# ENGINEER MANUAL

# Disclaimer

Ram Jack Systems Distribution, LLC ("RJSD") has written this manual solely for informational purposes to assist engineers, architects, and contractors in selecting appropriate pile specifications when developing deep foundation solutions. It discusses the design of Ram Jack® helical and driven steel piles and how professionals can utilize them as deep foundation elements to prevent, stabilize, or remediate foundation movement. Because piles interact with the soil, the manual offers an introduction to soil mechanics.

Since the design and application of helical and driven steel piles are dependent upon site-specific project conditions, including the geotechnical properties of soil, which can vary significantly between sampling points, it is up to the user, prior to installation, to seek independent engineering analysis and to verify compliance with professional standards and governmental rules and regulations.

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# PREFACE

The past 25 years have seen a tremendous growth in the use of helical piles and driven steel piles to underpin distressed foundations. Typically, the distress is caused by settlement of the soil supporting the foundation. Occasionally, it is caused by downslope movement of a foundation built on an unstable slope or by bowing-in of a basement wall due to lateral soil pressure from the outside. In some cases, the goal is simply to stabilize the foundation in its current state so that no further distress will occur. In others, remediation by returning the foundation as close to its original condition as possible is desired. More recently, foundations utilizing helical piles have been recognized as an excellent means of avoiding the distress in the first place. Ram Jack Systems Distribution has been providing helical and driven steel piles for these uses for more than 45 years.

Over the years, Ram Jack has developed an extensive arsenal of underpinning brackets for almost every situation. In the rare case that a standard bracket will not meet the project requirements or loading situation, Ram Jack's engineering department will work with the design engineer to find an appropriate solution and provide engineering calculations and details for their review.

The purpose of this book is to assist readers in understanding what Ram Jack helical and driven steel piles are, and how they can be utilized as deep foundation elements to preclude, stabilize, or remediate foundation movement. This manual will serve as a design guide for engineers and architects as well as assist contractors to select appropriate pile specifications for bidding purposes.

Because these piles interact with soil, a basic understanding of soil mechanics is necessary to understand how they work. Therefore, the fundamentals of soil mechanics and soil/structure interaction concepts governing the design and application of Ram Jack foundation repair products have been included in this manual.

Darin Willis, P.E.
*Director of Engineering,*
*Ram Jack Systems Distribution, LLC*

# INTRODUCTION

## Ram Jack® History 1.1

The history of the Gregory legacy begins with Bill Gregory. In 1968, Bill started a small pest control company in Ada, Oklahoma, Gregory Pest Control. His son, Steven D. Gregory, took control of the family business in the '70s and later changed the name to Gregory Enterprises, Inc. Steve incorporated foundation repair into the pest control business with the major revenue stream coming from repairing settling foundations. The innovative Ram Jack System was later developed due to unreliable methods of foundation repair at the time. We committed resources to the development of a viable system for stabilizing foundations and, as a result, the first Ram Jack steel pile was born. The first of many patent-filings began in 1985.

The carefully designed Ram Jack System offers a strong, reliable method for supporting residential and commercial foundations using unique brackets for driving piles and related foundation support products, manufactured from the finest grade American-made steel.

The success of Ram Jack stems from a commitment to providing:

- A quality-first system for repairing foundations
- Customer satisfaction

Engineers around the world have found Ram Jack's methods and engineering offer versatility and reliability. Ram Jack's staff of engineers are dedicated to answering questions and providing assistance in solving foundation issues. Additionally, Ram Jack continues to innovate and has played an integral part in the construction of the


**FIGURE 1-1 :** Ram Jack Employees at Ram Jack University 2013, Oklahoma largest solar field. Ram Jack continues to offer strong, reliable foundation solutions for projects relating to alternative energy.

**FIGURE 1-2 : Ram Jack Headquarters**

Ram Jack products are developed, manufactured, and distributed from the Ram Jack Manufacturing/ Distribution facilities located in Ada, OK. Strict quality control standards, exact specifications, and unwavering installation guidelines ensure that the Ram Jack System and products have virtually no failure when properly installed.

In 1999, Ram Jack started building a network of highly competent dealers to offer the Ram Jack System through its affiliate, Ram Jack Systems Distribution. Today, there are over 60 locations nationwide as well as internationally which meet the needs of consumers, in both residential and commercial applications. From your house to the largest solar field in the world, Ram Jack's commitment to quality first and customer satisfaction keeps Ram Jack at the forefront of the foundation repair industry.

Our employees are highly trained in the proper design and installation of helical and hydraulically driven piles. Ram Jack conducts training classes for its employees every year at its headquarters in Ada, OK. Franchisees from all over the world participate in Ram Jack University. (Figure 1-1)

## Why Ram Jack? 1.2

Ram Jack piles are used extensively to underpin and support new construction and existing residential buildings, commercial buildings, industrial buildings, bridges, solar fields, wind turbines, sea walls, and many other types of structures. Ram Jack also custom-designs piles and brackets for non-conventional applications which make it possible to complete most challenging projects on time and within budget. Please visit www.ramjack.com and browse through the case studies which showcase the versatility of Ram Jack's system and a wide variety of applications. You may also view a sample of our case studies in Section II of this technical manual.

One of the main ingredients of Ram Jack's success and sustained growth over the last 45 years has been our core motto:

**"Anticipate, Innovate, and Exceed Customer Needs"**

We understand in the service industry that reputation is everything. Not all foundation companies or the systems they use are equal. Underpinning a foundation whether it's for new construction or remedial repair is expensive. It's something no client wants to do twice.

Quality, track record, and reliability set Ram Jack apart from the competition.

### EXPERIENCE 1.2.1

Ram Jack has been in business for more than 45 years, is still growing at a steady pace, and has evolved as one of the market leaders in the helical pile industry. In the past 45 years, Ram Jack has earned thousands of satisfied customers and has a proven track record of excellence and quality.

**FIGURE 1-3 : Ram Jack Manufacturing Employees**

From the permafrost of Alaska's tundra to the rain forests of Central America and everything in between, Ram Jack's driven and helical pile systems have been used to underpin and raise structures in all types of soil conditions and environments. The knowledge gained from these experiences allows Ram Jack to provide the most economical solution that will meet the client's needs.

## INNOVATION 1.2.2

Since the beginning Ram Jack has invested in innovation and technology. Ram Jack currently owns 12 patents. New products and equipment are developed and tested at Ram Jack's Research & Development Division to meet the growing demands of steel piles both domestically and internationally.

Ram Jack has more than 150 different products to fit almost any foundation and structure. Most of our competitors only offer a few different brackets and pile diameters. They require the engineer to adapt the

**FIGURE 1-4 : Ram Jack Robotic Welder**

foundation of the structure to their bracket and pile. Ninety percent of the time Ram Jack has a bracket that will fit the needs. In the rare instances where a bracket does not satisfy the required criteria, Ram Jack engineers can custom-design a bracket and pile to fit the project needs.

As an example, Abengoa Solar contacted Ram Jack in 2010 to custom-design a deep foundation system to support solar panels in Gila Bend, AZ. The Solana Solar field was to be the largest in the world at the time producing 280 MW [380694 hp] of electricity and covering 1,920 acres (3 mi$^2$ / 7.8 km$^2$). A little over 71,000 piles would be required to support the solar panel frames.

The design criterion required a maximum placement tolerance and deflection in any direction due to wind, seismic (design category D), and gravity forces be limited to 0.157 in. [4 mm]. Extensive preproduction load tests were performed for design verification and more than 350 post-installation load tests were performed for quality assurance. The foundations also had to be installed within a 13-month time-frame with liquidated damages being applied. This would require a minimum of 300 piles to be installed per day. Once the installation crews had ramped up to full production, they were installing up to 680 piles per day. The piles were installed two months ahead of schedule. Figure 1-5 shows an aerial photo of the Solana Solar field taken on August 18, 2013.

**FIGURE 1-5 : Aerial Photo of Solana Solar Field (Taken On August 18, 2013)**

## ENGINEERING 1.2.3

Ram Jack has a highly qualified engineering team located in Dallas, TX, consisting of structural and geotechnical engineers. Darin Willis, P.E., is the Director

**FIGURE 1-6 : Darin Willis, P.E. in the Field**

of Engineering for Ram Jack Systems Distribution, LLC. Darin has 30 years of experience in designing, developing, installing, and testing hydraulically driven steel and helical piles. He has designed foundation underpinning systems for residential, commercial, and industrial projects throughout the United States, Canada, Puerto Rico, and Central America.

Darin began working for Ram Jack in Ada, OK, in 1984 but left Ram Jack in 2000 to focus more on the structural design of commercial and industrial structures. His structural design experience includes tilt wall, steel-framed, concrete, masonry, and wood structures. He has designed structures up to 850,000 ft$^2$ [78967 m$^2$](Rooms To Go Distribution Center, Arlington, TX) to a 10-story dormitory for the Dallas Theological Seminary. Darin returned to Ram Jack as the Director of Engineering in 2007.

The engineering team is available to provide technical and pile design assistance to engineers, architects, contractors, and to answer any questions regarding Ram Jack piles and components which includes:

- Custom pile and bracket designs
- Pile calculation packages

- Shop drawings of products
- Helical and hydraulically driven steel piling design seminars
- Consulting
- Any technical assistance concerning Ram Jack products

## BUILDING CODE COMPLIANCE 1.2.4

AC358 is a comprehensive acceptance criterion for helical foundation systems. The International Code Council (ICC) adopted AC358 in June 2007. As of this publication, only six helical pile manufacturers have passed the strict criterion. Ram Jack was the first helical manufacturer to have its products recognized by ICC per AC358. The criterion requires third-party testing by an IAS accredited lab and extensive engineering calculations performed to show compliance with the current IBC code. In order to maintain an ESR report, a manufacturer must have quarterly inspections by an ICC certified inspector to verify the materials, workmanship, and quality of the products. The ESR will outline the capacities of the system, prove compliance, state any limit-states the system wasn't evaluated for, how the system is to be installed and inspected. Without an ESR report, engineers and building officials must rely on manufacturer-provided capacity and strength data which may not be accurate or even code compliant.

- Ram Jack has the only driven pile recognized by ICC-ES as compliant with the International Building Code (IBC).
- Ram Jack has more helical pile applications and brackets listed in its ESR report than any other manufacturer.
- The products listed in ESR-1854 comply with:
    - 2006, 2009, 2012, and 2015 IBC
    - 2010 Florida Building Code
    - 2014 Los Angeles Building Code (RR-25909)

## RAM JACK INSTALLERS 1.2.5

Unlike most helical manufacturers, Ram Jack does not sell their products to the general public. Over the last 45 years, we have learned many things. One of the most important lessons is that underpinning the foundation of a structure is as much of an art as it is a science. In order to raise a structure one quarter of an inch or eight feet and stabilize it takes a special skill-set that not everyone has. The contractors who install helical piles are almost as important as the helical piles themselves. Some helical manufacturers sell their piles to the general public who are often untrained in installation procedures, mechanics, and engineering principles of the helical system.

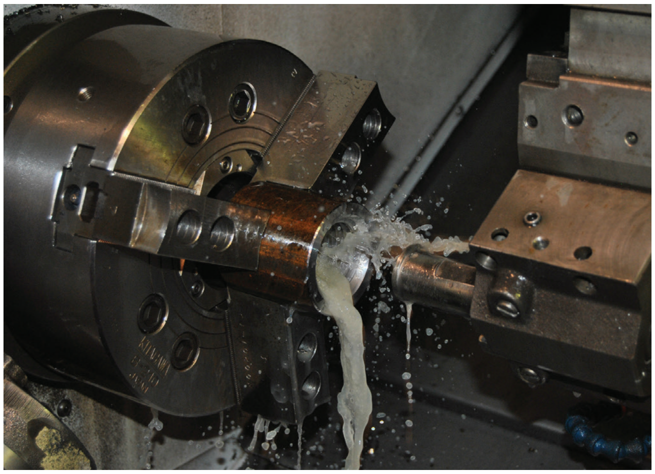


**FIGURE 1-7 : Machining Threaded Connections**

A manufacturer can fabricate the perfect product but if it's installed incorrectly, it won't work. It takes experience and training to install helical piles. Ram Jack only sells helical piling material and equipment to approved and trained Dealers and Franchises who are carefully vetted for their honesty and integrity. Our installers must also pass a rigorous written exam and field installation tests in order to become a certified Ram Jack installer.

## RAM JACK MANUFACTURING 1.2.6

Ram Jack manufactures all of its helical and driven steel piling components. Ram Jack's state of the art manufacturing plant is located in Ada, OK. From this location, Ram Jack distributes its helical and driven pile components throughout North and Central America. The plant consists of a 90,000 $ft^2$ [8362 $m^2$] facility which contains the following areas:

- Machine Shop Division
- Saw Division
- Drill Division
- Cut Division
- Weld Division
- Corrosion Protection Division
- Hydraulics Division
- Research and Development Division
- Shipping Division

At Ram Jack Manufacturing, quality assurance begins long before the production process starts. In fact, 100% of all product designs are reviewed by our engineering department and extensively tested by our Research and Development Division. Only after a potential product is fully tested and approved by our engineering department does it become introduced as a production item.

Ram Jack Manufacturing maintains strict quality control from start to finish. If you ever visit the plant, you will see our core values clearly displayed throughout the facility:

- Quality First
- Safety Always
- Complete Integrity
- On-time Delivery

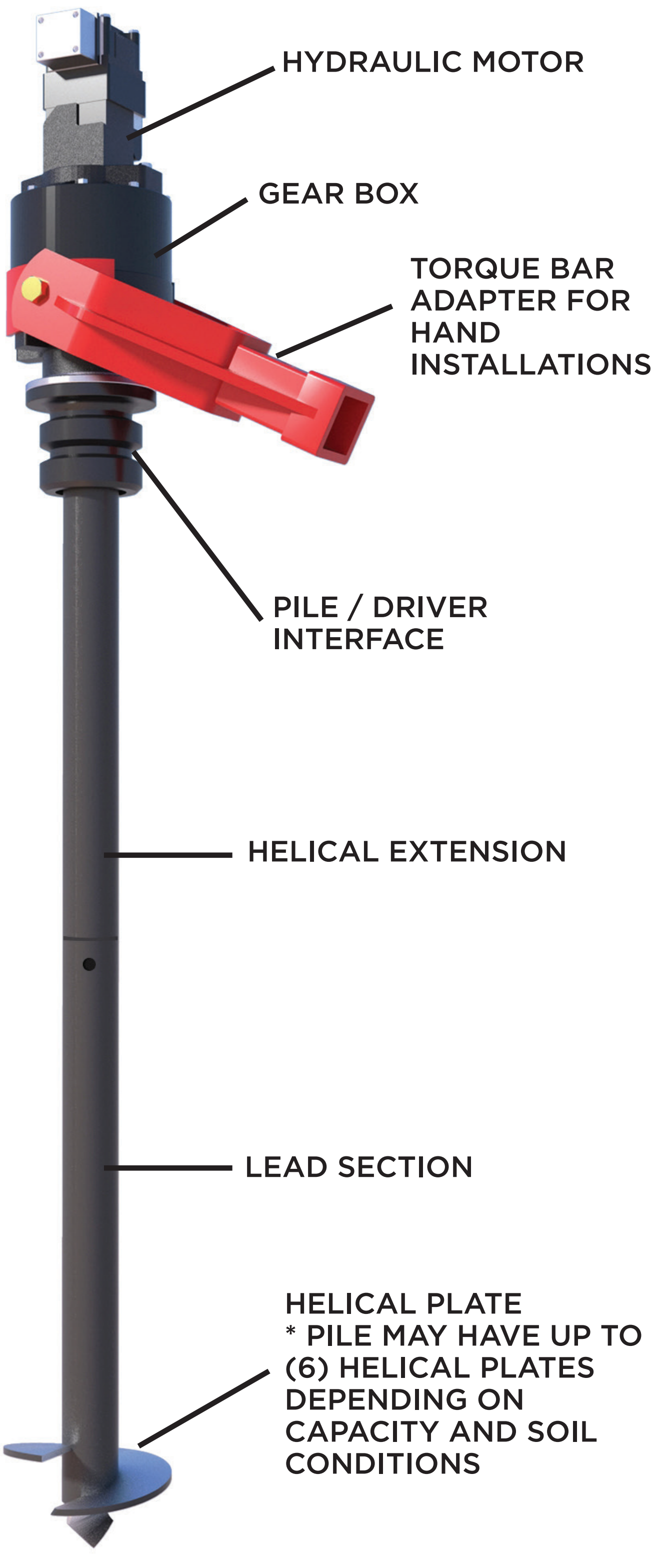
Since 2008 our manufacturing plant has had quarterly inspections by an independent ICC certified inspector for quality control and assurance to maintain our ESR Report. Ram Jack Manufacturing is also ISO 9001:2008 certified and an approved licensed fabricator by the City of Los Angeles building department.

## CORROSION PROTECTION 1.2.7

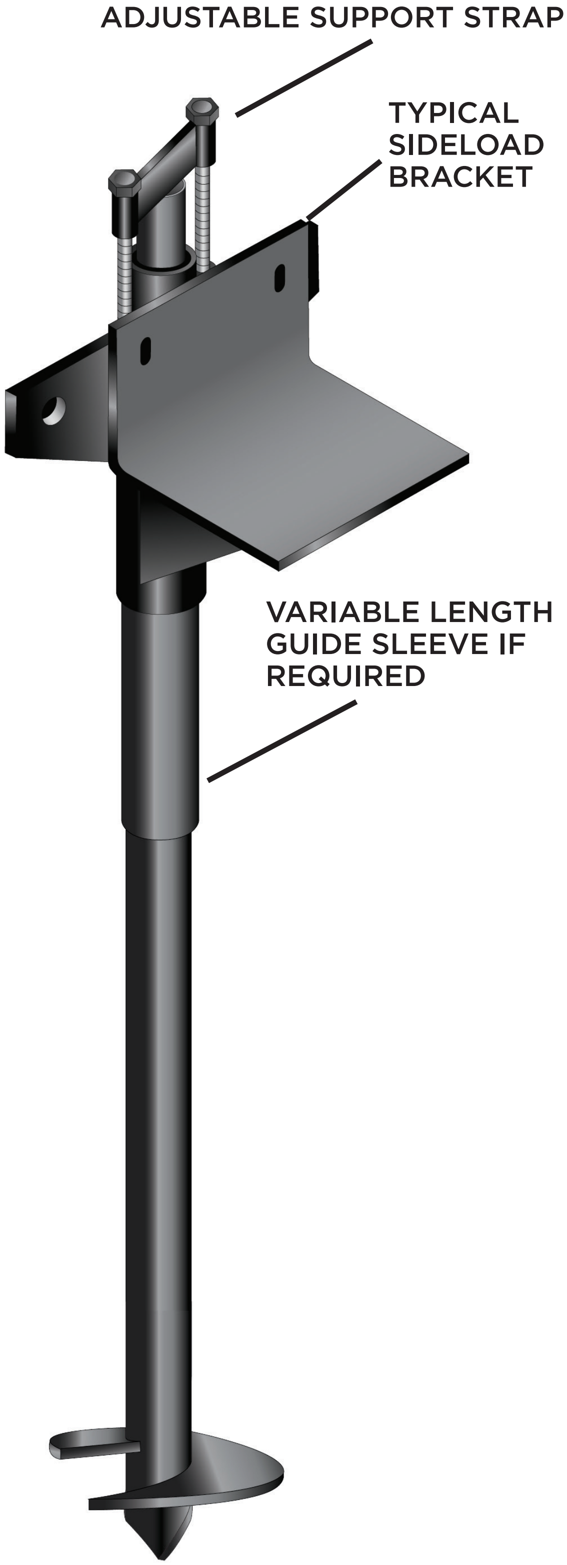
Ram Jack's standard corrosion protection on all of its products consists of a thermoplastic polymer powder coating. The powder coating is in compliance with the ICC adopted "*Acceptance Criteria for Corrosion Protection of Steel Foundation Systems Using Polymer Coating*" (AC228). The powder coating is long-lasting and environmentally friendly since it doesn't emit volatile compounds into the soil or ground water.

**FIGURE 1-8 :** Corrosion Protection

## TYPICAL HELICAL PILE SYSTEM

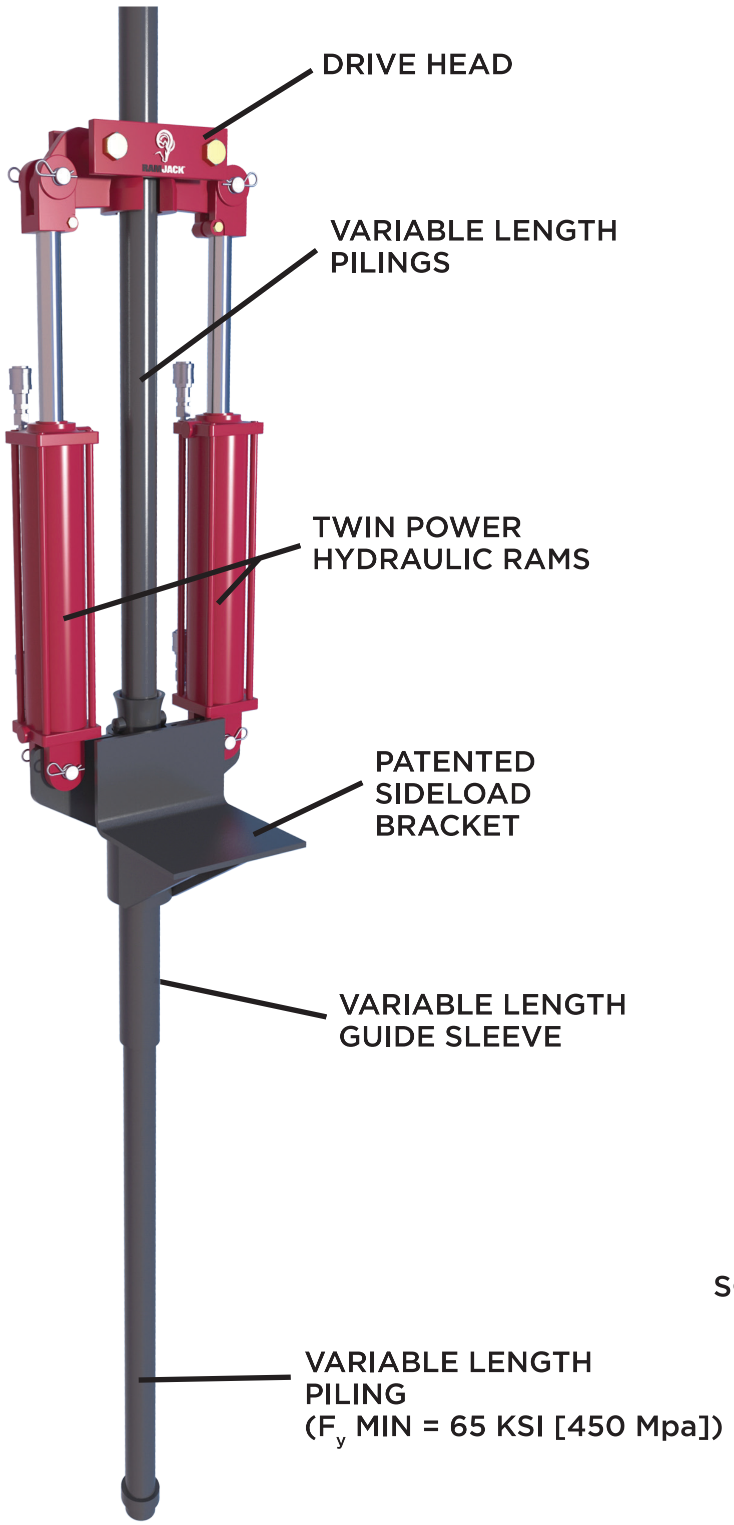



**INSTALLATION PROCESS**




**INSTALLED PILE**

## HYDRAULICALLY DRIVEN PILE SYSTEM




**INSTALLATION PROCESS**


**INSTALLED PILE**

Even though galvanized pile components are not recommended by Ram Jack since it is a sacrificial coating and can introduce zinc into the ground water system, the coating is allowed by AC358. Therefore, Ram Jack can supply galvanized-coated materials when specified by the engineer of record in his specifications.

# 2: BASIC SOIL MECHANICS

SAM ROSENBERK, PH.D., P.E.
SENIOR ENGINEER

## Introduction 2.1

Soil refers to the unconsolidated, granular product of the physical or chemical weathering of rock. It consists of discrete solid particles with gases or liquids filling the spaces between. Soil mechanics is concerned with the behavior and performance of soil as a construction material or as a support for engineered construction (Bowles, 1979).

Because soil is a natural material, its properties vary greatly in comparison to man-made construction materials such as steel and concrete. Because of the great bulk of soil typically involved in construction, it is often not practical either to transport soil over appreciable distances or to improve the soil's properties in-situ. So, rather than specifying a particular minimum value for a physical property of interest (as, for instance, the tensile strength of steel or compressive strength of concrete) and then obtaining material per the specification to use in construction, the engineer is faced with first determining as best as possible what the soil's properties are and then designing a structure to interact with the soil and produce the required results.

## Soil Classification 2.2

There are two major classification systems that are still in use: the American Association of State Highway and Transportation Officials (AASHTO) and the Unified Soil Classification System (USCS). Of these, one of the most widely used is the Unified Soil Classification System in a majority of geotechnical applications. Soil can be broadly classified into three major divisions as coarse grained, fine grained, and highly organic soils.

In the USCS, soils are first divided according to their organic content, with highly organic soils being labeled as peat (Pt). All other soils are grouped according to their prevalent grain size, coarse or fine, with the dividing line being the No. 200 sieve, or 0.06 mm grain size.

Coarse-grained soils are further subdivided into gravel and gravelly soils (G) vs. sand and sandy soils (S). These groups are subdivided into well graded, clean (W); well graded with excellent clay binder (C); poorly graded, fairly clean (P); and containing fine materials not covered in other groups (F).

Fine-grained soils are divided into three types: inorganic silts (M), inorganic clays (C), and organic silts and clays (O). These types are further subdivided into those of high (H) vs. low (L) compressibility and plasticity. The different symbols used in the USCS system are as summarized in Table 2-1.

The various soil classes are defined by soil type and sub-group and designated by combining the respective letter designations, as, for instance, SW (well graded sand) or CH (inorganic clay of high plasticity and compressibility).

One weakness of this system is the tests necessary to determine classification destroy the original soil

| Symbol | G | S | M | C | O | Pt | H | L | W | P |
|---|---|---|---|---|---|---|---|---|---|---|
| Description | Gravel | Sand | Silt | Clay | Organic Silts and Clay | Peat and Highly organic soils | High Plasticity | Low Plasticity | Well Graded | Poor Graded |

**TABLE 2-1 : Symbols Used for Identification in the Unified System**

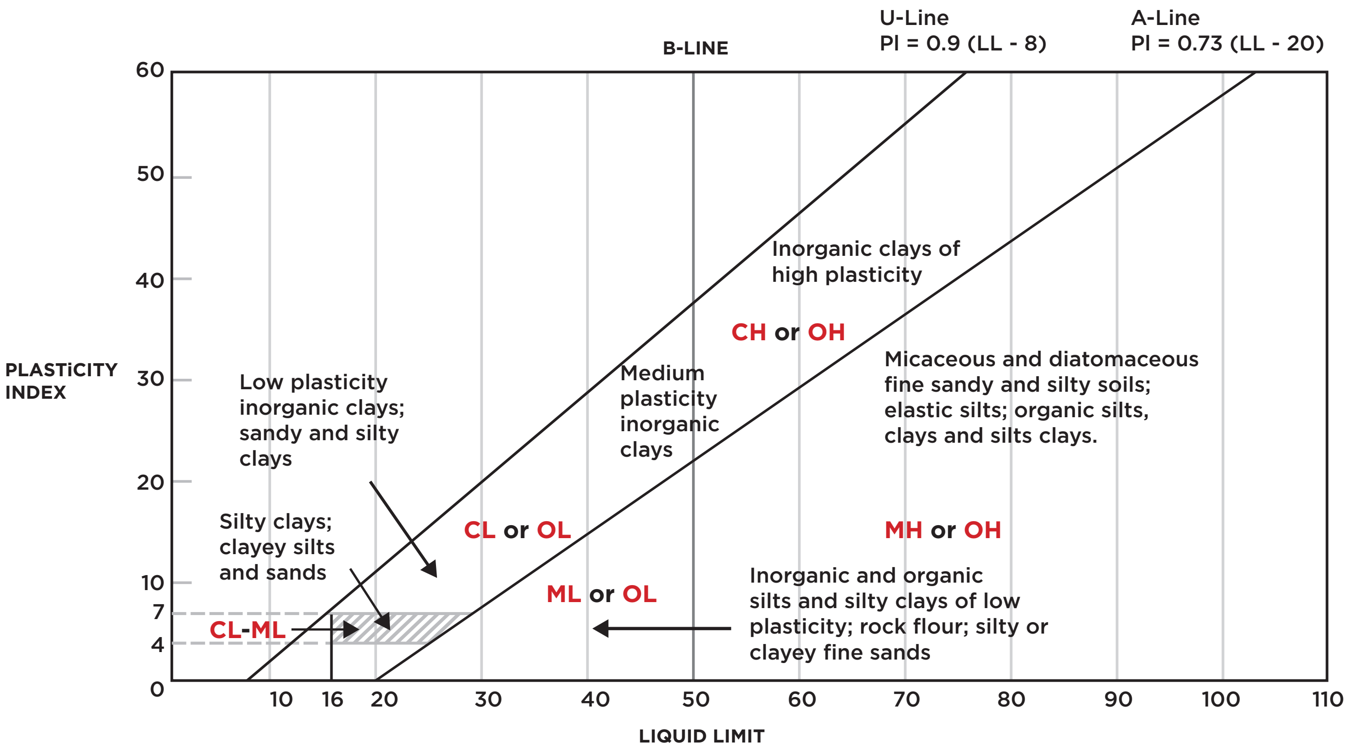



**FIGURE 2-1 :** Plasticity Chart

structure, thus the classification gives no information about this important characteristic. This weakness can be alleviated to some extent by describing the soil in accordance with its origin and method of deposition. Some terms used in this regard (Means and Parcher, 1963):

**Residual:**
A soil that was formed by weathering of the parent rock and still occupies the position of the rock from which it was formed.

**Transported:**
Any soil that has been transported from its place of origin by wind, water, ice, or other agency and then re-deposited. Transported soils are further classified according to the transporting agency and method of deposition as:

- Alluvial Soils that have been deposited from suspension in running water.
- Lacustrine Soils that have been deposited from suspension in quiet fresh water lakes.
- Marine Soils that have been deposited from suspension in salt water.
- Aeolian Soils that have been transported by wind.
- Glacial Soils that have been transported by ice.

## Soil Exploration 2.3

In order to select the type of foundation system, we need to know the soil conditions that exist at the project site. This can be done by performing a soil exploration program. The extent of the soil exploration program would depend on factors such as the sensitivity and nature of the proposed structure, budget, feasibility, and uniformity of the project site. Typically, a soils exploration program consists of test pits, grab/hand samples, geotechnical borings, disturbed/undisturbed sampling, in-situ field tests, etc. The purpose of the exploration program is to provide a generalized cross-section view of the soil profile existing at the respective site. The term soil profile typically refers to a vertical section through the soil layers showing the thickness and type of material encountered. The tests and sampling that are performed during the exploration program help to determine the physical and mechanical properties of the different soil layers that are encountered.

### SOIL BORING 2.3.1

The most common boring methods that are typically used currently are wash boring, rotary drilling, and auger drilling. Wash boring uses a casing, weight for driving the casing, wash pipe, and cutting edge. The cuttings are flushed out through the space between the casing and the wash pipe to the surface. Rotary drilling is very similar to wash boring with the circulating fluid most often being drilling mud in rotary drilling compared to water in wash boring. Auger boring is typically performed by advancing or turning an auger into the ground for a particular depth and then is removed along with the soil. The color and appearance of the flushing spoils at the surface is constantly observed by the driller during wash boring and rotary drilling. Similarly, the appearance and consistency of the removed soil from the auger is observed by the driller. Once any change is observed a sampler is attached to the end of the drill rod and a sample is obtained at the respective depth for further analysis and classification.

### SOIL SAMPLING 2.3.2

There are two types of samples that can be retrieved during a soils exploration program: disturbed and undisturbed. Disturbed soil samples are typically retrieved by shovel, test pits, hand/machine-driven auger or grab samples. Undisturbed or a slightly disturbed representative sample is obtained by using a split-spoon sampler, thin-walled tube sampler, and piston sampler. Although the samples obtained through these methods are slightly disturbed they are considered to be representative samples on which tests such as consolidation, hydraulic conductivity, and shear strength tests are performed.

## Soil Testing 2.4

Many other test methods and instruments have been devised in efforts to reduce the time and cost associated with the laboratory tests described in the previous section. Most do not directly measure the soil shear strength, instead measuring some other parameter and relying on an empirical correlation to establish an estimate of the shear strength that would be found were one of the direct-measuring laboratory tests performed.

### POCKET PENETROMETER AND TORVANE 2.4.1

The most widely used field test instruments are the Pocket Penetrometer and the Torvane shear tester. Both of these instruments are used with fine-grained soil samples and calibrated to predict results that would be obtained from unconfined compression testing of the sample. They can also be used for in-situ testing in test pits or large-diameter borings allowing personal access. The Pocket Penetrometer has a spring-loaded plunger that is pushed into the surface of a sample a calibrated distance. The deflection of the spring at the marked penetration distance is calibrated to give the estimated unconfined compressive strength of the sample.

The Torvane tester has a shaft with several vanes that is pushed into the sample surface a calibrated distance. Rotation of the shaft is restrained by a torsion spring. In use, the vanes are pushed into the sample and the head is then turned until the vanes shear out a plug of soil. The maximum torsional deflection of the spring is calibrated to give the estimated shear strength (cohesion) of the sample. Torvane results typically exceed unconfined compression results.

It should be noted that the shear strength results estimated using the pocket penetrometer are highly unreliable since the instrument is typically not calibrated by the manufacturer. Each penetrometer would have to be calibrated by the user based on the amount of data available. Therefore, the results would then rely on the accuracy of the available data and human error while obtaining the penetrometer readings. Further, the piston in the penetrometer has an approximate tip area of only 0.05 in$^2$ [32 mm$^2$] which is inserted into the soil sample. Since the tip area is relatively small there is significant potential for human errors and difficulty in calibration.

## STANDARD PENETRATION TEST 2.4.2

The most popular in-situ test is the Standard Penetration Test (SPT) as prescribed by ASTM D1586. It consists of:

1. Augering to test depth
2. Driving a thick-walled tubular sampler called a split-spoon into the bottom of the hole. The sampler (Figure 2-2) is driven by blows from a 140 lb. [63.5 kg] hammer dropped 30 in. [760 mm] onto the top end of a drive rod attached to the sampler. The sampler is driven a total of 18 or 24 in. [457 or 610 mm], at the driller's discretion.
3. Recording the number of hammer blows required to drive the sampler each successive 6-in. [150 mm] embedment increment.

The first 6 in. [150 mm] of embedment is considered seating of the sampler. The next 12 in. [300 mm] constitute the actual test; the sum of the blows required to drive the sampler these two increments is termed the standard penetration number, standard penetration resistance blow count, or N-value. SPT tests are most commonly taken in intervals of 5 ft. [1.52 m] or at changes in soil stratum.

Blow counts can be affected by many factors not related to soil properties. Studies by various investigators have led to the overall conclusion that the test is "difficult to reproduce" (Bowles, 1982). Though, it continues to enjoy widespread use due to its relatively low cost, the long service life of the existing test equipment, and the existence of an extensive experience database enabling successful design of many types of geotechnical works from SPT data.

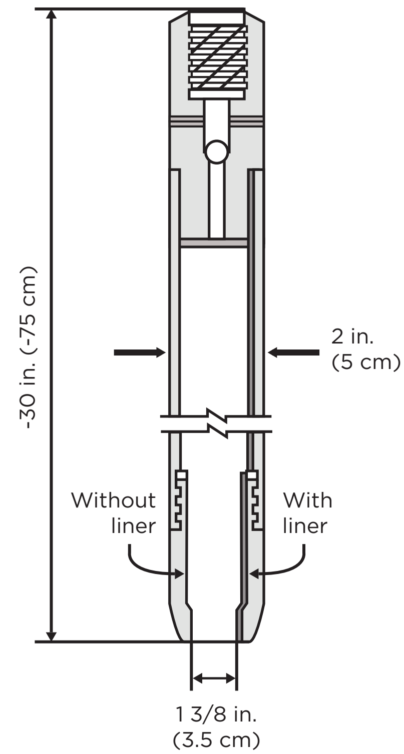



**FIGURE 2-2 : Standard Split-Spoon Sampler**

Assuming good workmanship and equipment, the most significant factors affecting reproducibility of the N-values are soil type, density, and overburden pressure (Bowles, 1982). Various investigators have recommended correcting observed blow count N-values for effective overburden pressure different from standard as

$$N' = C_N N \qquad (2.1)$$

| Blow Count, N (bpf) [blows/300 mm] | Consistency | Angle of Internal Friction $\phi$ (deg) | Moist Unit Weight $\gamma_m$ (pcf) | Moist Unit Weight $\gamma_m$ (kN/m³) | Submerged Unit Weight $\gamma_{sub}$ (pcf) | Submerged Unit Weight $\gamma_{sub}$ (kN/m³) |
|---|---|---|---|---|---|---|
| 0 - 4 | Very Loose | < 28 | < 100 | < 15.7 | < 60 | < 9.42 |
| 5 - 10 | Loose | 28 – 30 | 95 – 125 | 14.9 - 19.6 | 55 – 65 | 8.63 - 10.2 |
| 11 - 30 | Medium | 31 – 36 | 110 – 130 | 17.3 - 20.4 | 60 – 70 | 9.42 - 11.0 |
| 31 - 50 | Dense | 37 – 41 | 110 – 140 | 17.3 - 22.0 | 65 – 85 | 10.2 - 13.6 |
| > 50 | Very Dense | > 41 | > 130 | > 20.4 | > 75 | > 11.8 |

**TABLE 2-2 : Coarse-Grained Soil Parameters vs. SPT Blow Count (Teng, 1962)**

Where $N'$ is the corrected blow count and $C_N$ is a correction factor for overburden pressure for dense, very fine, or silty, saturated sand. It is further recommended that observed blow counts greater than 15 be corrected to account for increased resistance from excess pore pressure built up as the split-spoon sampler is advanced.

Terzaghi and Peck, 1948 recommended

$$N'=15 + 0.5(N-15) \qquad (2.2)$$

Whereas Bazaraa, 1967 recommended:

$$N'=0.6N \text{ for } N>15 \qquad (2.3)$$

Table 2-2 shows approximate relationships between blow count $N$, compactness, angle of internal friction $\phi$ under unconsolidated, drained test conditions, and unit weights in moist $\gamma_m$ and submerged $\gamma_{sub}$ condition for coarse-grained soil. Some general observations to consider when using this table are:

The minimum and maximum values of observed unit weights increase with increasing grain size and decrease with increasing grain size uniformity. The minimum and maximum values of observed angle of internal friction increase with increasing grain size and decrease with increasing grain size uniformity. Lower angles of internal friction are associated with more rounded grains, higher angles of internal friction with more angular grains. Tables such as this should always be used with caution. The geotechnical engineer should verify the appropriate angle of internal friction $\phi$ to use in the design whenever possible.

Table 2-3 shows approximate relationships between blow count N-values, consistency, unconfined compressive strength $q_u$ and saturated unit weight $\gamma_{sat}$ for fine-grained soil. Some general observations to consider when using this table are:

Correlations between SPT and unconfined compressive strength are poor due to inadvertent changes in water content, excess pore pressures during sampling, and other factors. Hence, it is preferable to measure unconfined compressive strength directly in the lab.

The undrained shear strength (cohesion) is usually expressed by a cohesion intercept only, and may be taken as $q_u/2$.

The correlation between the standard penetration test results and unconfined compressive strength in Table 2-3 is adequately represented by $q_u = N/4$, ksf [kN/m²].

| Blow Count, N (bpf) [blows/300 mm] | Consistency | Unconfined Compressive Strength $q_u$, (psf) | Unconfined Compressive Strength $q_u$, (kN/m²) | Saturated Unit Weight $\gamma_{sat}$ (pcf) | Saturated Unit Weight $\gamma_{sat}$ (kN/m³) |
|---|---|---|---|---|---|
| 0 - 2 | Very Soft | 0 – 500 | 0 - 24.0 | < 100 – 110 | < 15.7 - 17.3 |
| 3 - 4 | Soft | 500 – 1,000 | 24.0 - 47.9 | 100 – 120 | 15.7 - 18.9 |
| 5 - 8 | Medium | 1,000 – 2,000 | 47.9 - 95.8 | 110 – 125 | 17.3 - 19.6 |
| 9 - 16 | Stiff | 2,000 – 4,000 | 95.8 - 192 | 115 – 130 | 18.1 - 20.4 |
| 17 - 32 | Very Stiff | 4,000 – 8,000 | 192 - 383 | 120 – 140 | 15.7 - 22.0 |
| > 32 | Hard | > 8,000 | > 383 | > 130 | > 20.4 |

**TABLE 2-3 : Fine-Grained Soil Condition and Strength Parameters vs. SPT Blow Count (ASCE, 1996)**

Consequently, cohesion can be estimated as $c = N/8$, ksf [kN/m²].

Because sands exhibit no plasticity and silts exhibit very little, the plasticity index reflects the proportion of clay particles in the finest fraction of a soil. Sand, silt, and clay particles each have their own characteristic angles of internal friction, thus their relative proportions determine the average angle of shearing resistance in the overall sample.

## MODIFIED SPLIT-BARREL DRIVE SAMPLER 2.4.3

The modified split-barrel sampler has an external diameter of 3.0 in. [76.2 mm] and is lined with several 1 in. [25.4 mm] long, thin brass rings with an inside diameter of approximately 2.4 in. [61 mm]. The sampler is typically driven into the ground using the weight of the hammer of the drill rig in general accordance with ASTM D 3550-01. Typically the modified split-barrel sampler or the modified California sampler is driven using the same hammer weight and drop height as used in the standard penetration test procedure. However, due to the dimensional change in the sampler compared to the standard penetration test sampler, the amount of energy delivered varies. A number of empirical equations are available to convert the blow counts obtained using the modified California sampler to the SPT N-value such as:

Burmister, 1948

$$N=0.65\,N' \qquad (2.4)$$

Lacroix & Horn, 1973.

$$N=0.44\,N' \qquad (2.5)$$

where,

N' = measured blow count value

N = Standard Penetration Number equivalent blow count value

Based on these correlations an average conversion factor of 0.56 may be used to convert the blow count obtained using the modified California sampler to the SPT N-values.

### CONE PENETRATION TEST 2.4.4

The static cone penetration test (CPT) is generally recognized as technologically superior to the SPT. One of the greatest advantages is the ability to provide an accurate continuous profile of soil stratification. With the electric cone, tip resistance and skin friction can be measured separately, and with the piezocone, pore pressures near the tip can be measured. Soil types can be identified by correlation with the tip resistance and tip-resistance-to-friction ratio, and the visual cone provides a running video of the soil that is being tested.

No drilling is required for the CPT test since the penetrometer is simply pushed through the soil strata. Mechanical penetrometers are pushed to the test depth, then the head is advanced hydraulically and the resulting resistance measured. With the later electric cones, the penetrometer is advanced at a constant slow rate and the various parameters (dependent on the type of cone utilized) are measured and recorded continuously.

Two design approaches are then available. In the first, direct correlations are used to predict foundation response based on the cone penetrometer data itself. This approach is often used with driven piles. In the second, the cone penetrometer data is used to infer soil strength parameters, which are then used to predict foundation performance the same as would be done with laboratory or other in-situ test data. Typically, equivalent Standard Penetration Test (SPT) N-values are provided in the CPT logs. These Standard Penetration Numbers can then be correlated to soil strength properties and used to estimate the bearing capacity of the soil.

## The Soil-Water System 2.5

Soil properties are affected by the presence or absence of water to varying degrees, depending on the soil type and the specific property of interest. Some terms related to these relationships are:

**Void Ratio,** [$e$]**:** Ratio of volume of voids to volume of solids in a soil mass.

**Porosity,** [$n$]**:** Ratio of volume of voids to the total volume of a soil mass.

**Water Content,** [$w$]**:** Ratio of the weight of water to the weight of solids in a soil mass.

**Saturation,** [$S$]**:** Ratio of the volume of water in a soil mass to the volume of voids.

Fine-grained soils are affected by water in ways that do not apply to coarse-grained soils. Clays in particular are composed of very small, flat particles having high negative surface charges. These negative surface charges attract cations (Na+, Ca++, etc.) present in the ground water. Because water molecules are polar (i.e., the positive charges are distributed differently than the negative charges) they also are attracted and held by the surface charges of the clay particles. This adsorbed water forms a highly viscous, asphalt-like continuum surrounding the soil particles. These electrochemical bonds tend to retard movement of the particles in relation to one another, and the attraction persists even when the particles do move in relation to one another.

The more distant a water molecule is from the surface of a clay particle, the less tightly it is held to the particle. At lower water contents, all the water present is tightly held, while at higher water contents some of the water is loosely held. Thus, the water content of clay greatly affects the ease with which the particles can be displaced relative to one another, i.e., the clay's shear strength. This effect may be greater or less depending on the specific mineral forming the clay particles.

Clay soil can exist in any of four states depending on its water content. Consider a suspension of clay in water that is allowed to settle and then dry out. In suspension, the clay particles are surrounded by closely-held "solid" adsorbed water about 10 Å thick (1 Å = $10^{-7}$ mm), a layer of viscous "double-layer" water some 200 to 400 Å thick, then free water. The particles are so far apart and separated by water of such low viscosity that the suspension acts completely like a liquid. That is, it has zero shear strength. If we now progressively remove water from this initial suspension, we will see the resultant mix begin to develop significant viscosity. Eventually, the mix becomes so thick that it starts developing persistent shear strength. The water content at which the soil exhibits shear strength of about 20 to 40 psf [0.96 to 1.92 kPa] is termed the liquid limit, *LL*. Actually, the operational definition of the liquid limit is based on a somewhat arcane test, the results of which correlate with shear strength. However, the physical interpretation of the liquid limit is the water content at which the soil exhibits specific, minimal shear strength.

The shear strength at the liquid limit is so little that the clay is easily molded with the hands. Further, this remolding is accomplished without the soil cracking or breaking apart. This ability to be remolded without cracking is termed plasticity. It is due to the persistence of the electrochemical bonds even in the face of large strains in the soil. As more and more water is removed, the clay becomes stiffer and also less tolerant of remolding. The water content at which the soil just begins to crumble when rolled into a ⅛ in. [3.18 mm] diameter thread is termed the plastic limit, $w_p$. A number of researchers have reported the undrained shear strength at the plastic limit as being 2 to 4 ksf [95.8 to 192 kPa].

Further drying results in continued increase in shear strength as the soil particles move closer and closer together. However, the tendency to break apart on attempted remolding also increases. Up to this point, as water was removed the soil particles moved closer together and the total volume decreased. This action continues until the shrinkage limit is reached. Soil that has dried beyond the shrinkage limit is quite hard, brittle, and experiences little volume change with changes in water content. The liquid limit, plastic limit, and shrinkage limit are among five water content limits which were defined by the Swedish chemist A. Atterberg about 1911. These limits are also known as the Atterberg Limits named after A. Atterberg. Tests to determine the limits are done on fine-grained particles (< 75 μm) only and necessarily involve remolding of this fraction at varying water contents.

## PLASTICITY INDEX (PI) 2.5.1

The difference between the liquid limit and the plastic limit is termed the plasticity index, *PI*. It is a measure of the range of water contents over which the soil acts as a reasonably plastic solid. The plasticity index could be used as a measure to predict the swelling potential (expansiveness) of soil. Cheng, 1988, presented a method to identify the swelling potential of the soil based off the plasticity index value as shown in Table 2-4.

| Swelling Potential | Plasticity Index (PI) |
|---|---|
| Low | 0 - 15 |
| Medium | 10 - 35 |
| High | 20 - 55 |
| Very High | 35 and above |

**TABLE 2-4 : Swelling Potential Based on Plasticity Index (PI), Cheng 1988**

Special care would have to be taken during the design of helical piles in expansive soils. High *PI* clayey soils could potentially develop uplift pressures along the pile shaft surface and helical plates resulting in heave. Therefore, the design helical pile embedment depth should always take into account the zone of seasonal moisture variation. Potential heave issues due to expansive soils can be mitigated by embedding the helical plates beyond the zone of seasonal moisture variation.

Soils with high to very high plasticity index values are also commonly termed as expansive soils. Based on prior soil exploratory programs and geological data, general areas of expansive soils have been mapped as shown in Figure 2-3. General information about the presence of any potential problems at the site is very useful at the time of pile installation.

### LIQUIDITY INDEX (LI) 2.5.2

The liquidity index, *LI* is the ratio of the difference between the natural water content and the plastic limit divided by the plasticity index.

$$I_L = \frac{w_n - w_p}{PI} \quad (2.6)$$

It gives an indication of the degree of plasticity to be expected of the soil at its natural water content. Even at points on the surface of the earth that appear to be dry, the degree of saturation, *S,* typically becomes 100% at some moderate depth. That is, all the void space is filled with water. The water then forms a continuous body capable of transmitting compressive stress (pressure). This continuous body of water is called the ground water table. Soil particles below the ground water table are subjected to buoyancy forces that reduce the contact forces between the soil grains. This effect has a strong influence on the unit shear strength of soils.

## Helical Selection Process Based on Soil Property 2.6

The physical and chemical properties of the soil play a critical part in the design process of helical piles and its components. Factors such as SPT blow count values, active zone, liquefaction zone, fill soil, ground water table, and more influence the shaft diameter, helix configuration, embedment depth, and tip design.

### STANDARD PENETRATION TEST (SPT) 2.6.1

The Standard Penetration Number or in other terms the N-value as described in the previous sections indicate the relative stiffness/density of the soils at that particular location and depth. The N-values are also critical in determining the design unbraced length of the pile as well as to get a preliminary idea of the required minimum pile shaft diameter. Based on section 1810.1.3 of the IBC, a pile section standing unbraced in air, water, or fluid soil shall be designed as a column up to the point where it can be considered to be laterally supported. It is also stated that any soil other than fluid soil may be considered to provide sufficient lateral support to prevent buckling of the pile shaft in accordance with acceptable engineering practice. Section 1810.2.1 of the IBC states, *"Where deep foundation elements stand unbraced in air, water, or fluid soils, it shall be permitted to consider them laterally supported at a point 5 feet [1524 mm] into stiff soil or 10 feet [3048 mm] into soft soil."* Based on general practice in the industry, soils with a blow count of less than 5 are typically categorized as soft soils, and soils with blow count of equal to or greater than 5 are categorized as firm/stiff soils. Therefore, the design unbraced length of helical piles may be estimated based on the unbraced length and designed accordingly. Once the unbraced length has been estimated, a structural analysis can be performed on the selected pile shaft to verify the maximum allowable capacity based on the pile application. Some sample estimations of unbraced length in different soil conditions are provided in Figure 2-4.

Regardless of soft or stiff soils being present, helical piles under certain conditions can be defined as braced. Please refer to Chapter 4 of this manual for additional information on when helical piles can be considered as braced elements.

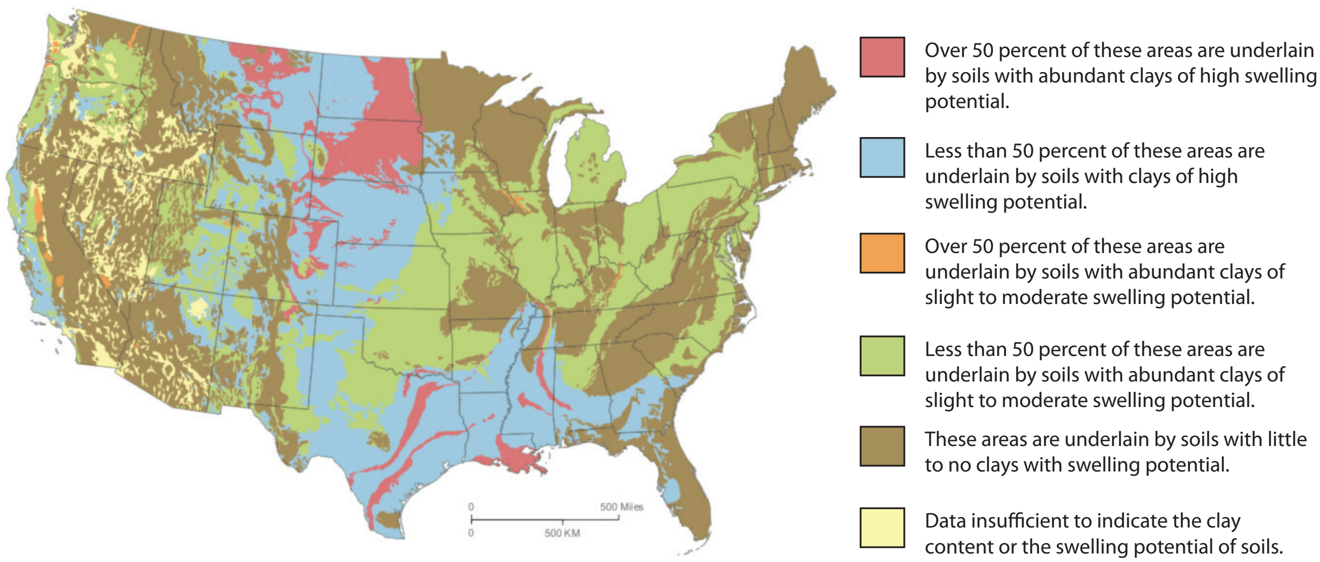



**FIGURE 2-3 : Distribution of Expansive Soil (Olive et al., 1989)**

### *FIELD CONDITION A* 2.6.1.1

It can be observed from Figure 2-4A, that the top 6 ft. [1.83 m] of soil is clayey fill with a SPT N-value expressed as weight of hammer (WOH). This implies that the sampler along with just the weight of the hammer was able to penetrate the respective 12 in.[305 mm] of soil. This is also typically expressed as soil with an SPT N-value of zero, zero blow count soil, or soil with weight of rod (WOR). Since the shear strength that is offered by these zero blow count soils are significantly less they are considered to be fluid soils. Thus a helical pile section through such a soil strata would have to be considered unbraced based on the referenced IBC section. It can also be observed that the clayey fill soil is underlain by loose sandy soil with a blow count of 4 [1.22 m] up to a depth of 20 ft. [6.10 m] below grade. Since soils with a blow count of less than 5 are considered to be soft soils, the pile shaft can be considered to be laterally supported only at a point 10 ft. [3.05 m] deep into this loose sandy stratum. Therefore, the design unbraced length would be the length of pile extending through the fluid soils plus 10 ft. [3.05 m] into the loose sandy stratum. Consequently, the design unbraced length would be 16 ft. [4.88 m] at this particular project site.

### *FIELD CONDITION B* 2.6.1.2

Based on Figure 2-4B, it appears that the boring log indicates loose well-graded sandy soil in the top 8 ft. [2.44 m] below grade underlain by organic peat soils up to a depth of 12 ft. [3.66 m] below grade. This is followed by medium to stiff clayey and silty soil up to the boring termination depth. Since the top 10 ft. [3.05 m] of soil is predominantly sandy soil with an SPT N-value of 10, the design unbraced length would be 5 ft. [1.52 m]. However, it should be noted that the boring log encountered peat soils between depths of 8 to 12 ft. [2.44 - 3.66 m] below grade. Peat soils are mainly made up of decaying wetland vegetation. It also has large pores and the typical water content value is around 80%. It is not surprising to encounter peat soils with moisture content even greater than 100%. Since it is a highly organic material it is subject to natural degradation due to decay and consequent change to its physical properties. Therefore, although the SPT N-value of the peat layer was 3 at the time when the boring was performed, it could decrease as it biologically degrades. Therefore, it is recommended that the length of pile through the peat layer be treated as unbraced and a structural column analysis be performed respectively. The pile can be considered to be fixed at the top and bottom of the peat layer based on the prevailing soil condition.

### *FIELD CONDITION C* 2.6.1.3

Figure 2-4C, indicates very loose sandy soil with a SPT N-value of 2 in the upper 6 ft. [1.83 m] below grade level underlain by 10 blow count sandy soil until a depth of 15 ft. [4.57 m] below grade level followed by stiff clayey soil until the boring termination depth. Since the soil in the upper 10 ft. [3.05 m] is predominantly comprised of very loose sandy soils, the pile shaft can be considered to be linearly supported only at a point 10 ft. [3.05 m] below grade as specified in the codes. Therefore, the design unbraced length at this particular site would be 10 ft. [3.05 m].

## LIQUEFACTION 2.6.2

Soil liquefaction can be defined as the phenomenon in which loose sandy saturated or partially saturated soil loses its strength under cyclic or dynamic loading, causing it to behave like a liquid medium. Typically liquefaction occurs in areas where loose, saturated sands are subjected to seismic loading or dynamic loading. The saturated loose sand is subjected to repeated loads within a short duration of time during an earthquake event. Consequently, the soil compresses in turn increasing the pore water pressure. Typically, under normal circumstances the increase in pore water pressure is dissipated as the water moves out of the soil pores. However, during a seismic event the rate of loading is so fast that the water does not have enough time to drain out of the pore spaces, consequently resulting in accumulation of more and more pore pressure. Eventually, the pore pressure reaches a magnitude which equals or exceeds the total stress (zero effective stress) resulting in the displacement of the soil structure. This displacement of the soil skeleton results in the loss of soil strength causing it to flow like a liquid. Therefore, during liquefaction the soil should be treated as a fluid soil.

Typically, a liquefaction analysis is performed by geotechnical engineers based on the information obtained from the soils report. Any potential liquefaction zones are identified in the soils report and these should be taken into consideration during the design of helical piles. It is critical that the helical piles bear on a suitable bearing strata well below the liquefaction zone to prevent any loss in bearing capacity. The shaft resistance/skin friction that is developed in the potential liquefaction zone should be neglected during bearing capacity estimations. Similarly, the length of pile through the liquefaction zone should be treated as unbraced and a structural analysis should be performed to determine the maximum allowable capacity of the pile based on the respective unbraced length. An external sleeve or a larger diameter pile may have to be used to provide the required allowable capacity in the presence of a liquefaction zone.

## EXPANSIVE SOIL 2.6.3

Soils with high plasticity indices, *PI,* are typically referred to as expansive soils. Expansive soils include minerals which have great affinity to water. Therefore, it is capable of absorbing a significant quantity of water which consequently results in an increase in volume. This change in volume results in the application of a force termed as swell pressure onto structures which results in heave. Expansive soils are also characterized by significant reduction in volume due to drying. As water is lost from the soil phase system due to evaporation, transpiration, etc., the soil volume is significantly reduced resulting in large fissures and cracks on the surface. This exacerbates soil heave as water is able to easily permeate through these cracks and penetrate much deeper during the following rainy season. This cycle of excessive heave and shrinkage

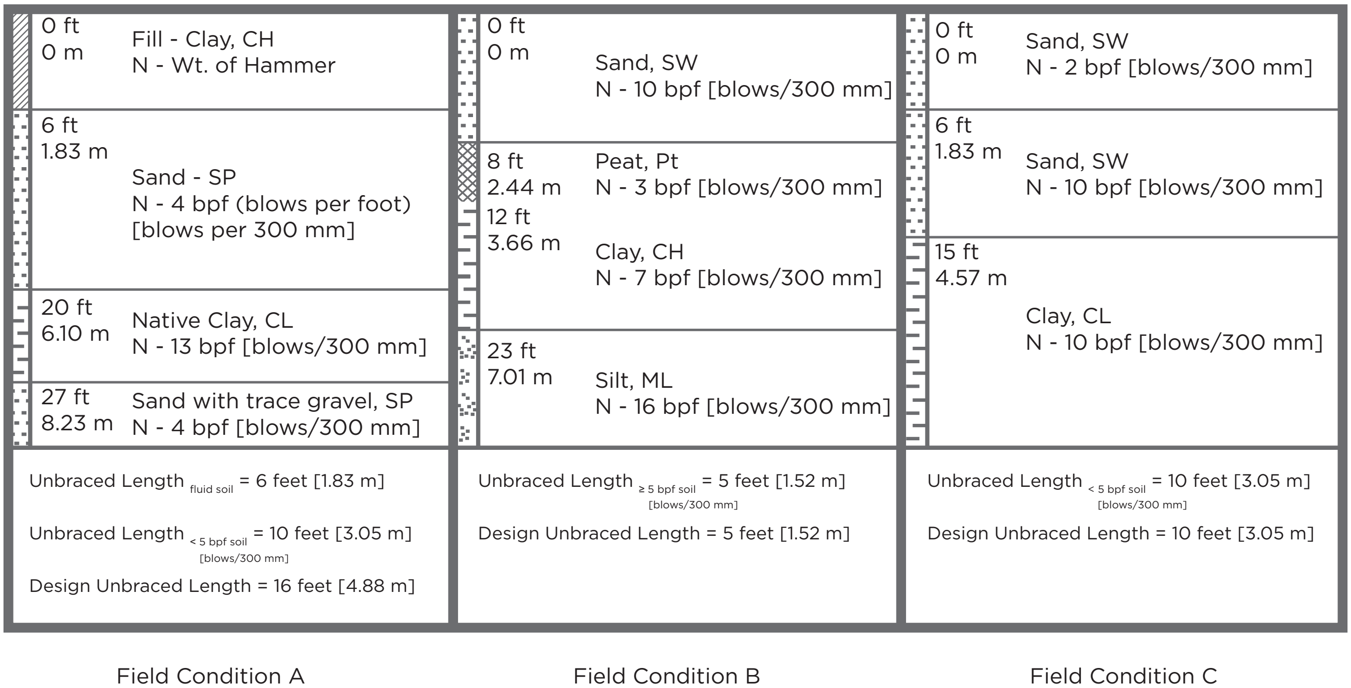



**FIGURE 2-4 : Design Unbraced Length Based on SPT – N-values**

causing cyclic movement of the structure results in distress. Expansive soils are not problematic as long as the moisture content is maintained at a constant level. Although, this is not typically the case as rain water is able to percolate into the soil and moisture is lost due to evaporation and transpiration due to natural causes. The zone through which this moisture variation occurs can be described as the active zone or the depth of wetting as shown in Figure 2-5. The maximum depth of the active zone is critical to the prediction of soil heave. The zone of seasonal moisture variation or the active zone varies with the geographical area. Geotechnical engineers who have been practicing for a while in their respective locales would have a well-defined idea on the active zone depth for that area.

Typically, the approximate embedment depth of helical piles may be optimized by varying the helix configuration. Using larger and more helical plates usually results in a lower embedment depth and vice-versa. Therefore, the helix configuration should be optimized in such a way that all the helical plates are located below the active zone prior to pile termination. This reduces or negates any swell pressure on the helical plates which might cause uplift of the piles.

## PEAT/ORGANIC SOILS 2.6.4

Organic soils are typically formed by the deposition and accumulation of partly decomposed organic matter. They are mostly found in areas with a high water table that promote the growth of aquatic plants which decompose to form the organic soils. Soils which consist predominantly of organic matter are called peat (Pt). They are generally classified based on the fiber content, ash content, water-holding capacity, and botanical composition (Holtz & Kovacs). It is critical to be aware of any peat deposits at the project site since they tend to have significantly poor engineering properties. The typical moisture content of organic soils are around 200 to 400% and are highly compressible material. Therefore, they are typically soft and weak with very low stability and shear strength.

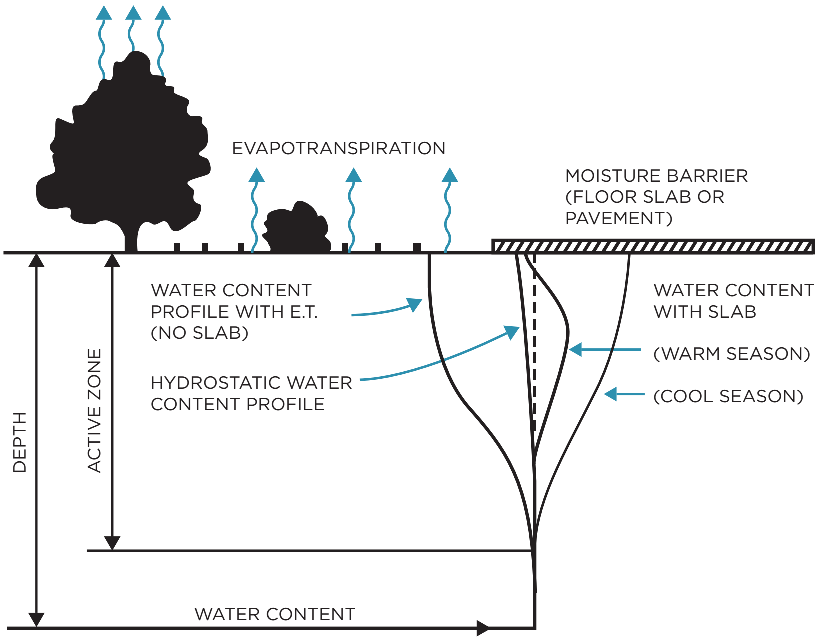



**FIGURE 2-5 : Moisture Content Profiles Within the Active Zone (Nelson & Miller, 1992)**

Peat soils most often return a SPT blow count value of zero. The recommended cohesion value for peat is 0 to 20 psf [0 to 0.96 kPa] with an adhesion coefficient of 0. Since peat is extremely porous and has significantly higher water content values (200 to 400%), the dry unit weight of peat is around 30 to 45 pcf [4.71 to 7.06 kN/m³]. It should also be noted that they go through a natural decay process and further lose strength. Therefore, it is recommended that these significantly weak soils be treated as fluid soils wherever they are encountered. The length of pile section through this layer should be considered as unbraced and a structural analysis be performed to determine the allowable capacity. The fixities of the pile at the top and bottom of the peat stratum would depend upon the depth at which the stratum was encountered and the relative stiffness of the soil layers above and below the peat strata.

Once all the above criteria have been taken into consideration, a preliminary helical pile shaft diameter and helix configuration can be selected and bearing capacities evaluated as described in Section 2.7.

## Soil Shear Strength 2.7

The shear strength of soils is a critical parameter which affects the bearing capacity of the soil. The primary shear strength parameters for sandy and clayey soils are provided by the friction angle ($\Phi$) and cohesion (c), respectively. These shear strength parameters are typically estimated from field test results such as the Standard Penetration Number (N) and Cone Penetrometer Test (CPT) values, etc. The shear strength parameters can be obtained by both field tests and laboratory tests conducted on soil samples obtained from the field. There are four methods by which soil bearing capacity or tension capacity of helical piles can be determined.

1. Individual Bearing Method
2. Cylindrical Shear Method
3. Torque Correlation Method
4. Load Testing

*Note: Reference Chapter 3.6 for additional information on the above mentioned methods.*

### TERZAGHI BEARING CAPACITY EQUATION 2.7.1

The individual bearing method utilizes the Terzaghi bearing capacity equation. This equation has been used for decades to estimate ultimate capacities of bearing-type foundation elements. The Terzaghi equation is

$$q_u = cN_c + q_v N_q + \tfrac{1}{2}\gamma B N_\gamma \qquad (2.7)$$

where,

$q_u$ = unit ultimate bearing capacity
c = cohesion soil strength parameter
$N_c$ = bearing capacity factor

$q_v$ = effective vertical stress
$N_q$ = bearing capacity factor
$\gamma$= soil unit weight
B = least lateral dimension of bearing area
$N_\gamma$ = bearing capacity factor

This equation applies to shallow foundations. It is often used with modifying factors that account for load, slope or foundation inclination, foundation shape and/or foundation depth. For deep foundations, the bearing capacity equation is usually modified to

$$q_u = cN_c + q_vN_q \quad (2.8)$$

since the $\frac{1}{2}\gamma BN_\gamma$ term proves to be negligible. Deep foundations consist of piles and drilled shafts having bearing areas located at sufficient depth to cause local shearing of the soil without propagation of the shear surface to the ground surface. With helical anchors, deep local shear occurs when the distance from the ground surface to the last helix exceeds some limiting number, reported by various investigators to be as little as four times the helix diameter (Trofimenkov and Mariupolskii, 1965; Sowers and Sowers, 1970) and as great as 16 times the helix diameter (Ghaly and Hanna, 1994). Ram Jack recommends that Equation 2.8 be used only to estimate compression capacities of helical anchors satisfying the following conditions:

- Last helix is embedded at least 4 ft. [1.22 m] vertically below the soil surface
- At least 5D in cohesive soil
- At least 7D in non-cohesive soil, respectively

Where D is defined as the largest helix diameter on the pile shaft. For compressive loading, the last helix should be embedded at least 2D vertically below the soil surface and along the shaft from the point where the shaft enters the soil.

## SHAFT RESISTANCE: 2.7.2

Shaft resistance is the resistance the soil presents to axial or rotational movement of the anchor shaft. There are several methods that can be used to determine the skin friction component along the pile shaft. Some of the methods such as the $\alpha$, $\beta$ or $\lambda$ method for computing shaft resistance is as described below.

The $\alpha$ method, proposed by Tomlinson (1971), is characterized by the modified Coulomb strength equation

$$f_s = \alpha c + q_vK \tan\delta \quad (2.9)$$

where,

$f_s$ = unit skin resistance
$\alpha$ = adhesion factor
c = cohesion soil strength parameter
$q_v$ = effective vertical stress
K = coefficient of lateral earth pressure
$\delta$ = effective friction angle between soil and pile

Others propose that unit skin resistance is more closely correlated with effective stress soil strength parameters.

$$f_s = \beta\, q_v \quad (2.10)$$

Vijayvergiya and Focht (1972) proposed yet another method for computing unit skin resistance in clays as

$$f_s = \lambda(q_v + 2c) \quad (2.11)$$

In all cases, the unit skin resistance $f_s$ is multiplied by the contact area to obtain the resisting force $F_s$.

$$f_s = Ac + Bq_v \quad (2.12)$$

where,

$f_s$ = unit skin resistance
A = adhesion coefficient
c = cohesion soil strength parameter
B = coefficient of external friction
$q_v$ = effective vertical stress

Note that with proper interpretation and input of the A and B coefficients, this equation can be used with either the α, β, or λ model of skin resistance.

For the α method,

$$A=\alpha \quad (2.13)$$

$$B=K\tan\delta \quad (2.14)$$

For the β method,

$$A=0 \quad (2.15)$$

$$B=\beta \quad (2.16)$$

For the λ method,

$$A=2\lambda \quad (2.17)$$

$$B=\lambda \quad (2.18)$$

In order to calculate the skin friction component, the shaft is first divided into two sections, a lead section and an extension section. Then the lead section is subdivided at the helix locations. For load capacity analysis, the lead shaft section ends are redefined to allow for a one-helix-diameter dead zone on the bearing side (upper side for tension, lower side for compression) of each helix. Then all sections are further subdivided at points where they cross the phreatic surface and soil stratum boundaries. For each of these sections, the unit skin resistance is computed as detailed above for the section midpoint and multiplied times the surface area of the section to obtain individual section resistances. Finally, the individual section resistances are summed to obtain the total shaft resistance.

## COHESION 2.7.3

Cohesion is the shear strength intercept of the Coulomb strength envelope for the soil in a stratum. It can be defined as the force that holds the clay particles within the soil mass. Typical cohesion values based on the SPT blow count values are provided in Table 2-3. The values in Table 2-3 are presented to help estimate an appropriate cohesion value with whatever information may be available. The undrained shear strength (cohesion) is usually expressed by a cohesion intercept only, and may be taken as $q_u/2$. The SPT/unconfined compressive strength, $q_u$, correlation in Table 2-3 is adequately represented by $q_u = N/4$, ksf [kN/m$^2$]. Consequently, the cohesion value can be estimated as follows

$$c = \frac{q_u}{2} \quad (2.19)$$

$$c = \frac{N}{8}\text{, ksf} \quad (2.20)$$

Other direct-measurement tests for cohesion include the tri-axial test, direct shear test, vane shear test, and borehole shear test. Cohesion can also be estimated by correlation with the standard penetration test, cone penetration test, California Bearing Ratio, liquidity index, plasticity index, and pocket penetration test.

## ADHESION 2.7.4

Adhesion could be defined as the attractive force between the clay particles to a surface made up of a different material. Adhesion is typically expressed as a fraction of cohesion using an adhesion coefficient. The adhesion

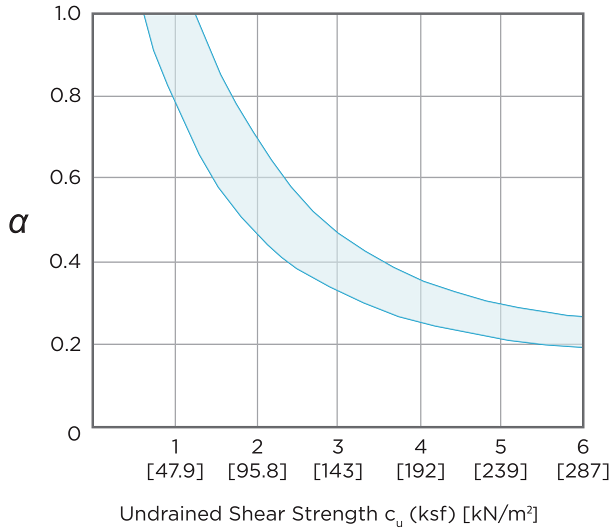



**FIGURE 2-6 : Adhesion Coefficient α vs. Undrained Shear Strength $C_u$ (Das, 1984).**

coefficient, A, is a factor that is multiplied by the soil's cohesion strength parameter to yield the adhesive component of skin resistance. Das, 1984 has provided the adhesion coefficient values based on the undrained shear strength values as shown in Figure 2-6. In the Tomlinson α model of skin resistance, A is equal to the Tomlinson factor α. In the β model of skin resistance, A is equal to 0. In the λ model of skin resistance, A is equal to 2λ.

It is cautioned that none of the three methods for which information is included here were developed for use with helical piles, and most of the information presented relates to driven piles. In comparison, helical piles typically produce lower lateral soil displacement, soil disturbance, and a different stress distribution near the pile toe. These differences between driven piles and helical piles should be considered when selecting the skin resistance factors.

## $N_c$ 2.7.5

$N_c$ is the bearing capacity factor that is multiplied times the cohesion strength parameter to obtain that part of the soil's unit bearing capacity that is related to soil cohesion. Values appropriate for deep local shear should be used. Typically for deep foundations a default value of 9 is recommended. An $N_c$ value of 9 has been found to work reasonably well with natural (un-remolded) cohesion values taken as one-half the unconfined compressive strength.

## ANGLE OF INTERNAL FRICTION 2.7.6

The angle of internal friction is the slope of the Coulomb strength envelope for the soil in a stratum. Typical values for the angle of internal friction based on SPT blow count values are as shown in Table 2-2. Although it does not appear directly in either the bearing capacity or skin friction equation, it is used to determine the default value of $N_q$. The angle of internal friction will be taken as constant for each defined stratum.

| Soil Type | Friction Angle, $\delta$ (˚) | Friction Coefficient, tan$\delta$ |
|---|---|---|
| Clean gravel, gravel–sand mixtures, well-graded rock fill with spoils | 22 | 0.40 |
| Clean sand, silty sand-gravel mixtures, single size hard rock fill | 17 | 0.30 |
| Silty sand, gravel or sand mixed with silt or clay | 14 | 0.25 |
| Fine sandy silt, non-plastic silt | 11 | 0.20 |

**TABLE 2-5 : Angle of External Friction $\delta$ and Friction Coefficient tan$\delta$ for Steel Against Various Types of Soils (NAVFAC, 1986).**

## COEFFICIENT OF EXTERNAL FRICTION 2.7.7

The coefficient of external friction, B, is a factor that is multiplied by the effective vertical stress to yield the frictional component of skin resistance in sand. In the Tomlinson $\alpha$ model of skin resistance, B is equal to K$\tan\delta$. In the $\beta$ model of skin resistance, B is equal to $\beta$. In the $\lambda$ model of skin resistance, B is equal to $\lambda$.

Table 2-5 gives some typical angles of external friction $\delta$ and friction coefficients $\tan\delta$ for steel against various non-plastic soil types. According to ASCE (1996), the ratio of the angle of external friction to the angle of internal friction may be taken as 0.54 for sand, silt, and clay. Consequently the coefficient of external friction B is estimated as $\tan\delta$ based on NAVFAC, 1986 (Table 2-5).

The skin resistance coefficient $\beta$ has been found to vary from about 0.25 to 0.40, with an average of about 0.32. A correction factor may be appropriate for embedment depth to shaft diameter ratios exceeding 15 to 20.

In the $\alpha$ method, the coefficient of lateral earth pressure K is often taken as the at-rest coefficient $K_0$ and the effective angle of external friction $\delta$ is often taken as the effective angle of internal friction $\Phi'$ (Bowles, 1982).

$K_0$ for piles is most commonly computed as (Bowles, 1982)

$$K_0 = (1 - \sin\phi')\sqrt{OCR} \quad (2.21)$$

$K_0$ can also be calculated from plasticity index as (Carter and Bentley, 1991).

$$K_0 = 0.44 + 0.0042 I_p \quad (2.22)$$

| Angle of Internal Friction, $\Phi$ (deg) | $N_q$ |
|---|---|
| 0 | 1.0 |
| 5 | 1.6 |
| 10 | 2.5 |
| 15 | 3.9 |
| 20 | 6.4 |
| 25 | 11 |
| 30 | 18 |
| 35 | 33 |
| 40 | 64 |
| 45 | 130 |
| 50 | 320 |

**TABLE 2-6 : Default Values of $N_q$ vs. Angle of Internal Friction $\Phi$**

## $N_q$ 2.7.8

$N_q$ is the bearing capacity factor that is multiplied times the effective vertical stress to obtain that part of the soil's unit bearing capacity that is related to soil internal friction. Values appropriate for deep local shear should be used. Table 2-6 provides the default values of $N_q$ with respect to the internal friction angle which have been found to work reasonably well when selected on the basis of undisturbed angle of internal friction deduced from the Standard Penetration Test. Correlations have also been developed between the angle of internal friction and the cone penetration test, plasticity index of cohesive soils and relative density. Direct-measurement tests include the direct shear test and borehole shear test. Wherever possible, the use of several methods together is recommended.

## ULTIMATE AXIAL CAPACITY, $Q_U$ 2.7.9

Based on section 4.4.1.2 of the AC358 the maximum/ ultimate load capacity *"shall be that which is achieved when plunging of the helical plate occurs or when net deflection exceeds 10 percent of the helix plate diameter, whichever occurs first"*. If the helical pile consists of more than one helical plate, then the average helical plate diameter is typically used to determine the 10% net deflection value. It is important to recognize that, should this load be applied to the pile, the pile head would be expected to move without limit. Many foundation engineers choose to define ultimate load in relation to a serviceability limit state, defined by a maximum allowable movement, say 1 in. [25.4 mm], at the anchor head. In such cases, it is recommended that a separate assessment of expected pile movement be made to ensure the pile will meet serviceability based performance requirements.

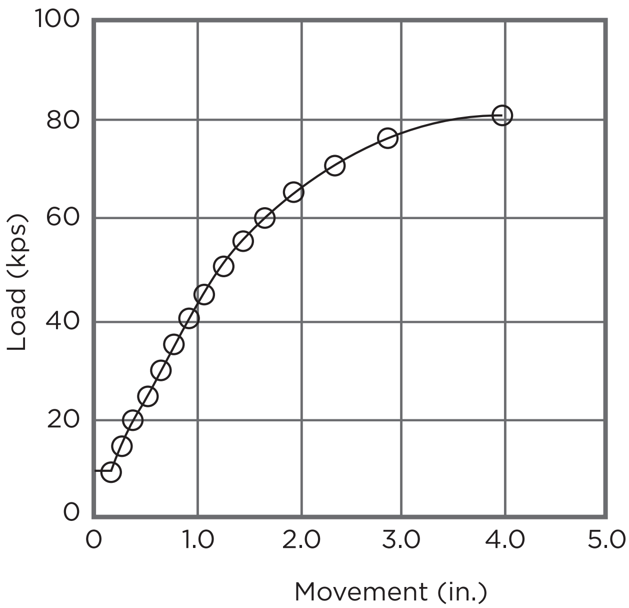



**FIGURE 2-7 : Load vs. Deflection Curve of Helical Anchor in Sand (Clemence, 1994)**

Figure 2-7 shows a typical load versus movement curve for a helical anchor. The anchor consisted of three 12 in. [305 mm] helices on a 2 in. [50.8 mm] round-cornered square bar shaft and was embedded 10 ft. [3.05 m] in medium dense, fine to medium sand (Clemence, et al, 1994). The ultimate pullout capacity was 80 kips [356 kN], attained at a head movement of 4 in. [102 mm] (0.33 times the largest helix diameter). Note that if the ultimate capacity were defined as that load which produces a head movement of, for instance, 1 in. [25.4 mm], the ultimate capacity would only be 42 kips [187 kN]. The shape of the load/movement curve indicates that if 80 kips [356 kN] were applied to the anchor in a stress-controlled test, head movement would continue without limit. In many cases, continuing a strain-controlled test beyond the ultimate pullout or plunging capacity will result in a reduction of load resistance to a residual value.

## REQUIRED ALLOWABLE CAPACITY, $Q_A$ 2.7.10

The allowable load capacity or the design/working load that is required to be resisted by the helical pile. Section 1810.3.3.1.9 of the IBC states that the ultimate capacity of a helical pile is the product of the safety factor and the allowable capacity.

A safety factor, *S.F.,* has to be applied to the required ultimate capacity, $Q_U$, to estimate the required allowable capacity, $Q_A$. This would typically be provided by the Engineer of Record (EOR). A safety factor of 2 is recommended by the IBC code based on Section 1810.3.3.1.9.

$$Q_A = \frac{Q_U}{S.F.} \tag{2.23}$$

## Soil Compressibility 2.8

Soil reacts to increased load in one of two distinctly different ways, depending on the magnitude of the increased stress. If the stress reaches the soil's strength limit, soil failure occurs, accompanied by large-scale distortion of the soil and instability of any supported structures. If the stress does not reach the soil's strength limit, the soil/structure system remains stable but relatively small-scale displacements of both occur as the soil/water system adjusts to the new loading condition. These adjustments include rearrangement, flexing, and fracture of soil grains and expulsion of fluids from voids between the soil grains. Soil response depends on whether the soil is saturated or not and on drainage conditions. Because our interest lies in the application of soil mechanics to helical piles and push piles—both deep foundation elements typically founded below the ground water table—we restrict the discussion here to saturated soils.

Saturated soil adjusts to increased load in three stages called primary compression, consolidation, and secondary compression. Primary compression occurs fairly quickly and results from reduction in the volume of voids due to expulsion of air, compression of the free water, and deformation of soil grains during which excess pore pressure arises to resist the additional load. Displacement of an overlying soil stratum or structure during this initial stage may be very small in saturated soils because there is no free air to compress, the continuous water phase is almost incompressible, and soil grain deformation is very limited until effective stress increases.

Consolidation is the reduction in void ratio that accompanies the expulsion of free and/or loosely-held water from the soil voids. It occurs rapidly in coarse-grained soils and slowly in fine-grained ones. In clays, it can continue for years after primary compression is complete. Displacement of an overlying soil stratum or structure due to consolidation depends on the thickness and void ratio of the affected soil layer and the change in external stress. The rate of settlement depends on the change in stress, the soil permeability, and the length of the drainage path(s). Because fine-grained soils tend to have large void ratios and low permeability, they may exhibit large deformation that occurs over a long time period. The average percent of the added total stress that is supported by increased effective stress is termed degree of consolidation.

Secondary compression is the reduction in void ratio due to expulsion of adsorbed water from between the grains and the resulting grain deformation and fracture under the increased contact pressure between the mineral particles. Secondary compression may continue for years after consolidation ends.

Settlement of coarse-grained soils occurs very rapidly because the large pore spaces allow water to escape quickly. Thus the estimation of total settlement comprises a complete settlement analysis for coarse-grained soils. With fine-grained soils, though, knowledge of how settlement will occur over time may be as important as knowledge of the eventual total settlement.

### SETTLEMENT MAGNITUDE 2.8.1

The total settlement of a pile for a given maximum allowable capacity is provided by the following equation

$$S_T = S_e + S_{tip} + S_{shaft} \qquad (2.24)$$

Where,

$S_e$ = elastic settlement of pile
$S_{tip}$ = settlement of pile due to the allowable load at the pile tip
$S_{shaft}$ = settlement of pile due to the skin friction developed along the pile shaft

Based on the principles of mechanics of materials, the elastic deformation of the pile shaft may be estimated as follows

$$S_e = \frac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p} \qquad (2.25)$$

where,

$Q_{wp}$ = load carried at pile tip under max. allowable load
$Q_{ws}$ = load carried by skin friction under max. allowable load
$A_p$ = cross sectional area of pile
L = length of pile
$E_p$ = Modulus of elasticity of pile

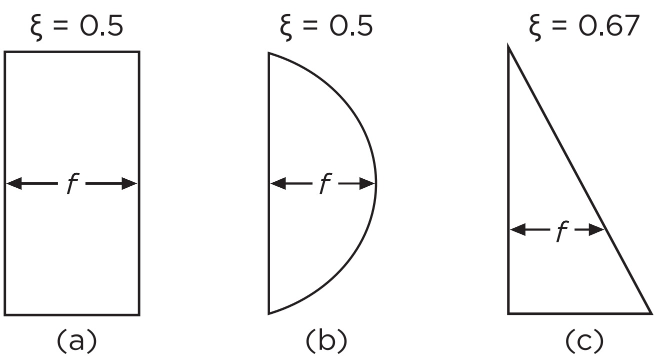



**FIGURE 2-8 : Types of Unit Skin Friction Distribution Over Pile Shaft**

The value of $\xi$ will vary based on the type of skin friction (f) resistance distribution over the pile shaft. The magnitude of $\xi$ can be estimated as shown in Figure 2-8. If the skin friction resistance distribution is uniform or parabolic as shown in Figure 2-8 then $\xi$ = 0.5. Further, $\xi$ = 0.67 for a triangular type distribution of f (Vesic, 1977).

The settlement of piles due to the load carried at the pile tip can be evaluated as follows

$$S_{tip} = \frac{(q_{wp}\, D)}{E_s}(1-\mu_s^2) I_{wp} \qquad (2.26)$$

where,

D = pile diameter
$q_{wp}$ = point load/unit area at pile tip = $Q_{wp}/A_p$
$E_s$ = modulus of elasticity of the soil at or below the pile tip
$\mu_s$ = Poisson's ratio of soil
$I_{wp}$ = influence factor

| Soil Type | Driven Pile | Bored Pile |
|---|---|---|
| Sand | 0.02 - 0.04 | 0.09 - 0.18 |
| Clay | 0.02 - 0.03 | 0.03 - 0.06 |
| Silt | 0.03 - 0.05 | 0.09 - 0.12 |

**TABLE 2-7 : Typical Values of $C_p$**

A semi-empirical formula was also proposed by Vesic (1977) to estimate the settlement magnitude at pile tip as follows

$$S_{tip} = \frac{(Q_{wp}\, C_p)}{D q_p} \qquad (2.27)$$

where,

$q_p$ = ultimate tip resistance of pile
$C_p$ = empirical coefficient

Similarly, the pile settlement due to the load carried by the pile shaft can be calculated as shown:

$$S_{shaft} = \left(\frac{Q_{ws}}{pL}\right) \frac{D}{E_s} (1-\mu_s^2) I_{ws} \qquad (2.28)$$

where,

p = perimeter of pile
L = embedment length

The influence factor $I_{ws}$ can be calculated by an empirical formula as proposed by Vesic (1977).

$$I_{ws} = 2 + 0.35 \sqrt{\left(\frac{L}{D}\right)} \qquad (2.29)$$

An empirical formula was also proposed by Vesic (1977) to calculate the settlement of the pile due to the load carried by the pile shaft.

$$S_{shaft} = \frac{(Q_{ws} C_s)}{Lq_p} \qquad (2.30)$$

where,

$C_s = (0.93+0.16\sqrt{(L/D)})\ C_p$

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# 3: RAM JACK HELICAL PILES

## Introduction 3.1

Helical piles were first invented in 1832 by Alexander Mitchell, an Irish engineer, by accident when experimenting with sail boats. He called his invention a screw pile and patented it in 1833 and initially used it for ship moorings. In 1838 Mitchell used helical piles to support the Maplin Sands Lighthouse in England. His technology was followed by Eugenius Birch to support seaside piers throughout England. During expansion of the British Empire, helical piles were used to support bridges on many continents.

Helical piles were first used in the United States by Major Hartman Bache for a lighthouse at Brandywine Shoal in Delaware Bay in 1850. Alexander Mitchell sailed to America and served as a consultant for the project. From 1850s through 1890s, several lighthouses were constructed on helical pile foundations along the East Coast of the United States and along the Gulf of Mexico.

Though helical piles were primarily used for anchoring, with advancement in modern installation equipment and manufacturing process, helical piles are now used throughout the world for both anchoring and compression applications.

## Ram Jack Helical Pile 3.2

A helical foundation (pile) is a factory manufactured steel foundation consisting of a central shaft with one or more helical shaped bearing plates. The terms "foundation" or "pile" typically refers to a compression application. A helical anchor has the same components as a helical pile but refers to a tension application.

When a helical pile is rotated or torqued into the ground, the pitch (typically 3 in. [76.2 mm]) of the helix plates generate an axial thrust, causing it to advance into the soil. Helical extensions are added to the pile as

**FIGURE 3-1 : Alexander Mitchell, Inventor of Helical Piles**

needed in order to continue advancing the pile into the soil. Installation is stopped when a specified torque is reached or the torque rating of the pile shaft has been reached. After installation, the helical plate transfers axial load into the soil through bearing. Ram Jack manufactures helical piles and extensions in a wide array of shaft diameters (2 ⅜ in., 2 ⅞ in., 3 ½ in. and 4 ½ in. [60.3 mm, 73.0 mm, 88.9 mm, 114 mm]). Additional shaft diameters are available by special order. All helical shafts and extensions are carbon steel with minimum yield strength of 65 ksi [450 Mpa]. All Ram Jack helical pile components have a thermoplastic coating which has been specifically designed to provide a long lasting, tough coating. The thermoplastic coating meets the Acceptance Criteria for Corrosion Protection of Steel Foundation Systems Using Polymer (EAA) Coatings (AC228). The acceptance criterion was adopted by the International Code Council (ICC) in October 2003. The thermoplastic coating is environment friendly and has a technical life span of 50 years. (Reference Section 6.5.1 for more detailed information on thermoplastic coating).

**Helical Plate:** The helical plates are circular steel plates pressed in spiral form with uniform pitch. The helix plates are cut to exact shape from 50 ksi [344 Mpa] plate steel on a computerized plasma table and pressed to the proper configuration with a die and hydraulic press. Each helix plate has a 360° spiral and approximately 3-in. [76.2 mm] spacing between the leading and tailing edges (referred to as the helix *pitch*). Plates are generally ⅜ in. [9.53 mm] to ½ in. [12.7 mm] thick, depending on the shaft diameter and loading conditions. Standard helix plate diameters are 8 in. [203 mm], 10 in. [254 mm], 12 in. [305 mm], 14 in. [356 mm], or 16 in. [406 mm]. When multiple helices are welded to the same shaft, smaller helices precede larger ones. The maximum number of helix plates that can be placed on a single pile is limited to six. The spacing distance between any two helices is a minimum of three times the diameter of the lower helix. Ram Jack's helix plates are welded to the shafts with a mig and argon wire/gas high amperage weld welder. This allows for the penetration necessary for a strong weld attachment.

**Helical Lead Section:** Helical lead sections are circular shafts with a pilot point at the tip, and one or more helical plates welded to the shaft.

**Pile Shaft Connections:** Ram Jack utilizes two coupling systems to make the pile shaft connections, through bolted and internally threaded. The through bolted connections are typically used when the piles will resist low lateral loads and an external sleeve will not be placed over the pile shaft. Internal threaded connections are unique to the industry and only used by Ram Jack. Internal threaded connections are used when high lateral loads will be resisted by the pile or an external sleeve will be placed over the pile shaft. This patented connection (U.S. Patent No. 6,514,012) is quick and easy to install in the field and produces a rigid joint with better flexural and torsional stiffness than competitive products, resulting in better buckling resistance under compressive and lateral loads.

The threaded connection also allows a smooth homogeneous surface on the perimeter of the pile shaft. This allows variable length (up to 30 ft. [9.14 m]) external sleeves to be placed over the piles. The external sleeve increases the moment of inertia (stiffness) of the pile shaft where the maximum stresses will occur. Therefore, the external sleeve allows a smaller, more economical pile to accept higher loads.

**Helical Extensions:** Helical extensions are pile shafts torqued into the ground following the lead section. The extensions may or may not have helical plates. The maximum number of helical plates that can be placed on a helical pile is limited to six.

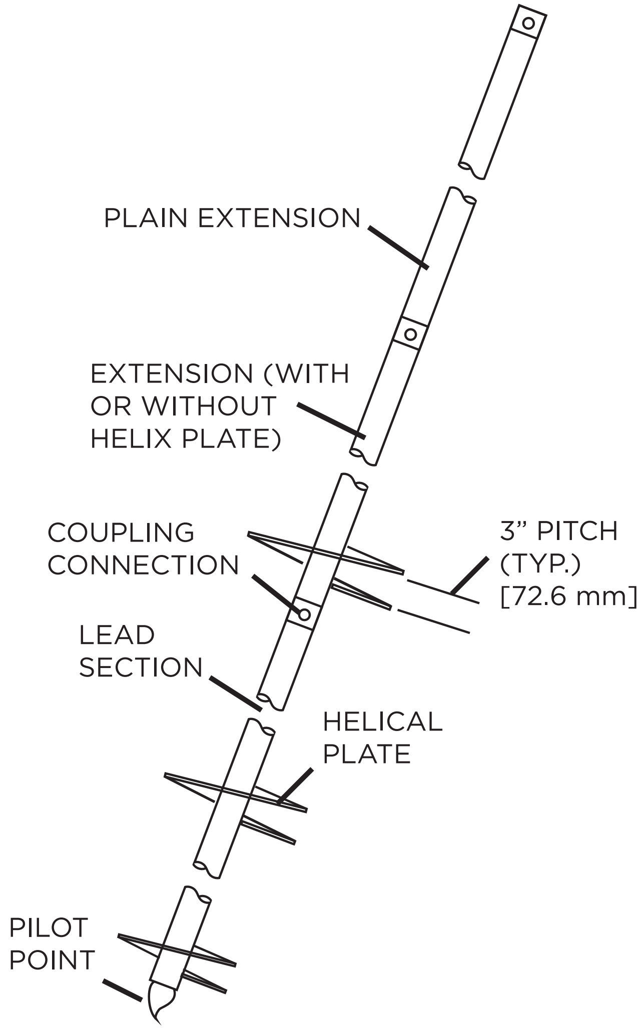



**FIGURE 3-2 : Typical Helical Pile Components**

## Helical Pile Applications 3.3

Ram Jack helical piles may be used in new construction where the costs or risks associated with conventional concrete foundations dictate, or they can be utilized to stabilize or remediate existing structures whose foundations are in distress. In these latter applications, they are often used in conjunction with prefabricated steel brackets that facilitate the load path and connection of the piles to the foundation.

*NEW CONSTRUCTION*

- Compression and tension applications such as foundations for buildings, pedestrian bridges, solar fields, launch pads, etc.
- Tension applications such as tie backs in retaining wall, basement walls, guy towers, etc.
- Shear and overturning applications such as lighting poles and self-supporting tower foundations.

*REPAIR AND REMEDIAL WORK*

- Underpinning distressed foundations or augment existing foundations for heavier loads.

## Advantages of Helical Piles 3.4

- Low mobilization costs due to small and inexpensive equipment.
- Low noise and minimal vibrations.
- Smaller shafts provide resistance to expansive soil heave and down drag forces.
- Can be loaded immediately after installation.
- Can be installed in confined spaces.
- Year-round installation in any weather.
- Installation produces no spoils to be disposed of or remediated.
- Installation torque can be used to verify capacity.
- Field welding is typically not required.

## Installation 3.5

Ram Jack helical piles are screwed into the ground by applying torque and axial thrust to the top of pile. These forces must be maintained during the installation to ensure the anchor does not either cease to rotate or cease to advance at the proper rate of about one pitch length (3 in. [76.2 mm]) per revolution. Installation can be done with either hand-held or machine-mounted

**FIGURE 3-3 :** New Construction Bracket

**FIGURE 3-4 :** Remedial Repair

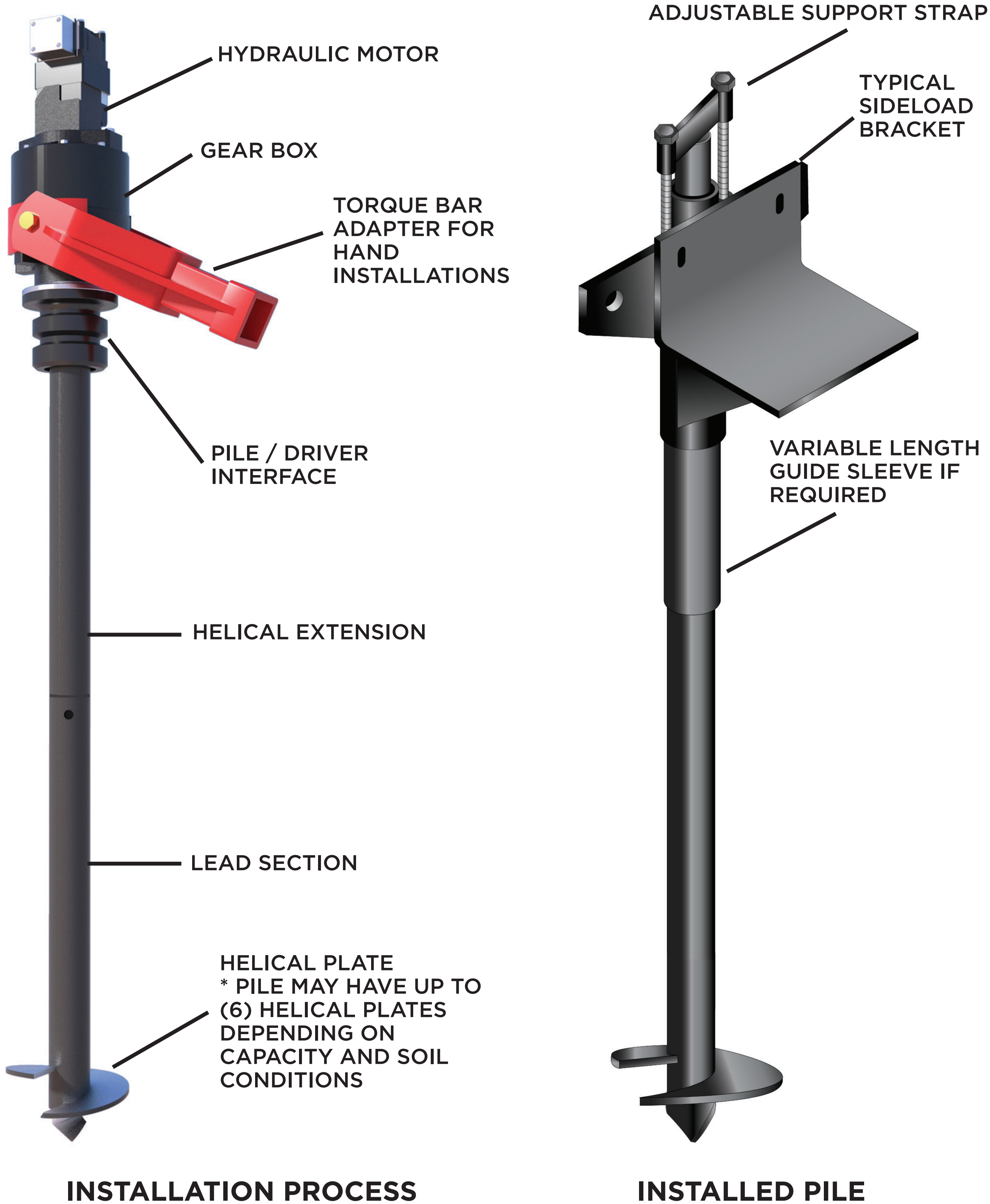



**FIGURE 3-5 :** Typical Helical Pile System equipment. The maximum load that an installed anchor can support is directly related to the torsional resistance that is generated during installation. In many cases, as torsional resistance increases, crowd must likewise increase. Therefore, anchors installed with hand-held equipment will necessarily be limited to supporting lighter loads because of the limited ability of humans to provide torsional reaction and crowd force to accomplish the installation. Although hand wrenches and electric motors can be used to supply the installation torque, the most commonly used equipment is a hydraulic motor in combination with a planetary gearbox to provide the low speed and high torque needed. The installation rate of a helical pile is typically limited to 25 revolutions per minute (RPM). This helps limit the risks of the helical plates 'auguring' the soil during installation.

## Calculating Soil Capacity 3.6

There are four methods by which soil bearing capacity or tension capacity of a helical pile can be determined.

- Individual Bearing Method
- Cylindrical Shear Method
- Torque Correlation Method
- Load Testing

Individual bearing method suggests that ultimate bearing capacity of a pile is the sum of bearing capacities of individual helical plates, whereas cylindrical shear method states that total bearing capacity is sum of resistance of the soil supporting the top helix plate, plus the cylindrical shear resistance of soil between plates, plus the soil friction/adhesion along the shaft above the top helix.

The first two methods utilize soil mechanics and the strength properties of soil in which the pile is embedded, thus they are generally used to select the proper pile configuration for a particular job. The third method uses an empirical correlation between the helical pile's ultimate resistant and the torsional resistance encountered during its installation. This method is particularly useful for quality control during construction. Hoyt and Clemence (1989) found the torque correlation method to be reliable. The fourth method consists of full-scale, site-specific load tests. The load tests are typically performed per ASTM standards for compression, tension, or lateral loads as applicable. The individual bearing method, torque correlation, and load testing are methods included in Section 1810.3.3.1.9 of the IBC.

### CYLINDRICAL SHEAR METHOD 3.6.1

The *cylindrical shear* method for calculating ultimate resistance of a multiple helix tension anchor assumes that when the soil fails, the helix plates and the soil between them are mobilized as one mass, while a bearing failure occurs above the top helix (Figure 3-6). The total resistance comprises the bearing resistance of the soil supporting the top helix plate, plus the cylindrical shear resistance of the soil between the plates, plus the soil friction/adhesion resistance along the shaft above the top helix (Mitsch and Clemence, 1985; Mooney, Adamczak and Clemence, 1985). This method is commonly extended to compression applications by assuming bearing failure below the bottom helix, rather than above the top helix, with no change to the other components of total resistance.

*Pullout Resistance* $Q_T$ is expressed as:

$$Q_T = A_{CS}S_u + A_T(cN_c + \sigma'_vN_q) + \pi D_sL_sq_s$$

*Plunging Resistance* $Q_C$ is expressed as:

$$Q_C = A_{CS}S_u + A_B(cN_c + \sigma'_vN_q) + \pi D_sL_sq_s$$

Where

$A_B$ = Area of bottom helix ($l^2$)
$A_{CS}$ = Surface area of the shear cylinder between top and bottom helix plates ($= \pi D_A L_H$) ($l^2$)
$A_T$ = Area of top helix ($l^2$)
$D_A$ = Average diameter of cylindrical shear surface, $(D_B + D_T)/2$ (l)
$D_B$ = Diameter of bottom helix (l)
$D_T$ = Diameter of top helix (l)
c = Soil cohesion ($f/l^2$)
$L_H$ = Distance between helices (l)
$L_s$ = Length of shaft above helices that is in contact with soil (l)
$N_q$ = Bearing capacity factor for overburden (dimensionless)
$Q_C$ = Ultimate plunging resistance (f)
$q_s$ = Unit ultimate shaft resistance ($f/l^2$)
$Q_T$ = Ultimate pullout resistance (f)
$S_u$ = Unit undrained soil shear strength ($f/l^2$)
$\sigma'_v$ = Effective vertical stress ($f/l^2$)

The value $q_s$ must be appropriate for soil that has been disturbed by the passage of helices. In the case of tension anchors, the same applies to the values of $\mathbf{N_c}$ and $\mathbf{N_q}$. For compression applications, the bottom helix bears on undisturbed soil. However, it is common (and conservative) to use the same bearing capacity factors for both tension and compression applications. Numerical values may be found in Mooney, Adamczak and Clemence (1985) and in Mitsch and Clemence (1985).

## INDIVIDUAL BEARING METHOD 3.6.2

The *individual bearing* method for calculating ultimate resistance assumes a bearing failure of the soil supporting each helix (Hoyt and Clemence, 1989). The total resistance for the helix anchor is the sum of the bearing resistances of the individual helix plates, plus the resistance due to soil friction/adhesion on the portion of the shaft above the top helix plate (Figure 3-7). The shaft friction/adhesion resistance may or may not be a significant part of the total.

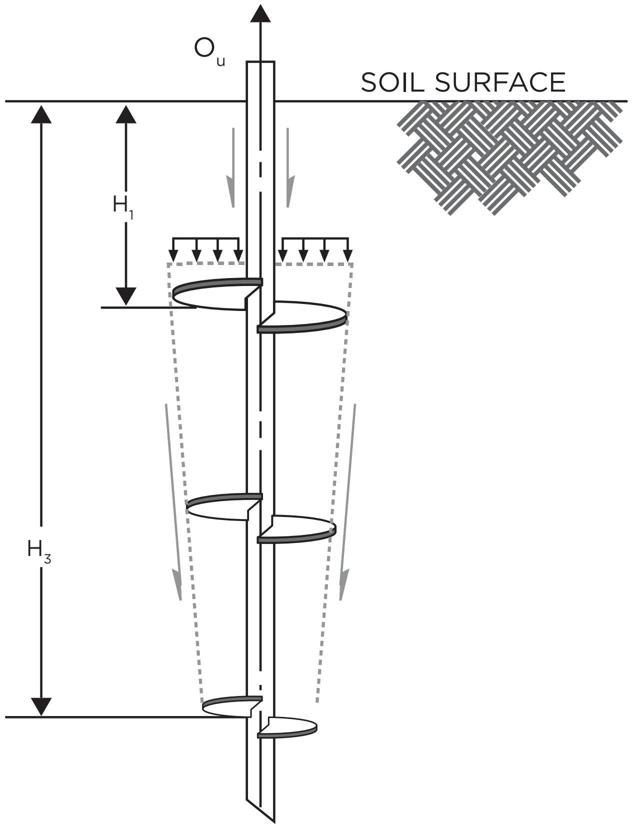



**FIGURE 3-6 : Cylindrical Shear Failure Mode in Tension (Hoyt and Clemence, 1989)**

*Plunging and Pullout Resistance* $Q_C$ and $Q_T$ are expressed as:

$$Q_C = Q_T = \sum[A_H\,(cN_c + \sigma'_v N_q)] + \pi D_s L_s q_s$$

Where,

$A_H$ = Area of individual helix plate ($l^2$)
$D_s$ = Outside diameter of shaft (l)
c = Soil cohesion ($f/l^2$)
$L_s$ = Length of shaft above helices that is in contact with soil (l)

$N_c$ = Bearing capacity factor for cohesion (dimensionless)
$N_q$ = Bearing capacity factor for overburden (dimensionless)
$Q_c$ = Ultimate plunging resistance (f)
$q_s$ = Unit ultimate shaft resistance ($f/l^2$)
$Q_T$ = Ultimate pullout resistance (f)
$Q_u$ = Ultimate bearing resistance (f)
$\sigma'_v$ = Effective vertical stress ($f/l^2$)

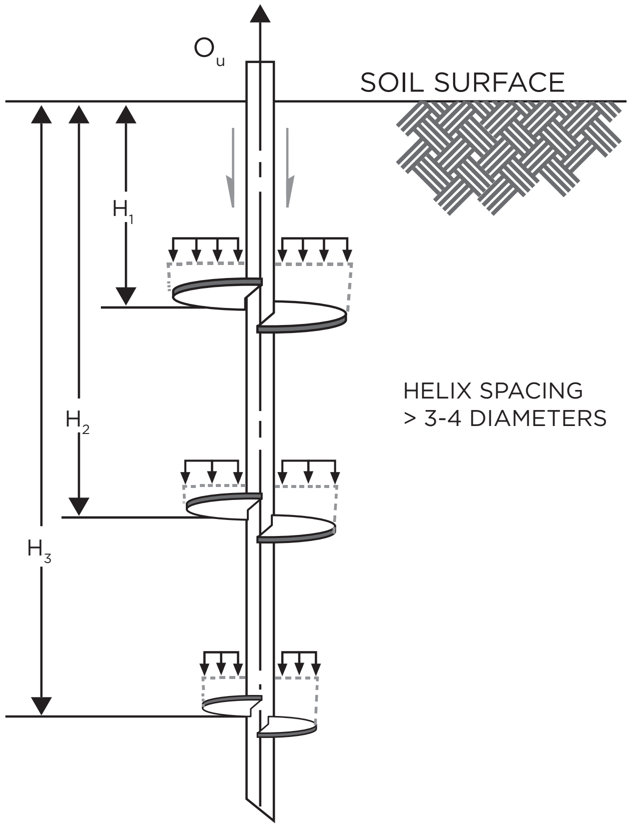



**FIGURE 3-7 : Individual Bearing Failure Mode in Tension (Hoyt and Clemence, 1989)**

Again, the shaft and bearing resistances must be calculated based on disturbed soil properties and it is common to use the same bearing capacity factors for both tension and compression applications. For deep anchors, Ram Jack recommends an $N_c$ value of 9. Recommended values for $N_q$ are given in Figure 3-8.

| Angle of Internal Friction, $\Phi$ (deg) | $N_q$ |
|---|---|
| 0 | 1.0 |
| 5 | 1.6 |
| 10 | 2.5 |
| 15 | 3.9 |
| 20 | 6.4 |
| 25 | 11 |
| 30 | 18 |
| 35 | 33 |
| 40 | 64 |
| 45 | 130 |
| 50 | 320 |

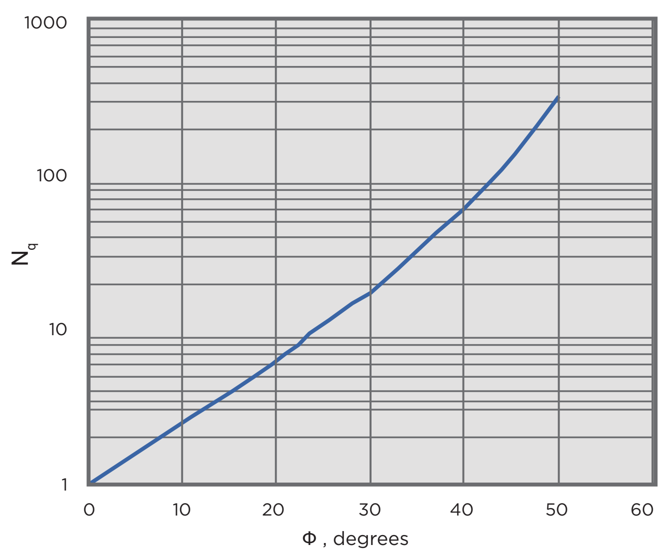



**FIGURE 3-8 : Bearing Capacity Factor $N_q$ vs. Soil Angle of Internal Friction $\Phi$**

The cylindrical shear method predicts that total resistance in a homogeneous clay deposit would increase linearly with the inter-helix spacing $L_H$. This is counter-intuitive for very large spacing. However, the individual bearing method would predict an unchanging helix bearing resistance as the spacing is varied, which is counter-intuitive for very small spacing. These relations are shown for a twin-helix anchor in Figure 3-9. Neither method makes sense for all values of helix spacing. Fortunately, the upper bound principles enable the development of a unified approach to resistance calculation that does work for all spacing.

The *upper bound principle* states that the load corresponding to any kinematically permissible failure mode is an upper bound to the true ultimate failure load. A kinematically permissible failure mode is one for which the material deformations do not violate any geometric (movement or slope) constraints. So any failure mode one can imagine that does not violate geometric constraints such as boundary support conditions or material continuity is a permissible failure mode and the load that would have to be applied to create that failure mode is an upper bound to (i.e., is greater than or equal to) the actual failure load. The *least upper bound principle* states that, of all the kinematically permissible failure modes, the one having the least corresponding load is the true failure mode and its corresponding load is the true ultimate failure load. That is, the structure will not support a load that is higher than the lowest one that will produce a kinematically permissible failure mode.

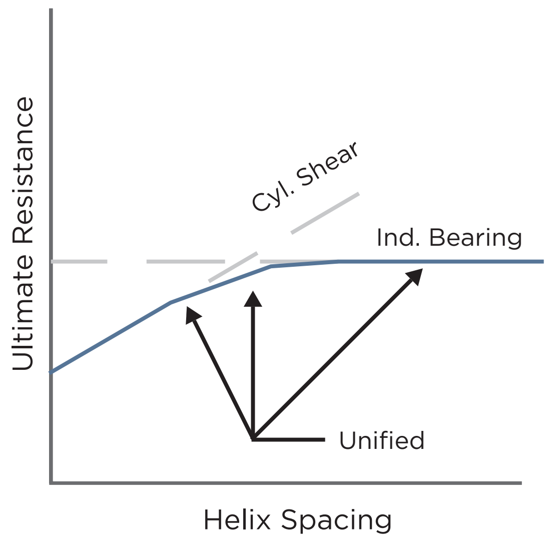



**FIGURE 3-9 : Ultimate Resistance of Twin-Helix Helical Pile vs. Helix Spacing**

Both the cylindrical shear and the individual bearing failure modes are kinematically permissible at all magnitudes of helix spacing. From Figure 3-9 it is clear that the cylindrical shear mode has a lower corresponding load than the individual bearing mode for closely spaced helices. On the other hand, the individual bearing mode has the lower corresponding load for helix spacing greater than some spacing, s. Assuming there are no other kinematically permissible failure modes (such as overhauling) to be considered, the least upper bound principle leads to the conclusions that:

1. The cylindrical shear mode controls for helix spacing less than s; therefore, the cylindrical shear method is the preferred method for computing resistance at these helix spacing.
2. The individual bearing mode controls for helix spacing greater than s; therefore, the individual bearing method is the preferred method for computing resistance at this helix spacing.
3. The cylindrical shear and individual bearing methods will yield similar results for helix spacing close to the value s at which the failure mode transition occurs.

In Figure 3-10, it is shown that the stress field produced by a bearing plate either at the surface or in the interior of an elastic half-space decreases in intensity with distance from the plate. At a perpendicular distance of three times the diameter of the plate, the stress is reduced to a minor level in comparison with the stress at the plate's bearing surface. On this basis, manufacturers started spacing helices three times their diameters apart some thirty years ago, reasoning that helices spaced that far apart would operate independently. Extensive laboratory testing by Bassett (1977) showed such a relation regarding multi-belled grout anchors. The stress field is even smaller than for a plate at the surface, so three-diameter spacing is considered conservative.

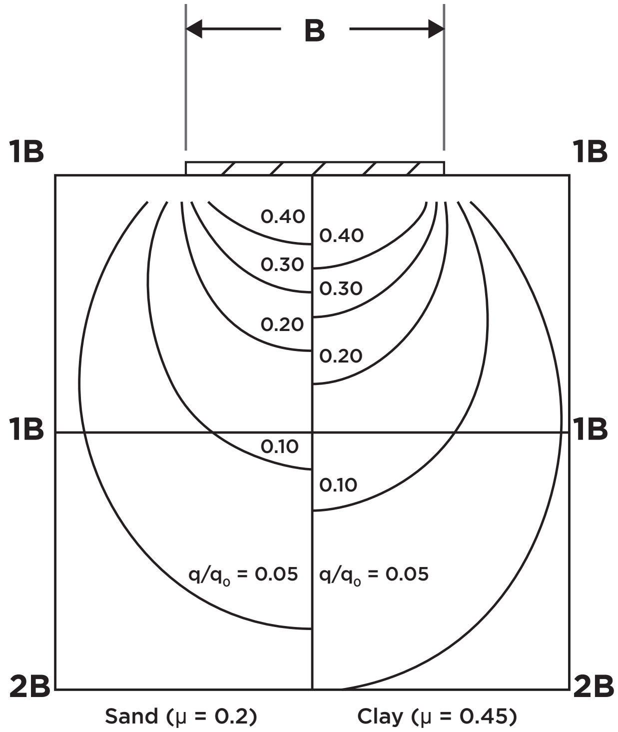



**FIGURE 3-10 :** Bearing Capacity Factors Proposed by Different Authors (Coyle and Castello, 1981, As Taken From Coduto, 1994 by Fellenius, 1999)

Given that Ram Jack helical foundations are manufactured with three-diameter helix spacing, one can use either the cylindrical shear method or the individual bearing method to predict their ultimate resistances. However, the computations for the individual bearing method are significantly simpler than those for the cylindrical shear method, thus *the individual bearing method is the preferred method for predicting ultimate pullout or plunging resistance of Ram Jack helical foundations based on soil strength properties*. The individual bearing method is also specified in the IBC. In calculating the ultimate resistance of each helix, the strength properties of the soil within three diameters above or below the helix should be used for anchors loaded in tension or compression, respectively.

Various investigators have studied the effect of anchor inclination on ultimate resistance. Though most of these studies have related to plates installed as shallow anchors, a few have looked at deep helical anchors (Ghaly and Clemence, 1998; Adams and Radhakrishna, 1977; Trofimenkov and Mariupolskii, 1965). The angle of inclination has been found to have no appreciable effect on the ultimate resistance of deep helical anchors. However, the three-diameter rule for selecting soil

| Blow Count, N (bpf) [blows/300 mm] | Consistency | Unconfined Compressive Strength $q_u$, (psf) | Unconfined Compressive Strength $q_u$, (kN/m$^2$) | Saturated Unit Weight $\gamma_{sat}$ (pcf) | Saturated Unit Weight $\gamma_{sat}$ (kN/m$^2$) |
|---|---|---|---|---|---|
| 0 - 2 | Very Soft | 0 – 500 | 0 - 24.0 | < 100 – 110 | < 15.7 - 17.3 |
| 3 - 4 | Soft | 500 – 1,000 | 24.0 - 47.9 | 100 – 120 | 15.7 - 18.9 |
| 5 - 8 | Medium | 1,000 – 2,000 | 47.9 - 95.8 | 110 – 125 | 17.3 - 19.6 |
| 9 - 16 | Stiff | 2,000 – 4,000 | 95.8 - 192 | 115 – 130 | 18.1 - 20.4 |
| 17 - 32 | Very Stiff | 4,000 – 8,000 | 192 - 383 | 120 – 140 | 15.7 - 22.0 |
| > 32 | Hard | > 8,000 | > 383 | > 130 | > 20.4 |

**TABLE 3-1 :** Fine-Grained Soil Condition and Strength Parameters vs. SPT Blow Count (ASCE, 1996) properties to be used in predicting resistance, given in the preceding paragraph, should be applied along the axis of inclined anchors, not vertically.

The stress created by each helix reduces to an insignificant level within a lateral distance of ¾ to 1 helix diameter. Ram Jack recommends helix screw anchors be spaced laterally at least three times the diameter of their largest helix to preclude capacity reduction due to group action. This requirement applies only at the final depth where the helices are located; where structural considerations require anchors to be closely spaced at the foundation or superstructure connection level, they may be inclined apart slightly to give the necessary lateral spacing at helix depth. For typical anchor embedment depths, this may be done without significant effect on the net resistance of the anchor group.

## *EXAMPLE CALCULATIONS*

**Example 1:** 10" – 12" [254-305 mm] double helix anchor with tip embedment of 12 ft. [3.65 m] in 12 blow count clay soil. Calculate the ultimate tension and compression capacities.

**Solution:** From individual bearing method

$$Q_C = Q_T = \sum[A_H\,(cN_c + \sigma'_v N_q)] + \pi D_s L_s q_s$$

The second term is usually dropped for deep foundations.

$$Q_C = Q_T = \sum[A_H\,(cN_c + \sigma'_v N_q)]$$

$$A_H(254\text{ mm}) = \pi(254)^2/4 = 0.05\text{ m}^2$$

$$A_H(305\text{ mm}) = \pi(305)^2/4 = 0.07\text{ m}^2$$

$c = q_u/2$, from Table 3-1, for blow count of 12,
$q_u = 137$ kN/m$^2$

$c = 68.4$ kN/m²
$N_c = 9$
$N_q = 1$

$$Q_C = Q_T = (0.05 + 0.073)\,(68.4 \times 9)$$

$$Q_C = Q_T = 76.2\text{ kN}$$

**Example 2:** A 10" – 12" [254-305 mm] double helix anchor with tip embedment of 12 ft. [3.65 m] in 12 blow count sand. The water table is and will remain below 12 ft. [3.65 m]. Calculate the ultimate tension and compression capacities.

**Solution:**
$A_H(254\text{ mm}) = 0.05\text{ m}^2$
$A_H(305\text{ mm}) = 0.07\text{ m}^2$
$H(254\text{ mm}) = 3.65\text{ m}$

Ram Jack helical plates are spaced at 3D. Distance between 10” [254 mm] & 12” [305 mm]
= 3 x 254 mm = 762 mm

H(305 mm) = 3.65 m - 0.762 m = 2.88 m

$\gamma_m = 17.4$ kN/m³ (Table 3-2)
$\Phi = 31$ degrees (Table 3-2)
c = 0 (Sand has no cohesion)
$N_q = 21$ (Figure 3-8)
$\sigma'_v$ (254 mm) = 3.65 x 17.4 = 63.6 kN/m$^2$
$\sigma'_v$ (305 mm) = 2.88 x 17.4 = 50.2 kN/m$^2$
$Q_c = Q_T = \sum[A_H(cN_c + \sigma'_v N_q)]$
$Q_c = Q_T = 0.05(63.6 \times 21) + 0.07(50.2 \times 21)$
$Q_c = Q_T = 141$kN

Alternatively, the helix depths may be averaged giving:

$\sigma'_v$ (avg) = 3.27 x 17.4 = 56.9 kN/m$^2$
$Q_c = Q_T = (0.05 + 0.07) \times 56.9 \times 21$
$Q_c = Q_T = 143$ kN

| Blow Count, N (bpf) [blows/300 mm] | Compact-ness | Density Index $I_D$ (%) | Angle of Internal Friction $\Phi$ (deg) | Moist Unit Weight $\gamma_m$ (pcf) | Moist Unit Weight $\gamma_m$ (kN/m³) | Submerged Unit Weight $\gamma_{sub}$ (pcf) | Submerged Unit Weight $\gamma_{sub}$ (kN/m³) |
|---|---|---|---|---|---|---|---|
| 0 - 4 | Very Loose | 0 - 15 | < 28 | < 100 | < 15.7 | < 60 | < 9.42 |
| 5 - 10 | Loose | 16 - 35 | 28 – 30 | 95 – 125 | 14.9 - 19.6 | 55 – 65 | 8.63 - 10.2 |
| 11 - 30 | Medium | 36 - 65 | 31 – 36 | 110 – 130 | 17.3 - 20.4 | 60 – 70 | 9.42 - 11.0 |
| 31 - 50 | Dense | 66 - 85 | 37 – 41 | 110 – 140 | 17.3 - 22.0 | 65 – 85 | 10.2 - 13.6 |
| > 50 | Very Dense | 86 - 100 | > 41 | > 130 | > 20.4 | > 75 | > 11.8 |

**TABLE 3-2 : Coarse-Grained Soil Condition and Strength Parameter vs. SPT Blow Count (after Teng, 1962)**

The change in this case is about 3%; it would decrease with increased embedment depth.

**Example 3:** A 10" – 12" [254-305 mm] double helix anchor with tip embedment of 12 ft. [3.65 m] in 12 blow sand. The highest anticipated water table is a depth of 10 ft. [3.05 m]. Calculate the ultimate tension and compression capacities.

$A_H(254 \text{ mm}) = 0.05 \text{ m}^2$
$A_H(305 \text{ mm}) = 0.07 \text{ m}^2$
$H(254 \text{ mm}) = 3.65 \text{ m}$
$H(305 \text{ mm}) = 2.90 \text{ m}$
$\gamma_m = 17.4 \text{ kN/m}^3$(Table 3-2)
$\gamma_{sub} = 9.5 \text{ kN/m}^3$ (Table 3-2)
$\Phi = 31$ degrees (Table 3-2)
$c = 0$
$N_q = 21$ (Fig 3-8)
$\sigma'_v (254) = 3.05 \times 17.4 + (3.65 - 3.05) \times 9.5$
$\sigma'_v (254) = 58.4 \text{ kN/m}^2$

For soil above water table use moist unit weight; soil below water table use submerged unit weight.

$\sigma'_v (305 \text{ mm}) = 2.90 \times 17.4 = 50.1 \text{ kN/m}^2$
$Q_c = Q_T = 0.05(58.4 \times 21) + 0.07(50.1 \times 21)$
$Q_c = Q_T = 61.32 + 73.65 \approx$ **135 kN**

**Example 4:** A 10" [254 mm] single helix anchor inclined 45 degrees with tip embedment of 14 ft. [4.27 m] in 20 blow sand. The water table will remain below a depth of 15 ft. [4.57 m]. Calculate the ultimate tension and compression capacities.

**Solution:**

$A_H(254 \text{ mm}) = 0.05 \text{ m}^2$
$H(254 \text{ mm}) = 4.27 \sin(45) = 3.02 \text{ m}$
$\gamma_m = 18.8 \text{ kN/m}^3$ (Table 3-2)
$\Phi =$ 33 degrees (Table 3-2)
$c = 0$
$N_q = 27$ (Fig 3-8)
$\sigma'_v (254 \text{ mm}) = 3.02 \times 18.8 = 56.8 \text{ kN/m}^2$
$Q_T = 0.05(56.8 \times 27)$
$Q_T \approx$ **76.7 kN**

## TORQUE CORRELATION METHOD 3.6.3

The torque required to advance a helical pile into the soil is a direct correlation to soil shear strength and capacity. Torque is used in the Torque Correlation Method and provides a field verification of the capacity of a helical pile. It is important to monitor the installation torque and record the final installation torque for every helical pile installed on a project. The Torque Correlation

Method has been widely used since the 1960s. The method gained notoriety based on the study performed by Hoyt and Clemence (1989). Their study analyzed 91 helical pile load tests at 24 different sites within various soil types ranging from sand, silt, and clay soils. They demonstrated the direct correlation of the installation torque of a helical pile to its ultimate capacity in compression or tension. The common denominator discovered from the study was a parameter referred to as the torque correlation factor, $K_t$. The equation is

$$P_u = K_t T$$

Where,

$P_u$ = Ultimate capacity of the helical pile or anchor (lb [kN])
$K_t$ = Empirical torque factor of the central shaft of the pile (ft$^{-1}$ [m$^{-1}$])
T = Final installation torque (ft-lb [m-kN])

$K_t$ is a constant that depends on the diameter of the pile shaft and is independent of the number of helical plates used and size of the plates. Refer to Table A-1 in the appendix for default $K_t$ factors and pile capacities for Ram Jack standard piles. Final installation torque can be measured in the field.

In order to obtain the allowable or service load capacity ($P_a$) of a helical pile, a safety factor of 2 should be applied to the ultimate capacity of the pile. The equation is:

$$P_a = \frac{K_t T}{2}$$

or

$$P_a = \frac{P_u}{2}$$

**FIGURE 3-11 :** Hydraulic Gauge

It is important to note that the pile capacity changes with the unbraced length of the pile.

There are several different methods of measuring torque during the installation of a helical pile. The most common methods used for measuring torque are:

**Hydraulic Gauges:** Ram Jack provides a driver torque chart with each helical driver it recommends. The torque chart provides the basic torque equation for a hydraulic driver. Each driver will have a specific motor displacement, gear ratio, motor efficiency, and gear drive efficiency. Therefore, it is important that the correct torque chart is used with its corresponding driver.
The variable in the equation is the hydraulic pressure within the hydraulic motor. The driver should be 'free spun' prior to installing a helical pile to record the back pressure. This back pressure should be subtracted from the hydraulic pressure reading in order to obtain the effective hydraulic pressure. The torque chart lists the torque of the driver at 250 psi [1724 kPa] increments. The chart also provides a psi/torque factor that can be multiplied by the effective hydraulic pressure that will allow the torque to be determined at any hydraulic pressure. Because mechanical parts can wear with time, the driver and hydraulic gauges should be calibrated annually or whenever torque readings are questionable.


**FIGURE 3-12 : PT-Tracker**

**PT-Tracker:** A PT-Tracker by Marian Technologies measures the pressure differential across a torque motor. A pair of pressure transducers in this device measures the input and back pressure. The inventors of the device provide software for a laptop that converts the pressure differential to a torque reading using torque correlation curves supplied by torque motor manufacturers. Calibration curves from almost all readily available torque motors are preloaded into the software. A digital readout can be purchased with the device wherein the torque correlations are hard-coded onto the firmware within the device itself. They should be calibrated annually or whenever torque readings are questionable.

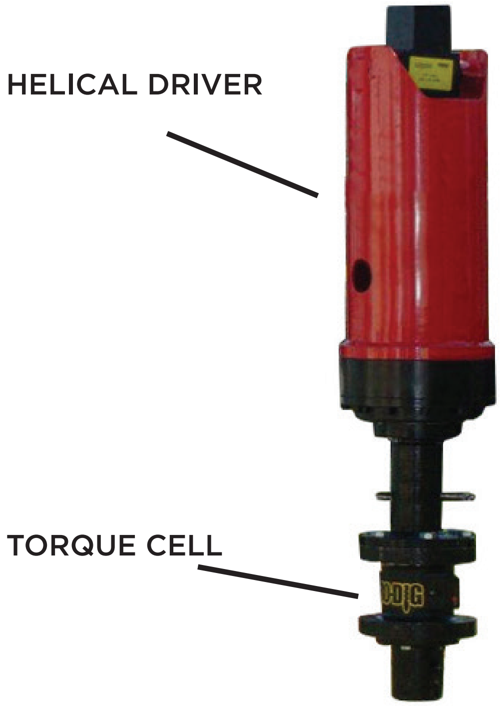



**FIGURE 3-13 : Torque Cell**

**Torque Cell:** These units provide a direct measurement of torque. They contain strain gauges that detect elastic deformation of the steel housing under torsion. Strain is measured based on the principle that a change in length translates to a change in wire diameter and resistance. A data acquisition device detects the change in resistance and displays torque digitally. Torque cells can be placed in-line between the torque motor and the pile shaft. They should be calibrated annually or whenever torque readings are questionable.

**Shear Pin Indicator:** This device consists of two circular plates attached to a central hub. The hub on one side of the device is connected to the torque motor and the hub on the other side of the device is connected to the pile shaft. The plates attached to each hub have a series of 10 to 20 holes arranged along the perimeter. A number of calibrated shear pins can be placed in the holes. Without the pins, the two plates are free to spin with respect to each other. The pins shear at a certain known

**FIGURE 3-14 : Shear Pin Indicator**

torque, providing a one-time measurement. The device provides a direct measurement of torque and can be used to limit the maximum installation torque to ensure the maximum torque rating of the pile shaft isn't exceeded. The device does not provide a continuous torque log or allow the installer to monitor the torque during installation. Shear pins can also get stuck in the plates and be difficult to remove.

Adhesion and/or friction on the pile shaft adds to both torsional and pullout/plunging resistance. This is ignored for square shafts or round shafts less than 2 in. [50.8 mm] in diameter. For larger round shafts, the addition to pullout/plunging resistance is direct, while the addition to torsional resistance is proportional to the radius of the shaft. Any length of shaft having a radius greater than 0.1 ft. (diameter greater than 2.4 in. [61 mm]) will generate pullout/plunging resistance at less than the 10 to 1 default ratio given above for the helices, thus degrading the overall $K_t$. The larger the shaft, the more likely it is to generate appreciable resistance in relation to the helices, and the lower the pullout/plunging resistance to torsional resistance ratio for the shaft (and thus for the overall anchor) will be. The appropriate modification, if any, will depend on the soil strength profile and the anchor's helix configuration, shaft length, and shaft diameter.

**Example 5:**

Find the maximum allowable load-carrying capacity ($P_a$) of a 2 ⅞ in. [73 mm] pile. Assume the pile is fully braced and no eccentricity.

**Solution:**

$P_u = K_t T$

$P_a = P_u$/S.F, (Factor of Safety = 2)

$K_t$ = 29.5 (Appendix, Table A-1)

T = 10.9 kN-m (maximum torque that can be applied to 73 mm pile) Table A-1

$P_u$ = (29.5 * 10.9) = 322 kN

$P_a = P_u$ / S.F.

$P_a$ = 322 / 2 = 161 kN

Maximum allowable load carrying capacity of a 73 mm pile = **161 kN**.

## PILE LOAD TESTS 3.6.4

Load tests are normally performed when the capacity of a deep foundation system is in question or settlement/deflection of a piling system is critical to the performance of the structure being supported. Load tests are also used to verify axial and lateral capacity as well as the empirical torque factor ($K_t$) for a pile shaft size at a specific site.

The pile deflection during the test should be clearly defined by the engineer of record in the project specification prior to the pile test being performed.

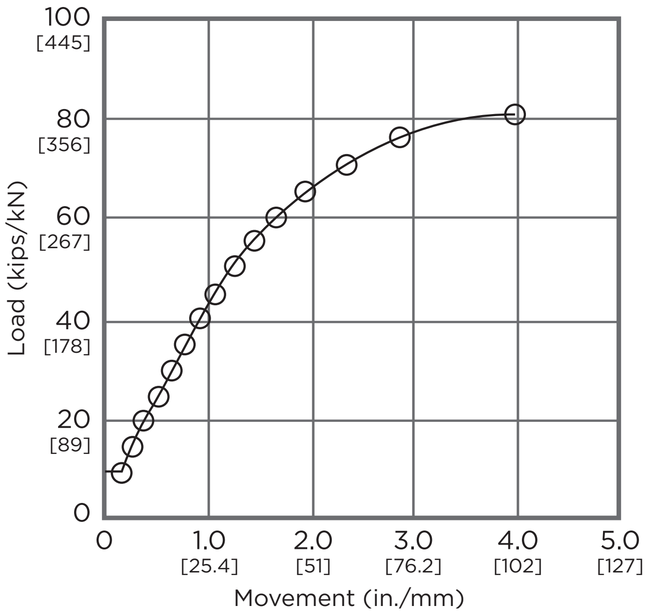



**FIGURE 3-15 : Pile Load Test Taken to Ultimate Capacity**

The ultimate capacity of a helical pile has traditionally been defined as the maximum load achieved that causes constant soil shear or plunging of the pile. Basically, when the helical pile can no longer accept any load. Figure 3-15 shows a typical load diagram of a pile loaded to its ultimate capacity. As shown on the chart, the ultimate capacity of the pile is 80 kips [356 kN]. Applying a safety factor of 2, the allowable or service load would be 40 kips [178 kN] with an anticipated deflection of 1 in. [25.4 mm].

Section 4.4.1.2 of AC358 defines the maximum (ultimate) axial load capacity of a helical pile when the net deflection of the pile exceeds 10% of the helix plate diameter. Net deflection is defined as the total deflection minus shaft elastic shortening or lengthening. For multiple helix configurations on a single pile, the average helix diameter should be used. A brief explanation of compression, tension, and lateral loads tests are provided in the following sections. Please contact Ram Jack's engineering department if a more detailed description of the load test procedures are needed.

**FIGURE 3-16 :** Pile Load Compression Test Setup

## *COMPRESSION TESTS* 3.6.4.1

Compression load tests are typically performed in accordance with the quick test procedure per ASTM D1143. A load frame designed to resist the appropriate test load is constructed over the test pile. Reaction piles must be placed on each end of the reaction frame a minimum of 8 ft. [2.44 m] away from the test pile. The reaction piles typically consist of one or two piles installed to a commutative tension capacity equal to the maximum load that will be applied to the test pile. A typical compression test setup is shown in Figure 3-16.

The hydraulic jack is placed directly on top of the test pile under the load frame. The ASTM standard requires a primary and secondary measurement method is used to monitor the axial deflection of the pile. Typically, two calibrated dial gauges with a +/- 0.001 accuracy tolerance are used as the primary method at the pile head.

The secondary method usually consists of attaching a ruler or tape measure to the pile shaft and monitoring the movement through a site level or survey transit. Care should be taken when cutting off the top of the pile shaft and leveling the load frame. Alignment of the pile shaft, hydraulic jack and load frame is critical to proper measurement of the compression capacity. Misalignment will create an eccentricity on the pile shaft which will induce combined axial and flexural loads. Eccentricities of several millimeters can cause the pile to fail at loads much less than the ultimate capacity of the piles due to increased flexural and buckling stresses.

The pile is slowly loaded in increments. The quick test method requires the test load to be applied in increments of 5% of the anticipated failure load. Each increment is held for a minimum of 4 minutes with readings taken immediately before and after each load increment.
The load is removed from the test pile in 5 to 10 approximately equal decrements to measure soil rebound. Each decrement is held for a minimum of 4 minutes.

### *TENSION TESTS* 3.6.4.2

There are two types of tension tests typically performed on helical anchors, performance tests and proof load tests. Performance tests are typically conducted in general accordance with ASTM D3689 (quick test) or the Federal Highway Administration (FHWA). Performance tests are destructive tests and should not be performed on production anchors. The test load normally ranges between 150 to 200% of the design load. The tension load is applied to the test anchor in increments of 5 to 25% and held for a minimum of 1 to 4 minutes depending on which test criteria is being used. Readings are taken before and after each load increment.

**FIGURE 3-17 :** Performance Tension Load Test Setup on Helical Anchor

The ASTM tension test standard illustrates several different test setups that can be used to test deep foundation systems. Figure 3-17 shows a typical performance tension test setup used for helical anchors. An appropriate sized test beam is placed over the test anchor. Each end of the test beam is supported on cribbing that is spaced a minimum of 8 ft. [2.44 m] away from the test anchor.

An adapter with a solid threaded rod is attached to the top of the test pile. The rod will extend through the test beam and a hollow core hydraulic jack that bears on the top of the test beam. A reference beam is place above or

**FIGURE 3-18 :** Proof Load Test Setup on Helical Tieback Anchor

below the test beam. Two dial gauges are typically used as the primary measuring method to monitor the pile movement as discussed in the compression test section.

Proof load tests are performed on production anchors. The proof load is typically 100-130% of the allowable design load of the helical anchor. The number of proof load tests an engineer may require will vary based on several factors, such as, the factor of safety used in the anchor design, designer or contractor's experience with the anchorage system, and importance of the structure. On smaller projects, it may be more economical to use a higher factor of safety. The costs of the tests may exceed the costs of a few extra anchors.

Proof load tests are performed similar to performance test procedures. However the cribbing is normally spaced closer to the test anchor and the load increments are typically larger. Figure 3-18 shows the setup of a typical proof load test being performed on tieback anchors for a retaining wall.

Tension tests are more economical to perform than compression tests as they don't require the installation of reaction piles. The tension test results will also be more

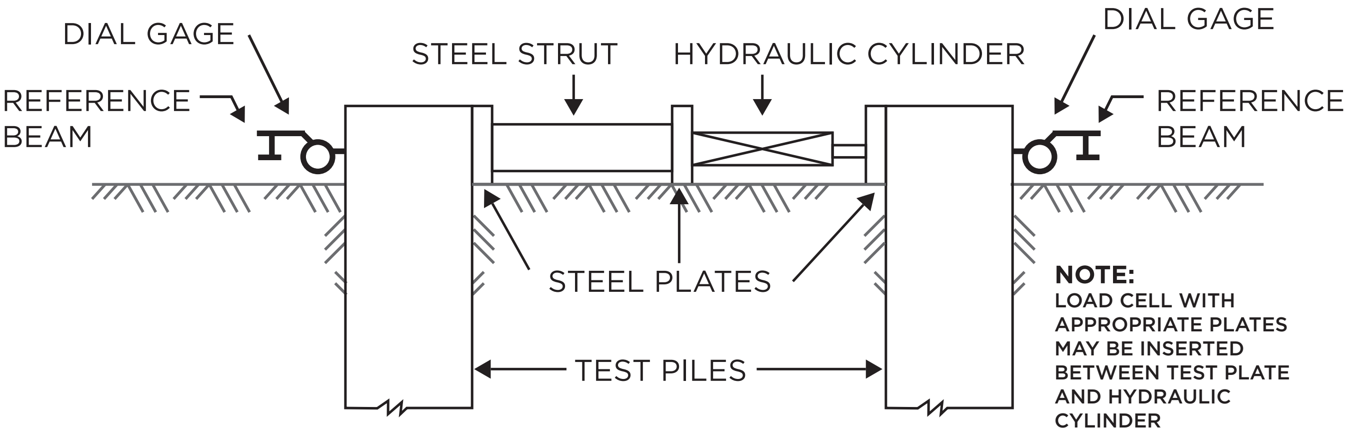



**FIGURE 3-19 : Typical Arrangement for Testing Two Piles Simultaneously**

conservative than performing compression load tests. The helical plates are being pulled against softer soil than the helical plates have sheared through. In a compression test, the helical plates are pushing against stiffer, denser soil than the helical plates were unable to shear through. For these reasons, performance tension tests are typically performed in lieu of compression tests. The only exception is when the design load is near the maximum allowable capacity of the pile. In this case, it may not be wise to lean too far to the conservative side or the testing may require a larger pile to be used. When a project requires a helical pile to be installed at or near their maximum compression load capacity, the piles should be tested in compression.

### *LATERAL TESTS* 3.6.4.3

Lateral load tests for helical piles are typically performed in accordance with ASTM D3966. The ASTM test standard provides several different load test setup procedures. The most common arrangement used is the simultaneous procedure as shown in Figure 3-19. This allows two piles to be tested at the same time by pushing or pulling on the piles which is more economical.

ASTM D3966 provides a standard lateral loading procedure. The procedure tests the pile to 200% of the lateral design load. The piles are loaded in 10 to 25% increments of the lateral design load. The hold time for each increment varies from 10 to 60 minutes. Deflection measurements are taken immediately before and after each load increment. Please reference the ASTM standard for the appropriate load percent and time interval for each increment.

In cases where the goal is to verify the lateral load capacity of a helical pile at a specific site, the test criteria per Section 4.4.2.1 of AC358 (ICC-adopted acceptance criteria for helical pile systems and devices) is used. The criterion requires the shaft of the test piles to extend a minimum of 12 in. [305 mm] above the ground surface. The lateral load should be applied to the pile shaft immediately above the ground surface and at a slow rate to simulate a statically applied load. The test should be conducted as a fixed or free head arrangement in accordance to ASTM D3966. Section 4.4.2.1 of AC358 and Section 1810.3.3.2 of the IBC defines the maximum allowable lateral load as, *"The allowable load capacity shall be half the load required to cause 1 in. [25.4 mm] of lateral deflection at the ground surface".*

When a soils report is available, a lateral load analysis can be performed with a finite element analysis such as LPILE™ from Ensoft, Inc. The software provides finite element modeling of piles in the soil conditions provided

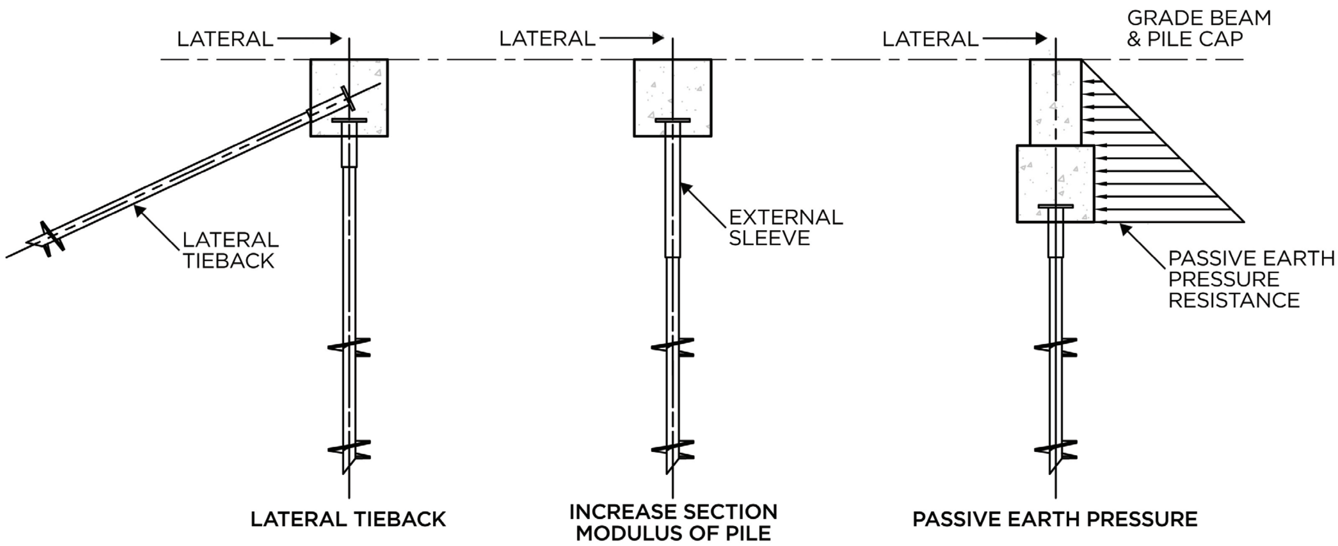



**FIGURE 3-20 : Methods to Increase Lateral Capacity**

in the soils report. The software uses discrete elements to solve the p-y method of analysis. The software has several predefined p-y curves for different soil and rock types. When performing an LPILE™ analysis the geotechnical engineer should provide the soil perimeters (c, $\phi$, $\gamma$ and ks) which are required to be entered into the software.

The software allows moments, axial, and lateral loads to be place at the pile head. The pile can also be modeled as battered, placed in a sloping soil, or with the pile head extended above the ground surface. Many geotechnical engineers use LPILE™ for their analysis and include the required soil strength input parameters for the software in their reports. If a soils report is available, Ram Jack's engineering department can assist in performing an LPILE™ analysis if flexural buckling or lateral deflection is a concern.

## Increasing Lateral Capacity 3.8

Due to the slenderness of the pile shaft, helical piles do not have high lateral resistance when compared to different large diameter deep foundation systems. However, there are several techniques that can be used to increase the lateral capacity of helical piles. The first approach would be to check if the passive earth pressure is enough to resist the excess lateral force. The next would be to place an external sleeve over the pile shaft to increase the moment of inertia (stiffness) of the pile where the maximum bending stresses are occurring. Unlike other helical manufacturers that have upset or through bolted connections, Ram Jack's internal thread connection allows external sleeves with variable lengths to be installed over the pile shaft. An LPILE™ analysis is typically used to design the length of the sleeve. This is a more economical method of increasing the lateral and buckling resistance of the pile. Instead of using a more expensive larger diameter pile, the external sleeve allows you to stiffen only the portion of the pile where it's needed.

Another option, as shown in Figure 3-20, is to use a tieback anchor or jack leg. These can be tied directly to the pile or cast in new concrete. They are typically used at corners of buildings or on thrust blocks and are ideal for resisting seismic loads.

Concrete can also be used by predrilling a hole at each pile location to a sufficient depth and diameter to meet

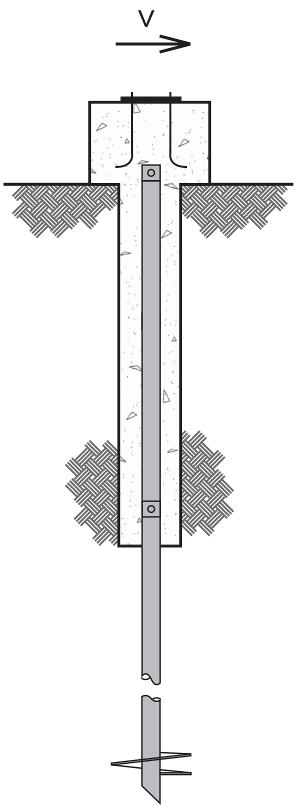



**FIGURE 3-21 :** Concrete Cap (Perko 2009)

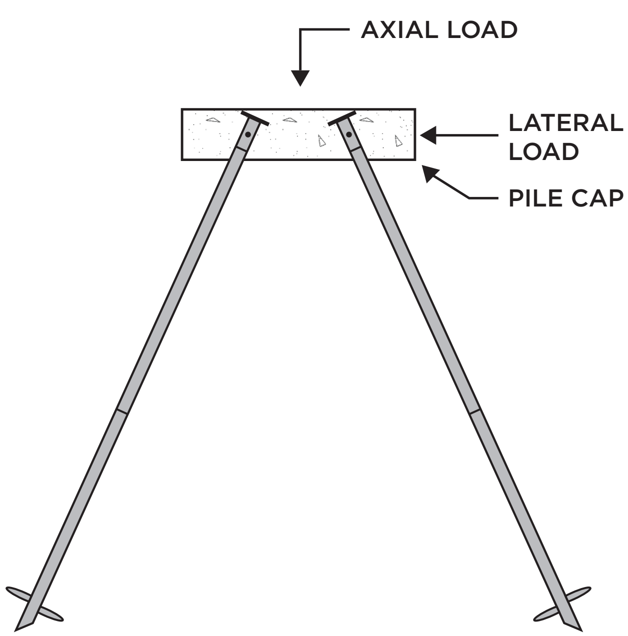



**FIGURE 3-22 :** Battered Piles

the lateral load requirements. Once completed, a helical pile is installed in the center of the hole to a sufficient depth and torque to meet the axial load requirements. A pile cap can be formed at or below the ground surface to facilitate attachment to the structure. Based on the design shear load and the anticipated bending moments, reinforcement can be placed around the pile shaft as necessary. The hole is then filled with concrete. The final product is shown in Figure 3-21.

Battering piles as shown in Figure 3-22 is a technique often used with screen walls, building columns with lateral loads, and in the solar and wind industries. By battering the piles, each pile will contribute a horizontal and vertical component to resist the loads exerted on the pile cap. This method requires that both piles are battered at the same inclination and in opposing directions. In order to take advantage of the horizontal component, the piles must be installed at a higher capacity than the axial compressive load alone. When the lateral load is zero, each pile has an equal share of the compressive load transferred to the vertical component of each pile. If the lateral load is applied to one side of the pile cap, the load on the far side pile increases while the load on the near side pile decreases.

There are an infinite number of angles the piles can be battered depending on the axial and lateral loads that must be resisted. However, by using simple algebraic and trigonometry functions, the optimum angle of batter can be determined to minimize the vertical load exerted on the piles based on the load combination used. Ram Jack's engineering department can assist in performing these calculations to determine the optimum angle of batter for the piles.

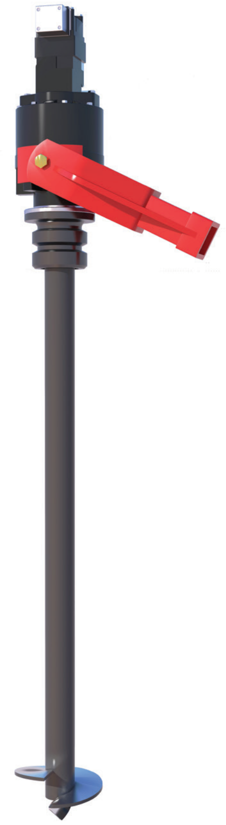
## Structural Strength 3.9

Even though the soil capacity (as determined from the methods listed in Section 2.6) will most often govern the capacity of a helical pile, the structural capacity of a helical pile may control in cases of lateral load, working load, or bracket eccentricity or buckling of the pile shaft. The structural capacity of a helical pile should be determined from an evaluation of the pile shaft, bracket connection (structure and pile), and the helical plates. Many helical manufacturers provide capacities of their helical pile components. However, they don't always provide capacities for how the helical pile components will function as a system. For example, a helical pile with a remedial repair (side load) bracket is defined as laterally unbraced per the International Building Code (IBC). Please reference *Chapter 4 – Designing Helical Piles per the IBC* for a more detailed explanation of when a deep foundation system is considered braced or unbraced.

The structural calculations should be performed in accordance with methods prescribed in the *Manual of Steel Construction* (AISC) and the *International Building Code* (IBC). ICC also provides test procedures for verifying the capacity of individual helical pile components as well as the capacity of these components as they function as a system. These procedures can be found in the acceptance criteria for helical pile systems and devices (AC358).

## Installation Equipment 3.10

One of the big advantages of helical piles is they can be installed by a wide variety of machines. Hand-held units are often used for limited access projects such as inside existing buildings or residential underpinning. Since the torque applied to a helical pile is a direct correlation to the capacity of the pile, the size of the torque motor is the limiting factor as to what size of pile can be installed with a hand-held unit. A torque motor with an output of 7,000 ft-lbs [9.49 kN-m] is about the maximum weight and torque that can be safely used by hand. This limits the maximum pile diameter that can be installed by hand to 2 ⅞ in. [73 mm] diameter or an allowable working load of 31.5 kips [140 kN]. The basic components of a



(a) Hand-Held Unit

(b) Mini-Excavator Used for Installation

**FIGURE 3-23 :** Small Installation Machines hand-held unit consists of a torque motor, torque bar, and independent hydraulic power source. One drawback of using a hand-held unit other than the capacity limit is that crowd cannot be effectively applied during pile installation. However, this is typically not an issue unless the soil is very hard and the leading helix plate has a difficult time biting into the soil. Once the leading helix bites into the soil, the pitch of the helical plates will provide sufficient thrust to pull the pile down to a load-supporting soil stratum.

**FIGURE 3-24 : Track Hoe Used to Install 5 ⅝ in. [143 mm] Helical Piles**

Small machines such as mini-excavators or boxers are sometimes used to raise and lower the torque motor. This reduces the physical strain on the installers and aids in the installation of the piles. These small machines typically do not have enough hydraulic power to operate the torque motor. Therefore, an independent hydraulic power source is still needed. Figure 3-23 shows an example of a hand-held unit and a small machine being used to install helical piles.

Larger machines such as skid steers, backhoes, and track hoes are used to install higher capacity piles as shown in Figure 3-24.

These machines are large enough that they can be fitted with an auxiliary hydraulic power kit. The auxiliary kits can be sized to provide the required hydraulic flow and pressure to operate the torque motor. Some advantages to using the installation machine as the power source:
1) The machine is typically heavy enough that it can provide more crowd when starting the pile in hard soil.
2) The helical piles can be installed with as few as two installers.


**FIGURE 3-25 : Installation of New Construction Piles**

## Installation Procedures 3.11

This section will cover the basic installation procedures for installing helical piles for common applications. The applications will include piles used for new construction, remedial repair of existing structures, underpinning floor slabs, and installing wall tieback anchors.

### NEW CONSTRUCTION PILE INSTALLATION 3.11.1

1. The lead helical section must be installed and successive extensions must be added as needed until the desired torque and capacity are achieved.

2. The pile must be cut squarely to the desired height upon termination of the pile.

3. The new construction bracket is placed over the top of the pile. If the pile is to be used to resist tension forces, the new construction bracket must be embedded the proper distance into the footing or grade beam as required to resist the tension loads as determined by a registered design professional, and must be through-bolted to the pile. Reference Table 4B of ESR-1854 (Section IV) for the proper embedment of the pile into the footing or grade beam for tension resistance.

4. Steel reinforcement bars are placed and tied to the bracket if applicable. The concrete can then be placed in accordance with the construction documents.

## REMEDIAL REPAIR PILE INSTALLATION 3.11.2

1. An area must be excavated immediately adjacent to the building foundation to expose the footing, bottom of grade beam, stem wall or column to a width of at least 24 in. [607 mm], and at least 12 in. [305 mm] below the bottom of the footing or grade beam.

**FIGURE 3-26 : Hydraulic Torque Driver Head**

2. The vertical and bottom faces of the footing or grade beam must, to the extent possible, be smooth and at right angles of each other for the mounting of the support bracket. The surfaces in contact with the support bracket must be free of all dirt, debris, and loose concrete so as to provide firm bearing surfaces.

3. The spread footing or grade beam, if applicable, must be notched to allow the support bracket seat to mount directly under the bearing load of the stem or basement wall.

4. A hydraulic torque driver head is used to install the helical pile. A helical lead section which has helical plates attached is installed first. The helical lead section must be pinned to the rotary torque driver and advanced into the ground by rotating the helical pile. Additional extension shafts must be added as required to advance the pile through unstable soils as required to bear in a load-bearing stratum. The support bracket can be placed on the pile after the lead section and any extensions with helical plates have been embedded into the soil. The remaining pile extensions can be installed through the bracket sleeve.

5. Advancement of the pile will continue until the minimum installation torque is achieved as specified by the torque correlation method to support the allowable design loads of the structure using a torque factor ($K_t$) of 9 $ft^{-1}$ [29.5 $m^{-1}$] for the 2 ⅞-inch-diameter [73 mm] pile; or a $K_t$ value of 7 $ft^{-1}$ [22.9$m^{-1}$] for the 3½-inch-diameter [89 mm] pile. The installation torque must not exceed 8,200 ft-lb [11.1 kN-m] for the 2 ⅞-inch-diameter [73 mm] pile; or 14,000 ft-lb [19.0 kN-m] for the 3½-inch-diameter [89 mm] pile.

6. After piling termination, the excess piling must be cut off squarely at a sufficient height to allow for foundation lifting. If the support bracket has not already been installed, it should be installed now. The support strap assembly must be installed on the support bracket, and the lifting tool placed on the head of the pile.

7. Lifting of the structure or proof loading of the pile can be performed using the hydraulic rams. Lifting of the structure must be verified by the registered design professional to ensure that the foundation and/or superstructure are not overstressed.

8. Once the foundation has been raised and/or stabilized, the nuts on the support strap assembly must be snug-tightened to secure the support strap and bracket to the pile. The lifting tool and hydraulics must then be removed.

9. The excavation must be back-filled and the soil properly compacted. Excess soil and any debris must be removed.

## FLOOR SLAB PILE INSTALLATION 3.11.3

1. A maximum 10-in. diameter [254 mm] hole through the concrete floor slab must be core drilled and an area below the floor slab must be excavated to allow placement of the floor slab bracket.

2. A helical lead section must be inserted into the floor opening and pin-connected to the rotary torque driver. The pile must then be driven into the ground by rotating the helical pile. Additional extension shafts must be added as required to advance the pile through unstable soils as required to bear in a load-bearing stratum. The support bracket can be placed on the pile after the lead section and any extensions with helical plates have been embedded into the soil. The remaining pile extensions can be installed through the bracket sleeve or the bracket can be placed on the pile after the pile installation is terminated.

**FIGURE 3-27 : Helical Installation Through Concrete Floor Slab**

3. Advancement of the pile continues until the minimum installation torque is achieved as specified by the torque correlation method to support the allowable design loads of the structure using a torque factor ($K_t$) of 9 ft$^{-1}$ [29.5 m$^{-1}$] for the 2 ⅞ in. diameter [73 mm] pile. The installation torque must not exceed 8,200 ft-lb [11.1 kN-m].

4. After piling termination, the excess piling must be cut off at a sufficient height to allow for foundation lifting. If the support bracket has not already been installed, it should be installed now. The support strap assembly must be installed on the support bracket, and the lifting tool placed on the head of the pile. Lifting of the structure must be verified by the registered design professional to ensure that the foundation and/or superstructure are not overstressed.

5. Lifting of the structure or proof loading of the pile may be performed using a hydraulic ram or as otherwise approved by the registered design professional and the code official.

6. Once the floor slab has been raised and/or stabilized, the nuts on the support strap assembly must be snug-tightened to secure the support strap and bracket to the pile. The lifting tool and hydraulic ram must then be removed.

7. The excavation must be back-filled and the concrete replaced in accordance with the registered design professional specifications. Excess soil and any debris must be removed.

## WALL TIEBACK ANCHOR INSTALLATION 3.11.4

1. Excavate soil on the earth side of the retaining wall to an appropriate depth where the helical tieback will be installed.

2. Core drill a maximum 6-in. [152 mm] diameter hole through the wall at the tieback location.

3. Insert extension through the hole in the wall and connect to lead section on opposite side of wall.

4. Connect torque driver to other end of the extension, align the tieback to the appropriate inclination as shown on the approved drawings, and begin rotating the tieback into the soil.

5. Advancement of the tieback continues until the minimum installation torque is achieved as specified by the torque correlation method to support the allowable design loads of the structure using a torque factor ($K_t$) of 9 ft$^{-1}$ [29.5 m$^{-1}$] for the 2 ⅞-in. diameter [73 mm] pile. The installation torque must not exceed 8,200 ft-lbs [11.1 kN-m].

6. Cut the tieback shaft off squarely on the earth retained side of the wall. Insert the all-thread connector through the hole in the wall and through-bolt the sleeve of the connector to the tieback shaft.

7. Use low slump grout to fill hole in wall. Place wall channel and wedge washer over all-thread and snug-tighten nut. Do not apply a tension force to the wall until the grout has cured to a sufficient strength as approved by a registered design professional. Please contact the Ram Jack Engineering Division if you have any technical questions or need assistance with the design of any helical piles or anchors.

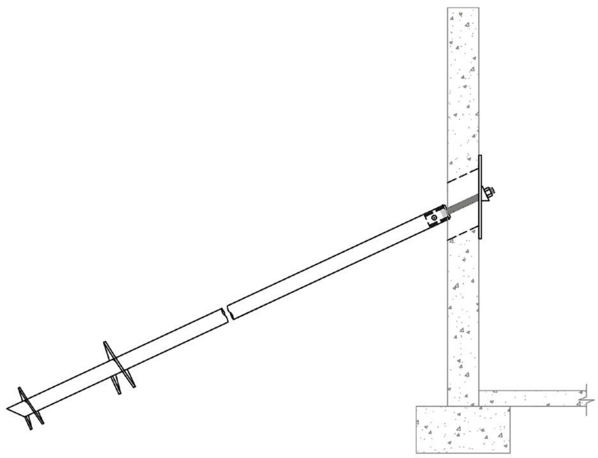


**FIGURE 3-28 :** Wall Tieback Anchor

# 4: DESIGNING HELICAL PILES PER THE IBC

## Introduction 4.1

Helical piles and anchors have been used in construction applications for more than 175 years. However, there was not a uniform standard of testing helical systems until the International Code Council (ICC) approved the Acceptance Criteria for Helical Foundation Systems (AC358) in June 2007. Before an acceptance criterion for helical foundation systems was adopted by ICC, the foundation industry relied on legacy evaluation reports. Only a few helical manufacturers held Legacy Reports, including Ram Jack. Legacy Reports were published by older model building codes but recognized by ICC. Legacy Reports were valuable in providing the capacities of helical foundation system components for engineers and building officials. However, Legacy Reports did not provide capacities of how the components functioned as a system.

AC358 recognizes four helical foundation applications: remedial repair (side load), direct load (new construction), floor slab, and tieback anchors (tension). For each application, there are four structural elements that must be evaluated: bracket capacity and its connection to the structure (P1), shaft capacities with consideration to buckling and eccentricities (P2), helix plate capacity (P3), and soil interaction capacity (P4). Please reference Figures 4-1 and 4-2.

Helical piles are important foundation innovations because of their versatility. Ram Jack was the first helical manufacturer to meet all the requirements of AC358 and receive their Evaluation Service Report (ESR-1854) on February 1, 2011. Ram Jack is also the only helical manufacturer that currently has ICC-recognized piles and brackets listed in their ESR report for all the helical foundation applications listed in AC358.

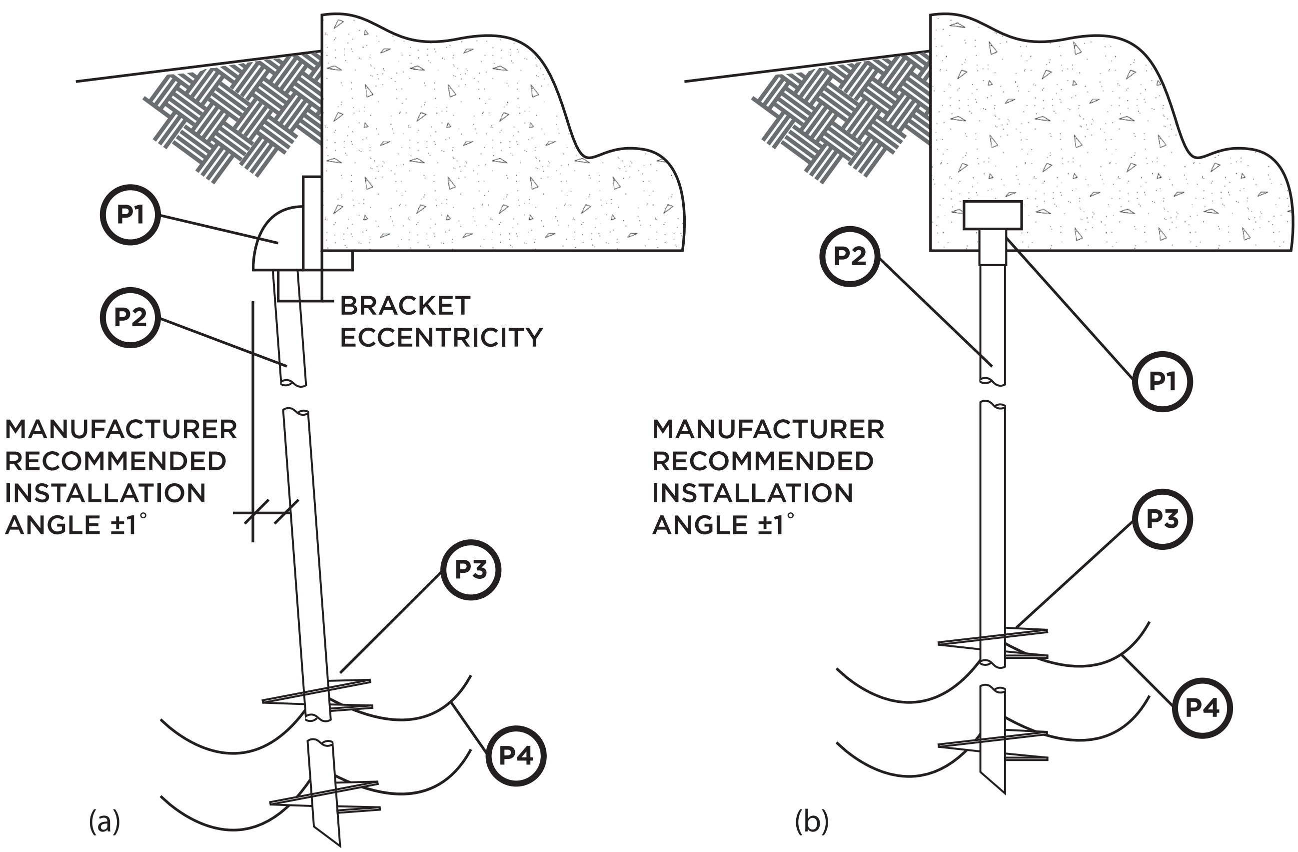



**FIGURE 4-1 :** Helical Foundation Applications – (a) Remedial Repair and (b) Direct Load

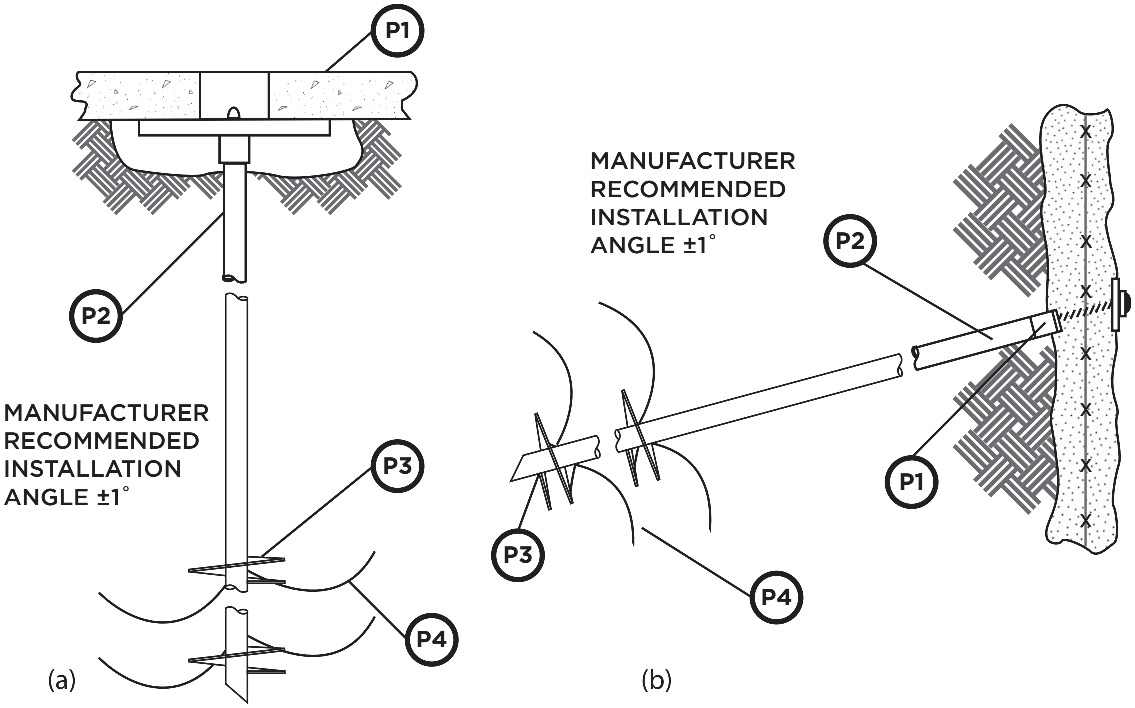



**FIGURE 4-2 : Helical Foundation Applications – (a) Floor Slab and (b) Tieback Anchor**

With the adoption of AC358, ICC also completely revised Chapter 18, "Soils and Foundations", of the 2009 International Building Code (IBC) and added helical piles to the model code. The intent of this chapter is to aid foundation design engineers. This chapter specifically discusses how to design helical piles per the 2009 and 2012 IBC with excerpts and commentary applicable to helical piles. Most of the IBC section excerpts presented here are not new and exist in past IBC codes. This chapter also includes all the applicable code sections so that the designer will have a better understanding of the design process of helical piles and the intent of the code.

## SECTION 1802 - DEFINITIONS

Section 1802.1 defines a helical pile as:

**Helical Pile:** *Manufactured steel deep foundation element consisting of a central shaft and one or more helical bearing plates. A helical pile is installed by rotating it into the ground. Each helical bearing plate is formed into a screw thread with a uniform defined pitch.*

## SECTION 1810 – DEEP FOUNDATIONS

Section 1810 includes all deep foundation elements specifically mentioned in the IBC. Excerpts pertaining to the design criterion of helical piles have been extracted from this section for review and discussion.

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

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

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

Traditionally the IBC has used the section for lateral support to define the minimum unbraced length a deep foundation element must be designed for. The first paragraph of Section 1810.2.1 is new in reference to the previous International Building Codes and is somewhat contradictory to Section 1810.2.2 – Stability, which defines a laterally braced deep foundation element.

Most engineers interpret the code to mean that deep foundation elements that are not clearly defined, as a laterally braced pier or pile in accordance with Section 1810.2.2 must be designed in accordance to the minimum unbraced length as defined in the code. The minimum unbraced length is defined in the second paragraph of Section 1810.2.1.

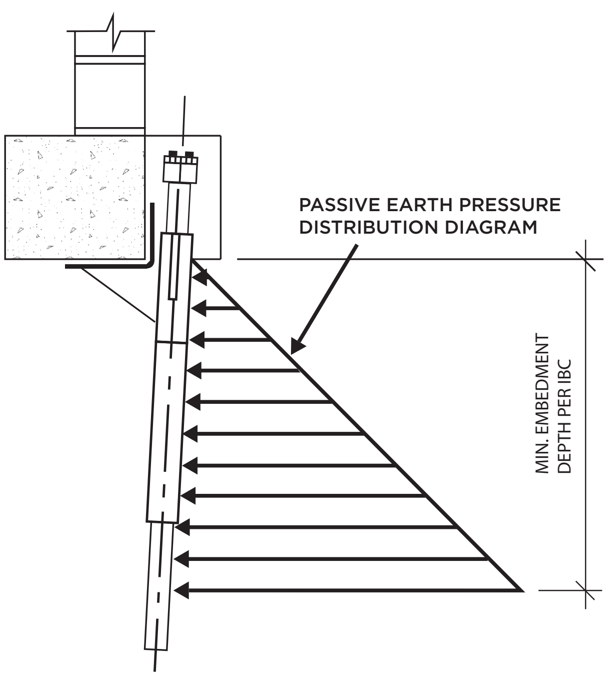



**FIGURE 4-3 : Typical Remedial Repair Bracket Installation**

The first 5 ft. [1.52 m] of a pier or pile placed in a stiff soil should be considered unbraced unless the pier or pile is defined as laterally braced in all directions by the IBC in Section 1810.2.2. In a soft soil, the first ten (10'-0) ft. [3.05 m] is considered unbraced. Most geotechnical engineers classify a soft soil as having an N-value of 4 or less and a stiff soil as having an N-value of 5 or greater. Section 3.11.2.1 of AC358 has also adopted this classification for stiff and soft soils. Section 3.11.2.1 also defines fluid soils as any soil with a Standard Penetration Test blow count (N-value) of zero [weight of hammer (WOH) or weight of rods (WOR)].

Most remedial underpinning performed on existing structures will be defined as an unbraced deep foundation element. A side load bracket or haunch used to transfer structural loads to a pile or pier will not meet the stability or bracing requirements of Section 1810.2.2 of the IBC unless an engineered means of bracing the pile is provided.

Figure 4-3 illustrates a remedial repair bracket installed on an existing foundation. Since an engineered means of bracing is not typically provided with remedial repair brackets, the pile must be designed for the minimum unbraced length as described in Section 1810.2.1 of the IBC. The code is implying that the passive earth pressure distribution along the pile shaft is similar to the lateral soil load on a retaining or basement wall. The soil will not provide sufficient lateral support to the pile shaft to prevent buckling of the shaft until the minimum embedment prescribed by the code has been met. At the minimum embedment depth, the pile shaft can be considered fixed for structural analysis.

As can be seen in Figure 4-3, the pile must also be designed to resist the eccentricity of the bracket. Figure 4-1 shows the moment created from the bracket eccentricity is taken as the distance from the vertical face of the footing or bearing wall to the center of the

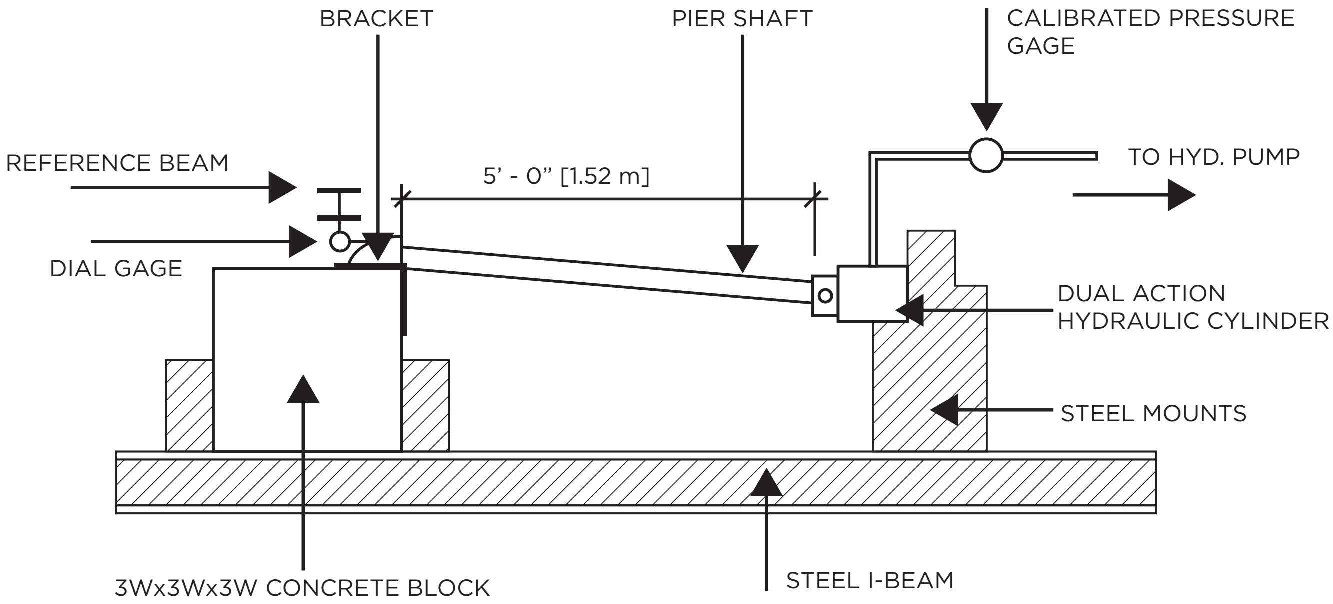



**FIGURE 4-4 : Typical AC358 Remedial Repair (Side Load) Bracket Test Setup**

pile. Calculating capacities of remedial repair brackets can be extremely difficult when taking the interactions of bearing seats, gussets, sleeves, welds, and friction into account. Therefore, ICC Evaluation Services included a full scale test procedure in AC358 for remedial repair brackets as shown in Figure 4-4. The test setup complies with the intent of Section 1810.2.1 of the IBC by including the minimum 5-ft. [1.52 m] unbraced length of the pile shaft. It also provides true bracket capacities that would be obtained in the field as test procedure measures the capacity of the bracket and pile components as an interactive system. Ram Jack has several brackets and their capacities that are recognized by ICC in Table 1 of ESR-1854. A copy of ESR-1854 can be found in Section IV of this Technical Manual.

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

*Deep foundation elements supporting walls shall be placed alternately in lines spaced at least 1 foot [305 mm] apart located symmetrically under the center of gravity of the wall load carried, unless effective measures are taken to provide for eccentricity and lateral forces, or the foundation elements are adequately braced to provide for lateral stability.*

**Exceptions:**

1. *Isolated cast-in-place deep foundations elements without lateral bracing shall be permitted where the least horizontal dimension is no less than 2 feet [610 mm], adequate lateral support in accordance with Section 1810.2.1 is provided for the entire height and the height does not exceed 12 times the least horizontal dimension.*

2. *A single row of deep foundations without lateral bracing is permitted for one- and two-family dwellings and*

*lightweight construction not exceeding two stories above grade plane or 35 feet [10.7 m] in building height, provided the centers of the elements are located within the width of the supported wall.*

As previously stated, Section 1810.2.2 defines a fully braced deep foundation element. In previous International Building Codes, the code always preceded the definition of an unbraced foundation element with that of a braced foundation element which seems a more logical order. The reason for the change in the 2009 and 2012 IBC is unclear.

For stability of the structure, Section 1810.2.2 requires all deep foundation elements to be braced to provide lateral stability in all directions. This section goes on to provide four (4) prescriptive conditions where piles can be considered laterally braced:

1. Three or more piles connected by a rigid cap, provided the piles are located in radial directions from the centroid of the group not less than 60 degrees apart. (Figure 4-5a)

2. A two pile group connected by a rigid cap is braced along the axis connecting the two piles. A wall or grade beam would have to be connected perpendicular to the cap in order to provide bracing in the opposing direction. (Figure 4-5b)

3. Piles supporting walls shall be staggered on each side of the wall at least 1 ft. [305 mm] apart and located symmetrically under the center of gravity of the wall provided measures are taken to provide for eccentricity, lateral forces, or piles to be adequately braced to provide lateral stability. (Figure 4-5c)

4. A single row of piles without lateral bracing is permitted for one- and two-family dwellings not exceeding two stories or 35 ft. [10.7 m], provided the centers of the piles are located within the width of the supported wall.

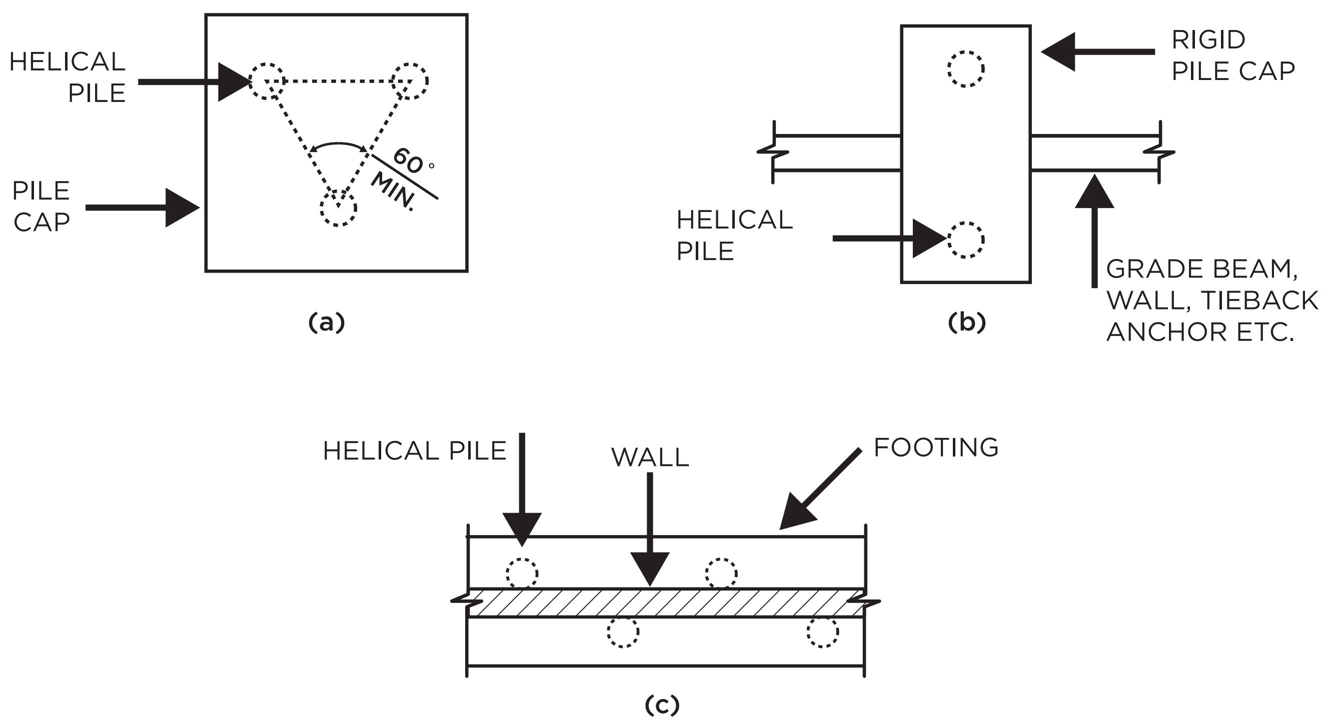



**FIGURE 4-5 : Prescriptive Lateral Bracing Requirements per Section 1810.2.2 of IBC**

**FIGURE 4-6 : Lateral Bracing – (a) Helical Tiebacks and (b) Diagonal Bracing**

If none of the above four requirements are met, the pile is considered unbraced by default. Therefore, an engineered means must be used to brace the structure and piles if the piles cannot resist the buckling stresses due to the minimum unbraced length defined in Section 1810.2.1. There are several different means of providing lateral bracing to a pile. The most common methods are using helical tiebacks, a strong-back (waler) connected to several pile shafts, or diagonal bracing to an adjacent pile. (Figure 4-6)

**1810.2.5 Group Effects.** *The analysis shall include group effects on lateral behavior where the center-to-center spacing of deep foundation elements in the direction of lateral force is less than eight times the least horizontal dimension of the element. The analysis shall include group effects on axial behavior where the center-to-center spacing of the deep foundation elements is less than three times the least horizontal dimension of an element.*

Due to the slenderness of the central shaft of a helical pile, the group effect on the minimum horizontal spacing is much larger than the code allows. For axial loaded helical piles, Section 6.7 of AC358 requires that a group efficiency analysis must be performed on helical piles with a center-to-center spacing less than three times the largest helical plate diameter of the pile. For example, a helical pile with an 8”-10”-12” [2.44 - 3.05 - 3.66 m] helical plate configuration on the lead section must not be spaced closer than 3 ft. [0.91 m] center-to-center without performing a group efficiency analysis.

It’s important to note that it isn’t the center-to-center spacing of the piles that is required to be spaced 3D apart; it’s the center-to-center spacing of the largest helical plates on the piles. In most cases, piles can be battered slightly in opposing directions to obtain the helical plate spacing to avoid group effects. (Figure 4-7)

## *SECTION 1810.3 – DESIGN AND DETAILING*

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

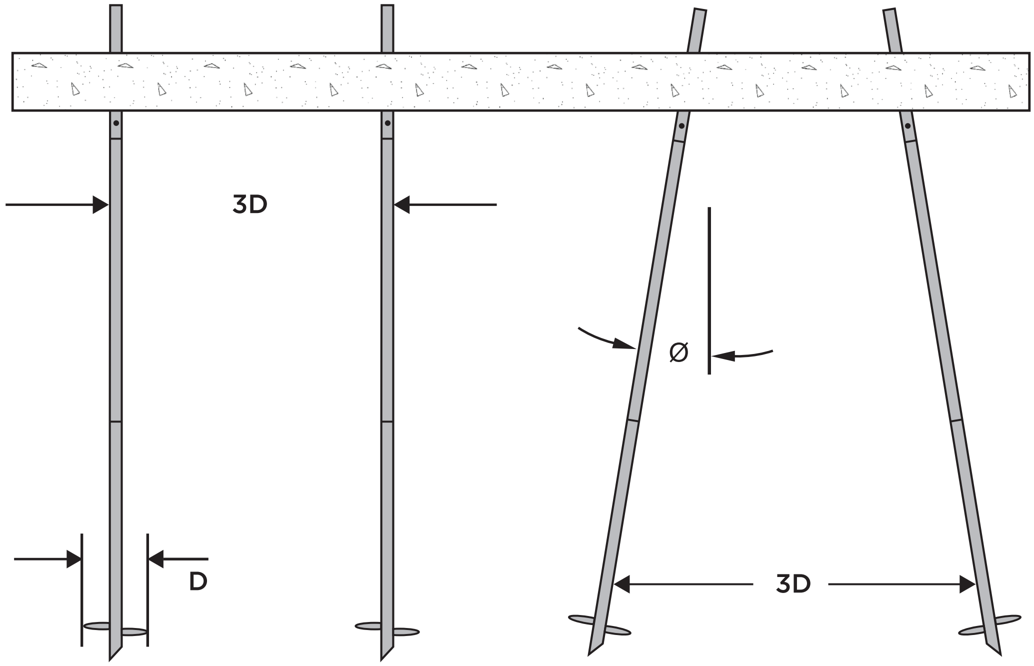



**FIGURE 4-7 : Minimum Helical Pile Spacing**

Please reference Tables A-1 through A-6 in the Appendix for mechanical strengths, section properties, and torque ratings of all the Ram Jack helical standard piles and allowable capacities of common brackets. This information as well as AutoCAD drawings and specifications are available online by clicking on the engineering portal at www.ramjack.com.

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

All of Ram Jack's standard central pile shafts (2 ⅜ in. [60 mm], 2 ⅞ in. [73 mm], 3 ½ in. [89 mm] and 4 ½ in. [114 m]) are a special manufactured ASTM-A500 Grade C steel and have a minimum yield strength of 65 ksi [450 Mpa] and a minimum tensile strength of 80 ksi [552 Mpa]. All the helical plates are manufactured from ASTM-A36 steel plate specifications but have a minimum yield strength of 50 ksi [344 Mpa].

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

$$P_a = 0.5\ P_u \quad \text{(Equation 18-4)}$$

*where $P_u$ is the least value of:*

1. *Sum of the areas of the helical bearing plates times the ultimate bearing capacity of the soil or rock comprising the bearing stratum.*
2. *Ultimate capacity determined from well-documented correlations with installation torque.*
3. *Ultimate capacity determined from load tests.*
4. *Ultimate axial capacity of pile shaft.*
5. *Ultimate capacity of pile shaft couplings.*
6. *Sum of the ultimate axial capacity of helical bearing plates affixed to pile.*

An explanation and summary of each of the six design criteria required per the IBC for helical pile design have been listed below to better explain the design process and intent of the code.

**Item 1** is in reference to the Individual Bearing Method. This method was discussed previously in Section 3.6.2 of Chapter 3. The method requires prior knowledge of the soil properties at the site via a soils report or boring logs. Please note that most soil reports only report the allowable bearing capacity of a soil or stratum. This allowable capacity normally has a safety factor of two or three applied. Applying another factor of safety of two per the IBC would be extremely conservative.

Typical helical plate sizes are 8", 10", 12", 14" and 16" [2.44, 3.05, 3.66, 4.27, 4.88 m] in diameter. The maximum number of helical plates placed on a single pile is normally set at six (6). The central area of the shaft is typically omitted from the effective area of the helical plate when using the Individual Bearing Method.

**Item 2** is in reference to the Torque Correlation Method. The Torque Correlation Method is an empirical method that distinguishes the relationship between helical pile

| Material Type and Condition | Maximum Allowable Stress [a] |
|---|---|
| Structural steel in compression **Helical Piles** | $0.6\ F_y \leq 0.5\ F_u$ |
| Structural steel in tension **Helical Piles** | $0.6\ F_y \leq 0.5\ F_u$ |

**TABLE 1810.3.2.6 Allowable Stress for Materials Used in Deep Foundation Elements**

*a. $F_y$ is the specified minimum yield stress of structural steel; $F_u$ is the specified minimum tensile stress of structural steel.*

capacity and installation torque and has been widely used since the 1960s. The process of a helical plate shearing through the soil or weathered bedrock in a circular motion is equivalent to a plate penetrometer test. The method gained notoriety based on the study performed by Hoyt and Clemence (1989). Their study analyzed 91 helical pile load tests at 24 different sites within various soil types ranging from sand, silt, and clay soils. They demonstrated the direct correlation of the installation torque of a helical pile to its ultimate capacity in compression or tension. The common denominator discovered from the study was a parameter referred to as the torque correlation factor, $K_t$.

The equation is:

$$P_u = K_t T$$

Where:
$P_u$ = ultimate capacity of the helical pile or anchor (lb [kN])
$K_t$ = empirical torque factor of the central shaft of the pile ($ft^{-1}$ [$m^{-1}$])
T = final installation torque (ft-lb [kN-m]).

It's important to point out that the tests analyzed by Hoyt and Clemence (1989) were in tension. It was shown in sub-sequential studies that the tension capacity of helical piles was 16 to 33% less than the measured compression capacity. The difference is attributed to the fact that the lead helical plate is bearing on relatively undisturbed soil in compression applications. In tension applications, the leading and trailing helical plates are bearing on soil affected by the installation of the helical plates. It has become common practice to use the same torque correlation factor for a helical pile of the same size for tension and compression and ignore the slight increase in compression capacity. This creates a more conservative compression capacity for helical piles when compared to the Individual Bearing Method. Also, unlike the Individual Bearing Method, the number of helical plates on a pile is completely independent of the piles capacity based on the Torque Correlation Method. Please reference Section 3.6.3 of this manual for additional information on the Torque Correlation Method.

Ram Jack uses four (4) standard central pile shaft diameters. The recommended default torque correlation factor, $K_t$, for each diameter is listed in Table 4-1.

**Item 3** is in reference to IBC approved load tests. The approved compression load test is ASTM D1143 Standard Test Methods for Deep Foundations Under Axial Compressive Load. The approved tension load test is ASTM D3689 Standard Test Methods for Deep Foundations Under Axial Tensile Load. Please reference Sections 3.6.4 through 3.3.4.3 of this manual for a summary of the test procedures.

There are three basic reasons to load test a helical pile:

1. When a project requires a more aggressive design approach. Please note that the default $K_t$ values shown in Table 4-1 are conservative and are based on numerous load test analyses. A load test would provide the ultimate load capacity and actual $K_t$ value for the specific project site.

2. Where the design compressive loads are greater than those determined using the allowable stresses specified in Section 1810.3.2.6.

3. Where the design load for any deep foundation element is in doubt.

**Item 4** is in reference to the structural axial capacity of the central pile shaft. The structural calculations should be performed in accordance with methods prescribed in the *Manual of Steel Construction* (AISC) and the *International Building Code* (IBC). The structural analysis should take into account any applicable unbraced length per Section 1802.1, pile eccentricities, as well as flexural and bending stresses. The pile eccentricity for helical piles used with remedial brackets is normally measured from the vertical face of the wall load or the concrete footer for a column. Ram Jack pile materials are manufactured from specialty materials. Please reference Table A-5 of the Appendix for applicable mechanical strengths and section properties. Allowable loads for common helical piles and brackets can also be found in Tables A-1 through A-3 of the Appendix.

**Item 5** Ram Jack's helical pile couplings are designed to exceed the structural capacity of the central pile shaft based on the Torque Correlation Method. Please reference Table 4-1 for maximum allowable torque ratings of Ram Jack's standard pile shafts.

| **Pile Diameter** | **2 ⅜"** [60 mm] | **2 ⅞"** [73 mm] | **3 ½"** [89 mm] | **4 ½"** [114 mm] |
|---|---|---|---|---|
| **$K_t$** (default) | 10 [32.8] | 9 [29.5] | 7 [22.9] | 6 [19.7] |
| **Torque Rating**, $T$ (ft-lb [kN-m]) | 4,000 [5.42] | 8,000 [10.9] | 14,000 [19.0] | 23,000 [31.2] |
| **Ultimate Capacity**, $P_u$ (lb [kN]) | 40,000 [178] | 72,000 [320] | 98,000 [436] | 138,000 [614] |

**TABLE 4-1 : Default Torque Correlation Factors and Torsional Ratings** [a]

*a. The structural capacity of the pile must be checked per item 4 of Section 1810.3.3.1.9 to ensure it does not govern the limit state of the pile. Ram Jack pile materials are manufactured from specialty materials. Please reference Table A-5 in the Appendix for applicable mechanical strengths and section properties.*

**Item 6** is in reference to the structural capacity of the helical plate welded to the central shaft of the pile. Table A-6 in the Appendix provides a complete list of strength ratings for each helical plate Ram Jack manufactures based on the plate's diameter, thickness, and central shaft it is attached to.

**1810.3.3.2 Allowable Lateral Load.** W*here required by the design, the lateral load capacity of a single deep foundation element or a group thereof shall be determined by an approved method of analysis or by lateral load tests to at least twice the proposed design working load. The resulting allowable load shall not be more than one-half of the load that produces a gross lateral movement of 1 inch [25 mm] at the lower of the top of the foundation element and the ground surface, unless it can be shown that the predicted lateral movement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any element to be loaded beyond its capacity.*

Helical piles are slender deep foundation elements that typically range from 2 ⅜” [60 mm] to 4 ½” [114 mm] in diameter. When helical piles are subjected to lateral loads, a lateral load analysis should be performed using a program such as LPILE™ or equivalent or field tested on the project site as described in Section 1810.3.3.2. When performing an LPILE™ analysis the geotechnical engineer should provide the soil perimeters (c, $\phi$, $\gamma$ and ks) which are required to be entered into the software.

When the lateral loads are large there are several methods available to resist the forces. The moment of inertia (stiffness) of the upper portion of the pile can be increased; a pile cap can be used to create a couple moment; the piles can be battered; or a helical tieback anchor can be connected to the foundation system or vertical helical pile. Please reference Section 3.8 in Chapter 3 for a list of common methods used to increase the lateral capacity of helical piles.

**1810.3.5.3 Steel.** *Steel deep foundation elements shall satisfy the requirements of this section.*

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

Reference Appendix Tables A-1 and A-6 for Ram Jack’s standard central shaft and helical plate sizes as well as strength and section properties.

**1810.3.11 Pile Caps.** *Pile caps shall be of reinforced concrete, and shall include all elements to which vertical deep foundation elements are connected, including grade beams and mats. The soil immediately below the pile cap shall not be considered as carrying any vertical load. The tops of the vertical deep foundation elements shall be embedded not less than 3 inches [76 mm] into pile caps and the caps shall extend at least 4 inches [102 mm] beyond the edges of the elements. The tops of the elements shall be cut or chipped back to sound material before capping.*

Section 1810.3.11 applies to all deep foundation elements. The minimum embedment depth of 3 in. [76 mm] is intended for compression loads only. It’s important to point out that where a helical pile is required to resist tension loads or a combination of tension and compression, the embedment depth of the helical pile may need to be increased or applicable nelson studs, weldable deformed bars, concrete anchor bolts, or headed studs will be needed to provide a tension connection between the concrete and helical pile. If the depth of the concrete pile cap, grade beam, or mat foundation is sufficient, the embedment depth of the helical pile could be analyzed as a single headed stud per ACI 318 Appendix D.

Tension load capacities for Ram Jack’s standard new construction brackets for 2 ⅞” [73 mm] and 3 ½” [89 mm] diameter piles can be found in Table 3B of ESR-1854. A copy of ESR-1854 has been included in Section

IV. Table 3B provides ICC rated tension capacities for the 4075, 4079, and 4076 new construction brackets based on varying embedment depths. The tension capacities are based on localized limit states such as mechanical strength of steel components, concrete bearing, and punching shear capacity.

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

The torque ratings of Ram Jack's helical piles can be found in Table 4-1.

**1810.4.12 Special Inspection.** *Special inspections in accordance with Sections 1704.8 and 1704.9 shall be provided for driven and cast-in-place deep foundation elements, respectively. Special inspections in accordance with Section 1704.10 shall be provided for helical piles.*

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

Per Section 1704.10 item 3 under exceptions, *"Unless otherwise required by the building official, special inspections are not required for Group U occupancies that are accessory to a residential occupancy including, but not limited to, those listed in Section 312.1."*

## *SECTION 312 UTILITY AND MISCELLANEOUS GROUP U*

**312.1 General.** *Buildings and structures of an accessory character and miscellaneous structures not classified in any specific occupancy shall be constructed, equipped and maintained to conform to the requirements of this code commensurate with the fire and life hazard incidental to their occupancy. Group U shall include, but not be limited to, the following:*

- *Agriculture buildings*
- *Aircraft hangers, accessory to a one- or two-family residence (see Section 412.5)*
- *Barns*
- *Carports*
- *Fences more than 6 feet [1829 mm] high*
- *Grain silos, accessory to a residential occupancy*
- *Greenhouses*
- *Livestock shelters*
- *Private garages*
- *Retaining walls*
- *Sheds*
- *Stables*
- *Tanks*
- *Towers*

Please feel free to contact Ram Jack's Engineering Division should you have any questions or need technical assistance when designing a deep foundation system with helical piles.

# 5: RAM JACK DRIVEN STEEL PILES

## Introduction 5.1

Driven steel piles are sometimes referred to as push or resistance piles. They are used to underpin the foundation of an existing structure. The piles are pushed into the soil with hydraulic cylinders by using the weight of the structure as a reaction force. Each pile is installed individually thereby utilizing the maximum resistance of the structure as a reaction force. The piles are advanced into the soil until the required resistance is reached by bearing the pile on rock or other suitable bearing stratum. Once a pile has been installed, the entire load is removed from the pile before installation of the adjacent pile begins. Once all the piles have been installed in a given area, the brackets attached to the piles are utilized to raise the foundation of the structure (if required) and stabilize it against future settlement.

Ram Jack manufactures three different sizes of driven steel piles, 2 ⅜" [60 mm], 2 ⅞" [73 mm] and 3 ½" [89 mm] diameters. Ram Jack's driven piles have four primary components (Figure 5-1).

1. Bracket
2. Guide Sleeve
3. Piling
4. Starter

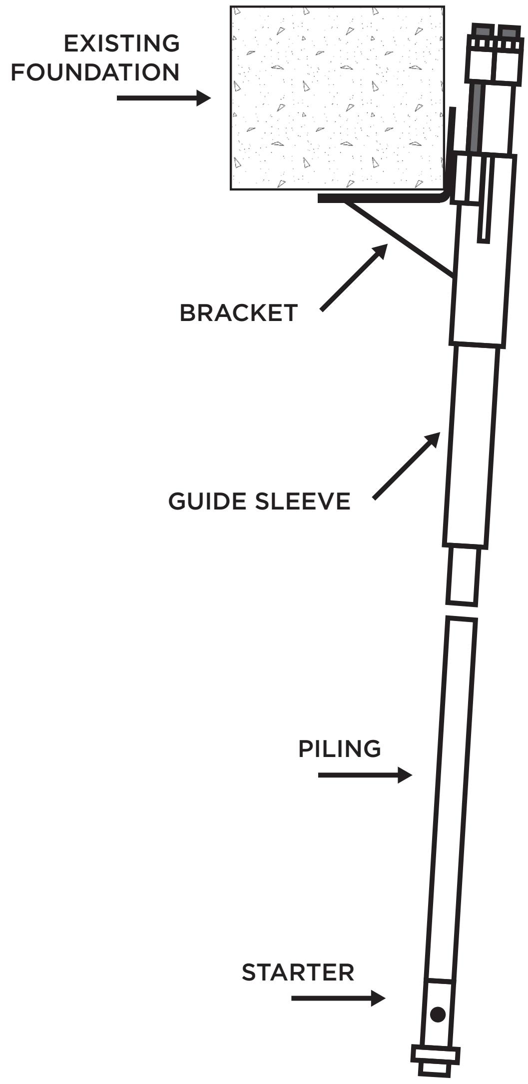



**FIGURE 5-1 : Primary Driven Steel Pile Components**

## Driven Steel Pile Components 5.2

### BRACKETS 5.2.1

Side load brackets as shown in Figure 5-1 are typically used in conjunction with driven steel piles. Ram Jack manufactures a wide assortment of brackets that can be used with the driven steel pile based on the foundation type and structural loads it will be supporting. The brackets are designed to support tensile and compressive loads that are not concentric with the primary axis of the pile shaft. Rotational moments which are caused by load eccentricity are subdivided into two components, bracket eccentricity and structural eccentricity. The structural strength of the bracket, pile shaft, and their connection must resist the stresses induced by the bracket eccentricity. The structural eccentricity varies with application and is normally resisted by the internal strength of the structure to which the bracket is attached. Therefore, resistance to structural eccentricity is generally determined on a case-by-case basis.

There are basically two types of driven pile steel brackets: concrete-bearing as shown in Figure 5-1 and shear plates. Each bracket has a bracket sleeve attached to it which the pile is installed through. The brackets which are concrete-bearing have a flat steel bearing plate on the bracket that fits underneath the foundation being supported. The bearing plate accepts the compression force and transfers the force through the bracket components and to the pile. The size of the bearing plate can vary with different brackets. However, the bearing plates on Ram Jack's standard brackets typically have 90 $in^2$ [0.06 $m^2$] of bearing area. The concrete bearing surface is assumed to be uniform on the bearing plate. In some cases where the bottom of the foundation is curved or jagged, a non-shrink grout is used to provide a uniform surface. The allowable concrete bearing stress of the foundation is limited to $0.35f'c$ for Allowable Stress Design (ASD) and $0.55f'c$ for Load Resistance Factored Design (LRFD). For example, a foundation with a minimum concrete compressive strength of 3,000 psi [20.7 Mpa] will have an allowable uniform concrete bearing stress of 1,050 psi [7.24 Mpa] (ASD) and 1,650 psi [11.4 Mpa] (LRFD).

Driven steel brackets with shear plates are fastened to the side face of the foundation with concrete anchors. These brackets are generally used when underpinning foundations with deep foundation walls. The load is transferred to the bracket by the shear resistance of the concrete anchors. Therefore, these brackets can only be used in conjunction with structurally sound concrete. Due to the variance in load requirements, the shear plate and anchor bolts are custom-designed and manufactured by Ram Jack on a project-by-project basis.

## GUIDE SLEEVE 5.2.2

Ram Jack was the first foundation company in the industry to develop and patent an external guide sleeve with their piling system. During the early development of the guide sleeve, Ram Jack was looking for a method or component to guide the driven pile on a straight and true trajectory as it rifled through unstable soil. Thus, the name guide sleeve was given to the new component. The guide sleeve is installed through the bracket sleeve and the driven pile shaft is then installed through the guide sleeve.

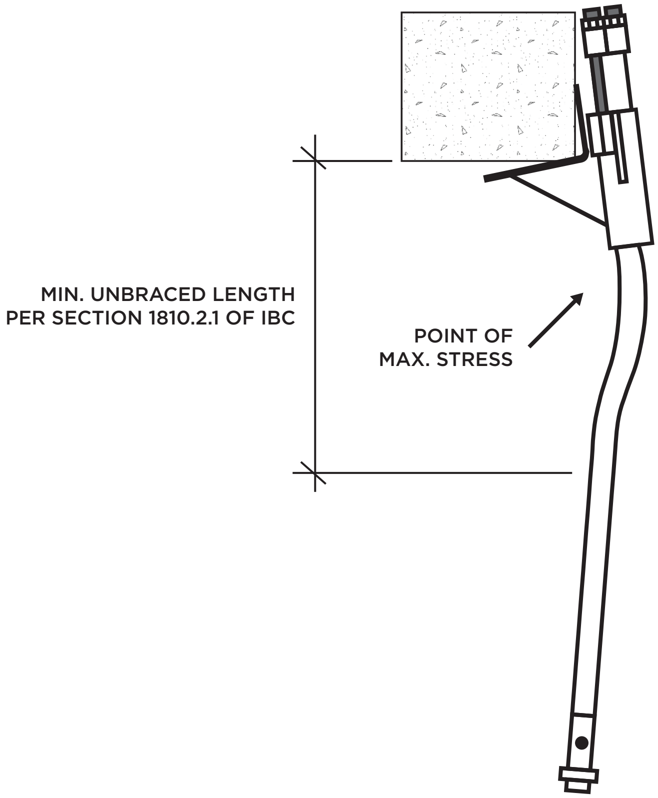



**FIGURE 5-2 : Buckling Mode of a Typical Piling with No Guide Sleeve**

Even though the primary purpose of the guide sleeve was to create a rifle barrel for the driven pile, it also increases the moment of inertia (stiffness) of the pile shaft where the maximum bending stresses are occurring. The maximum bending stresses in the pile shaft occur at the bottom of the bracket sleeve. Figure 5-2 illustrates the typical failure mode. By increasing the stiffness of the pile at its maximum stress point, it greatly increases the capacity of a smaller diameter pile which is more economical for the client.

| Pile Diameter | Guide Sleeve | Ultimate Capacity (kip) | |
|---|---|---|---|
| | | No Sleeve | With Sleeve |
| 2 ⅜" [60 mm] | 2 ⅞" [73 mm] x 4'-0 [1.22 m] | 14.5 [64.5 kN] | 38.0 [169 kN] |
| 2 ⅞" [73 mm] | 3 ½" [89 mm] x 4'-0 [1.22 m] | 40.6 [181 kN] | 67.3 [299 kN] |
| 3 ½" [89 mm] | 4 ½" [114 mm] x 4'-0 [1.22 m] | 45.2 [201 kN] | 110.0 [489 kN] |

**TABLE 5-1 : Ultimate Driving Force of Driven Piles with 5-foot Unbraced Length**

*Note: Capacities ratings for 2 ⅞" [73 mm] and 3 ½" [89 mm] brackets are in ESR-1854*

A driven pile with a side load bracket is defined as an unbraced system per the IBC. The system also needs to meet the same stability and lateral support criteria that was discussed in Chapter 4 (Sections 1810.2.1 and 1810.2.2 of the IBC). Assuming a minimum 5-foot [1.52 m] unbraced length for the driven pile, the ultimate (maximum) driving force capacities of Ram Jack's driven piles are shown in Table 5-1. The table represents the maximum driving force that can be applied to a driven pile with typical bracket eccentricities included. All of Ram Jack's driven piles include a standard 4-foot [1.22 m] long guide sleeve. The capacity of a driven pile without a guide sleeve has been provided for comparison purposes only.

As can be seen in Table 5-1, the ultimate capacity of a 2 ⅞" [73 mm] diameter pile is essentially the same as a 3 ½" [89 mm] diameter pile. The 3 ½" [89 mm] pile is inherently stronger in bending than a 2 ⅞" [73 mm] pile. The difference is the eccentricity. Because of the larger diameter of the 3 ½" [89 mm] pile, the eccentricity on the pile increases which also increases the bending stresses the pile shaft must resist. Again by increasing the moment of inertia at the point of maximum stress, a higher driving force can be applied to the driven pile which allows the pile to penetrate and bear in stronger, denser soils or rock.

## DRIVEN PILING SECTION 5.2.3

All of Ram Jack's driven pilings consist of ASTM-A500 material with a minimum yield strength of 65 ksi [450 Mpa]. Each piling is connected with a slip joint connection as shown in Figure 5-3. The coupler consists of a smaller diameter 12 in. [305 mm] long pipe that extends 6 in. [152 mm] into one end of the piling. The connector end of the piling is shop-crimped to hold the coupler in place. This allows the ends of the pilings to be

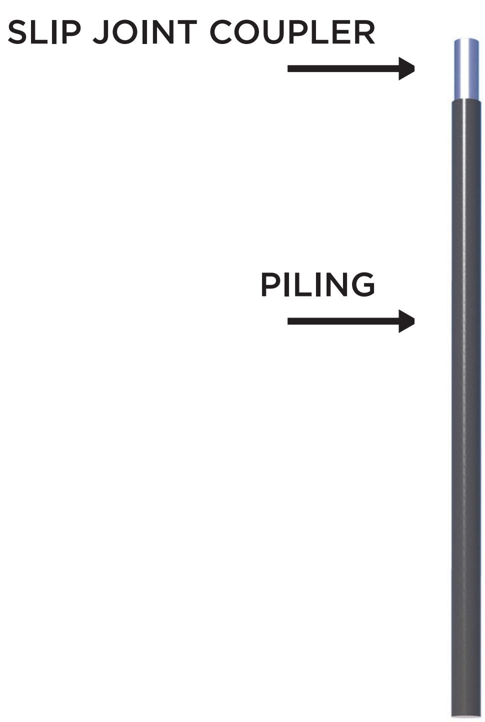



**FIGURE 5-3 : Piling Section**

smooth and even so the compressive forces being applied to a piling section have a clean load path to transfer the load to each subsequent section.

Piling section lengths vary from 2 to 10 ft. [0.61 to 3.05 m]. The length of the piling sections are typically governed by the overhead clearance available. The most common lengths are 5 and 7 feet [1.52 and 2.13 m]. The piling sections are installed with the slip connection on the top end. Therefore once the piling has been installed in the field and the piling is cut-off at the proper elevation height, the shorter scrap piece that is removed can be used when installing another pile. This eliminates material waste as much as possible.

### STARTER 5.2.4

The starter is the lead section which is placed on the toe of the piling as illustrated in Figure 5-1. The starter section is shown in Figure 5-4. It consists of a 12 in. [305 mm] long piece of piling material with the same slip joint coupler as the piling section. A soil plug is welded on the inside of the starter. This allows the soil to tightly compact inside the starter as the pile is being installed. The soil plug allows the pile to have a solid bearing surface once the pile reaches a load bearing strata. It also prevents the soil from sliding up the inside diameter of the pile over time which would allow the pile to bear only on the cross sectional area of the pipe edge. The toe of the starter is expanded to a diameter ½ in. [13 mm] larger than the diameter of the piling. The purpose of expanding the toe of the starter is to reduce the soil skin friction on the surface of the pile during installation. This allows more of the driving force used to install the pile to be transferred to the toe of the pile. The pile is able to penetrate into denser soils. As the displaced soil rebounds and remolds around the pile shaft, the skin friction on the pile also increases which increases the overall capacity of the pile.

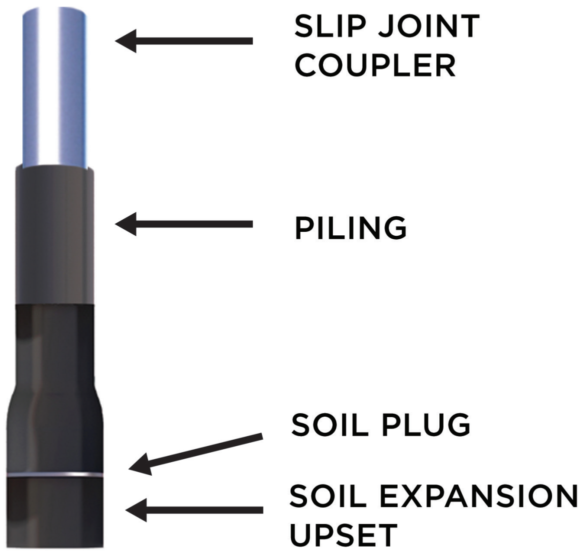



**FIGURE 5-4 : Starter Section**

## Applications 5.3

Hydraulically driven steel piles use the weight of the structure as a reaction force during installation. Therefore, they are only used for remedial foundation repairs. The structure must have sufficient weight to allow enough force to be applied to the pile to reach a load-supporting strata. Driven piles should not be used on light structures such as wing walls, carports, interior floor slabs, light weight porches, etc. Typically a minimum wall load of 700 pounds per linear foot [10.2 kN/m] is required to install driven piles.

Due to the slip joint couplers, the driven steel piles are typically only used for compression loads. Even though helical piles are typically used for tension loads, hydraulically driven piles can be adapted to accept tension loads when necessary. Either Ram Jack's threaded helical piles are used or a reinforcing bar is grouted inside the driven pile to create a tension load path.

Ram Jack's driven steel piles are also fast and efficient to install. Depending on the depth of the footing and the embedment depth of the piles, it's not uncommon for a single crew of 3 to 4 members to install as many as 10 to 30 piles in a single day. Most projects can be completed in one or two days.

The equipment used to install Ram Jack's driven steel piles is small and lightweight compared to the equipment used to construct other types of deep foundations, allowing installation in tight quarters and on sites with soft surface conditions. All the equipment can be hand carried. A driven pile can be installed in areas as small as 2 ft. [0.61 m] by 2 ft. [0.61 m] with 4 ft. [1.22 m] of clearance height. This means the disturbance to the existing grade and landscaping is minimal.

Driven steel piles are excellent for remedial underpinning of existing structures. Each pile is basically load tested during installation. Since the hydraulic cylinders have a constant effective area, the force being applied to a pile is easy to calculate by multiplying the hydraulic pressure being applied by the effective area of the cylinders being used. Standard hydraulic cylinders recommended by Ram Jack with their effective areas are provided in Section 5.4.

## Installation System 5.4

Unlike many driven steel pile systems that use a single hydraulic cylinder (ram) that pushes down on the top of a pile section, Ram Jack's driven pile system utilizes twin hydraulic cylinders (rams) to pull the pile into the soil. There are several advantages to using twin cylinders.

First, using twin cylinders allows more driving force to be applied to the pile during installation. This ensures that enough driving force can be applied to the pile to allow it to rifle through soft, unstable soils and bear on a strong load-bearing stratum.

It increases the stability of the system as a whole during installation. Single hydraulic cylinders must have heavy frames attached to the bracket during installation to provide stability. The frame is meant to keep the cylinder from kicking to one side during installation which is a safety risk to the installers and the walls of the structure under heavy loads.

Single cylinder systems only install short sections of pilings. This means that more couplings must be used on a single pile. The couplings are typically the weakest point of a driven pile especially if the coupling is in a region of the pile experiencing bending stresses due to the eccentric load on the pile. As explained in Chapter 4 *Designing Helical Piles per the IBC* of this manual, a driven steel pile with a side load bracket must be designed as an unbraced system as it does not meet the bracing requirements described in Section 1810.2.2 of the IBC. The minimum unbraced length of a pile is provided in Section 1810.2.1 of the IBC as previously mentioned in Chapter 4.

Ram Jack's patented drive head as shown in Figure 5-5 allows twins cylinders to be used and longer piling sections (typically 5 to 7 ft. [1.52 to 2.13 m]) to be installed. By using longer piling sections, the pile is inherently stronger by having fewer couplings. Longer pile sections are typically less expensive than shorter sections since they require less material and handling to manufacture. The external guide sleeve previously mentioned provides added reinforcement to any coupler that might be present in the unbraced zone defined in the IBC where the maximum bending stresses will occur.

When installing a driven pile, the hydraulic cylinders are attached to the underpinning bracket and the drive head. The piling is placed through the drive head, bracket sleeve, and guide sleeve. Hydraulic cylinders are then reciprocated up and down to install the piling sections. The drive head functions similar to a pipe wrench. It releases the piling on the up stroke and grabs the piling on the down stoke to pull it into the soil.

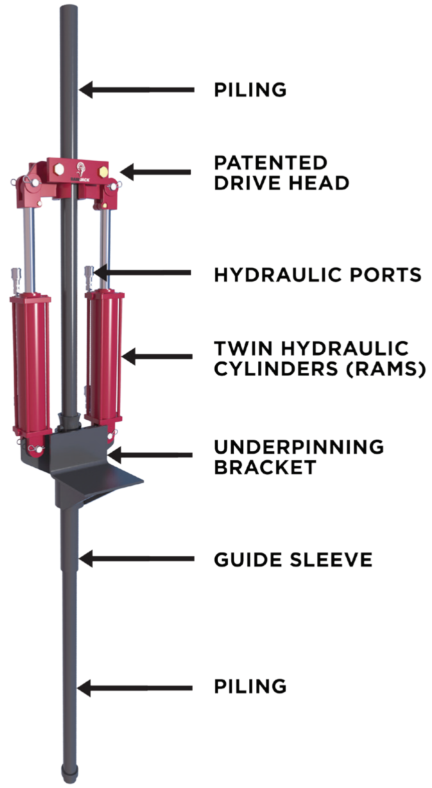



**FIGURE 5-5 : Driven Pile Installation System**

Ram Jack utilizes three different hydraulic cylinder sizes to install their driven piles (3 ½”, 4 ½” and 5” [89, 114, and 127 mm] bore cylinders). The size and capacity of a hydraulic cylinder is based on the internal diameter of the cylinder which is referred to as the bore. The rod that extends out of the top side of the cylinder has a piston attached to the other end which travels up and down on the inside of the cylinder. A double action hydraulic cylinder has hydraulic ports at each end of a cylinder. When hydraulic fluid flows into the upper port as shown in Figure 5-5, the cylinder rod retracts. When hydraulic fluid flows into the lower port, the cylinder rod extends.

The force exerted by a hydraulic cylinder is a function of the hydraulic pressure and cross-sectional area inside the cylinder. In order to calculate the force, the hydraulic pressure is multiplied by the surface area on the top or bottom of the piston which is dependent on whether the cylinder rods are extending or retracting. When the rods are extending, the surface area of the bottom of the piston is the cross-sectional area of the cylinder. For example, a 4-inch [102 mm] bore hydraulic cylinder has a cross-sectional area of 12.57 $in^2$ [81.1 $cm^2$]. In the retraction (pull) mode which the driven piles are being installed, the surface area on the top side of the piston is the cross-sectional area of the cylinder minus the cross-sectional area of the rod.

$$\text{Working Area} = \frac{\pi D^2}{4} - \frac{\pi d^2}{4}$$

Where:
D = bore diameter of cylinder
d = diameter of cylinder rod

**Example**, a 4-inch [102 mm] bore hydraulic cylinder has a rod diameter of 1.75 in. [44.5 mm]. Therefore the working area is:

$$\text{Working Area} = \frac{\pi(4)^2}{4} - \frac{\pi(1.75)^2}{4}$$

$$= 12.57 - 2.41$$

$$= \mathbf{10.16\ in^2}\ [65.5\ cm^2]$$

Since Ram Jack uses dual hydraulic cylinders during installation, the effective working area of twin 4-inch installation cylinders is **20.32 $in^2$** [131 $cm^2$]. Load charts are provided in Figure 5-6 for Ram Jack’s standard installation cylinders.

## Driven Pile Installation Procedure 5.5

1. An area shall be excavated immediately adjacent to the building foundation to expose the footing, bottom of grade beam, stem wall, or column to a

| **3 ½" [89 mm] Bore (rod = 1 ⅜" [35 mm] dia.)** (Working Area = 16.27 in² [0.0104 m²]) | | | |
|---|---|---|---|
| **PSI** | **[kPa]** | **Force** (lbs) | [kN] |
| 500 | [3,447] | 8,135 | [36.18] |
| 1,000 | [6,895] | 16,270 | [72.37] |
| 1,500 | [10,342] | 24,405 | [108.55] |
| 2,000 | [13,789] | 32,540 | [144.75] |
| 2,500 | [17,237] | 40,675 | [180.93] |
| 3,000 | [20,684] | 48,810 | [217.12] |
| 3,500 | [24,132] | 56,945 | [253.30] |
| 4,000 | [27,579] | 65,080 | [289.50] |

| **4" Bore [102 mm] (rod = 1 ¾" [44 mm] dia.)** (Working Area = 20.32 in² [0.0131 m²]) | | | |
|---|---|---|---|
| **PSI** | **[kPa]** | **Force** (lbs) | [kN] |
| 500 | [3,447] | 10,160 | [45.19] |
| 1,000 | [6,895] | 20,320 | [90.34] |
| 1,500 | [10,342] | 30,480 | [135.58] |
| 2,000 | [13,790] | 40,640 | [180.78] |
| 2,500 | [17,237] | 50,800 | [225.96] |
| 3,000 | [20,684] | 60,960 | [271.16] |
| 3,500 | [24,132] | 71,120 | [316.35] |
| 4,000 | [27,579] | 81,280 | [361.56] |

| **5" [127 mm] Bore (rod = 2" [51 mm] dia.)** (Working Area = 32.90 in² [0.0212 m²]) | | | |
|---|---|---|---|
| **PSI** | **[kPa]** | **Force** (lbs) | [kN] |
| 500 | [3,447] | 16,450 | [73.17] |
| 1,000 | [6,895] | 32,900 | [146.35] |
| 1,500 | [10,342] | 49,350 | [219.52] |
| 2,000 | [13,789] | 65,800 | [292.69] |
| 2,500 | [17,237] | 82,250 | [365.86] |
| 3,000 | [20,684] | 98,700 | [439.04] |
| 3,500 | [24,132] | 115,150 | [512.21] |

*Notes:*

1. *Working area is the area of the bore of each cylinder minus the area of the rod. Two cylinders are used during pile installation.*
2. *Force = working area x PSI [kPa]*
3. *Max. working pressure is 4,000 psi [27,579 kPa]*
4. *Bursting pressure is 5,000 psi [34,474 kPa]*

**FIGURE 5-6 : Ram Jack Installation Driving Force Load Charts**

width of at least 24 in. [610 mm] and at least 12 in. [305 mm] below the bottom of the footing or grade beam.

2. The vertical and bottom face of the footing shall, to the extent possible, be smooth and at right angles of each other for the mounting of the pile bracket. The surfaces in contact with the support bracket shall be free of all dirt, debris, and loose concrete so as to provide firm bearing surfaces.

3. The spread footing, if applicable, shall be notched to allow the support bracket seat to mount directly under the bearing load of the stem or basement wall.

4. Insert pile lead section, guide sleeve, and first pile section through the bracket sleeve. Connect double action hydraulic rams to the support bracket. Pile should not exceed more than a 5-degree angle from vertical. Hydraulic rams used to install the pile shall have the capability of exerting the minimum

installation force required to meet or exceed the ultimate installation force required. The ultimate installation force is defined as the allowable or service load requirement multiplied by an appropriate safety factor of no less than two.

5. The hydraulic rams shall be reciprocated up and down, with the pile being advanced with each downward stroke. Pile sections shall be continuously added as required to advance the pile through unstable soils as required. Advancement of the pile will continue until one of the following occurs: the structure begins to experience uplift flexure as the pile is being advanced; the desired installation force is achieved or as determined by the foundation investigation. All piles shall be installed individually utilizing the maximum resistance of the structure as a reaction force to install each pile. The location of the driven pile system must be determined by a design professional. Lifting of the structure must be verified by a design professional to ensure that the foundation and/or superstructure are not overstressed.

**FIGURE 5-7 : Installed Driven Pile with Support Strap Assembly**

6. After piling termination, the excess piling shall be cut off at a sufficient height to allow for foundation lifting if required. The support strap assembly shall be installed as shown in Figure 5-7.

7. If the foundation is to be stabilized against settlement only, tighten the nuts on the top of the support strap until the bracket is firmly engaged with the bottom of the foundation. The excavation shall be back-filled and soil properly compacted. Excess soil shall be removed.

## Driven Pile Lifting System 5.6

All of Ram Jack's remedial underpinning brackets for driven and helical piles can be used to raise the foundation of a structure. There are a few different methods and assemblies used by Ram Jack to lift a structure. Figure 5-8 shows the basic lifting system that is used with driven and helical piles.

The basic components consist of a custom jack base commonly referred to as a rocket, two lifting arms, and a reaction bar. The rocket is slotted on one end and is designed to fit over the support strap and the head of the pile shaft. The rocket provides stability to the hydraulic jack during lifting operations.

The arms of the lifting system attach to the same holes in the bracket ears as the hydraulic cylinders used to install the piles. There are a number of holes punched in the lifting arms that allow the height of the reaction bar to be adjusted up or down. This allows the lifting system to accommodate any type of jack or jacking system used to raise the structure.

When multiple lift points are needed to lift a structure uniformly, Ram Jack's multi-lift system is used. The system is very similar to the basic system in Figure 5-8. However, the reaction bar is customized to receive a double action hydraulic cylinder. A single hydraulic cylinder is placed at each pile. Each hydraulic cylinder is connected in series with the cylinder on the adjacent pile and all the cylinders are operated by a central hydraulic pump. Each hydraulic cylinder also has an independent needle valve which controls the hydraulic flow to the lifting cylinder. Once the proper elevation is achieved at a certain lift point, the hydraulic flow can be turned off and the remaining hydraulic cylinders can continue lifting the structure as needed. Figure 5-9 shows a picture of Ram Jack's multi-lift system being utilized on a project.

Once the lifting operation is completed, the fastening nuts are tightened and the lifting assembly and hydraulic cylinder is removed from the pile.

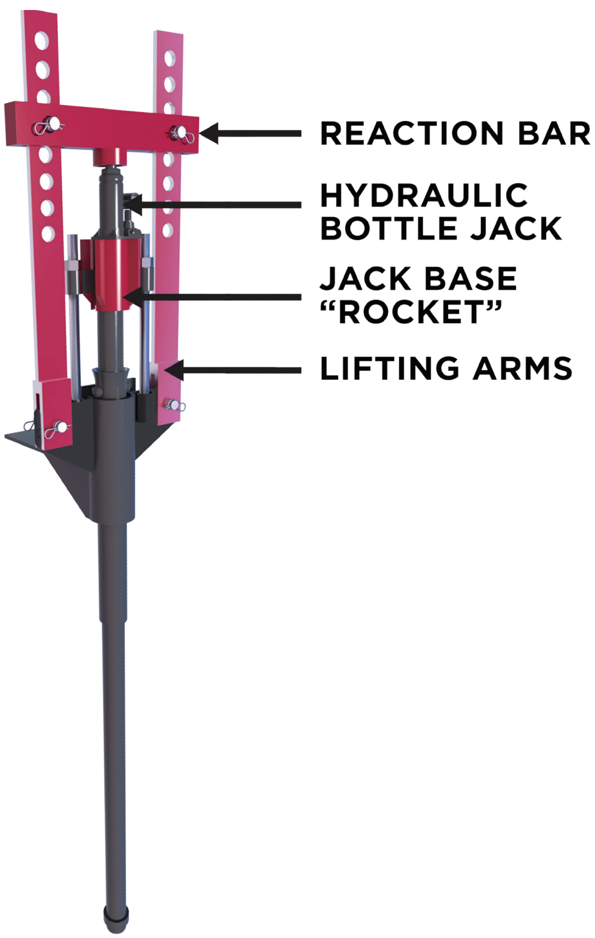



**FIGURE 5-8 :** Basic Pile Lifting System with Hydraulic Bottle Jack

**FIGURE 5-9 :** Ram Jack Multi-Lift System

# 6: CORROSION: IMPACT ON STEEL PILES

## Introduction 6.1

The subject of corrosion is complex with multiple variables dictating the rate at which corrosion occurs. There have been many studies conducted over the years on the prediction of corrosion rates of steel underground. Due to the extensive infrastructure of oil, gas, and water pipelines throughout the world, most of the corrosion studies have been centered around steel pipes buried in disturbed soil where heavy concentrations of oxygen are present. Oxygen is a major driving force of corrosion in steel whether it is above or below grade.

In the 1950s and 60s the National Bureau of Standards (NBS) did extensive research on the corrosion of steel in soil. The studies were authored by Melvin Romanoff. Mr. Romanoff's research was so extensive his reports are referenced and findings confirmed in almost every corrosion study of steel in soil since. Mr. Romanoff's first report was titled *Underground Corrosion* (NBS Circular 579) which was first published in 1957 and rereleased in 1982. This study consisted of more than 36,500 specimens, representing 333 varieties of ferrous, nonferrous, and protective coating materials from 128 test sites from across the nation. The electrical and electrochemical aspects of underground corrosion were monitored and studied extensively. It's important to point out that all of the steel specimens from this study were from sites where the steel had been placed in disturbed soil (trenches and excavation pits).

In 1962, Mr. Romanoff authored another report for the NBS titled *Corrosion of Steel Pilings in Soils* (NBS Monograph 58). The study consisted of inspections of steel pilings that had been in service for up to 40 years. Since steel piles are normally installed into undisturbed soil, the findings varied greatly from the results reported in NBS Circular 579. The difference in the behavior of the corrosion of steel is largely attributed

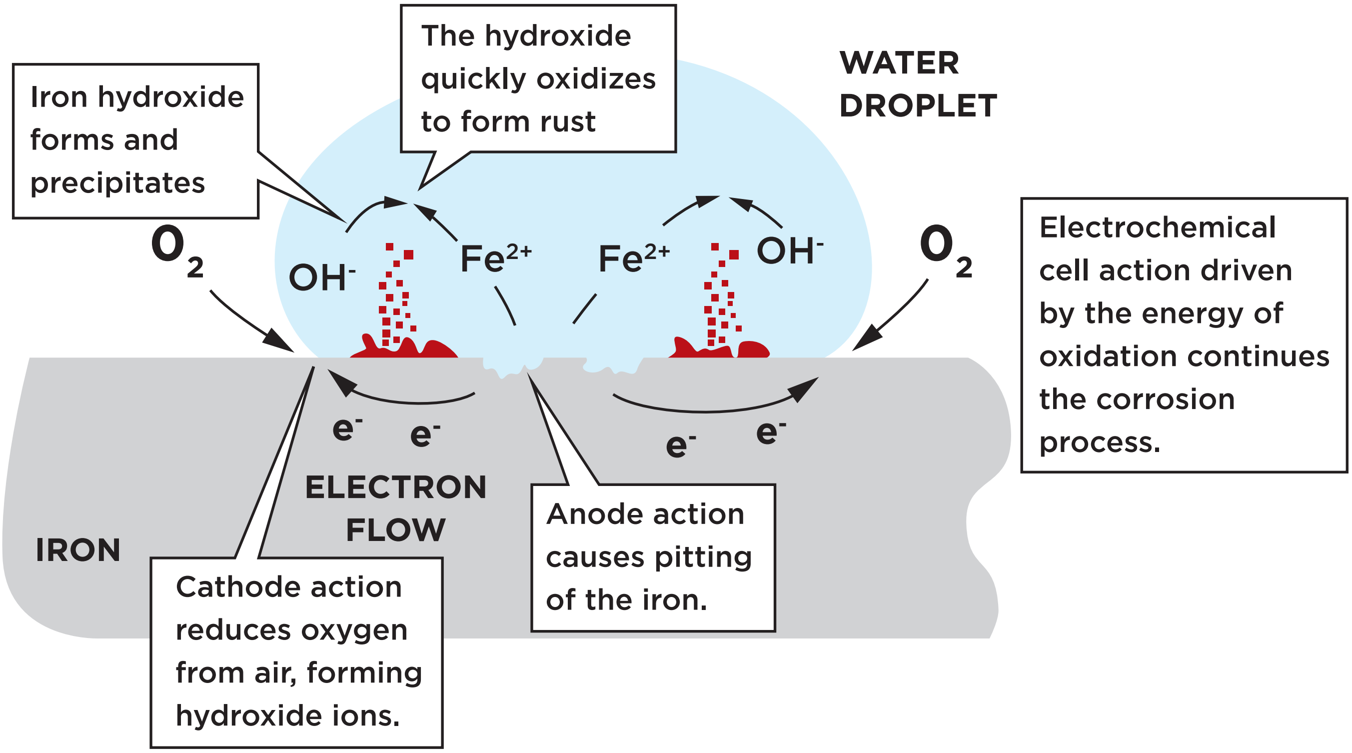



**FIGURE 6-1 :** Electrochemical Corrosion of Iron to concentration differentials of oxygen in disturbed and undisturbed soil.

Both of the NBS reports (NBS Circular 579 and NBS Monograph 58) are readily available on the web. Due to the complexity of corrosion, our discussion will be limited to the corrosion effects on steel pilings.

## What is Corrosion 6.2

Broadly speaking, corrosion is essentially the tendency of a refined metal to return to its native state. Different types of corrosion can occur in soils, such as soil corrosion and galvanic corrosion. In the case of steel piles and steel sheet piles, soil corrosion is most important. Corrosion is an electrochemical process.

There are certain conditions which must exist before a corrosion cell can function. Figure 6-1 illustrates the essential elements of a corrosion cell. Necessary elements include an anode, a cathode, oxygen, an electrical path between each, and an electrical conductive electrolyte. Once each of these elements is in place, an active corrosion cell is formed.

A metal surface will have discrete areas with differences in potential (voltage difference) caused by variations. These small surface variations set up anodic and cathodic sites. Metal ions dissolve into the electrolyte (water) leaving the anode with a positive charge which releases electrons.

$$Fe \rightarrow Fe^{2+} + 2e^{-}$$

These electrons travel through the steel to a cathode area which is negatively charged. Once the electrons reach a cathode area, they react with a depolarizer. The resulting hydroxide ions react with the $Fe^{2+}$ to form the mixture of hydrous iron oxides known as rust. The most common depolarizers are:

<u>Oxygen and Moisture</u>

$$4e^{-} + O_2 + 2H_2O \rightarrow 4OH^{-}$$

<u>Acid</u>

$$2e^{-} + 2H^{+} \rightarrow H_2$$

<u>Cation of More Nobel Metal</u>

$$2e^{-} + Cu^{2} \rightarrow Cu$$

Conventional current flows from positive to negative and thus current discharges from the anode and is picked up at the cathode. The current then returns from the cathode to the anode through the electrical path. This flow has a detrimental effect on the anode known as corrosion.

As the electrochemical reaction takes place, differences in surface potential can also change with time. The anode can move to another location, even one that had previously been a cathode. The result is an appearance of general corrosion which covers a large area and is common in atmospheric conditions. When an anode stays in one location for an extended period of time, the corrosion is concentrated at the anode which is also known as pitting.

## Galvanic Corrosion 6.3

When two different metals or alloys are immersed in a corrosive solution or regularly connected by moisture, each will develop a corrosion potential. If the conditions for galvanic corrosion are present, the more noble metal will become the cathode and the more active metal will become the anode. A measurable current may flow between the anode and the cathode. If this occurs, the anode's rate of corrosion in the service environment will be increased while the cathode's corrosion rate will decrease. The increased corrosion of the anode is called galvanic corrosion (also known as dissimilar metal corrosion).

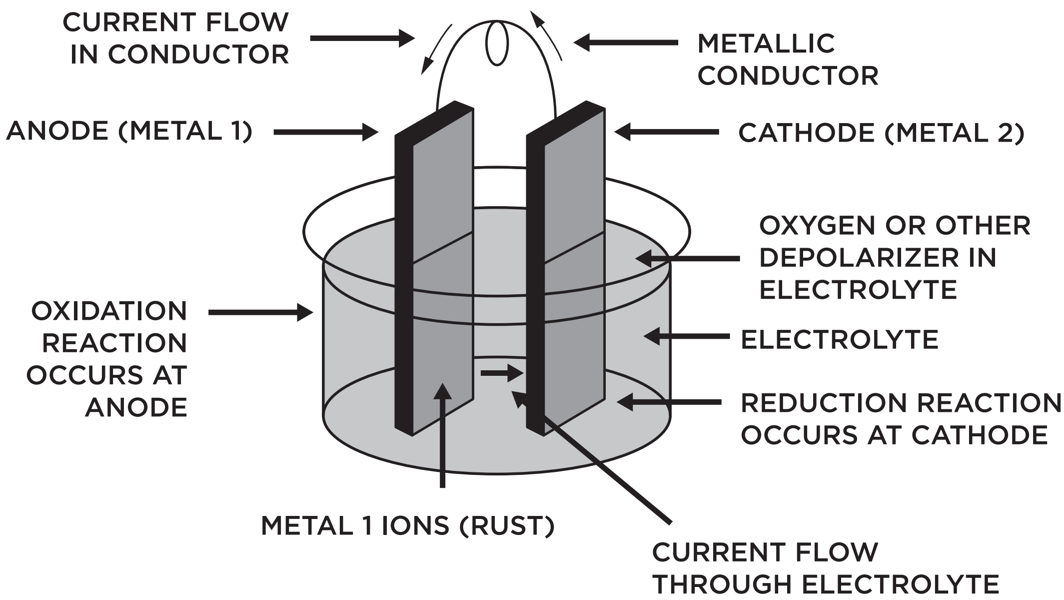



**FIGURE 6-2 : Galvanic Corrosion Cell**

For galvanic corrosion to occur, three conditions must be present:

1. Two metals with different electrical potential must be present. One of the metals must act as an anode and generate electrons that create an electrical current. The other metal must act as a cathode and collect these flowing electrons.

2. The two dissimilar metals must be in direct contact with each other.

3. There must be an electrolyte (water or moist soil) covering the two metals at the area where they are in contact.

Figure 6-2 shows the elements and electron flow of a galvanic couple. The less nobel metal (anode) releases electrons through the electrolyte. The more noble cathode collects the electrons. In order to complete the return path of the electrical circuit, the two metals must be connected. In almost all cases concerning steel piles, the two metals are in direct contact. Figure 6-3 shows an example of galvanic corrosion.

**FIGURE 6-3 : Galvanic Corrosion**

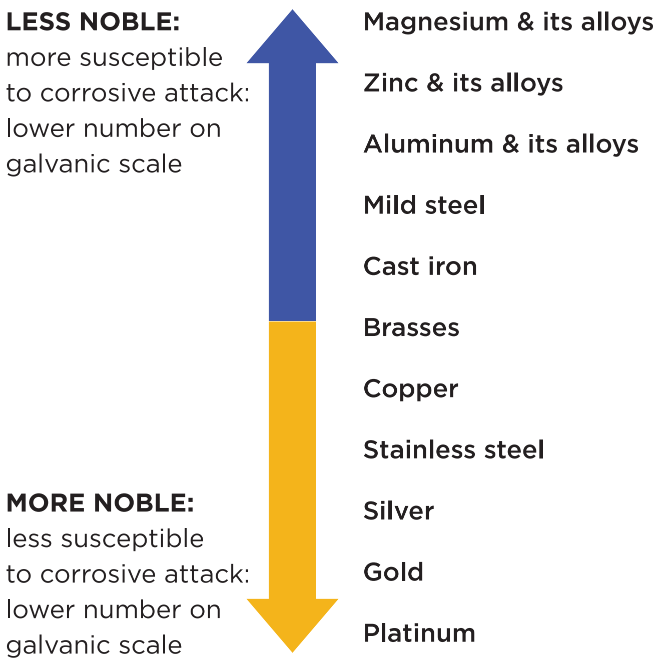



**FIGURE 6-4 : Susceptible Metals Per the Galvanic Series**

The relative nobility of a material can be predicted by measuring its corrosion potential. The galvanic series lists the relative nobility of certain materials in sea water. Figure 6-4 show a list of some of the most susceptible metals listed in the galvanic series. The further the two metals are apart from each other in the galvanic series, the higher the electrical current and corrosion will be.

The relative surface area (not mass) of each of the exposed metals is also an important factor. A small anode/cathode area ratio is highly undesirable. If the area of the cathode (noble metal) is very large, and the anode (active metal) is very small, the current produced is likely to be very high. Rapid thickness loss of the dissolving anode tends to occur under these conditions. Galvanic corrosion problems should be solved by designing to avoid these problems in the first place.

There are several ways to prevent galvanic corrosion from occurring. One is not mixing dissimilar metals together when possible. In the event that circumstances require dissimilar metals to be connected, provisions should be made to prevent moisture from collecting where the two metals are in contact. Another method would be to prevent the two metals from contacting each other by using a plastic coating or plastic washer to isolate one of the metals from the other.

## Soil Corrosion 6.4

Soil corrosion is influenced by several factors, including soil resistivity, degree of saturation, pH, dissolved salts, availability of trapped oxygen in the soil and total acidity. These parameters are interrelated but may be measured independently. Based on studies of buried metals, a general consensus has been established that resistivity is the most accurate indicator of corrosion potential of soil.

| **Soil Resistivity** (ohm-cm) | **Corrosion Potential** |
|---|---|
| Greater than 10,000 | Very mild |
| 5,000 to 10,000 | Mild |
| 3,000 to 5,000 | Moderate |
| 1,000 to 3,000 | Severe |
| Less than 1,000 | Highly Severe |

**TABLE 6-1 : Corrosion Ratings Based on Soil Resistivity**

Soil resistivity, which quantifies the resistance to an electrical current across a cubic meter of earth, becomes the electrolyte in the electrochemical corrosion process. Please note that moisture and oxygen also need to be present to complete the corrosion cell. The electrolytic behavior of soils is an indirect measurement of the soluble salt content. In other words, the amount of dissolved inorganic solutes (anions and cations) in water or in soil is directly proportional to the soil's electrolytic conductivity. The major dissolved anions in soils are

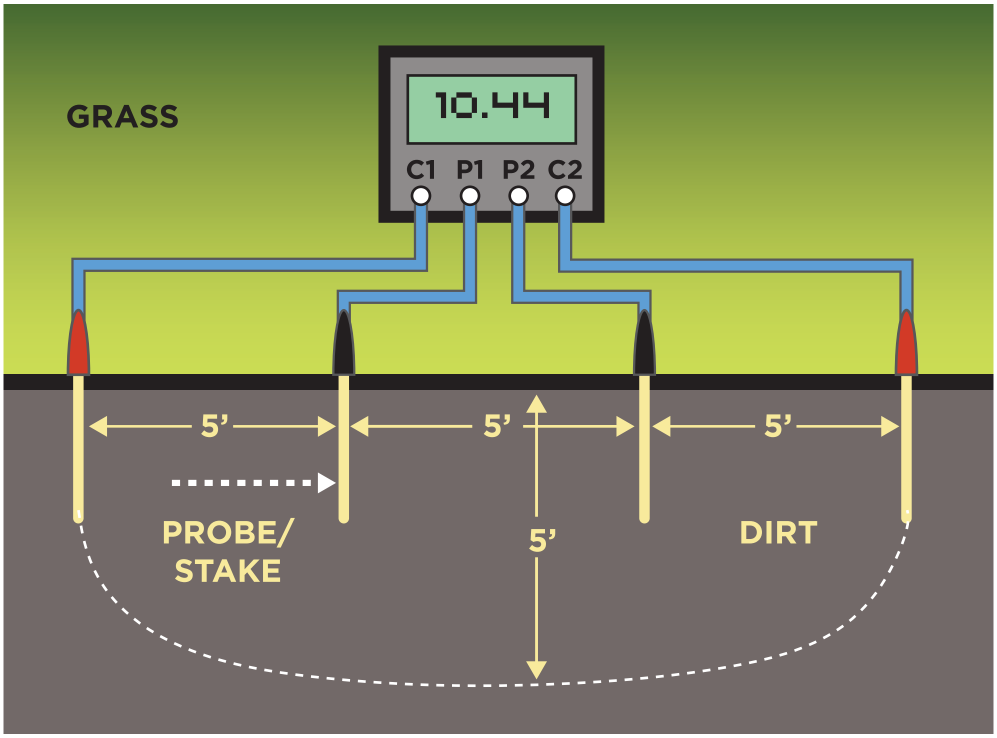



**FIGURE 6-5 : Wenner 4-Point Test**

chloride and sulfate which increase the conductivity of the soil. Table 6-1 lists the corrosion potential of soils with different resistivity levels. Notice that the higher the soil resistivity is, the lower the potential for corrosion is. Soil resistivity typically decreases in moist soil conditions and a concentration of dissolved anions which increases the soil's corrosiveness.

There are several methods available for measuring soil resistivity in the field. The most widely used method is the Wenner 4-point test. The test involves driving four metal electrodes into the ground in a straight line and equidistant from one another as shown in Figure 6-5. The outer electrodes are called current probes. These are the electrodes that inject an electrical current into the ground. The two inner probes are the potential probes which take the actual soil resistance measurement. The basic premise of the soil resistivity test is that probes spaced equidistant across the earth will read the same equidistant in depth. The example in Figure 6-5 shows the electrodes spaced on 5-ft. [1.52 m] centers and measuring the resistivity to a depth of 5 ft. [1.52 m] below grade. However, the electrodes can measure to depths of 6 in. [152 mm] to 100 ft. [30.5 m] depending on the spacing of the probes. Manufacturer software and formulas are used to convert the measured resistance (ohms) into soil resistivity (ohm-cm) and create a soil resistivity chart or graph based on the spacing of the electrodes. The soil resistivity will often vary with depth depending on the soil conditions.

## UNDISTURBED SOILS 6.4.1

Even though steel piles have been used to support structures since the early 1900s, much of the research on soil corrosion comes from the corrosion of pipelines and storage tanks. In the majority of studies, the steel samples were placed in disturbed soils.

It wasn't until 1962 that the first major study of the corrosion effect on steel pilings was published by the National Bureau of Standards (NBS). The study *Corrosion of Steel Pilings in Soils* was authored by Melvin Romanoff, (NBS Monograph 58, 1962). The study consisted of exhuming and inspecting steel piles that had been installed in a wide range of soil conditions for as much as 40 years. The soil types in which the steel piles were placed is well-documented and ranged from well-drained sands to impervious clays, soil resistivity ranging from 300 to 50,200 ohm-cm, and soil pH which ranged from 2.3 to 8.6. The report found that there was little to no corrosion of steel piles installed in undisturbed soils or below the water table. The full report is readily available online.

From Mr. Romanoff's discussion section:

*"Examination of the data shows that in general soil type, drainage, soil resistivity, pH or chemical compositions of soils are of no importance in determining the corrosion of steel piles driven in undisturbed soils. This is contrary to everything published pertaining to the behavior of iron and steel under disturbed or backfilled soil conditions. Hence, soil corrosion data published in NBS Circular 579 are not applicable and should not be used for estimating the behavior of steel pilings driven in undisturbed soils."*

Mr. Romanoff attributed the differences in the corrosion of steel placed in undisturbed soil and disturbed soil as being the differences in oxygen concentration. Undisturbed soils are so deficient in oxygen levels a few feet below the ground line or below the water table, that steel pilings are not appreciably affected by corrosion, regardless of soil types or the soil properties. Please recall that corrosion is an electrochemical process that requires four elements for an active corrosion to form:

1. Anode and cathode which is found in all steel.
2. Electrolyte (water, moisture, corrosive soil)
3. Oxygen

Therefore, an active corrosion cell cannot form if the electrolyte or oxygen can be isolated from reaching the steel. Since steel piles are typically installed in undisturbed soils, the highest degree of corrosion risk occurs in the upper few feet of the pile where the soil may have been disturbed during the installation of the pile.

## DISTURBED SOILS 6.4.2

A disturbed soil is a soil in which digging, backfilling, or other soil upheaval has taken place. The soil disturbance can significantly impact the oxygen concentration in the soil when compared to undistrurbed soil. During construction, the excavation of footings and pile caps can create a region of aerated soil near the top of the pile. However, the corrosion potential can easily be mitigated by appropriate soil compaction which will drive the concentrated oxygen from the soil. Properly compacted soils have corrosion rates similar to those of natural, undisturbed soil.

Steel piles are sometimes installed through fill soil. Mr. Romanoff found that piles installed through loose, under- or non-compacted soil experienced some corrosion pitting. He attributed the pitting to pockets of oxygen found in the disturbed soil and that once the oxygen was depleted in the disturbed soil the corrosion stopped. The pitting found was mild to moderate. The maximum pit depth found in the piles place in disturbed soil regions was in the order of 0.145 in. [3.68 mm]. The report concluded that while pitting corrosion might cause considerable concern for a fluid-carrying structure it doesn't significantly affect the structural capacity of a steel piling. Because of the localized and isolated nature of the attack, corrosion by pitting covers a relatively small area of the piling. Therefore, the reduction in the steel piling's cross-section is not significant.

**FIGURE 6-6 : Helical Lead Sections Hung From Conveyor System**

## Corrosion Protection for Piles 6.5

When steel piles are going to be installed in a corrosive environment and corrosion protection is needed, there are many options available for minimizing the effects of soil corrosion on steel. The most common methods are thermoplastic powder coating, epoxy coating, zinc coating, designing the pile cross section for the sacrificial steel loss, concrete encasement and installing sacrificial anodes. These options have various advantages and drawbacks. More importantly, they each have different cost benefits.

### THERMOPLASTIC POWDER COATING 6.5.1

Thermoplastic (polymer) coatings were first developed in the 1950s. The coating offers excellent protection of metal structures against corrosion, wear and tear and chemical attack. Thermoplastic coatings have excellent adhesion to steel, iron and aluminum. It is used in high corrosive environments such as oil and gas pipelines, marine harbors, battery boxes, playground equipment, and many others. The adhesion of the coating to the steel prevents oxygen from reaching the steel in disturbed soil. Since the coating is also a thick durable plastic, it also isolates the steel pile from an electrolyte and prevents an electrical current from forming an electrochemical corrosion cell. The plastic also serves as an insulation barrier to prevent galvanic corrosion from occurring in case a dissimilar metal comes into direct contact with the pile or bracket.

Helical and hydraulically driven piles are low-displacement piles, meaning the piles produce no soil spoils and displace very little soil. Therefore, surface damage to the thermoplastic coating can occur when a pile is installed through an abrasive soil or gravel layer. Most of the abrasion occurs on the lower portion of the pile as it is clearing a path for the pile. This portion of the pile should be in undisturbed soil where oxygen is deficient. Since a path has already been cleared for the trailing pile shaft, the surface damage should be very minimal around the upper portion of the pile where oxygen levels will be higher a few feet below the ground line.

The International Code Council (ICC) adopted the *Acceptance Criteria for Corrosion Protection of Steel Foundation Systems Using Polymer Coatings* (AC228) in October 2003. Thermoplastic powder coatings are also an approved method of corrosion protection in the *Acceptance Criteria for Helical Pile Systems* (AC358).

The most common way to apply a thermoplastic powder coating to steel piles is with an electrostatic spray. An electrostatic spray gun has an electrode at its tip that charges the powder particles with a highly negative charge upon exiting the spray gun. The particles are then attracted to the steel with an attraction that is 75

**FIGURE 6-7 : Helical Lead Sections Leaving Shot Blast Cabinet**

**FIGURE 6-8 : Application of Powder Coating**

times stronger than gravity. This is important for two reasons. It allows the overspray particles that float past the target to reverse direction and wrap around the target thereby coating all hidden surfaces. This allows 95 to 100% of the powder to stick to the target which reduces waste, increases efficiency and makes the thermoplastic powder coating more cost effective than most corrosion protection methods.

Following is a summary of the thermoplastic powder coating process and the seven stages required.

1. The parts to be powder coated are loaded (suspended) on a conveyor system.

2. The parts then travel through a two-stage wash consisting of a mild acid and filtered water rinse. The purpose of this stage is to remove oil, grease dirt and paint markings from the parts.

3. The parts then travel through a dryer to remove excess water and humidity from the parts.

4. The next stage is a steel shot blast cabinet. This stage preps the steel to what is commonly called "white steel." It removes any remaining mill scale and oxidation from the steel and increases adhesion for the thermoplastic bond to the steel.

5. The parts then enter a preheat oven that heats the steel parts to 220˚F [104˚C].

6. The next stage is the spray booth where multiple spray guns apply an electrostatically charged thermoplastic powder.

7. The parts then travel through a 500˚F [260˚C] cure oven. This is where the powder melts into a smooth, continuous film and is tightly bonded to the steel.

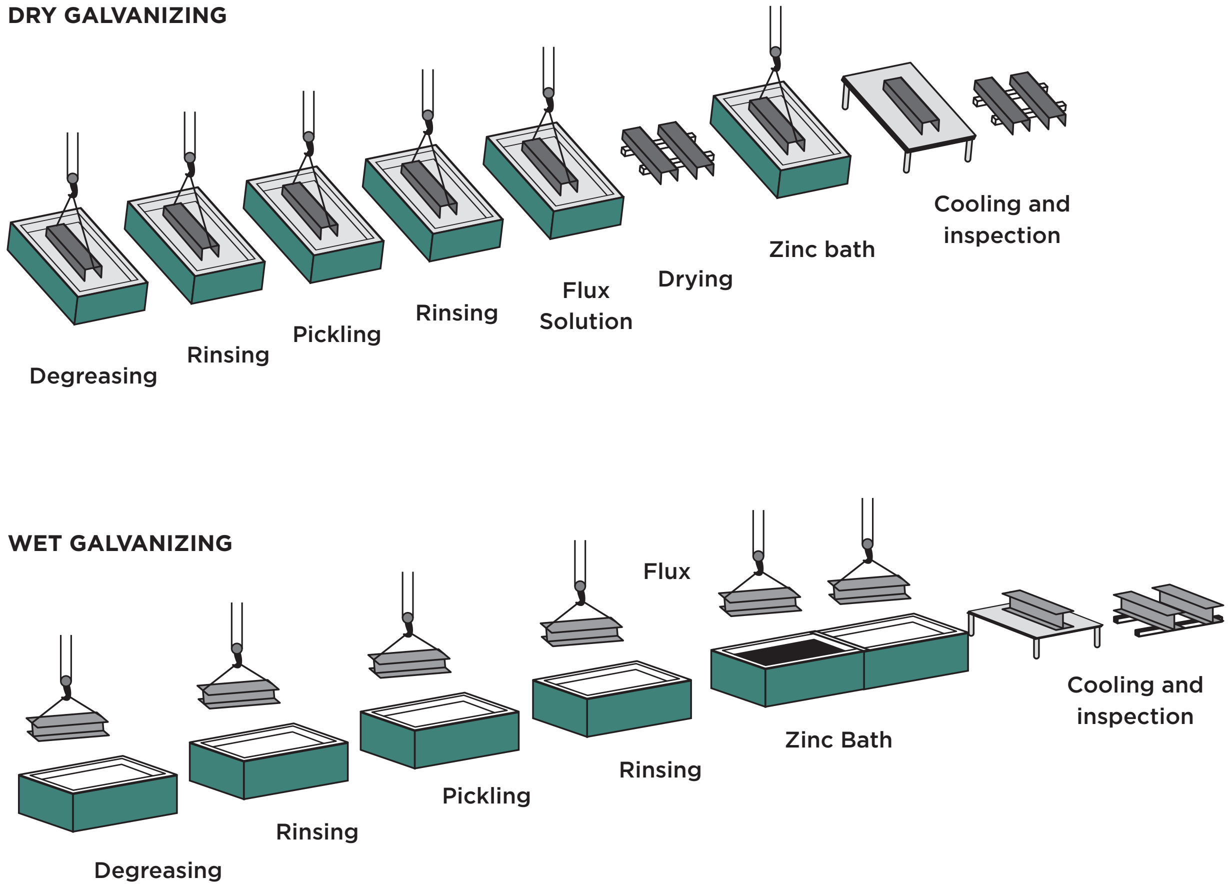



**FIGURE 6-9 : Hot-Dipped Galvanizing Process**

8. The final stage is the ambient cure. Once the temperature of the powder and steel reaches 100˚F [37.8˚C], the parts can be removed from the conveyor system.

## ZINC COATINGS 6.5.2

Zinc is also a common coating to protect steel from corrosion. It has been used for decades to protect steel in atmospheric conditions such as light and fence posts. Zinc is very durable and provides a sacrificial anode because it is higher on the galvanic scale (Figure 6-4). The zinc is corroded away before corrosion can attack the steel.

Zinc can be applied by several methods, including hot dipped galvanization and zinc plating. The application process is similar to that of the thermoplastic powder coating as far as prepping the steel. Figure 6-9 shows a summary of the wet and dry hot dipped galvanizing process.

In the true galvanizing step of the process, the part being coated is completely immersed in a bath of molten zinc. The bath contains at least 98% pure zinc and is heated to approximately 840˚F [449˚C]. Zinc chemistry is specified by ASTM B 6.

While immersed in the kettle, the zinc reacts with the iron in the steel to form a series of zinc/iron intermetallic alloy layers. Once the coating growth is complete, the part is slowly withdrawn from the galvanizing bath. The excess zinc is removed by draining, vibrating and/or centrifuging.

The disadvantage of a zinc coating is that it will eventually be consumed in a disturbed soil condition. Also in a corrosive soil with a high chloride content or low pH (pH < 6.0), the zinc will be consumed at an accelerated rate and shorten the service life of the steel. The typical coating thickness of zinc for hot dipped galvanizing is 3.4 mils [0.0034 in [0.09 mm]). In situations where the soil is acidic or has a high chloride content, hot dipped galvanizers can double-dip the steel material. This doubles the thickness of the galvanizing and generally doubles the life of the coating.

## SACRIFICIAL ANODES 6.5.3

Galvanic or sacrificial anodes are made in various shapes using alloys of zinc, magnesium and aluminum all of which are less noble metals than steel (Figure 6-4). The electrochemical potential, current capacity and consumption rate of these alloys are superior for cathodic protection (CP) to steel. The potential of the steel surface is polarized (pushed) more negative until the surface has a uniform potential. At that stage, the driving force for the corrosion reaction is halted. The galvanic anode continues to corrode. Thereby, consuming the anode material until eventually it must be replaced. The polarization is caused by the electron flow from the anode to the cathode. The driving force for the CP current flow is the difference in electrochemical potential between the anode and the cathode.

For example, the natural corrosion potential of iron is about -0.550 volts in seawater. The natural corrosion potential of zinc in seawater is about -1.2 volts. Thus if the two metals are electrically connected, the corrosion of the zinc becomes a source of negative charge which prevents corrosion of the iron.

Sacrificial anodes are normally supplied with either lead wires or cast-m straps to facilitate their connection to the structure being protected. The lead wires may be attached to the structure by welding or mechanical connections. These should have a low resistance and should be insulated to prevent increased resistance or damage due to corrosion. Figure 6-10 illustrates a cathodic protection system with a sacrificial anode.

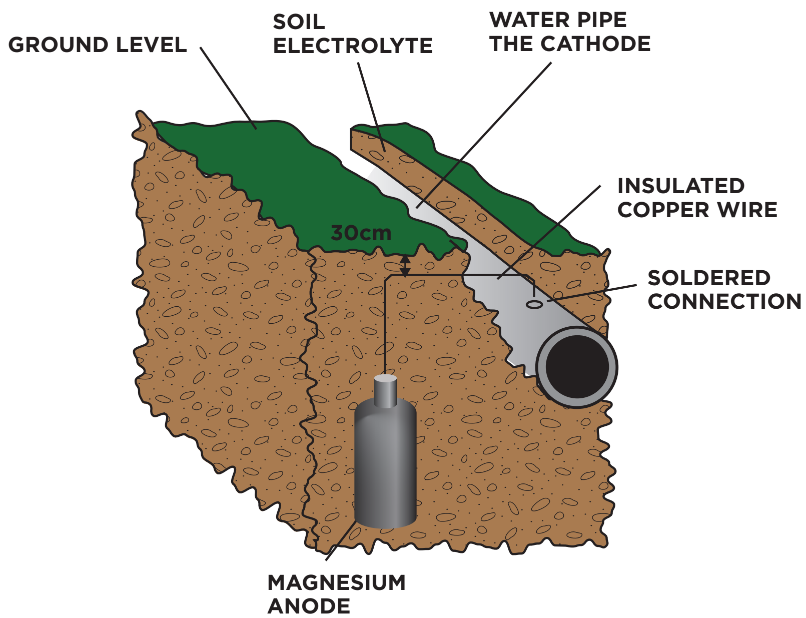



**FIGURE 6-10 :** Cathodic Protection System

The more active metal (anode) is sacrificed to protect the less active metal (cathode). The amount of corrosion depends on the metal being used as an anode but is directly proportional to the amount of current supplied. When sacrificial anodes are used, a corrosion engineer should be consulted to determine the appropriate mass of the anode required. The anodes in a sacrificial anode cathodic protection system must be periodically inspected and replaced when consumed. Even though sacrificial anodes are an effective way to prevent corrosion, they are typically not used in conjunction with steel pilings unless the soil environment is extremely corrosive, dissimilar metals must be used in the piling system or required by a corrosion engineer.

## SACRIFICIAL STEEL LOSS 6.5.4

A sacrificial steel or corrosion allowance is the thickness of steel (above what is structurally required for the pile) needed to compensate for the loss of steel that will occur as the pile corrodes. This extra steel is added to all surfaces of the pile exposed to the corrosive soil or water. As previously stated, most corrosion studies including NBS Circular 579 (Romanoff, 1957) has been performed on steel and iron placed in disturbed soil. Most of the sacrificial steel loss equations have been based on these studies which are conservative in relation to the corrosion rates of steel piles. For example, the corrosion rates for helical piles which are included in the *Acceptance Criteria for Helical Pile Systems* (AC358) comes from a Federal Highway Administration (FHWA) publication (FHWA-NHI-00-044, 2000). The FHWA report is on the corrosion of soil reinforcements for mechanically stabilized earth walls and reinforced slopes which is steel placed in backfill soils (disturbed soil). The FHWA report is based on NBS Circular 579 which Romanoff concluded in his NBS Monograph 58 report *Corrosion of Steel Pilings in Soils*, 1962 was not applicable to steel pilings. Therefore, the sacrificial steel loss equations typically used for steel pilings are conservative and are assumed to be a worst case scenario.

The corrosion equations provided in AC358 are based on the following maximum values:

Resistivity – no less than 1,000 ohm-cm

pH – no less than 5.5

Sulfate – no more than 1,000 ppm

AC358 provides steel loss equations for powder coated, zinc coated and bare steel. For pile design purposes, the sacrificial steel loss (Ts) is reduced by ½ Ts on each side of each component for a net cross-sectional thickness reduction of Ts. For steel pipe piles used in corrosive soil or water, the corrosion allowance is only taken for the exterior surface of the pile. The interior surface of the pile will not be exposed to sufficient oxygen to drive significant corrosion. For H-piles or square shaft piles, ½ Ts would be applied to all exterior surfaces. The sacrificial steel loss equations provided in AC358 are:

Powder Coated: $T_s = 40\ (t - 16)^{0.80}$

Zinc Coated: $T_s = 25\ t^{0.65}$

Bare Steel: $T_s = 40\ t^{0.80}$

Where:

$T_s$ = microns ($\mu m$)
$t$ = years

Corrosion losses are taken for the entire length of the pile and its components regardless of whether the pile is below or above the ground. Zinc coated and bare steel should not be combined in the same system, except where the sacrificial thickness for the zinc coated steel is taken as that for bare steel components. Powder coated steel is allowed to be combined with zinc coated or bare steel.

The California Department of Transportation (Caltrans) published *Corrosion Guidelines*, version 1.0 in September 2003 which includes sacrificial steel loss equations for steel piles. The guidelines follow the conclusions of NBS Monograph 58 concerning steel piles. Caltrans divides the behavior of structural steel in soil into two categories, disturbed soil and undisturbed soil.

Caltrans states that no corrosion allowance is required to be taken for the length of a steel pile placed in undisturbed soil. For steel pilings placed in undisturbed soil, corrosion rates should be taken for the region of the pile below the pile cap or footing down to 3 ft. [0.91 m] below the water table. This region of undisturbed soil is defined as the soil embedment zone and is assumed to have a replenishable source of oxygen needed to sustain corrosion. The corrosion guidelines also provide corrosion rates for immersed zone and scour zone. The immersed zone is the portion of a steel pile that is immersed in corrosive water such as salt water.

Caltrans currently uses the following corrosion rates for steel pilings exposed to a corrosive soil and/or water:

**Soil Embedment Zone** (undisturbed soil)
0.001 in. [0.025 mm] per year

**Immersed Zone** (pile in corrosive water)
0.004 in. [0.100 mm] per year

**Scour Zone**
0.004 in. [0.100 mm] per year

Please note that these corrosion rates are for bare steel only and are applied to all exterior surfaces exposed to corrosive soil and/or water. Caltrans does not provide any corrosion allowances for protective coatings such as powder, epoxy or zinc coatings or cathodic protection. In highly severe corrosive soil (soil restively below 1,000 ohm-cm and/or pH less than 5.5), protective coatings and sacrificial thickness losses are typically used in the pile design.

## Conclusions 6.6

Steel piles have been used to support structures for many years in a variety of applications and structures including marine applications such as dams, bridges and docks. Common steel piles include helical piles, pipe piles, H-piles and sheet piles. None of them produce soil spoils during installation and therefore have low soil displacement and are typically driven into undisturbed soil where oxygen concentration is limited. They are commonly used on projects in an unprotected state with design lives ranging from 75 to 100 years. However in aggressive environments where corrosion can compromise the structural integrity of a steel pile, there are many proven corrosion protection techniques that can be incorporated into the pile design.

- **Barrier Method:** thermoplastic powder or epoxy coating
- **Sacrificial Anode:** zinc coating or cathodic protection system
- **Sacrificial Steel Allowance**

In highly severe corrosive environments such as piles immersed in sea water, soil resistivity levels less than 1,000 ohm-cm or pH levels of 5.5 or less, a corrosion engineer with experience in steel pile corrosion should be consulted during the pile design.

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ICC Evaluation Service, *Acceptance Criteria for Corrosion Protection of Steel Foundation Systems using Polymer (EAA) Coatings (AC228)*, (Approved October 2003)

ICC Evaluation Service, *Acceptance Criteria for Helical Pile Systems and Devices (AC358)*, (Approved June 2013)

US Federal Highway Administration, *Corrosion/ Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes* (FHWA-NHI-00-044), September 2000

Scott Canada, *Corrosion Impacts on Steel Piles, Solarpro*, (December/January 2012)

California Department of Transportation, *Corrosion Guidelines*, version 1.0, (September 2003)

# APPENDIX

| Pile Diameter (in [mm]) | Torque Rating (ft-lbs [kN-m]) | $K_t$ | Fully Braced Pile (kip [kN]) | | 5'-0 [1.52 m] Unbraced Length (kip [kN]) | | 10'-0 [3.05 m] Unbraced Length (kip [kN]) | |
|---|---|---|---|---|---|---|---|---|
| | | | Ultimate | Allowable (S.F.=2) | No Sleeve | 5'-0 [1.52 m] Sleeve | No Sleeve | 10'-0 [3.05 m] Sleeve |
| 2 ⅜ [60] | 4,000 [5.42] | 10 [32.8] | 40 [178] | 20 [89] | 20 [89] | N/A | 8 [36] | 20 [89] |
| 2 ⅞ [73] | 8,000 [10.85] | 9 [29.5] | 72 [320] | 36 [160] | 36 [160] | N/A | 18.5 [82] | 36 [160] |
| 3 ½ [89] | 14,000 [18.98] | 7 [22.9] | 98 [436] | 49 [218] | 49 [218] | N/A | 35.5 [158] | 49 [218] |
| 4 ½ [114] | 23,000 [31.18] | 6 [19.7] | 138 [614] | 69 [307] | 69 [307] | N/A | 69.0 [307] | N/A |

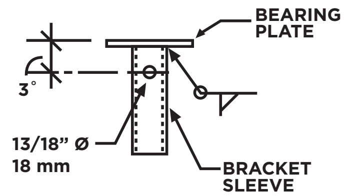
**TABLE A-1 : New Construction Pile Capacities[1,2,3] (ecc. = 0)**

NOTES:

1. Recommended capacities indicated in Table A-1 are based on the lowest value between the mechanical strength of the pile and the torque correlation method and assumes no load eccentricity (e) is being resisted by the pile.

2. A fully braced pile is defined in Section 1810.2.2 of the 2009 and 2012 International Building Code (IBC). Please ensure the definition is clearly understood before relying on the fully braced pile capacities listed in Table A-1.

3. A pile not defined as laterally braced per Section 1810.2.2 of the IBC is considered unbraced by default. The minimum unbraced lengths are defined based on the soft and firm soil conditions the piles will be installed in accordance with Section 1810.2.1 of the 2009 and 2012 IBC. A minimum 5'-0 [1.52 m] unbraced length is applicable to a pile being installed in a firm soil (N-value ≥ 5). A minimum 10'-0 [3.05 m] unbraced length is applicable to a pile being installed in a soft soil (N-value < 5).




| Part Number | Pile Diameter (in. [mm]) | Allowable Load Capacity (kips [kN]) | Bearing Plate Size (in. [mm]) | Bracket Sleeve and Length (in. [mm]) |
|---|---|---|---|---|
| 4074 | 2 ⅜" [60] | 24.2 [108] | PL ⅝"x 8"x 0'-8 [16 x 203 x 203] | 2 ⅞" dia. x 8" [73 dia. x 203] |
| 4075 | 2 ⅞" [73] | 20.6 [92] | PL ⅝"x 4"x 0'-8 [16 x 102 x 203] | 3 ½" dia. x 10" [89 dia. x 254] |
| 4079 | 2 ⅞" [73] | 36.5 [162] | PL ⅝"x 8"x 0'-8 [16 x 203 x 203] | 3 ½" dia. x 10" [89 dia. x 254] |
| 4076 | 3 ½" [89] | 65.1 [290] | PL 1"x 9"x 0'-9 [25 x 229 x 229] | 2 ⅞" dia. x 10" [73 dia. x 254] |
| 4077 | 4 ½" [114] | 75.8 [337] | PL 1"x 9"x 0'-9 [25 x 229 x 229] | 3 ½" dia. x 10" [89 dia. x 254] |

**TABLE A-2 : Standard New Construction Pile Brackets**

*Custom sizes are available. Please contact engineering department for bracket sizing.

| Pile Dia. (in.) [mm] | Torque (ft-lbs) [kN-m] | $K_t$ | No Sleeve (5'-0 [127 mm] unbraced length) | | | Bracket with External Sleeve (5'-0 [127 mm] & 10'-0 [254 mm] unbraced length) | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | Bracket | ecc. (in.) [mm] | Allow. Capacity (kips) [kN] | Bracket | Sleeve Dia. (in.) [mm] | ecc. (in.) [mm] | 5'-0 [127 mm] Sleeve (kips) [kN] | 10'-0 [254 mm] Sleeve (kips) [kN] |
| 2 ⅜ [60] | 4,000 [5.42] | 10 [32.8] | 4045 | 1.937 [49] | 7.25 [32.2] | 4038 | 2 ⅞ [73] | 2.625 [67] | 19.0 [84.5] | 10.75 [47.8] |
| 2 ⅞ [73] | 8,000 [10.9] | 9 [29.5] | 4038 | 2.625 [66.7] | 20.3 [90.3] | 4021 | 3 ½ [89] | 3.125 [79] | 33.5 [149] | 22.5 [100] |
| 3 ½ [89] | 14,000 [19] | 7 [22.9] | 4021 | 3.125 [79] | 20.0 [89] | 4021.55 | 4 ½ [114] | 4.000 [102] | 49.0 [218] | 45.25 [201] |
| 4 ½ [114] | 23,000 [31] | 6 [19.7] | 4032 | 4.000 [102] | 42.0 [187] | N/A | N/A | N/A | N/A | N/A |

**TABLE A-3 : Capacity of Piles with Side Load Brackets[1,3]**

NOTES:

1. A pile with a side load bracket is defined in the International Building Code (IBC) in Chapter 18 as laterally unbraced. The minimum unbraced lengths are defined in accordance with Section 1810.2.1 of the 2009 and 2012 IBC based on the soft and firm soil conditions the piles will be installed in. A minimum 5'-0 [127 mm] unbraced length is applicable to a pile being installed in a firm soil (N-value ≥ 5). A minimum 10'-0 [254 mm] unbraced length is applicable to a pile being installed in a soft soil (N-value < 5).

2. The eccentricity, e, of on a helical pile is measured from the center of the pile to the face of the footing per ICC-ES AC358.The eccentricities shown are for the brackets indicated.

3. The capacities shown in Table A-3 are based on the assumption that the footing/grade beam has been properly prepared and that the bracket has a smooth and flush fit to the footing/grade beam. If the brackets do not have a proper and flush fit to the footing/grade beam, the eccentricity on the pile will be greater resulting in a higher bending stress and a lower capacity than that shown in Table A-3.

| **f'c** (psi) [kPa] | **Floor Slab Depth** (in.) [mm] | **As, min. per ACI** ($in^2$) [$mm^2$] | **Live Load** (psf) [kPa] | **Maximum Pile Spacing** | | **Pile Load**[3] (kip) [kN] | |
|---|---|---|---|---|---|---|---|
| | | | | **1 & 2 Span** (ft.-in.) [m] | **3 Span** (ft.-in.) [m] | **1 & 2 Span** | **3 Span** |
| 2,500 [17,237] | 4" [102] | 0.06 [38.7] | 40 [1.96] | 5'-9" [1.75] | 6'-5" [1.96] | 3.02 [13.4] | 3.78 [16.8] |
| | | | 50 [2.39] | 5'-5" [1.65] | 6'-1" [1.85] | 2.98 [13.3] | 3.72 [16.5] |
| | | | 100 [4.79] | 4'-4" [1.32] | 4'-10" [1.47] | 2.84 [12.6] | 3.55 [15.8] |
| | 5" [127] | 0.075 [48.4] | 40 [1.96] | 6'-10" [2.08] | 7'-7" [2.31] | 4.80 [21.4] | 6.00 [26.7] |
| | | | 50 [2.39] | 6'-5" [1.96] | 7'-2" [2.18] | 4.73 [21.0] | 5.91 [26.2] |
| | | | 100 [4.79] | 5'-3" [1.60] | 5'-10" [1.78] | 4.50 [20.0] | 5.63 [25.0] |
| | 6" [152] | 0.09 [58.1] | 40 [1.96] | 7'-9" [2.36] | 8'-8" [2.64] | 7.00 [31.1] | 8.75 [38.9] |
| | | | 50 [2.39] | 7'-5" [2.26] | 8'-3" [2.51] | 6.89 [30.6] | 8.62 [38.3] |
| | | | 100 [4.79] | 6'-1" [1.85] | 6'-10" [2.08] | 6.56 [29.1] | 8.20 [36.5] |
| | 8" [203] | 0.12 [77.4] | 40 [1.96] | 10'-4" [3.15] | 11'-7" [3.53] | 14.95 [66.5] | 18.78 [83.5] |
| | | | 50 [2.39] | 9'-10" [3.00] | 11'-0" [3.35] | 14.50 [64.5] | 18.15 [80.73] |
| | | | 100 [4.79] | 8'-1" [2.46] | 9'-1" [2.77] | 13.07 [58.1] | 16.50 [73.4] |
| 3,000 [20,684] | 4" [102] | 0.07 [45.2] | 40 [1.96] | 6'-0" [1.83] | 6'-9" [2.06] | 3.32 [14.8] | 4.16 [18.5] |
| | | | 50 [2.39] | 5'-8" [1.73] | 6'-4" [1.93] | 3.27 [14.5] | 4.09 [18.2] |
| | | | 100 [4.79] | 4'-6" [1.37] | 5'-1" [1.55] | 3.12 [13.9] | 3.90 [17.3] |
| | 5" [127] | 0.08 [51.6] | 40 [1.96] | 7'-2" [2.18] | 8'-0" [2.44] | 5.28 [23.5] | 6.60 [29.3] |
| | | | 50 [2.39] | 6'-9" [2.06] | 7'-7" [2.31] | 5.19 [23.1] | 6.49 [28.9] |
| | | | 100 [4.79] | 5'-6" [1.67] | 6'-2" [1.88] | 4.95 [22.0] | 6.19 [27.5] |
| | 6" [152] | 0.10 [64.5] | 40 [1.96] | 8'-2" [2.49] | 9'-1" [2.77] | 7.70 [34.2] | 9.62 [42.8] |
| | | | 50 [2.39] | 7'-9" [2.36] | 8'-8" [2.64] | 7.58 [33.7] | 9.47 [42.1] |
| | | | 100 [4.79] | 6'-5" [1.96] | 7'-2" [2.18] | 7.21 [32.1] | 9.02 [40.1] |
| | 8" [203] | 0.13 [83.9] | 40 [1.96] | 10'-10" [3.30] | 12'-0" [3.66] | 16.43 [73.1] | 20.00 [89.0] |
| | | | 50 [2.39] | 10'-4" [3.15] | 11'-6" [3.51] | 16.01 [71.2] | 20.00 [89.0] |
| | | | 100 [4.79] | 8'-6" [2.59] | 9'-6" [2.90] | 14.45 [64.3] | 18.05 [80.3] |

**TABLE A-4 : Max. Pile Spacing for Minimally Reinforced Concrete Floor Slab (ACI 318)**[1,2]

1. The maximum pile spacing shown are for floor slabs constructed of normal weight concrete (150 pcf) [.02 kN/m³]with minimum reinforcement per ACI 318.

2. The floor slab spans shown assumes the minimum floor slab reinforcement is placed in the center of the slab. Longer spans can be achieved if the slab reinforcement is verified to be larger and/or placed below the center line of the floor slab.

3. Pile loads include the dead load of the concrete for the slab thickness and pile spacing shown

| **Central Pile Shaft (in. [mm])** | **2 ⅜" [60]** | **2 ⅞" [73]** | **3 ½" [89]** | **4 ½" [114]** |
|---|---|---|---|---|
| Weight *(lb/ft) [kg/m]* | 4.43 [6.59] | 7.00 [10.4] | 8.67 [12.9] | 19.0 [28.3] |
| Outside Diameter *(in.) [mm]* | 2.375 [60.0] | 2.875 [73.0] | 3.500 [89.0] | 4.500 [114] |
| Inside Diameter *(in.) [mm]* | 1.995 [50.0] | 2.441 [62.0] | 2.992 [76.0] | 3.624 [92.0] |
| Wall Thickness, *t (in.) [mm]* | 0.190 [4.82] | 0.217 [5.51] | 0.254 [6.45] | 0.438 [11.1] |
| Cross-Sectional Area, *A* $(in.^2)$ $[cm^2]$ | 1.304 [8.41] | 1.812 [11.7] | 2.590 [16.7] | 5.589 [36.1] |
| Yield Strength, $F_y$ *(ksi) [Mpa]* | 65.0 [450] | 65.0 [450] | 65.0 [450] | 65.0 [450] |
| Tensile Strength, $F_u$ *(ksi) [Mpa]* | 80.0 [552] | 80.0 [552] | 80.0 [552] | 80.0 [552] |
| Moment of Inertia, *I* $(in.^4)$ $[cm^4]$ | 0.784 [32.6] | 1.61 [67.1] | 3.43 [143] | 11.7 [485] |
| Elastic Section Modulus, *S* $(in.^3)$ $[cm^3]$ | 0.660 [10.8] | 1.12 [18.4] | 1.96 [32.1] | 5.18 [84.9] |
| Plastic Section Modulus, *Z* $(in.^3)$ $[cm^3]$ | 0.909 [14.9] | 1.54 [25.2] | 2.68 [44.0] | 7.26 [119] |
| Radius of Gyration, *r (in.) [mm]* | 0.775 [19.7] | 0.923 [23.5] | 1.15 [29.2] | 1.45 [36.7] |
| Torsional Constant, *J* $(in.^4)$ $[cm^4]$ | 1.57 [65.6] | 3.22 [134] | 6.87 [286] | 23.3 [971] |
| Torsional Yield Moment *(ft-lb) [kN-m]* | 4,130 [5.59] | 7,800 [10.6] | 12,100 [16.4] | 23,000 [31.2] |
| Torsional Plastic Moment *(ft-lb) [kN-m]* | 4,470 [6.06] | 8,500 [11.5] | 14,140 [19.2] | 25,200 [34.2] |
| Default Torque Correlation Factor, $K_t$ $(ft^{-1})$ $[m^{-1}]$ | 10 [32.8] | 9 [29.5] | 7 [22.9] | 6 [19.7] |
| Ram Jack Torque Rating *(ft-lb) [kN-m]* | 4,000 [5.42] | 8,000 [10.9] | 14,000 [19.0] | 23,000 [31.2] |

**TABLE A-5 :** Ram Jack Helical Pile/Anchor Shaft Sizes and Properties

| **Helical Plate** (in.) [mm] | **Central Pile Shaft** (in.) [mm] | **Effective Area** ($ft^2$) [$cm^2$] | **Plate Thickness** (in.) [mm] | **Maximum Allowable Load Rating** (lbs) [kN] |
|---|---|---|---|---|
| 8 [203] | 2 ⅜ [60] | 0.318 [295] | ⅜ [9.50] | 47,500 [211] |
| 10 [254] | | 0.515 [478] | | 43,500 [193] |
| 12 [305] | | 0.755 [701] | | 37,500 [167] |
| 8 [203] | 2 ⅞ [73] | 0.304 [282] | ⅜ [9.50] | 73,000 [325] |
| 10 [254] | | 0.500 [465] | | 64,000 [285] |
| 12 [305] | | 0.740 [687] | | 45,500 [202] |
| 12 [305] | | 0.740 [687] | ½ [12.7] | 55,400 [246] |
| 14 [356] | | 1.02 [948] | | 46,800 [208] |
| 16 [406] | | 1.35 [1254] | | 40,200 [179] |
| 8 [203] | 3 ½ [89] | 0.283 [263] | ⅜ [9.50] | 79,800 [355] |
| 10 [254] | | 0.479 [445] | | 66,300 [295] |
| 12 [305] | | 0.719 [668] | | 65,700 [292] |
| 12 [305] | | 0.719 [668] | ½ [12.7] | 80,500 [358] |
| 14 [356] | | 1.00 [929] | | 64,000 [285] |
| 16 [406] | | 1.33 [1236] | | 58,000 [258] |
| 10 [254] | 4 ½ [114] | 0.435 [404] | ½ [12.7] | 117,000 [520] |
| 12 [305] | | 0.675 [627] | | 94,500 [420] |
| 14 [356] | | 0.959 [891] | | 83,000 [369] |
| 16 [406] | | 1.290 [1198] | | 76,000 [338] |

**TABLE A-6 :** Ram Jack Helical Plate Load Diameters and Load Ratings

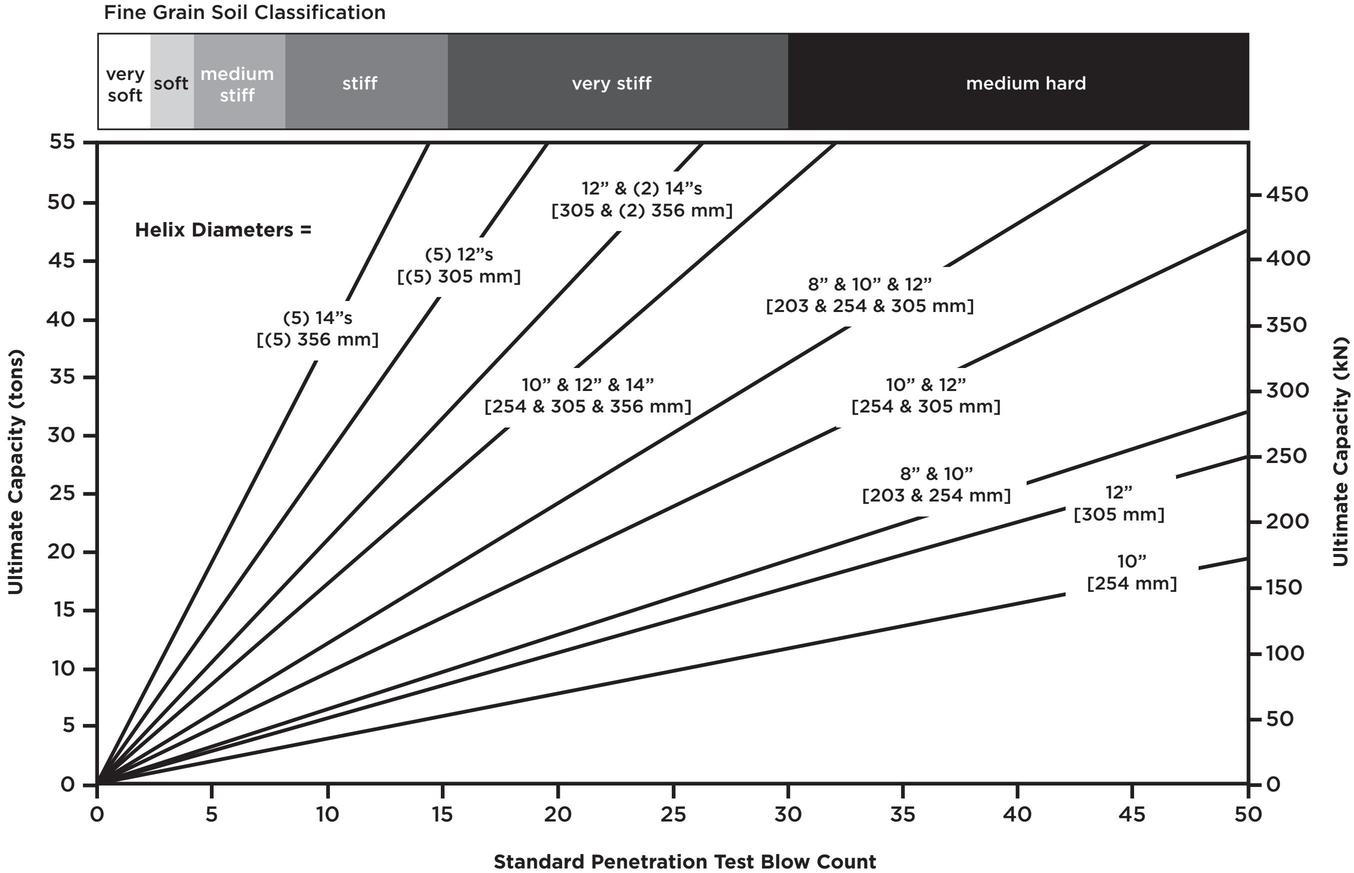



**FIGURE A-1 :** Helical Plate Configuration Selection Diagram for Fine Grained Soils[1]

1. Perko, H.A., Helical Piles – *A Practical Guide to Design and Installation*, John Wiley, 2009

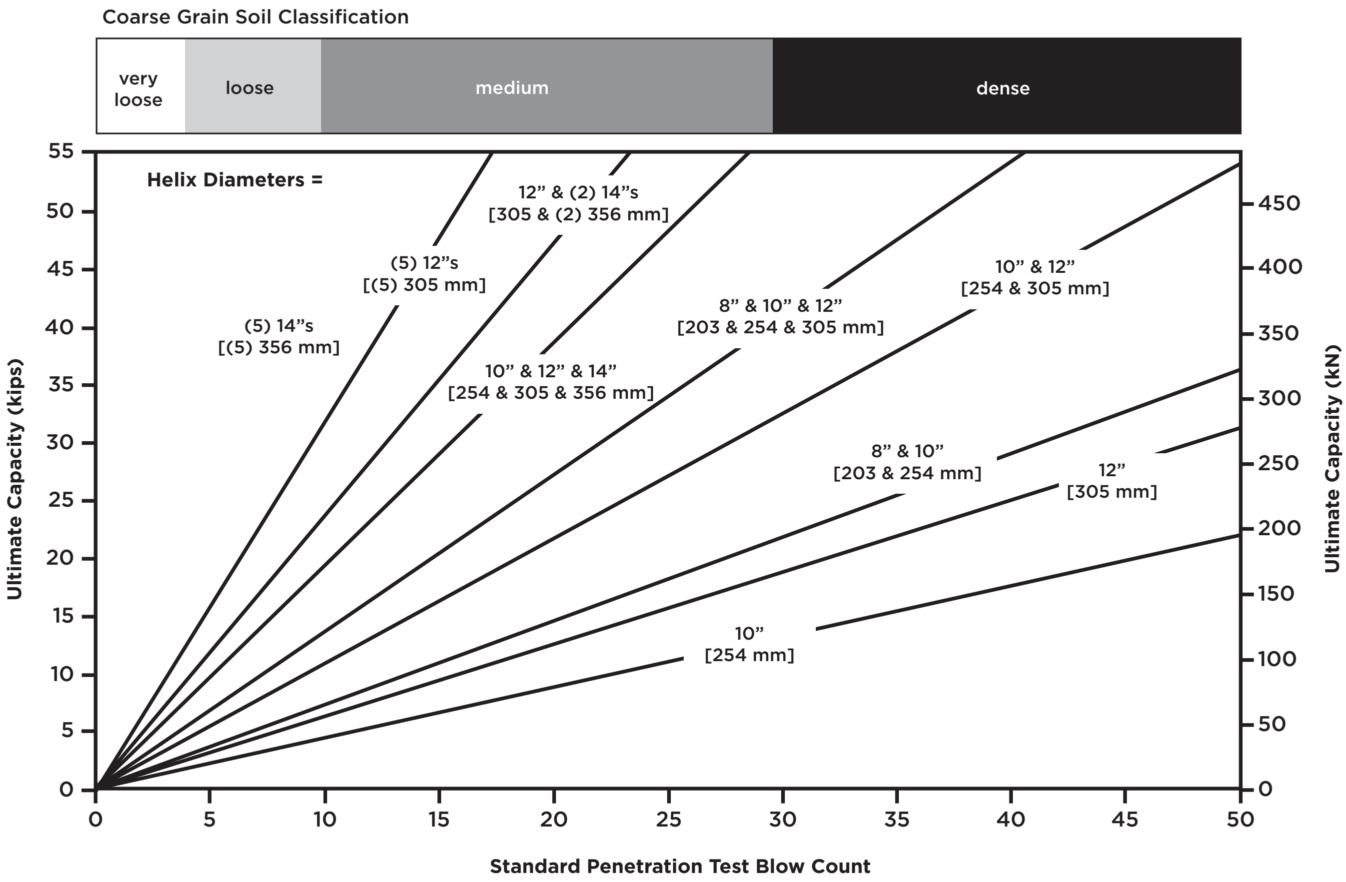



**FIGURE A-2 :** Helical Plate Configuration Selection Diagram for Coarse Grained Soils[1]

1. Perko, H.A., Helical Piles – *A Practical Guide to Design and Installation*, John Wiley, 2009

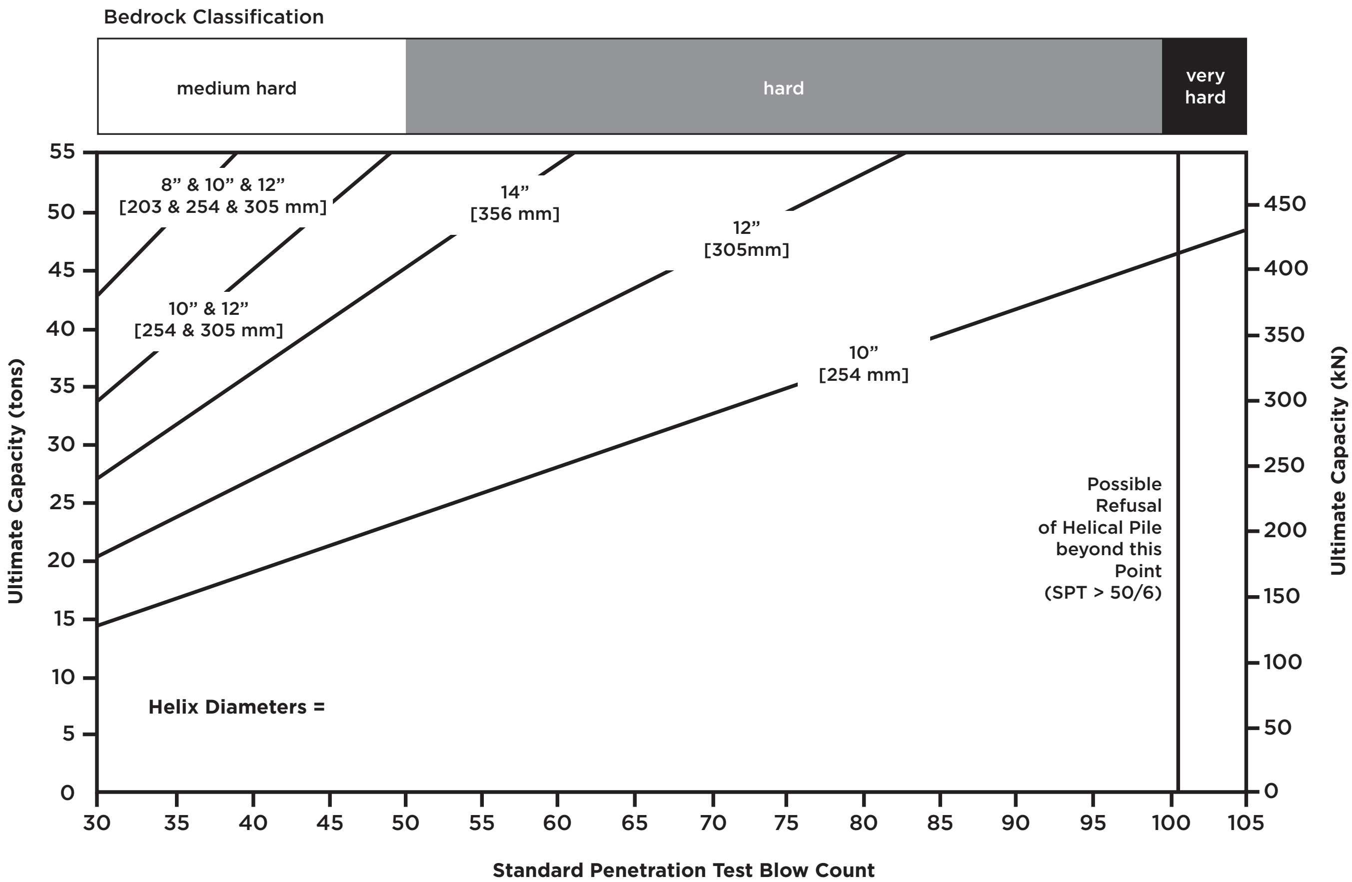



**FIGURE A-3 :** Helical Plate Configuration Selection Diagram for Weathered Bedrock[1]

1. Perko, H.A., Helical Piles – *A Practical Guide to Design and Installation*, John Wiley, 2009

| Blow Count, N (bpf) [blows/300 mm] | Consistency | Angle of Internal Friction $\Phi$ (deg) | Moist Unit Weight $\gamma_m$ (pcf) | Moist Unit Weight $\gamma_m$ (kN/m$^3$) | Submerged Unit Weight $\gamma_{sub}$ (pcf) | Submerged Unit Weight $\gamma_{sub}$ (kN/m$^3$) |
|---|---|---|---|---|---|---|
| 0 - 4 | Very Loose | < 28 | < 100 | < 15.7 | < 60 | < 9.42 |
| 5 - 10 | Loose | 28 – 30 | 95 – 125 | 14.9 - 19.6 | 55 – 65 | 8.63 - 10.2 |
| 11 - 30 | Medium | 31 – 36 | 110 – 130 | 17.3 - 20.4 | 60 – 70 | 9.42 - 11.0 |
| 31 - 50 | Dense | 37 – 41 | 110 – 140 | 17.3 - 22.0 | 65 – 85 | 10.2 - 13.6 |
| > 50 | Very Dense | > 41 | > 130 | > 2082 | > 75 | > 1201 |

**TABLE A-7 :** Coarse-Grained Soil Condition and Strength Parameters vs. SPT Blow Count (after Teng, 1962)

| Blow Count, N (bpf) [blows/300 mm] | Consistency | Unconfined Compressive Strength $q_u$, (psf) | Unconfined Compressive Strength $q_u$, (kN/m$^2$) | Saturated Unit Weight $\gamma_{sat}$ (pcf) | Saturated Unit Weight $\gamma_{sat}$ (kN/m$^3$) |
|---|---|---|---|---|---|
| 0 - 2 | Very Soft | 0 – 500 | 0 - 24.0 | < 100 – 110 | < 15.7 - 17.3 |
| 3 - 4 | Soft | 500 – 1,000 | 24.0 - 47.9 | 100 – 120 | 15.7 - 18.9 |
| 5 - 8 | Medium | 1,000 – 2,000 | 47.9 - 95.8 | 110 – 125 | 17.3 - 19.6 |
| 9 - 16 | Stiff | 2,000 – 4,000 | 95.8 - 192 | 115 – 130 | 18.1 - 20.4 |
| 17 - 32 | Very Stiff | 4,000 – 8,000 | 192 - 383 | 120 – 140 | 15.7 - 22.0 |
| > 32 | Hard | > 8,000 | > 383 | > 130 | > 20.4 |

**TABLE A-8 :** Fine-Grained Soil Condition and Strength Parameters vs. SPT Blow Count (ASCE, 1996)

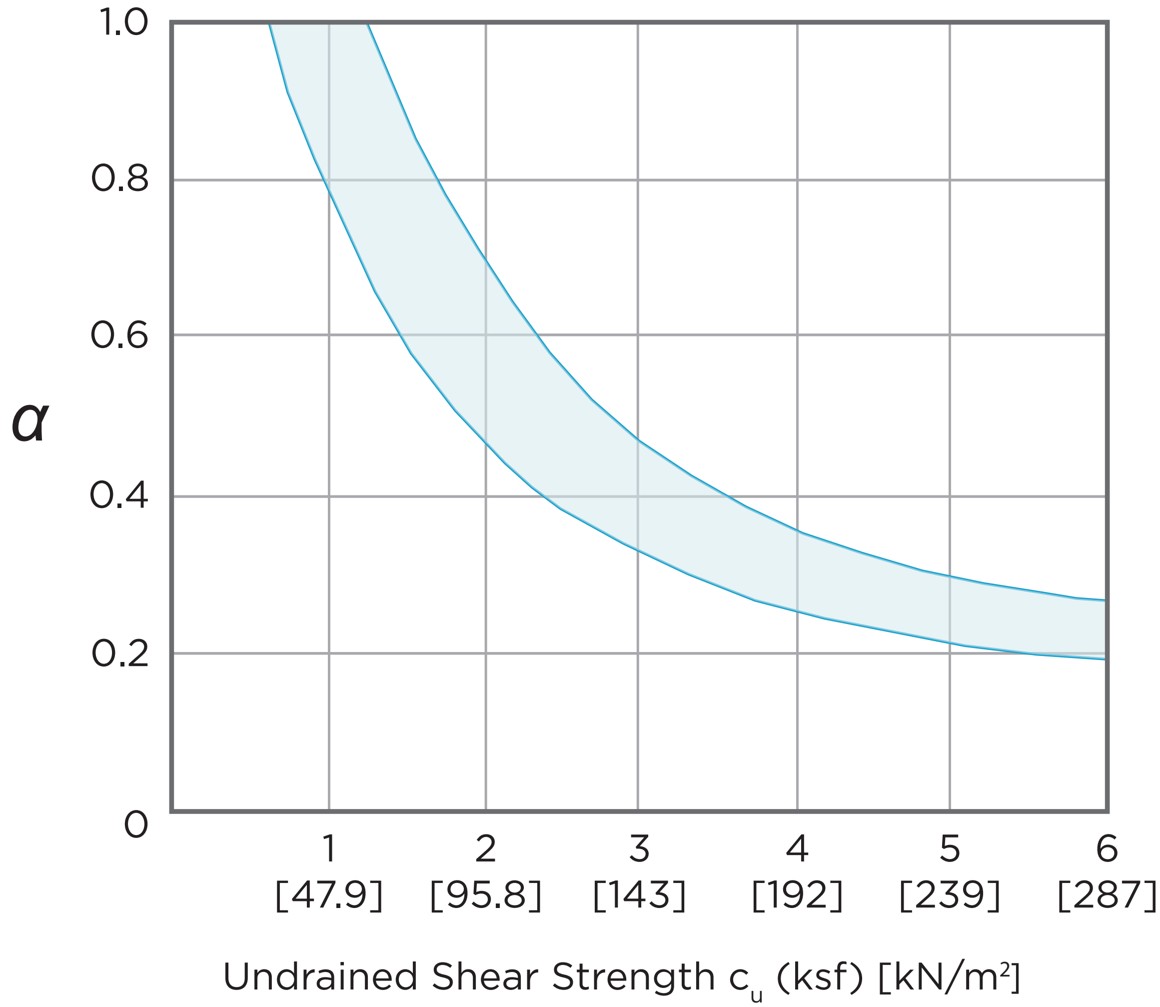



**FIGURE A-4 : Adhesion Coefficient α vs. Undrained Shear Strength cu (after Das, 1984)**

# GLOSSARY OF TERMS

**Adhesion** – Similar to cohesion except adhesion is the shear strength between the soil and another material, such as a pile shaft or concrete foundation. (Also referred to as skin friction)

**Allowable Load** – (reference Design Load)

**Allowable Stress Design (ASD)** – A code approved method of sizing structural members such that elastically computed stresses produced in the members by nominal loads do not exceed specified allowable stresses (also called working stress design).

**Angle of Internal Friction** ($\phi$) – Soil strength parameter that governs strength due to confining stress. It is only applicable to slits, sands and gravels and determined in the soils laboratory by using two primary tests: direct shear test and triaxial test.

**Battered Piles** – Helical piles installed at an angle other than vertical. Typically used for lateral stability or in areas where the pile heads are constrained together by a footing, grade beam or pile cap and the pile tips (toes) splay apart to avoid group effects.

**Bearing Stratum** – The undisturbed soil layer at any pile location which provides a significant portion of the axial resistance of an installed helical pile bearing on one or more helix plates.

**Bedrock** – Continuous rock layer that forms the Earth's crust; may outcrop at the ground surface or be buried under a thick layer of soil, cobble and/or boulders.

**Bentonite** – Highly expansive clay soil derived from chemical alteration of volcanic ash.

**Braced Pile** – A pile that has resistance to lateral forces and provides lateral stability in all directions.

**Bracket** – Manufactured steel cap or assembly that attaches to the head of the helical pile/anchor and is used to transfer loads to new or existing foundation elements; examples include side load brackets, new construction and wall plate assemblies.

**Buckling Capacity** – The capacity of a pile to resist a sudden change in its geometry or any of its elements under a loading condition.

**Buttress** – Section of reinforced concrete or masonry that is constructed inside of a structure and is oriented perpendicular to the primary wall system in order to resist lateral movement.

**Cohesion** (c) – Soil shear strength parameter of fine grained (clay) soil that is constant with respect to confining pressure. Basically, the forces within a soil that hold it together. Is determined in the soils laboratory by using two primary tests: direct shear test and triaxial test.

**Concentric Loading** – When the load or force is parallel to and transferred through the center of the pile shaft. An axial loaded pile with no load eccentricity. (Example: new construction piles, floor slab piles and tension anchors can be concentrically loaded).

**Consolidation** – Change in volume of soil involving the drainage of pore water over time under applied pressures. Typically associated with fine-grain soil (silts and clays) having low permeability. Can also be triggered by self-weight of soil, periodic changes in groundwater, placement of fill or other surcharge loads on the ground surface.

**Counterfort** – Section of reinforced concrete or masonry that is constructed outside of a structure and is oriented perpendicular to the primary wall system in order to resist lateral movement.

**Crowd** – Axial compressive force applied to the head (top) of the helical pile shaft during installation as required to ensure the pile progresses into the ground with each revolution a distance approximately equal to the helix pitch.

**Dead Load** – The weight of all building materials of construction incorporated into a structure including but not limited to walls, floors, roofs, ceilings, built-in partitions, cladding, architectural and structural items and fixed service equipment.

**Design Load** – The summation of all the loads exerting a force or action onto building. Engineers normally provide the design loads on the drawings. It is often referred to as allowable load, working load or service load.

**Down Drag** – The phenomenon where soft soils surrounding a pile consolidate and produce a downward force on the pile tending to increase the load requirements of the pile. The geotechnical engineer normally provides the additional down drag force in the soils report in a psf unit. This force is multiplied by the surface area of the pile (in ft. [m]) and the length of the pile (in ft. [m]) noted in the soils report. The design load of the pile must be increased by this additional down drag force.

**Dunnage** – (Also referred to as cribbing) Pieces of timber, steel or other material used to build up an area to form a temporary structure, such as a footing pad or construction platform. Often used on each end of the test beam when performing tension load tests on piles.

**Eccentric Loading** – When the load is not transferred through the center of the pile. The condition creates a moment which induces bending stresses into the pile shaft.

**Eccentricity** – The distance between the load and the center of the pile. It is also referred as a moment arm.

**End Bearing** – Term used to describe a pile that generates a majority of its capacity from the pile tip (toe).

**Extension** – A pile section with or without helical plates used to extend the lead section of the pile to a load supporting stratum at greater embedment depths. Each extension is connected with integral couplings which provide a rigid load transferring connection which form a continuous column of steel in the soil. Extensions with helical plates are often required in very soft soils and in soil nail applications.

**Factored Load** – Nominal loads that have been multiplied by a load factor.

**Glacial Till** – Material deposited by a glacier without subsequent transport by water. Typically, a heterogeneous mixture of soil and rock varying in size from soil to boulders.

**Gumbo** – Very plastic, typically very soft to medium stiff, clay soil which is dark brown or black in color.

**Hardpan** – Stratum of hard soil that is difficult to excavate, drill or install a helical pile through. Typically makes a good bearing stratum for helical piles to bear on.

**Helical Anchor** – Same as helical foundation / pile except the term anchor implies a tension application.

**Helical Foundation / Pile** – A factory manufactured steel foundation consisting of a central shaft with one or more helical shaped bearing plates. The terms foundation or pile typically refers to a compression application.

**Helix Driver** – A high torque hydraulic motor used to advance (screw) a helical pile into the soil to a load bearing stratum. Depending on the capacity of the helix driver, it may be either hand held or machine operated.

**Helix Plate** – A round plate formed into a ramped spiral. When rotated into the soil, the helical shape provides thrust along its longitudinal axis thus aiding in pile installation. After installation, the plate transfers axial load into the soil through bearing.

**In situ** – (pronounced "in sit-you") State of being undisturbed and in place.

**Installation Torque** – The resistance generated by a helical pile when installed into the soil. The installation resistance is a function of the strength properties of the soil the helical piles are being installed in as well as the shaft geometry of the pile shaft and helical plates.

**Kip** – A unit of force used by engineers and contractors. One kip [4.45 kN] is equal to 1,000 pounds lbs. [4450 kg]. (Example: 3,500 lbs is the same as 3.5 kip).

**Lateral Load** – A force acting perpendicular to a structure or structural member. Lateral loads are typically generated from wind and seismic loads as well as from soil and water pressures against retaining and basement walls.

**Lead Section** – The first helical pile section installed into the soil consisting of one or more helix plates welded to the pile shaft.

**Live Load** – A load produced by the use and occupancy of the structure that does not include construction or environmental loads such as wind load, snow load, rain load. Earthquake load, flood load or dead load. Minimum live loads structures must be designed for are listed in the building code and *ASCE 7 Minimum Design Loads for Buildings and Other Structures.*

**Load Factor** – A factor that accounts for deviations of the actual load from the nominal load, for uncertainties in the analysis that transforms the load into a load effect and for the probability that more than one extreme load will occur simultaneously.

**Loess** – Very porous aeolian (wind) deposited silt soil cemented by calcium carbonate or clay; can stand on vertical slopes and often collapses when wet.

**Marl** – Calcareous silt or clay of marine origin; ranges from very soft to stiff, typically very moist to wet.

**Micropile** – Small diameter (typically 4 in. [102 mm] to 12 in. [305 mm]) drilled pile with a single central reinforcing steel bar surrounded by lean cement grout; can be installed in very hard soils and rock; will not accept lateral loads or moments without a properly sized steel casing being placed in the upper portion of the pile.

**Moment** – A bending or buckling stress in a structural member. Normally induced by an eccentric loading condition.

**N-value** – The sum of the last two 6-inch blow count intervals required to drive a split spoon sampler into the soil per the standard penetration test. The soil boring logs will list the N-value directly or provide the blow counts for each of the 6-inch [152 mm] intervals. The N-value is very useful as it can be used to estimate several useful correlations such as the strength and consistency of a soil layer.

**Nominal Loads** – The magnitudes of loads specified in the building or design code for dead, live, soil, wind, snow, rain, flood and earthquake.

**Over Consolidated** – Refers to the stress history of the soil; indicates that the soil has previously experienced significantly higher confining pressures than its current state.

**Performance Test** – Full-scale load test on a sacrificial pile which typically consists of applying 150% to 200% of the design load to the pile and monitoring the pile's displacement.

**Phreatic Surface** – (see water table)

**Pier** – A deep foundation element that is plac in the soil as one unit. (No connections)

**Pile** – A deep foundation element that is made up of segments which are driven or torqued into the soil and are connected together.

**Pile Cap** – A steel or reinforced concrete structure of variable thickness and geometry placed over one or a group of piles and used to transfer loads to a column, grade beam, wall or other structure.

**Pilot Hole** – A pre-drilled hole made by a conventional drill rig or down-hole hammer and used in difficult ground conditions to break through obstructions, guide the helical or driven pile shaft and improve penetration; the pilot hole is typically the same size or slightly larger than the pile shaft diameter.

**Pitch** – Vertical distance between the leading edge and the trailing edge of a helix plate; also the approximate distance a helical pile should advance per revolution with proper installation.

**Plunging** – Continuous deflection of a pile under constant load.

**Proof Test** – Abbreviated load test on a production pile which typically consists of applying 100% of the design load to the pile and monitoring the pile's displacement.

**Safety Factor** – A load factor applied to the ultimate capacity of a helical or driven pile. The ultimate capacity of a pile is divided by the safety factor to derive the allowable or design load capacity of the pile. Typical safety factors for helical and driven piles are 2 and 3, respectfully.

**Soil Nail** – Helical anchor used in earth retention with helical plates spaced along the entire shaft so as to reinforce an entire zone of retained earth; also a term used with grouted anchors and other systems with continuous bond length.

**Solider** – Vertical pile used in earth retention.

**Spoil** – Excess soil or rock produced during excavation or drilling activities. Helical and driven piles do not produce drill spoil.

**Standard Penetration Test (SPT)** – The test is performed at predetermined depths during the soil boring activities. A split spoon sampler is placed in the boring hole and is driven into the soil by hammer blows. The standard weight of the hammer is 140 lbs [63.5 kg] and for each blow the hammer falls a distance of 30 in. [762 mm] The number of blows required to drive the sampler (3) 6-inch [152 mm] intervals are recorded. The number of blows recorded for the last two intervals are added together to give the standard penetration number at that depth. This number is generally referred to as the N-value.

**Tieback** – Helical anchor used in earth retention with helical bearing plates located a significant distance past the active zone of retained earth and a central shaft that extends through the active zone to a rigid wall facing that is capable of withstanding the tension force of the helical anchor.

**Ton** – A unit of force that is equal to 2,000 lbs [907 kg] or 2 kips [8.9 kN]. (Example: 15 tons is the same as 30,000 lbs. or 30 kips [13608 kg or 133 kN]).

**Torque Bar** – (Also referred to as reaction or torsion bar) A long tube, bar or I-beam used as counter resistance to the torsional force generated from a portable, hand operated, free standing torque motor (driver) during helical pile installation. One end of the bar is attached to the torque motor and the other end is attached to an immovable structure.

**Torque Rating** – The maximum torque energy that can be applied to a helical pile shaft during installation into the soil.

**Unbraced Length** – Piles standing in air, water, and fluid soil as well as the distance between braced points of a pile. The International Building Code (IBC) defines the minimum unbraced length a pile must be designed for, if it is not defined by as a braced system, as 5 ft. [1.52 m] for firm soil (N-value > 5) and 10 ft. [3.01 m] for soft soil (N-value < 5).

**Under Consolidated** – Refers to soil that is not in equilibrium with its self-weight and is currently undergoing consolidation over time.

**Waler** – Horizontal beam spanning between tiebacks, rakers or braces in earth retention systems. Commonly used on sheetpile and solider walls.

**Water Table** – Depth of ground water below the surface of the soil. (Also referred to as phreatic surface).