1.1 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 foundations of lighthouse structures (*Figure 1.1* and *Figure 1.2*).

Figure 1.1 Lighthouse supported by screw (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 drive heads 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 product and the installation equipment.

Figure 1.2 Early patent of screw (helical) piles

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 in the International Building Code since the 2009 edition.

1.2 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, flights or helices 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 capacity. Brackets are used at the top 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 (*Figure 1.3*).

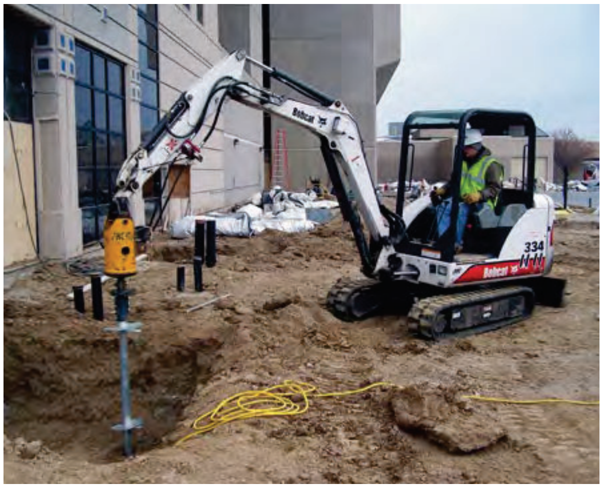


Figure 1.3 New construction helical pile installation

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 tiebacks and helical soil nails are types of helical anchors differentiated by their 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. (*Figure 1.4* and *Figure 1.5*). 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.

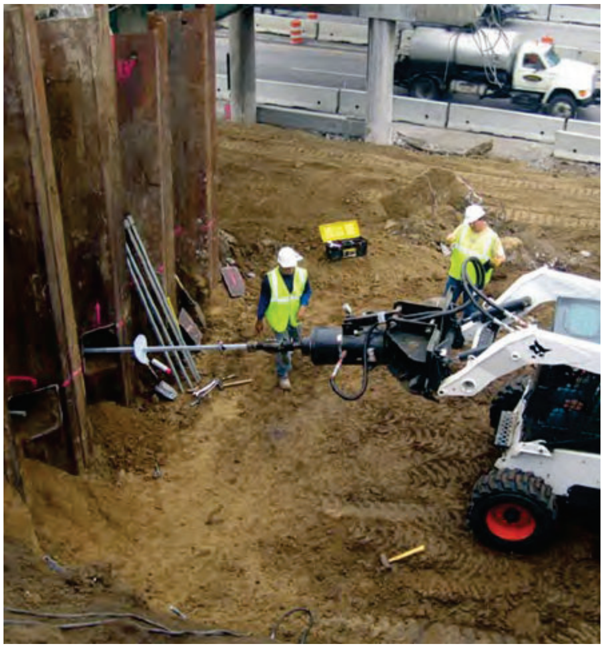


Figure 1.4 Sheet-pile wall stabilization with helical tiebacks

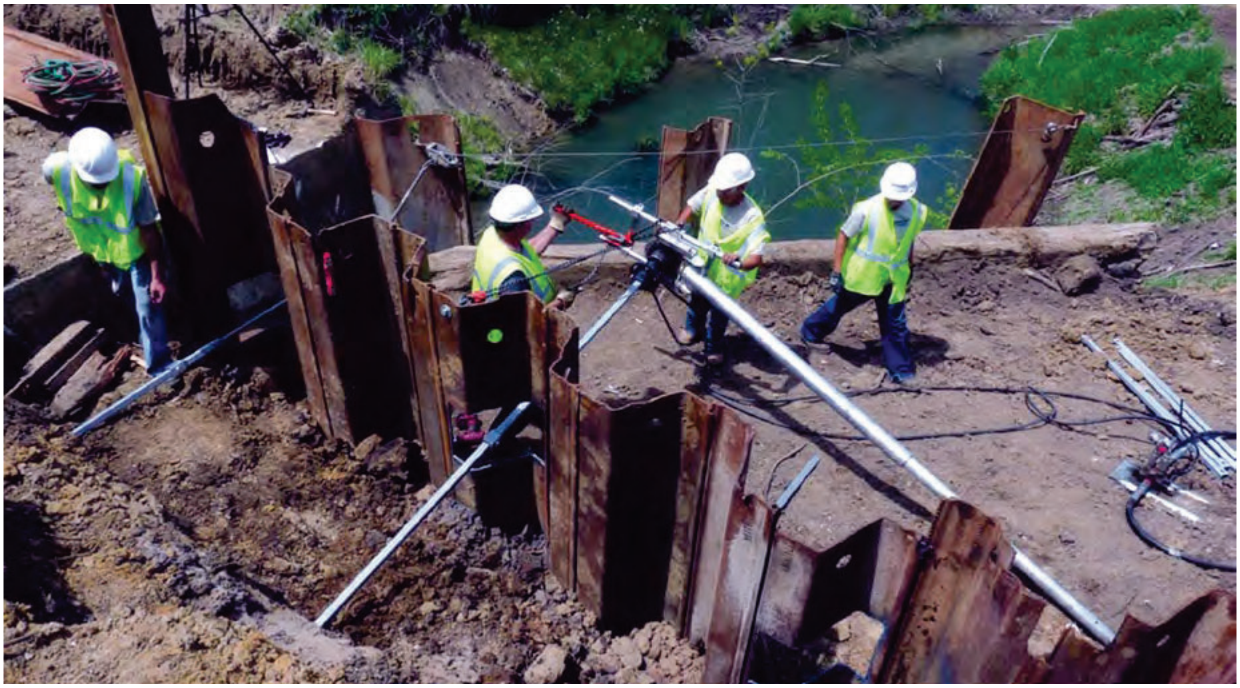


Figure 1.5 Helical tieback installation with hand-held equipment

1.3 Helical Foundation System Components

1.3.1 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 2 to 3 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 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 1.6*.

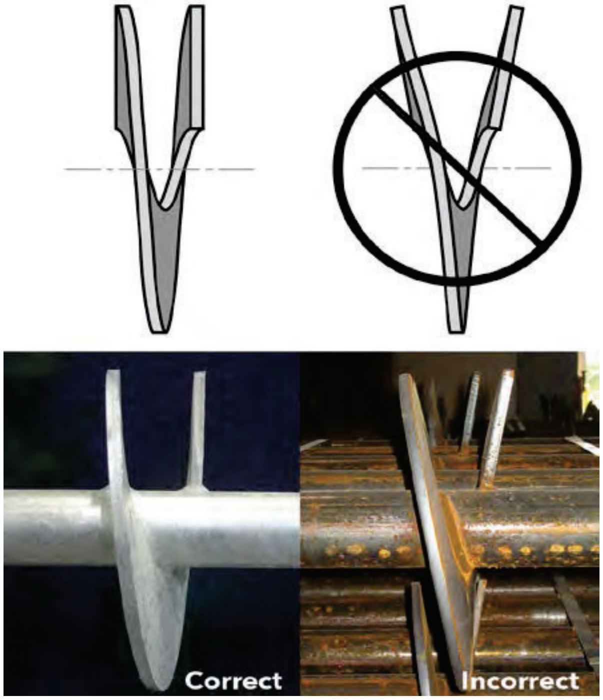


Figure 1.6 Well-formed helix (left) versus poorly formed helix (right)

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.

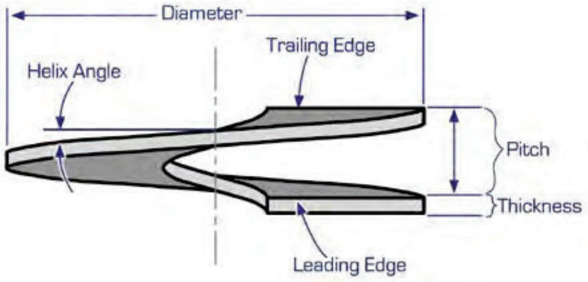


Figure 1.7 Helix Plate Geometry

A helix plate is further defined by geometric parameters including diameter, thickness, pitch, helix angle and edge geometry (*Figure 1.7*). 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 ⅟₁₆ 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. Foundation Supportworks currently offers:

- Helix plate diameters ranging from 6 inches to 16 inches
- Standard helix plate thicknesses of 5⁄16 inch, 3⁄8 inch, and ½ inch
- Plate steel yield strengths of at least 50 ksi (Grade 50).
- Standard H-style cut and V-style cut plates (*Figure 1.8*). 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.

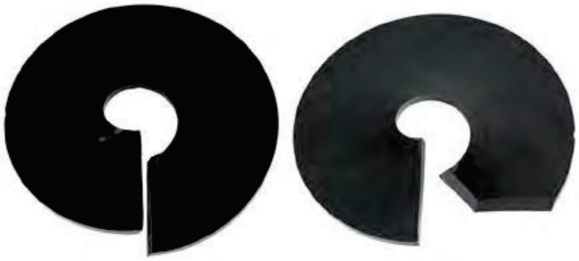


Figure 1.8 Standard H-style and V-style plates

1.3.2 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 either a 45-degree, bevel-cut or spiral-cut tip (*Figure 1.9*) with one or more helix plates welded along its length. The spiral-cut tip assists with pile advancement and penetration into dense to hard soil while the 45-degree, bevel-cut tip is generally acceptable for less dense soil conditions.

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 efficiency. Generally, a large track excavator would be required to provide the reach necessary to install these longer sections.

Figure 1.9 45-degree, bevel-cut tip and spiral-cut tip geometry

1.3.2.1 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 sectionof 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 (*Figure 1.10, Figure 1.11, and Figure 1.12*). These couplers can be sized to provide tight connections that reduce angular movement 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 1.10 Foundation Supportworks' HP288 with external welded coupler

Figure 1.11 Foundation Supportworks' HP350 with external detached coupler

Internal detached couplers are made from solid round stock, tube, or pipe material with an outside diameter smaller than the inside diameter of the central shaft material. 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.

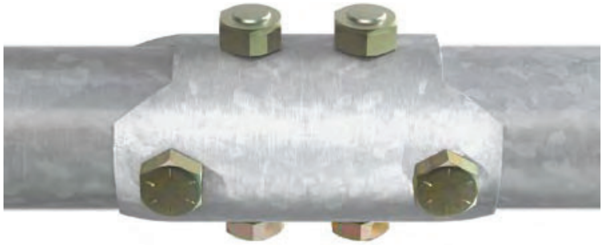


Figure 1.12 Foundation Supportworks' HP450 with external detached coupler

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 1.13*).

Figure 1.13 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 1.14*).

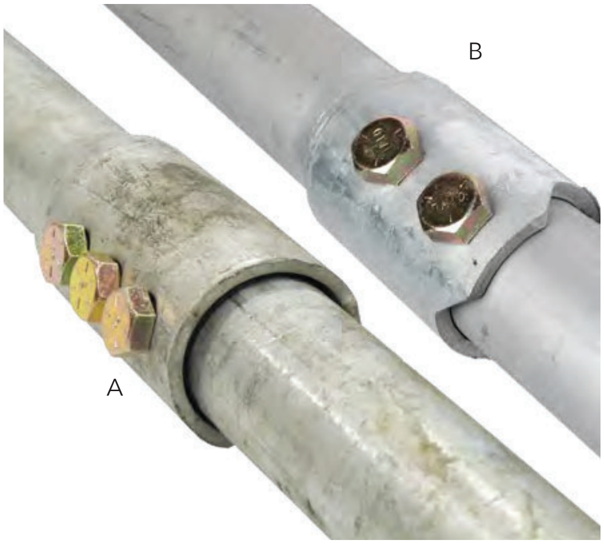


Figure 1.14 Coupler tolerances, (A) Competitor upset coupler, (B) Foundation Supportworks' external welded coupler

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 1.15*). 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 1.15 Competitor upset coupler variance from straightness

Foundation Supportworks' round shaft helical piles are manufactured with external welded or external 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 couplers. Foundation Supportworks 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 1.16*).

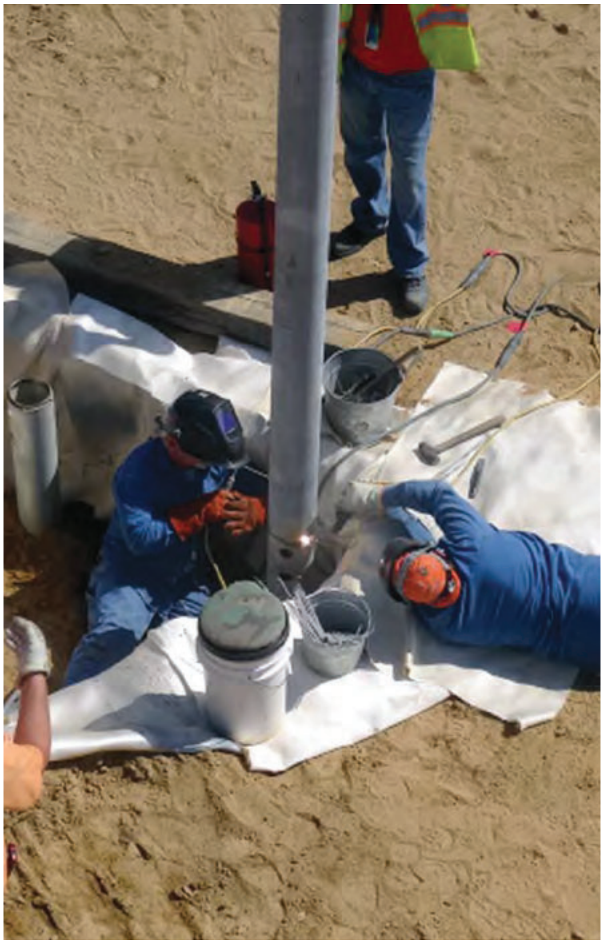


Figure 1.16 7-inch diameter pile with complete joint penetration welds between sections

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 1.17*. 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 (*Figure 1.18* and *Figure 1.19*).

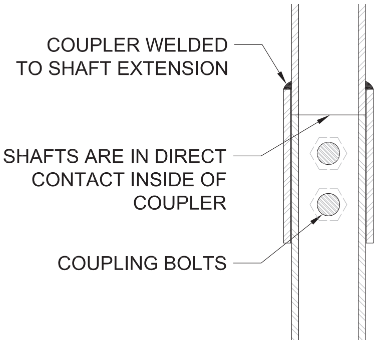


Figure 1.17 Coupler detail showing shaft contact within coupler

Figure 1.18 Foundation Supportworks' external welded coupler

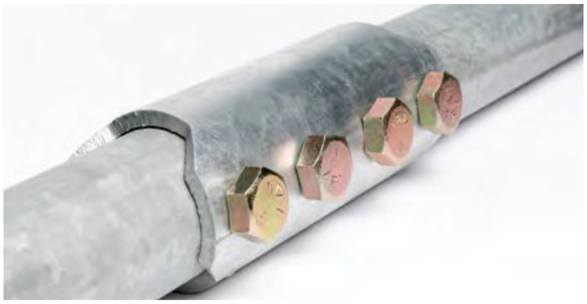


Figure 1.19 Foundation Supportworks' external detached coupler

The most common coupler detail for solid square shaft utilizes a forged and upset end (*Figure 1.20* and *Figure 1.21*). 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 1.22* clearly shows a comparison of coupling rigidity between a Foundation Supportworks external welded 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.

Foundation Supportworks engineers recommend 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.

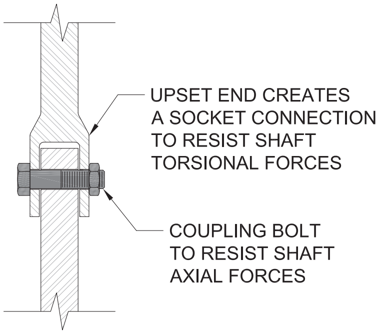


Figure 1.20 Schematic of square shaft forged and upset coupler

Figure 1.21 Square shaft forged and upset coupler

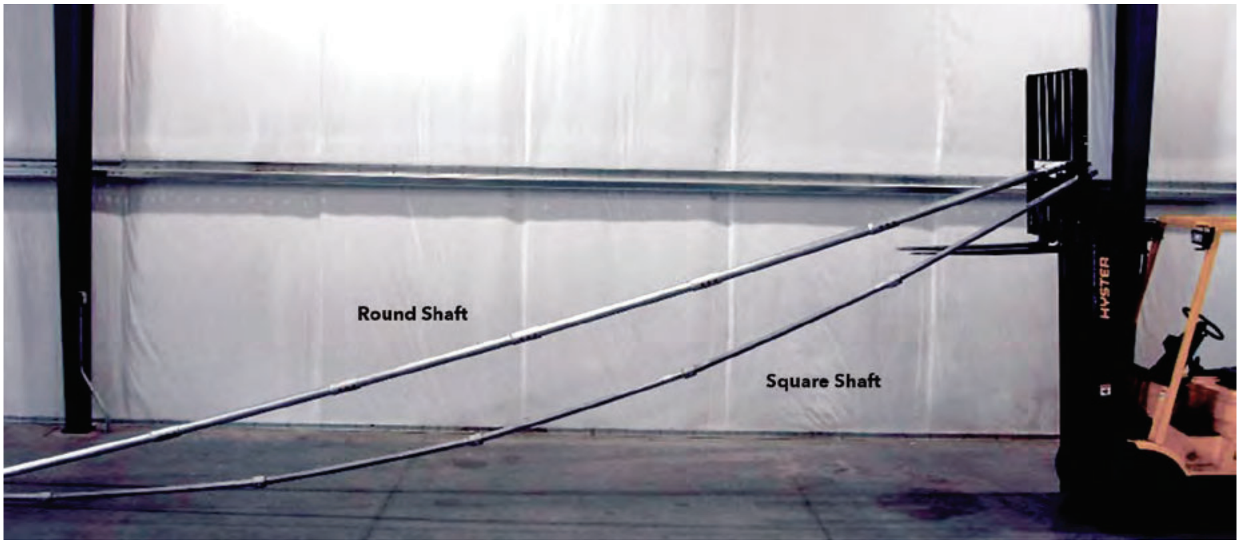


Figure 1.22 Coupler Rigidity Comparison: Foundation Supportworks' round-shaft external welded coupler vs. typical upset coupler for square shaft

1.3.2.2 Round vs. Square

Solid square shaft helical piles have been used successfully for decades in tension applications, e.g., 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 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, hollow round shafts are better suited for compression whereas solid square shaft may provide some advantages in certain tension applications. 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 Foundation Supportworks 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 solid square shaft piles, i.e., piles with a similar amount of steel in the cross section.

- 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 1.22*). Use of square shaft piles in compression should be reserved for light compression load applications in soil profiles that offer sufficient lateral support, e.g., standard penetration test (SPT) N-values ≥ 10 blows/foot (American Society for Testing and Materials (ASTM) D1586).

- As stated in the Coupler Detail section, Foundation Supportworks' round shaft helical piles are designed so the pile shafts are in direct contact within the coupling connections (refer back to *Figure 1.17*). 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 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 1.23*). 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.

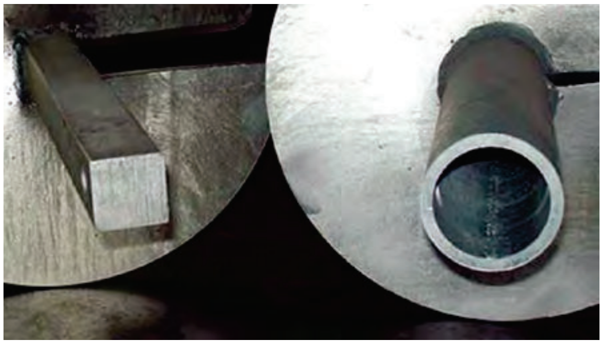


Figure 1.23 Section comparison between 2.875" diameter, 0.276" wall round shaft and 1.5" square shaft

- 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 therefore will achieve greater soil penetration for a given amount of torque. There is less skin friction during installation. 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 generally 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 1D* 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 passed this threshold when the shaft twist is not recovered after the installation torque is released. Although these observations can be used as a guide or point of reference during installation, Foundation Supportworks engineers do 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. Therefore, square shaft is much more forgiving during installation, allowing less experienced installers to decrease the applied torque before shaft damage may occur.

1.3.3 Brackets

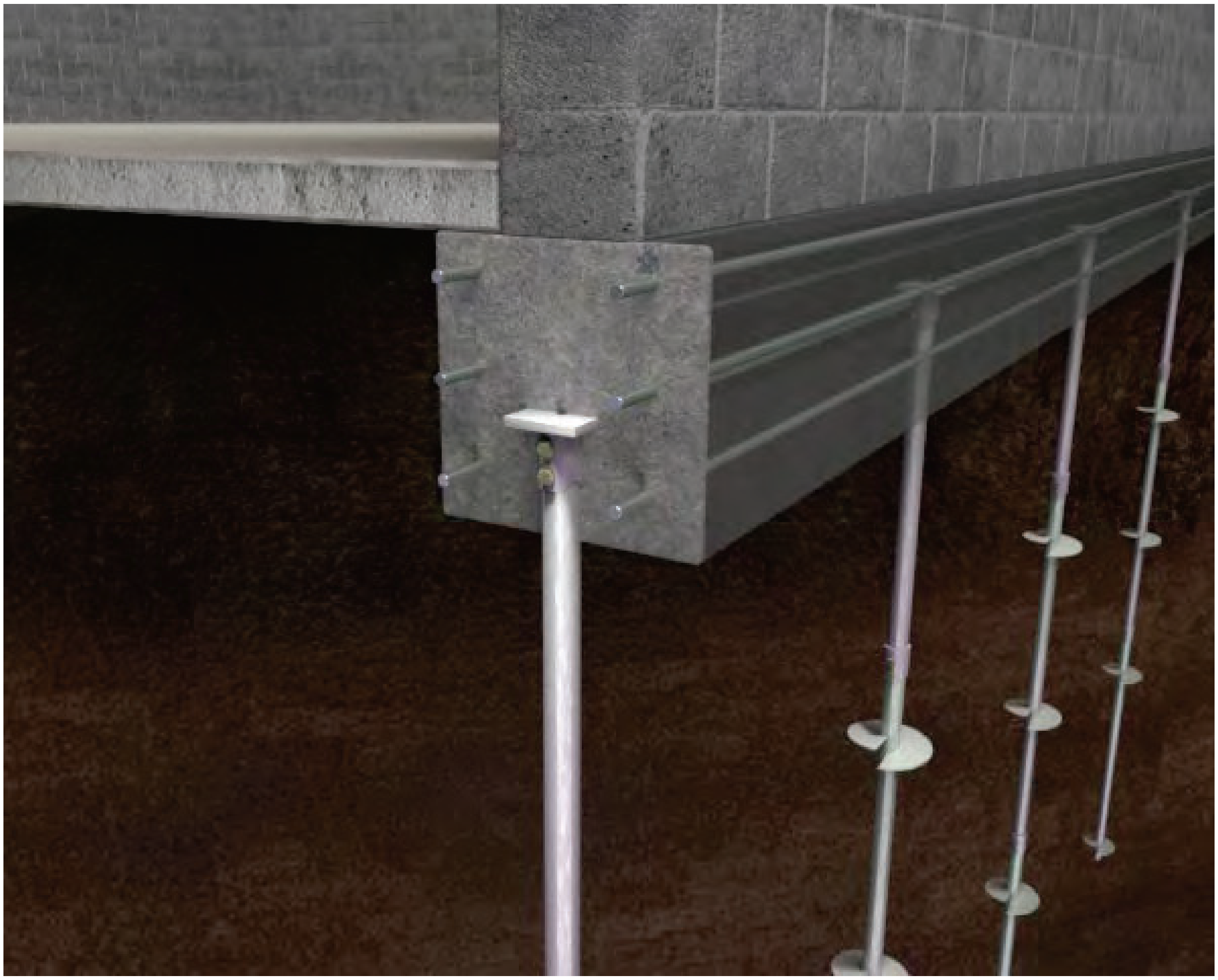


Figure 1.24 Rendering of new construction helical piles cast into a structural grade beam

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 of the grade beam or pile cap. New construction brackets often consist of round shaft sleeve (coupler) material with a flat plate welded to the top (*Figure 1.24* and *Figure 1.25*). 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. However, Foundation Supportworks engineers still recommend 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 1.25 Schematic of new construction bracket

Figure 1.26 Rendering of retrofit helical piers

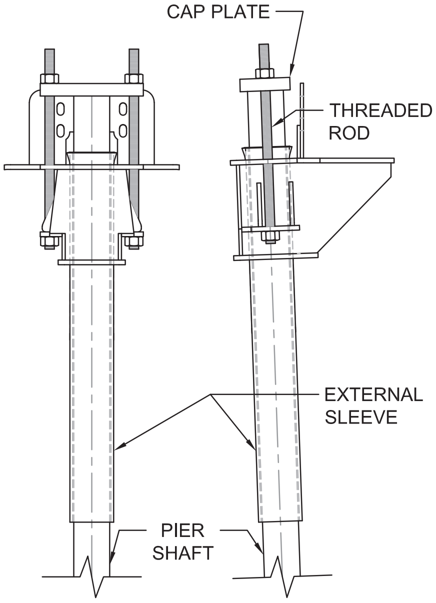


Figure 1.27 Retrofit bracket detail

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 (*Figure 1.26* and *Figure 1.27*). 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 1.28 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 1.28*). Helical tiebacks and helical soil nails may consist of either hollow round shaft or solid square shaft, although square shaft 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 a 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 load to the wall (*Figure 1.29*).

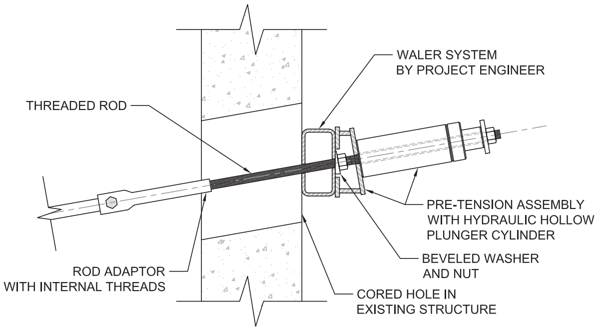


Figure 1.29 Schematic of helical tieback and pre-tension assembly

Foundation Supportworks engineers recommend that all helical anchors and tiebacks (excluding soil nails) be pretensioned or proof tested following installation. Pretensioning to 1.0 to 1.33 times the service 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 service load after proof testing. Helical anchors and tiebacks to be cast into new concrete retaining walls may be completely unloaded, locked off with a modest seating load, or locked off near the service load after proof testing. The design professional should determine pretensioning and lock-off procedures based upon project conditions, anticipated tieback deflections and the estimated tolerable movement of the supported structure. Tiebacks can be pull tested or load tested to typically two (2) times the service 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 service 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 (*Figure 1.30* and *Figure 1.31*), elevated structures in high tide or hurricane-prone areas, pipe buoyancy control, guy wires, tie downs, etc. Please contact the Foundation Supportworks Engineering Department with any questions regarding bracket details or availability.

Figure 1.30 Boardwalk supported on vertical and battered helical piles

Figure 1.31 Custom saddle bracket connected with clevis to battered helical pile

1.4 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-correlated capacities of 100 kips may be achieved with helical shaft sizes of 4.5 inches in diameter. 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" helix plates, thicker helix plates and spiral-cut lead shaft tips are also available for penetrating and/or bearing in dense soils.
- **Well-established torque-to-capacity relationship** – Empirical torque factors have been established through years of product testing. Default torque-to-capacity 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 (*Figure 1.32*, *Figure 1.33* and *Figure 1.34*). The equipment and drive heads can be sized according to the project design loads as well as site access.

Figure 1.32 Skid steer installing helical piles within limited space at a substation

Figure 1.33 Mini-excavator lowered by crane into excavation to install helical tiebacks

Figure 1.34 Helical piles installed with hand-held equipment to support new elevator within existing school

- **Low mobilization costs** – Helical piles can achieve moderate to high capacities, yet be installed with smaller equipment than most other deep foundation systems. This results in lower mobilization costs 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 haul-off 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, auger-cast piles, or drilled and grouted micropiles. It is common to install a helical test pile and then test the pile 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, auger-cast piles, or drilled and grouted micropiles. On schedule-sensitive projects, the contractor may place reinforcing steel and pour foundation concrete directly behind the helical pile installation.

- **Clean installation** – Installation of standard helical piles, helical tiebacks and helical soil nails does not include concrete or grout, thereby minimizing equipment, vehicles and mess on the construction site.

1.5 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.

- Standard helical piles, tiebacks and soil nails are steel foundation elements that generate capacity through interaction with the soil in which they are embedded. 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 1D* for additional information about corrosion.

- Helical piles may 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 1.35*). The use of a spiral-cut tip may also assist penetration through or into dense soils (refer back to *Figure 1.9*). 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.

- Helical piles will not typically penetrate hard rock, defined by auger refusal by the drill rig or SPT N-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 or a spiral-cut lead tip on hollow round shaft may again be considered along with the larger installation equipment.

Figure 1.35 Combination pile with HA175 stinger coupled immediately to 3.5" O.D. shaft

- The slenderness of helical pile shafts and their limited exposed area to the surrounding soil does not allow for generation of high lateral capacities. In competent soils, ultimate lateral capacities may range from less than 2 kips to more than 6 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.

1.6 Design Considerations

1.6.1 Spacing & 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 1.36*). Helical piles designed with helix plate spacing in accordance with AC358 could, therefore, use either the Individual Bearing or Cylindrical Shear Methods for 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 1.7*.

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 battered away from each other in order to meet the AC358 spacing criteria at the helix bearing depth.

The center-to-center spacing of laterally loaded piles shall be considered both at the ground surface and the depth of helix plate bearing. Center-to-center spacing at the ground surface shall be at least eight (8) times the diameter of the pile shaft. Spacing between helix plates shall not be less than three (3) times the diameter of the largest helix plate measured from the edge of the helix plate to the edge of the adjacent helical pile plate, or four (4) diameters where the spacing is measured from the center to center of adjacent pile plates (ICC-ES AC358). If these criteria are not met, an analysis should be completed to determine if there should be a reduction in the lateral capacity per pile.

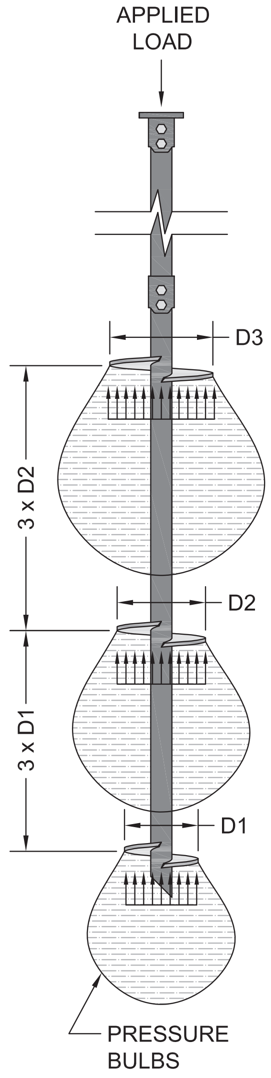


Figure 1.36 Helix plate spacing with bulbs of significant stress influence

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 minimum depth of the uppermost helix plate would therefore be determined from the greatest of these values, or deeper if required to achieve adequate bearing for the project.

Expansive soils are prevalent in many regions of the United States and Canada with varying degrees of shrink/swell potential. Volumetric changes (shrink/swell) occur in expansive soils from seasonal fluctuations of the natural moisture content. As mentioned above, the depth of soil that is affected by changes in seasonal moisture is commonly referred to as the "active zone" or the "depth of wetting". Below this depth, the soil moisture content is presumed to remain relatively constant, thereby limiting any volumetric changes within the deeper strata. The geotechnical investigation may identify layers of expansive soils and swell testing may be performed on soil samples to determine the potential vertical rise (PVR) and swell pressure of the expansive zone upon wetting. When helical piles are installed in expansive soil profiles, it is important to advance the helix plates to depths below the active zone to limit pile and structure movement.

If expansive soils are found to exist under shallow footings or pile caps, swell pressures determined from testing can be used to estimate the uplift force on the foundation. This potential uplift pressure, or force, must be considered when designing helical piles for both retrofit and new construction applications. In new construction applications, the helical piles and connections to the structure, along with the structure itself, may be designed to resist the uplift forces. In retrofit applications, modifications to the bracket/pile/structure connections may be necessary to provide uplift restraint.

It may also be possible to design the foundation system so that the dead loads from the structure produce bearing pressures that exceed the swell pressures and resulting forces from the underlying soils. When this is not possible and a deep foundation is required, void form is often used in new construction to provide a gap between the soil and pile supported grade beams and pile caps. This would allow the soil to expand several inches upon wetting without inducing an uplift force on the foundation system. Void form typically consists of rigid cardboard box shapes that can support the weight of the new concrete, but then weakens over time as it absorbs moisture. In retrofit applications, the contractor may first support and level the existing structure with helical piers, and then excavate a void between the concrete footing and the soil. If expansive soils are a concern, at a minimum, the geotechnical engineer should provide guidance regarding the depth of wetting, PVR and swell pressures that may be applied to the foundation. Any resulting uplift forces should be considered in the helical pile design, and additional appropriate measures taken.

1.6.2 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 American Institute of Steel Construction (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 1.37*).

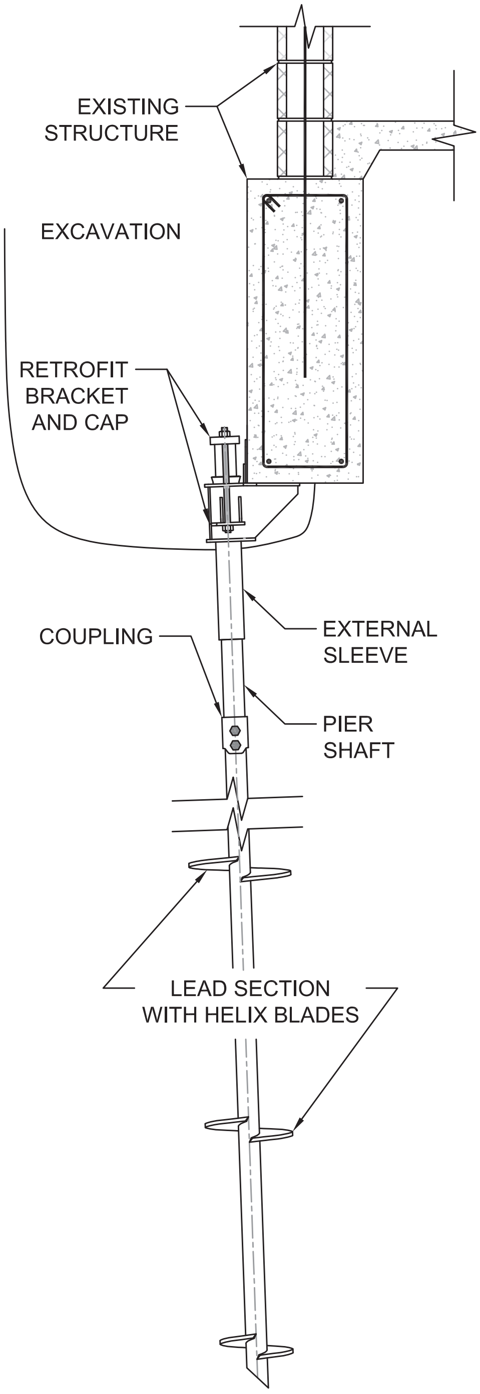


Figure 1.37 Schematic of retrofit helical pier installation

Most helical piles used in residential and commercial 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, which when considered 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 2 inches, which is less than what would be considered typical for most retrofit piering systems.

Foundation Supportworks engineers address the issue of retrofit helical pier eccentricities in 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.

1.7 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 engineers recommend 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 service 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 1E* ***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 engineers recommend that helical pile design be completed by an experienced geotechnical engineer or other qualified design professional.

1.7.1 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 three times the diameter of the leading plate, the uppermost helix plate is embedded to a depth of at least five diameters, and skin friction along the shaft is generally ignored for shaft sizes less than 6 inches in outside diameter. *Figure 1.38* illustrates the load transfer mechanism for the Individual Bearing Method in compression loading.

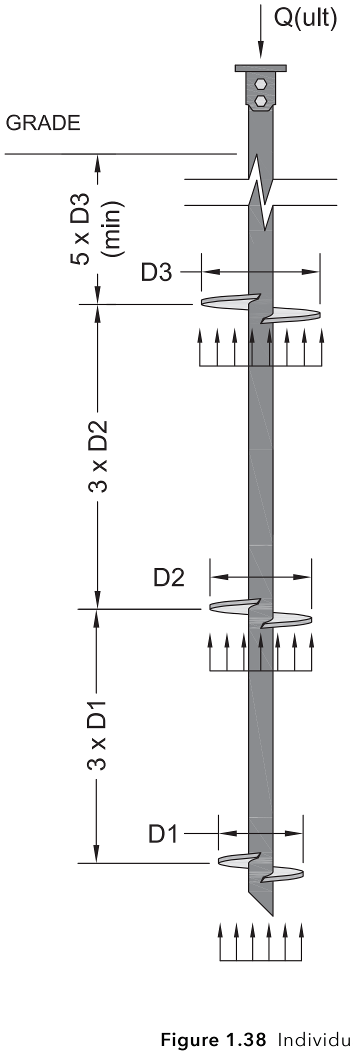


Figure 1.38 Individual Bearing Method

Helical pile capacity by the Individual Bearing Method can be calculated from:

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

Where,

Q_u = Ultimate Pile Capacity (lb)
c = Cohesion at Helix Depth (lb/ft²)
q' = Effective Vertical Overburden Stress at Helix Depth (lb/ft²)
γ = Soil Unit Weight (lb/ft³)
B = Diameter of Helix Plate (ft)
A_h = Area of Helix Plate (ft²)
N_c, N_q, N_γ = 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:

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

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:

$$Q_u = \sum A_h(9c)$$

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

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

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 displacement piles due to the lack of spoils during installation, 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 engineers recommend N_c and N_q bearing capacity factors calculated by the following equations and shown graphically in *Figure 1.39*:

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

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

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

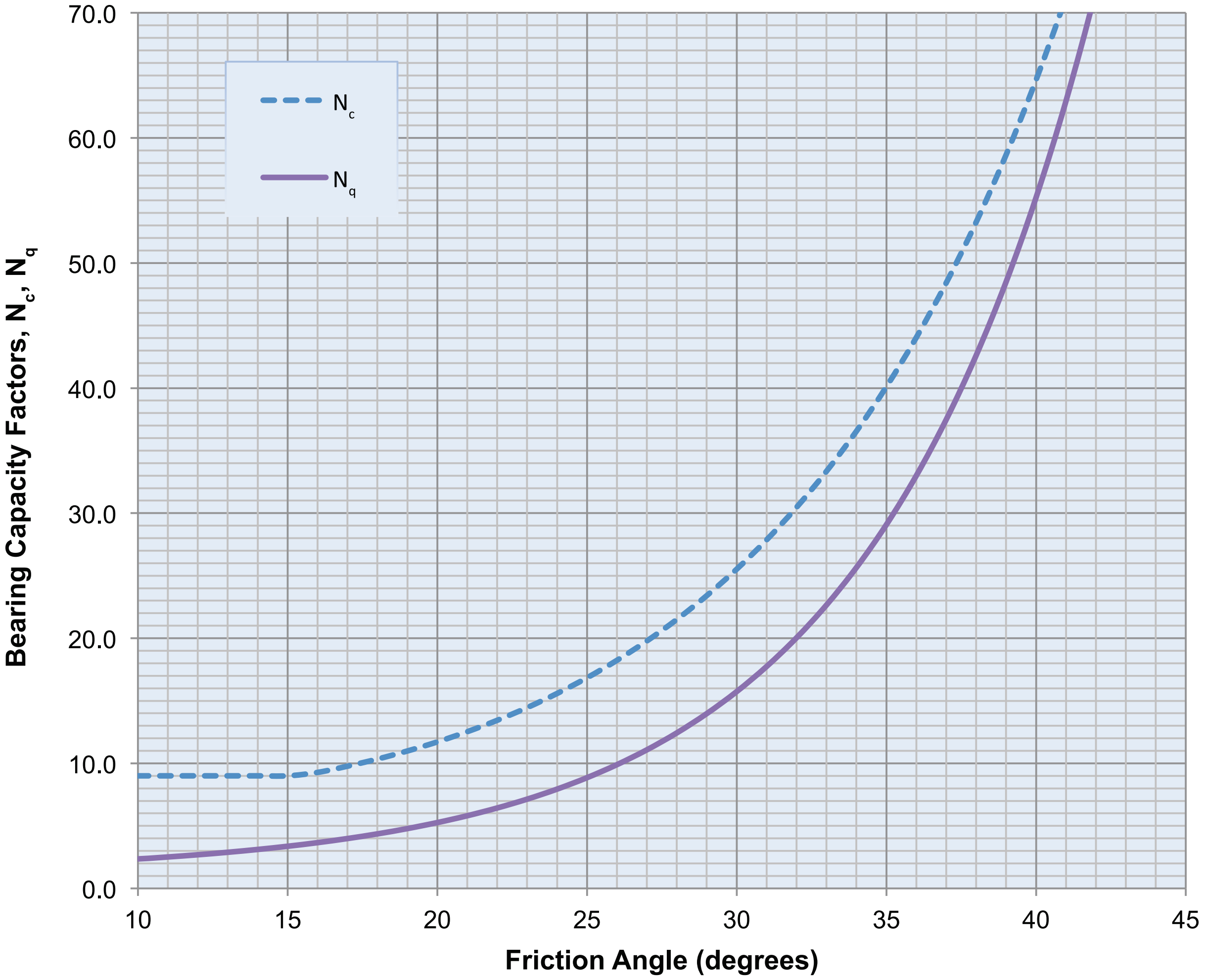


Figure 1.39 Recommended Bearing Capacity Factors N_c and N_q versus Soil Friction Angle

1.7.1.1 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 below the critical 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. Foundation Supportworks engineers recommend 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.

1.7.2 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 three times the diameter of the leading plate and the uppermost helix plate is embedded to a depth of at least five diameters. Skin friction along the shaft is generally ignored for shaft sizes less than 6 inches in outside diameter. *Figure 1.40* illustrates the load transfer mechanism for the Cylindrical Shear Method in compression loading.

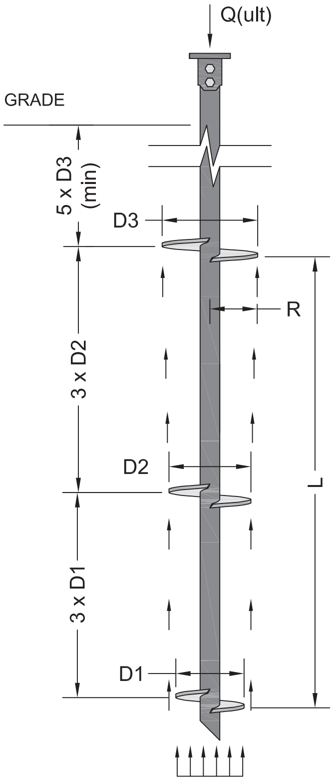


Figure 1.40 Cylindrical Shear Method

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/ft²)
K_o = Dimensionless At-Rest Earth Pressure Coefficient
Φ = Soil Friction Angle (degrees)
A_h = Area of the Top or Bottom Helix Plate (ft²)
q' = Effective Vertical Overburden Stress at A_h (lb/ft²)
N_c, N_q = Dimensionless Bearing Capacity Factors

Refer to Section 1.7.1 for discussions regarding bearing capacity factors and critical depth.

Foundation Supportworks engineers promote the use of the Individual Bearing Method for determination of pile capacity when helix spacings are ≥ 3D 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.

1.7.3 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:

$$Q_u = K_t \cdot T$$

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 2 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. Notable exceptions include K_t values for square shaft piles larger than 1.75 inches and less than or equal to 3 inches. In these cases, AC358 recommends K_t values lower than 10 ft^{-1}, determined from an equation that considers the diagonal dimension of the shaft as the "effective diameter." These K_t values for larger square shaft are then comparable to their round shaft counterparts.

ICC-ES AC358 recognizes the following helical pile shaft sizes and respective 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 square shaft $K_t = 10$ ft^{-1}
- 2.375-inch O.D. round $K_t = 10$ ft^{-1}
- 2.875-inch O.D. round $K_t = 9$ ft^{-1}
- 3.5-inch O.D. round $K_t = 7$ ft^{-1}
- 4.5-inch O.D. round $K_t = 5.5$ ft^{-1}

The K_t factors above may be considered conservative for most applications, and even though they are often presented as constant values, K_t can vary depending upon the soil conditions; i.e., K_t factors are generally higher in well-graded sand and gravel versus silt and clay soils. K_t is also inversely proportional to the shaft dimension/diameter as shown above.

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 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.

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 drive heads will likely affect the K_t determined from field load testing. The helical pile industry has long used the differential pressure across the drive head for correlation to installation torque. The installation torque is then correlated to pile capacity. In other words, the differential pressure across the drive head 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 drive head manufacturers based on theoretical equations and will vary with the planetary gear ratio, hydraulic motor displacement and motor efficiency. Drive head manufacturers typically show a linear fit between the differential pressure and output torque with no scaling effect. Research has shown that the drive head 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 drive head efficiency. Therefore, it is highly recommended that drive heads 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 heads 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 1.3.1* for an in-depth discussion about helix plate geometry. Proper installation procedures and well-formed helix plates are critical to minimize soil disturbance.

1.8 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 (*Figure 1.42* and *Figure 1.43*)
- 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 1.44*)

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 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 1.29*).

Figure 1.42 Rendering of helical tieback installation for soldier pile and timber lagging wall

Figure 1.43 Multi-tier helical tieback installation to support sheet pile wall

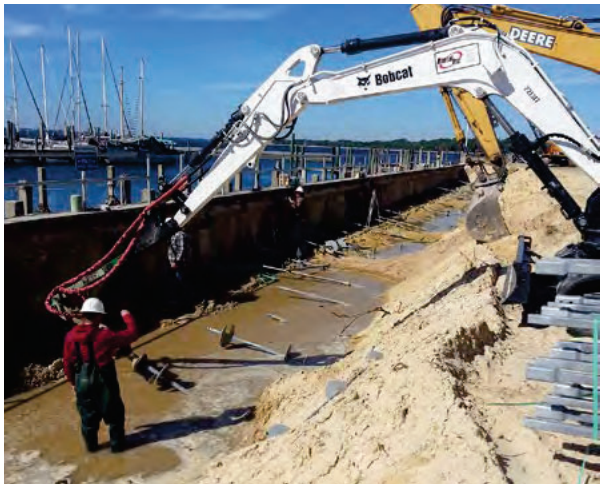


Figure 1.44 Helical tiebacks stabilize marina seawall

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 1.7*.

Helical tiebacks are often used to stabilize existing earth retaining structures that have experienced excessive movement, e.g., walls that are cracked, leaning and/or bowing (*Figure 1.45* and *Figure 1.46*). 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 1.45 Helical tiebacks stabilize sheet pile wall below historic home

Figure 1.46 Helical tiebacks and tube steel walers stabilize concrete retaining wall

1.8.1 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 (plate closest to the wall) shall be at least five (5) times its diameter beyond the estimated failure plane (*Figure 1.47*). The effective tieback length, i.e., the axial embedment of the last helix plate from the tip, should also be a minimum distance of twelve (12) times its diameter from the wall face, following the general guidelines in AC358 for K_t verification for tension piles. Again, the design professional may determine effective tieback lengths more or less than this value based upon site-specific soil and project conditions. 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.

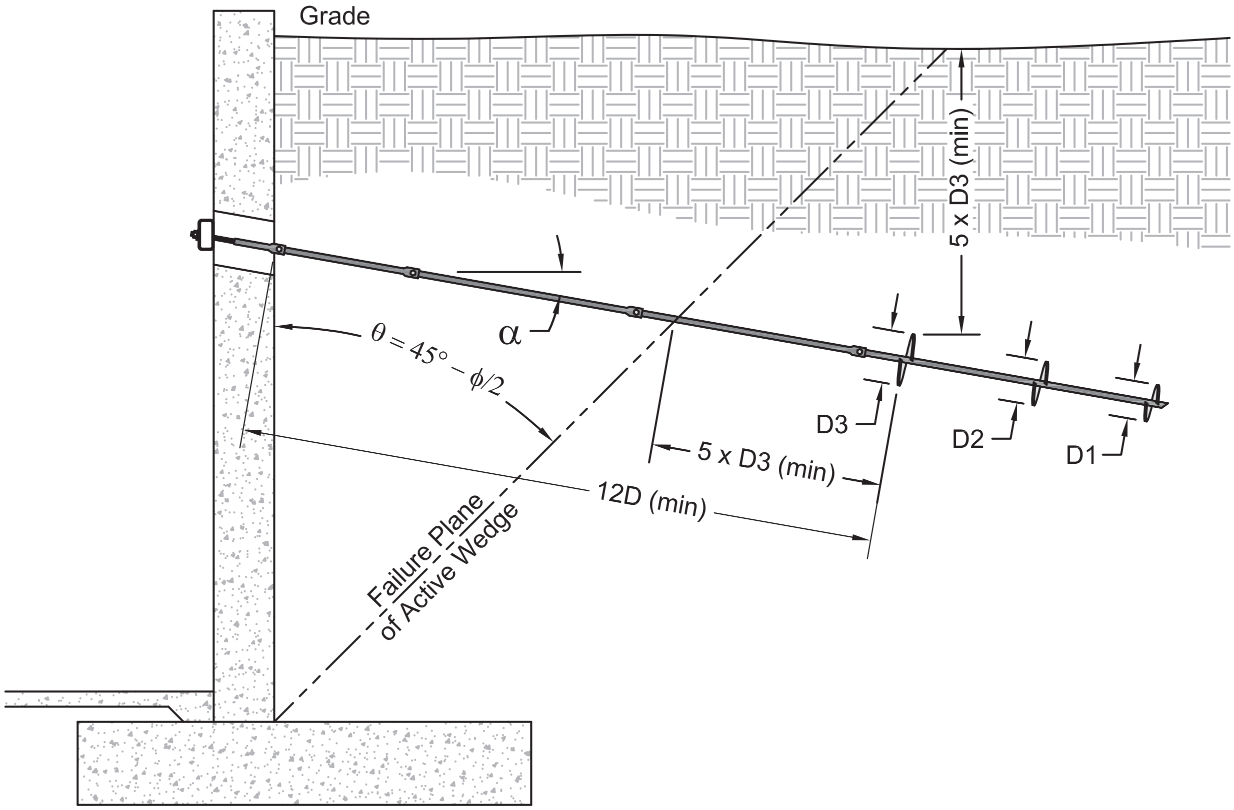


Figure 1.47 Helical tieback design considerations
(Failure plane origin varies based on project-specific parameters)

Tiebacks designed with a downward installation angle (α) should be installed to a torque-correlated capacity equal to or greater than the required axial tieback capacity (T_R) (*Figure 1.48*). The required axial tieback capacity increases with an increase in tieback installation angle, provided the calculated horizontal tieback capacity (T_{CH}) is held constant. The calculated horizontal tieback capacity (T_{CH}) is determined from analysis considering the various loads on the wall. ***Remember that the torque-correlated ultimate capacity should exceed the service load by an appropriate factor of safety.*** The equation for determining the required axial capacity of a downward battered tieback is:

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

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 component of the tieback force (T_{CV}) will increase with an increase in installation angle, provided the calculated horizontal tieback capacity (T_{CH}) 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

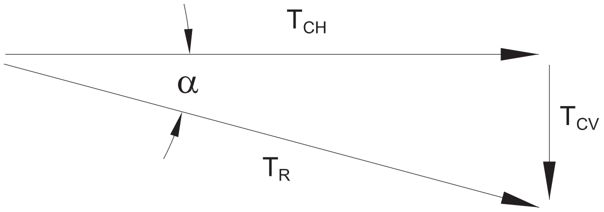


Figure 1.48 Vector mechanics of tieback forces

Angled or beveled washers are recommended at the tieback/bearing plate interface to more uniformly transfer the tension loads (refer back to *Figure 1.29*). Beveled washers are especially critical when the tiebacks are pretensioned 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.

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 or wall footing upward at an angle of 45-Φ/2 (degrees) from vertical for both active and at-rest conditions. For basement walls, this failure plane is usually assumed to begin at the bottom of the wall. For cantilevered retaining walls, it may be appropriate to model the failure plane beginning at the bottom of the wall or the bottom back edge of the footing heel. For sheet-pile walls, the failure plane is usually assumed to begin at or slightly below 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. Helical tiebacks used in conjunction with earth retention systems should have a minimum factor of safety of two.

Foundation Supportworks engineers recommend that all helical anchors and tiebacks (excluding soil nails) be pretensioned or proof tested following installation (*Figure 1.49****). Pretensioning to 1.0 to 1.33 times the service 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 service load after proof testing. Helical anchors and tiebacks to be cast into new concrete retaining walls may be completely unloaded, locked off with a modest seating load, or locked off near service load after proof testing. The design professional should determine pretensioning and lock-off procedures based upon project conditions, anticipated tieback deflections and the estimated tolerable movement of the supported structure. Tiebacks can be pull tested or load tested to typically two (2) times the service 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 service load, or failure, are generally considered sacrificial and should not be used as production tiebacks.***

Figure 1.49 Pretensioning helical tieback

1.9 Helical Soil Nails

Soil nailing is a method of earth retention that relies on a grid of individual reinforcing 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.

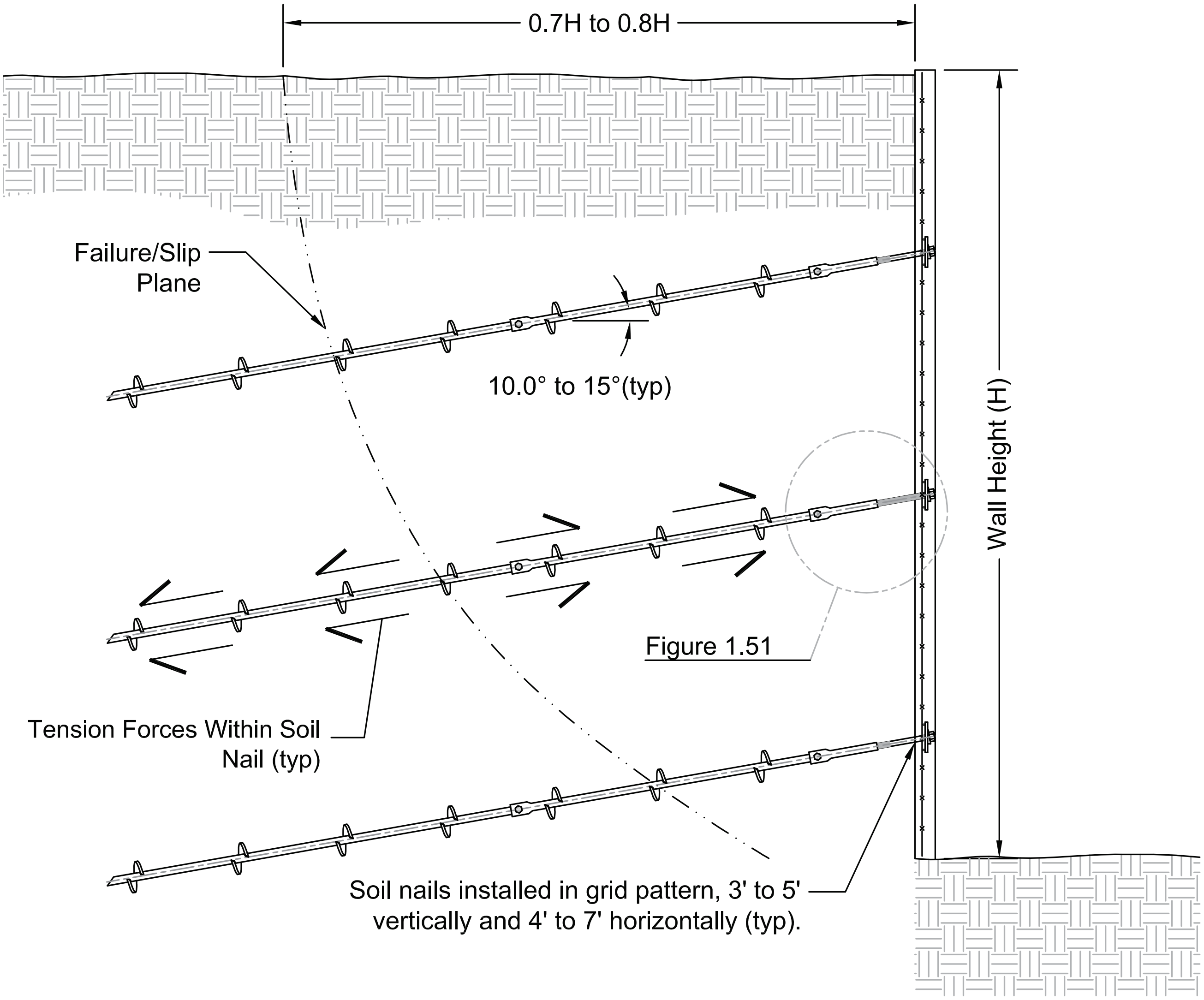


Figure 1.50 Helical soil nail wall installation

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.

- 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.

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 (*Figure 1.50* and *Figure 1.51*). 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 pretensioned 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.

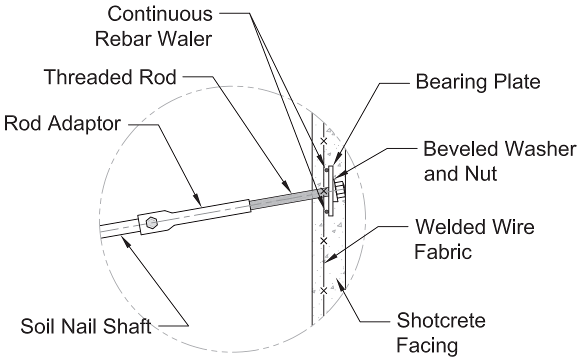
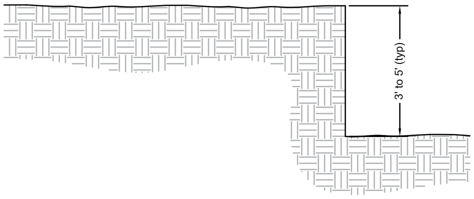


Figure 1.51 Nail head to wall detail

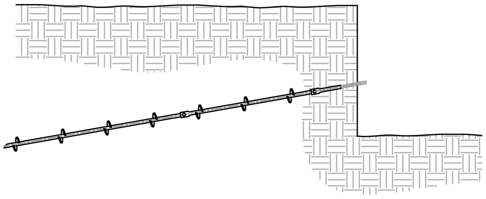
1.9.1 Construction Methodology

Soil nail walls are constructed from the top down where the excavation proceeds as shown in *Figure 1.52*. The construction sequence for a typical helical soil nail wall includes:

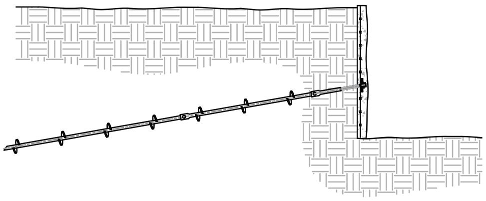
- Initial excavation about 3 to 5 feet deep depending upon design parameters and soil conditions
- Installation of the first row of helical soil nails to the required inclination angle, torque and embedment length
- Placement of drainage medium (if required)
- Placement of wall reinforcement and bearing 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).



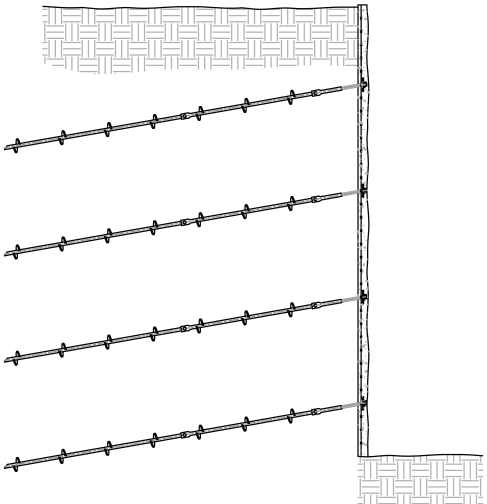
Step 1 - Initial Excavation



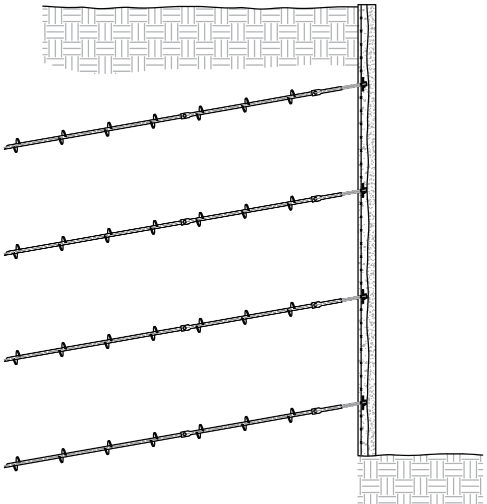
Step 2 - Install top row of soil nails



Step 3 - Install drainage strips, reinforcing steel and anchor plates, and apply initial shotcrete layer.



Step 4 - Repeat steps 1 through 3 to bottom of wall.



Step 5 - Apply final or permanent wall facing, if required.

Figure 1.52 Typical soil nail wall installation sequence

1.9.2 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:
    - Dry, poorly-graded cohesionless soils, e.g., clean sands or sands with SPT N-values less than 5 blows/foot
    - Highly plastic clays, expansive soils, organic soils, or soils with a liquidity index of 0.2 or greater
    - Clays with SPT N-values less than 4 blows/foot
    - Soil profiles with high groundwater levels - dewatering may be required to facilitate installation
    - Soil with cobbles, boulders or weathered rock lenses
    - Highly corrosive soils
    - Collapsible soils
    - Very dense sands and hard clays - may be difficult to penetrate without predrilling 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, 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 is not required. The downward installation angle is a carryover from grouted nail design where an angle is required to limit 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 1.9.2.2*.

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 1.7* and is correlated to bond stress by:

$$q_u = Q_u / L\pi D_h FOS$$

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)
FOS = Factor of Safety for Uncertainties in Soil Conditions (Typically 1.5 to 2.0 Based on Quality of Soil Information)

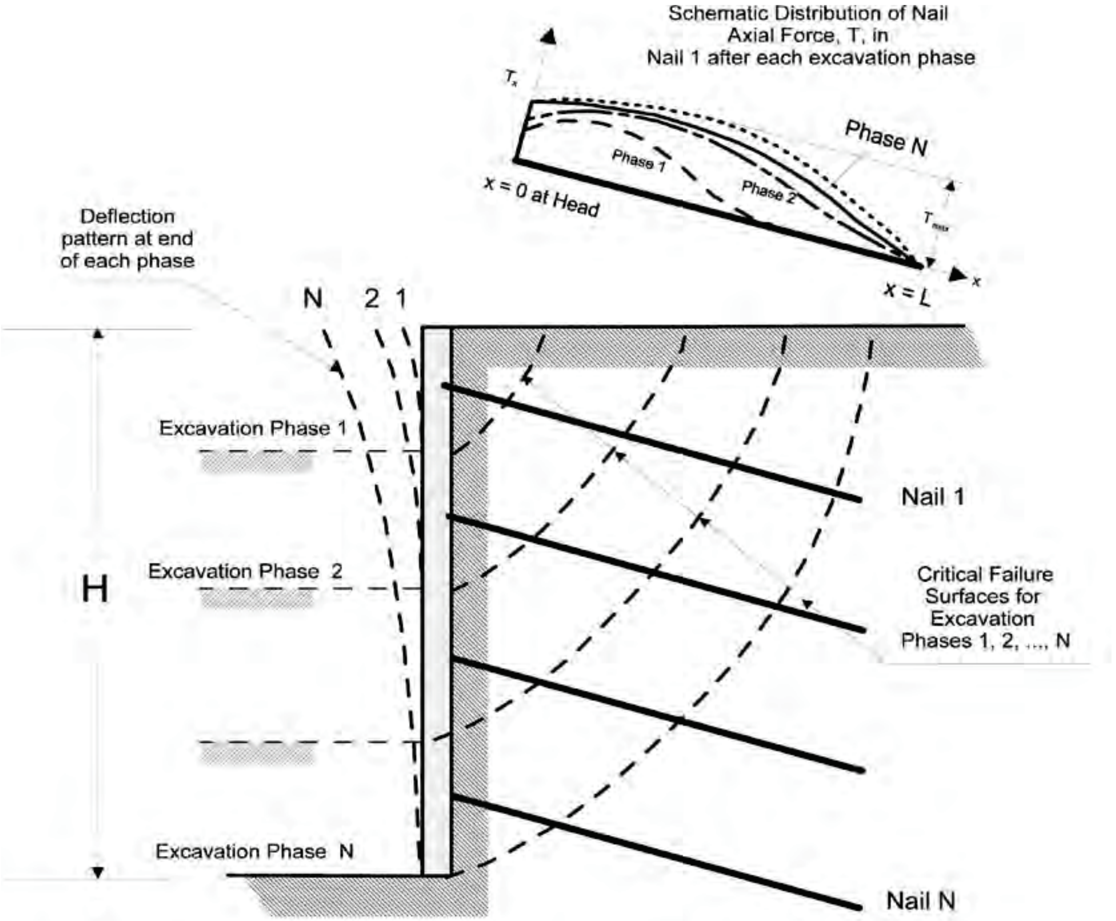


Figure 1.53 Potential failure surfaces and soil nail tensile forces (Lazarte, Elias et al. 2003)

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 1.53* 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 1.53* illustrates the tensile force distribution along the top soil nail as construction continues through the various excavation phases. The tensile force in Nail 1 is shown to decrease once Phase N is completed due to load redistribution among the lower rows of soil nails as the excavation progresses between Phase 2 and Phase N.

The design of helical soil nail walls should be performed in general accordance with requirements detailed in FHWA Geotechnical Engineering Circular No. 7 (Lazarte, Elias et al. 2003).

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 1.9.2.2*.

The design of helical soil nails should be completed by experienced design professionals. Installation of Foundation Supportworks helical soil nails shall be by certified Foundation Supportworks installing contractors trained specifically for helical soil nail installations. Foundation Supportworks engineers recommend 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 Foundation Supportworks 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.

1.9.2.1 Temporary & Permanent Wall Facing

Helical soil nail walls may be classified 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.

1.9.2.2 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).

1.10 Load Tests

Figure 1.54 Compression load test

Load tests are routinely completed on helical piles to establish nominal 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 (*Figure 1.54* and *Figure 1.55*)
- ASTM D3689, Standard Test Methods for Deep Foundations Under Static Axial Tensile Load (*Figure 1.56*)
- ASTM D3966, Standard Test Methods for Deep Foundations Under Lateral Load

Figure 1.55 Close up of test pile, hydraulic cylinder, dial gauges and hemispherical bearing plate

Figure 1.56 Tension load test

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.

1.11 Design Examples

Three common methods for determining helical pile end-bearing capacity are presented in *Section 1.7*. The individual bearing and cylindrical shear methods are used during the design phase to calculate or estimate the pile end-bearing capacity. The Torque Correlation Method is generally used to confirm or verify pile capacity during field installation. Foundation Supportworks engineers promote 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 1A*.

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, visit the Introduction section of this manual and the Foundation Supportworks website: *www.OnStableGround.com*.

1.11.1 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 service load of 30 kips in compression per pile with a factor of safety (FOS) = 2. Preliminary product selection suggests that the Model 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 1.57*.

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 Individual Bearing Method equation from *Section 1.7.1* for purely cohesive soils with Φ = 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) × FOS (2) = 60,000 lb
c = 3,000 lb/ft^2
$\mathbf{A_h}$ = 60,000 / (9)(3,000)
$\mathbf{A_h}$ = 2.22 ft^2

Helix plate areas for the various shaft sizes can be found in *Appendix 1A*. For the Model HP288 shaft (2.875-inch O.D.), a total helix plate area of 2.22 ft^2 can be most efficiently achieved with a 10/12/14 triple-helix plate configuration.

$\mathbf{A_{10"}}$ = 0.50 ft^2
$\mathbf{A_{12"}}$ = 0.74 ft^2

$A_{14"}$ = 1.02 ft^2

$\sum A_h$ = 2.26 ft^2

Solve for the ultimate and allowable pile capacities:

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

The allowable pile capacity,

$$Q_a = Q_u / FOS$$

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

Determine the required final installation torque in accordance with the equations and procedures of *Section 1.7.3*:

$$Q_u = K_t T$$

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 2.875-inch O.D. shaft, $K_t = 9 \text{ ft}^{-1}$:

$$T = 60{,}000 / 9 = 6{,}667 \text{ ft-lb}$$

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

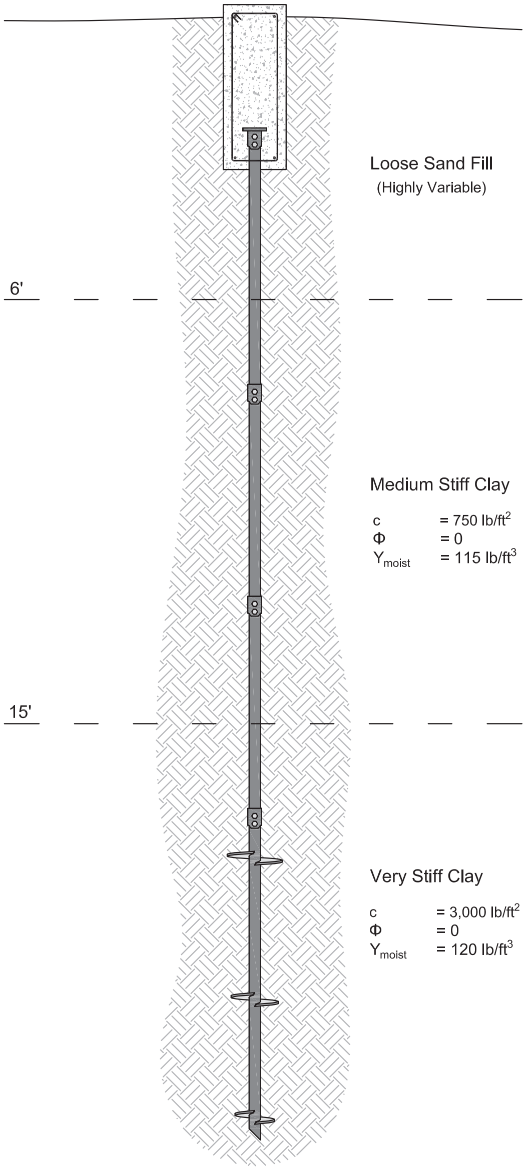


Figure 1.57 Example 1. Helical Pile Capacity

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 service 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 1.58*. Groundwater was encountered at a depth of 10 feet below the surface. Preliminary product selection suggests that the Model 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 torque-rated capacity of 61.3 kips. 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 1.58*.

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

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

Solve for the required helix plate area:

$$A_h = Q_u \,/\, q' N_q$$

The helix plates should be embedded several plate diameters into the dense sand to provide uplift resistance. This depth depends both on soil conditions and 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 into the dense sand, as measured to the uppermost plate. 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. In granular soils, helical pile capacity is dependent on the effective overburden stress (refer back to *Section 1.7*). 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) = 19.3 feet.

The 14-inch diameter helix plate is located at a depth of 16.5 feet. Torque correlation factors determined in accordance with ICC-ES AC358 consider embedment depths for tension loading applications of 12 plate diameters or more from the ground surface. Therefore, the 14-inch plate should be located at a depth of at least 14 feet. That criteria is met in this example. The design professional shall select an appropriate embedment depth and torque correlation factor when the 12D criteria is not met.

q' = $(110\ \text{lb/ft}^3)(10\ \text{ft}) + ((115-62.4)\ \text{lb/ft}^3)(3\ \text{ft}) + ((130-62.4)\ \text{lb/ft}^3)(6.3\ \text{ft}) = 1{,}683\ \text{lb/ft}^2$
Q_u = Service Load (40,000 lb) × FOS (2) = 80,000 lb
N_q = $1+0.56(12\Phi)^{\Phi/54} = 42.6$ (for $\Phi = 38°$)

And the minimum required helix plate area is:

A_h = $80{,}000 / (1{,}683)(42.6)$
A_h = $1.11\ \text{ft}^2$

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.

$\mathbf{A_{10''}}$ = 0.48 ft^2
$\mathbf{A_{12''}}$ = 0.72 ft^2

$\mathbf{\sum A_h}$ = 1.20 ft^2

Solve for the ultimate and allowable pile capacities:

$\mathbf{Q_u}$ = (1.20)(1,683)(42.6) = 86,000 lb
= 86 kips
$\mathbf{Q_{a,\ compression}}$ = 86,000 / 2 = 43,000 lb
= 43 kips…OK

To maintain the average vertical effective overburden stress at a depth of 19.3 feet, the 12-inch blade would be installed to a depth of 18.0 feet and the 10-inch blade would be installed to a depth of 20.5 feet. The upper helix plate is now 5.0 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.

The vertical effective overburden stress, $q'_{12''}$, at 18.0 feet:

$$q'_{12''} = (110\ \text{lb/ft}^3)(10\ \text{ft}) + ((115\text{-}62.4)\ \text{lb/ft}^3)(3\ \text{ft}) + ((130\text{-}62.4)\ \text{lb/ft}^3)(5.0\ \text{ft})$$
$$= 1{,}595\ \text{lb/ft}^2$$

A critical depth of 20 feet is used for this design. See *Section 1.7.1.1* for more information related to the critical depth. Since the 10-inch plate is below the critical depth, the vertical effective overburden stress, $q'_{10''}$, is equal to the effective stress at the critical depth.

$$q'_{10''} = (110\ \text{lb/ft}^3)(10\ \text{ft}) + ((115\text{-}62.4)\ \text{lb/ft}^3)(3\ \text{ft}) + ((130\text{-}62.4)\ \text{lb/ft}^3)(7\ \text{ft})$$
$$= 1{,}731\ \text{lb/ft}^2$$

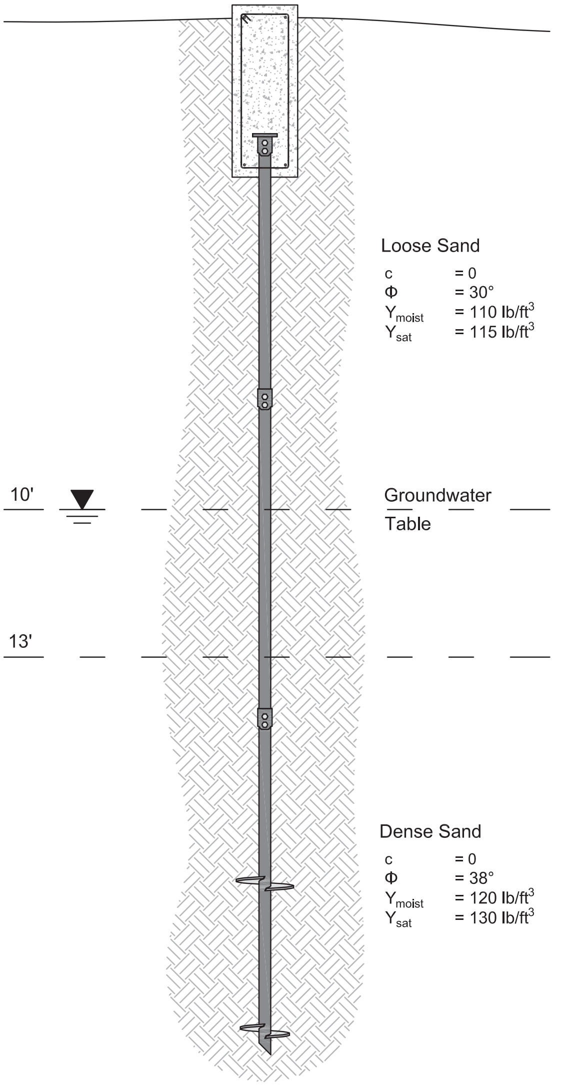


Figure 1.58 Example 2. Helical Pile Capacity

Solve for the ultimate and allowable pile capacities:

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

$\mathbf{Q_u} = (0.72)(1{,}595)(42.6) + (0.48)(1{,}731)(42.6) = 84{,}310 \text{ lb} = 84.3 \text{ kips}$

The allowable pile capacity:

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

$\mathbf{Q_{a,\,compression,\,uplift}} = 84.3 / 2 = 42.1$ kips… OK

Again, 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 above the 12-inch plate to represent the loose sand strata.

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

With the loose sand profile extending down to the 12-inch plate, the vertical effective overburden stress, $q'_{12"}$, at 18.0 feet is:

$\mathbf{q'_{12"}} = (110 \text{ lb/ft}^3)(10 \text{ ft}) + ((115\text{-}62.4) \text{ lb/ft}^3)(8 \text{ ft}) = 1{,}521 \text{ lb/ft}^2$

The vertical effective overburden stress, $q'_{10"}$, at 20.5 feet with a critical depth of 20 feet is:

$\mathbf{q'_{10"}} = (110 \text{ lb/ft}^3)(10 \text{ ft}) + ((115\text{-}62.4) \text{ lb/ft}^3)(8 \text{ ft}) + ((130\text{-}62.4) \text{ lb/ft}^3)(2 \text{ ft})$

$= 1{,}656 \text{ lb/ft}^2$

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

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

$\mathbf{Q_u} = (0.72)(1{,}521)(15.7) + (0.48)(1{,}656)(42.6) = 51{,}055 \text{ lb} = 51.0 \text{ kips}$

$\mathbf{Q_{a,uplift}} = 51.0 / 2 = 25.5$ kips…OK

Determine the required final installation torque in accordance with the equations and procedures of *Section 1.7.3*:

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

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 3.5-inch O.D. shaft, $K_t = 7 \text{ ft}^{-1}$:

$\mathbf{T} = 80{,}000 / 7 = 11{,}428$ ft-lb

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

Example 3

Helical tiebacks are being considered to stabilize an existing 12-inch thick reinforced concrete retaining wall. A geotechnical investigation found the retained soils to consist of silty sand. The design engineer proposed a Model HA150 shaft (1.5-inch solid square shaft) with a 10/12/14 helix plate configuration. The design length of the tieback was selected to meet minimum length criteria for the closest helix plate (14-inch plate) of 12D from the wall face and 5D from the assumed soil failure plane. Since the 12D criterion governed, the 14-inch helix plate was set approximately 14 feet from the wall face (longitudinally along the anchor). See *Section 1.8* for additional information regarding the design of helical tiebacks. The soil parameters and preliminary tieback design are shown on *Figure 1.59*. The engineer must determine the allowable tieback capacity so tieback spacing can be established.

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

$A_{14"} = 1.05\ \text{ft}^2$

$A_{12"} = 0.77\ \text{ft}^2$

$A_{10"} = 0.53\ \text{ft}^2$

$q'_{14"} = (120)(7.5) = 900\ \text{lb/ft}^2$

$q'_{12"} = (120)(8.0) = 960\ \text{lb/ft}^2$

$q'_{10"} = (120)(8.4) = 1{,}008\ \text{lb/ft}^2$

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

$Q_u = (1.05)(900)(15.7) + (0.77)(960)(15.7) + (0.53)(1{,}008)(15.7) = 34{,}800\ \text{lb} = 34.8\ \text{kips}$

$Q_a = 34.8 / 2\ (\text{FOS}) = 17.4\ \text{kips}$

The Model HA150 shaft has an allowable mechanical (galvanized corroded) capacity of 27.1 kips….OK

The horizontal and vertical components of the tieback force can be calculated in accordance with *Section 1.8.1*.

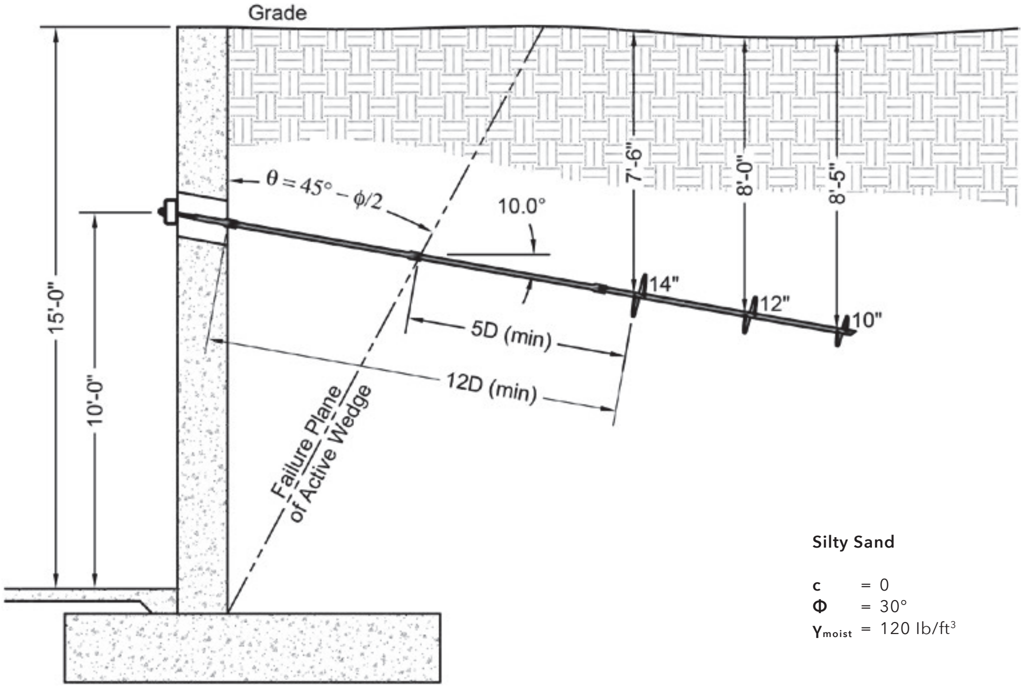


Figure 1.59 Example 3. Helical Tieback Capacity
(Failure plane origin varies based on project-specific parameters)

1.12 Installation

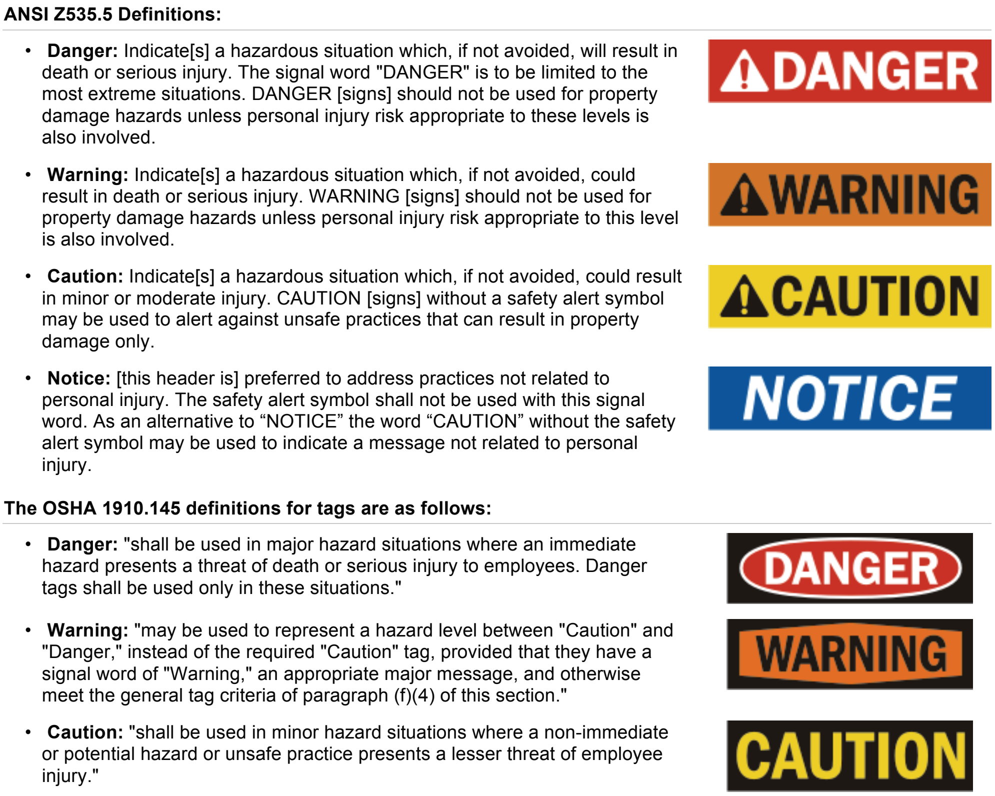
1.12.1 General Information

1.12.1.1 Preparation

All utilities, pipelines, cables, or any other service line or buried structure shall be identified and marked prior to any excavation or installation of helical piles, tiebacks and soil nails. 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 free of charge.

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 1.60*).



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 1.60 ANSI and OSHA Tag Definitions

1.12.1.2 Crowd

Axial force or "crowd" is applied to helical piles, tiebacks and soil nails during installation 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 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.

1.12.1.3 Penetration Rate

Helical piles, tiebacks and soil nails should ideally be advanced into the soil at a rate equal to the pitch of a properly formed, conforming helix plate per ICC-ES AC358, i.e., 3 inches per revolution. ICC-ES AC358 further states that 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 no more than 25 revolutions per minute (rpm) 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.

1.12.2 Equipment

1.12.2.1 Drive Heads

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. Foundation Supportworks engineers recommend 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 1.61*), mini-excavators (*Figure 1.62*), backhoes (*Figure 1.63*), and full-size excavators (*Figure 1.64*). 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. Although conservative, a basic rule of thumb has been a pound of machine weight for each ft-lb of torque from the drive head. Machine weight and structural capacity become increasingly more important with greater output torque.

Figure 1.61 Skid steer

Figure 1.63 Backhoe

Figure 1.62 Mini-excavator

Figure 1.64 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.

Foundation Supportworks 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.

The drive head is mounted to the frame of the hand-held equipment (*Figure 1.65*) 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 1.66*) 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. Please contact the Foundation Supportworks Engineering Department with any questions regarding the rated capacities of hand-held equipment.

Figure 1.65 Hand-held equipment

Figure 1.66 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 restraint 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. Safe operation of the hand-held equipment also requires lateral restraint at the drive head.

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.

1.12.2.2 Installation Tooling

Installation tooling consists of the components that are attached in-line between the drive head and the helical pile, generally an output shaft adaptor and a product adaptor. The drive head output shaft is typically a hexagonal or square shape with measurements ranging from 2 inches to 4 inches across flats. The output shaft adaptor slides over and is pinned to the output shaft of the drive head (*Figure 1.67*).

Figure 1.67 Hexagonal output shaft adaptor

The flange plate of the output shaft 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, bent arm pins, or bolts (*Figure 1.68*). 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 1.69*).

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

Figure 1.68 Round shaft product adaptor

Figure 1.69 Square shaft adaptor

1.12.2.3 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 1.7.3*. 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 (*Figure 1.70* and *Figure 1.71*) can be used to measure the "pressure drop" across a hydraulic drive head. This method is based on the principle that the work output of the drive head is directly related to measurement of the pressure drop across the drive head 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 1.70 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 1.71 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 range from 50 psi to over 800 psi. Systems with high return line pressures may damage a hydraulic drive head. The installation of a "case drain" on the hydraulic drive head can prevent damage to the drive head 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 overestimation of applied torque if back pressure is underestimated or ignored all together.

Shear pin torque limiters are mechanical in-line devices consisting of two 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.

The **shaft twist method** is simply a visual observation of the shaft deformation or twist that occurs with square bar helical products (*Figure 1.72*) 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.

Figure 1.72 Shaft twist

Electronic torque transducers such as the Pro-Dig Intelli-Tork® are placed in line with the drive tooling. Torque is a true real time measurement that is generated continuously during the installation of a helical pile or tieback. The Intelli-Tork (*Figure 1.73* and *Figure 1.74*) measures the torque applied between two flanges and transmits the torque reading to a hand-held unit for display and logging.

Figure 1.73 Electronic torque transducer

Figure 1.74 Torque transducer logging real-time installation data on an iPhone

A built in torque sensor within the housing of the flanged instrument transfers data via Bluetooth wireless technology or via a wireless network signal which allows multiple smart devices in the vicinity to receive the information. The software provides a remote visual indication of the torque during the installation. The free app on smart devices 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.

1.12.3 Installation Guidelines

1.12.3.1 New Construction Helical Piles

Installing the Lead Section:

1. Align the lead section with the product adaptor and install the temporary hitch pins, bent arm pins or bolts.

2. Position the installation equipment and pile directly over the marked location.

3. Apply a small amount of crowd to seat the pile shaft tip into the soil.

4. Use a level or digital gauge to plumb or set the installation angle (batter) of the pile shaft.

5. 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 soil strength profile relative 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.

6. Terminate installation of the lead section before the product adaptor penetrates the soil.

7. Remove the hitch pins, bent arm pins or bolts and carefully disconnect and raise the drive head.

Installing Extension Sections:

1. 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 to align the bolt holes.

2. Install the coupler bolts taking care not to damage threads. Tighten nuts to a snug-tight condition (*Figure 1.75*).

3. Align the drive head and product adaptor over the extension shaft to allow for insertion of the hitch pins, bent arm pins or bolts.

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

Figure 1.75 Install coupler bolts

Termination of Installation:

1. Over the final 3 to 5 feet of installation, assuming depth and minimum torque requirements are being met, reduce the rotational speed to approximately 10 rpm while providing proper alignment and crowd. **Refer to the Model Specifications for Helical Pile Foundations located on the Foundation Supportworks commercial website (*www.OnStableGround.com*) for termination criteria when the minimum overall length or minimum torsional resistance is not met.**

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

3. Install new construction bracket as specified (*Figure 1.76*). For compression applications, the new construction bracket could theoretically be set on top of the pile without bolting or welding. However, 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 Foundation Supportworks engineers to maintain bolt hole size, location and spacing tolerances.

Figure 1.76 Install new construction bracket

4. 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.

Excavation:

1. 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.

2. The 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 1.77*).

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

Figure 1.77 Excavation at bracket

Preparation of Footing:

1. 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 1.78*). 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.

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

Figure 1.78 Footing preparation

Installation of Helical Pier:

1. Attach lead section to drive head and product adaptor.

2. Place the lead section's first blade under the footing and the shaft of the lead 1½ inches away from bottom edge of footing (*Figure 1.79*). This will allow alignment of the lead to the required angle of inclination specific for the bracket system used. 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.

3. Advance the lead section and extensions to the design depth and minimum torque requirements.

Figure 1.79 Alignment of lead

4. Record the differential pressure or torque readings at appropriate depth intervals. Although the final installation torque is arguably the most critical, it is good practice to record differential 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 differential pressure or torque for every lead section and extension.

5. If necessary, cut the last extension shaft to an elevation about 13 inches above bottom of footing.

6. 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:

1. Place the external sleeve through the bracket.
2. Lower the bracket and external sleeve assembly over the pier shaft with the bracket bearing plate facing away from the footing (*Figure 1.80*).
3. Rotate the bracket body 180 degrees toward the footing.
4. Raise the 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 1.81*).
5. Tap the external sleeve down until the top flange or flared end rests on the bracket.
6. Install the cap plate and all thread rods (or coil rods) and tighten the nuts to snug the bracket to the bottom of the footing (*Figure 1.82*).
7. Remove the bracket RAYser™, backfill and compact soil up to the bottom of the bracket.

Figure 1.80 Lower assembly over pile shaft

Figure 1.81 Bracket RAYser™

Figure 1.82 Install cap plate and rods

Load transfer and Lift:

Figure 1.83 Set hydraulic lift cylinders

1. Set the lift cylinders and apply load to project specifications. Discontinue if structure begins to lift prior to achieving the service load. Alternatively, load can be increased until the structure lifts and the desired elevation is met (*Figure 1.83*).
2. Lock off and transfer the load to the piers by tightening the nuts down to cap plate.
3. Remove the lifting hardware and hydraulics.
4. Complete the field installation logs.
5. Establish benchmarks (if required).

Backfill and cleanup:

Figure 1.84 Backfill and compact soil

1. Backfill holes or trenches with the excavated, on-site material or imported soil.
2. Follow proper backfill/compaction procedures and tamp in maximum 6 to 12-inch lifts depending upon type and weight of compaction equipment (*Figure 1.84*).
3. When possible, establish grades to allow positive surface drainage away from the structure.
4. 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.

1.12.3.3 Helical Tiebacks

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

1. Attach the lead section to the product adaptor and insert the hitch pin(s), bent arm pin(s) or bolt(s).

2. Elevate the lead section along with the drive head assembly and place the tip of the tieback at the marked location.

3. Establish the proper angle of inclination and align the lead section per the design specifications.

4. Provide minimal crowd to seat the lead shaft tip.

5. Install the lead section while maintaining the proper installation angle.

6. Connect the extension section to the lead and snug tighten the bolted connection(s) at coupler.

7. Continue adding extensions until embedment length and minimal torque requirements are met. **Refer to the Model Specifications for Helical Anchor Foundations located at the Foundation Supportworks commercial website (*www.OnStableGround.com*) for termination criteria when the minimum overall length or minimum torsional resistance is not met.**

8. Disconnect the installation equipment and install a threaded transition assembly or other termination device on the end of the extension.

9. Install threaded rod into the transition so the threads are fully engaged.

10. Place a wall plate, bracket or waler system over the threaded rod. Place a nut and beveled washer on the threaded rod and tighten to the bearing plate.

11. Pretension and lock off the tieback as recommended by the engineer of record (see *Section 1.8.1* for additional information related to pretensioning tiebacks). Some systems may allow the lock-off nut to be tightened to a predetermined torque which correlates to an axial tensile force. For larger projects, a calibrated hydraulic cylinder may be used to pretension the tieback to the required lock-off load.

12. Remove equipment and cut threaded rod.