or unprotected. The character and magnitude of any such hazard will depend on a broad range of factors which are controlled and influenced by the manufacturing and production process, by the mode of application or installation and by the function and usage of the particular product. ***Any flammability rating or reference contained in this or other PolyLEVEL® 250 literature is not intended to reflect hazards presented by this or any other material under actual fire conditions. These ratings are used solely to measure and***

describe the product's response to heat and flame under controlled laboratory conditions. Each person, firm or corporation engaged in the manufacture, production, application, installation, or use of any polyurethane product should carefully determine whether there is a potential fire hazard associated with such product in a specific usage, and utilize all appropriate precautionary and safety measures.

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PolyLEVEL® 400
Technical Information

Description

PolyLEVEL® 400 is a two-component, water blown, nominal 4 pcf density, polyurethane foam system designed for concrete slab raising, under-slab void fill and cavity fill applications.

Typical Resin Properties		
	PolyLEVEL® 400 R	PolyLEVEL® 400 A
Viscosity(1)	700 cps	200 cps
Unit Weight	9.4 lb/gal	10.25 lb/gal.
Shelf Life	6 months	6 months

Mix Ratio		
	PolyLEVEL® 400 R	PolyLEVEL® 400 A
By Weight	100 parts	109 parts
By Volume	100 parts	100 parts

Typical Reaction Properties	
Cream Time(1)	25 sec
Rise Time(1)	90 sec
Density(2)	4 pcf
Typical Compressive Strength(2)	60 psi

Typical Physical Properties(3)	
Typical In-Place Density (ASTM D1622)	6.5 pcf
Compressive Strength (ASTM D1621)	100 psi
Shear Strength (ASTM C273)	140 psi
Closed Cell Content	>90%
Water Absorption (ASTM D2842)	<0.02 lb/sq ft
Resistance to Solvents	Excellent
Resistance to Mold and Mildew	Excellent
Maximum Service Temp	200°F

(1) At 77°F

(2) FRC = Free Rise Condition

(3) Average value from laboratory testing and should serve only as a guideline.

Storage and Cleanup

Shelf life is 6 months, in original, unopened factory containers, under normal storage conditions of 55° to 85° F. Do not store in direct sunlight. Keep drums tightly closed when not in use. Cured product may be disposed of without restriction. Excess liquid 'A' and 'R' material should be mixed together and allowed to cure, then disposed of in the normal manner.

Safe Handling of Liquid Components

Use caution in removing caps and bungs from the container. Loosen caps and bungs first to let any built up gas escape before completely removing. Avoid prolonged breathing of vapors. In case of chemical contact with eyes, flush with water for at least 15 minutes and get medical attention. For further information refer to "MDI-Based Polyurethane Foam Systems: Guidelines for Safe Handling and Disposal" publication AX-119 published by the Center for the Polyurethanes Industry 1300 Wilson Blvd, Suite 800, Arlington, VA 22209.

Caution

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describe the product's response to heat and flame under controlled laboratory conditions. Each person, firm or corporation engaged in the manufacture, production, application, installation, or use of any polyurethane product should carefully determine whether there is a potential fire hazard associated with such product in a specific usage, and utilize all appropriate precautionary and safety measures.

DISCLAIMER

The information on our data sheets is to assist customers in determining whether our products are suitable for their applications. The customers must satisfy themselves as to the suitability for specific cases. Foundation Supportworks®, Inc. warrants only that the material shall meet its specifications; this warranty is in lieu of all other written or unwritten, expressed or implied warranties and Foundation Supportworks®, Inc. expressly disclaims any warranty of merchantability, fitness for a particular purpose, or freedom from patent infringement. Accordingly, buyer assumes all risks whatsoever as to the use of the material. Buyer's exclusive remedy as to any breach of warranty, negligence or other claim shall be limited to the purchase price of the material. Failure to adhere strictly to any recommended procedures shall relieve Foundation Supportworks®, Inc. of all liability with respect to the material or the use thereof.

PolyLEVEL® 400H
Technical Information

Description

PolyLEVEL® 400H is a two-component hydrophobic, closed cell, water blown, nominal 4 pcf density, polyurethane foam system designed for concrete slab raising, under-slab void fill and cavity fill applications.

Typical Resin Properties		
	PolyLEVEL® 400H R	PolyLEVEL® 400H A
Viscosity(1)	800 cps	200 cps
Unit Weight	9.7 lb/gal	10.25 lb/gal
Shelf Life	6 months	6 months

Mix Ratio		
	PolyLEVEL® 400H R	PolyLEVEL® 400H A
By Weight	100 parts	106 parts
By Volume	100 parts	100 parts

Typical Reaction Properties	
Cream Time(1)	20 sec
Rise Time (1)	95 sec
Density(2)	4 pcf
Typical Compressive Strength(2)	60 psi

Typical Physical Properties(3)	
Typical In-Place Density (ASTM D1622)	6.5 pcf
Compressive Strength (ASTM D1621)	100 psi
Shear Strength (ASTM C273)	140 psi
Closed Cell Content	>90%
Water Absorption (ASTM D2842)	<0.02 lb/sq ft
Resistance to Solvents	Excellent
Resistance to Mold and Mildew	Excellent
Maximum Service Temp	200°F

(1) At 77°F

(2) FRC = Free Rise Condition

(3) Average value obtained from laboratory testing and should serve only as a guideline.

Storage and Cleanup

Shelf life is 6 months, in original, unopened factory containers, under normal storage conditions of 55° to 85° F. Do not store in direct sunlight. Keep drums tightly closed when not in use. Cured product may be disposed of without restriction. Excess liquid 'A' and 'R' material should be mixed together and allowed to cure, then disposed of in the normal manner.

Safe Handling of Liquid Components

Use caution in removing caps and bungs from the container. Loosen caps and bungs first to let any built up gas escape before completely removing. Avoid prolonged breathing of vapors. In case of chemical contact with eyes, flush with water for at least 15 minutes and get medical attention. For further information refer to "MDI-Based Polyurethane Foam Systems: Guidelines for Safe Handling and Disposal" publication AX-119 published by the Center for the Polyurethanes Industry 1300 Wilson Blvd, Suite 800, Arlington, VA 22209.

Caution

Polyurethane products manufactured or produced from this liquid system may present a serious fire hazard if improperly used or allowed to remain exposed or unprotected. The character and magnitude of any such hazard will depend on a broad range of factors which are controlled and influenced by the manufacturing and production process, by the mode of application or installation and by the function and usage of the particular product. ***Any flammability rating or reference contained in this or other PolyLEVEL® 400H literature is not intended to reflect hazards presented by this or any other material under actual fire conditions. These ratings are used solely to measure and***

describe the product's response to heat and flame under controlled laboratory conditions. Each person, firm or corporation engaged in the manufacture, production, application, installation, or use of any polyurethane product should carefully determine whether there is a potential fire hazard associated with such product in a specific usage, and utilize all appropriate precautionary and safety measures.

DISCLAIMER

The information on our data sheets is to assist customers in determining whether our products are suitable for their applications. The customers must satisfy themselves as to the suitability for specific cases. Foundation Supportworks®, Inc. warrants only that the material shall meet its specifications; this warranty is in lieu of all other written or unwritten, expressed or implied warranties and Foundation Supportworks®, Inc. expressly disclaims any warranty of merchantability, fitness for a particular purpose, or freedom from patent infringement. Accordingly, buyer assumes all risks whatsoever as to the use of the material. Buyer's exclusive remedy as to any breach of warranty, negligence or other claim shall be limited to the purchase price of the material. Failure to adhere strictly to any recommended procedures shall relieve Foundation Supportworks®, Inc. of all liability with respect to the material or the use thereof.

PolyLEVEL® Trench-Breaker
Technical Information

Description

PolyLEVEL® Trench-Breaker is a two-component, water blown, nominal 2.2 pcf density, polyurethane foam system designed for exterior applications around utilities and pipelines.

PolyLEVEL® Trench-Breaker is applied as a liquid mixture that reacts almost immediately to expand approximately 35 times in volume in 15 to 20 seconds. PolyLEVEL® Trench-Breaker exhibits low exothermic reaction temperature and therefore can be placed in continuous, successive lifts well beyond 2 inches in thickness without danger of charring or ignition.

Typical Resin Properties		
	Trench-Breaker R	Trench-Breaker A
Viscosity(1)	800 cps	225 cps
Unit Weight	9.3 lb/gal	10.25 lb/gal
Shelf Life	6 months	6 months

Mix Ratio		
	Trench-Breaker R	Trench-Breaker A
By Weight	100 parts	110 parts
By Volume	100 parts	100 parts

Typical Reaction Physical Properties	
Cream Time(1)	4.5 sec
Rise Time(1)	17 sec
Typical In-Place Density	2.2 pcf
Tensile Strength (ASTM D1623)(2)	71 psi
Resistance to Mold and Mildew	Excellent

(1) At 77°F

(2) Average value from laboratory testing and should serve only as a guideline.

Storage and Cleanup

Shelf life is 6 months, in original, unopened factory containers, under normal storage conditions of 55° to 85° F. Do not store in direct sunlight. Keep drums tightly closed when not in use. Cured product may be disposed of without restriction. Excess liquid “A” and “R” material should be mixed together and allowed to cure, then disposed of in the normal manner.

Safe Handling of Liquid Components

Use caution in removing caps and bungs from the container. Loosen caps and bungs first to let any built up gas escape before completely removing. Avoid prolonged breathing of vapors. In case of chemical contact with eyes, flush with water for at least 15 minutes and get medical attention. For further information refer to “MDI-Based Polyurethane Foam Systems: Guidelines for Safe Handling and Disposal” publication AX-119 published by the Center for the Polyurethanes Industry 1300 Wilson Blvd, Suite 800, Arlington, VA 22209.

Caution

Polyurethane products manufactured or produced from this liquid system may present a serious fire hazard if improperly used or allowed to remain exposed or unprotected. The character and magnitude of any such hazard will depend on a broad range of factors which are controlled and influenced by the manufacturing and production process, by the mode of application or installation and by the function and usage of the particular product. ***Any flammability rating or reference contained in this or other PolyLEVEL® Trench-Breaker literature is not intended to reflect hazards presented by this or any other material under actual fire conditions. These ratings are used***

solely to measure and describe the product's response to heat and flame under controlled laboratory conditions. Each person, firm or corporation engaged in the manufacture, production, application, installation, or use of any polyurethane product should carefully determine whether there is a potential fire hazard associated with such product in a specific usage, and utilize all appropriate precautionary and safety measures.

DISCLAIMER

The information on our data sheets is to assist customers in determining whether our products are suitable for their applications. The customers must satisfy themselves as to the suitability for specific cases. Foundation Supportworks®, Inc. warrants only that the material shall meet its specifications; this warranty is in lieu of all other written or unwritten, expressed or implied warranties and Foundation Supportworks®, Inc. expressly disclaims any warranty of merchantability, fitness for a particular purpose, or freedom from patent infringement. Accordingly, buyer assumes all risks whatsoever as to the use of the material. Buyer's exclusive remedy as to any breach of warranty, negligence or other claim shall be limited to the purchase price of the material. Failure to adhere strictly to any recommended procedures shall relieve Foundation Supportworks®, Inc. of all liability with respect to the material or the use thereof.

Appendix 5B

Model Specifications

MODEL SPECIFICATION FOR POLYURETHANE VOID FILLING OR SOIL STABILIZATION APPLICATIONS

1 SCOPE

1.1 The work consists of designing, furnishing, and installing single-part, high-density polyurethane material according to the project plans and this specification.

1.2 The parties and contract terms referred to in this specification are as follows:

1.2.1 The Owner is the person or entity that owns the facility or will own the facility once it is completed. The Owner may have contractual agreements with, and be represented by, other parties such as engineers, architects, or contractors that preform services under the direction of the Owner. Where Owner is used in the specification, it refers to the Owner or the Owner's contracted representatives separate from the Installing Contractor.

1.2.2 The Installing Contractor installs the polyurethane material and possibly performs other tasks associated with the project.

1.2.3 The Plans refer to the contract documents; including but not limited to the drawings and specifications for the project.

1.3 The work may include void filling, stabilizing loose or sandy soil, or undersealing pavement and faulted joints.

1.4 The Owner will provide suitable access to the construction site for the Installing Contractor's personnel and equipment.

1.5 Unless otherwise noted, the Installing Contractor shall provide all labor, tools, equipment, and material necessary to accomplish the work.

1.6 Unless specifically noted otherwise in the contract documents, the Owner will remove and replace structures, utilities, or other surficial improvements in the work area as necessary to facilitate the work.

1.7 The Owner will be responsible for overall construction oversight to preclude the development of unsafe conditions.

1.8 The Owner will be responsible for any soil density testing subsequent to the polyurethane injection, unless otherwise noted.

1.9 The work does not include and post-installation monitoring unless specifically noted otherwise in the contract documents.

2 APPROVED POLYURETHANE MANUFACTURERS

2.1 Foundation Supportworks®, Inc., 12330 Cary Circle, Omaha, NE 68128; Phone: (800) 281-8545; Fax (402) 393-4002.

2.2 Due to the special requirements for design and manufacturing of polyurethane products, the system shall be obtained from Foundation Supportworks®, Inc., or other qualified manufacturer with an approved equivalent product. A request to substitute any other manufactured polyurethane product must be submitted to the Owner for review not less than seven (7) calendar days prior to the bid date. The request must include:

2.2.1 Documentation of at least five years of production experience manufacturing polyurethane products for similar applications,

2.2.2 Documentation that the manufacturer's polyurethane products have been used successfully on at least five similar projects within the last three (3) years, and/or,

2.2.3 Product acceptance by the local building code official(s) having jurisdiction over the project.

3 ACCEPTABLE PRODUCTS

3.1 PolyLEVEL® 100-SS Single-part Polyurethane Foam manufactured in accordance with the requirements of Section 4 of this specification.

4 MATERIALS

4.1 PolyLEVEL® 100-SS Single-part Polyurethane

4.1.1 Single-part polyurethane and activating agent must have at least the following material properties:

4.1.1.1 Viscosity: The viscosities of the soil stabilizer and activator are 25 to 50 centipoise (cps) and 100 to 200 cps, respectively, in accordance with ASTM D2196.

4.1.1.2 Unit Weight: The unit weights of the soil stabilizer and activator are 10.25 lb/gal and 8.5 lb/gal, respectively, in accordance with ASTM D1475.

5 QUALIFICATIONS OF INSTALLING CONTRACTOR

5.1 The Installing Contractor shall submit to the Owner a proposal including the documentation required in this section. Work shall not begin until all the submittals have been received and approved by the Owner. All costs associated with incomplete or unacceptable submittals shall be the responsibility of the Installing Contractor.

5.2 Evidence of the Installing Contractor's competence in the installation of polyurethane materials shall be provided to the Owner's satisfaction and may include any or all of the following:

5.2.1 Polyurethane manufacturer's certificate of competency in installation of polyurethane materials,

5.2.2 A list of at least three projects completed in the previous three years wherein the Installing Contractor installed polyurethane similar to those shown in the project Plans. Such a list shall include names and phone numbers of those project representatives who can verify the Installing Contractor's participation in those projects, and/or, 5.2.3 A letter from the polyurethane manufacturer or manufacturer's representative expressing ability and intent to provide on-site supervision of the polyurethane installation.

5.3 A listing of all safety violations lodged against the Installing Contractor within the previous three years and the current status or final resolutions thereof. Descriptions of safety improvements instituted within the previous three years may also be submitted, at the Installing Contractor's discretion.

6 PRE-CONSTRUCTION SUBMITTALS

6.1 Within two (2) weeks of receiving the contract award, the Installing Contractor shall submit the following documentation:

6.1.1 Certification that the proposed polyurethane material meets the requirements of Section 4.

6.1.2 Qualifications of the Installing Contractor per Section 5.2.

6.1.3 Minimum and/or maximum quantity of polyurethane material.

6.1.4 Soil testing procedures and failure criteria, if applicable.

6.1.5 Provide proof of insurance coverage as stated in the general specifications and/or contract.

7 POLYURETHANE INSTALLATION

7.1 Installing Contractor shall furnish and install polyurethane material per the project Plans. In the event of conflict between the project Plans and the Installing Contractors proposed installation method, the Installing Contractor shall not begin work until conflict has been resolved with the Owner.

7.2 The Owner shall request marking of underground utilities by an underground utility location service as required by law, and the Installing Contractor shall avoid contact with all marked underground utilities.

7.3 The portion of the construction site occupied by the Installing Contractor, including equipment and material stockpiles, shall be kept reasonably clean and orderly.

7.4 The installation of polyurethane may be observed by representatives of the Owner for quality assurance purposes. The Installing Contractor shall give the Owner at least 24 hours' notice prior to starting the polyurethane installation.

7.5 The polyurethane will be installed with an industrial pumping unit capable of output pressures of at least 3,000 psi. The pumping unit will be capable of controlling the pressure and rate of flow of the material, as well as, measuring the total amount of material injected.

7.6 If ⅝ inch diameter holes are required for the placement of the polyurethane material, the hole locations may be approved by the Owner prior to installation. After installation, the drilled holes will be cleaned out and filled with non-shrink grout or high-strength mortar mix.

7.7 The polyurethane material will be injected until all known or encountered voids under the pavement are filled or until the soil matrix has reached the required density.

7.8 The rate and quantity of material required will be determined by the Installing Contractor and approved by the Owner.

8 INSTALLATION RECORD SUBMITTALS

8.1 The Installing Contractor shall provide the Owner copies of the polyurethane material usage record within 24 hours after each installation section has been completed. Formal copies shall be submitted within 30 days following the completion of the polyurethane installation. The material usage record shall include, but is not limited to, the following information:

8.1.1 Date and time of installation

8.1.2 Location of installation

8.1.3 Total material used

8.1.4 Comments pertaining to interruptions, obstructions, other relevant information

9 CLEANUP

9.1 With one week of completion of the work, the Installing Contractor shall remove any and all material, equipment, tools, debris, of other items belonging to the Installing Contractor or used under the Installing Contractor's direction.

10 METHOD OF MEASUREMENT

10.1 The high-density polyurethane material shall be measured by the pound. Weight of the injected material will be recorded and documented at each location and at the end of each work shift.

MODEL SPECIFICATION FOR POLYURETHANE INSTALLATION FOR CONCRETE PAVEMENT/SLAB STABILIZATION APPLICATIONS

1 SCOPE

1.1 The work consists of designing, furnishing, and installing two-part, high-density polyurethane material according to the project Plans and this specification.

1.2 The parties and contract terms referred to in this specification are as follows:

1.2.1 The Owner is the person or entity that owns the facility or will own the facility once it is completed. The Owner may have contractual agreements with, and be represented by, other parties such as engineers, architects, or contractors that preform services under the direction of the Owner. Where Owner is used in the specification, it refers to the Owner or the Owner's contracted representatives separate from the Installing Contractor.

1.2.2 The Installing Contractor installs the polyurethane material and possibly performs other tasks associated with the project.

1.2.3 The Plans refer to the contract documents; including but not limited to the drawings and specifications for the project.

1.3 The work may include void filling, stabilizing and/or lifting pavement and slab structures, or undersealing pavement and faulted joints where required.

1.4 The Owner will provide suitable access to the construction site for the Installing Contractor's personnel and equipment.

1.5 Unless otherwise noted, the Installing Contractor shall provide all labor, tools, equipment, and material necessary to accomplish the work.

1.6 Unless specifically noted otherwise in the contract documents, the Owner will remove and replace structures, utilities, or other surficial improvements in the work area as necessary to facilitate the work.

1.7 The Owner will be responsible for overall construction oversight to preclude the development of unsafe conditions.

1.8 The Owner will be responsible for any soil density testing subsequent to the polyurethane foam injection, unless otherwise noted.

1.9 The work does not include any post-installation monitoring unless specifically noted otherwise in the contract documents.

2 REFERENCES

2.1 American Society for Testing and Materials (ASTM)

2.1.1 ASTM S1621: Compressive Properties of Rigid Cellular Plastics

2.1.2 ASTM D1622/D1622M: Apparent Density of Rigid Cellular Plastics

2.1.3 ASTM C273: Shear Properties of Sandwich Core Materials

2.1.4 ASTM D2842: Standard Test Method for Water Absorption of Rigid Cellular Plastics

3 APPROVED POLYURETHANE FOAM MANUFACTURERS

3.1 Foundation Supportworks®, Inc., 12330 Cary Circle, Omaha, NE 68128; Phone: (800) 281-8545; Fax (402) 393-4002.

3.2 Due to the special requirements for design and manufacturing of polyurethane foam, the system shall be obtained from Foundation Supportworks®, Inc., or other qualified manufacturer with an approved equivalent product. A request to substitute any other manufactured polyurethane foam product must be submitted to the Owner for review not less than seven (7) calendar days prior to the bid date. The request must include:

3.2.1 Documentation of at least five years of production experience manufacturing polyurethane products for similar applications,

3.2.2 Documentation that the manufacturer's polyurethane products have been used successfully on at least five similar projects within the last three (3) years, and/or,

3.2.3 Product acceptance by the local building code official(s) having jurisdiction over the project.

4 ACCEPTABLE PRODUCTS

4.1 Two-part, closed-cell, polyurethane foam products PolyLEVEL® 400 and PolyLEVEL® 400H manufactured in accordance with the requirements of Sections 5.1 and 5.2 of this specification.

5 MATERIALS

5.1 PolyLEVEL® 400 Two-part, High-density Polyurethane Foam

5.1.1 Two-part, one to one ratio by volume, closed-cell, high-density polyurethane foam system.

5.1.2 Viscosity: The viscosities of the resin and diisocyanate are 700 to 900 centipoise (cps) and 150 to 250 cps, respectively, in accordance with ASTM D2196.

5.1.3 Unit Weight: The unit weights of the resin and diisocyanate are 9.4 lb/gal and 10.25 lb/gal, respectively, in accordance with ASTM D1475.

5.1.4 Minimum free-rise density of at least 3.8 lb/cubic foot per ASTM D1622.

5.1.5 Minimum molded compressive strength of at least 85 psi per ASTM D1621.

5.1.6 Minimum molded shear strength of at least 120 psi per ASTM C273.

5.1.7 Maximum water absorption of less than or equal to 0.03 lb/square foot when tested per ASTM D2842.

5.1.8 Achieve 90% compressive strength in 15 minutes.

5.2 POLYLEVEL® 400H Two-part, High-density, Hydrophobic Polyurethane Foam

5.2.1 Used in applications where water is present beneath the slab.

5.2.2 Viscosity: The viscosities of the resin and diisocyanate are 700 to 950 centipoise (cps) and 150 to 250 cps, respectively, in accordance with ASTM D2196.

5.2.3 Unit Weight: The unit weights of the resin and diisocyanate are 9.7 lb/gal and 10.25 lb/gal, respectively, in accordance with ASTM D1475.

5.2.4 Minimum free-rise density of at least 3.8 lb/cubic foot per ASTM D1622.

5.2.5 Minimum molded compressive strength of at least 85 psi per ASTM D1621.

5.2.6 Minimum molded shear strength of at least 120 psi per ASTM C273.

5.2.7 Maximum water absorption of less than or equal to 0.03 lb/square foot when tested per ASTM D2842.

5.2.8 Achieve 90% compressive strength in 15 minutes.

6 QUALIFICATIONS OF INSTALLING CONTRACTOR

6.1 The Installing Contractor shall submit to the Owner a proposal including the documentation required in this section. Work shall not begin until all the submittals have been received and approved by the Owner. All costs associated with incomplete or unacceptable submittals shall be the responsibility of the Installing Contractor.

6.2 Evidence of the Installing Contractor's competence in the installation of polyurethane materials shall be provided to the Owner's satisfaction and may include any or all of the following:

6.2.1 Polyurethane manufacturer's certificate of competency in installation of polyurethane materials,

6.2.2 A list of at least three similar projects completed in the previous three years wherein the Installing Contractor installed polyurethane similar to those shown in the project Plans. Such a list shall include names and phone numbers of those project representatives who can verify the Installing Contractor's participation in those projects, and/or,

6.2.3 A letter from the polyurethane manufacturer or manufacturer's representative expressing ability and intent to provide on-site supervision of the polyurethane installation.

6.3 A listing of all safety violations lodged against the Installing Contractor within the previous three years and the current status or final resolutions thereof. Descriptions of safety improvements instituted within the previous three years may also be submitted, at the Installing Contractor's discretion.

7 PRE-CONSTRUCTION SUBMITTALS

7.1 Within two (2) weeks of receiving the contract award, the Installing Contractor shall submit the following documentation:

7.1.1 Certification that the proposed polyurethane material meets the requirements of Section 5.

7.1.2 Qualifications of the Installing Contractor per Section 6.2.

7.1.3 Minimum and/or maximum quantity of polyurethane material.

7.1.4 Soil testing procedures and failure criteria, if applicable.

7.1.5 Provide proof of insurance coverage as stated in the general specifications and/or contract.

8 POLYURETHANE INSTALLATION

8.1 Installing Contractor shall furnish and install polyurethane material per the project Plans. In the event of conflict between the project Plans and the Installing Contractors proposed installation method, the Installing Contractor shall not begin work until conflict has been resolved with the Owner.

8.2 The Owner shall request marking of underground utilities by an underground utility location service as required by law, and the Installing Contractor shall avoid contact with all marked underground utilities.

8.3 The portion of the construction site occupied by the Installing Contractor, including equipment and material stockpiles, shall be kept reasonably clean and orderly.

8.4 The installation of polyurethane may be observed by representatives of the Owner for quality assurance purposes. The Installing Contractor shall give the Owner at least 24 hours' notice prior to starting the polyurethane installation.

8.5 The polyurethane will be installed with a truck, trailer, or buggy mounted pumping unit capable of injecting high-density polyurethane material under the concrete slab or pavement. The pumping unit will be capable of controlling the temperature and rate of flow of the material, as well as, measuring the total amount of material injected.

8.6 If 5/8 inch diameter holes are required for the placement of the polyurethane material, the hole locations may be approved by the Owner prior to installation. After installation, the drilled holes will be cleaned out and filled with non-shrink grout or high-strength mortar mix.

8.7 Provide laser levels, manometers, dial indicators, or other measuring devices capable of detecting slab movement within 0.1 inches to verify stabilization and/or lift of the slab and/or structure.

8.8 The rate, temperature, and amount of material required will be determined by the Installing Contractor and approved by the Owner.

9 INSTALLATION RECORD SUBMITTALS

9.1 The Installing Contractor shall provide the Owner copies of the polyurethane material usage record within 24 hours after each installation section has been completed. Formal copies shall be submitted within 30 days following the completion of the polyurethane installation. The material usage record shall include, but is not limited to, the following information:

9.1.1 Date and time of installation

9.1.2 Location of installation

9.1.3 Total material used

9.1.4 Comments pertaining to interruptions, obstructions, other relevant information

10 CLEANUP

10.1 With one week of completion of the work, the Installing Contractor shall remove any and all material, equipment, tools, debris, of other items belonging to the Installing Contractor or used under the Installing Contractor's direction.

11 METHOD OF MEASUREMENT

11.1 The high-density polyurethane material shall be measured by the pound. Weight of the injected material will be recorded and documented at each location and at the end of each work shift.

EXPECT EXCELLENCE

Foundation Supportworks®, Inc. (FSI), headquartered in Omaha, Nebraska, is a leading manufacturer of helical pile systems, hydraulically-driven push pier systems, wall anchoring and wall bracing systems, polyurethane injection systems, and supplemental crawl space support systems. FSI places a strong focus not only on innovative solutions that are appropriately designed and tested, expertly installed and dependable to perform as promised, but also on providing a level of support and assistance to design professionals and installing contractors that is unparalleled in the industry.

When you work with FSI and our network of installing contractors, you can be assured you are working with a team of people who take great pride in their work and who are on a mission to change the level of service typically provided by contractors. We understand that the excellent products we discuss in detail in this technical manual are only half of the equation. The other half is being committed to do whatever it takes to make your project a success, and that includes sticking within project timeframes, communicating frequently and effectively, and offering flexibility and problem solving when project conditions change. We are committed to it all.

We believe that Foundation Supportworks is a different and special kind of company, and we look forward to you experiencing it firsthand.