Technical Design Manual Edition 3

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SECTION 1

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SYMBOLS USED IN THIS SECTION

FS	Factor of Safety	1-4
ASTM	American Society for Testing and Materials	1-6
BOCA	Building Officials and Code Administrators International	1-6
ICBO	International Conference of Building Officials	1-6
SBCCI	Southern Building Code Congress International	1-6
ICC	International Code Council	1-6
PISA	Power Installed Screw Anchor	1-9
RR	Round Rod	1-9
SS	Square Shaft	1-9
HS	High Strength	1-10
PIF	Power Installed Foundation	1-10
SLF	Street Light Foundation	1-10
T/C	Tension/Compression	1-11
ICC-ES	ICC Evaluation Service, Inc.	1-13
kips	Kilopound	1-11

DISCLAIMER

The information in this manual is provided as a guide to assist you with your design and in writing your own specifications.

Installation conditions, including soil and structure conditions, vary widely from location to location and from point to point on a site.

Independent engineering analysis and consulting state and local building codes and authorities should be conducted prior to any installation to ascertain and verify compliance to relevant rules, regulations and requirements.

Hubbell Power Systems, Inc., shall not be responsible for, or liable to you and/or your customers for the adoption, revision, implementation, use or misuse of this information. Hubbell, Inc., takes great pride and has every confidence in its network of installing contractors and dealers.

Hubbell Power Systems, Inc., does NOT warrant the work of its dealers/installing contractors in the installation of CHANCE® Civil Construction foundation support products.

DEFINITION of ATLAS RESISTANCE® PIERS

The ATLAS RESISTANCE® Pier utilizes the weight of the structure as its reaction system to drive or push the pipe pier sections into the soil. Hubbell/CHANCE® has developed a lasting solution for many distressed foundation problems through its patented and tested ATLAS RESISTANCE® Pier System. The pier is an assembly of structural steel components that include a pier head assembly attached to the foundation or slab, which is then mounted on a steel pier that is installed to bedrock or firm bearing stratum. The unique friction reduction collar on the lead section of the pier reduces skin friction on the pier pipe during installation. The pier capacity is primarily from end bearing on a hard/dense soil stratum. The ATLAS RESISTANCE® Pier has been successfully driven to depths of 200 feet to ensure proper and verified support.

Hubbell Power Systems, Inc. offers a broad range of applications for ATLAS RESISTANCE® Piers, including foundation underpinning and slab underpinning applications.

The ATLAS RESISTANCE® Pier is a manufactured, two-stage product designed specifically to produce structural support strength. First, the pier pipe is driven to a firm bearing stratum; then the lift equipment is typically combined with a manifold system to lift the structure (if required). This procedure provides measured support strength. Piers are spaced at adequate centers where each pier is driven to a suitable stratum and then tested to a force greater than required to lift the structure. This procedure effectively load tests each pier prior to lift and provides a measured Factor of Safety (FS) on each pier at lift.

Workspace is not normally a problem when using ATLAS RESISTANCE® Piers. They can be installed using portable equipment in an area that measures approximately three feet square. The pier may be installed from the interior or on the exterior of the footing.

HISTORY of PUSHED STEEL PILE SYSTEMS

The history of piling systems extends back to the ancient Greek, Roman and Chinese societies. Although numerous methods and materials have been utilized throughout the centuries, modern construction methods and practices have mandated the repair and remediation techniques of today's structures. The use of excavated foundations, footings, walls and beams, although providing adequate support in some soil conditions, have proven to be less desirable in a multitude of soil and site profiles. Fill areas, compressible soils, organics and expansive soils offer a greater challenge in the long term stability of foundations and are an underlying cause of billions of dollars of structural remedial repairs worldwide. The need for deep foundation underpinning systems increased dramatically in the 20th century with the building booms and growth in metropolitan areas.

In 1896, Jules Breuchaud, a contractor and civil engineer residing in New York, patented an "improved method of underpinning the walls of existing buildings" by a system of driving hollow, tubular column sections to bedrock or other firm strata using hydraulic jacks and a transverse beam system. Two sets of columns driven at opposite sides of the wall and beneath a transverse beam or beams utilized "the superincumbent weight of the building to resist the pressure of the hydraulic jacks, whereby the latter exerts a very powerful force in driving the column sections to bearing strata". This method allowed for permanent or temporary support and raising or lowering of structures by patent definition.

In 1897, Richard S. Gillespie, another New York entrepreneur, patented a similar method of underpinning existing buildings by means of a reaction, or "pressure-resisting" column that provided the reaction force to drive "cylindrical columns" using a system of cantilevered beams, tie-rods and hydraulic rams restrained to the reaction column to allow for sinking pipe sections to bearing strata for support. This cantilevered approach allowed for placement of pipe supports beneath the middle of the building wall in lieu of the twin-column method developed by Breuchaud and also provided a method for driving deep foundation piles for new construction.

Another substantial advancement was developed and patented by Lazarus White, again of New York, in 1917. White addressed long-term stability issues encountered in previous similar methods by introducing the practice of pre-loading or as he termed it "the first or temporary load" encountered from the reaction during pushing the pipe against the structure load to a pre-determined capacity equal to 150% of the required load which is consistent with the installation methodology ATLAS RESISTANCE® Piers use today. Additionally, White also documented theories of the soil "pressure bulb" created at the pile tip which assumes compression of the soil beyond the periphery of the pile for contributing to "a load in excess of that attributable to the resistance of the area of the end of the pile".

One early documented adaptation incorporating the use of a steel, eccentrically loaded bracket with pushed piles as a load transfer method was revealed in a 1959 patent application by Guy Henry Revesz and Jack C. Steinsberger of Illinois. This patent, which was recognized in 1961, cited references to the early work of Breuchaud and Gillespie. The method of 150% pre-loading which was prevalent in the White Patent of 1917 is also a standard criterion in this 1961 patent methodology. Numerous similar patents for pushed or jacked piers surfaced in the 60's and 70's, further extending the work of these early pioneers.

APPLIED RESEARCH and DEVELOPMENT

The development of the ATLAS RESISTANCE® Pier system early in the 1980's created new opportunities for building owners to reclaim the hard-earned equity of their structure's previously de-valued state as a result of settlement. Since the ATLAS RESISTANCE® Pier is designed to actually restore the structural integrity and original elevation, building values and salability are usually recovered. Their two stage installation method provides validation of load capacity along with a verifiable Factor of Safety for each pier installed.

Essentially, every single pier is load tested during the installation process. The friction reduction collar on the lead pier section reduces skin friction during installation which allows less driving force to required to reach the bearing stratum. From the early three-piece ATLAS RESISTANCE® Pier System patent, numerous products and specialty equipment have been developed to serve the industry. The ATLAS RESISTANCE® 2- Piece, Plate Pier, Continuous Lift and Pre-Drilled systems represent the flexibility in design and application of the ATLAS RESISTANCE® product line. New applications and modifications of these systems are continually in a state of expansion and growth to meet the needs of the deep foundation industry and to maintain the "state of the art" status and reputation of the ATLAS RESISTANCE® Product line.

ATLAS RESISTANCE® Piers have earned the support of the engineering community through years of focus on engineering, preliminary design, continuing education through formal training and overall team effort philosophy of Hubbell Power Systems, Inc., its application engineers and its installing contractor force. The broad Hubbell Power Systems, Inc. product line is a direct result of the effort and interaction of innovative engineers, installing contractors and owners to provide sound, economical solutions to structure settlement in a multitude of environments throughout the country.

TESTING and CODE COMPLIANCE

ATLAS RESISTANCE® Pier products have been subjected to full scale load tests under actual field conditions to determine their ultimate capacity. These tests were designed, conducted and certified under the direction by Dr. David C. Kraft, Ph.D., PE. The field load tests were carried out in close conformance to ASTM D1143-81, Piles under Static Axial Compressive Load. These field load tests were conducted in Independence, Missouri between June 3, and July 6, 1989.

ATLAS RESISTANCE® Models AP-2-3500.165 and AP-2-3500.165(M) comply with the structural provisions of the most recent editions of the Building Officials and Code Administrators International (BOCA) National Code, International Conference of Building Officials (ICBO) Uniform Code, Southern Building Code Congress International (SBCCI) Standard Code and the 2000 International Building and Residential Codes of the International Code Council (ICC) with the new 2002 Accumulative Supplement. A copy of this evaluation report, NER-579, is available online at www.abchance.com.

APPLICATIONS

ATLAS RESISTANCE® Piers are used primarily for underpinning and the repair of residential and commercial buildings, retaining structures and slabs. They can be installed in either interior or exterior locations. They have been used to repair equipment and machinery foundations, warehouse buildings, tower foundations, etc. Special remedial repair brackets can be connected to either the bottom or side of an existing foundation. They can also be connected to the sides of circular or flat building columns. ATLAS RESISTANCE® Piers not only stop settlement, but can also be used to raise the structure, thus closing cracks and correcting other structural flaws resulting from settlement and/or ground movement. The design process should involve professional engineering input. Specific information involving the structure, soil characteristics and foundation conditions must be evaluated and incorporated into the final design.

ADVANTAGES of ATLAS RESISTANCE® PIERS

The advantages of ATLAS RESISTANCE® Piers are similar in nature to those cited later in this section for CHANCE® Helical Piles/Anchors. They are used when a deep foundation solution is required. They are installed with light weight, portable equipment that allows for installations in limited access areas and in low overhead conditions. Their installation is not weather dependent. They are ideal in contaminated soil areas, since no soil has to be removed for installation. Table 1-1 summarizes some of the advantages of ATLAS RESISTANCE® Piers.

INTRODUCTION

ATLAS RESISTANCE® PIER ADVANTAGES, TABLE 1-1

Summary of ATLAS RESISTANCE® Pier Advantages	
• No need for concrete to cure	• All weather installation
• Fast turnkey installation	• Solution for:
• Immediate loading	- Restricted access sites
• Equipment portability	- High water table
• Pre-engineered system	- Weak surface soils
• Easily field modified	• Environmentally friendly
• On site load test on each pier	• No vibration
• Two stage installation for load capacity checks	• No spoils to remove

DEFINITION of HELICAL PILES/ANCHORS

The helical pile/anchor is basically a deep foundation system used to support or resist any load or application. Installed by mobile equipment ranging in size from lightweight units to heavier units depending on the load requirements, it can be loaded immediately. The helical pile/anchor's elegant simplicity is its greatest asset. Its mechanical design and manufacture balance the capacities of its three basic parts and maximize the efficient use of their material.

Essential Elements:

1. At least one bearing plate (helix)

Dies form each steel bearing plate into a true helix. The plates are formed in a true helical shape to minimize soil disturbance during installation (as opposed to the inclined plane of an auger which mixes soil as it excavates). Properly formed helical plates do not measurably disturb the soil. The helical bearing plates transfer the load to the soil bearing stratum deep below the ground surface. Hubbell Power Sytems, Inc. defines "deep" as five helix diameters vertically below the surface where the helical plate can develop full capacity of the plate-to-soil interaction.

2. A central shaft

During installation, the central steel shaft transmits torque to the helical plate(s). The shaft transfers the axial load to the helical plate(s) and on to the soil bearing stratum. Theoretically, the shaft needs to be larger than the shaft material's allowable stress. Realistically, the shaft also needs to be strong enough to resist the torque required for installation and large enough in section for the soil to resist buckling, if used in a compression application.

3. A termination

The termination connects the structure to the top of the helical pile/anchor transferring the load down the shaft to the helical plate(s) to the bearing soil. To evenly distribute the structure load to the helical piles/anchors, the termination may be a manufactured bracket or an attachment produced on site as designed by the structural engineer. Such aspects dictate the termination's configuration as a function of its application and may range from a simple threaded bar to a complex weldment, as is appropriate to interface with the structure.

HISTORY and SCIENCE of CHANCE® HELICAL PILES/ANCHORS

In 1833, the helical pile was originally patented as a "screw pile" by English inventor Alexander Mitchell. Soon after, he installed screw piles to support lighthouses in tidal basins of England. The concept also was used for lighthouses off the coasts of Maryland, Delaware and Florida.

Innovations of the helical pile/anchor have been advanced by both its academic and commercial advocates. Considerable research has been performed by public and private organizations to further advance the design and analysis of helical piles and anchors. A partial list of publications related to helical pile research is included at the end of this chapter. Much of the research was partially funded or assisted by Hubbell Power Systems, Inc. Contributions of financial, material and engineering support for research ventures related to helical piles is continued today by Hubbell Power Systems, Inc.

Today, readily available hydraulic equipment, either small or large, can install helical pile/anchors almost anywhere. Backhoes, skid-steer loaders and mini-excavators are easily fitted with hydraulically driven torque motors to install helical pile/anchors in construction sites inaccessible by the larger equipment required for other deep foundation methods. According to site conditions, installation equipment can include guided-head and articulated-head torque-head machinery, self-propelled, carrier-mounted, tracked, wheeled or floating.

The following summarizes a short list of Hubbell Power Systems, Inc. contributions to the helical pile/anchor industry. In 1940, the A.B. Chance Company sold the first commercially offered helical anchor tension application. It was installed by hand using a small tubular wrench. Other early developments include soil classifying measurement devices.

- **PISA® (Power Installed Screw Anchors)**

In the late 1950's, the A.B. Chance Company introduced the patented PISA® system. This coincided with the invention of truck-mounted hole-digging equipment following World War II. The PISA® system has become the worldwide method of choice for guying pole lines of electric and telephone utilities.

The PISA® system's all-steel components include one or two helix plates welded to a square hub, a rod threaded on both ends, a forged guy wire eye nut, and a special installing wrench. The square-tube anchor wrench attaches to the kelly bar of a digger truck, fits over the rod, engages the helical hub and typically installs a PISA® anchor in 8 to 10 minutes. Rod and wrench extensions may be added to reach soil layers which develop enough resistance to achieve capacity. PISA® rods come in 5/8", 3/4" and 1" diameters.

Through A.B. Chance Company testing and close contact with utilities, the PISA® anchor family soon expanded to develop higher strengths capable of penetrating harder soils including glacial till. This quickly gave rise to the development of CHANCE® Helical Piles/Anchors with higher capacities and larger dimensions.

More recent developments include the SQUARE ONE® (1980) and the TOUGH ONE® (1989) patented guy anchor families with 10,000 and 15,000 ft-lb installing torque capacities. Unlike previous PISA® designs, these anchor designs are driven by a wrench that engages inside, rather than over, their welded socket hubs. Both use the PISA® extension rods with threaded couplings.

- **Round Rod (RR) Anchors**

In 1961, the A.B. Chance Company developed extendable Type RR multi-helix anchors, originally for use as tiedowns for underground pipelines in poor soil conditions on the Gulf of Mexico coast. These anchors are not driven by a wrench; instead, installing torque is applied directly to their 1-1/4" diameter shafts. Type RR anchors worked well in weak surficial soils, but their shaft (although extendable by plain shafts with bolted upset couplings) did not provide enough torque strength to penetrate very far into firm bearing soils.

- **Square Shaft (SS) Anchors**

Development of a high-torque, shaft-driven, multi-helix anchor began in 1963, culminating in the introduction of CHANCE® Type SS 1½" Square Shaft multi-helix anchors in 1964-65. The SS anchor family since has expanded to include higher-strength 1-3/4", 2" and 2-1/4" square shafts. With the acquisition of Atlas Systems, Inc., in 2005, the Type SS product line has been expanded to include 1-1/4" square shafts. Extension shafts with upset sockets for the 1-1/4", 1-1/2", 1-3/4", 2" and 2-1/4" square shafts also lengthen these anchors to penetrate most soils at significant depths for many civil construction applications including guying, foundations, tiebacks and more recently, soil nails (the CHANCE SOIL SCREW® Retention Wall System, 1997).

- **High Strength (HS) Anchors/Piles [now called Round Shaft (RS) Piles]**

Later in the 1960's, Type HS anchors developed first for high-torque guying requirements later were applied as foundation helical piles for utility substations and transmission towers. The HS anchor family has 3-1/2" pipe shafts which may be lengthened by extensions with swaged couplings. HS anchors now are used for a wide array of foundation applications. The Type HS Piles/Anchors are now referred to as Type RS Piles/anchors. Hubbell Power Systems, Inc. now offers 2-7/8" (RS2875.203, RS2875.276), 4-1/2" (RS4500.337), 6" (RS6625.280) and 8" (RS8625.250) pipe shafts in addition to the 3-1/2" (RS3500.300).

- **Power Installed Foundation (PIF) Anchors/Piles**

Also launched in the 1960's were non-extendable anchors termed Power Installed Foundations. PIF sizes and load capacities support requirements for foundations that support a broad range of equipment, platforms and field enclosures. Most versatile are the 5-ft to 10-ft-long PIFs with pipe shafts of 3-1/2", 4", 6-5/8", 8-5/8" and 10-3/4" diameters, each with a single helix of 10", 12", 14" or 16" diameter. Integral base plates permit direct bolt-up connections on either fixed or variable bolt-circle patterns.

Bumper post anchors are similar to the 3½"-shaft PIF, but with fence-type caps instead of base plates, to serve as traffic barriers around booths, cabinets, doorways, etc. One with a 2-3/8" pipe shaft 69" long is called a Square Drive Foundation for its 2"- square drive head. The solid head is internally threaded for adding a straight stud or adjustable leveling pad after installation.

- **Street Light Foundation (SLF) Anchors/Piles**

In 1972, CHANCE® Street Light Foundations (SLF) were introduced. Anchors with pipe shaft diameters of 6-5/8", 8-5/8" and 10-3/4" in fixed lengths of 5, 8 and 10 feet. Complete with an internal cableway, these foundations with bolt-up base plates deliver the quick solution their name implies and now are used to support similar loads for a variety of applications.

- **Helical Pier Foundation Systems/Piles**

In 1985, CHANCE® patented products for repairing foundations of all residential and commercial buildings were introduced. Originally based on Type SS helical anchors, its special foundation repair brackets transfer structural loads to stable soil strata below weak surface conditions. Since then, the product also has been used to deepen foundations for new construction by installing the helical piles at intervals between footing forms prior to pouring reinforced concrete.

- **CHANCE HELICAL PULLDOWN® Micropiles**

Developed in 1997, for sites with especially weak surface soils, this patented innovative application of the helical pile integrates portland-cement-based grout to stiffen the shaft. By "pulling down" a special flowable grout as the foundation is screwed into the soil, the result is a pile with both a friction-bearing central shaft and end-bearing helical plates in competent substrata. Where needed for poor surface conditions, this performance combination converts sites previously deemed as "non-buildable" to usable sites suited for not only building construction but also telecom tower foundations in areas inaccessible by equipment utilized for other deep foundation methods. It employs SS, RS and combinations of these two types of helical piles.

- **Large Diameter Pipe Piles (LDPP)**

To meet an industry need for helical piles with higher tension/compression capacities and larger bending resistance, the large diameter pipe pile research project was initiated in 2007. The research culminated in product offerings including extendable large diameter piles with a box coupling system capable of installation torques as high as 60,000 ft-lbs and compression capacities of 300 kips.

APPLIED RESEARCH and DEVELOPMENT

In addition to products developed for specific applications, significant contributions to the applied science of helical piles and anchors by Hubbell Power Systems, Inc. have been achieved. Among the various subjects which have expanded the body of knowledge are:

- **CHANCE® Civil Construction Soil Classification**

In 1945, A.B. Chance Company listed the first earth anchoring manual, which classified soils according to holding capacities as related to proper anchor selection. At sites where soil data was available, either by sample excavation or some rudimentary means of probing subsurface strata, this chart imparted a valuable basis for recommending the proper helical pile or anchor for a given load.

CHANCE® CIVIL CONSTRUCTION SOIL CLASSIFICATION , TABLE 1-2

Class	Common Soil-Type Description	Geological Soil Classification	Probe Values in/lbs (nm)	Typical Blow Count N per ASTM D1586
0	Sound hard rock, unweathered	Granite, Basalt, Massive Limestone	N.A	N.A
1	Very dense and/or cemented sands; coarse gravel and cobbles	Caliche, (Nitrate-bearing gravel/rock)	750-1600 (85-181)	60-100+
2	Dense fine sands; very hard silts and clays (may be preloaded)	Basal till; boulder clay, caliche; weathered laminated rock	600-750 (68-85)	45-60
3	Dense sands and gravel; hard silts and clays	Glacial till; weathered shales, schist, gniess and siltstone	500-600 (56-68)	35-50
4	Medium dense sand and gravel; very stiff to hard silts and clays	Glacial till; hardpan; marls	400-500 (45-56)	24-40
5	Medium dense coarse sands and sandy gravels; stiff to very stiff silts and clays	Saprolites, residual soils	300-400 (34-45)	14-25
6	Loose to medium dense fine to coarse sands to stiff clays and silts	Dense hydraulic fill; compacted fill; residual soils	200-300 (23-34)	7-14
**7	Loose fine sands; Alluvium; loess; medium-stiff and varied clays; fill	Flood plain soils; lake clays; adobe; gumbo, fill	100-200 (11-23)	4-8
**8	Peat, organic silts; inundated silts, fly ash very loose sands, very soft to soft clays	Flood plain soils; lake clays; adobe; gumbo, fill	less than 100 (0-11)	0-5

Class 1 soils are difficult to probe consistently and the ASTM blow count may be of questionable value.

* Probe values are based on using CHANCE® Soil Test Probe, catalog number C309-0032

** It is advisable to install anchors deep enough, by the use of extensions, to penetrate a Class 5 or 6, underlying the Class 7 or 8 Soils.

- **Torque-to-Capacity Relationships**

Installation torque-to-load capacity relationship is an empirical method that the A.B. Chance Company originally developed in the 1960's. The idea was that the installation energy (torque) required to install a helical pile/anchor can be correlated to its ultimate load capacity in soil. The analogy is similar to screwing a wood screw into a piece of wood. It takes more torsional energy to screw into dense wood, such as oak, than it does to screw into a soft wood, such as pine. Likewise, a wood screw in oak will require more effort to pull out than the same wood screw in pine. The same is true for helical piles/anchors in soil. Dense soil requires more torque (more energy) to install compared to a soft soil; and likewise dense soil will generate higher load capacity compared to a soft soil.

For the torque correlation method to work, torque must be measured. Hubbell Power Systems, Inc. Engineers have developed both mechanical and electronic indicators over the years, many of which are commercially available for torque measurement in the field. The most recent addition to the product line is the C3031578 Digital Torque Indicator, which features a continuous reading digital readout of installation torque up to 30,000 ft-lb. The Digital Torque Indicator is also available with a wireless remote display and a data logger. The data logger records torque and other installation data that is used as a permanent record.

- **Soil Mechanics Principles**

In the 1970s and early 1980s, changes in design philosophy led Hubbell Power Systems, Inc. Engineers to recognize that a deep buried plate (i.e., pile/anchor helix) transferred load to the soil in end-bearing. Theoretical capacity could then be calculated based on Terzaghi's general bearing capacity equation. The individual bearing method, discussed in detail in Section 5, calculates the unit bearing capacity of the soil and multiplies it by the projected area of the helix plate. The capacity of individual helix plate(s) is then summed to obtain the total ultimate capacity of a helical pile/anchor. Today, the individual bearing method is commonly used in theoretical capacity calculations and is recognized as one method to determine helical pile capacity in the International Building Code (IBC).

- **100+ Years of Field Test Data**

Hubbell Power Systems, Inc. Engineers continuously prove theory by conducting literally thousands of load tests in the field. It has been said that soil occurs in infinite variety of engineering properties can vary widely from place to place. This variability makes in-situ testing a vital part of sound geotechnical engineering judgment. Test results are available from Hubbell Power Systems, Inc. for typical capacity of helical piles/anchors in soil.

- **HeliCAP® Helical Capacity Design Software**

Hubbell Power Systems, Inc. Engineers developed HeliCAP® Helical Capacity Design Software to assist the designer to select the correct helical lead configuration and overall pile/anchor length. It also estimates the installation torque. This program

makes the selection of helical piles/anchors easier and quicker than hand calculations. To obtain a copy of the software, please contact your local Hubbell Power Systems, Inc. Distributor. Contact information for each distributor can be found at www.abchance.com.

- **SELECT-A BASE™ Lighting Base Program**

The SELECT-A BASE™ Lighting Base Program is an on-line program developed in 2009 by Hubbell Power Systems, Inc. Engineers for preliminary foundation selection for roadway, area, and site lighting poles and luminaires. The program incorporates a database of CHANCE® Lighting Bases designed using more than 100 years of research, development and testing of earth anchor systems. The program inputs include loading conditions (wind, moment, and/or lateral), pole/pole arm details and soil data. The software is free and easy to use on-line at www.abchance.com.

- **Inter-Helix Spacing**

Load transfer either above or below the helix plate results in a stress zone within a defined soil volume. For individual bearing to work properly, the helix plates must be spaced far enough apart to avoid overlapping their stress zones. The key is to space the helix plates just far enough apart to maximize the bearing capacity of a given soil. This works to reduce the overall length of the helical pile/anchor and increases the likelihood for all helix plates to be located in the same soil layer; which in turn leads to more predictable torque-to-capacity relationships and better load/deflection characteristics. Through years of research, the Hubbell Power Systems, Inc. Engineers determined that the optimal spacing for helix plates is three diameters. More specifically, the optimum space between any two helical plates on a helical pile/anchor is three times the diameter of the lower helix. Today, all CHANCE® Helical Piles/Anchors are manufactured using the industry standard of three diameter spacing.

- **Industry Standard: Helical Pile/Anchor Form Fits Function**

The helical pile/anchor is not a complex product, but it continues to serve ever-expanding roles in civil construction applications. However, you will probably not find helical piles/anchors mentioned in most foundation engineering textbooks; and as such familiarity with helical piles/anchors is still lacking among most civil and structural engineers with a foundation background. This trend is slowly changing. Since the first edition of this technical manual, helical piles are now listed as a deep foundation system in the 2009 and 2012 editions of the International Building Code. In addition, ICC-ES Acceptance Criteria AC358 for Helical Systems and Devices was published in 2007 and is now on its third revision. Hubbell Power Systems, Inc. was the first manufacturer of helical piles and anchors to obtain evaluation reports from all three model building code agencies – ICBO, BOCA, and SBCCI. Today Hubbell Power Systems, Inc. has evaluation reports for helical products both in the US and Canada. ESR-2794 is an ICC-ES evaluation report that demonstrates Code compliance with the IBC, and CCMC Report 13193-R is an NRC evaluation report that demonstrates Code compliance with the Canadian Building Code. Copies of ICC-ES ESR-2794 and CCMC 13193-R Evaluation Reports are available on www.abchance.com.

- **Instructor's Curriculum for Foundation Engineering Courses**

In 2012, Hubbell Power Systems, Inc. contracted with Dr. Alan Lutenegger to develop an instructor's curriculum on helical piles and anchors to be used for foundation engineering courses for undergraduates. The curriculum includes all the information needed for two lectures, design examples and homework. Also included is a Student Guide, which serves as the "textbook" for students.

APPLICATIONS

In its simplest form, the helical pile/anchor is a deep foundation element, i.e., it transfers a structure's dead and live loads to competent soil strata deep below grade. This is the same for any deep foundation element such as driven piles, drilled shafts, grouted tendons, auger-cast piles, belled piers, etc. Therefore, helical piles/anchors can be used as an alternative method to drilled shafts and driven piles. Practical constraints, primarily related to installation, currently limit the maximum design load per helical pile/anchor to 100 kips in tension and 200 kips in compression, which means helical piles/anchors can resist relatively light to medium loads on a per pile/anchor basis, and much heavier loading when used in pile groups. But as is the case with virtually all engineering problems, more than one solution exists. It is the responsibility of the engineer to evaluate all possible alternatives, and to select the most cost-effective solution.

Today, helical piles/anchors are commonly used for residential and light commercial and heavy commercial construction, machinery/equipment foundations, telecommunication and transmission towers, tie-downs for wind and/or seismic forces, and virtually any application where site access is limited or remote. They have become the deep foundation of choice for walkways and boardwalks in environmentally sensitive areas, such as wetlands and protected forestland. In expansive soil areas, helical piles can save money and time when compared to expensive over-excavation and fill options. Helical piles/anchors do have several advantages (see following section) that make them the foundation of choice for many applications including these general categories:

- Machinery/Equipment Foundations
- Limited Access Sites
- Wind and Seismic Loading
- Replacement for Drilled/Driven Piles

ADVANTAGES of CHANCE® HELICAL PILES/ANCHORS

Each project has unique factors that determine the most acceptable foundation system. The following summarizes situations where helical piles/anchors present sensible solutions.

- **Projects Requiring Deep Foundations due to Weak Surface Soil**

Helical piles/anchors are designed as end-bearing piles which transfer loads to competent, load-bearing strata. Helical piles/anchors eliminate high mobilization costs associated with driven piles, drilled shafts or auger-cast piles. They also don't require spoils to be removed and for flowable

CHANCE® HELICAL PILE/ANCHOR ADVANTAGES TABLE 1-3

Summary of CHANCE® Helical Pile/Anchor Advantages	
• No need for concrete to cure	• Install in any weather
• Quick, easy turnkey installation	• Solution for:
• Immediate loading	- Restricted access sites
• Small installation equipment	- High water table
• Pre-engineered system	- Weak surface soils
• Easily field modified	• Environmentally friendly
• Torque-to-capacity relationship for production control	• No vibration
	• No spoils to remove

sands, soft clays and organic soils, no casings are required, unlike drilled shafts or caissons. When using the CHANCE HELICAL PULLDOWN® Micropiles, you have not only end-bearing capacity, but also the additional capacity from the friction developed along the grout/soil interface.

- **Flooded and/or Poor Surface Conditions**

When surface conditions make spread footings impossible and equipment mobilization difficult, helical piles/anchors are a good alternative since installation requires only a mini-excavator, a rubber-tired backhoe or small tracked machine.

- **Limited Access**

In confined areas with low overhead, helical piles/anchors can be installed with portable equipment. This is particularly useful for rehabilitation work.

- **Expansive Soils**

The depth of expansive soils from the surface varies, but a typical depth is approximately 10 feet. The bearing plates of a helical pile/anchor are usually placed well below this depth. This means that only the small-cross-section shaft of the helical pile/anchor is affected by the expansive soils. The swell force on the shaft is directly proportional to the surface area between the soil and the shaft, and the swell adhesion value. Since helical piles have much smaller shafts than driven piles or auger-cast piles, uplift forces on helical piles are much smaller. Research by R.L. Hargrave and R.E. Thorsten in the Dallas area (1993) demonstrated helical piles' effectiveness in expansive soils.

- **Bad weather installation**

Because helical piles/anchors can be installed in any weather, work does not need to be interrupted.

- **Contaminated soils**

Helical piles/anchors are ideal for contaminated soils because no spoils need to be removed.

- **Temporary structures**

Helical piles/anchors can easily be removed by reversing the installation process. This makes removal of temporary structures simple.

- **Remedial applications**

Helical piles can supplement or replace existing foundations distressed from differential settlement, cracking, heaving, or general foundation failure. Patented products such as the CHANCE® Helical Pier Foundation System provide a complete solution. Hubbell Power Systems, Inc. uses patented products to attach the helical piles to existing foundations and either stabilize the structure against further settlement or lift it back to near original condition. This system is installed only by trained, authorized, and certified dealers/installing contractors.

Helical piles are ideal for remedial work since they can be installed by portable equipment in confined, interior spaces. Additionally, there is no need to worry about heavy equipment near existing foundations. And, unlike driven piles, helical piles are vibration-free. The building can continue to operate with little inconvenience to its occupants. Other deep foundation systems such as auger-cast piles disturb the soil, thereby undermining existing foundations.

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SOIL MECHANICS

REVIEW OF SOIL MECHANICS, SOIL BEHAVIOR, & GEOTECHNICAL SITE INVESTIGATIONS
SECTION 2

CONTENTS

SYMBOLS USED IN THIS SECTION

SOIL MECHANICS

DISCLAIMER

The information in this manual is provided as a guide to assist you with your design and in writing your own specifications.

Installation conditions, including soil and structure conditions, vary widely from location to location and from point to point on a site.

Independent engineering analysis and consulting state and local building codes and authorities should be conducted prior to any installation to ascertain and verify compliance to relevant rules, regulations and requirements.

Hubbell Power Systems, Inc., shall not be responsible for, or liable to you and/or your customers for the adoption, revision, implementation, use or misuse of this information. Hubbell, Inc., takes great pride and has every confidence in its network of installing contractors and dealers.

Hubbell Power Systems, Inc., does NOT warrant the work of its dealers/installing contractors in the installation of CHANCE® Civil Construction foundation support products.

INTRODUCTION

The use of manufactured steel foundation products generally requires a prior geotechnical investigation of the subsurface condition of the foundation soils at the site of a proposed project. In addition to the geotechnical investigation, it is necessary to define the structural load requirements and required Factor of Safety (FS) for use in the overall design approach. CHANCE® Civil Construction manufactures or supplies two main lines of steel foundation products:

- ATLAS RESISTANCE® piers for underpinning and repair of residential and commercial buildings, retaining structures and slabs.
- CHANCE® Helical Piles for new construction and repair of residential and commercial buildings; CHANCE® Helical Tiebacks and a SOIL SCREW® Retention System used in excavation shoring systems, retaining walls and slope stabilization; and CHANCE® Helical Anchors are utilized for communication towers, transmission & distribution power lines, signs, light standards and commercial buildings subject to wind and earthquake load.

SOIL MECHANICS

Terzaghi stated in his book Theoretical Soil Mechanics (1943): ". . . the theories of soil mechanics provide us only with a working hypothesis, because our knowledge of the average physical properties of the subsoil and of the orientation of the boundaries between the individual strata is always incomplete and often utterly inadequate. Nevertheless, from a practical point of view, the working hypothesis furnished by soil mechanics is as useful as the theory of structures in other branches of civil engineering."

Advance planning and careful observation by the engineer during the construction process can help fill the gaps between working hypothesis and fact. The intent of this section of the Design Manual is to provide a basic background or review of soil mechanics so the engineer can develop a useful "working hypothesis" for the design and use of CHANCE® Helical Piles and ATLAS RESISTANCE® Piers.

THE SOIL PROFILE

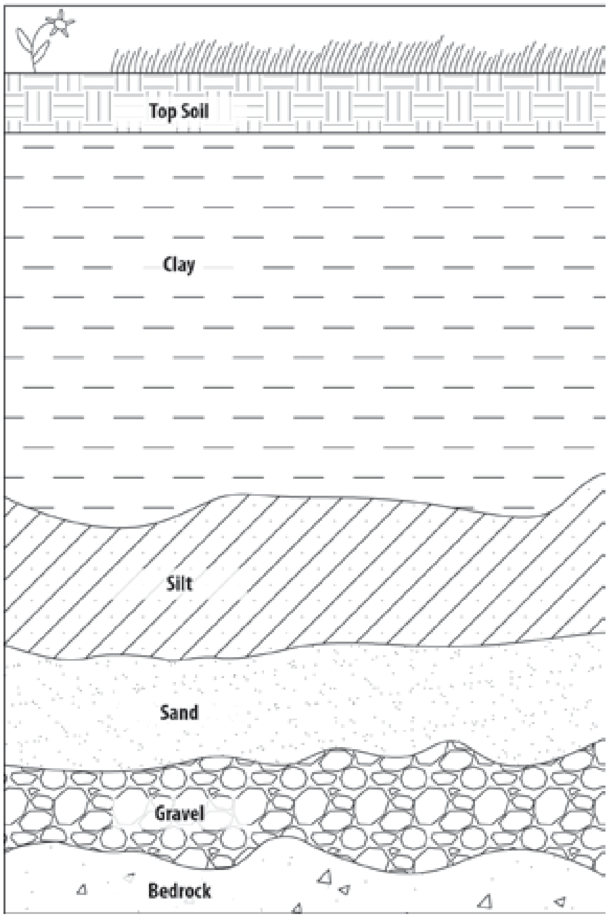
Rock or soil material, derived by geologic processes, are subject to physical and chemical changes brought about by the climate and other factors prevalent at the location of the rock or soil material. Vegetation, rainfall, freeze/thaw cycles, drought, erosion, leaching, and other natural processes result in gradual but profound changes in the character of the soil over the passage of time. These processes bring about the soil profile.

The soil profile is a natural succession of zones or strata below the ground surface. It may extend to various depths, and each stratum may have various thicknesses. The upper layer of the profile is typically rich in organic plant and animal residues mixed with a given mineral-based soil. Soil layers below the topsoil can usually be distinguished by a contrast in color and degree of weathering. The physical properties of each layer usually differ from each other. Topsoil is seldom used for construction. Figure 2-1 shows a typical generalized soil profile.

Deeper layers will have varying suitability depending on their properties and location. It is important to relate engineering properties to individual soil layers in order for the data to be meaningful. If data from several layers of varying strength are averaged, the result can be misleading and meaningless. Equally misleading is the practice of factoring a given soil's engineering properties for design. This can lead to overly conservative foundation design.

DEFINITION of SOIL

Soil is defined as sediments or other accumulation of mineral particles produced by the physical or chemical disintegration of rock, plus the air, water, organic matter, and other substances that may be included. Soil is typically a non-homogeneous, porous, earthen material whose engineering behavior is influenced by changes in composition, moisture content, degree of saturation, density, and stress history.



Generalized Soil Profile
Figure 2-1

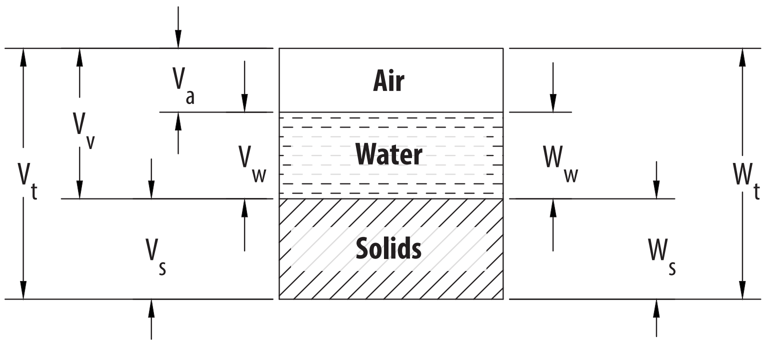
The origin of soil can be broken down to two basic types: residual and transported. Residual soil is produced by the in-place weathering (decomposition) of rock by chemical or physical action. Residual soils may be very thick in areas of intense weathering such as the tropics, or they may be thin or absent in areas of rapid erosion such as steep slopes. Residual soils are usually clayey or silty, and their properties are related to climate and other factors prevalent at the location of the soil. Residual soils are usually preferred to support foundations, as they tend to have better and more predictable engineering properties.

Transported or deposited soils are derived by the movement of soil from one location to another location by natural means. The means are generally wind, water, ice, and gravity. The character of the resulting deposit often reflects the modes of transportation and deposition and the source material. Deposits by water include alluvial floodplains, coastal plains, and beaches. Deposits by wind include sand dunes and loess. Deposits by melting ice include glacial till and outwash. Each of these materials has behavioral characteristics dependent on geological origin, and the geological name, such as loess, conveys much useful information. Transported soils – particularly by wind or water – can be of poor quality in terms of engineering properties.

A soil mass is a porous material containing solid particles interspersed with pores or voids. These voids may be filled with air, water, or both. Figure 2-2 shows a conceptual block diagram of relative volumes of air, water, and soil solids in a given volume of soil. Pertinent volumes are indicated by symbols to the left while weights of these material volumes are indicated by symbols to the right. Figure 2-2 also provides several terms used to define the relative amounts of soil, air, and water in a soil mass. Density is the mass of a unit volume of soil. It is more correctly termed the unit weight. Density may be expressed either as a wet density (including both soil and water) or as a dry density (soil only). Moisture content is the ratio of the weight of water to the weight of soil solids expressed at a percent. Porosity is the ratio of the volume of voids to the total volume of the soil mass regardless of the amount of air or water contained in the voids. Void ratio is the ratio of the volume of voids to the volume of solids.

The porosity and void ratio of a soil depend upon the degree of compaction or consolidation. For a particular soil in different conditions, the porosity and void ratio will vary and can be used to judge relative stability and load-carrying capacity – i.e., stability and load capacity increase as porosity and void ratio decrease. If water fills all the voids in a soil mass, the soil is said to be saturated, i.e., S = 100%.

Permeability or hydraulic conductivity is the property of soil that allows it to transmit water. Its value depends largely on the size and number of the void spaces, which in turn depends on the size, shape, and state of packing of the soil grains. A clay soil can have the same void ratio and unit weight as a sand soil, but the clay will have a lower permeability because of the much smaller pores or flow channels in the soil structure. Water drains slowly from fine-grained soils like clays. As the pore water drains, clays creep, or consolidate slowly over time. Sands have high permeability, thus pore water will drain quickly. As a result, sands will creep, or consolidate quickly when loaded until the water drains. After drainage, the creep reduces significantly.



Mositure Content	W_n	$(W_w$ / $W_s)$ x 100%
Degree of Saturation	S	$(V_w$ / $V_v)$ x 100%
Void Ratio	e	V_v / V_s
Porosity	n	$(V_v$ / $V_t)$ x 100%
Dry Unit Weight (Dry Density)	γ_d	W_s / V_t
Total Unit Weight	γ_t	$(W_s + W_w)$ / V_t
Saturated Unit Weight	γ_s	$(W_s + V_v \gamma_w)$ V_t
Effective (Submerged) Unit Weight	γ^1	$\gamma_s - \gamma_w$

Soil Phases and Index Properties
Figure 2-2

BASIC SOIL TYPES

As stated above, soil is typically a non-homogeneous material. The solid mineral particles in soils vary widely in size, shape, mineralogical composition, and surface-chemical characteristics. This solid portion of the soil mass is often referred to as the soil skeleton, and the pattern of arrangement of the individual particles is called the soil structure.

The sizes of soil particles and the distribution of sizes throughout the soil mass are important factors which influence soil properties and performance. There are two basic soil types that are defined by particle size. The first type is coarse-grained soils. Coarse-grained soils are defined as soil that have 50% or more particles retained by the #200 sieve (0.074 mm). The #200 sieve has 200 openings per inch.

Coarse-grained soils consist of cobbles, gravels, and sands. Coarse-grained soils are sometimes referred to as granular or cohesionless soils. The particles of cohesionless soils typically do not stick together except in the presence of moisture, whose surface tension tends to hold particles together. This is commonly referred to as apparent cohesion.

The second type of soil is fine-grained soil. Fine-Grained soils consist of soils in which 50% or more of the particles are small enough to pass through the #200 sieve. Typical Fine-Grained soils are silts and clays. Silt particles typically range from 0.074 to 0.002 mm. Clay particles are less than 0.002 mm. It is not uncommon for clay particles to be less than 0.001 mm (colloidal size). Fine-grained soils are sometimes referred to as cohesive soils. The particles of cohesive soils tend to stick together due to molecular attraction.

For convenience in expressing the size characteristics of the various soil fractions, a number of particle-size classifications have been proposed by different agencies. Table 2-1 shows the category of various soil particles as proposed by the Unified Soil Classification System (USCS), which has gained wide recognition.

An effective way to present particle size data is to use grain-size distribution curves such as shown in Figure 2-3. Such curves are drawn on a semi-logarithmic scale, with the percentages finer than the grain size shown as the ordinate on the arithmetic scale. The shape of such curves shows at a glance the general grading characteristics of soil. For example, the dark line on Figure 2-3 represents a "Well-Graded" soil – with particles in a wide range. Well-graded soils consist of particles that fall into a broad range of sizes class, i.e., gravel, sand, silt-size, clay-size, and colloidal-size.

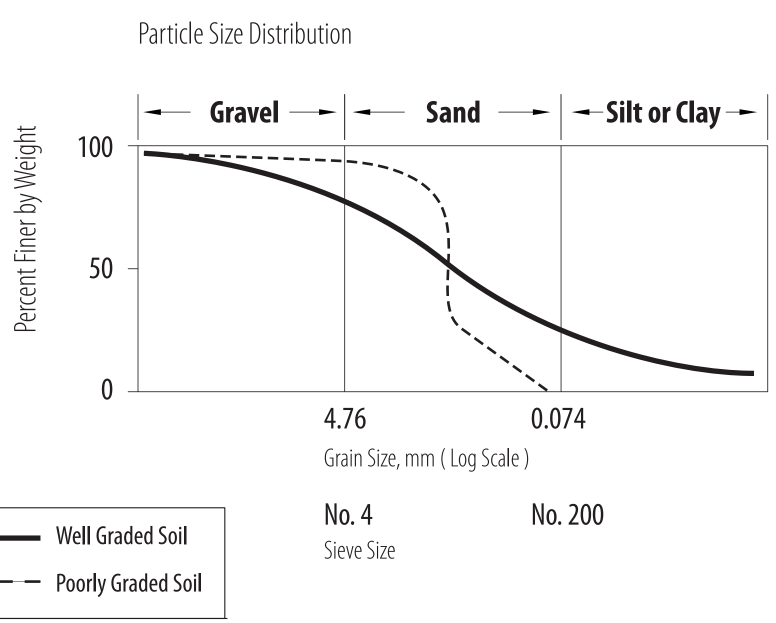
Soil Particle Sizes, Table 2-1

PARTICLE SIZE TERM	FRACTION	SIEVE SIZE	DIAMETER	FAMILIAR REFERENCE
Boulders	---	12" Plus	300 mm Plus	Volleyball
Cobbles	---	3"-12"	75 - 300 mm	Baseball
Gravels	Coarse Fine	0.75"- 3" No. 4 - 0.75"	19 - 75 mm 4.76 - 19 mm	Marbles & Peas
Sand	Coarse Medium Fine	No. 10 - No. 4 No. 40 - No. 10 No. 200 - No. 40	2 - 4.76 mm 0.42 - 2 mm 0.074 - .042 mm	Rock Salt, Table Salt, Sugar
Fines (silts and clays)	---	Passing No. 200	0.074 mm	Flour

SOIL CONSISTENCY STATES and INDEX PROPERTIES

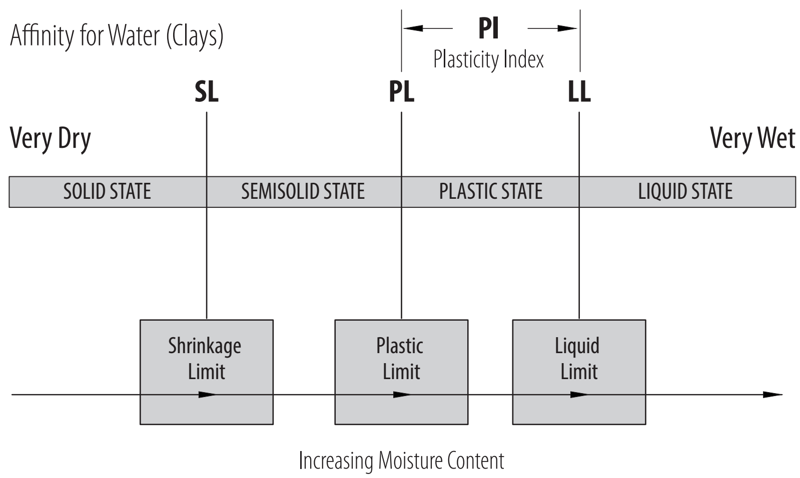
The consistency of fine-grained soils can range from a dry solid condition to a liquid form with successive addition of water and mixing as necessary to expand pore space for acceptance of water. The consistency passes from solid to semi-solid to plastic solid to viscous liquid as shown in Figure 2-4. In 1911, Atterberg, a Swedish soil scientist, defined moisture contents representing limits dividing the various states of consistency. These limits are known as Atterberg Limits. The shrinkage limit (SL) separates solid from semisolid behavior, the plastic limit (PL) separates semisolid from plastic behavior, and the liquid limit (LL) separates plastic from liquid state. Soils with water content above the liquid limit behave as a viscous liquid.

The width of the plastic state (LL-PL), in terms of moisture content, is defined as the plasticity index (PI). The PI is an important indicator of the plastic behavior a soil will exhibit. The Casagrande Plasticity Chart, shown in Figure 2-5, is a good indicator of the differences in plasticity that different fine-grained soils can have. The softness of saturated clay can be expressed numerically by the liquidity index (L.I.) defined as L.I. = (w_n –P.L.)/(L.L.-P.L). Liquidity Index is a very useful parameter to evaluate the state of natural fine-grained soils and only requires measurement of the natural water content, the Liquid Limit and the Plastic Limit. Atterberg limits can be used as an approximate indicator of stress history of a given soil. Values of L.I. greater than or equal to one are indicative of very soft sensitive soils. In other words, the soil structure may be converted into a viscous fluid when disturbed or remolded by pile driving, caisson drilling, or the installation of CHANCE® Helical Piles/Anchors, or ATLAS RESISTANCE® Piers.



Typical Grain Size Distribution Curves
Figure 2-3

If the moisture content (w_n) of saturated clay is approximately the same as the L.L. (L.I. = 1.0), the soil is probably near normally consolidated. This typically results in an empirical torque multiplier for helical piles/anchors (K_t) = 10. If the w_n of saturated clay is greater than the L.L. (L.I. > 1.0), the soil is on the verge of being a



Plasticity and Atterberg Limits
Figure 2-4

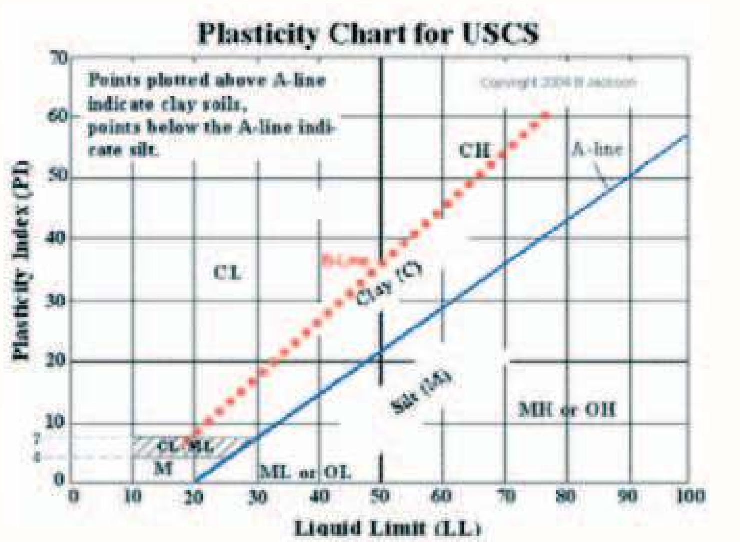


Figure 2-5

viscous liquid and K_t will be less than 10. If the w_n of saturated clay is close to the P.L. (L.I. = 0), the soil is dry and overconsolidated and K_t typically ranges between 12 and 14. If the w_n of a saturated clay is intermediate (between the PL and LL), the soil is probably over consolidated and Kt will be above 10. Many natural fine-grained soils are over consolidated, or have a history of having been loaded to a pressure higher than exists today. Some common causes are desiccation, the removal of overburden through geological erosion, or melting of overriding glacial ice.

Clays lying at shallow depth and above the water table often exhibit overconsolidated behavior known as desiccation. They behave as overconsolidated, but the overburden pressure required has never existed in the soil. Desiccated clays are caused by an equivalent internal tension resulting from moisture evaporation. This is sometimes referred to as negative pore pressure. The problems with desiccated or partly dry expansive clay are predicting the amount of potential expansion and the expansion or swell pressure so that preventive measures can be taken.

Sensitivity of fine grained soils is defined as the ratio of the undrained shear strength of a saturated soil in the undisturbed state to that of the soil in the remolded state $S_t = su_{und}/su_{rem}$. Most clays are sensitive to some degree, but highly sensitive soils cannot be counted on for shear strength after a CHANCE® Helical Pile, ATLAS RESISTANCE® Pier, drilled shaft, driven pile, etc. has passed through it. Some soils are "insensitive", that is, the remolded strength is about the same as the undisturbed strength. Highly sensitive soils include marine deposited in a salt water environment and subsequently subjected to flushing by fresh water. Typical values of soil sensitivity are shown in Table 2-2.

ENGINEERING SOIL CLASSIFICATION

The engineering soil classification commonly used by Geotechnical Engineers is the Unified Soil Classification System (USCS). The Unified System incorporates the textural characteristics of the soil into engineering classification and utilizes results of laboratory grain-size data and Atterberg Limits shown in Table 2-1. The basics of the system are shown in Table 2-4. All soils are classified into 15 groups, each group being designated by two letters. These letters are abbreviations of certain soil characteristics as shown in Table 2-3.

Sensitivity of Soils, Table 2-2

Soil TYPE	Description	Sensitivity
Overconsolidated, Low to Medium Plastic Clays & Silty Clays	Insensitive	1-3
Normally Consolidated, Medium Plastic Clays	Medium Sensitivity	4-8
Marine Clays	Highly Sensitive	10-80

USCS Soil Group Symbol Characteristics, Table 2-3

	1st Symbol		2nd Symbol
G	Gravel	O	Organic
S	Sand	W	Well Graded
M	Non-plastic or Low Plasticity Fines	P	Poorly Graded
C	Plastic Fines	L	Low Liquid Limit
Pt	Peat, Humus, Swamp Soils	H	High Liquid Limit

COARSE-GRAINED SOILS (G & S)

GW and SW groups comprise well-graded gravely and sandy soils that contain less than 5% of non-plastic fines passing the #200 sieve. GP and SP groups comprise poorly graded gravels and sands containing less than 5% of non-plastic fines. GM and SM groups generally include gravels or sands that contain more than 12% of fines having little or no plasticity. GC and SC groups comprise gravelly or sandy soils with more than 12% of fines, which exhibit either low or high plasticity.

FINE-GRAINED SOILS (M & C)

ML and MH groups include the predominately silty materials and micaceous or diatomaceous soils. An arbitrary division between the two groups is where the liquid limit is 50. CL and CH groups comprise clays with low (L.L. < 50)and high (L.L. > 50) liquid limits, respectively. They are primarily inorganic clays. Low plasticity clays are classified as CL and are usually lean clays, sandy clays, or silty clays. Medium-plasticity and high plasticity clays are classified as CH.

ORGANIC SOILS (O & Pt)

OL and OH groups are characterized by the presence of organic matter, including organic silts and clays. The Pt group is highly organic soils that are very compressible and have undesirable construction characteristics. Peat, humus, and swamp soils with a highly organic texture are typical.

Classification of a soil in the United Soil Classification System will require laboratory tests to determine the critical properties, but a tentative field classification is often made by drillers, geologists, or engineers; but considerable skill and experience are required. Soil boring logs often include the engineering classification of soils as described by the USCS.

Specifics of the Unified Soil Classification System (USCS), Table 2-4

Major Divisions			Group Symbols	Typical Descriptions
Coarse Grained Soils- more than 50% retained on #200 sieve.*	Gravels - 50% or more of coarse fraction retained on #4 sieve.	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures. Little or no fines.
			GP	Poorly graded gravels and gravel-sand mixtures. Little or no fines.
		Gravels with Fines.	GM	Silty gravels. Gravel-sand-silt mixtures.
			GC	Clayey gravels. Gravel-sand-clay mixtures.
	Sands - 50% or more of coarse fraction passes #4 sieve.	Clean Sands.	SW	Well-graded sands and gravelly sands. Little or no fines.
			SP	Poorly graded sands and gravelly sands. Little or no fines.
		Sand with Fines	SM	Silty sands. Sand-silt mixtures.
			SC	Clayey sands. Sand-clay mixtures.
Fine-Grained Soils - 50% or more passes #200 sieve.*	Silts and Clays - Liquid limit less than 50.		ML	Inorganic silts, very fine sands, rock flour, silty or clayey find sands.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	Silts and Clays - Liquid limit 50 or more		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity.
Highly Organic Soils.			PT	Peat, muck and other highly organic soils.
*Based on the material passing the 3” (76 mm) sieve.				

EFFECTIVE STRESS and PORE WATER PRESSURE

The total stress within a mass of soil at any point below a water table is equal to the sum of two components, which are known as effective stress and pore water pressure. Effective stress is defined as the total force on a cross section of a soil mass which is transmitted from grain to grain of the soil, divided by the area of the cross section, including both solid particles and void spaces. It sometimes is referred to as inter-granular stress. Pore water pressure is defined as the unit stress carried by the water in the soil pores in a cross section. Effective stress governs soil behavior and can be expressed as:

$$\sigma' = \sigma - u \qquad \textbf{Equation 2-1}$$

where: σ' = the effective stress in the soil
σ = total (or applied) stress
u = pore water pressure

SOIL STRENGTH

One of the most important engineering properties of soil is its shearing strength, or its ability to resist sliding along internal surfaces within a given mass. Shear strength is the property that materially influences the bearing capacity of a foundation soil and the design of CHANCE® Helical Piles/Anchors, or ATLAS RESISTANCE® Piers. The basic principle is similar in many respects to an object that resists sliding when resting on a table.

The shear strength is the maximum shear resistance that the materials are capable of developing. Shear strength of soil consists of two parts. The first part is the friction between particles (physical property). The second part is called cohesion, or no-load shear strength due to a chemical bond between particles.

DRAINED SHEAR STRENGTH

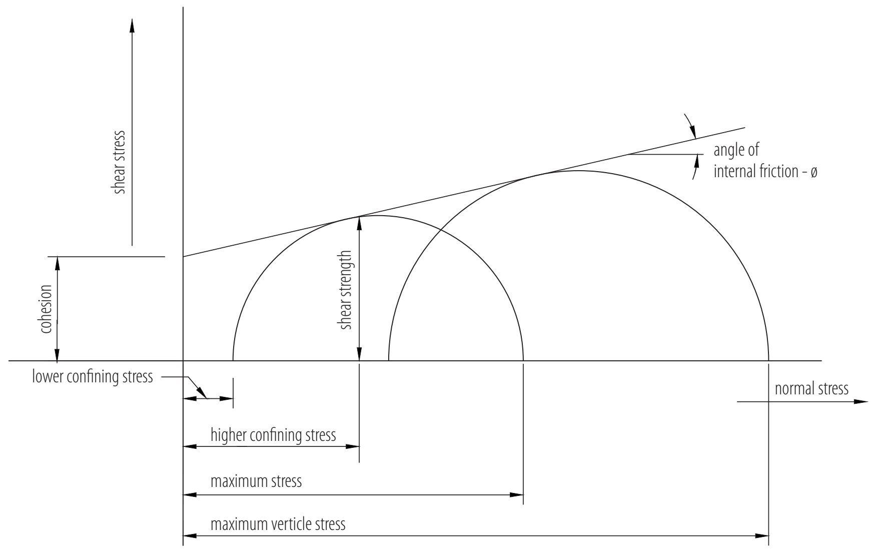
Most unsaturated coarse-grained soils and some mixed grain soils, have sufficiently high permeability that applied loads do not generate pore water pressures or any pore water pressures can dissipate during shear. This is also true if the load is applied very slowly and water is allowed to drain. The shear strength of these soils generally consists of both a "cohesive" component and a "frictional" component so that the shear strength may be reasonably described by the Mohr-Coulomb equation as shown in Equation 2-3.

UNDRAINED SHEAR STRENGTH

Saturated fine-grained soils, such as clays and silty clays subjected to rapid loading have a low enough permeability that excess pore water pressures cannot dissipate during shear. The behavior of these soils is controlled by undrained shear strength. The strength is composed of only a "cohesive" component and not a "frictional" component. The strength of these soils, is sometimes called "cohesion" (c), but a better term is simply undrained shear strength, s_u. The undrained shear strength is controlled by stress history, stress path, loading rate and vertical effective stress.

ANGLE of INTERNAL FRICTION

The shear strength of coarse-grained soils, such as sands, gravels and some silts, is closely analogous to the frictional resistance of solids in contact. The relationship between the normal stress acting on a plane in the soil and its shearing strength can be expressed by the following equation, in terms of stress:



Mohr's Diagram for Moderately Plastic Soil
Portland Cement Association (1996)
Figure 2-6

$\tau = \sigma \tan \varphi$ **Equation 2-2**

where: τ = the shearing stress at failure, or the shear strength

σ = normal stress acting on the failure plane

φ = friction angle

The internal friction of a given soil mass is related to the sliding friction between individual soil grains and the interlocking of soil particles. Shear strength attributable to friction requires a normal force (σ), and the soil material must exhibit friction characteristics, such as multiple contact areas. In dense soils, the individual soil grains can interlock, much like the teeth of two highly irregular gears. For sliding to occur, the individual grains must be lifted over one another against the normal stress (σ). Therefore, the force required to overcome particle interlock is proportional to the normal stress, just the same as sliding friction is proportional to normal stress. In soil mechanics, φ is designated the angle of internal friction, because it represents the sum of sliding friction plus interlocking. The angle of internal friction (φ) is a function of density, roundness or angularity, and particle size.

COHESION

When saturated clay is consolidated, that is, when the volume of voids decreases as a result of water being squeezed out of the pores, the shear strength increases with normal stress. If the shear strength of clays which have a previous history of consolidation (i.e., pre-consolidated) is measured, the relationship between shear strength and normal stress is no longer a line intersecting the ordinate at zero. The clays exhibit a memory, or cohesive shear strength. In other words, the clays remember the pre-consolidation pressure they were previously subjected to. This means considerable shear strength is retained by the soil. Figure 2-6 is an example of the relationship between shear strength and normal stress for a pre-consolidated plastic clay as derived from a triaxial shear test. The intersection of the line at the ordinate is called the cohesion.

Cohesion is analogous to two sheets of flypaper with their sticky sides in contact. Considerable force is required to slide one over the other, even though no normal stress is applied. Cohesion is the molecular bonding or attraction between soil particles. It is a function of clay mineralogy, moisture content, particle orientation (soil structure), and density. Cohesion is associated with fine grain materials such as clays and some silts.

COULOMB EQUATION for SHEAR STRENGTH

The equation for shear strength as a linear function of total stress is called the Coulomb equation because it was first proposed by Coulomb in 1773.

$\tau_f \;= c + \sigma \tan \varphi$ **Equation 2-3**

In terms of effective stress:

$\tau_f \;= c' + (\sigma - u) \tan \varphi'$ **Equation 2- 4**

where: τ_f = shear strength at failure
c' = cohesion
σ = total stress acting on the failure plane
φ' = friction angle
u = pore water pressure

Equations 2-3 and 2-4 are two of the most widely used equations in geotechnical engineering, since they approximately describe the shear strength of any soil under drained conditions. They are the basis for bearing capacity Equations 5-6 and 5-31 presented in Section 5.

SITE INVESTIGATIONS

To this point, various definitions, identification properties, limit states, engineering classifications, and soil strength properties have been discussed. This section details some of the more common soil exploration methods used to determine these various soil parameters.

The primary purpose of a geotechnical site investigation is to identify the subsurface stratification, and the key soil properties for design of the steel foundation elements. Such studies are useful for the following reasons:

ATLAS RESISTANCE® Piers:

- To locate the depth of a suitable bearing stratum for end bearing support of the underpinning pier.

- To establish the location of any weak or potentially liquefiable soil zones in which column stability of the pier shaft must be considered.

- To determine if there are any barriers to installing the pier to the required depth such as rubble fill, boulders, zones of chert or other similar rock, voids or cavities within the soil mass, any of which might require pre-drilling.

- To do a preliminary evaluation of the corrosion potential of the foundation soils as related to the performance life of the steel pier.

CHANCE® Helical Piles/Anchors, Tiebacks and SOIL SCREW® Anchors:

- To locate the depth and thickness of the soil stratum suitable for seating the helical plates of the pile and to determine the necessary soil strength parameters of that stratum.

- To establish the location of weak zones, such as peat type soils, or potentially liquefiable soils in which column stability of the pile for compression loading situations may require investigation.

- To locate the depth of the groundwater table (GWT).

- To determine if there are any barriers to installing the piles to the required depth such as fill, boulders or zones of cemented soils, or other conditions, which might require pre-drilling.

- To do a preliminary evaluation of the corrosion potential of the foundation soils as related to the performance life of the steel pile.

The extent to which a soil exploration program should reach depends on the magnitude of the project. If the proposed construction program involves only a small expenditure, the designer cannot afford to include more in the investigation than a small number of exploratory borings, test pits or helical trial probe piles and a few classification tests on representative soil samples. The lack of information about subsoil conditions must be compensated for by using a liberal factor of safety. However, if a large-scale construction operation is to be carried out under similar soil conditions, the cost of a thorough and elaborate subsoil investigation is usually small compared to the savings that can be realized by utilizing the results in design and construction, or compared to the expense that would arise from a failure due to erroneous design assumptions. The designer must be familiar with the tools and processes available for exploring the soil, and with the methods for analyzing the results of laboratory and field tests.

A geotechnical site investigation generally consists of four phases: (1) Reconnaissance and Planning, (2) Test Boring and Sampling Program, (3) Laboratory Testing, and (4) a Geotechnical Report. A brief description of the requirements and procedures, along with the required soil parameters used in designing manufactured steel foundation products, is given in the following sections.

INITIAL RECONNAISSANCE and PLANNING

The first step in any subsoil exploration program should be an investigation of the general geological character of the site. The more clearly the site geology is understood, the more efficiently the soil exploration can be performed.

Reconnaissance and Planning includes: (1) review of the proposed project and structural load requirements and size of the structure and whether the project is new construction or structure repair, (2) a review of the general soil and geologic conditions in the proximity of the site, and (3) a site visit to observe topography and drainage conditions, rock outcrops if present, placement of borings, evidence of soil fill, including rubble and debris and evidence of landslide conditions. The planning portion includes making a preliminary determination of the number and depth of each boring as well as determining the frequency of soil sampling for laboratory testing and requesting the marking of all utilities in the zone in which borings will be conducted. Indicated below are recommended guidelines for determining the number of borings and the depth to which the boring should be taken based on the project type.

Minimum Number of Test Boring(s)

Whether the project involves underpinning/repair of an existing structure or new construction, borings should be made at each site where helical piles or resistance piers are to be installed. The recommended minimum number of borings necessary to establish a foundation soil profile is given below:

- Residential Home - One (1) boring for every 100 to 150 lineal feet of foundation.

- Commercial Building - One (1) boring for every 50 to 100 lineal feet for multistory-story structures, and every 100 to 150 lineal feet of foundation for other commercial buildings, warehouses and manufacturing buildings.

- Communication Towers - One (1) boring for each location of a cluster of piles or anchors, and one (1) boring at the tower center foundation footing.

- Sheet Pile/Earth Stabilization for Earth Cuts - One (1) boring for every 200 to 400 feet of project length.

- If the project is small or when the project has a restricted budget, helical trial probe piles installed at the site can provide information regarding the depth to the bearing strata and pile capacity.

- Or, boring number can be based on the overall project area, or based on minimum requirements per applicable building codes.

Depth of Test Boring(s)

The depth of each boring will vary depending on the project type, magnitude of foundation loads and area extent of the project structure. Some general guidelines for use in estimating required boring depths are given below:

- Residential Home - At least 15 feet deep with final 5 feet into good bearing stratum, generally "N" > 8 to 10 (See next section "Test Boring and Sampling Program" for a description of Standard Penetration Test and "N" values.)

- Commercial Building - For a single story structure at least 20 feet deep with final 5 to 10 feet into good bearing stratum (generally "N" > 15); add 5 foot depth for each additional story.

Auger Drilling Operation
Figure 2-7

- Communication Towers - Minimum of 35 feet for towers over 100 feet tall and at least 20 feet into a suitable bearing stratum (typically medium dense to dense for sands and stiff to very stiff for clays) for helical anchors/piles. The suitable bearing stratum should have a minimum “N” value of 12 for sands and a minimum of 10 for cohesive soils.

- Sheet Piling/Earth Stabilization - Boring should be taken to a depth that is at least as deep as the structure (sheet pile, retaining wall, etc.) to be anchored or until a suitable stratum is reached for seating the helical plates of the tiebacks (generally medium or denser sand or stiff clays).

- Active Seismic Areas - Depth per local codes.

TEST BORING and SAMPLING PROGRAM

In some cases, especially for small projects and shallow conditions, test borings may be conducted using hand augers or other portable equipment. In most cases, however, the site investigation will typically require drilling using a truck mounted drill rig.

The second step of the site investigation is to make exploratory boreholes or test pits that furnish more specific information regarding the general character and thickness of the individual soil strata. This step and an investigation of the general geological character of the site are recommended minimums. Other steps depend on the size of the project and the character of the soil profile.

Method of Boring and Frequency of Sampling

Drilling is typically the most economical and most expedient procedure for making borings although test pits can be an alternative for some projects. Three common types of borings obtained using truck or track mounted drill rigs are 1) wash borings (mud rotary), and 2) solid-stem continuous flight (CFA) auger drilling and 3) hollow stem flight auger (HSA) drilling. Any one of the three can be used, but CFA auger drilling is the most common – particularly for shallow borings. Wash borings or mud rotary drilling use casings to hold the borehole open and a drilling fluid to bring solid cuttings to the surface. The casing is either driven with a hammer or rotated mechanically while the hole is being advanced. The cutting bit and drill rods are inserted inside the casing and are rotated manually or mechanically. The cuttings allow the driller to visually classify the soil as to its type and condition and record the data on a log sheet at the depth of the cutting bit. Wash borings typically use water or drilling mud such as bentonite slurry depending on the soil. In some soil profiles, drilling mud prevents caving, making full-length casing unnecessary. While drilling proceeds, the driller observes the color and appearance of the mixture of soil and water/mud. This enables the driller to establish the vertical sequence of the soil profile. At 5 ft (1.5 m) intervals, or when a change in strata is noticed, the cutting bit is removed and a spoon sample is taken.

Auger drilling typically uses a continuous solid-stem flight auger rotated mechanically while the hole is being advanced. The continuous flight auger (CFA) often includes a hollow stem, which acts as a casing to hold the borehole open. Water or drilling mud is typically not used. Cuttings are carried to the surface by the auger flights, which allow visual

Figure 2-8

classification of the soil. The advantage of the hollow stem auger is to permit the sampler and rod to be inserted down through the auger without removing the auger sections each time a sampler is inserted. The auger acts as a temporary casing. Samplers are inserted inside the auger casing to retrieve disturbed and undisturbed soil samples typically at 5 ft (1.5 m) intervals. Figure 2-7 demonstrates an auger drilling operation. Solid-stem augers are designated by the outside diameter of the auger flights. Common sizes are 3 inch, 4 inch, and 6 inch. Hollow-stem augers are designated by the inside diameter of the pipe. 3-1/4 inch and 4-1/4 inch are common sizes.

Solid-stem continuous flight augers consist of a solid steel central shaft with a continuous auger, typically available in 5 foot sections. The borehole is advanced by rotating the auger, which brings soil cuttings to the ground surface. Disturbed samples of soil may be taken from the augers, but in order to obtain undisturbed samples, the augers must be removed and a sampling tool placed in the bottom of the borehole. Continuous Flight Augers work well in stiff to very stiff fine-grained soils that maintain an open borehole, but do not work in very soft clays or sands and loose silts below the water table. These conditions require either wash boring or the use of Hollow Stem Augers (HSA).

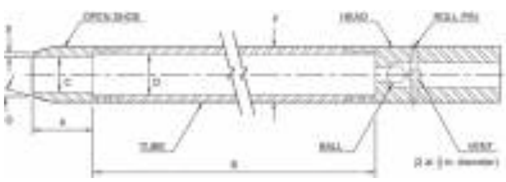
The groundwater table (GWT), or phreatic surface is defined as the elevation at which the pressure in the water is equal to that of the atmosphere. Information regarding the location of the groundwater table is very important to the design and construction of deep foundations – especially in granular soils. Careful observations should always be made and recorded, if circumstances permit, during exploratory drilling. It is customary to note the water level on completion of the hole and after allowing the hole to stand overnight or for 24 hours before backfilling. The use of drilling mud to stabilize the walls of the hole may preclude obtaining this information.

Soil Sampling

Geotechnical Site Investigations almost always include the collection of soil samples for identification and description, laboratory testing for soil classification and laboratory testing for soil strength and stiffness. There are two broad types of soil samples that are often collected; 1) disturbed samples, and 2) undisturbed samples. In general, disturbed samples may either be obtained from augers as previously discussed or more commonly they are obtained using the Standard Penetration Test (SPT). Undisturbed samples are typically obtained with thin-walled push tubes called Shelby Tubes (ST).

Standard Penetration Test and Sampling

The cuttings from exploratory drill holes are inadequate to furnish a satisfactory conception of the engineering characteristics of the soils encountered, or even the thickness and depths of the various strata. To obtain soil samples from test borings, a sampling spoon is attached to the drill rod and lowered to the bottom of the hole. The spoon is driven into the soil to obtain a sample and is then removed from the hole. The spoon is opened up and the recovery (soil sample length inside the spoon) is recorded. The soil is extracted from the spoon and inspected and described by the driller. A portion of the sample is placed in a glass jar and sealed for later visual inspection and laboratory determination of index properties.



A = 1.0 to 2.0 in (25 to 50 mm)
B = 18.0 to 30.0 in (0.457 to 0.762 m)
C = 1.375 ± 0.005 in (34.93 ± 0.13 mm)
D = 1.50 + 0.05 - 0.00 in (38.1 + 1.3 = 0.0 mm)
E = 0.10 ± 0.02 in (2.54 ± 0.25 mm)
F = 2.00 + 0.05 - 0.00 in (50.8 + 1.3 - 0.0 mm)
G = 16.0° to 23.0°

Geometry of Standard Penetration Test
Split-Barrel Sampler (ASTM D 1586)
Figure 2-9

Split Spoon Sample

The 1½ in (38 mm) inside diameter split barrel may be used with a 16-gauge wall thickness split liner. The penetrating end of the drive shoe may be slightly rounded. Metal or plastic retainers may be used to retain soil samples.

The most common method of obtaining some information concerning relative density or the stiffness of in-situ soil consists of counting the number of blows of a drop weight required to drive the sampling spoon a specified distance into the ground. This dynamic sounding procedure is called the standard penetration test (SPT). The essential features include a drop hammer weighing 140 lb (63.5 kg) falling through a height of 30” (0.76 m) onto an anvil at the top of the drill rods, and a split spoon (SS) sampler having an external diameter of 2” (50.8 mm) and a length of 30” (0.76 m). The spoon is attached to the drill rods and lowered to the bottom of the drill hole. After the spoon reaches the bottom, the number of blows of the hammer is counted to achieve three successive penetrations of 6” (0.15 m). The number of blows for the first 6” is disregarded because of the disturbance that exists at the bottom of the drill hole. The number of blows for the second and third 6” increments are added and designated the standard penetration test (SPT), “N” value, or blow count. The data obtained from SPT tests are commonly recorded on soil boring logs relative to the sounding depth where the sample was taken. SPT values are widely used to correlate the shearing strength of soil for the design of shallow and deep foundations – including CHANCE® Helical Piles and ATLAS RESISTANCE® Piers. The SPT values also can assist in determining the depth of installation requirements for ATLAS RESISTANCE® Piers. Values of soil friction angle “φ” and cohesion “c” can be selected through correlation with the SPT “N” values. Details of the equipment and standardized procedures are

CPT/CPTU
Figure 2-10

specified in ASTM D 1586. Figure 2-8 illustrates a drill crew conducting a Standard Penetration Test. The split spoon sampler is shown in Figure 2-9.

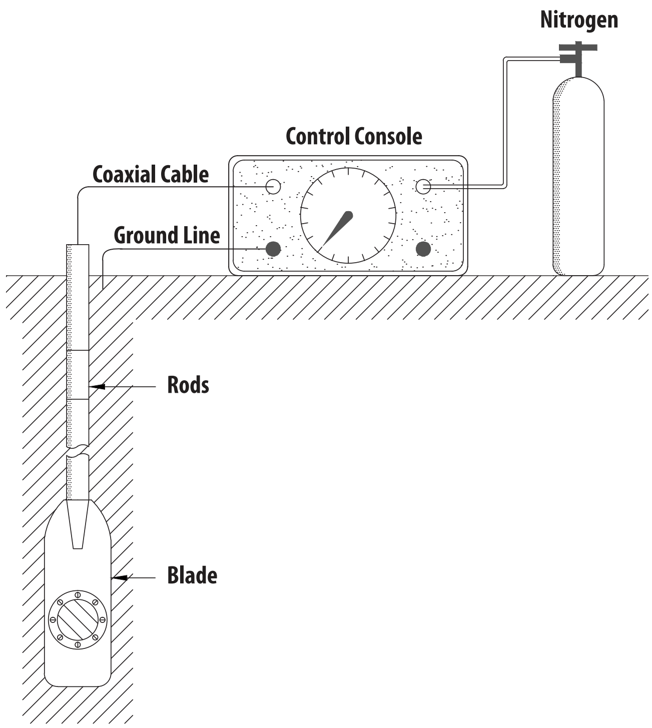


Figure 2-12

Undisturbed Samples

In general, soil samples taken from split spoon samplers are always considered disturbed to some degree for two reasons: 1) the sampler is driven into the soil, and 2) the split spoon is very thick. For soil samples to be used for laboratory analysis, the degree of disturbance of the samples must be reduced to a minimum. Reasonably satisfactory samples can be obtained in 50 and 76 mm samplers made of steel tubing about 1.5 mm thick. The lower ends are beveled to a cutting edge to give a slight inside clearance. This type of sampler is commonly referred to as a “Shelby tube”. The Shelby tube is attached to the end of the drill rod and pushed vertically down into the soil to obtain an undisturbed sample. Hand samples or grab samples are sometimes taken from cuttings or test pits and are useful for soil classification and determining index properties. Details of the equipment and proper procedures for obtaining thin-walled Shelby Tube samples are specified in ASTM D1587.

IN-SITU TESTING METHODS

Cone Penetration Test (CPT) / Piezocone (CPTU)

The Cone Penetration Test consists of a cylindrical probe with a cone tip having an apex angle of 60° that is pushed slowly into the ground. The standard size cone has a diameter of 1.405 inch, which gives a projected end area of 10 cm^2. Most cones also have a short section behind the tip that is called the sleeve. The force on the tip and the sleeve are measured independently during penetration to give the cone tip resistance, q_c, and the sleeve resistance, f_s. These values may then be used to evaluate changes in soil layering at a site and to estimate individual soil properties, such as shear strength and stress history. Some cones are also equipped with a porewater pressure sensor to measure the excess porewater pressure as the cone advances. This is called a piezocone. The cone tip resistance obtained from a piezocone is defined as q_t, the “effective” or corrected cone tip resistance since it is corrected for porewater pressure. A figure of a CPT and CPTU are shown in Figure 2-10.


Figure 2-13


Figure 2-11

Cone penetrometers cannot penetrate more than a few meters in dense sand, but they have been used to depths up to 60 m or more in soft soils. The friction ratio, defined as the friction resistance divided as the tip resistance can be correlated with the type of soil encountered by the penetrometer. Since no samples are obtained by use of cone penetrometers, borings and sampling are usually needed for definitive information about the type of soil being investigated.

Dilatometer Test (DMT)

The Dilatometer Test consists of a flat stainless steel blade with a circular, flexible membrane mounted on one side of the blade, as shown on Figure 2-11. The blade is pushed into the ground, much like a CPT or CPTU, but instead of providing continuous data, pushing is stopped every 1 foot. Immediately after pushing is stopped, the flexible membrane is expanded into the soil using nitrogen gas and a control console at the ground surface. Two pressure readings are taken; 1) the A-Reading, which is the pressure required to just initiate movement of the membrane into the soil, and 2) the B-Reading, which is the pressure required to expand the center of the membrane 1 mm into the soil. The two Readings are corrected for the stiffness of the membrane to give two pressure readings, P_0 and P_1. P_0 and P_1 are then used along with the soil effective stress at each test depth to obtain estimates of specific soil properties such as shear strength, modulus, stress history and in-situ lateral stress. The specific requirements of the test are given in ASTM D6635.

Field Vane Test (FVT)

The Field Vane Test (FVT) or Vane Shear Test (VST) is used to measure the undrained shear strength and Sensitivity of medium stiff to very soft saturated fine-grained soils. It is considered one of the most reliable and direct in-situ test methods for determining undrained shear strength and the only in-situ test that may be used to determine Sensitivity. The test consists of inserting a thin four-bladed vane into the soil and rotating slowly to create a shear failure in the soil. The vane is usually rectangular with a height to diameter ratio (H/D) of 2, as shown in Figure 2-13. Initially, the maximum torque is measured to obtain the peak or undisturbed undrained shear strength. Then, the vane is rotated 10 times and the test is repeated to obtain the remolded undrained shear strength. The ratio of undisturbed to remolded strength is defined as Sensitivity, as previously described. The specific requirements of the test are given in ASTM D2573.

Mechanical Properties of Various Rocks, Table 2-5

ROCK	YOUNG'S MODULUS AT ZERO LOAD (10^5 kg/cm^2)	BULK DENSITY (g/cm^3)	POROSITY (percent)	COMPRESSIVE STRENGTH (kg/cm^2)	TENSILE STRENGTH (kg/cm^2)
Granite	2 - 6	2.6 - 2.7	0.5 - 1.5	1,000 - 2,500	70 - 250
Microgranite	3 - 8				
Syenite	6 - 8				
Diorite	7 - 10			1,800 - 3,000	150 - 300
Dolerite	8 - 11	3.0 - 3.05	0.1 - 0.5	2,000 - 3,500	150 - 350
Gabbro	7 - 11	3.0 - 3.1	0.1 - 0.2	1,000 - 3,000	150 - 300
Basalt	6 - 10	2.8 - 2.9	0.1 - 1.0	1,500 - 3,000	100 - 300
Sandstone	0.5 - 8	2.0 - 2.6	5 - 25	200 - 1,700	40 - 250
Shale	1 - 3.5	2.0 - 2.4	10 - 30	100 - 1,000	20 - 100
Mudstone	2 - 5				
Limestone	1 - 8	2.2 - 2.6	5 - 20	300 - 3,500	50 - 250
Dolomite	4 - 8.4	2.5 - 2.6	1 - 5	800 - 2,500	150 - 250
Coal	1 - 2			50 - 500	20 - 50
Quartzite		2.65	0.1 - .05	1,500 - 3,000	100 - 300
Gneiss		2.9 - 3.0	0.5 - 1.5	500 - 2,000	50 - 200
Marble		2.6 - 2.7	0.5 - 2	1,000 - 2,500	70 - 200
Slate		2.6 - 2.7	0.1 - 0.5	1,000 - 2,000	70 - 200

Notes:
1) For the igneous rocks listed above, Poisson's ratio is approximately 0.25
2) For a certain rock type, the strength normally increases with an increase in density and increase in Young's Modulus (after Farmer, 1968)
3) Taken from *Foundation Engineering Handbook* , Winterkom and Fong, Van Nostrand Reinhold, page 72

The maximum torque (T) is measured during rotation and for a vane with H/D = 2 the undrained shear strength is determined from:

$s_u = (0.273T)/D^3$ **Equation 2- 5**

Vanes are available is different sizes to suit the soil at a particular site. The Field Vane Test may be especially useful in evaluating sites for helical piles/anchors as it may give some insight to the engineer into the degree of disturbance and strength reduction that the soil may experience during installation, depending on the Sensitivity. It is important that the exact geometry of the vane (e.g., H, D, thickness of blades) and test procedures used be described in a Geotechnical Report so that the engineer may make any adjustments to the test results for the equipment used.

Helical Probe

Shear strength also can be estimated by installing a helical pile "probe" and logging installation torque vs. depth. The torque values can be used to infer shear strength based on the torque-to-capacity relationship discussed in Section 6.

Rock Coring and Quality of Rock Measurement

When bedrock is encountered, and rock anchors are a design consideration, a continuous rock core must be recovered to the depth or length specified. Typical rock anchors may be seated 20 ft. or 30 ft. into the rock formation.

In addition to conducting compressive tests on the recovered rock core samples (See Table 2-5), the rock core is examined and measured to determine the rock competency (soundness or quality). The rock quality designation (RQD) is the most commonly used measure of rock quality and is defined as:

$$\text{RQD} = \frac{\Sigma \text{ Length of intact pieces of core } (>100 \text{ mm})}{\text{Length of core run}}$$

The values of RQD range between 0 and 1.0 where an RQD of 0.90 or higher is considered excellent quality rock.

Helical piles/anchors rotated or torqued into the ground cannot be installed into hard, competent bedrock. However, in upper bedrock surfaces comprised of weathered bedrock material such as weathered shale or sandstone, the helix plates can often be advanced if the RQD is 0.30 or less.

The presence of an intact bedrock surface represents the ideal ground condition for ATLAS RESISTANCE® Piers. In this ground condition, the ATLAS RESISTANCE® Pier is installed to the rigid bearing surface represented by the bedrock layer.

Laboratory Testing of Recovered Soil Samples

Laboratory testing is typically part of a subsurface investigation and may vary in scope depending upon project requirements or variability in soil conditions. Some of the more typical laboratory tests are described below:

Classification / Characterization Tests

- **Visual Classification** – Samples collected during the drilling operations should be visually classified. Every recovered sample from the field boring and sampling program is inspected visually and given a visual description as to its collection depth, percent recovery, moisture conditions, soil color, inclusion type and quantity, approximate strength, odor and composition (See Table 2-4). In addition to this visual classification, a representative number of samples are selected to conduct the following tests:

- **Water Content** – measures the amount of moisture in the soil. Moisture or water content is measured by weighing a soil sample taken from the field on a laboratory scale. The soil sample is then placed in a standard oven for a sufficient time to allow all the moisture to evaporate. After being removed from the oven, the soil sample is weighed again. The dried weight is subtracted from the original weight to determine the water weight of the sample. These methods are also used to determine the total (wet) unit weight and the dry unit weight.

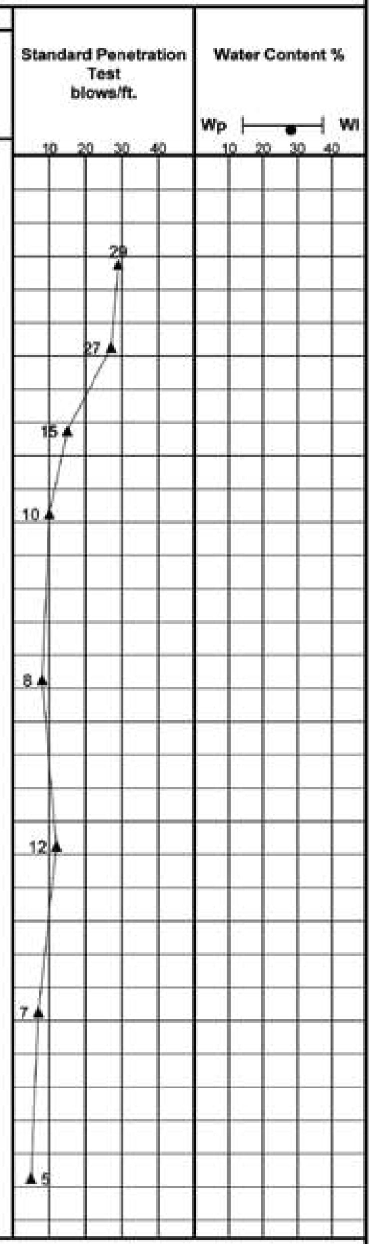
Sample Boring Log in Coarse-Grained Soil, Table 2-6

Project No.: 12-1122
Project: Doe Run Test Borings - 2012
Client: Hubbel Power Systems
Boring No.: 1

Boring Log

Rig: CME 75 with 140 lb Auto Hammer
Location: Leadwood, MO
Driller: MAS

Depth (ft.)	Description (Subsurface Profile)	Qp, T.S.F.	Dry Density, P.C.F.	Depth/Elev.	Sample Number	Type	Blows/ft.	Qu, T.S.F.
0	Ground Surface			100.0				
	Crushed Stone: Poorly Graded Sand with Silt (SP-SM), Light Gray, Trace Gravel, Fine to Coarse, Dry				0	HA		
	(SP-SM), Trace Gravel, Fine to Coarse, Medium Dense, Dry Blow Sequence = 6-13-16 Recovery = 14"				1	SS	29	
5	(SP-SM), Trace Gravel, Fine to Coarse, Medium Dense, Dry Blow Sequence = 10-13-14 Recovery = 18"				2	SS	27	
	(SP-SM), Fine to Medium, Medium Dense, Moist Blow Sequence = 8-8-7 Recovery = 16"				3	SS	15	
10	(SP-SM), Fine to Medium, Medium Dense, Moist Blow Sequence = 3-5-5 Recovery = 19"				4	SS	10	
15	(SP-SM), Fine to Medium, Loose, Moist Blow Sequence = 2-4-4 Recovery = 17"				5	SS	8	
20	(SP-SM), Fine to Medium, Medium Dense, Moist Blow Sequence = 3-6-6 Recovery = 18"				6	SS	12	
25	(SP-SM), Fine to Medium, Loose, Moist Blow Sequence = 2-3-4 Recovery = 15"				7	SS	7	
30	(SP-SM), Fine to Medium, Loose, Moist/Wet Blow Sequence = 1-2-3 Recovery = 15"			68.5	8	SS	5	
	End of Boring @ 31½ Ft.			31.5				



Drill Method: 3 1/4" HSA with AW Rod
Boring Started: 9-10-2012
Boring Completed: 9-10-2012
Tested By: N/A
Logging By: PEB

GEOTECHNICS
Soil & Material Testing

Groundwater Elev. During Drilling: 69.0
Groundwater Elev. @ Comp.:
Groundwater Elev. @ 1 **Hrs.:**
Boring Location: West Boring
Sheet 1 of 1

- **Particle Size Analysis** – measures the distribution of particle sizes within the soil sample.

- **Atterberg Limits** – Liquid Limit (LL), Plastic Limit (PL), Shrinkage Limit (SL), and Plastic Index (PI) – applies to cohesive types of soil and is a measure of the relative stiffness of the soil and potential for expansion. Index properties (LL, PL, SL, and PI) are determined using specially developed apparatus and procedures for performing these tests. The equipment, specifications and procedures are closely followed in ASTM D 4318 Classification / Characterization Tests. The Liquid Limit and the Plastic Limit are particularly important since they may be used along with the natural water content to determine the Liquidity Index.

STRENGTH CHARACTERISTICS

In some instances undisturbed soil samples are recovered in the field using a thin wall Shelby tube. These recovered samples are tested either in triaxial or direct shear tests to determine directly the friction angle "φ" and the cohesion "c" of the soil. For cohesive (clay) soil samples, an unconfined compression test "UC" is often conducted. The unconfined compression test is used to determine the unconfined compression strength "q_u" of the clay soil. The cohesion of the clay sample is then taken to be one-half of "qu". The unconfined compression test is commonly performed due to its low cost; however the results tend to be conservative and simulate only total stress conditions with no confining pressure which may not be appropriate for the project. For granular soils, the Direct Shear test is a relatively inexpensive test to determine the soil friction angle and may also be used for undrained testing of cohesive samples. More refined laboratory testing may be appropriate for large projects and may offer a cost saving potential by justifying higher soil strength than using less sophisticated test methods. Some of the more complex strength tests include, Consolidated Drained (CD), Consolidated Undrained (CU) and Unconsolidated Undrained (UU) Triaxial tests for total and effective stress paths at project specific confining stresses.

THE GEOTECHNICAL REPORT

The geotechnical report provides a summary of the findings of the subsurface investigation, and the results of the laboratory testing. Geotechnical reports usually include an introduction detailing the scope of work performed, site history including geology, subsurface conditions, soil profile, groundwater location, potential design constraints such as seismic parameters and corrosion potential, foundation options, allowable load capacities, and an appendix which includes soil boring logs. Soil boring logs provide a wealth of information that is useful in the design of CHANCE® Helical Piles and ATLAS RESISTANCE® Piers. Boring logs come in variety of designs since there is no standard form, but they contain basically the same type of information – most of which has been discussed in this section. Items to expect on a soil boring are: total boring depth, soil profile, description of soil samples, sample number and type, Standard Penetration Test N-values, moisture content, Atterberg limits, unconfined compression strength or undrained shear strength (cohesion), groundwater table location, type of drilling used, type of SPT hammer used, and sample recovery. An example boring log is shown in Table 2-6 & 2-7. Table 2-6 is a soil boring taken in a coarse-grained sand soil. Table 2-7 is a soil boring taken in a fine-grained clay soil.

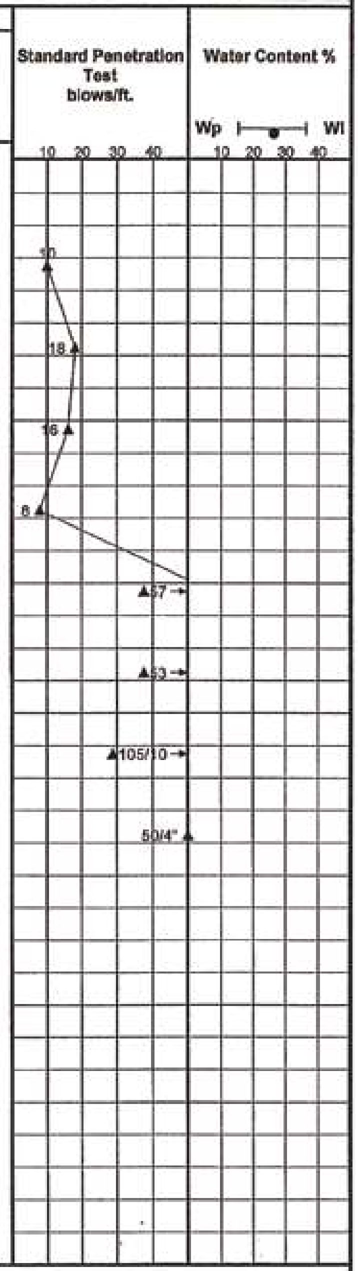
Sample Boring Log in Fine-grained Soil, Table 2-7

Project No.: 09-1219
Project: Mexico and Eaton Dam Drill Sites
Client: Chance Civil Construction
Boring No.: Mexico

Boring Log

Rig: CME 55
Location: Mexico/Park Hills, Missouri
Driller: MAS

SUBSURFACE PROFILE						SAMPLE			
Depth (ft.)	Symbol	Description	Qp, t.s.f.	Dry Density, P.C.F.	Depth/Elev.	Number	Type	Blows/ft.	Qu, T.S.F.
0		Ground Surface			0.0				
		Fill: Yellow Brown Silty Clay, w/Sand, (CL)			-2.5 2.5	0	HA		
		Fill: Yellow Brown Micaceous Clay, Trace Sand, Stiff, (CH)	3.50			1	SS	10	
5		Fill: Mottled Reddish Brown, Trace Sand and Gravel, Very Stiff, (CH)	3.50		-7.5 7.5	2	SS	18	
		Fill: Light Gray mottled Yellow Brown Clay, w/Sand and Gravel, Pieces of Coal and Shale, Very Stiff, (CH)	4.00			3	SS	16	
10		Fill: W/Pieces of Limestone, Stiff, (CH)			-12.5 12.5	4	SS	8	
		Light Gray Weathered Micaceous Clay (Shaly Residuum), Hard, (CH)	4.5+			5	SS	57	
15		Hard, (CH)				6	SS	53	
		Hard, (CH)				7	SS	105/10	
20		Hard, (CH)			-21.5 21.5	8	SS	50/4"	
		End of Boring @ 21½ Ft.							
25									
30									



Drill Method: 3 1/4" HSA
Boring Started: 11-23-2009
Boring Completed: 11-23-2009
Tested By:
Logging By: PEB/BJS

GEOTECHNICS
Soil & Material Testing

Groundwater Elev. During Drilling:
Groundwater Elev. @ Comp.:
Groundwater Elev. @ Hrs.:
Boring Location: Client Provided

Sheet 1 of 1

Problem Soil Conditions

All natural materials, such as soil, will exhibit conditions of variability that may make a single solution inadequate for inevitable problems that arise. It is wise to remember Dr. Terzaghi's emphasis to have a secondary solution ready when dealing with the variability of soils.

Deep Fill, Organic and Collapsible Soils

The existence of deep fills, organic and collapsible soils on a given project site are typically known before the start of the project. This is usually determined during the subsurface investigation by means of drilling or sounding. However, on large projects like an underground pipeline or transmission line that covers many miles, these soils may occur in undetected pockets and hence present a potential problem. The best solution is to be aware of the possibility of their existence and be prepared to install CHANCE® Helical Piles and ATLAS RESISTANCE® Piers deeper to penetrate through this material into better bearing soil. It is not recommended to locate the helical bearing plates or the tip of the ATLAS RESISTANCE® Pier in these soils.

Loose Liquefiable Soils

Some deposits of saturated sand and silty sand are naturally loose and may be prone to lose strength or liquefy during an earthquake or other dynamic loading. These soils are typically identified by very low SPT N-values (typically less than about 6) and should be viewed with caution.

Sensitive Clays

Some marine clay deposits are also very sensitive and can lose most of their shear strength when disturbed and when loaded dynamically. These deposits are typically indentified with Liquidity Index greater than about 1.2.

Expansive Soils

Expansive soils exist all over the earth's surface, in nearly every region. These soils are often described as having high shrink-swell behavior since they can also shrink if dried out. The natural in-place weathering of rock produces sand, then silt, and finally clay particles – hence the fact that clay is a common soil type. Most clay soils exhibit volume change potential depending on moisture content, mineralogy, and soil structure. The upward forces (swell pressure) of expansive clay may far exceed the adfreeze forces generated by seasonally frozen ground, yet foundations continue to be founded routinely in expansive soil with no allowance for the potential expansion. Foundations should be designed to penetrate below the expansive soil's active zone, or be designed to withstand the forces applied the foundation, e.g., to prevent "slab dishing" or "doming." The active zone is defined as the depth of expansive soil that is affected by seasonal moisture variation. Another method used to design foundations on expansive soil is to prevent the soil's moisture content from changing. Theoretically, if the moisture content does not change, the volume of the clay soil will not change. This is typically difficult to control.

The tensile strength of deep foundations must be sufficient to resist the high tensile forces applied to the foundation by expansive soil via skin friction within the active zone. As an expansive soil swells or heaves, the adhesion force between the soil and the side of the foundation can be of sufficient magnitude to "jack" a foundation out of the ground. CHANCE® Helical Piles are a good choice in expansive soils due to their relatively small shaft size – which results in less surface area subjected to swell pressures and jacking forces. Isolating footings, slabs, and grade beams from subgrade soils by using void form is a typical detail used in areas like Denver, Colorado, where expansive soil is present. The void form isolates the structure from contact with the expansive soil, thereby eliminating the destructive effects of swell pressures.

A Plasticity Index (PI) greater than 25 to 30 is a red flag to the geotechnical engineer. A PI ≥ 25 to 30 indicates the soil has significant volume change potential and should be investigated further. There are fairly simple tests (Atterberg, soil suction test, swell potential) that can be conducted but should be practiced by the informed designer.

Seasonally Frozen Ground

The most obvious soil in this category is the frost susceptible soils (typically, silt) as illustrated by the growth of frost needles and ice lenses in freezing weather. This leads to a commonly observed expansion phenomenon known as frost heave. Frost heave is typically observed on roadbeds, under concrete slabs, and along freshly exposed cuts. Capillary breaks and vapor barriers in conjunction with proper drainage will do much to control this problem, before CHANCE® Helical Piles or ATLAS RESISTANCE® Piers are installed.

A subcategory of this condition is seasonal permafrost. If possible, these ice lenses should be penetrated and not relied on for end bearing.

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PRODUCT FEASIBILITY

PRODUCT FEASIBILITY
SECTION 3

CONTENTS

PRODUCT FEASIBILITY

SYMBOLS USED IN THIS SECTION

FEASIBILITY of USING CHANCE® HELICAL or ATLAS RESISTANCE® PRODUCTS

Hubbell Power Systems, Inc. manufactures steel foundation products that can be designed for a wide range of soil conditions. In order to assist the designer/user in selecting the proper product for the application, Figure 3-1 shows the product type suitable for various soils and rock conditions. When reviewing Figure 3-1, the designer/user should note the following items:

- The most common selection of soil parameters for design is from field testing using the ASTM D 1586 Standard Penetration Test (SPT) and field or laboratory testing of shear strength (cohesion "c" and friction angle "φ"). Refer to Section 2 in this manual for a detailed discussion of geotechnical investigation requirements and to Section 4 for a detailed discussion of structural load requirements for projects using CHANCE® Helical Piles/Anchors and/or ATLAS RESISTANCE® Piers.
- A range is noted based on SPT "N-" values where the ATLAS RESISTANCE® type of pier will provide the foundation underpinning support in an end-bearing mode. This "N-" value is generally above 30 to 35 in cohesionless (sands and gravels) soils and above 35 to 40 in cohesive clay soils.
- A range is indicated for use of the helical piles (compression) and helical anchors (tension). As noted on the chart, there are certain conditions for weathered rock and cemented sands where an initial predrilling will permit the installation of helical plates under relatively high installing torque (generally above 10,000 ft-lbs). Helical piles/anchors have been successfully installed on projects where the target depth is not homogenous or consists of hard clays, cemented sands or weathered rock. These factors must be considered and evaluated before a design can be finalized. Modifications may have to be made to the design to be able to accomplish embedment into the target stratum such as:
  - Cutting a "sea shell" shape into the leading edge of one or more of the helical plates.
  - Predrilling prior to the installation of a helical pile/anchor.
  - Using a shaft configuration that provides adequate torques and resistance to "spikes" during installation.

The product selection chart shown in Figure 3-1 is intended for use on a preliminary basis. Hubbell Power Systems, Inc. assumes no responsibility for the accuracy of design when based solely on Figure 3-1. A Preliminary Design Request Form is provided at the end of this section. This form can be copied and then completed with the required information to request a preliminary design (application) by the Hubbell Power Systems, Inc. engineering department. The completed form can be sent to Hubbell Power Systems, Inc. or directly to your local CHANCE® Distributor.

⚠	*All foundation systems should be designed under the direct supervision of a Registered Professional Engineer knowledgeable in product selection and application.*

Hubbell Power Systems, Inc. steel foundation products offer simplicity in design and flexibility in adapting to the project. The design for ultimate and allowable bearing capacities, anchor or tieback loads for helical products, is established using classical geotechnical theory and analysis, and supplemented by empirical relationships developed from field load tests. In order to conduct the design, geotechnical information is required at the site. The design and data shown in this manual are not intended for use in actual design situations. Each project and application is different as to soils, structure, and all other related factors.

FACTORS of SAFETY

To recognize the variability of soil conditions that may exist at a site, as well as the varied nature of loading on structures and how these loads are transferred through foundations, Hubbell Power Systems, Inc. recommends an appropriate Factor of Safety (FS) when using CHANCE® Helical and ATLAS RESISTANCE® Pier foundation products. Generally, the minimum FS is 2 on all permanent loading conditions and 1.5 for any temporary load situation. National and local building codes may require more stringent Factors of Safety on certain projects. Refer to Section 5 for a discussion of Factors of Safety when using ATLAS RESISTANCE® Piers for underpinning (remedial repair) applications.

SITE ACCESS

The proximity to other structures, rights-of-way and obstructions are some of the first considerations for any construction or improvement. Equipment access may be restricted due to overhead limits and safety issues. The designer needs to consider all the possible limitations when selecting a foundation system. CHANCE® Helical Piles/Anchors and ATLAS RESISTANCE® Piers can generally be used anywhere a soil boring can be taken and are virtually the most access-problem-free foundation systems available today. Restricted access and similar concerns should be shown on the bid documents with the usual notes concerning site conditions.

Vibration and noise can be another limitation to conventional deep foundations (i.e., driven piles, drilled piers). CHANCE® Helical Piles/Anchors and ATLAS RESISTANCE® Piers have been installed inside office buildings, restaurants, retail shops and hospitals without interrupting their normal routines. CHANCE® Helical Pile and ATLAS RESISTANCE® Pier certified installers can assist the designer in determining the best type of product for the application.

WORKING LOADS

Helical piles have been used in the compressive mode to working (design) loads of 200 kip, in the form of the CHANCE HELICAL PULLDOWN® Micropile which is detailed later in this manual. In a “normal consolidated” soil, the working load per foundation is typically less than 100 kip, but special cases may apply.

Working tension loads are typically 100 kip or less. The soil is generally the limiting factor as the number and size of helical piles/anchors can be varied to suit the application. The designer should determine the shaft series of products to use from the information provided in Section 7 – Product Drawings and Ratings.

ATLAS RESISTANCE® Piers have been used in the compressive mode to working (design loads) of 70 kip+. The soil conditions, weight of the existing foundation, and type of foundation are generally the limiting factors when determining the number and size of ATLAS RESISTANCE Piers to use in a given application. The designer should determine the shaft series of products to use from the information provided in Section 7 - Product Drawings and Ratings.

SOILS

Soil may be defined for engineering purposes as the unconsolidated material in the upper mantle of the earth. Soil is variable by the nature of its weathering and/or deposition. The more accurately one can define the soil at a particular site; the better one can predict the behavior of any deep foundation, such as a CHANCE® Helical Pile, HELICAL PULLDOWN® Micropile or ATLAS RESISTANCE® Pier. In the absence of sufficient soil data, assumptions can be made by the designer. The field engineer or responsible person needs to be prepared to make changes in the field based on the soil conditions encountered during construction.

As noted earlier, ATLAS RESISTANCE® Piers will provide the foundation underpinning support in an end-bearing mode provided N-values are generally above 30 to 35 in cohesionless (sands and gravels) soils and above 35 to 40 in cohesive clay soils. CHANCE® Helical Piles can be installed into residual soil and virgin or undisturbed soils other than rock, herein defined as having a SPT "N-value" less than 80 to 100 blows per foot per ASTM D1586. This implies that the correct shaft series of helical piles must be chosen to match to the soil density. For example, a standard 1-1/2" shaft, Type SS helical pile with a total helix area of 1 square foot may require so much installing torque that it may have difficulty penetrating into the bearing stratum without exceeding the torsional strength of the shaft.

Water-deposited soil, marine, riverene (terraces or delta) and lacustrine have a high degree of variability. They may be highly sensitive and may regain strength with time. In these conditions, it is good practice to extend helical piles and and resistance piers deeper into more suitable bearing soil.

Very soft or very loose natural, virgin or undisturbed soils overlying a very dense soil layer, such as unweathered rock, present an ideal situation for the installation of ATLAS RESISTANCE® Piers. Similar soil profiles could present a challenge to the installation of helical piles depending on the weathered nature of the underlying rock. The helices may not develop enough downward thrust in upper soils to penetrate into the hard underlying material. Down pressure is often applied to the shaft to assist in penetration of the helices into the hard underlying material.

The use of helical piles/anchors in controlled or engineered fill is another good application. For example, helical tiebacks are used in the controlled fills of roadway and railway fills to make improvements to the infrastructure.

Helical piles should be capable of penetrating the collapsible soils (such as loess) and poorly cemented granular soils in the southwestern United States.

EQUIPMENT

Equipment suitability consideration and selection is the domain of the contractor. Certified CHANCE® Installers are familiar with the various spatial requirements for his equipment and is best able to determine the type of mounted or portable equipment they can utilize to do the work. The designer may contact the local CHANCE® Distributor or certified installer for guidance on this matter. A wide variety of equipment can be utilized for projects based on such considerations as interior vs. exterior construction and headroom. Mini-excavators have been used indoors to install helical piles.

CONTRACTORS

Certified CHANCE® Installers are available in nearly all areas of North America. These installers should be experienced in the type of work specified. A current project list should be submitted as evidence of experience.

CODES

Building codes may have restrictions regarding the foundation type. Generally, CHANCE® Helical Piles and ATLAS RESISTANCE® Piers fall under the category of deep foundations, such as driven piles or drilled piers. The underpinning shaft series of CHANCE® Helical Piles and ATLAS RESISTANCE® Models AP-2-3500.165 and AP-2-3500.165 (M) have been evaluated to show compliance with past and also the latest revisions of the International Building Code (IBC). CHANCE® Type SS5 and SS175 helical piles and bracket assemblies have been evaluated per International Code Council Evaluation Services (ICC-ES) Acceptance Criteria AC358 for Helical Systems and Devices. In Canada, CCMC Report 13193-R shows compliance with the latest revisions of the Canadian Building Code (CBC). The current evaluation reports can all be found at www.abchance.com.

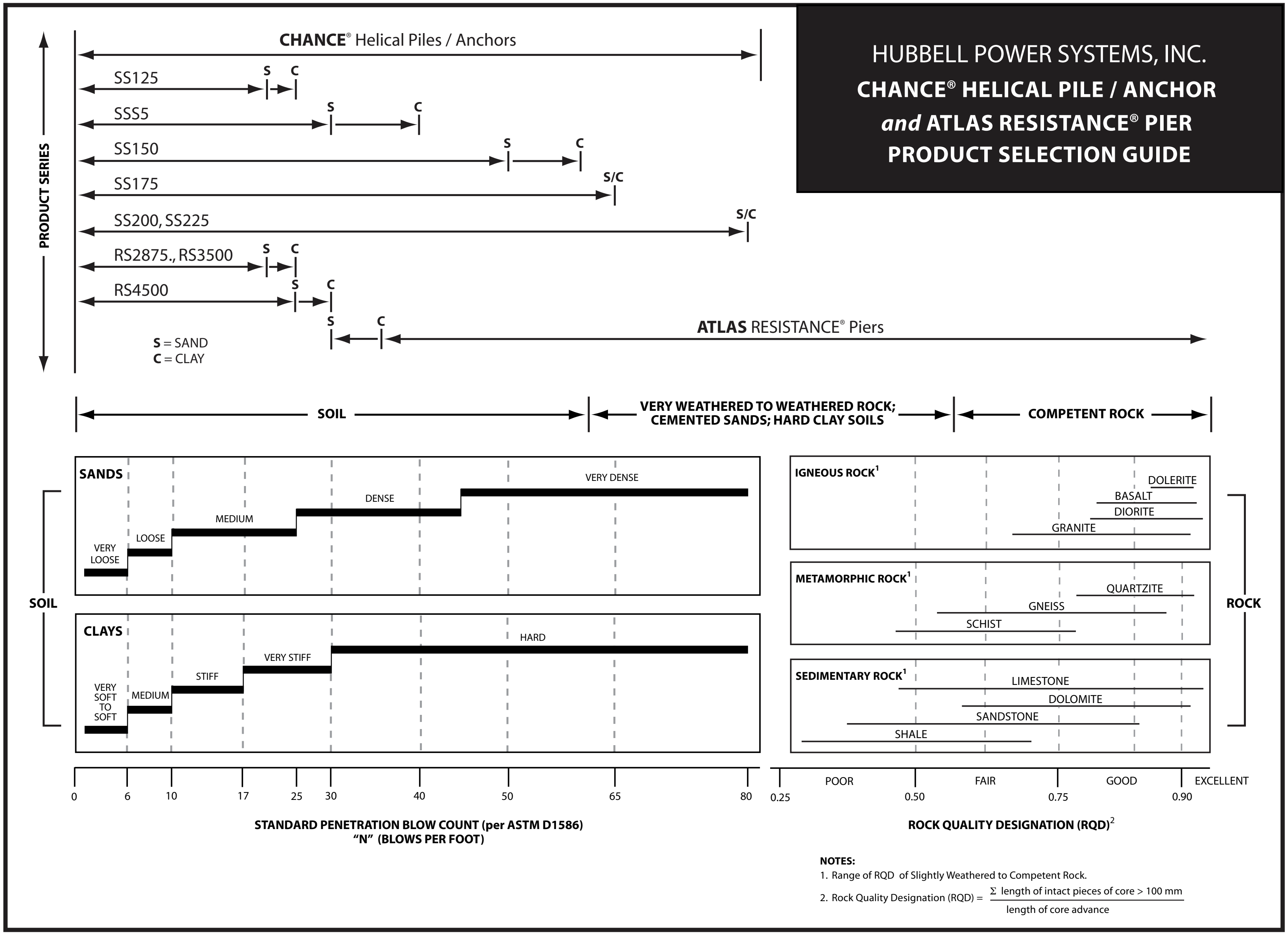
SHAFT SIZE SELECTION BASED on SOIL PARAMETERS

An additional condition that must be evaluated is the ability of the helical pile to penetrate soil to the required depth. For example, a foundation design may require an installation that penetrates a dense fill layer consisting of compacted construction debris (concrete, rubble, etc.) through a compressible organic layer below the fill and finally into the bearing strata. A helical pile shaft with a higher torque rating may be required to adequately penetrate through the fill even though a helical pile shaft with a lower torque rating would satisfy the ultimate capacity requirement. Table 3-1 outlines the maximum blow count or N-value that a particular shaft will typically penetrate. Note that the Type SS helical piles with higher strength shafts and helix material will penetrate harder/denser soils than the Type RS helical piles. Penetrating into harder/denser soils is generally required to support larger loads. The N-values listed in this table are intended to serve as a guide in the preliminary selection of the appropriate shaft series based on using multi-helix configurations. The limits are not intended to be absolute values and higher N-value soils may be penetrated by varying helix diameter, quantity and geometry. Therefore, local field installation experience may indicate more appropriate maximum N-values.

CHANCE® Helical Shaft Series Selection, Table 3-1

SHAFT SERIES	SHAFT SIZE in (mm)	TORQUE RATING Ft-lb (N-m)	MAX N-VALUE* Clay	MAX N-VALUE Sand
SS125	1-1/4 (32)	4,000 (5,400)	25	20
SS5	1-1/2 (38)	5,700 (7,730)	40	30
SS150	1-1/2 (38)	7,000 (9,500)	60	50
SS175	1-3/4 (44)	10,500 (14,240)	65	65
SS200	2 (51)	16,000 (21,700)	<80	<80
SS225	2-1/4 (57)	21,000 (28,475)	<80	<80
RS2875.203	2-7/8 (73)	5,500 (7,500)	25	20
RS2875.276	2-7/8 (73)	8,000 (10,847)	25	20
RS3500.300	3-1/2 (89)	13,000 (17,600)	25	20
RS4500.337	4-1/2 (114)	23,000 (31,200)	30	25
Large Diameter Pipe Pile (LDPP)		Varies based on Shaft Size	30	30
*N-value or Blow Count, from Standard Penetration Test per ASTM D 1586				

Figure 3-1 on page 3-7 shows the same information as contained in the above table along with soil conditions suited for ATLAS RESISTANCE® Piers. This figure does not address the proper product selection based on its application. ATLAS RESISTANCE Piers are used primarily for remedial repair applications involving an existing structure. CHANCE® Helical Piles/Anchors are used for not only remedial repair applications, but for new commercial and residential construction, tieback walls, SOIL SCREW® walls, telecommunication towers, electric utility towers, pipeline buoyancy control, etc.



Product Selection Guide, Figure 3-1

PRELIMINARY CHANCE® HELICAL PILE/ANCHOR and ATLAS RESISTANCE® PIER DESIGN GUIDE

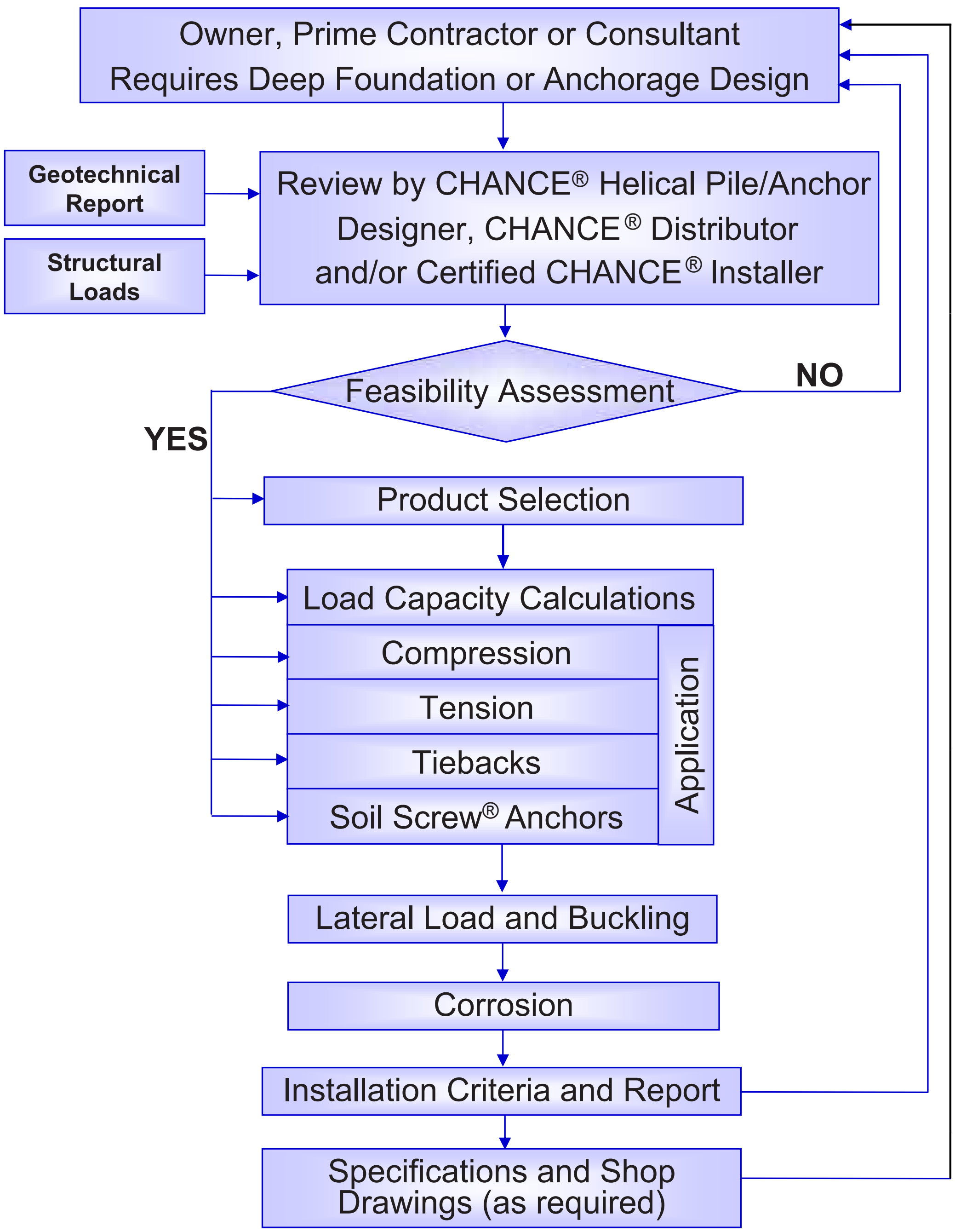
Hubbell Power Systems, Inc. manufactures CHANCE® Helical Piles/Anchors and ATLAS RESISTANCE® Pier products for use as tension anchors and/or compression piles for varied foundation support applications. There are many different applications for these end bearing piles and each application will require:

- An evaluation of the soil strata and soil characteristics of that stratum in which the helical plates or ATLAS RESISTANCE® Pier tip will be seated.
- A selection of the appropriate ATLAS RESISTANCE® Pier, including shaft type and bracket type or CHANCE® Helical Pile foundation, including shaft type, helical plate size, number and configuration. (Note: Type RS piles or CHANCE HELICAL PULLDOWN® Micropiles are strongly recommended in bearing/compression applications where the N-value of supporting soil around the shaft is less than 4. These piles have greater column stiffness relative to the standard CHANCE® Type SS piles. Refer to Buckling/Slenderness Considerations in Section 5 of this Technical Design Manual for a detailed discussion of this subject).
- A determination of the ultimate bearing capacity and suitable FS.

The preliminary design guide shown in Figures 3-2 and 3-3 is intended to assist certified installers, general contractors and consulting engineers in the selection of the appropriate CHANCE® Helical Pile or ATLAS RESISTANCE® Pier.

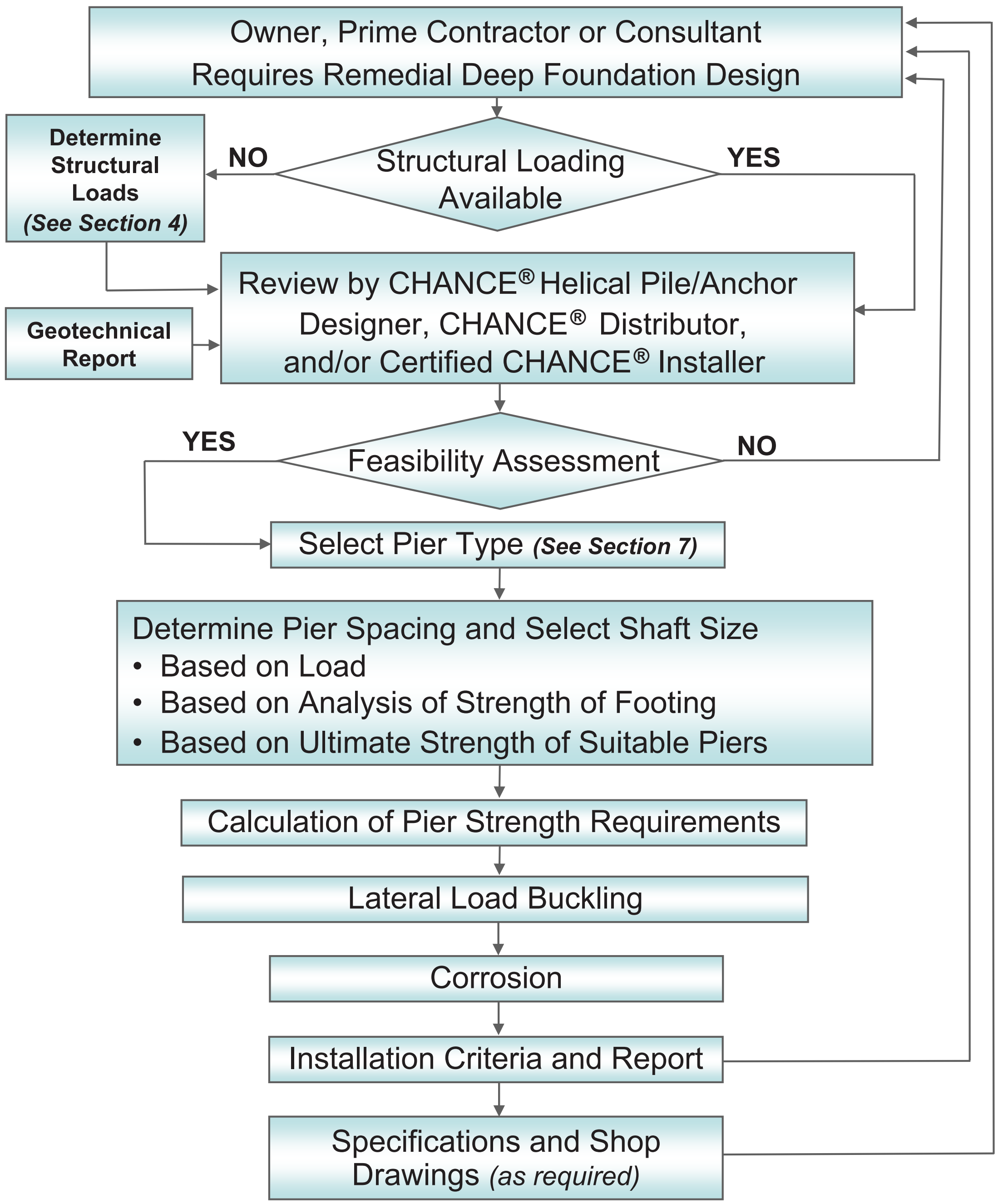
Design should involve professional geotechnical and engineering input. Specific information involving the structures, soil characteristics and foundation conditions must be used for the final design.

Preliminary Design Flowchart for New Construction CHANCE® Helical Piles / Anchors



Design Flowchart for CHANCE® Helical Piles and Anchors (New Construction), Figure 3-2

Design Steps
Atlas Resistance® Piers



Design Flowchart for ATLAS RESISTANCE® Piers (Remedial Repair Applications), Figure 3-3

PRELIMINARY DESIGN REQUEST FORM

Contact at Hubbell Power Systems, Inc., CHANCE® Civil Construction: ______________________________

Installing Contractor

Firm:	Contact:	
Phone:	Fax:	Cell:

Project

Name:	Type:	☐ Foundation	☐ Underpinning/Shoring
Address:		☐ New Construction	☐ Rock
		☐ Tieback Retaining	☐ Other:
		☐ Soil Nail Retaining	

Project Engineer? ☐ Yes ☐ No

Firm:	Contact:
Address:	Phone:
	Fax:
	Email:

Geotechnical Engineer? ☐ Yes ☐ No

Firm:	Contact:
Address:	Phone:
	Fax:
	Email:

Loads

	Design Load	FS (Mech) #1	FS (Geo) #1	Design Load	FS (Mech) #2	FS (Geo) #2
Compression						
Tension						
Shear						
Overturning						

Define the owner's expectations and the scope of the project:

____

____

____

The following are attached: ☐ Plans ☐ Soil Boring ☐ Soil Resistivity ☐ Soil pH

If any of the above are not attached, please explain:

____

____

Date:__________________ Requested Response:______________________ CHANCE® #:__________ Response:__________

Please copy and complete this form to submit a design request.

PRODUCT FEASIBILITY

LOAD DETERMINATION
SECTION 4

CONTENTS

SYMBOLS USED IN THIS SECTION

FS	Factor of Safety	4-4
GWT	Ground Water Table	4-7
T	Tension Load	4-7
SL	Snow Load	4-11
S_K	Snow Load Factor	4-11
ksi	Kips (kilo-pounds) per square inch	4-11
ACI	American Concrete Institute	4-23
AISC	American Institute of Steel Construction	4-23

DISCLAIMER

The information in this manual is provided as a guide to assist you with your design and in writing your own specifications.

Installation conditions, including soil and structure conditions, vary widely from location to location and from point to point on a site.

Independent engineering analysis and consulting state and local building codes and authorities should be conducted prior to any installation to ascertain and verify compliance to relevant rules, regulations and requirements.

Hubbell Power Systems, Inc., shall not be responsible for, or liable to you and/or your customers for the adoption, revision, implementation, use or misuse of this information. Hubbell, Inc., takes great pride and has every confidence in its network of installing contractors and dealers.

Hubbell Power Systems, Inc., does NOT warrant the work of its dealers/installing contractors in the installation of CHANCE® Civil Construction foundation support products.

STRUCTURAL LOADS

TYPES of LOADS

There are generally four common loads that may be resisted by a given foundation element. These are compression, tension, lateral and moment loads. It is anticipated that anyone reading this manual will know the meanings of these loads, but for completeness we will describe them for our purposes here.

A compression load is one that will axially shorten a foundation and is typically considered to act vertically downward. The tension load tends to lengthen a foundation and is often taken to be acting vertically upward. A lateral load is one that acts parallel to the surface of the earth or perpendicular to a vertically installed foundation. The lateral load can also be referred to as a shear load. Moment load tends to bend the foundation about one of its transverse axis. A fifth load is torsion. It tends to twist the foundation about its longitudinal axis. This is a load that is seldom applied except during installation of a helical pile/anchor.

This design manual generally assumes the use of allowable strength design (ASD), i.e., the entire Factor of Safety (FS) is applied to the ultimate capacity of the steel foundation product in the soil to determine a safe (or design) strength. Section 7 of this Design Manual provides the Nominal, LRFD Design, and Allowable Strength of helical pile/anchor. Therefore, the designer can choose to use either limit states or allowable strength design for helical pile/anchor.

DESIGN or WORKING LOAD

The design load or working load is typically considered to be the same load. This is a combination of dead loads and live loads. The dead loads are simply the gravity load of structure, equipment, etc. that will always be there to be resisted by the foundation. The live load takes into account seismic events, wind load, snow load, ice, and occupancy activities. They are transient loads that are dynamic in nature. These loads are sometimes referred to as Unfactored Loads. They do not include any Factor of Safety.

Loads associated with backfill soil should be considered in any type of structural underpinning application. Soil load may be present in foundation lifting or restoration activities and can represent a significant percentage of the overall design load on an individual underpinning element, sometimes approaching as much as 50% of the total design load.

ULTIMATE LOAD

The ultimate load is the combination of the highest dead loads and live loads including safety factors. This load may or may not be the load used for foundation design.

FACTOR of SAFETY

Before a foundation design is complete a Factor of Safety (FS) must be selected and applied. In allowable strength design, the Factor of Safety (FS) is the ratio between the ultimate capacity of the foundation and the design load. A Factor of Safety of 2 is usual but can vary depending on the quality of the information available for the design process and if testing or reliable production control is used. Hubbell Power Systems, Inc. recommends a minimum Factor of Safety of 2 for permanent loading conditions and 1.5 for any temporary loading condition. See page 5-5 for a discussion of Factors of Safety when using ATLAS RESISTANCE® Piers for underpinning (remedial repair) applications.

NOTE: Ultimate load is not the same as ultimate capacity. A foundation has some finite capacity to resist load. The ultimate capacity may be defined as the minimum load at which failure of the foundation is likely to occur, and it can no longer support any additional load.

REVERSING LOADS

Foundation design must allow for the possibility that a load may reverse or change direction. This may not be a frequent occurrence, but when wind changes course or during seismic events, certain loads may change direction. A foundation may undergo tension and compression loads at different times or a reversal in the direction of the applied shear load. The load transfer of couplings is an important part of the design process for reversing loads.

DYNAMIC LOADS

Dynamic or cyclic loads are encountered when supporting certain types of equipment or conditions involving repetitive impact loads. They are also encountered during seismic events and variable wind events. These loads can prove destructive in some soil conditions and inconsequential in others. The designer must take steps to account for these possibilities. Research has shown that multi-helix anchors and piles are better suited to resist dynamic or cyclic loads. Higher factors of safety should be considered when designing for dynamic loads.

CODES and STANDARDS

The minimum load conditions, especially live loads for buildings are usually specified in the governing building codes. There are municipal, state and regional as well as model codes that are proposed for general usage. The designer must adhere to the codes for the project location. Chapter 18 of the IBC 2009 and 2012 contain Code sections for helical piles, as well as sections for general design of deep foundations. Section 4 of ICC-ES ESR-2794 provides guidelines for the design and installation of helical piles.

PRELIMINARY TIEBACK DESIGN GUIDE

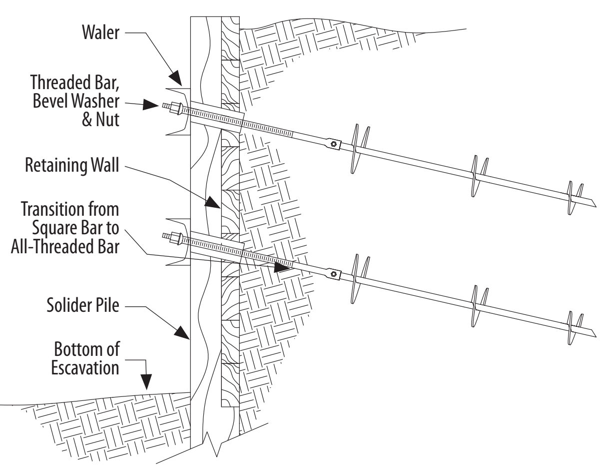
Hubbell Power Systems, Inc. manufactures multi-helix products for use as tiebacks to assist in stabilizing and anchoring structures subjected to lateral loads from earth and water pressure. There are many applications for these tieback products and each application will require:

- An evaluation of the soil characteristics and the lateral earth and water loads on the retaining structure,
- A selection of the appropriate tieback product, including shaft type, helix size(s) and configuration, and
- A determination of the tension load capacity and suitable Factor of Safety.

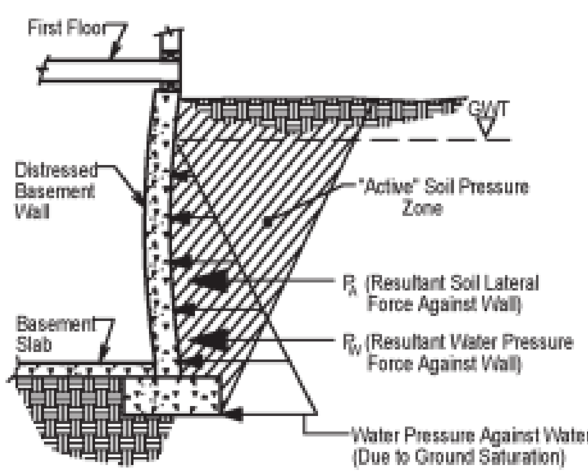
The following preliminary design guide information is intended to assist dealers, installing contractors, and consulting engineers in estimating the required tieback force and placement for the more common tieback applications and to select the appropriate CHANCE® Helical Tieback product. Figure 4-1 illustrates a typical temporary soldier beam and lagging retaining wall utilizing CHANCE® Helical Tiebacks. The commercial uses of CHANCE® Helical Tiebacks include both permanent and temporary sheet pile walls, bulkheads for marine applications, concrete reinforced walls, precast concrete panel walls, etc. They have been used in multi-tier tieback walls to heights of 50’-0.

When using an external waler system consisting of double channels, WF or HP sections, these members shall be positioned relative to the wall face so that their webs are collinear with the tieback tendon. If the waler is not properly oriented with respect to the tieback tendon, then bending moments and shear loads could be introduced into the tieback tendon that could result in a premature failure of the tendon. The tieback tendon is intended to resist only axial loading.

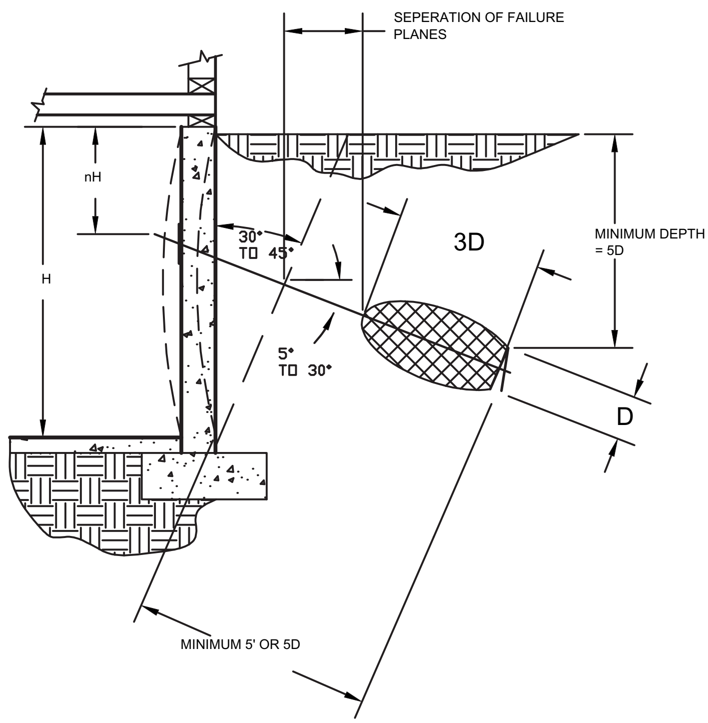
⚠	*It is recommended that a Registered Professional Engineer conduct the design.*



Typical Retaining Wall Tieback Configuration
Figure 4-1



Distressed Basement Wall with “Active” Soil Pressure and Water Pressure Acting Against the Wall
Figure 4-2



Guidelines for Depth and Length for a Typical Installation with Helical Tiebacks
Figure 4-3

TIEBACK DESIGN CONSIDERATIONS

Basement and Retaining Wall Applications

In most regions of the United States, many residential homes have basement walls below grade. Over time, the settling of the ground, plugging of drain tile, extensive rains, plumbing leaks and other environmental factors can cause these basement walls to inwardly bulge, crack, or be subjected to other forms of distress. The CHANCE® Helical Tieback can be an effective repair method for distressed basement walls (See Figure 4-2 and 4-3). There are, however, some general considerations that are important to understand and follow when specifying wall tiebacks.

Active and Passive Pressure Conditions

Figure 4-2 shows a distressed basement wall with the earth pressure "actively" pushing against the wall, as well as water pressure due to the indicated soil saturation condition. Most often it is the combined effect of "active" earth pressure and water pressure that leads to basement wall bulges and cracks. Active earth pressure is defined as the pressure exerted by the earth on a structure that causes movement of the structure away from the soil mass. When a helical tieback is installed and anchored in place, two options are available:

- A portion of the soil is removed, the helical tieback is used to restore the wall toward its original position and the soil is backfilled against the wall, or
- The helical tieback is merely loaded and locked in position with no restoration. In this case, the wall is merely stabilized in its' deflected position.

In either case, the soil will continue to exert an "active" pressure against the wall.

The installed helical tieback anchor develops anchoring resistance capacity through development of "passive" earth pressure against the helical plate. Passive earth pressure is defined as the pressure a structure exerts directly on the earth that causes the structure to move in the direction of the soil mass. Thus it is necessary that the helical tieback anchor be installed properly to ensure the ability to develop full "passive" pressure resistance.

It is very important that the basement wall repair should also include remedial drainage work in order to prevent any future condition of soil saturation and resulting water pressure against the wall and/or take into account the full effect of water pressure against the wall in the tieback design. (See Figure 4-2.)

Location and Placement of Tiebacks

Every tieback wall situation is unique, but there are some aspects that merit extra attention. The placement of the anchor is influenced by the height of the soil backfill against the wall. Figure 4-3 shows this condition and a guide for setting the location and minimum length of installation of the tieback. Experience indicates that the tieback should be located close to the point of maximum wall bulge and/or close to the most severe transverse crack. In cases where walls are constructed of concrete block walls or severe cracking occurred in solid concrete walls, a vertical and/or transverse steel channel (waler) or plate must be used to maintain wall integrity.

For other types of wall distress such as multiple cracking or differential settlement induced cracking, the tieback placement location must be selected on a case by case basis.

Another factor to consider is the height of soil cover over the helical tieback. Figure 4-3 shows the recommended minimum height of soil cover is five times the diameter of the largest helical plate. Finally, the helical anchor must be installed a sufficient distance away from the wall in order that the helical plate(s) can fully develop an anchoring capacity by "passive" pressure as shown in Figure 4-3. This requires the length of installation to be related to the height of soil backfill also shown in Figure 4-3. The top-most or last helix installed must be located a minimum of five times its diameter beyond the assumed "active" failure plane.

Estimating Tieback Load Requirements

Estimating the lateral loads acting against basement walls or retaining walls as exerted by the earth requires knowledge of:

- The soil type and condition,
- The structural dimensions of the retaining structure, and.
- Other geotechnical conditions (e.g. ground water table).

Figures 4-4, 4-5, and 4-6 were prepared for preliminary design assistance for estimating tieback load requirements. Figures 4-5 and 4-6 illustrate cases where no ground water table (GWT) is present at the site. If hydrostatic water pressure is present, the magnitude of this pressure is determined and added to the tieback load requirement from the earth pressure.

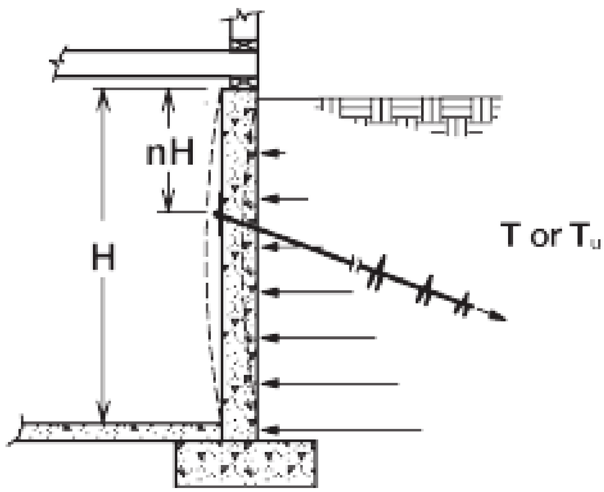
In those cases where the soil and subsurface drainage conditions are not known, it should be assumed in the design that water pressure will be present. As a guideline in preparing tieback load requirement estimates, one tieback row (tier) was used for walls of 15 feet of height or less and two tieback rows (tiers) for walls ranging in height from 15 feet to 25 feet. Individual project conditions and design considerations can cause changes in these guidelines.

PLACEMENT OF TIEBACK ANCHORS

TYPICAL BASEMENT WALL

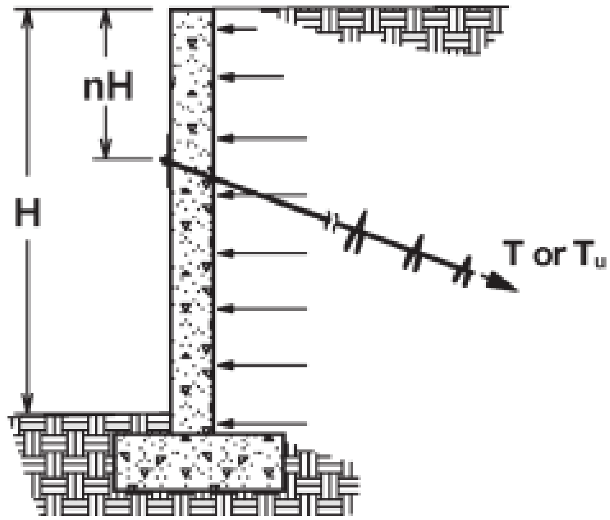
H = Height of backfill
n = Tieback location from top of wall = 0.2 to 0.6
FS = Factor of Safety = 1.5 < FS <2.5
T = Tension load (lb/ft of wall)/cos ø. Assumes tie-back provides 80% of lateral support.
T_U = 18 x (H^2) x FS/cos ø (no water pressure present)
T_U = 45 x (H^2) x FS/cos ø (water pressure present)

Note: Top of wall is assumed to be restrained in the lateral direction



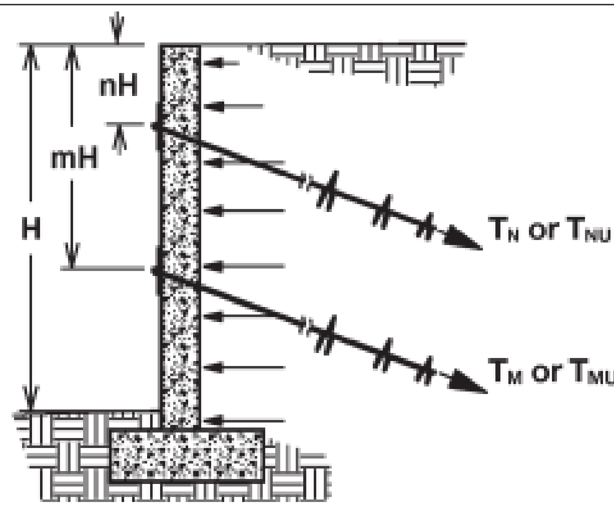
Estimated Tieback Force Required for Basement Applications
Figure 4-4

H = Height of backfill (walls 15 ft or less)
n = Tieback location from top of wall = 0.25 to 0.40
FS = Factor of Safety = 1.5 < FS <2.5
T = Tension Load (lb/ft of wall)/cos ø
T_U = 25 x (H^2) x FS/cos ø
Note: Top of wall is assumed free to translate.



Estimated Tieback Force Required for Retaining Walls 15 Feet High or Less
Figure 4-5

H = Height of backfill (walls 15 to 25 ft)
n = Tieback location from top of wall = 0.20 to 0.30
m = Lower tieback location from top of wall = 0.50 to 0.75
FS = Factor of Safety = 1.5 < FS <2.5
T = Tension Load (lb/ft of wall)/cos ø
T_{NU} = 12 x (H^2) x FS/cos ø
T_{MU} = 18 x (H^2) x FS/cos ø
Note: Top of wall is assumed free to translate.



Estimated Tieback Force Required for Retaining Walls 15 Feet to 25 Feet
Figure 4-6

TECHNICAL DESIGN ASSISTANCE

The engineers at Hubbell Power Systems, Inc. have the knowledge and understand all of the elements of design and installation of CHANCE® Helical Piles/Anchors, Tiebacks, SOIL SCREW® Anchors and ATLAS RESISTANCE® Piers. Hubbell Power Systems, Inc. will prepare a complimentary product selection ("PRELIMINARY DESIGN") on a particular project for use by the engineer of record and our installing contractor or dealer.

If you require engineering assistance in evaluating an application, please contact your CHANCE® Distributor or Certified CHANCE® Installer in your area. These professionals will assist you in collecting the data required to submit the PRELIMINARY DESIGN INITIATION FORM and job specific data. The distributor, installing contractor or dealer will either send Preliminary Design requests to Hubbell Power System, Inc. or will provide the complimentary service themselves.

The PRELIMINARY DESIGN INITIATION FORM may be found on the last page of Section 3 in this manual. Please familiarize yourself with the information that you will need before calling for assistance.

TABLES for ESTIMATING DEAD LINE (DL) and LIVE LINE (LL) LOADS

Tables 4-1 though 4-5 below are provided solely as estimates of the dead and live line loads acting along a perimeter grade beam. It is recommended that a Registered Professional Engineer who is familiar with the site and site specific structural loading conduct the final analysis of the dead and live line loads acting along the perimeter grade beam.

Residential Buildings with Concrete Slab Floors, Table 4-1

BUILDING CONSTRUCTION	BUILDING DIMENSIONS (ft)								
	20′ x 20′	20′ x 30′	20′ x 40′	30′ x 30′	30′ x 45′	30′ x 60′	40′ x 40′	40′ x 60′	40′ x 80′
	ESTIMATED DEAD LOAD at FOUNDATION, DL (lb/ft)								
One Story - Wood/metal/vinyl walls with wood framing on footing.	725	742	753	742	758	768	776	797	810
One Story - Masonry walls with wood framing on footing.	975	992	1003	992	1008	1018	1026	1047	1060
Two Story - Wood/metal/vinyl walls with wood framing on footing.	965	1004	1012	1004	1040	1063	1082	1129	1160
Two Story - First floor masonry, second floor wood/metal.	1215	1254	1280	1254	1290	1313	1332	1379	1410
Two Story - Masonry walls with wood framing on footing.	1465	1504	1530	1504	1540	1563	1582	1629	1660

Residential Buildings with Basements, Table 4-2

BUILDING CONSTRUCTION	BUILDING DIMENSIONS (ft)								
	20′ x 20′	20′ x 30′	20′ x 40′	30′ x 30′	30′ x 45′	30′ x 60′	40′ x 40′	40′ x 60′	40′ x 80′
	ESTIMATED DEAD LOAD at FOUNDATION, DL (lb/ft)								
One Story - Wood/metal/vinyl walls with wood framing on footing.	1060	1092	1114	1092	1121	1140	1156	1195	1220
One Story - Masonry walls with wood framing on footing.	1310	1342	1364	1342	1371	1390	1406	1445	1470
Two Story - Wood/metal/vinyl walls with wood framing on footing.	1300	1354	1390	1354	1403	1435	1462	1528	1570
Two Story - First floor masonry, second floor wood/metal.	1550	1604	1640	1604	1653	1685	1712	1778	1820
Two Story - Masonry walls with wood framing on footing.	1800	1854	1890	1854	1903	1935	1962	2028	2070

Commercial Buildings, Table 4-3

BUILDING CONSTRUCTION	BUILDING DIMENSIONS (ft)								
	20′ x 20′	20′ x 30′	20′ x 40′	30′ x 30′	30′ x 45′	30′ x 60′	40′ x 40′	40′ x 60′	40′ x 80′
	ESTIMATED DEAD LOAD at FOUNDATION, DL (lb/ft)								
One Story - Precast concrete walls on footing with slab floor.	2150	2175	2192	2175	2198	2213	2225	2255	2275
One Story - Precast concrete walls and basement on footing.	3130	3175	3205	3175	3217	3243	3265	3320	3355
Two Story - Precast concrete walls on footing with slab floor.	3425	3475	3508	3475	3521	3550	3611	3636	3675
Two Story - Precast concrete walls and basement on footing.	4490	4560	4607	4560	4624	4665	4700	4786	4840

Estimating Live Loads, Table 4-4

<table>
<tr><th rowspan="3">BUILDING CONSTRUCTION</th><th colspan="9">BUILDING DIMENSIONS (ft)</th></tr>
<tr><th>20′ x 20′</th><th>20′ x 30′</th><th>20′ x 40′</th><th>30′ x 30′</th><th>30′ x 45′</th><th>30′ x 60′</th><th>40′ x 40′</th><th>40′ x 60′</th><th>40′ x 80′</th></tr>
<tr><th colspan="9">ESTIMATED LIVE LOAD at FOUNDATION, LL (lb/ft)</th></tr>
<tr><td>One Story - Residential on slab.</td><td colspan="9">N/A</td></tr>
<tr><td>One Story - Residential on basement.</td><td rowspan="3">250</td><td rowspan="3">300</td><td rowspan="3">333</td><td rowspan="3">300</td><td rowspan="3">346</td><td rowspan="3">375</td><td rowspan="3">400</td><td rowspan="3">461</td><td rowspan="3">500</td></tr>
<tr><td>One Story - Residential over crawl space.</td></tr>
<tr><td>Two Story - Residential on slab.</td></tr>
<tr><td>Two Story - Residential on basement.</td><td rowspan="2">500</td><td rowspan="2">600</td><td rowspan="2">667</td><td rowspan="2">600</td><td rowspan="2">692</td><td rowspan="2">750</td><td rowspan="2">800</td><td rowspan="2">923</td><td rowspan="2">1000</td></tr>
<tr><td>Two Story - Residential over crawl space.</td></tr>
<tr><td>One Story - Commercial on slab.</td><td colspan="9">N/A</td></tr>
<tr><td>One Story - Commercial on basement.</td><td rowspan="2">450</td><td rowspan="2">540</td><td rowspan="2">600</td><td rowspan="2">540</td><td rowspan="2">623</td><td rowspan="2">675</td><td rowspan="2">720</td><td rowspan="2">831</td><td rowspan="2">900</td></tr>
<tr><td>Two Story - Commercial on slab.</td></tr>
<tr><td>Two Story - Commercial on basement.</td><td>900</td><td>1080</td><td>1200</td><td>1080</td><td>1246</td><td>1350</td><td>1440</td><td>1662</td><td>1800</td></tr>
</table>

ESTIMATING SNOW LOADS (SL)

The required Snow Load Factor (S_K) can be determined from the locally approved building code. This factor will be given in pounds per square foot. To determine the Snow Load along the perimeter of the structure used the following:

$SL = S_K \times [(w \times L) /2 \times (w + L)]$

NOTE: w = width of building, L = length of building

TABLES for ESTIMATING FREE SPANS BETWEEN SUPPORTS

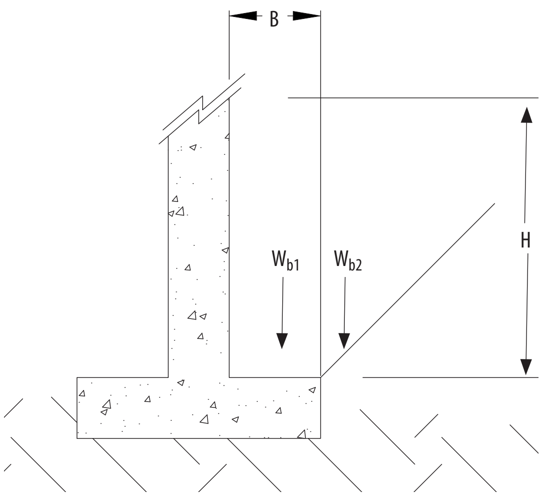
Tables 4-6 through 4-9 are provided to help estimate spacing of CHANCE® Helical Piles or ATLAS RESISTANCE® Piers. One must clearly understand that the tables were calculated assuming that the foundation element was fabricated using proper construction techniques, with properly embedded reinforcing bars rated at 60 ksi and with high quality concrete having a 28-day compressive strength of 3,000 psi. After calculating maximum free span using Equation 4-1 below, the results were checked to ensure that beam shear did not yield a shorter maximum span. Keep in mind that poor construction techniques and/or substandard materials will shorten the allowable span. A Factor of Safety must be applied to the calculated maximum CHANCE® Helical Pile or ATLAS RESISTANCE® Pier spacing based upon experience and judgment.

$$L_s = [(F_y \times d \times A_s) / 1.875 \times P)]^{1/2}$$ **Equation 4-1**

	L_s	=	Maximum footing free span (ft)
	F_y	=	Rebar yield strength = 24,000 psi
where	d	=	Moment arm distance (in)
	A_s	=	Cross section area of steel (in^2)
	P	=	Structural line load (lb/ft)

Estimating Foundation Soil Load (W), Table 4-5

LOAD FROM SOIL OVERBURDEN



Note: W_{b2} may be reduced or may not apply when only stabilizing the structure

FOOTING TOE WIDTH B (in)	HEIGHT OF SOIL OVERBURDEN H (ft)	SOIL TYPE			
		COHESIVE		GRANULAR	
		W_{b1}	W_{b2}	W_{b1}	W_{b2}
3	2	55	220	75	240
	4	110	880	125	960
	6	165	1980	188	2160
	8	220	3520	250	3840
6	2	110	220	125	240
	4	220	880	250	960
	6	330	1980	375	2160
	8	440	3520	500	3840
9	2	165	220	500	240
	4	330	880	1000	960
	6	495	1980	1500	2160
	8	660	3520	2000	3840
12	2	220	220	250	240
	4	440	880	500	960
	6	660	1980	750	2160
	8	880	3520	1000	3840

Use Table 4-5 for structural underpinning applications.

$$x = \frac{(L_S + w_p/12)}{FS_f}$$

Equation 4-2

where

x	=	Pile/pier spacing
W_p	=	Width of foundation repair bracket (in)
FS_f	=	Factor of Safety based upon field conditions and engineering judgment.

Example: The structure has a 6″ thick footing along with an 8″ tall stem wall that was cast with the footing. It was reported that building code required a minimum of two #4 reinforcing bars spaced 3″ from the bottom and sides of the concrete. The structure is a single story wood frame building with masonry veneer and a 4″ concrete slab. The structural load on the perimeter footing was calculated at 1,020 lb/ft plus 250 lb/ft soil overburden.

$$L_S = [(F_y \times d \times A_s) / 1.875 \times P)]^{1/2} \qquad \textbf{Equation 4-3}$$
$$= [(24{,}000 \times 11 \times 0.3926) / (1.875 \times 1270)]^{1/2}$$
$$= [43.526]^{1/2}$$
$$L_S = 6.6 \text{ ft} = \text{maximum free span}$$

where

$d = (6'' - 3'') + 8'' = 11''$

$A_S = 2 \times 0.1963 = 0.3926 \text{ in}^2$

$P = 1020 + 250 = 1270 \text{ lb/ft}$

$$x = \frac{(L_S + w_p/12)}{FS_f} \qquad \textbf{Equation 4-4}$$

where

w_p = 10" (Atlas AP-2-UFB-3500.165 Pier Bracket) or CHANCE® Underpinning Helical Pile Bracket C1500121

FS_f = 1.2 (Inspection revealed a well built foundation)

$x = \frac{(6.6 + 10/12)}{FS_f} = \frac{7.43 \text{ ft}}{1.2}$

x = 6.19 ft (specify pier spacing at 6 feet on center)

For this project specify the spacing at a maximum 6 feet on center to allow for unexpected defects in the beam or foundation loading, or for possible field adjustments caused by obstructions or utilities.

It is important to keep in mind when one wants to reduce the number of piles/piers on a project, the distances in the tables are for a free span between supports. A supplemental steel footing could be offered to the client, which will effectively expand the distance between piles/piers while maintaining the required free span distance.

If we consider the example above, depending upon the complexity of the architecture, the number of piles/piers could be reduced by perhaps 10% to 15% on the total project by simply installing a 24" long, 3/8" x 6" x 6" supplemental steel beam under the footing.

$$x = \frac{(L_S + L_b/12)}{FS_f} \qquad \textbf{Equation 4-5}$$

where

L_b = 24" (supplemental steel beam length)

FS_f = 1.2 (Inspection revealed a well built foundation)

$x = \frac{8.6 \text{ ft}}{1.2}$ = 7.17 ft (pier spacing can be increased to 7 ft on center)

The piles/piers could, if the architecture allows, be spaced on 7-foot centers, while still maintaining the desired 6-foot free span distance.

Tables 4-6 through 4-9 will assist the designer and installer to estimate the maximum free span allowable for some common foundation configurations.

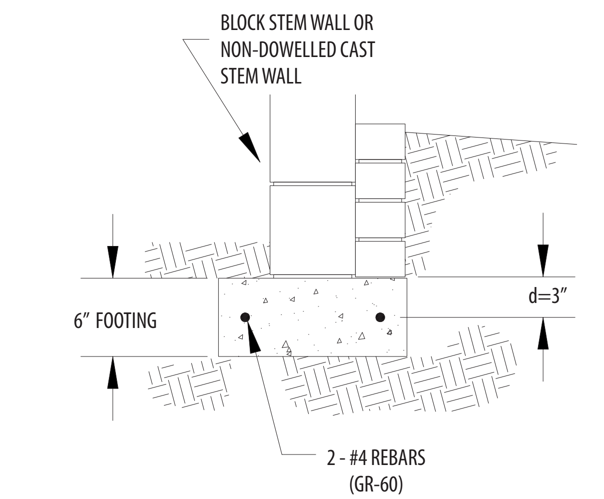


Figure 4-7

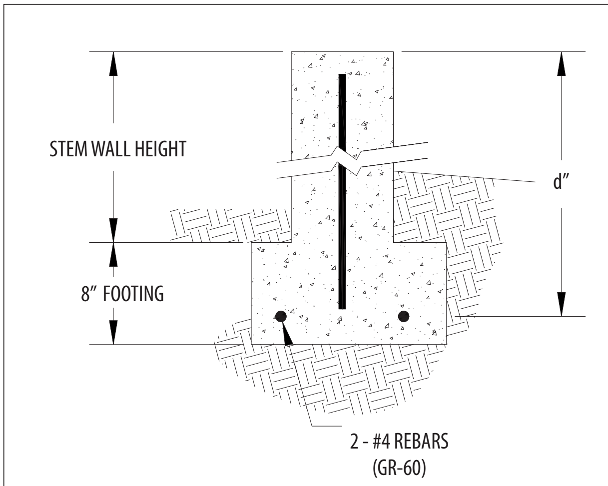


Figure 4-8

WARNING! **THE DESIGNER MUST APPLY A FACTOR OF SAFETY TO THE MAXIMUM FREE SPAN WHEN PLANNING THE UNDERPINNING DESIGN SO THAT BEAM FAILURE IS NOT EXPERIENCED.**

6″ Thick Reinforced Concrete Spread Footings Maximum Free Spans, Table 4-6

6″ THICK x 16″ SPREAD FOOTING (See Figure 4-7)	BUILDING LINE LOAD (lb/ft.)											
	1,000	1,500	2,000	2,500	3,000	3,500	4,000	4,500	5,000	5,500	6,000	6,500
	MAXIMUM FREE SPAN BETWEEN SUPPORTS											
2 - #4 Rebar (Gr 60): concrete block or cast stem wall (not dowelled) d = 3″	3′-11	3′-2	–	–	–	–	–	–	–	–	–	–
2 - #4 Rebar (Gr 60): 6″ x 12″ tall cast stem wall (dowelled or monolithic) d = 15″	8′-8	7′-1	6′-2	5′-6	5′-0	4′-8	4′-4	4′-1	–	–	–	–
2 - #4 Rebar (Gr 60): 6″ x 18″ tall cast stem wall (dowelled or monolithic) d = 21″	–	8′-5	7′-3	6′-6	5′-11	5′-6	5′-2	4′-10	4′-7	4′-5	4′-2	–
2 - #4 Rebar (Gr 60): 6″ x 24″ tall cast stem wall (dowelled or monolithic) d = 27″	–	–	8′-5	7′-4	6′-9	6′-3	5′-10	5′-6	5′-2	5′-0	4′-9	4′-7
2 - #4 Rebar (Gr 60): 6″ x 48″ tall cast stem wall (dowelled or monolithic) d = 51″	–	–	–	–	–	8′-7	8′-0	7′-7	7′-2	6′-10	6′-6	6′-3

8" Thick Reinforced Concrete Spread Footings Maximum Free Spans, Table 4-7

8" THICK x 16" SPREAD FOOTING (See Figure 4-8)	BUILDING LINE LOAD (lb/ft.)											
	1,500	2,000	2,500	3,000	3,500	4,000	4,500	5,000	5,500	6,000	6,500	7,000
	MAXIMUM FREE SPAN BETWEEN SUPPORTS											
2 - #4 Rebar (Gr 60): concrete block or cast stem wall (not dowelled) d = 5"	4′-6	3′-9	3′-6	–	–	–	–	–	–	–	–	–
2 - #4 Rebar (Gr 60): 8" x 12" tall cast stem wall (dowelled or monolithic) d = 17"	7′-7	6′-6	5′-10	5′-4	4′-11	4′-7	4′-4	4′-2	3′-11	3′-9	3′-8	3′-5
2 - #4 Rebar (Gr 60): 8" x 18" tall cast stem wall (dowelled or monolithic) d = 23"	–	7′-7	6′-10	6′-2	5′-9	5′-5	5′-1	4′-10	4′-7	4′-5	4′-3	4′-1
2 - #4 Rebar (Gr 60): 8" x 24" tall cast stem wall (dowelled or monolithic) d = 29"	–	8′-6	7′-8	7′-0	6′-5	6′-0	5′-8	5′-5	5′-2	4′-11	4′-9	4′-7
2 - #4 Rebar (Gr 60): 8" x 48" tall cast stem wall (dowelled or monolithic) d = 53"	–	–	–	–	–	8′-2	7′-8	7′-4	7′-0	6′-8	6′-5	6′-2

12" Thick Reinforced Concrete Spread Footings Maximum Free Spans, Table 4-8

12" THICK x 24" SPREAD FOOTING (See Figure 4-9)	BUILDING LINE LOAD (lb/ft.)											
	3,500	4,000	4,500	5,000	5,500	6,000	6,500	7,000	7,500	8,000	8,500	9,000
	MAXIMUM FREE SPAN BETWEEN SUPPORTS											
3 - #5 Rebar (Gr 60): 10" x 12" tall cast stem wall (dowelled or monolithic) d = 21"	8′-4	7′-10	7′-2	7′-0	6′-8	6′-5	6′-2	5′-11	5′-9	5′-7	5′-5	5′-3
3 - #5 Rebar (Gr. 60): 10" x 18" tall cast stem wall (dowelled or monolithic) d = 27"	–	–	8′-5	8′-0	7′-7	7′-3	7′-0	6′-9	6′-6	6′-4	6′-1	5′-11
3 - #5 Rebar (Gr. 60): 10" x 24" tall cast stem wall (dowelled or monolithic) d = 33"	–	–	–	–	–	8′-0	7′-9	7′-5	7′-2	7′-0	6′-9	6′-7

Monolithic Reinforced Concrete Grade Beam Footing Maximum Free Spans, Table 4-9

TURNED DOWN FOUNDATION CONSTRUCTION (See Figure 4-10)	BUILDING LINE LOAD (lb/ft.)											
	1,000	1,500	2,000	2,500	3,000	3,500	4,000	4,500	5,000	5,500	6,000	6,500
	MAXIMUM FREE SPAN BETWEEN SUPPORTS											
12″ high perimeter beam: 2-#4 bottom rebars (Gr 60) d = 9″	6′-9	5′-6	4′-9	4′-3	3′-11	3′-7	–	–	–	–	–	–
20″ high perimeter beam: 2-#5 bottom rebars (Gr 60) d = 17″	–	–	8′-2	7′-5	6′-8	6′-2	5′-9	5′-6	5′-2	4′-11	4′-9	4′-6
24″ high perimeter beam: 2-#5 bottom rebars (Gr 60) d = 21″	–	–	–	8′-1	7′-5	6′-10	6′-5	6′-1	5′-9	5′-6	5′-3	5′-0

WARNING! THE DESIGNER MUST APPLY A FACTOR OF SAFETY TO THE MAXIMUM FREE SPAN WHEN PLANNING THE UNDERPINNING DESIGN SO THAT BEAM FAILURE IS NOT EXPERIENCED.

PRELIMINARY DESIGN GUIDELINES for REINFORCED CONCRETE GRADE BEAMS

Building loads are most commonly transferred to helical piles through concrete grade beams. Figures 4-11 through 4-15 below provide preliminary design guidance for grade beam sizing and steel reinforcement configuration. The grade beam sizing and selection of steel reinforcement tables below include the total line load for live loads on the beam and the dead load of the beam and structure. The 4″ void under the grade beam is for illustration purposes only. The thickness of the void form will depend on site specific conditions. The final design should be conducted and approved by a Registered Professional Engineer.

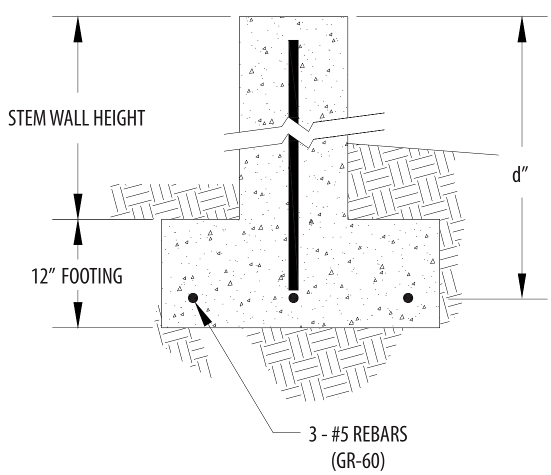


Figure 4-9

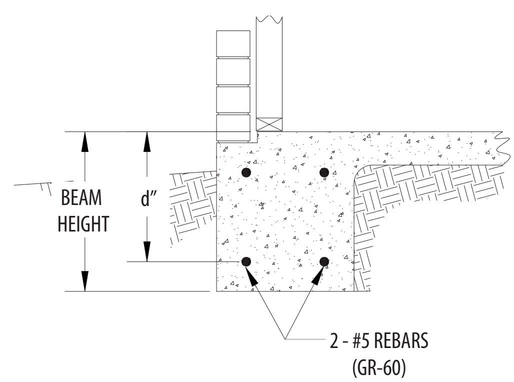
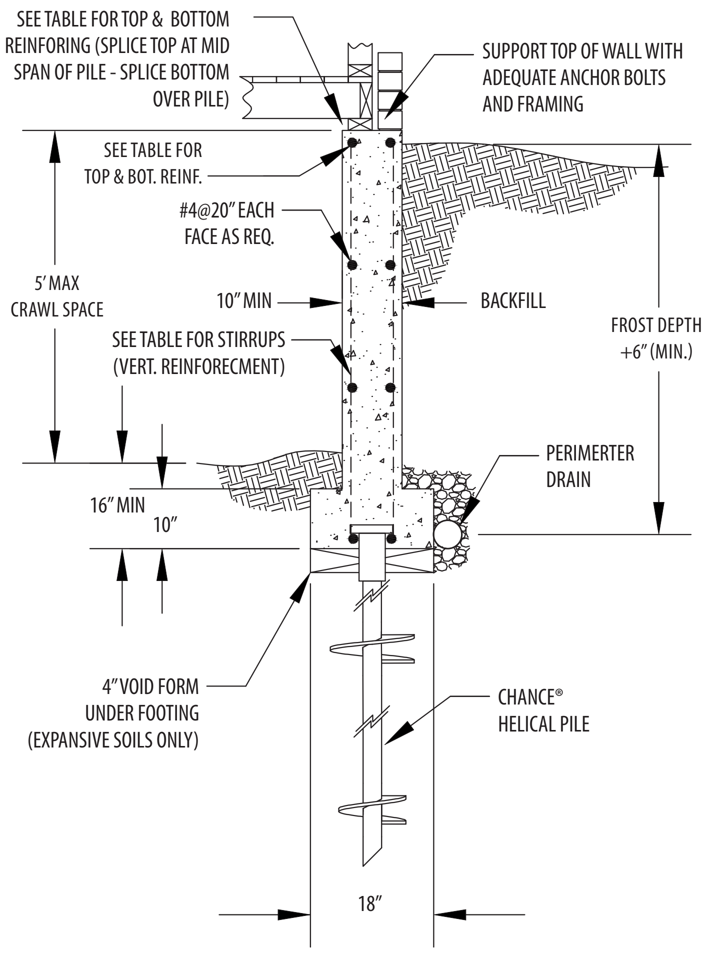
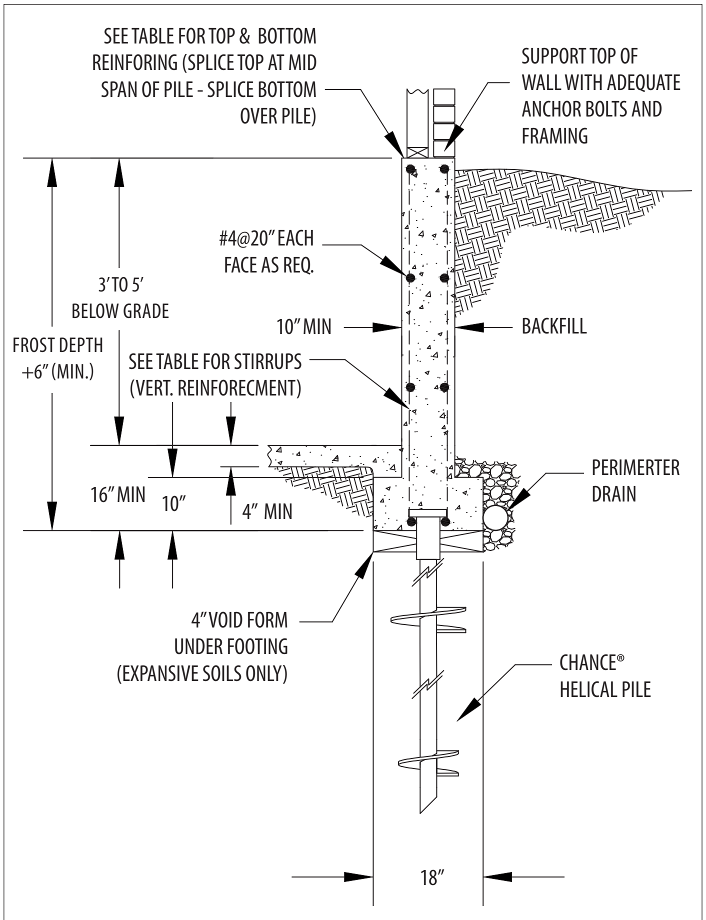


Figure 4-10



Crawl Space Wall Steel Configuration
(See Table 4-10)
Figure 4-11



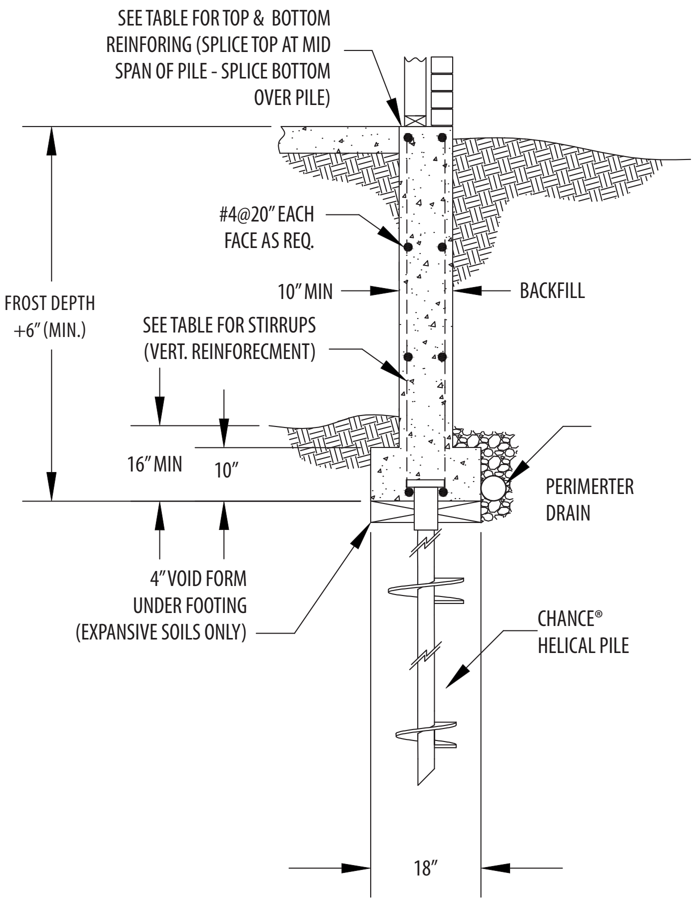
Garden Level Wall Steel Configuration
(See Table 4-11)
Figure 4-12

Crawlspace Wall Reinforcing Steel, Table 4-10

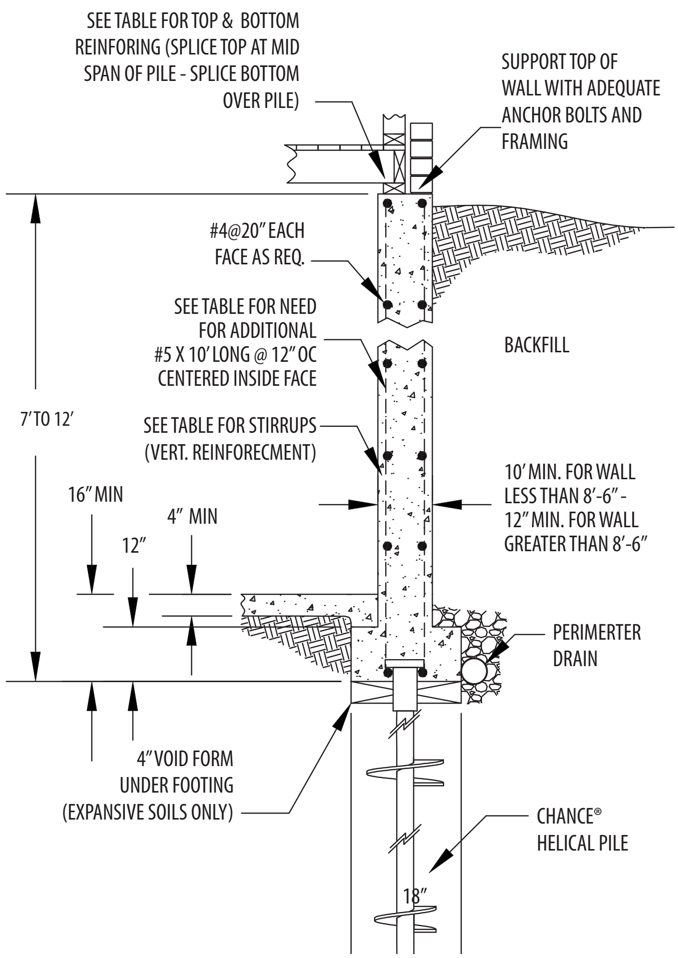
PILE SPACING	WALL HEIGHT	TOTAL FOUNDATION LINE LOAD									
		3,000 (lb/ft)		4,000 (lb/ft)		5,000 (lb/ft)		6,000 (lb/ft)		7,000 (lb/ft)	
		STEEL REINFORCING BARS REQUIRED									
		Top & Bottom	Stirrup ("O.C.)	Top & Bottom	Stirrup ("O.C.)	Top & Bottom	Stirrup ("O.C.)	Top & Bottom	Stirrup ("O.C.)	Top & Bottom	Stirrup ("O.C.)
8′	3′	2- #5		2- #6		2- #6		2 - #7		2 - #7	
	4′	2- #4	#3 @ 15″	2- #5	#3 @ 15″	2- #6	#3 @ 15″	2 - #6	#3 @ 15″	2 - #7	#3 @ 15″
	5′	2- #4		2- #4		2- #5		2 - #5		2 - #6	
10′	3′	2- #6		2- #7		2- #7		2 - #8		2 - #8	
	4′	2- #5	#3 @ 15″	2- #6	#3 @ 15″	2- #7	#3 @ 15″	2 - #8	#3 @ 15″	2 - #8	#3 @ 15″
	5′	2- #5		2- #5		2- #6		2 - #7		2 - #7	
12′	3′	2- #7		2- #7		2- #8		4 - #6		4 - #7	
	4′	2- #6	#3 @ 15″	2- #7	#3 @ 15″	2- #8	#3 @ 15″	4 - #6	#3 @ 15″	2 - #8	#3 @ 15″
	5′	2- #6		2- #7		2- #7		2 - #8		4 - #6	
15′	3′	2- #8		4- #6		4 - #7		4 - #8	#3 @ 11″	5 - #8	#3 @ 9″
	4′	2- #8	#3 @ 15″	2- #8	#3 @ 15″	4 - #6	#3 @ 15″	4 - #7	#3 @ 15″	4 - #8	#3 @ 15″
	5′	2- #7		2- #8		4 - #7		4 - #7		4 - #7	

Garden Level Wall Reinforcing Steel, Table 4-11

PILE SPACING	WALL HEIGHT	TOTAL FOUNDATION LINE LOAD									
		3,000 (lb/ft)		4,000 (lb/ft)		5,000 (lb/ft)		6,000 (lb/ft)		7,000 (lb/ft)	
		STEEL REINFORCING BARS REQUIRED									
		Top & Bottom	Stirrup ("O.C.)	Top & Bottom	Stirrup ("O.C.)	Top & Bottom	Stirrup ("O.C.)	Top & Bottom	Stirrup ("O.C.)	Top & Bottom	Stirrup ("O.C.)
8′	3′	2- #5	#3 @ 15″	2- #6	#3 @ 15″	2- #6	#3 @ 15″	2 - #7	#3 @ 15″	2 - #7	#3 @ 15″
	4′	2- #4		2- #5		2- #6		2 - #6		2 - #7	
	5′	2- #4	#3 @ 12″	2- #4	#3 @ 12″	2- #5	#3 @ 12″	2 - #5	#3 @ 12″	2 - #6	#3 @ 12″
10′	3′	2- #6	#3 @ 15″	2- #7	#3 @ 15″	2- #7	#3 @ 15″	2 - #8	#3 @ 15″	2 - #8	#3 @ 15″
	4′	2- #5		2- #6		2- #7		2 - #8		2 - #8	
	5′	2- #5	#3 @ 12″	2- #6	#3 @ 12″	2- #6	#3 @ 12″	2 - #7	#3 @ 12″	2 - #7	#3 @ 12″
12′	3′	2- #7	#3 @ 15″	2- #7	#3 @ 15″	2- #8	#3 @ 15″	4 - #6	#3 @ 15″	4 - #7	#3 @ 15″
	4′	2- #6		2- #7		2- #8		2 - #8		2 - #8	
	5′	2- #6	#3 @ 12″	2- #7	#3 @ 12″	2- #7	#3 @ 12″	2 - #8	#3 @ 12″	4 - #6	#3 @ 12″
15′	3′	2- #8	#3 @ 15″	4- #6	#3 @ 15″	4 - #7	#3 @ 15″	4 - #8	#3 @ 10″	5 - #8	#3 @ 9″
	4′	2- #8		2- #8		4 - #6		4 - #7	#3 @ 15″	4 - #8	#3 @ 15″
	5′	2- #7	#3 @ 12″	2- #8	#3 @ 12″	4 - #7	#3 @ 12″	4 - #7	#3 @ 12″	4 - #7	#3 @ 12″



Grade Frost Wall Steel Configuration
(See Table 4-12)
Figure 4-13



Basement Wall Steel Configuration
(See Table 4-13)
Figure 4-14

Grade Frost Wall Reinforcing Steel, Table 4-12

PILE SPACING	WALL HEIGHT	TOTAL FOUNDATION LINE LOAD									
		3,000 (lb/ft)		4,000 (lb/ft)		5,000 (lb/ft)		6,000 (lb/ft)		7,000 (lb/ft)	
		STEEL REINFORCING BARS REQUIRED									
		Top & Bottom	Stirrup (“O.C.)	Top & Bottom	Stirrup (“O.C.)	Top & Bottom	Stirrup (“O.C.)	Top & Bottom	Stirrup (“O.C.)	Top & Bottom	Stirrup (“O.C.)
8′	3′	2- #5		2- #6		2- #6		2 - #7		2 - #7	
	4′	2- #4	#3 @ 15″	2- #5	#3 @ 15″	2- #6	#3 @ 15″	2 - #6	#3 @ 15″	2 - #7	#3 @ 15″
	5′	2- #4		2- #4		2- #5		2 - #5		2 - #6	
10′	3′	2- #6		2- #7		2- #7		2 - #8		2 - #8	
	4′	2- #5	#3 @ 15″	2- #6	#3 @ 15″	2- #7	#3 @ 15″	2 - #8	#3 @ 15″	2 - #8	#3 @ 15″
	5′	2- #5		2- #5		2- #6		2 - #7		2 - #7	
12′	3′	2- #7		2- #7		2- #8		4 - #6		4 - #7	
	4′	2- #6	#3 @ 15″	2- #7	#3 @ 15″	2- #8	#3 @ 15″	2 - #8	#3 @ 15″	2 - #8	#3 @ 15″
	5′	2- #6		2- #7		2- #7		2 - #8		4 - #6	
15′	3′	2- #8		4- #6		4 - #7		4 - #8	#3 @ 12″	4 - #8	#3 @ 9″
	4′	2- #8	#3 @ 15″	2- #8	#3 @ 15″	4 - #7	#3 @ 15″	4 - #7	#3 @ 15″	4 - #8	#3 @ 15″
	5′	2- #7		2- #8		4 - #7		4 - #7		4 - #7	

Basement Wall Reinforcing Steel Configuration, Table 4-13

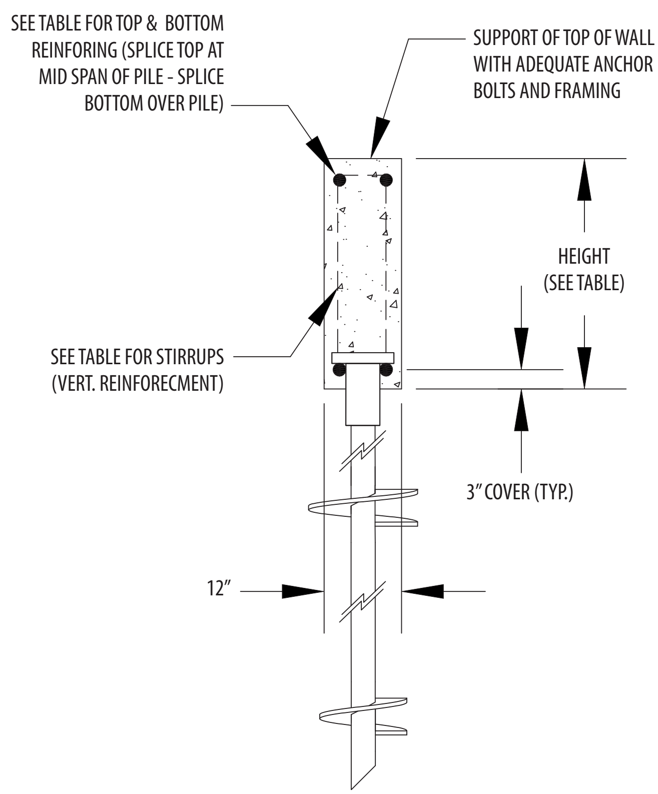
PILE SPACING	WALL HEIGHT	TOTAL FOUNDATION LINE LOAD									
		3,000 (lb/ft)		4,000 (lb/ft)		5,000 (lb/ft)		6,000 (lb/ft)		7,000 (lb/ft)	
		STEEL REINFORCING BARS REQUIRED									
		Top & Bottom	Stirrup ("O.C.)	Top & Bottom	Stirrup ("O.C.)	Top & Bottom	Stirrup ("O.C.)	Top & Bottom	Stirrup ("O.C.)	Top & Bottom	Stirrup ("O.C.)
8′	7′	2- #4	#3 @ 11″	2- #4	#3 @ 11″	2- #4	#3 @ 11″	2 - #5	#3 @ 11″	2 - #5	#3 @ 11″
	8′	2- #4	#3 @ 8″	2- #4	#3 @ 8″	2- #4	#3 @ 8″	2 - #4	#3 @ 8″	2 - #5	#3 @ 8″
	9′	2- #4	#4 @ 12″	2- #4	#4 @ 12″	2- #4	#4 @ 12″	2 - #4	#4 @ 12″	2 - #4	#4 @ 12″
	10′	2- #4	#4 @ 9″	2- #4	#4 @ 9″	2- #4	#4 @ 9″	2 - #4	#4 @ 9″	2 - #4	#4 @ 9″
	11′	2- #4	#4 @ 16″ *	2- #4	#4 @ 16″ *	2- #4	#4 @ 16″ *	2 - #4	#4 @ 16″ *	2 - #4	#4 @ 16″ *
	12′	2- #4	#4 @ 12″ *	2- #4	#4 @ 12″ *	2- #4	#4 @ 12″ *	2 - #4	#4 @ 12″ *	2 - #4	#4 @ 12″ *
10′	7′	2- #4	#3 @ 11″	2- #5	#3 @ 11″	2- #5	#3 @ 11″	2 - #6	#3 @ 11″	2 - #6	#3 @ 11″
	8′	2- #4	#3 @ 8″	2- #4	#3 @ 8″	2- #5	#3 @ 8″	2 - #5	#3 @ 8″	2 - #6	#3 @ 8″
	9′	2- #4	#4 @ 12″	2- #4	#4 @ 12″	2- #5	#4 @ 12″	2 - #5	#4 @ 12″	2 - #6	#4 @ 12″
	10′	2- #4	#4 @ 9″	2- #4	#4 @ 9″	2- #4	#4 @ 9″	2 - #5	#4 @ 9″	2 - #5	#4 @ 9″
	11′	2- #4	#4 @ 16″ *	2- #4	#4 @ 16″ *	2- #4	#4 @ 16″ *	2 - #5	#4 @ 16″ *	2 - #5	#4 @ 16″ *
	12′	2- #4	#4 @ 12″ *	2- #4	#4 @ 12″ *	2- #4	#4 @ 12″ *	2 - #4	#4 @ 12″ *	2 - #5	#4 @ 12″ *
12′	7′	2- #5	#3 @ 11″	2- #6	#3 @ 11″	2- #6	#3 @ 11″	2 - #7	#3 @ 11″	2 - #7	#3 @ 11″
	8′	2- #5	#3 @ 8″	2- #5	#3 @ 8″	2- #6	#3 @ 8″	2 - #6	#3 @ 8″	2 - #7	#3 @ 8″
	9′	2- #4	#4 @ 12″	2- #5	#4 @ 12″	2- #6	#4 @ 12″	2 - #6	#4 @ 12″	2 - #7	#4 @ 12″
	10′	2- #4	#4 @ 9″	2- #5	#4 @ 9″	2- #5	#4 @ 9″	2 - #6	#4 @ 9″	2 - #6	#4 @ 9″
	11′	2- #4	#4 @ 16″ *	2- #5	#4 @ 16″ *	2- #5	#4 @ 16″ *	2 - #6	#4 @ 16″ *	2 - #6	#4 @ 16″ *
	12′	2- #4	#4 @ 12″ *	2- #4	#4 @ 12″ *	2- #5	#4 @ 12″ *	2 - #5	#4 @ 12″ *	2 - #6	#4 @ 12″ *
15′	7′	2- #6	#3 @ 11″	2 - #7	#3 @ 11″	2 - #8	#3 @ 11″	4 - #6	#3 @ 11″	4 - #7	#3 @ 11″
	8′	2- #6	#3 @ 8″	2 - #7	#3 @ 8″	2 - #7	#3 @ 8″	2 - #8	#3 @ 8″	4 - #6	#3 @ 8″
	9′	2- #5	#4 @ 12″	2 - #6	#4 @ 12″	2 - #7	#4 @ 12″	2 - #8	#4 @ 12″	2 - #8	#4 @ 12″
	10′	2- #5	#4 @ 9″	2 - #6	#4 @ 9″	2 - #7	#4 @ 9″	2 - #7	#4 @ 9″	2 - #8	#4 @ 9″
	11′	2- #5	#4 @ 16″ *	2 - #6	#4 @ 16″ *	2 - #6	#4 @ 16″ *	2 - #7	#4 @ 16″ *	2 - #7	#4 @ 16″ *
	12′	2- #5	#4 @ 12″ *	2 - #5	#4 @ 12″ *	2 - #6	#4 @ 12″ *	2 - #7	#4 @ 12″ *	2 - #7	#4 @ 12″ *

* Note: Requires added #5 x 10′ long @ 12″ O.C. bars centered vertically on inside wall face – See Figure 4-14.

LOAD DETERMINATION

Reinforcing Configuration Table, Table 4-14

Pile Spacing	TOTAL FOUNDATION LINE LOAD								
	2,000 (lb/ft)			3,000 (lb/ft)			4,000 (lb/ft)		
	STEEL REINFORCING BARS REQUIRED								
	Height	Top & Bottom	Stirrups (in. O.C.)	Height	Top & Bottom	Stirrups (in. O.C.)	Height	Top & Bottom	Stirrups (in. O.C.)
8′	18″	2 x #5	#3 @ 12″	20″	3 x #5	#3 @ 12″	24″	4 x #5	#3 @ 12″
10′	18″	3 x #5	#3 @ 12″	22″	3 x #5	#3 @ 12″	30″	4 x #5	#3 @ 15″
12′	24″	3 x #5	#3 @ 12″	27″	4 x #5	#3 @ 15″	30″	4 x #5	#3 @ 15″
15′	24″	4 x #5	#3 @ 12″	30″	4 x #5	#3 @ 15″	36″	4 x #6	#3 @ 18″



Grade Beam Design
Figure 4-15

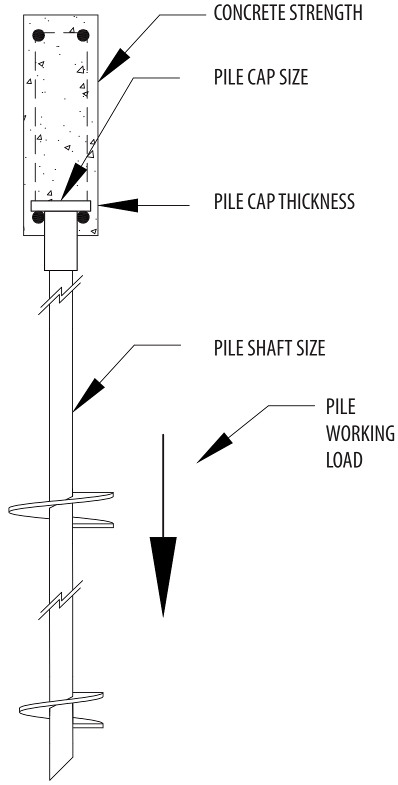
PRELIMINARY DESIGN GUIDELINES for REINFORCED PILE CAPS

Pile cap configurations may be determined from Table 4-15. The table is based upon American Concrete Institute (ACI) criteria for concrete bearing stress from external bearing plates at working loads and from the American Institute of Steel Construction (AISC) criteria for bending stress in the steel plate overhang. Step 1 is based upon a yield-line theory whether bending is across a corner or parallel to an edge.

STEP 1. Select a pile cap plate size from Table 4-15 by looking at the proper row for applicable concrete strength. Locate the lowest value that exceeds the expected pile working load. The proper pile cap plate size is indicated at the bottom of the table.

STEP 2. The pile cap thickness is then determined from the lower portion of Table 4-15. Select the group of rows for the desired pile shaft size. Under the column for the desired pile cap plate size (as determined in Step 1), select the smallest pile cap thickness that exceeds the expected pile working load.

⚠	*It is recommended that a Registered Professional Engineer conduct the design.*



Pile Cap Design
(See Table 4-15)
Figure 4-16

Pile Cap Configuration Table, Table 4-15

STEP 1	PILE CAP PLATE SIZE SELECTOR Limiting Pile Working Loads Controlled by Compressive Strength of Concrete			
Concrete Compressive Strength (psi)	Compressive Working Load on Helical Pile (lb)			
3,000	14,100	32,400	57,600	90,000
3,500	16,800	37,800	67,200	105,000
4,000	19,200	43,200	76,800	120,000
4,500	21,600	48,600	86,400	
5,000	24,000	54,000	96,000	
	RECOMMENDED PILE CAP SIZE			
	4″ x 4″	6″ x 6″	8″ x 8″	10″ x 10″

STEP 2		PILE CAP PLATE SIZE SELECTOR Limiting Pile Working Loads Controlled by Bending Stress in Plate Overhang			
Helical Pile Shaft Series	Pile Cap Thickness	PILE CAP SIZE (From Step 1 above)			
		4″ x 4″	6″ x 6″	8″ x 8″	10″ x 10″
		Compressive Working Load on Helical Pile (lb)			
RS2875.203 RS2875.262	1/4″	23,200	9,780	7,080	5,330
	3/8″	52,200	22,000	15,900	12,000
	1/2″		39,100	28,300	21,300
	3/4″		88,000	63,700	47,900
RS3500.300	1/4″		12,100	8,080	6,250
	3/8″		27,200	18,200	14,100
	1/2″		48,300	32,300	25,000
	3/4″		109,000	72,700	56,300
	1″				100,000
RS4500.337	1/4″		20,000	10,800	8,080
	3/8″		45,000	24,400	18,200
	1/2″		80,000	43,300	32,300
	3/4″			97,500	72,700
SS5 SS150	1/4″	10,000	6,000	5,000	4,000
	3/8″	21,000	12,000	10,000	9,000
	1/2″	40,000	25,000	18,000	16,000
	3/4″	85,000	50,000	40,000	35,000
	1″		90,000	75,000	65,000
SS175	1/4″	14,000	7,000	6,000	5,000
	3/8″	31,000	15,000	11,000	10,000
	1/2″	56,000	27,000	20,000	18,000
	3/4″		60,000	45,000	38,000
	1″		105,000	80,000	70,000
SS200	1/4″	21,000	9,000	6,500	5,500
	3/8″	45,000	18,000	13,000	11,000
	1/2″	82,000	32,000	22,000	19,000
	3/4″		71,000	50,000	42,000
	1″			90,000	75,000

DESIGN METHODOLOGY
SECTION 5

CONTENTS

SYMBOLS USED IN THIS SECTION

DESIGN METHODOLOGY

A_h	Projected Helix Area	5-9
c	Soil Cohesion	5-9
q'	Effective Overburden Pressure	5-9
B	Helix Diameter & Footing Width (Base)	5-9
γ '	Effective Unit Weight of the Soil	5-9
N_c	Bearing Capacity Factor for Cohesive Component of Soil	5-9
N_q	Bearing Capacity Factor for Non-Cohesive Component of Soil	5-9
N_γ	Bearing Capacity Factor for Soil Weight and Foundation Width	5-9
Q_t	Total Ultimate Multi-Helix Anchor/Pile Capacity	5-25
Q_h	Individual Helix Capacity	5-11
Q_s	Capacity Upper Limit	5-21
D	Vertical Depth to Helix Plate	5-10
φ	Angle of Internal Friction	5-11
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K_0	Coefficient of Earth Pressure at Rest	5-14
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K_p	Coefficient of Passive Earth Pressure	5-45
H	Height of Wall or Resisting Element	5-46
P_a	Active Earth Pressure	5-46
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P_{crit}	Critical Compression Load	5-49
E	Modulus of Elasticity	5-49
I	Moment of Inertia	5-49
K	End Condition Parameter	5-49
L_u	Unsupported Length	5-49
Kl/r	Slenderness Ratio	5-49
P_{cr}	Critical Buckling Load	5-50
E_p	Modulus of Elasticity of Foundation Shaft	5-50
I_p	Moment of Inertia of Foundation Shaft	5-51
k_h	Modulus of Subgrade Reaction	5-51
d	Foundation Shaft Diameter	5-51
L	Foundation Shaft Length	5-51
U_{cr}	Dimensionless Ratio	5-51
y	Lateral Deflection of Shaft at Point x	5-51

x	Distance Along the Axis	5-51
EI	Flexural Rigidity of the Foundation Shaft	5-51
Q	Axial Compressive Load	5-51
$E_s y$	Soil Reaction per Unit Length	5-51
E_s	Secant Modulus of the Soil Response Curve	5-51
D	Diameter of Timber, Steel or Concrete Pile Column	5-38
f_s	Sum of Friction and Adhesion Between Soil and Pile	5-38
ΔL_f	Incremental Pile Length	5-38
C_a	Adhesion Factor	5-39
σ_0	Mean Normal Stress	5-38
psf	Pounds per Square Foot	5-23
q	Effective Vertical Stress on Element	5-39
K	Coefficient of Lateral Earth Pressure	5-39
ø	Effective Friction Angle Between Soil & Pile Material	5-39
S	Average Friction Resistance on Pile Surface Area	5-39
P_0	Average Overburden Pressure	5-39
s_u	Undrained Shear Strength	5-12
$(N_1)_{60}$	Normalized SPT N-value	5-32

DISCLAIMER

The information in this manual is provided as a guide to assist you with your design and in writing your own specifications.

Installation conditions, including soil and structure conditions, vary widely from location to location and from point to point on a site.

Independent engineering analysis and consulting state and local building codes and authorities should be conducted prior to any installation to ascertain and verify compliance to relevant rules, regulations and requirements.

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5.1 ATLAS RESISTANCE® PIER CAPACITY

ATLAS RESISTANCE® Piers develop their capacity primarily through end bearing. The current accepted state of the art practice is for ATLAS RESISTANCE® Piers to be installed to a preset performance design criterion. The development of a theoretical capacity model is under study. Current and planned research projects and studies should provide meaningful data for the development of this model in the future.

In general, the tip of the ATLAS RESISTANCE® Pier should be embedded in cohesionless soils with Standard Penetration Test (SPT) "N" values above the 30 to 35 range and in cohesive soils with SPT "N" values above the 35 to 40 range. The ATLAS RESISTANCE® Pier will provide foundation underpinning support in end-bearing when positioned into these SPT "N" value ranges based on past installation experience. See Figures 5-1 and 5-2 for assumed failure patterns under a pile tip in dense sand.

The ATLAS RESISTANCE® Pier is a manufactured, two-stage product designed specifically to produce structural support strength. First, the pier pipe is driven to a firm-bearing stratum then the lift equipment is combined with a manifold system to lift the structure. The ATLAS RESISTANCE® Pier System procedure provides measured support strength. ATLAS RESISTANCE® Piers are spaced at adequate centers where each pier is driven to a suitable stratum and then tested to a force greater than required to lift the structure. ***This procedure effectively load tests each pier prior to lift and provides a measured Factor of Safety (FS) on each pier at lift.***

Performance Design Criterion

The following guidelines are intended to serve as a basis for the selection and installation of a proper ATLAS RESISTANCE® Pier.

- Pier Spacing: The required working load per pier is calculated based on the dead loads and live loads and the ability of the existing foundation to span between the proposed pier locations.

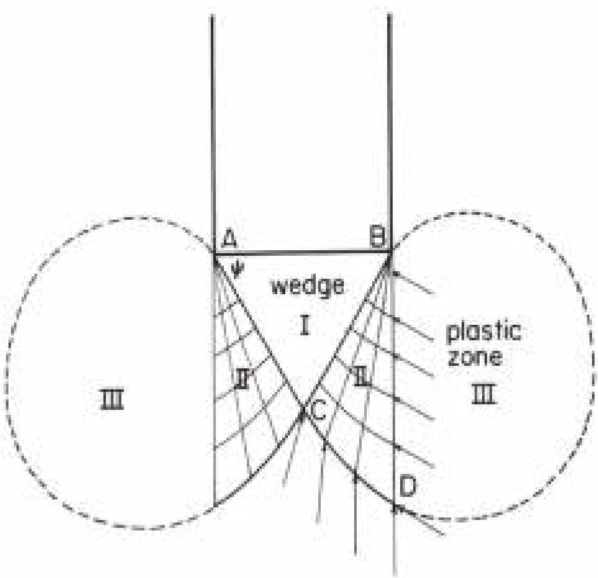

Figure 5-1 Assumed Failure Pattern Under Pile Point

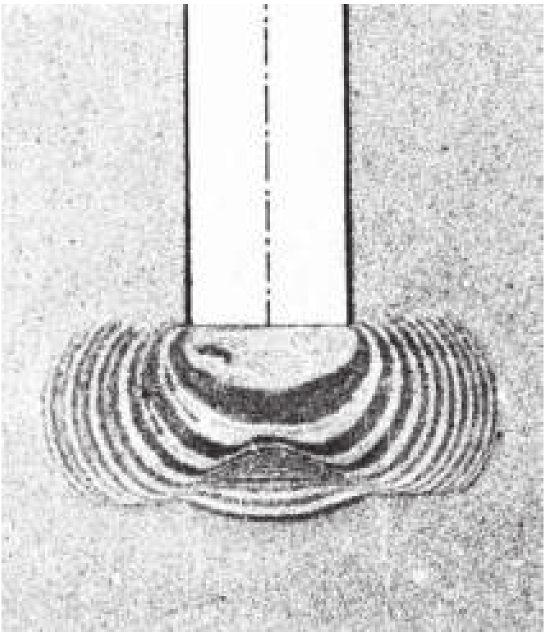

Figure 5-2 Failure Pattern Under Pile Point in Dense Sand

	P	=	DL + LL + SL + W
	P_w	=	(x) x (P)
where	P	=	Line load on footing
	P_w	=	Pier working load
	DL	=	Dead load
	LL	=	Live load
	SL	=	Snow load
	W	=	Soil load
	x	=	Selected pier spacing

- Select Factor of Safety: Hubbell Power Systems, Inc. recommends a minimum Factor of Safety (FS_h) for mechanical strength of the hardware of 2.0.

	FS_h	=	2.0 (may be varied based on engineering judgment)
where	$R_{w\ ULT}$	=	P_w x FS_h
	$R_{w\ ULT}$	=	Minimum ultimate hardware strength based on structural weight

- Select a Pier System with an adequate minimum ultimate strength rating.

	$R_{h\ ULT}$	≥	2 x P_w
where	$R_{h\ ULT}$	=	Minimum ultimate hardware strength based on the published strength rating found in Section 7 of this Technical Design Manual

- Check the maximum pier spacing (x_{MAX}) based upon the selected hardware capacity.

x_{MAX}	=	($R_{h\ ULT}$) / (FS_h) x (P) (wall and footing must be structurally capable of spanning this distance)
x	≤	x_{MAX}

- Proof Load: ATLAS RESISTANCE® Piers are installed using a two-step process as noted above. First, the ATLAS RESISTANCE® Pier is driven to a firm bearing stratum. The resistance force applied during this step is called the Proof Load (R_p). Hubbell Power Systems, Inc. recommends a minimum Factor of Safety[1] (FS_P) of 1.5 at installation unless structural lift occurs first.

R_p = $(FS_P) \times (P_W)$

R_p = $1.5 \times (P_W)$

$R_{h\,MAX}$ = $(R_{h\,ULT} / FS_h) \times 1.65$

$R_{h\,MAX}$ = $(R_{h\,ULT} / 2.0) \times 1.65$

R_p < $R_{h\,MAX}$

where $R_{h\,MAX}$ = Maximum installation force based on hardware ultimate capacity[2]

[1] Experience has shown that in most cases the footing and stem wall foundation system that will withstand a given long term working load will withstand a pier installation force of up to 1.5 times that long term working load. If footing damage occurs during installation, the free span between piers ($L_{P\,MAX}$) may be excessive.
[2] It is recommended that $R_{h\,MAX}$ not exceed $(R_{h\,ULT} / 2) \times 1.65$ during installation without engineering approval.

Additional Notes:

Current practice by Hubbell Power Systems, Inc. is to limit the unsupported pier pipe exposure to a maximum of 2 feet at the published working loads for the standard pier systems. The soil must have a SPT "N" of greater than 4. The pier pipe must be sleeved for pier pipe exposures greater than 2 feet and up to 6 feet and/or through the depths where the SPT value "N" is 4 or less. Sleeve must extend at least 36" beyond the unsupported exposure and/or the area of weak soil. If the anticipated lift is to exceed 4", then the ATLAS RESISTANCE® Continuous Lift Pier System should be used.

ATLAS RESISTANCE® Piers can be located as close as 12" (305 mm) between adjacent piers to develop a "cluster" of load bearing elements.

5.2 CHANCE® HELICAL PILE/ANCHOR ULTIMATE BEARING CAPACITY

The capacity of a helical pile/anchor is dependent on the strength of the soil, the projected area of the helix plate(s), and the depth of the helix plate(s) below grade. The soil strength can be evaluated by use of various field and lab techniques. The projected area is controlled by the size and number of helix plates. Helical anchors and screw piles may be used for a variety of applications involving both tension loading (helical anchors) and compression loading (screw piles or helical piles). Screw piles and helical anchors are generally classified as either "shallow" or "deep" depending on the depth of installation of the top helix below the ground surface, usually with respect to the helix diameter. There are some situations in which the installation may be considered partway between "shallow" and "deep", or "intermediate". In this Manual, only design procedures for "shallow" and "deep" installations will be described. Table 1 gives a summary of the most common design situations involving screw-piles and helical anchors that might be encountered. Note that the use of "shallow" multi-helix anchors for either compression or tension loading is not a typical application and is not covered in this Technical Design Manual.

The dividing line between shallow and deep foundations has been reported by various researchers to be between three and eight times the foundation diameter. To avoid problems with shallow installations, the minimum recommended embedment depth of helical piles and anchors is five helix diameters (5D). The 5D depth is the vertical distance from the surface to the top-most helix. Whenever a CHANCE® Helical Pile/Anchor is considered for a project, it should be applied as a deep foundation for the following reasons:

1. A deep bearing plate provides an increased ultimate capacity both in uplift and compression.
2. The failure at ultimate capacity will be progressive with no sudden decrease in load resistance after the ultimate capacity has been achieved.

The approach taken herein for single-helix piles/anchors assumes that the soil failure mechanism will follow the theory of general bearing capacity failure. For multi-helix helical piles and anchors, two possible modes of failure

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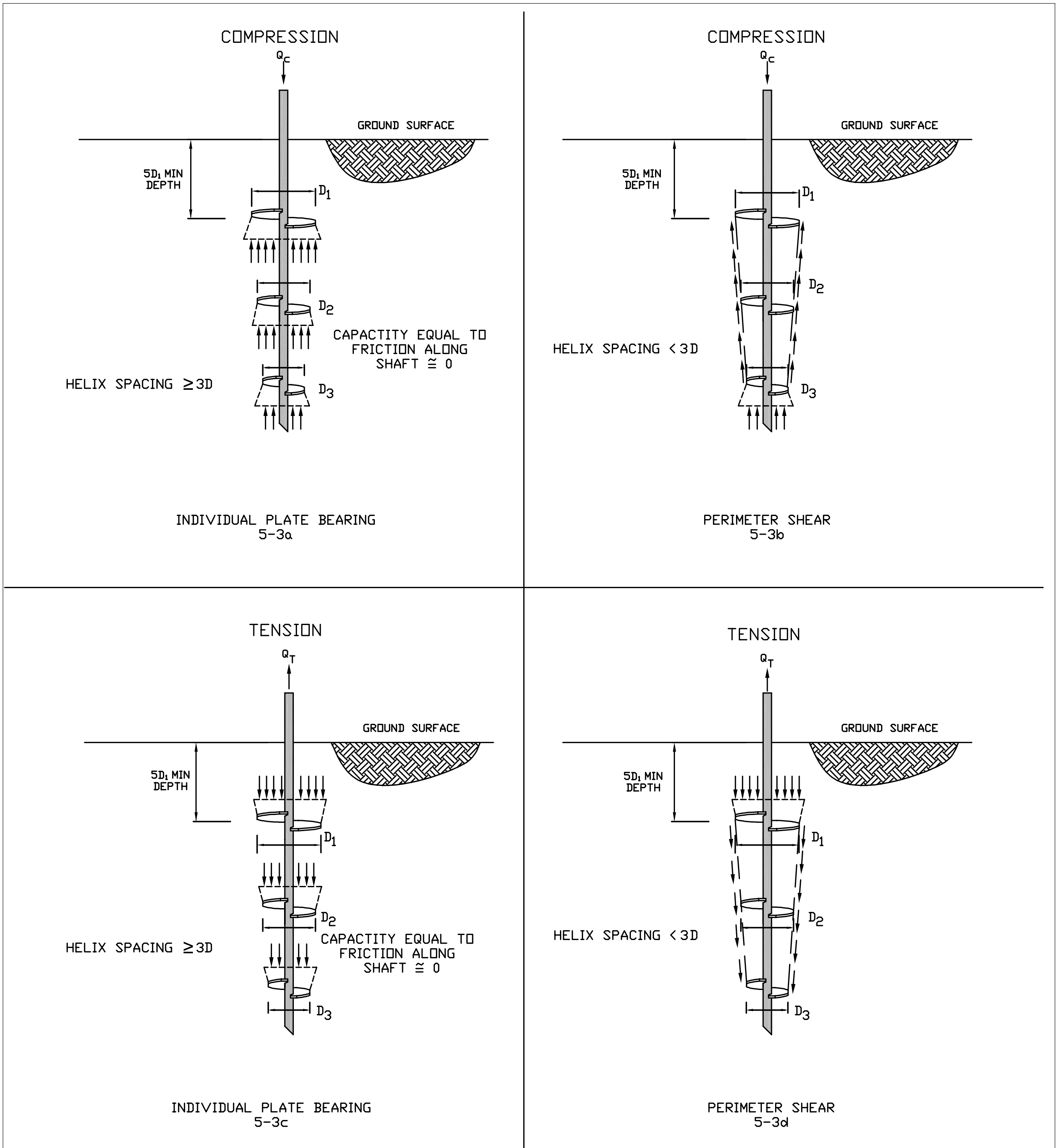

Individual Bearing and Perimerter Shear Models for Helical Piles with Slender Shafts
Figure 5-3

are considered in design, depending on the relative spacing of the helix plates. For wide helix spacing (s/B ≥ 3), the Individual Plate Bearing Method is used; for close helix spacing (s/B < 3), the Perimeter Shear Method is used. These two methods are illustrated in Figures 5-3a & c (Individual Plate Bearing) and 5-3b & d (Perimeter Shear). With Individual Plate Bearing, the helix capacity is determined by calculating the unit bearing capacity of the soil at each helix and then multiplying the result by the individual helix's projected area. Friction along the central shaft is typically not used to determine capacity, but may be included when the central shaft is round, as will be discussed later in this section. The Individual Plate Bearing Method assumes that load capacity will be developed simultaneously and independently by each helix; i.e. no interaction between helix plates. The Perimeter Shear Method assumes that because of the close helix spacing, a prism of soil will develop between the helix plates

Table 5-1 Typical Design Situations for Single-Helix and Multi-Helix Screw-Piles and Helical Anchors

Single-Helix				Multi-Helix			
Failure Condition				Failure Condition			
Shallow		Deep		Shallow		Deep	
C	T	C	T	C	T	C	T
Clay	Clay	Clay	Clay	N/A	N/A	Clay	Clay
Sand	Sand	Sand	Sand	N/A	N/A	Sand	Sand
Mixed Soils	Mixed Soils	Mixed Soils	Mixed Soils	N/A	N/A	Mixed Soils	Mixed Soils

C = Compression T = Tension

and failure in this zone occurs along a plane as shown in Figure 5-3b & d. In reality, the Perimeter Shear Method includes both plate bearing and perimeter shear failure as illustrated.

The following is Terzaghi's general bearing capacity equation, which allows determination of the ultimate capacity of the soil. This equation and its use will be discussed in this section, as it forms the basis of determining helix capacity in soil.

$$Q_{ult} = A_h \left(cN_c + q'N_q + 0.5\,\gamma'\,BN\gamma \right)$$

where

Q_{ult} = Ultimate capacity of the soil
A_h = Projected helix area
c = Soil cohesion
q' = Effective overburden pressure
B = Footing width (base width)
γ' = Effective unit weight of the soil

and Nc, Nq, and $N\gamma$ are bearing capacity factors

Terzaghi's Bearing Capacity Factors are shown in the Table 5-2.

DESIGN METHODOLOGY

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DESIGN METHODOLOGY

Table 5-2. Terzaghi's Shallow Foundation Bearing Capacity Factors [from and Bowles (1988) and ASCE (1993a)]

ϕ'	Nc	$N\gamma$	Nq
0	5.7	0.0	1.0
10	9.6	1.2	2.7
12	10.8	1.7	3.3
14	12.1	2.3	4.0
16	13.7	3.0	4.9
18	15.5	3.9	6.0
20	17.7	4.9	7.4
22	20.3	5.8	9.2
24	23.4	7.8	11.4
26	27.1	11.7	14.2
28	31.6	15.7	17.8
30	37.2	19.7	22.5
32	44.0	27.9	28.5
34	52.6	36.0	36.5
36	63.5	52.0	47.2
38	77.5	80.0	61.5
40	95.7	100.4	81.3
42	119.7	180.0	108.7
44	151.9	257.0	147.7
46	196.2	420.0	204.2
48	258.3	780.1	287.8

Following is quoted from Bowles (1988) concerning the use of Equation 5-6 for deep foundations where the various terms of the bearing capacity equation are distinguished.

"1. The cohesion term predominates in cohesive soil.

2. The depth term ($q'N_q$) predominates in cohesionless soil. Only a small D (vertical depth to footing or helix plate increases Q_{ult} substantially.

3. The base width term $0.5\gamma'BN\gamma$ provides some increase in bearing capacity for both cohesive and cohesionless soils. In cases where B is less than about 2 feet (0.61 m), this term could be neglected with little error."

The base width term of the bearing capacity equation is not used when dealing with helical anchors/piles because, as Bowles indicates, the resulting value of that term is quite small. The effective overburden pressure (q', of consequence for cohesionless soils) is the product of depth and the effective unit weight of the soil. The water table location may cause a reduction in the soil bearing capacity. The effective unit weight of the soil is its in-situ unit weight when it is above the water table. However, the effective unit weight of soil below the water table is its in-situ unit weight less the unit weight of water.

Notes on use of Terzaghi's Bearing Capacity equation:

1. Because helix plates are generally round, Terzaghi's adjustment for round footings is sometimes used for compression loading:

    a. $Q_H = A_H(1.3c'N_C + q'N_q + 0.6\gamma'BN\gamma)$

2. Because B is considered very small for screw-piles and helical anchors, relative to most concrete footings, most engineers choose to ignore the term $0.5\gamma'BN\gamma$ in design.
3. In saturated clays under compression loading, Skempton's (1951) Bearing Capacity Factor for shallow round helical plates can also be used:

    a. $N_C = 6.0(1 + 0.2D/B) \leq 9.0$

4. The unit weight of the soil is the total (wet) unit weight if the helical plate (s) is above the water table and the buoyant unit weight if the helical plate(s) is below the water table.
5. For saturated clay soils, $N_q = 1.0$; For sands, N_q is a function of the friction angle, φ'.
6. For square-shaft anchors/piles, the shaft resistance is generally ignored. For round shaft piles/anchors there may be a component of shaft resistance that contributes to capacity depending on the configuration of connections between extension sections.
7. In all cases, for both compression and tension loading, the upper limit of capacity is governed by the mechanical strength of the pile/anchor as provided by the manufacturer. See Section 7 of this Manual for mechanical strength ratings of CHANCE® Helical Piles/Anchors.

Concern can develop when a helical pile/anchor installation is terminated in sand above the water table with the likelihood that the water table will rise with time to be above the helix plates. In this situation, the helical pile/anchor lead section configuration and depth should be determined with the water at its highest anticipated level. Then the capacity of the same helical-pile/anchor should be determined in the same soil with the water level below the helical pile/anchor, which will typically produce higher load capacities and a more difficult installation, i.e., it will require more installation torque. It is sometimes the case that a larger helical pile/anchor product series, i.e., one with greater torque capacity, must be used in order to facilitate installation into the dry conditions.

5.2.1 Single-Helix Screw-Piles and Helical Anchors – Shallow Installation

5.2.1.1 Compression Loading (Shallow Single-Helix)

A shallow installation, like a shallow foundation, is one in which the ratio of depth (D) of the helix to diameter (B) of the helix is less than or equal to about 5, i.e., $D/B \leq 5$. In this case, the design is very analogous to compression loading of a shallow foundation.

5.2.1.1.a Saturated Clays $\varphi' = 0$; $c > 0$

In saturated clays with $\varphi' = 0$, the term $N\gamma = 0$ and $Nq = 1.0$. The bearing capacity equation becomes:

$$Q_H = A_H(cN_C + \gamma' D) \quad \textbf{Equation 5-9}$$

where:

Q_H = Ultimate Bearing Capacity
A_H = Projected Helix Area
c = "cohesion"; for $\varphi' = 0$; c = undrained shear strength = s_u
N_C = Bearing Capacity Factor for $\varphi' = 0$; for round plates $N_C = 6.0(1 + 0.2D/B) \leq 9$
γ' = effective unit weight of soil above screw-pile
D = Depth

Note: The term $\gamma' D$ is sometimes ignored because it is very small.

5.2.1.1.b Sands $\varphi' > 0$; $c' = 0$

In clean sands with zero cohesion, the cohesion term of the bearing capacity equation drops out and only two terms remain:

$$Q_H = A_H(q' N_q + 0.5\gamma' BN\gamma) \quad \textbf{Equation 5-10}$$

where:

q' = effective surcharge (overburden pressure) = $\gamma' D$
N_q and $N\gamma$ are evaluated from the Table of Bearing Capacity Factors

Note: The term $0.5\gamma' BN\gamma$ is typically ignored for helical piles because the helix plate is small

5.2.1.1.c Mixed Soils $\varphi' > 0$; $c' > 0$

Many soils, such as mixed-grain silty sands, sandy silts, clayey sands, etc., have both a frictional and cohesive component of strength. In these cases, the bearing capacity equation includes all three terms:

$$Q_H = A_H(c' N_C + q' N_q + 0.5\gamma' BN\gamma) \quad \textbf{Equation 5-11}$$

Note: The term $0.5\gamma' BN\gamma$ is typically ignored for helical piles because the helix plate is small.

5.2.1.2 Tension Loading - Axial Uplift (Shallow Single Helix)

Under tension loading in axial uplift, the behavior of a shallow single-helix helical anchor is currently approached more-or-less as an "inverse" bearing capacity problem and the concern is for the failure surface to reach the ground surface, producing "breakout" of the helical plate. Helical anchors should not be installed at vertical depths less than 5 ft. for tension loading. The design approach is similar to that under compression loading, except that instead of using a Bearing Capacity Factor, N_C, a Breakout Factor, F_C, is used.

5.2.1.2.a Saturated Clays φ′ = 0; c > 0

Test results and analytical studies indicate that the Breakout Factor for saturated clays in undrained loading varies as a function of the Relative Embedment of the plate, i.e., D/B. This is much like the transition of shallow to deep foundation behavior under compression loading. Table 5-3 shows the variation in F_C vs. D/B for circular plates. This figure (from Das (1990) shows that $F_C = 1.2(D/B) \le 9$, so that at D/B > 7.5, $F_C = 9$ (i.e., the transition from shallow to deep behavior under tension in clays occurs at about D/B > 7.5). In this case, the ultimate uplift capacity is similar to Equation 5-9 and is given as:

$$Q_{HU} = A_H(cF_C + \gamma' D)$$

where:

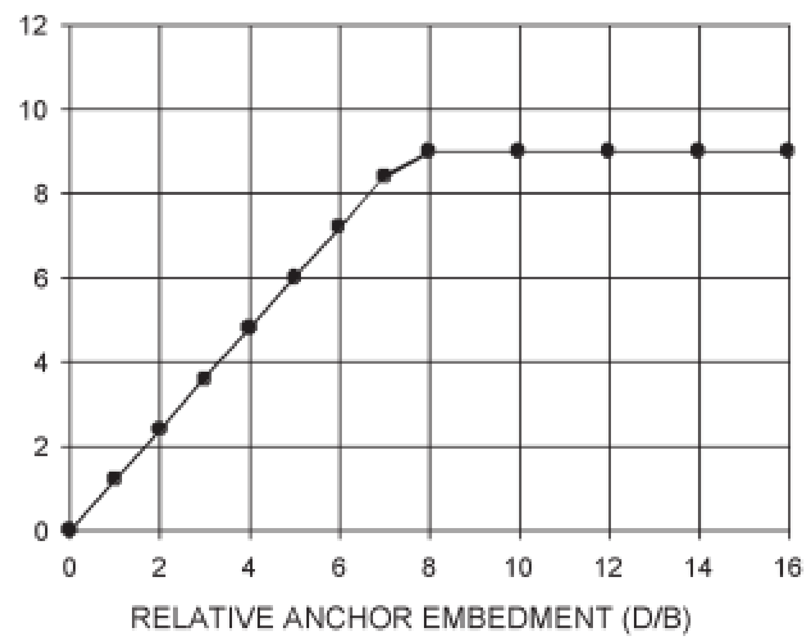
Q_{HU} = Ultimate Uplift Capacity
c = "cohesion"; for φ′ = 0 c = undrained shear strength = s_u

F_C = Breakout Factor for φ′ = 0; $F_C = 1.2(D/B) \le 9$
γ' = effective unit weight of soil above helical anchor plate
D = Depth

Note: The term $\gamma'D$ is sometimes ignored because it is very small.

In some situations the undrained shear strength of clays under tension loading may be reduced to account for some disturbance effects of the clay above the helical plate but this is a matter of engineering judgment.

Table 5-3 Variation in Uplift Breakout Factor for Shallow Single-Helix Anchors in Clay



5.2.1.2.b Sands $\varphi' > 0$; $c' = 0$

In sands the uplift behavior of shallow (generally $D/B \leq 5$) single-helix anchors develops a failure zone that looks similar to an inverted truncated cone. The failure is assumed to take place by the perimeter shear acting along this failure surface, which is inclined from the vertical at an angle of about $\varphi'/2$, as shown in Figure 5.4, and also includes the mass of the soil within the truncated cone. The Ultimate Uplift Capacity is calculated from:

$$Q_{HU} = W_S + \pi\gamma K_0(\tan\varphi')(\cos^2\varphi'/2)\ [(BD^2/2) + (D^3\tan\varphi'/2)/3)] \qquad \textbf{Equation 5-13}$$

where:

W_S = Mass of Soil in Truncated Cone = γV
γ = Total (wet) Unit Weight
V = Volume of Truncated Cone
K_0 = At-Rest Lateral Earth Pressure Coefficient
B= helix diameter
D = vertical plate depth

The volume of the truncated cone is determined from:

$$V = [\pi D/3][B^2 + (B + 2D\tan\varphi'/2)^2 + (B)(B + 2D\tan\varphi'/2)] \qquad \textbf{Equation 5-14}$$

Values of the at-rest lateral earth pressure coefficient for sands can reasonably be taken as:

$K_0 = 1 - \sin\varphi'$

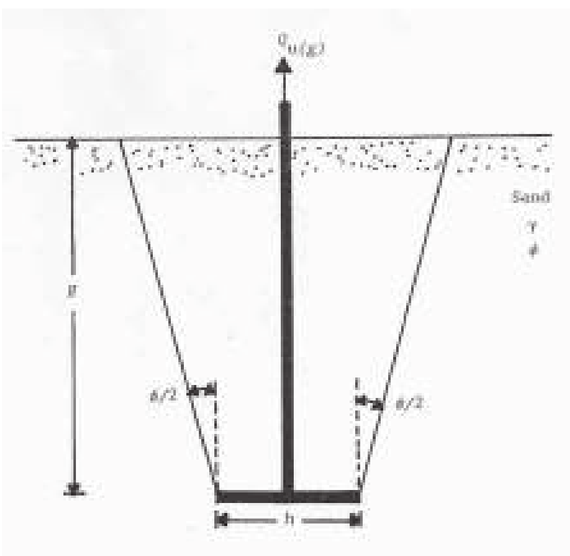


Figure 5-4 Proposed Failure Mechanism for Shallow Single-Helix Anchors in Dense Sand.

5.2.1.2.c Mixed Soils $\varphi' > 0$; $c' = 0$

In mixed soils with both frictional and cohesive components of shear strength, there is an added resisting force in uplift for shallow installations above the value given by Equation 5-13. This added component results from cohesion acting along the surface of the truncated cone failure zone between the helical plate and the ground surface so that an additional term may be added to Equation 5-13 giving:

$$Q_{HU} = W_S + \pi\gamma K_0(\tan\varphi')(\cos^2\varphi'/2) \qquad \textbf{Equation 5-15}$$
$$[(BD^2/2) + (D^3\tan\varphi'/2)/3)] + (c)(A_C)$$

where:

A_C = Surface Area of Truncated Cone

The surface area of a truncated cone can be obtained from:

$$A_C = \pi[(R^2 + r^2) + [(R^2 - r^2) + (D(R + r))^2]^{0.5}] \qquad \textbf{Equation 5-16}$$

where:

r = Radius of Helical Plate = $B/2$
R = Radius of Cone Failure Surface at the Ground Surface = $B/2 + (D)\tan(\varphi'/2)$

The additional component of uplift resulting from soil cohesion, is sometimes ignored since soil cohesion is often lost from water infiltration or rising water table.

5.2.2 Single-Helix Screw-Piles and Screw-Anchors – Deep Installation

Deep installations of screw-piles and helical anchors are generally more common than shallow installations, provided there is sufficient soil depth to actually perform the installation. The reason is simply that higher load capacities are generally developed from a deeper installation in the same soil so it makes more sense economically to go for a deep installation when possible. Figure 5.5 below demonstrates the single-helix plate capacity model, where the soil failure mechanism will follow the theory of general bearing plate capacity. Compression capacity is mobilized from soil below the helix plate and tension capacity from soil above the helix plate.

5.2.2.1 Compression Loading (Deep Single-Helix)

A deep installation, like a deep foundation, is one in which the ratio of depth (D) of the helix to diameter (B) of the helix is greater than 5 - 7, i.e., D/B > 5 - 7. In this case, the design is very analogous to compression loading of deep end bearing foundation.

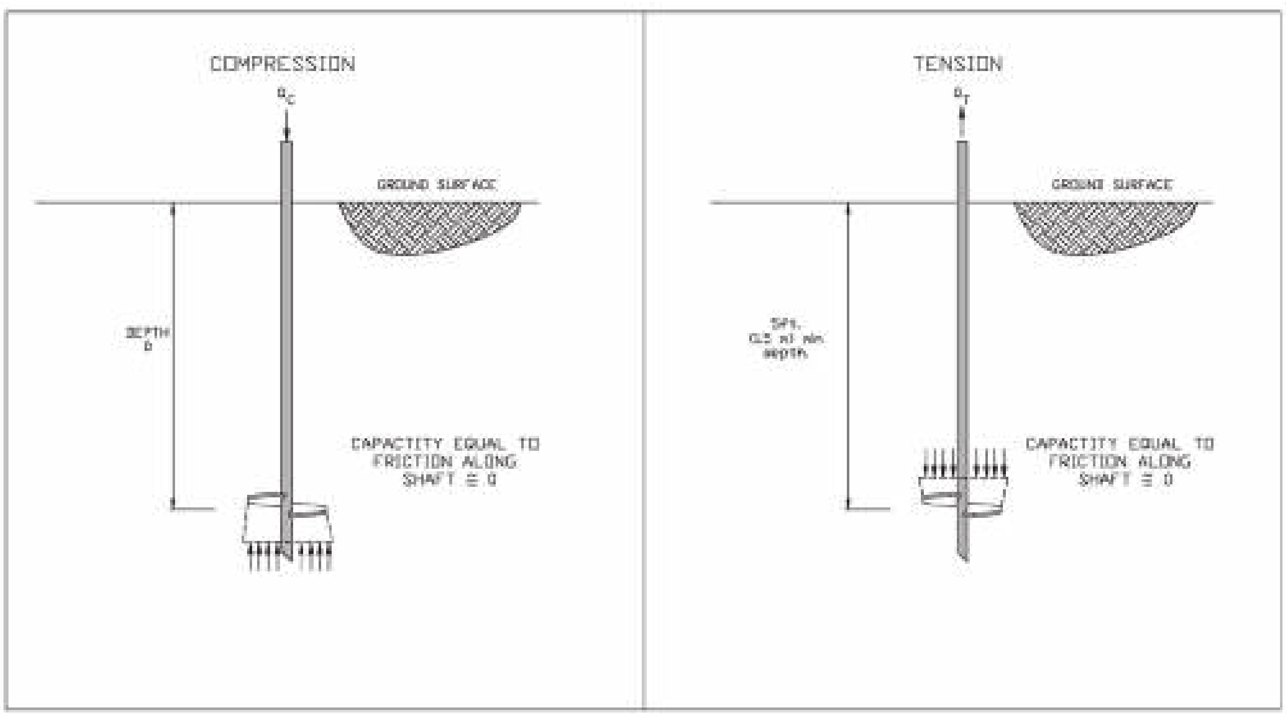


Figure 5-5 Single-Helix Plate Bearing Capacity Model – Helical Piles with Slender Shafts

5.2.2.1.a Saturated Clays φ' = 0; c' > 0

Under compression loading, the ultimate capacity of a single-helix screw-pile in clay is calculated from Equation 5-9 as:

$Q_H = A_H[(N_C)(s_u) + \gamma' D]]$

where:

N_C = Bearing Capacity Factor for Deep Failure = 9

Which gives:

$Q_H = A_H[(9)(s_u) + \gamma' D]$ **Equation 5-17**

5.2.2.1.b Sands φ' > 0; c' = 0

For clean, saturated sands, the "cohesion" is normally taken as zero, reducing the ultimate capacity, as in Equation 5-10, to:

$$Q_H = A_H(q'N_q + 0.5\gamma'BN\gamma)$$

Even in moist sands or sand with a small amount of fines that may give some "cohesion", this is usually ignored. Because the area of the plate is small, the contribution of the "width" term to ultimate capacity is also very small and the width term is often ignored leaving:

$$Q_H = A_H(q'N_q) \qquad \textbf{Equation 5-18}$$

For deep installations, the bearing capacity factor Nq is usually obtained from values used for determining the end bearing capacity for deep pile foundations, which is different than the values used for shallow foundations. There are a number of recommendations for Nq available in foundation engineering textbooks as shown in Figure 5-6. The difference in Nq values shown in Figure 5-6 is largely related to the assumptions used in the failure mechanism. Figure 5-7 gives a reasonable chart of Nq values as a function of the friction angle of the soil, φ', that may be used for screw-piles and helical anchors. The value of Nq in Figure 5-7 is obtained from:

$$Nq = 0.5\ (12 \times \varphi')^{\varphi'/54} \qquad \textbf{Equation 5-19}$$

Note: In some sands, the unit end bearing capacity of deep foundations may reach a limiting value. The point at which this occurs is generally termed the "critical depth". Critical depth is defined as the depth at which effective vertical stress, a.k.a. overburden pressure, will not increase with depth. Critical depth is not specifically defined for screw-piles and helical anchors, but engineers often use it with deep installation in saturated sands.

5.2.2.1.c Mixed Soils φ' > 0; c' > 0

The ultimate capacity of a deep single-helix screw-pile in mixed-grain soils can be taken from traditional bearing capacity theory using Equation 5-11:

$$Q_H = A_H(cN_c + q'N_q + 0.5\gamma BN\gamma)$$

Note: The term $0.5\gamma'BN\gamma$ is typically ignored for helical piles because the helix plate is small.

5,2.2.2 Tension Loading –Axial Uplift (Deep Single-Helix)

5.2.2.2.a Saturated Clays φ' = 0; c' > 0

Under tension loading, the ultimate capacity of a single-helix screw-anchor in clay the ultimate capacity is calculated using the same approach given in Section 5.2.2.1.a. In some cases a reduction may be made in the undrained shear strength to account for soil disturbance above the helical plate as a result of installation, depending on the Sensitivity of the clay. Also, as previously noted in Section 5.2.1.2.a, for a deep installation (D/B > 7.5) the Breakout Factor, F_C has a default value of 9. The bearing capacity equation becomes:

$$Q_{HU} = A_H[(9)s_u + \gamma'D]$$

5.2.2.2.b Sands φ' > 0; c' = 0

In sands, the tension capacity of a helical anchor is generally assumed to be equal to the compression capacity provided that the soil above the helix is the same as the soil below the helix in a zone of about 3 helix diameters. Again, for clean, saturated sands, the "cohesion" is normally taken as zero, reducing the ultimate capacity to:

$Q_H = A_H(q'N_q + 0.5\gamma'BN\gamma)$

Also, because the area of the plate is small, the contribution of the "width" term to ultimate capacity is also very small and the width term is often ignored leaving:

$Q_H = A_H(q'N_q)$

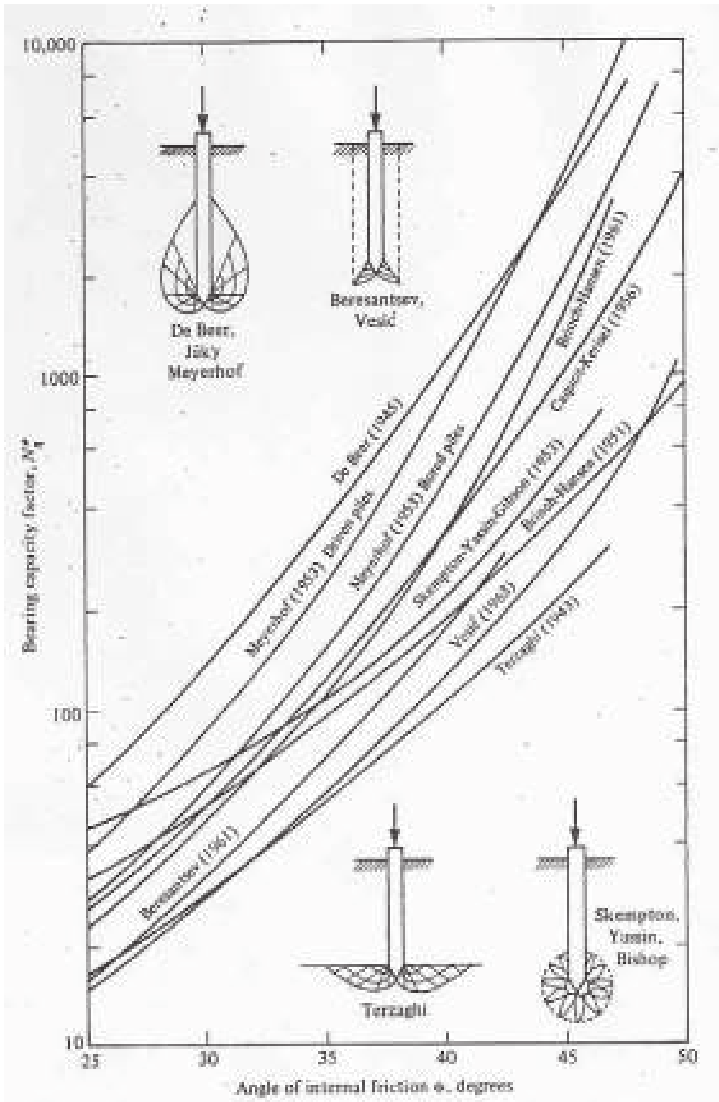


Fig. 19.49 Bearing capacity factors vs. angle of internal friction, according to various authors.

Figure 5-6 Reported Values of Nq for Deep Foundations in Sands [from Winterkorn & Fang (1983)].

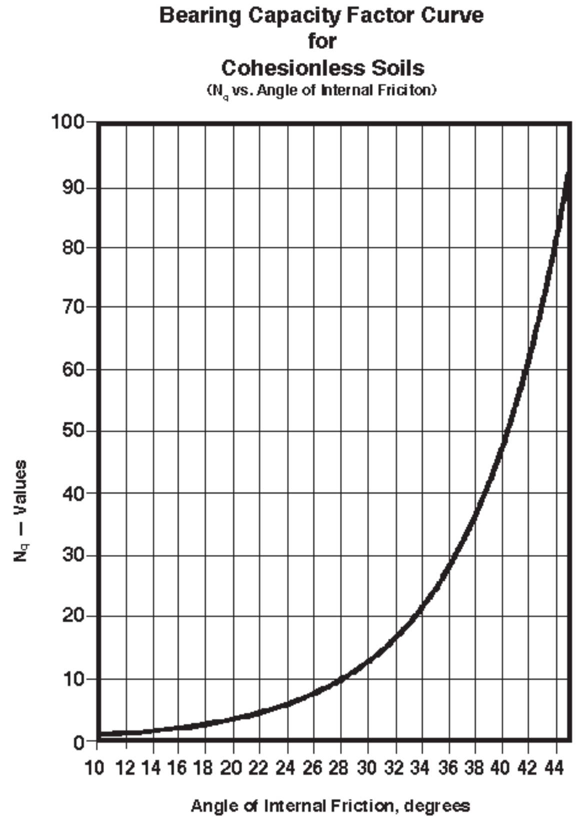


Figure 5-7 Recommended Bearing Capacity Factor Nq for Deep Screw-Piles and Helical Anchors in Sand.

5.2.2.2.c Mixed Soils φ' > 0; c' > 0

The ultimate capacity of a deep screw-pile in mixed-grain soils can be taken from traditional bearing capacity theory using Equation 5-11:

$$Q_H = A_H(cNc + q'Nq + 0.5\gamma BN\gamma)$$

Note: The term $0.5\gamma'BN\gamma$ is typically ignored for helical piles because the helix plate is small.

5.2.3 Multi-Helix Screw-Piles and Screw-Anchors – Deep Installation

The ultimate capacity of deep multi-helix screw-piles and screw-anchors depends on the geometry of the helical section, namely the size and number of helical plates and the spacing between the plates. As shown in Figure 5-3b and 5-3d, if the spacing of helix plates is close, the capacity is developed from a zone of failure between the helical plates and from end bearing from the end helix plate (either the lowest plate for compression loading or the top helix plate for tension loading), i.e., the helix plates interact with each other. If the spacing of the helix plates is sufficiently large, the capacity is taken as the sum of the capacity developed from the individual helix plates, i.e., there is no interaction between helix plates. Also, there is no capacity taken along the shaft between the helix plates.

In the U.S., most manufacturers of screw-piles and helical anchors produce elements with a standard helix spacing of 3 times the helix diameter. This spacing was originally used by CHANCE® over 30 years ago and is assumed to allow individual helix plates to develop full capacity with no interaction between helix plates and the total capacity is taken as the sum of the capacities from each plate as shown in Figure 5-3a and 5-3c. Most CHANCE® Screw-Piles and Helical Anchors use inter-helix spacing that is based on the diameter of the lower helix. For example, the distance between a 10 inch (254 mm) and a 12 inch (305 mm) helix is three times the diameter of the lower helix, or 10 x 3 = 30 inches (762 mm).

The first section, called the lead or starter, contains the helix plates. This lead section can consist of a single helix or multi-helices, typically up to four. Additional helix plates can be added, if required, with the use of helical extensions. Standard helix sizes and projected areas are shown in Table 5-4. Comprehensive tables of helix projected areas, showing both the full plate area and the area less the shaft for both square shaft and pipe shaft helical piles, is included in Section 7 of this Manual. The helix plates are usually arranged on the shaft such that their diameters stay the same size or increase as they get farther from the pilot point (tip). The practical limits on the number of helix plates per anchor/pile is usually four to five if placed in a fine-grained soils and six if placed in a coarse-grained or granular soils.

5.2.3.1 Compression Loading

The ultimate capacity of a multi-helix screw-pile with an inter-helix spacing greater than or equal to 3 (s/B ≥3) is generally taken as the summation of the capacities of the individual plates:

Table 5-4 Standard Helix Sizes

LEAD SECTION AND EXTENSIONS	
DIAMETER in (cm)	**AREA ft^2 (m^2)**
6 (15)	0.185 (0.0172)
8 (20)	0.336 (0.0312)
10 (25)	0.531 (0.0493)
12 (30)	0.771 (0.0716)
14 (35)	1.049 (0.0974)
16 (40)	1.385 (0.1286)

$Q_M = \sum Q_H$ **Equation 5-20**

where:

Q_M = Total Capacity of a Multi-Helix Screw-Pile/Helical Anchor
Q_H = Capacity of an Individual Helix

5.2.3.2 Tension Loading

As previously noted in soft clays, especially those with high Sensitivity, it may be appropriate to reduce the undrained shear strength of the undisturbed clay for design of anchors in tension to account for some disturbance of the clay as the helical plates have passed through. This is left to the discretion of the Engineer. Most of the evidence shows that in uniform soils, the tension capacity of multi-helix anchors is the same as in compression. This means that the ultimate capacity of a multi-helix helical anchor with plate spacing of 3B or more may be taken as the summation of the capacities of the individual plates:

$$Q_M = \sum Q_H$$

There is some evidence that shows that in tension the unit capacity of the trailing helix plates is somewhat less than the leading helix. Engineers may wish to apply a reduction factor to account for this behavior; of about 10% for each additional helix on the helical anchor.

5.2.4. Round Shaft Screw-Piles and Helical Anchors

Screw-piles and helical anchors are available with both square shaft and round steel pipe shafts. Square shaft is used for tension applications and also for compression applications when shaft buckling or bracing is not an issue. Pipe shaft helical piles have become increasingly popular for use in compression loading for both new construction and remediation or underpinning of existing structures. They may be either single or multi-helix. Typical round shaft pile diameters range from 2-7/8 inches (73 mm) to 12 inches (305 mm). For the most part, the design is essentially the same as with square shaft screw-piles as previously described with two simple modifications: 1) some provision is usually made to include the additional load capacity developed via skin friction by the round shaft; and 2) in tension loading, the area of the helical plate is reduced to account for the central shaft as shown in Figure 5-11b. In compression loading, the full projected area of the helix plate develops capacity since the pipe generally plugs with soil.

Typically, the length of the shaft for about one helix diameter above the helix is not included in calculating shaft resistance due to skin friction. In addition, load capacity due to friction along the pile shaft is generally mobilized only if the shaft diameter is at least 3 inches (89 mm).

5.2.4.1 Shaft Resistance in Clay $\varphi' = 0$; $c' > 0$

In clays, the shaft resistance developed by round shaft screw-piles and helical anchors is considered in much the same way that shaft resistance in a driven pile develops. In this traditional approach that is used for many driven piles in clays and available in most textbooks, the available "adhesion" between the shaft and the clay is obtained as a percentage of the undrained shear strength of the clay. This is the undrained or "Alpha" method in which:

$\alpha = f_S/s_u$ **Equation 5-21**

where:

α = Adhesion Factor
f_S = Unit Side Resistance
s_u = Undrained Shear Strength of the Clay

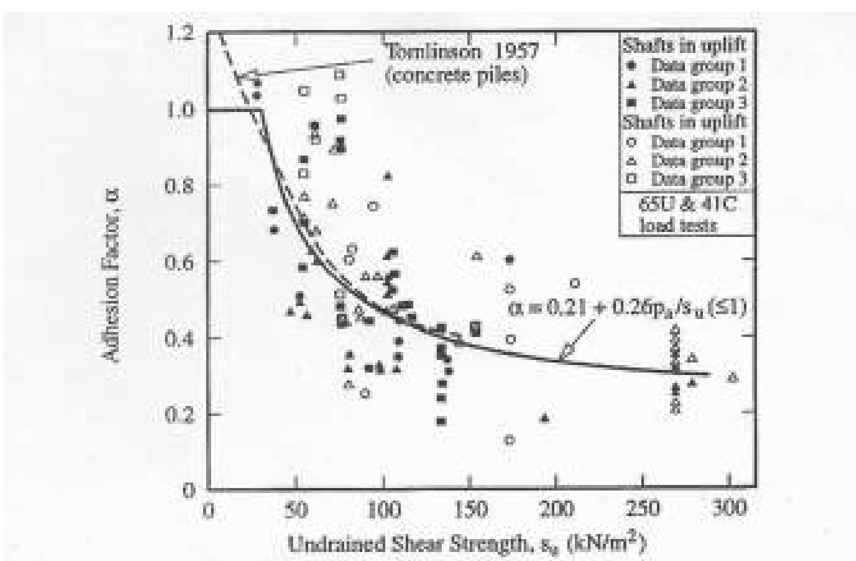


FIGURE 18.1 *Adhesion as a function of undrained shear strength*

Figure 5-8 Variation in Adhesion Factor with Undrained Shear Strength of Clays [from Canadian Foundation Manual (2006)].

The value of α is usually obtained from any one of a number of published charts and is typically related to the absolute value of the undrained shear strength of the clay. Figures 5-8 and 5-9 give typical plots of α vs. undrained shear strength for a number of cases in which f_S has been back calculated from actual pile load tests. Generally it is sufficient to select an average value of α for a given undrained shear strength for use in design.

The total shaft resistance is then obtained from:

$Q_S = (f_S)(\pi)(d)(L)$ **Equation 5-22**

where:

Q_S = Total Shaft Resistance
d = Diameter of Central Shaft
L = Length of Round Shaft in Contact with Soil

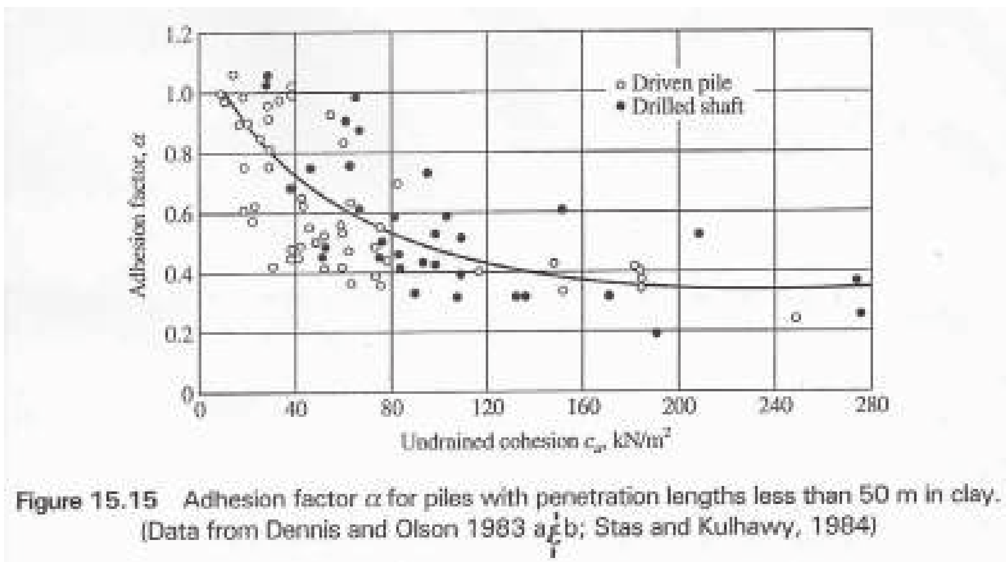


Figure 5-9 Variation in Adhesion Factor with Undrained Shear Strength of Clays (from Murthy 2003).

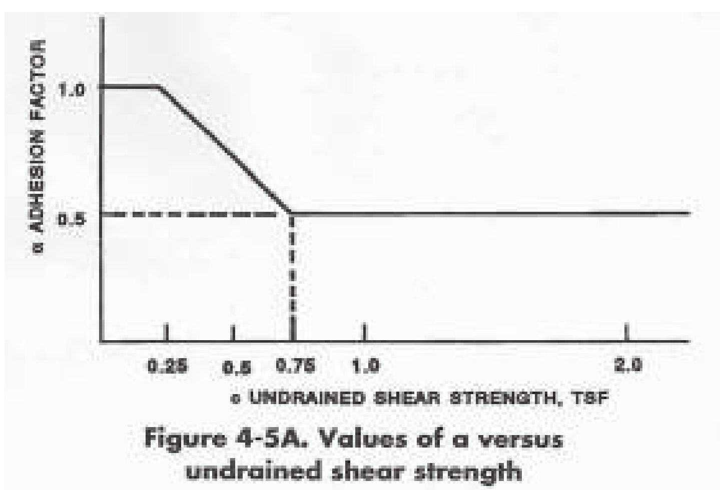
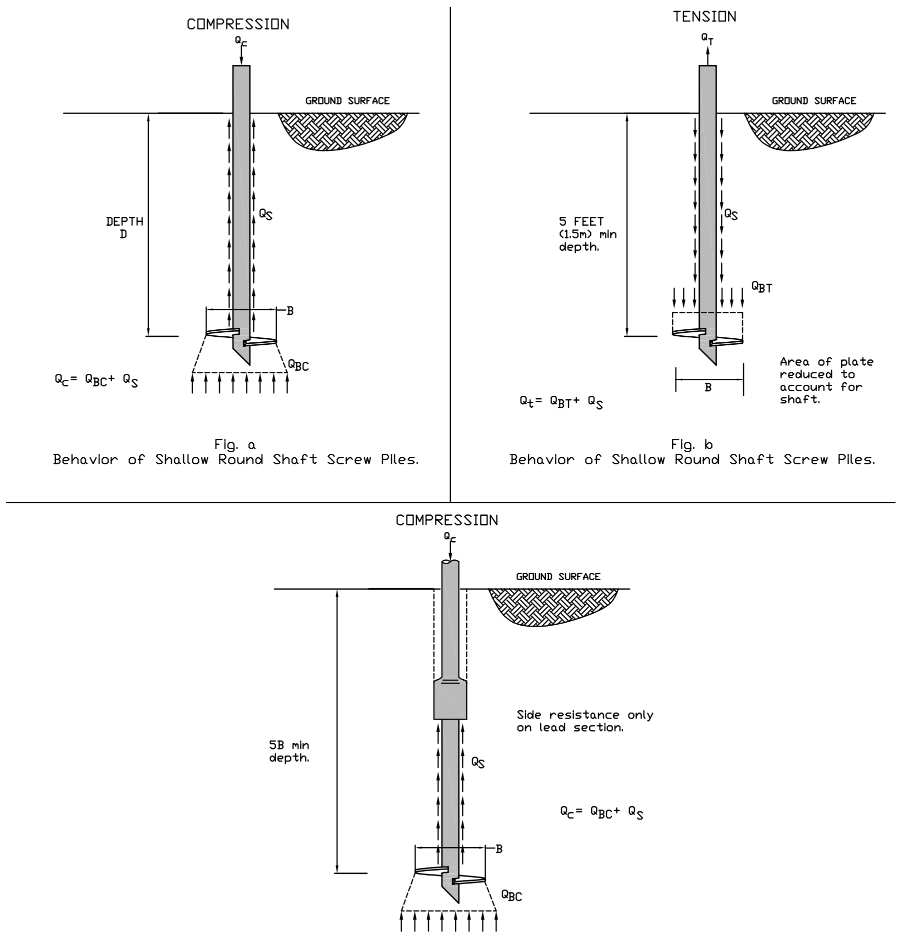


Figure 5-10 Variation in Adhesion Factor from American Petroleum Institute [from ASCE (1993b)].



Individual Bearing and Skin Friction Models for Helical Piles with Round (Pipe) Shafts
Figure 5-11

The design line given by the American Petroleum Institute (API) shown Figure 5-10 may also be used in which:

For $s_u < 500$ psf; $\alpha = 1.0$

For $s_u > 1500$ psf; $\alpha = 0.5$

For 500 psf < su < 1500 psf; α varies linearly between 1.0 and 0.5

The shaft resistance should only be calculated for that portion of the shaft length that is in full contact with the soil. This will depend on the length of the lead section, the design of the shaft couplings that connect the pile sections, and the type of soil. For example, flanged and bolted connections generally create an annulus between the shaft and the soil as the pile or anchor is installed as shown in Figure 5-11. This is because the coupling, being larger than the shaft, displaces and compacts soil. Generally, the length of the central shaft between couplings is not considered to develop shaft resistance unless the disturbed soil moves back against the shaft, or sufficient time is allowed for the soil to recover. In this situation, reduced shear strength should be used for shaft resistance capacity.

On the other hand, in the case of true flush connections between extension sections, the entire shaft may develop side resistance.

Table 5-5 Values of Unit Side Resistance for Steel Piles in Sand (from Navy Manual DM-7)

σ'_{vo} (psf)	Friction Angle of Soil φ'				
	20	25	30	35	40
	Unit Side Resistance f_S (psf)				
500	137	175	217	263	315
1000	273	350	433	525	629
1500	410	524	650	788	944
2000	546	700	866	1050	1259
2500	683	875	1082	1313	1574
3000	819	1049	1300	1575	1888
3500	956	1244	1516	1838	2203
4000	1092	1399	1732	2101	2517

5.2.4.2 Shaft Resistance in Sand and Mixed Soils $\varphi' > 0$; $c' = 0$

The shaft resistance of steel pipe shaft piles in coarse-grained soils, such as sands and mixed soils is more complex than in clays but can still be determined using traditional deep foundation analyses. The Department of Navy Design Manual DM-7 also gives a simplified method for estimating the unit side resistance for straight shaft steel piles. The value of f_S is related to the friction angle of the soil, φ', and the effective vertical stress, σ'_{vo}, as given in Table 5-5.

5.2.5 HELICAL ANCHOR/PILE SPACING & MINIMUM DEPTH

Reasonability Check

Consideration should be given to the validity of the values obtained when determining the bearing capacity and shaft resistance of the soil. The calculated theoretical ultimate capacity is no better than the data used to obtain that value. Data from soils reports, boring logs, the water table depth, and load information may not accurately represent actual conditions where the helical pile/anchor must function. Empirical values that are used and estimates of strength parameters, etc. that must be made because of lack of data affect the calculated bearing capacity and shaft resistance value. In those situations where soil data is insufficient or not available, a helical trial probe pile can help determine such items as, location of bearing strata, pile capacity, location of soft/loose soil, and the presence of obstructions, such as, cobbles, boulders, and debris.

An important step in the process of determining the capacity of a helical pile/anchor is to conduct a reasonability check. The engineer should use the best engineering judgment to perform the reasonability check. This should be based on experience, historical test data and consulting colleagues. This is easily overlooked but must be performed by the designer or by others.

Helical Pile/Anchor Spacing

Once the capacity of the helical pile/anchor is determined, concern may turn to location of the foundation element with respect to the structure and to other helical pile/anchors. It is recommended that the center-to-center spacing between adjacent anchors/piles be no less than five times the diameter of the largest helix. The minimum spacing is three feet (0.91 m). This latter spacing should be used only when the job can be accomplished no other way and should involve special care during installation to ensure that the spacing does not decrease with depth. Minimum spacing requirements apply only to the helix bearing plate(s), i.e., the pile/anchor shaft can be battered to achieve minimum spacing. Spacing between the helical anchors/piles and other foundation elements, either existing or future, requires special consideration and is beyond the scope of this section.

Group effect, or the reduction of capacity due to close spacing, has never been accurately measured with helical piles. However, bearing capacity theory would indicate that capacity reduction due to group effect is possible, so it's considered good practice to install helical piles into dense bearing stratum when center-to center spacing is less than 4 feet (1.2 m).

Minimum Depth

As mentioned earlier, the minimum embedment depth recommended by Hubbell Power Systems, Inc. for a helical deep foundation is five helix diameters (5D), where D is the diameter of the largest helix. The 5D depth is the vertical distance from the surface to the top-most helix. Standard practice is to locate the top-most helix 6D to 8D vertical below the ground surface where practical. Minimum depth is also a function of other factors, such as seasonally frozen ground, "active" zones (depth of wetting) and depth of compressive soils. These factors are generally related to seasonal variations to soil strength parameters, but can also be related to long-term conditions, such as periods of drought or extended wet conditions. The minimum embedment depth recommended by Hubbell Power Systems, Inc. for a helical deep foundation due to seasonal variations is three diameters (3D) below the depth of soil where these seasonal variations will occur. For example, frost depths may require embedment depths that exceed the 5D minimum, depending on the project location. ICC-ES Acceptance Criteria AC358 has specified a minimum depth for helical tension anchors. AC358 states that for tension applications, as a minimum, the helical anchor must be installed such that the minimum depth from the ground surface to the uppermost helix is 12D, where D is the diameter of the largest helix. This disparity between minimum depth requirements can be reconciled by reviewing published literature on the subject, or by performing load tests.

Critical Depth

In granular soils, helical pile capacity is a function of both angle of internal friction (φ) and vertical effective overburden stress. Therefore, as a helical pile is extended deeper into soil, theoretical methods predict that the pile capacity would increase without limit as the effective vertical stress increases with increasing depth. In reality, there may be a critical depth where any further increase in depth results in only a small increase in the bearing capacity of the helical pile. Critical depth for helical piles is best determined by an experienced foundation engineer. Hubbell Power Systems, Inc. recommends critical depths of 20D to 30D be used in loose saturated soils at deep depth, where D is the diameter of the largest helix plate. The 20D to 30D length is the depth into a suitable bearing stratum, and is not necessarily measured from the ground surface.

Table 5-6 Soil Properties Required for Helical Pile/Anchor/Pile Design for Various Site Conditions

	Required Soil Properties		
Soil Property Category	**Saturated Fine-Grained**	**Coarse-Grained**	**Unsaturated Fine-Grained, Mixed Soils**
Shear Strength	s_u	φ'	c', φ'
Unit Weight	γ_{sat}	γ_{wet} or γ_{buoy}	γ_{wet}

5.3 EVALUATING SOIL PROPERTIES FOR DESIGN

The design of helical piles/anchors using the traditional soil mechanics approach described in the previous section requires evaluation of soil properties for input into the various bearing and friction capacity equations. Table 5-6 summarizes the soil properties for different site conditions for design of both single-helix and multi-helix helical piles/anchors.

Geotechnical design of helical piles/anchors requires information on the shear strength of saturated fine-grained soils, i.e., undrained shear strength, s_u, and the drained friction angle of coarse-grained soils, φ'. The best approach to evaluating these properties for design is a thorough site investigation and laboratory testing program on high quality undisturbed samples. However, this is not always possible or practical and engineers often rely on information obtained from field testing, such as the Standard Penetration Test (SPT). Whenever possible, other high quality field tests, such as Field Vane Tests (FVT), Cone Penetration Tests (CPT), Piezocone Tests (CPTU), Dilatometer Tests (DMT), Pressuremeter Tests (PMT) or Borehole Shear Tests (BST) are preferred. THERE IS NO SUBSTITUTE FOR A SITE SPECIFIC GEOTECHNICAL INVESTIGATION.

Estimating Undrained Shear Strength, s_u, in clays:

The undrained shear strength of saturated clays, silty clays and clayey silts is not a unique soil property, like Liquid Limit of clay content, but depends on the test method used for the measurement. Correlations are available for estimating undrained shear strength from the results obtained from several of the field tests noted above. The most common field results that may be available to engineers for design of helical piles/anchors are the SPT and CPT/CPTU.

s_u from SPT

A number of correlations exist for estimating both the undrained shear strength and unconfined compressive strength, q_u, of fine-grained soils from SPT results. Several of these correlations are given in Tables 5-7 and 5-8. The undrained shear strength is generally taken as one-half the unconfined compressive strength. Caution should be used when using these correlations since they have been developed for specific geologic deposits and the SPT field procedure used may not have been the same in all cases.

s_u from CPT/CPTU

The undrained shear strength may also be estimated from the tip resistance obtained from the total cone tip resistance from a CPT or the effective (net) cone tip resistance from a CPTU (e.g., Lunne et al. 1995).

Estimating s_u from the CPT total tip resistance is from a form of the bearing capacity equation as:

$s_u = (q_c - \sigma_{vo})/N_k$ **Equation 5-23**

where:

q_c = CPT tip resistance
σ_{vo} = total vertical stress at the cone tip = depth x total soil unit weight
N_k= empirical cone factor

The value of N_k varies somewhat with soil stiffness, plasticity, stress history and other factors, however many reported observations where s_u has been obtained from both laboratory triaxial tests and field vane tests suggest that a reasonable value of N_k for a wide range of soils is on the order of 16.

Estimating s_u from the CPTU effective tip resistance uses a modified approach since the tip resistance is corrected for pore pressure effects to give the effective tip resistance, q_t, as the undrained shear strength is obtained from:

$s_u = (q_t - \sigma_{vo})/N_{kt}$ **Equation 5-24**

where:

q_t = CPTU effective tip resistance
N_{kt}= empirical cone factor

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Table 5-7. Reported Correlations Between SPT N-Value and Undrained Shear Strength, s_u

Correlation to Undrained Shear Strength	Units of s_u	Soil Type	Reference
$s_u = 29N^{0.72}$	kPa	Japanese cohesive soils	Hara et al. (1974)
$s_u = 4.5N$	tsf	Insensitive Overconsolidated Clays in U.K.	Stroud (1974)
$s_u = 8N$ $N < 10$ $s_u = 7N$ $10 < N < 20$ $s_u = 6N$ $20 < N < 30$ $s_u = 5N$ $30 < N < 40$	kPa	Guabirotuba Clay	Tavares (1988)
$s_u = 1.39N + 74.2$	tsf	tropical soil	Ajayi & Balogun (1988)
$s_u = 12.5N$ $s_u = 10.5N_{60}$	kPa tsf	Sao Paulo overconsolidated clay	Decourt (1989)

Note: 1 kPa = 20.9 psf

Table 5-8. Reported Correlations Between SPT N-Value and Unconfined Compressive Strength, q_u

Correlation to Unconfined Compressive Strength	Units of q_u	Soil Type	Reference
$q_u = 12.5N$	kPa	Fine-Grained	Terzaghi & Peck (1967)
$q_u = N/8$	tsf	Clay	Golder (1961)
$q_u = 25N$ $q_u = 20N$	kPa kPa	Clay Silty Clay	Sanglerat (1972)
$q_u = 25N$ $q_u = 15N$ $q_u = 7.5N$	kPa	Highly Plastic Clay Medium Plastic Clay Low Plasticity Clay	Sowers (1979)
$q_u = 24N$	kPa	Clay	Nixon (1982)
$q_u = 62.5 (N-3.4)$	kPa		Sarac & Popovic (1982)
$q_u = 15N$	kPa	CL and CL-ML	Behpoor & Ghahramani (1989)
$q_u = 58N^{0.72}$	kPa	Fine-Grained	Kulhawy & Mayne (1990)
$q_u = 13.6 N_{60}$ $q_u = 9.8N_{60}$ $q_u = 8.6N_{60}$ $q_u = (0.19PI + 6.2)N_{60}$	kPa	CH CL Fine-Grained Fine-Grained	Sivrikaya & Togrol (2002)

The value of N_{kt} also has been shown to vary for different soils but a reasonable conservative value for massive clays is on the order of 12. For very stiff, fissured clays, the value of N_{kt} may be as high as 30.

Other methods are available for estimating undrained shear strength from the pore pressure measurements from a CPTU or by first estimating the stress history from CPT/CPTU results and then converting to undrained shear strength, e.g., NCHRP (2007); Schnaid (2009), both of which are viable approaches.

Estimating Shear Strength of Fine-Grained Soil – Other Methods

Vane Shear Test: Shear strength of fine-grained soils may be measured both in the field and in the laboratory. One of the most versatile devices for investigating undrained shear strength and sensitivity of soft clays is the vane shear test. It generally consists of a four-bladed rectangular vane fastened to the bottom of a vertical rod. The blades are pressed their full depth into the clay surface and then rotated at a constant rate by a crank handle. The torque required to rotate the vane is measured. The shear resistance of the soil can be computed from the torque and dimensions of the vane.

One such type of the portable vane shear test is the Torvane. It is a convenient hand-held device useful for investigating the strength of clays in the walls of test pits in the field or for rapid scanning of the strength of Shelby tubes or split spoon samples. A calibrated spring allows undrained shear strength (cohesion) to be read directly from the indicator.

Pocket Penetrometer Test: Another device used to estimate undrained shear strength in the laboratory or the field is the Pocket Penetrometer. As with the vane shear test, the pocket penetrometer is commonly used on Shelby tube and split spoon samples, and freshly cut test pits to evaluate the consistency and approximate unconfined compressive strength (q_u) of clay soils. The penetrometer's plunger is pushed into the soil ¼" and a reading taken on the sliding scale on the side. The scale is a direct reading of shear strength. Pocket Penetrometer values should be used with caution. It is not recommended for use in sands or gravel soils.

Unconfined Compression Test: The unconfined compression (UC) test is used to determine the consistency of saturated clays and other cohesive soils. A cylindrical specimen is set up between end plates. A vertical load is applied incrementally at such a rate as to produce a vertical strain of about 1 to 2% per minute – which is rapid enough to prevent a volume change in the sample due to drainage. The unconfined compressive strength (q_u) is considered to be equal to the load at which failure occurs divided by the cross-sectional area of the sample at the time of failure. In clay soils where undrained conditions are expected to be the lower design limit (i.e. the minimum Factor of Safety), the undrained shear strength (i.e., cohesion) governs the behavior of the clay. This undrained shear strength is approximately equal to ½ the unconfined compressive strength of undisturbed samples (see Laboratory Testing of Recovered Soil Samples in Section 2 of this Technical Manual).

The consistency of clays and other cohesive soils is usually described as soft, medium, stiff, or hard. Tables 5-9 and 5-10 can be found in various textbooks and are reproduced from Bowles, 1988. Values of consistency, overconsolidation ratio (OCR), and undrained shear strength (cohesion) empirically correlated to SPT N-values per ASTM D 1586 are given in Table 5-9. It should be noted that consistency correlations can be misleading because of the many variables inherent in the sampling method and the soil deposits sampled. As such, Table 5-9 should be used as a guide.

The relative density of sands, gravels, and other granular soils is usually described as very loose, loose, medium dense, dense, very dense, or extremely dense. The standard penetration test is a good measure of granular soil density. Empirical values for relative density, friction angle and unit weight as correlated to SPT "N" values per ASTM D 1586 are given in Table 5-10. It should be noted that SPT values can be amplified in gravel because a 1"+ gravel particle may get lodged in the opening of the sampler. This can be checked by noting the length of sample recovery on the soil boring log (see Table 2-6). A short recovery in gravelly soils may indicate a plugged sampler. A short or "low" recovery may also be indicated by loose sand that falls out of the bottom of the sampler during removal from the borehole.

Estimating Friction Angle, φ', in sands

Results from both the SPT and CPT may be used to estimate the drained friction angle of sands and other coarse-grained soils. Generally, most site investigations involving coarse-grained soils will include the use of either the Standard Penetration Test (SPT) or the Cone Penetrometer (CPT).

φ' from SPT

Several correlations have been proposed to estimate the drained friction angle in sands from SPT results. A summary of several of the more popular correlations are given in Table 5-11. The correlation of Hatanaka & Uchida (1996) is shown in Figure 5-12, taken from FHWA Reference Manual on Subsurface Investigations (2002).

Table 5-9. Terms to Describe Consistency of Saturated Cohesive Soils

Consistency Term	Stress History	SPT N_{60}-Values	Undrained Shear Strength skf (kPa)	Comments
Very Soft	Normally Consolidated OCR = 1	0 - 2	<0.25 (12)	Runs through fingers.
Soft	Normally Consolidated OCR ≅ 1 – 1.2	3 - 5	0.38 (18.2) to 0.63 (30.2)	Squeezes easily in fingers.
Medium	Normally Consolidated OCR = 1 to 2	6 - 9	0.75 (36) to 1.13 (54.1)	Can be formed into a ball.
Stiff	Normally Consolidated to OCR of 2-3.	10 - 16	1.25 (59.9) to 2 (95.8)	Hard to deform by hand squeezing.
Very Stiff	Overconsolidated OCR = 4 – 8	17 - 30	2.13 (102) to 3.75 (179.6)	Very hard to deform by hand.
Hard	Highly Overconsolidated OCR > 8	>30	>3.75 (179.6)	Nearly impossible to deform by hand.

φ' from CPT/CPTU

A similar approach may be used to estimate the friction angle of sands from the CPT/CPTU tip resistance based on a modified bearing capacity theory. Robertson and Campanella (1983) summarized a number of available calibration chamber tests on five sands and suggested a simple correlation between the normalized CPT tip resistance and a cone bearing capacity factor, N_q as:

$$N_q = (q_c/\sigma_{v0}') = 0.194\exp(7.63\tan\varphi') \qquad \textbf{Equation 5-26}$$

where:

σ_{v0}' = vertical effective (corrected for pore water pressure) stress at cone tip

This relationship is shown in Figure 5-14.

The friction angle may also be estimated from the effective tip resistance from the CPTU. Early calibration chamber data suggested a simple empirical correlation as:

$$\varphi' = \arctan[0.1 + 0.38 \log (q_t/\sigma'_{vo})] \qquad \textbf{Equation 5-27}$$

Equation 5-27 is shown in Figure 5-16.

Table 5-10. Empirical Values for Dr, Friction Angle and Unit Weight vs SPT (Assuming a 20 ft (6 m) depth of overburden and 70% rod efficiency on hammer)

Description		Very Loose	Loose	Medium	Dense	Very Dense
Relative Density (D_r) (%)		0	15	35	65	85
SPT (N_{70})	Fine	1-2	3-6	7-15	16-30	?
	Medium	2-3	4-6	8-20	21-40	40+
	Coarse	3-6	5-9	10-25	26-45	45+
Friction Angle (ϕ)	Fine	26-28	28-30	30-33	33-38	38+
	Medium	27-29	29-32	32-36	36-42	50+
	Coarse	28-30	30-34	34-40	40-50	50+
Total Unit Weight (g_{wet}) (PCF)		70-100	90-115	110-130	110-140	130-150

Additional test results from 24 different sands were compiled by Kulhawy and Mayne (1990) who proposed the following expression:

$\varphi' = 17.70 + 11.0 \log (q_{t1})$ **Equation 5-28**

where:

$(q_{t1}) = (q_t/\sigma_{atm})/(\sigma'_{vo}/\sigma_{atm})^{0.5}$

σ_{atm} = atmospheric pressure (1 atm = 1 bar = 100 kPa = 1tsf = 14.7 psi)

Table 5-11. Reported Correlations between SPT N-Value and φ' for Coarse-Grained Soils

Correlation	Reference
$\varphi' = (0.3N)^{0.5} + 27^0$	Peck et al. (1953)
$\varphi' = (10N)/35 + 27^0$	Meyerhof (1956)
$\varphi' = (20N)^{0.5} + 15^0$	Kishida (1967)
$\varphi' = (N/\sigma'_{vo})^{0.5} + 26.9^0$ (σ'_{vo} in MN/m^2)	Parry (1977)
$\varphi' = (15N)^{0.5} + 15^0$	Shioi & Fukui (1982)
$\varphi' = (15.4(N_1)_{60})^{0.5} + 20^0$	Hatanaka & Uchida (1996)

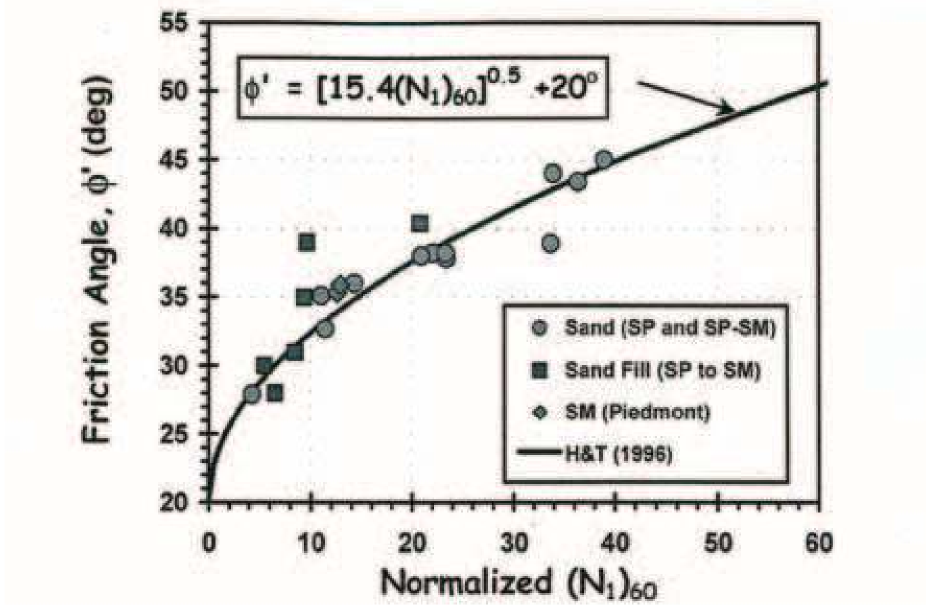
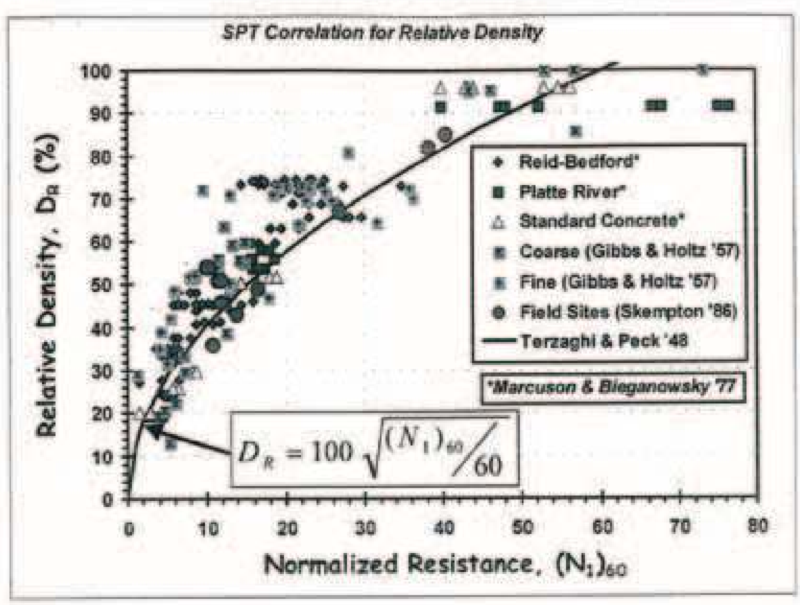


Figure 5-12 Peak Friction Angle of Sands from SPT Resistance - Correlation of Hatanaka & Uchida (1996) from FHWA Reference Manual on Subsurface Investigations (2002)



Direct Estimate of Unit Shaft Resistance, f_s, of Steel Round Shaft Piles and Grouted Helical Micropiles

Suggestions for estimating the unit side resistance, f_s, of deep foundations in a variety of soils have been presented. This approach is convenient for helical piles/anchors and reduces assumptions in first estimating shear strength and then estimating other factors to obtain f_s. Poulos (1989) summarized a number of reported correlations between pile unit side resistance and SPT N-value and suggested that most of these correlations could be expressed using the general equation:

$$f_s = \beta + \alpha N \qquad \textbf{Equation 5-29}$$

Lutenegger (2011) presented a summary of more-or-less "global" reported correlations between SPT N-values and unit side resistance friction for both driven and bored piles in a number of different soil materials and shown in Table 5-12.

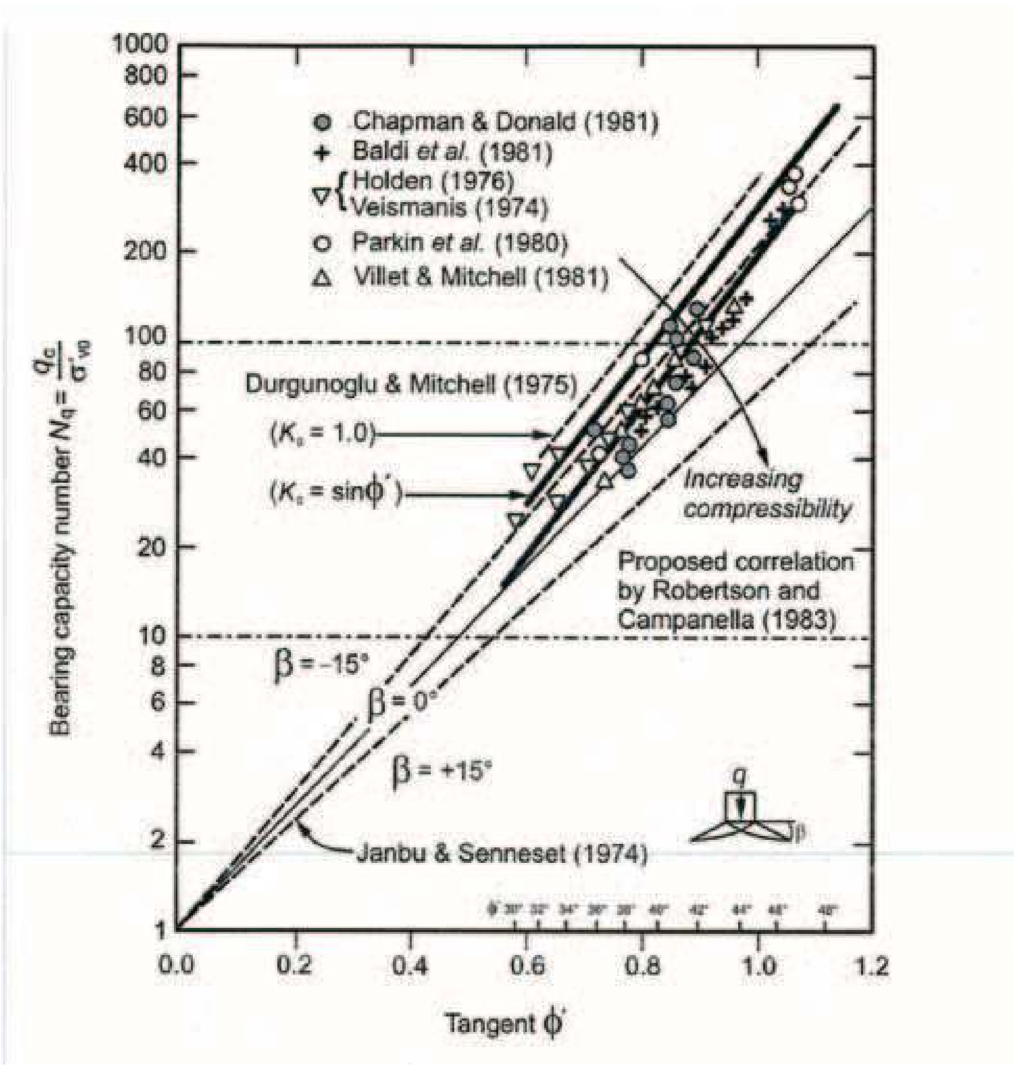


Figure 5-14. Relationship between Bearing Capacity Number and Friction Angle from Normalized CPT Tip Resistance – from Robertson and Campanella (1983)

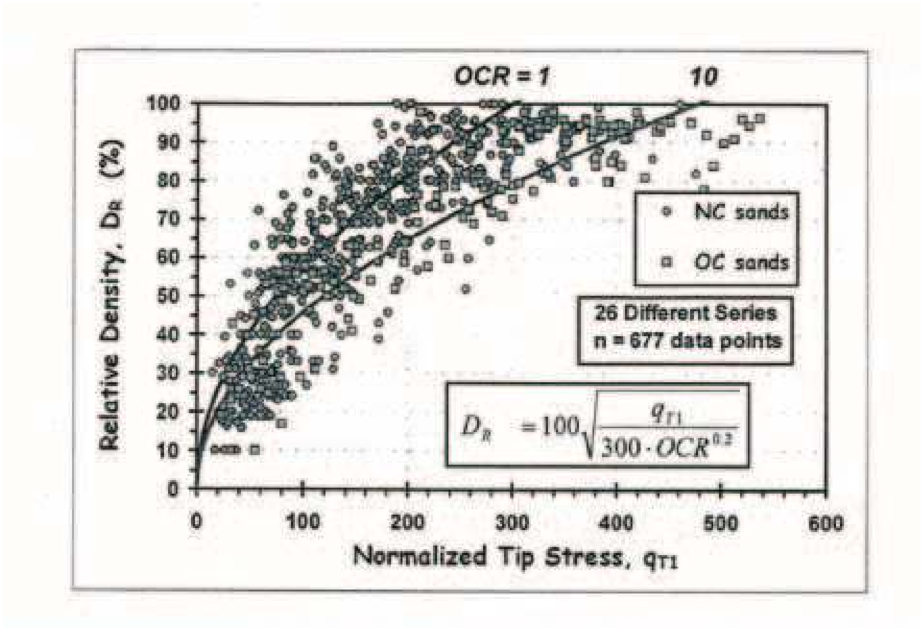


Figure 5-15. Relationship Between Relative Density for Normally Consolidated (NC) and Over Consolidated (OC) Sands from CPT Data.

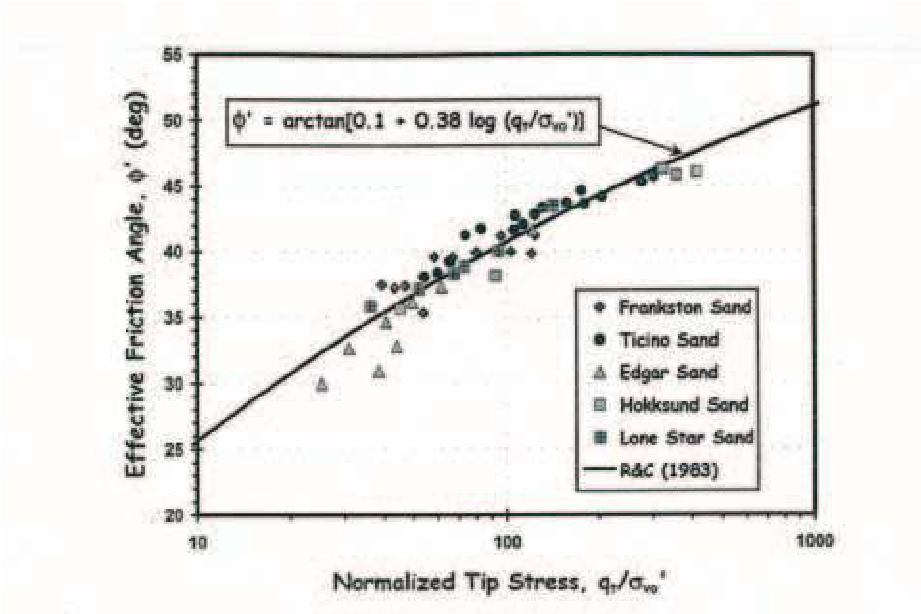


Figure 5-16. Relationship Between Friction Angle and the Effective Tip Resistance from CPTU Data

$(N_1)_{60} = N_{60}/(\sigma'_{vo})^{0.5}$ σ'_{vo} = effective overburden stress in tsf

Engineers might ask "Why should the SPT N-value correlate to pile side resistance?" Other than being purely coincidental, there must be a rational and logical explanation for such observations. The range in reported values of α given in Table 5-12 is quite large and the results might seem of limited use. Nonetheless, we can make some general observations and summarize these observations: 1) For most of these correlations, the value of β is very low and for practical purposes may be reasonably taken as zero with little effect on the correlation, which simplifies Eq. 5-29 to:

$$fs = \alpha N \quad \textbf{Equation 5-30}$$

2) The value of α ranges from 0.3 to 12.5; 3) The observations presented in Table 5-12 generally suggest higher values of α for fine-grained soils as compared to coarse-grained soils; and 4) Values of α are generally higher for driven piles as compared to bored piles.

The values of α vary considerably for a number of obvious reasons, deriving from both the pile data as well as the SPT data. In regard to the pile data: 1) The data represent a wide range of pile types, i.e., different geometry, such as open and closed end pipe, H-Piles and construction practices; such as dry bored vs. wet bored as well as pile size, pile plugging, L/d, and other factors; 2) Different methods may have been used to interpret the ultimate capacity and to isolate the side resistance from end bearing; 3) The unit side resistance from pile tests is typically averaged over the length of the pile except in the case of well instrumented piles. Regarding the SPT data:1) The results most likely represent a wide range in field practice including a wide range in energy or hammer efficiency; 2) It is likely that other variations in field practice or equipment such as spoon geometry are not consistent among the various studies and may affect results. Engineers should use the correlations in Table 5-12 with caution.

In fact, Equation 5-30 is similar to Equation 5-21, suggesting a correlation between SPT N-values and undrained shear strength (s_u) in fine-grained soils.

5.4 FACTOR of SAFETY

The equations discussed above are used to obtain the ultimate capacity of a helical anchor/pile. For working, or allowable stress design (ASD), an appropriate Factor of Safety must be applied to reduce the ultimate capacity to an acceptable design (or working) capacity. The designer determines the Factor of Safety to be used. In general, a minimum Factor of Safety of 2 is recommended. For tieback applications, the Factor of Safety typically ranges between 1.25 and 2.

Design or working loads are sometimes referred to as unfactored loads and do not include any Factor of Safety. They may arise from dead loads, live loads, snow loads and/or earthquake loads for bearing (compression) loading conditions; from dead loads, live loads, snow loads and/or wind loads for anchor loading conditions; and earth pressure, water pressure and surcharge loads (from buildings, etc.) for helical tieback or SOIL SCREW® earth retention conditions.

Ultimate loads, sometimes referred to as fully factored loads, already fully incorporate a Factor of Safety for the loading conditions described above. Hubbell Power Systems, Inc. recommends a minimum Factor of Safety of 2.0 for permanent loading conditions and 1.5 for temporary loading conditions. This Factor of Safety is applied to the

Table 5-12. Reported Correlations between SPT N-Value and Pile Side Resistance (from Lutenegger 2011)

Pile Type	Soil	β	α	Reference
driven displacement	granular	0	2.0	Meyerhof (1976)
	miscellaneous soils .(f_s < 170 kPa)	10	3.3	Decourt (1982)
	cohesive	0	10	Shioi & Fukui (1982)
	cohesive cohesionless	0 0	3 1.8	Bazaraa & Kurkur (1986)
	sandy clayey	29 34	2.0 4.0	Kanai & Yubuuchi (1989)
	misc	0	1.9	Robert (1997)
bored	granular	0	1.0	Meyerhof (1976)
	granular	55	5.8	Fujita et al. (1977)
	cohesionless	0	3.3	Wright & Reese (1979)
	cohesive (f_s < 170 kPa)	10	3.3	Decourt (1982)
	cohesive	0	5.0	Shioi & Fukui (1982)
	cohesive cohesionless	0 0	1.8 0.6	Bazaraa & Kurkur (1986)
	residual soil & weathered rock	0	2.0	Broms et al. (1988)
	clay sand	0 0	1.3 0.3	Koike et al. (1988)
	sandy soil cohesive	35 24	3.9 4.9	Kanai & Yubuuchi (1989)
	residual soil	0	4.5	Winter et al. (1989)
	gravel sand silt clay	0 0 0 0	6.0 4.0 2.5 1.0	Hirayama (1990)
	residual soils	0	2.0	Chang & Broms (1991)
	clayey soil sandy soil	0 0	10.0 3.0	Matsui (1993)
	misc.	17.3 18.2	1.18 0.65	Vrymoed (1994)
	misc.	0	1.9	Robert (1997)
	sand	0	5.05	Kuwabara & Tanaka (1998)
	weathered rock	0	4	Wada (2003)
cast-in-place	cohesionless cohesive	0 0	5.0 10.0	Shoi & Fukui (1982)
	cohesionless (f_s < 200 kPa) cohesive (f_s < 150 kPa)	30 0	2.0 5.0	Yamashita et al.(1987)

Note: $f_s = \beta + \alpha N$ (f_s in units of kPa) design or working loads as defined above to achieve the ultimate load requirement. National and local building code regulations may require more stringent Factors of Safety on certain projects.

Most current structural design standards in Canada use a Limit States Design (LSD) approach for the structural design of helical piles/anchors rather than working or allowable stress design (WSD). All specified loads (dead, live, snow, wind, seismic, etc.) are factored in accordance with appropriate load factors and load combinations should be considered. In addition, the geotechnical resistance of the helical pile/anchor must be factored. Geotechnical resistance factors for helical piles/anchors are not yet clearly defined. Therefore, a rational approach should be taken by the designer and resistance factors should be considered that are suitable to specific requirements.

5.5 HeliCAP® HELICAL CAPACITY DESIGN SOFTWARE

Hubbell Power Systems, Inc. engineers developed HeliCAP® design software to determine the bearing capacity of helical piles and anchors in soil. Since then, it has been revised several times to provide additional features such as side resistance for steel pipe piles and grouted shaft helical piles. HeliCAP® software is available to engineers and designers upon request. The software uses the same theory of general bearing capacity as presented in Section 5.2 for deep foundations (minimum depth ≥ 5D). A key feature of HeliCAP is it's designed to work with the information commonly available from soils reports. In North America, soil investigation usually includes a soil boring as described in Section 2 of this Technical Design Manual. The most common information available from the soils boring is the soil profile, groundwater location, and SPT blow count data per ASTM D-1586. As such, HeliCAP® includes blow count correlations for shear strength, angle of internal friction, and unit weight. These correlations are generally accepted as reasonable approximations given the available blow count data.

The following equations, factors, empirical values, etc., presented in this section are the algorithms used in the HeliCAP® v2.0 Helical Capacity Design Software. This program makes the selection of a helical anchor/pile much quicker than making hand calculations. It allows calculations to be made quickly while varying the different parameters to arrive at the most appropriate solution. As with any calculations, the results from this program are no better than the input data used to generate them.

The program will assist in determining an appropriate helical lead configuration and overall anchor/pile length. It also provides an estimate of the installation torque. The helical lead configuration can vary by the number and sizes of helix plates required to develop adequate capacity. Helical anchor/pile length may vary due to the combined effects of the lead configuration and soil strength. Generally speaking, the shorter the pile length for a given load, the better the performance will be in regard to deflection under load.

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HeliCAP® BEARING CAPACITY METHODOLOGY

As detailed earlier in this Section, the Individual Plate Bearing Method states the capacity of a single or multi-helix anchor/pile is determined by summing the bearing capacity of the individual helix plate elements specific to a given pile. Thus:

$$Q_t = \Sigma Q_h$$

where:

Q_t = Total ultimate multi-helix anchor/pile capacity
Q_h = Individual helix capacity

HeliCAP determines the ultimate bearing capacity of an individual helix as per the following equation. An upper limit for this capacity is based on helix strength that can be obtained from the manufacturer. See Section 7 of this Technical Design Manual for the mechanical strengths of helix plates.

$$Q_h = A_h\,(cN_c + q'N_q) \leq Qs \qquad \textbf{Equation 5-31}$$

where:

A_h = Projected helix area
Q_s = Capacity upper limit, determined by the helix mechanical strength

Sands φ' > 0; c' = 0

HeliCAP® determines the ultimate bearing capacity in a non-cohesive sand or gravel soil with Equation 5-32 in which the fine-grain (clay) term has been eliminated.

The bearing capacity factor N_q is dependent on the angle of internal friction (φ') of the non-cohesive sand or gravel soil. When a value is provided for the friction angle, HeliCAP uses Figure 5-7 (Nq vs φ') and Equation 5-19 to determine the value for N_q. When the angle of internal friction is not known, HeliCAP estimates it (and Nq) by using blow counts obtained from the Standard Penetration Test per ASTM D 1586. Equation 5-33 allows an estimate of the angle of internal friction from SPT blow count data. This equation is based on empirical data given by Bowles (1968) and its results should be used with caution. The graph in Figure 5-7 allows the determination of N_q for a specific angle of internal friction when measured in degrees. This curve was adapted from work by Meyerhof (1976). Equation 5-19 was written for the curve shown in Figure 5-7, which is Myerhof's N_q values divided by 2 for long term applications. **Note the correlated φ' and Nq values determined by HeliCAP® can be overridden. This is encouraged when more reliable soil data are available.**

$$Q_h = A_h q' N_q = Ah\, \gamma' D N_q$$ **Equation 5-32**

where:

A_h = Projected helix area
D = Vertical depth to helix plate
N_q = Bearing capacity factor for non-cohesive component of soil
γ' = Effective unit weight of the soil

$$\varphi' = 0.28\, N + 27.4$$ **Equation 5-33**

where:

φ' = Angle of internal friction
N = Blow count per ASTM D 1586 Standard Penetration Test

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Fine-Grain Cohesive Soil, φ' = 0; c' > 0

HeliCAP® determines the ultimate bearing capacity in a cohesive or fine-grained soil with Equation 5-17 with the overburden term not used. The Nc factor is 9, provided the installation depth below grade is greater than five times the diameter of the top most helix.

$$Qh = A_h c N_c = A_H[(9)(s_u)]$$ **Equation 5-34**

where:

A_h = Projected helix area
c = "cohesion"; for φ' = 0; c = undrained shear strength = s_u
N_c = Bearing Capacity Factor for Deep Failure = 9 (minimum depth ≥ 5D)

In the event that cohesion or undrained shear strength values are not available, HeliCAP® uses the following equation to obtain estimated undrained shear strength values when blow counts from ASTM D 1586 Standard Penetration Tests are available. This equation is based on empirical values and is offered only as a guide when undrained shear strength values are otherwise not available. It is suggested that results be used with caution. **(NOTE: The correlated undrained shear strength values determined by HeliCAP® can be overridden. This is encouraged when more reliable soil data are available.)**

$c\ (\text{ksf}) = N / 8 \text{ or } = 0.125(N)$ **Equation 5-35**

$c\ (\text{kPa}) = 6N$

where:

c = "cohesion"; for $\varphi' = 0$; c = undrained shear strength = s_u
N = Blow count value per ASTM D 1586 Standard Penetration Test

Unit Weight Correlation

In the event unit weight values are not available, HeliCAP® uses the following equations to obtain estimated unit weight values when blow counts from ASTM D 1586 Standard Penetration Tests are available.

Clay (Fine-Grain) Soils: **Equation 5-36**

$N > 0$ & $N \le 19$: $\gamma = 80 + (2N)$ (lb/ft^3)
$N \ge 20$ & $N \le 40$ $\gamma = 120$ (lb/ft^3)

$N \ge 41$ & $N < 50$ $\gamma = 120 + 2(N\text{-}40)$ (lb/ft^3) **Equation 5-37**
$N \ge 50$ $\gamma = 140$ (lb/ft^3)

Sand (Coarse-Grain) Soils:
$N = 0$ $\gamma = 65$ (lb/ft^3)

$N > 0$ & $N \le 7$ $\gamma = 60 + 5N$ (lb/ft^3) **Equation 5-38**
$N \ge 8$ & $N \le 10$ $\gamma = 100$ (lb/ft^3)

$N \ge 11$ & $N < 50$ $\gamma = 90 + N$ (lb/ft^3) **Equation 5-39**
$N \ge 50$ $\gamma = 140$ (lb/ft^3)

These correlations were originally determined from Tables 3-2 and 3-3 in Bowles first edition of Foundation Analysis and Design. These relationships provide an approximation of the total unit weight. They have been modified slightly from how they were originally presented as experience has suggested. **(NOTE: The correlated total unit weight values determined by HeliCAP® can be overridden. This is encouraged when more reliable soil data are available.)**

Mixed Soils φ' > 0; c' > 0

The determination of the bearing capacity of a mixed soil, one that exhibits both cohesion and friction properties, is accomplished by use of Equation 5-31. This is fairly uncomplicated when accurate values are available for both the cohesion (undrained shear strength) and friction terms (φ' & γ') of the equation. It is not possible to use ASTM D 1586 Blow Count correlations to determine all soil strength variables in the bearing capacity equation. Therefore, unless the designer is quite familiar with the project soil conditions, it is recommended that another approach be taken when accurate values are not available for both terms of the equation.

One suggestion is to first consider the soil as fine-grained (cohesive) only and determine capacity. Then consider the same soil as coarse-grained (cohesionless) only and determine capacity. Finally, take the lower of the two results and use that as the soil bearing capacity and apply appropriate Factors of Safety, etc.

HeliCAP® SHAFT RESISTANCE METHODOLOGY

As discussed earlier in this section, the shaft resistance developed by pipe shaft or grouted shaft screw-piles is considered in much the same way that shaft resistance in a driven pile develops. HeliCAP® uses this traditional approach that is available in most foundation design textbooks.

The general equation is:

$$Qf = \Sigma[\pi(D)fs(\Delta Lf)] \qquad \textbf{Equation 5-40}$$

where:

D = Diameter of steel or concrete pile column
f_s = Sum of friction and adhesion between soil and pile
ΔL_f = incremental pile length over which πD and fs are taken as constant

HeliCAP® uses two empirical methods to calculate shaft resistance - the Gouvenot Method and the US Department of Navy Method. The Gouvenot Method is named after the French researcher; who conducted tests on a variety of grouted shaft micropiles including gravity fed grout columns. HeliCAP® uses the Gouvenot method to calculate shaft resistance for grouted columns only (HELICAL PULLDOWN® Micropiles). The US Navy method uses the Dept. of Navy Design Manual 7, Soil Mechanics, Foundations and Earth Structures (1974). HeliCAP® uses the Navy method to calculate shaft resistance for both grouted columns and straight steel pipe shafts.

- Gouvenot reported a range of values for skin friction of concrete/grout columns based on a number of field load tests. The soil conditions are divided into three categories based on friction angle (ϕ) and cohesion (c). The equations used to calculate f_s are:

  Type I: Sands and gravels with 35° < φ < 45° and c' = 0:

  $$f_s = \sigma_o \tan \varphi \qquad \textbf{Equation 5-41}$$

  where: σ_o = Mean normal stress for the grout column

  Type II: Mixed soils; fine loose silty sands with 20° < φ < 30° and sandy clays with 205 psf < C < 1024 psf (9.8 kPa < c < 49 kPa)

$f_s = \sigma_o(\sin \varphi) + c(\cos \varphi)$ **Equation 5-42**

Type III: Clays with 1024 psf < c < 4096 psf (49 kPa < c < 196 kPa)

$f_s = C$ **Equation 5-43**

where: 1024 psf < c < 2048 pfs (49 kPa < c < 98 kPa)
and:

f_s = 2048 psf (98 kPa) **Equation 5-44**

where: 2048 psf < c < 4096 psf (98 kPa < c < 196 kPa)

In HeliCAP® this analysis assumes a uniform shaft diameter for each soil layer and, if required, the friction capacity of the pile near the surface can be omitted.

- **Department of the Navy Design Manual 7 Method:**

For cohesive soils (α Method):

$Q_f = \Sigma[\pi(D)C_a(\Delta L_f)]$ **Equation 5-45**

where: C_a = Adhesion factor (See Table 5-13)

For cohesionless soils (α Method):

$Q_f = \Sigma[\pi D(qK\tan\phi)\Delta L_f]$

Equation 5-46

where: q = Effective vertical stress on element ΔL_f

K = Coefficient of lateral earth pressure ranging from K_o to about 1.75 depending on volume displacement, initial soil density, etc. Values close to K_o are generally recommended because of long-term soil creep effects. As a default, use K_o = 1.
φ = Effective friction angle between soil and plate material

$Q_f = \Sigma[\pi D(S)\Delta L_f]$ **Equation 5-47**

where: S = Average friction resistance on pile surface area = $P_o \tan \varphi$ (See Tables 5-5 & 5-14)
P_o = Average overburden pressure

For straight steel pipe shaft piles in sand, HeliCAP® uses Table 5-5 to calculate shaft resistance in sand layers using the Alternate Navy Method.

Tables 5-13, 5-14 and 5-5 are derived from graphs in the Department of the Navy Design Manual 7, Soil Mechanics, Foundations and Earth Structures (1974). Later editions of this manual limit the depth at which the average overburden pressure is assumed to increase. The following is an excerpt from the manual regarding this limiting depth:

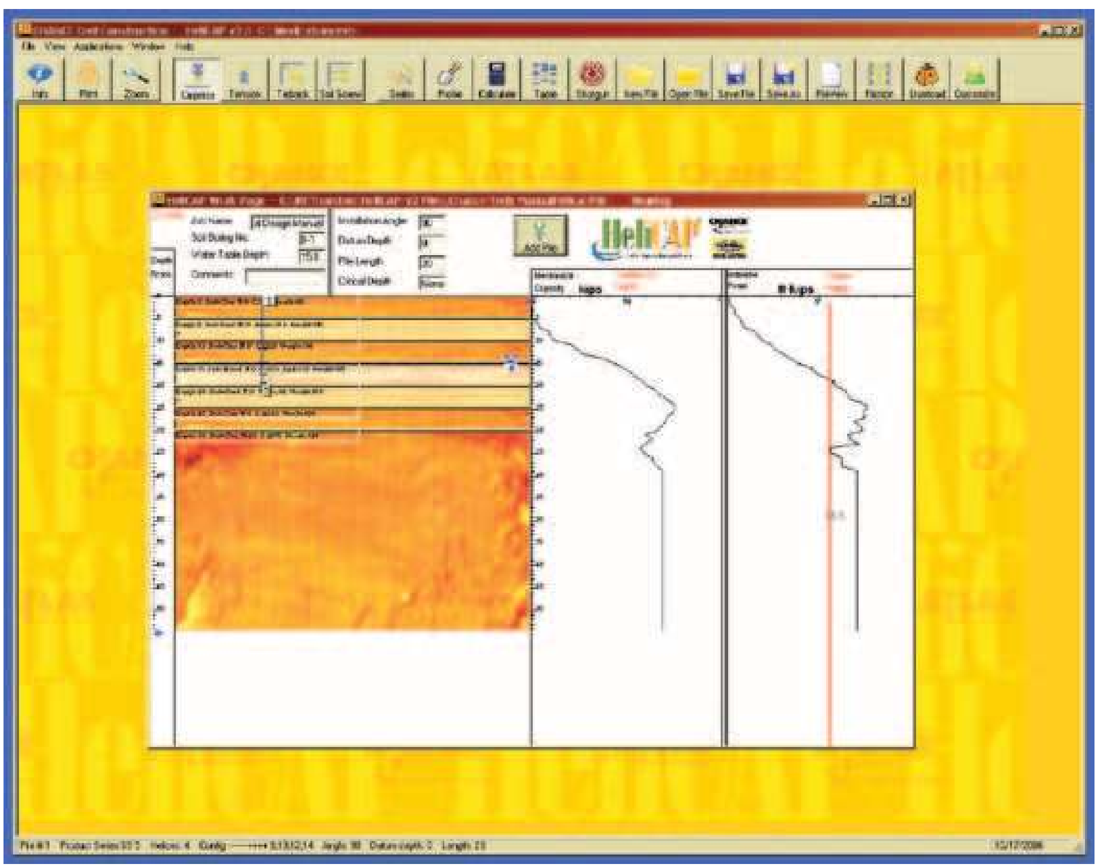
"Experimental and field evidence indicate that bearing pressure and skin friction increase with vertical effective stress (P_o) up to a limiting depth of embedment, depending on the relative density of the granular soil and position of the water table. Beyond this limiting depth (10B± to 40B±) there is very little increase in end bearing, and increase in side friction is directly proportional to the surface area of the pile. Therefore, if D is greater than 20B, limit P_o at the pile tip to that value corresponding to D = 20B" where D = depth of the pile embedment over which side friction is considered and B = diameter of the pile.

Design Example 8-5 in Section 8 illustrates the use of the Navy Design Manual 7 method to calculate the friction capacity of a CHANCE HELICAL PULLDOWN® Micropile.

HeliCAP® v2.0 Helical Capacity Design Software calculates ultimate capacity and must have an appropriate Factor of Safety applied to the results. The program has additional features that allow it to be used for other applications, but it is beyond the scope of this manual to present all facets of the program. For additional assistance, refer to the Help screen or contact Hubbell Power Systems, Inc. application engineers.

The following screen is from HeliCAP® v2.0 Helical Capacity Design Software. It shows a typical workpage with the soil profile on the left and helical pile capacity on the right.

Design Examples 8-3 through 8-12 in Section 8 illustrate the use of the standard bearing equation to determine the bearing capacities of helical piles/anchors. These design examples are taken from actual projects involving residential and commercial new construction, boardwalks, tiebacks, telecommunication towers, pipeline buoyancy control, etc.



5.6 APPLICATION GUIDELINES for CHANCE® HELICAL PILES/ANCHORS

- The uppermost helix should be installed at least three diameters below the depth of seasonal variation in soil properties. Therefore, it is important to check the frost depth or "mud" line at the project site. Seasonal variation in soil properties may require the minimum vertical depth to exceed five helix diameters. The influence of the structure's existing foundation (if any) on the helical pile/anchor should also be considered. Hubbell Power Systems, Inc. recommends helical piles/anchors be located at least five diameters below or away from existing foundation elements.
- The uppermost helix should be installed at least three helix diameters into competent load-bearing soil. It is best if all helix plates are installed into the same soil stratum.
- For a given shaft length, use fewer longer extensions rather than many shorter extensions. This will result in fewer connections and better load/deflection response.
- Check economic feasibility if more than one combination of helical pile/anchors helix configuration and overall length can be used.

Table 5-13. Recommended Adhesion Values in Clay *

PILE TYPE	SOIL CONSISTENCY	COHESION, c (psf)	ADHESION, Ca (psf)
Concrete	Very Soft	0 – 250	0 – 250
	Soft	250 – 500	250 – 480
	Medium Stiff	500 – 1000	480 – 750
	Stiff	1000 – 2000	750 – 950
	Very Stiff	2000 – 4000	950 – 1300
Steel	Very Soft	0 – 250	0 – 250
	Soft	250 – 500	250 – 460
	Medium Stiff	500 – 1000	460 – 700
	Stiff	1000 – 2000	700 – 720
	Very Stiff	2000 – 4000	720 - 750

* From Department of the Navy Design Manual 7, Soil Mechanics, Foundations and Earth Structures (1974).

Table 5-14. Straight Concrete Piles in Sand

P_o (psf)	Effective Angle of Internal Friction (degrees) (φ')				
	20	25	30	35	40
	S= Average Friction Resistance on Pile Surface (psf)				
500	182	233	289	350	420
1000	364	466	577	700	839
1500	546	699	866	1050	1259
2000	728	933	1155	1400	1678
2500	910	1166	1443	1751	2098
3000	1092	1399	1732	2100	2517
3500	1274	1632	2021	2451	2937
4000	1456	1865	2309	2801	3356

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5.7 LATERAL CAPACITY OF HELICAL PILES

Introduction

The primary function of a deep foundation is to resist axial loads. In some cases they will be subjected to horizontal or lateral loads. Lateral loads may be from wind, seismic events, live loads, water flow, etc. The resistance to lateral loads is in part a function of the near surface soil type and strength, and the effective projected area of the structure bearing against these soils. This section provides a summarized description of the methods and procedures available to determine the lateral capacity of helical piles/anchors in soil.

The analysis of deep foundations under lateral loading is complicated because the soil reaction (resistance) at any point along the shaft is a function of the deflection, which in turn is dependent on the soil resistance. Solving for the response of a deep foundation under lateral loading is one type of soil-structure interaction problem best suited for numerical methods on a computer. Square shaft (SS) helical piles/anchor do not provide any significant resistance to lateral loads. However, Round Shaft (RS) helical piles/anchor and HELICAL PULLDOWN® Micropiles can provide significant resistance to lateral loads depending on the soil conditions. Over the past 7 seven years, there has been considerable research done on the lateral capacity of grouted shaft helical piles – both with and without casing. Abdelghany & Naggar (2010) and Sharnouby & Naggar (2011) applied alternating cyclic lateral loads to helical piles of various configurations in an effort to simulate seismic conditions. Their research showed that helical piles with grouted shafts retain all their axial load capacity after being subjected to high displacement lateral load.

Lateral Resistance - Methods Used

Most helical piles/anchors have slender shafts [less than 3 inch (89 mm)] that offer limited resistance to lateral loads when applied to vertically installed shafts. Load tests have validated the concept that vertical pile foundations are capable of resisting lateral loads via shear and bending. Several methods are available to analyze the lateral capacity of foundations in soil including: 1) Finite Difference method; 2) Broms' Method (1964a) and (1964b); 3) Murthy (2003) Direct Method; and 4) Evans & Duncan (1982) Method as presented by Coduto (2001). Each of these methods may be applied to Round Shaft helical piles..

Lateral resistance can also be provided by passive earth pressure against the structural elements of the foundation. The resisting elements of the structure include the pile cap, grade beams and stem walls. The passive earth pressure against the structural elements can be calculated using the Rankine Method.

Battered or inclined helical piles/anchors can be used to resist lateral loads by assuming that the horizontal load on the structure is resisted by components of the axial load. The implicit assumption in this is that inclined foundations do not deflect laterally, which is not true. Therefore, it is better practice to use vertically installed helical piles/anchors to resist only vertical loads and inclined helical piles/anchors to resist only lateral loads. When inclined piles are required to resist both vertical and lateral loads, it is good practice to limit the pile inclination angle to less than 15°.

Friction resistance along the bottom of a footing, especially in the case of a continuous strip footing or large pile cap, can be significant. The friction component in a sandy soil is simply the structure's dead weight multiplied by the tangent of the angle of internal friction. In the case of clay, cohesion times the area of the footing may be used for the friction component. When battered piles are used to prevent lateral movement, the friction may be included in the computation. The designer is advised to use caution when using friction for lateral resistance. Some building codes do not permit friction resistance under pile supported footings and pile caps due to the possibility the soil will settle away from the footing or pile cap. Shrink-swell soils, compressible strata, and liquefiable soil can result in a void under footings and pile caps.

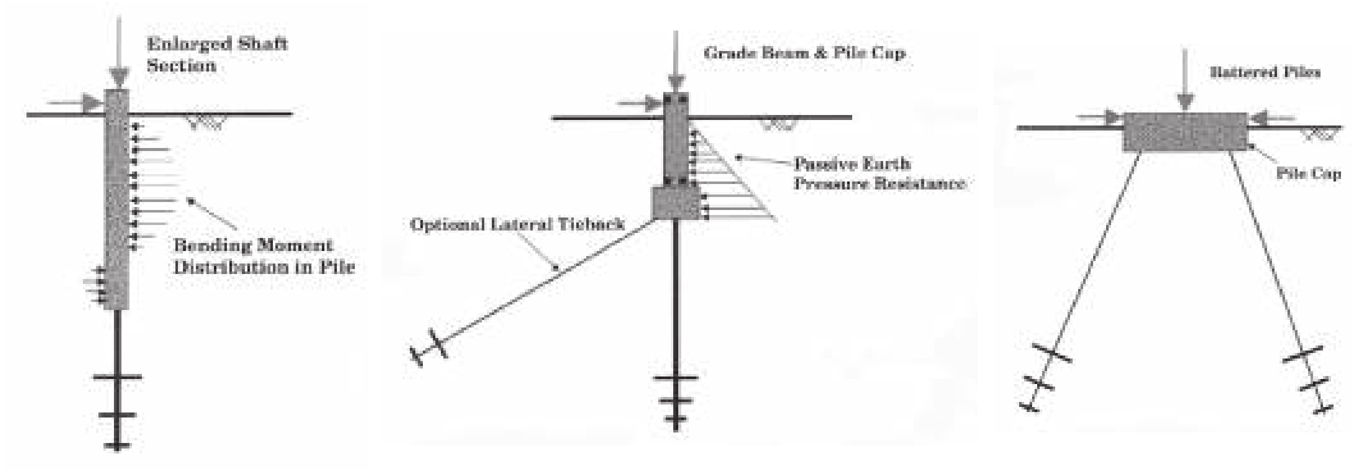

Figure 5-17. Lateral Resistance Methods

Finite Difference Method

Several computer programs, such as LPILEPLUS (ENSOFT, Austin, TX) are revisions of the COM624 program (Matlock and Reese) and its predecessor Beam-Column 28 (Matlock and Haliburton) that both use the p-y concept, i.e., soil resistance is a non-linear function of pile deflection, which was further developed by Poulos (1973). This method is versatile and provides a practical design method. This is made possible by the use of computers to solve the governing non-linear, fourth-order differential equation, which is explained in greater detail on page 5-20. Lateral load analysis software gives the designer the tools necessary to evaluate the force-deflection behavior of a helical pile/anchor embedded in soil.

Figures 5-18 and 5-19 are sample LPILE^{PLUS} plots of lateral shaft deflection and bending moment vs. depth where the top of the pile is fixed against rotation. From results like these, the designer can quickly determine the lateral response at various horizontal loads up to the structural limit of the pile, which is typically bending. Many geotechnical consultants use LPILE^{PLUS} or other soil-structure-interaction programs to predict soil-pile response to lateral loads.

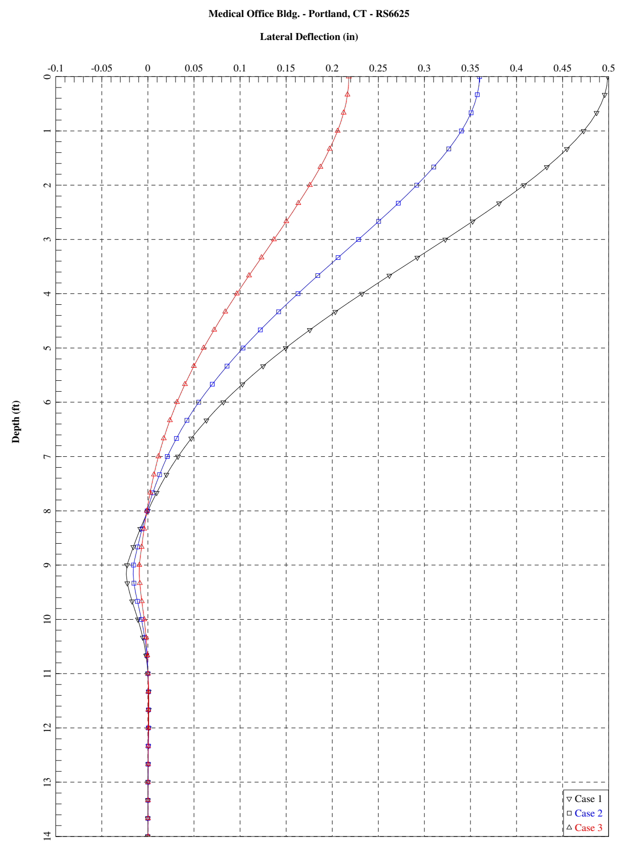

Figure 5-18. LPILE[PLUS] Sample Plot
Deflection vs Depth

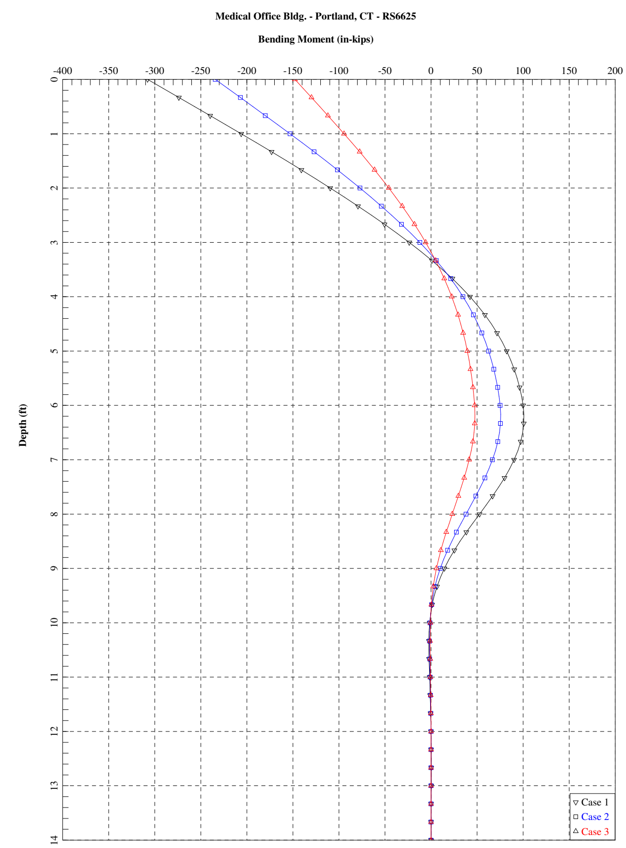

Figure 5-19. LPILE[PLUS] Sample Plot
Bending Moment vs Depth

Brom's (1964a & 1964b) Method

Broms' Method is best suited for applications where the top section of the helical pile/anchor/pile is a greater diameter than the bottom section. Enlarged top sections are commonly used to increase the lateral capacity of the foundation shaft. Design Example 8-13 in Section 8 gives an example of this. It uses Broms' method for short piers in cohesive soil. A "short" pier is one that is rigid enough that it will move in the direction the load is tending by rotation or translation. A "long" pier is one that the top will rotate or translate without moving the bottom of the foundation, i.e., a plastic hinge will form.

Broms developed lateral capacity methods for both short and long piles in cohesive and non-cohesive soil. Broms theorized that a short free-headed pier rotates about a center, above the lower end of the foundation, without substantial deformation along its axis. The resistance is the sum of the net of the earth pressures above and the passive earth pressure below the center of rotation. The end bearing influence or effect is neglected. Likewise, the passive earth pressure on the uppermost 1.5 diameters of shaft and the active earth pressure on the back of the pile are neglected.

Figure 5-20 is a reaction/shear/moment diagram that demonstrates the Broms theory for laterally loaded short piles in cohesive soils. A simple static solution of these diagrams will yield the required embedment depth and shaft diameter of the top section required to resist the specified lateral load. It is recommended the designer obtain and review Broms' technical papers (see References at the end of this section) to familiarize themselves with the various solution methods in both cohesive and non-cohesive soils. The Broms Method was probably the most widely used method prior to the finite difference and finite element methods used today and gives fair agreement with field results for short piles.

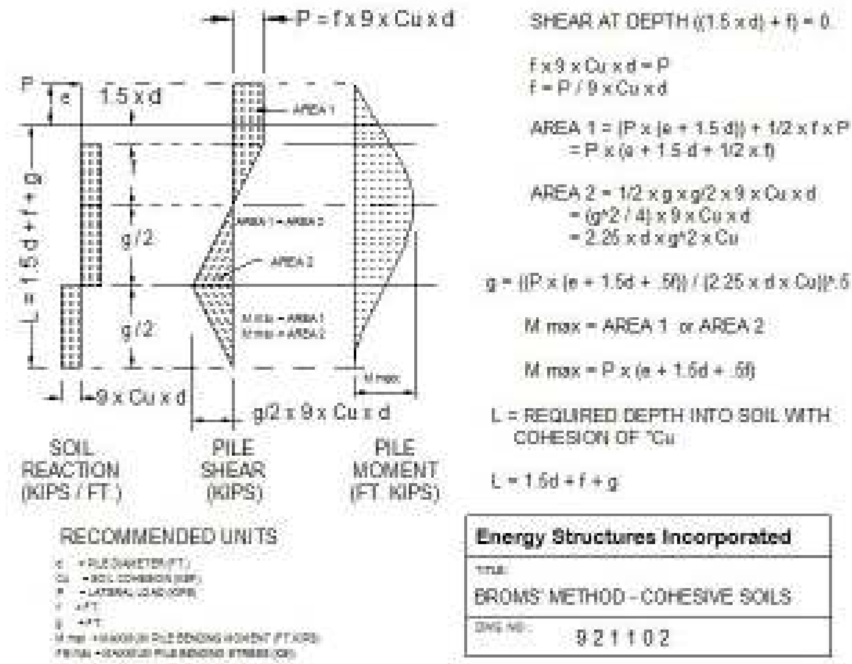

Figure 5-20. Broms' Method for Short Piles in Clay
(Energy Structures, Inc., 1994)

Lateral Capacity By Passive Earth Pressure

Passive earth pressure on the projected area of the pile cap, grade beam, or stem wall can be calculated by the Rankine (ca. 1857) method, which assumes no soil cohesion or wall-soil friction. One can use known or assumed soil parameters to determine the sum of the passive earth pressure minus the active earth pressure on the other side of the foundation as shown in Figure 5-21. The following are general equations to calculate active and passive pressures on a wall for the simple case on a frictionless vertical face and a horizontal ground surface. Equations 5-51 and 5-52 are Rankine equations for sand. Equations 5-53 and 5-54 are the addition of the cohesion for clay or cohesive soils. Three basic conditions are required for validity of the equations:

1. The soil material is homogenous.
2. Sufficient movement has occurred so shear strength on failure surface is completely mobilized.
3. Resisting element is vertical; resultant forces are horizontal.

$K_0 = 1 - \sin \phi'$ **Equation 5-48**

$K_a = \tan^2 (45 - \phi'/2)$ **Equation 5-49**

$K_p = \tan^2 (45 + \phi'/2)$ **Equation 5-50**

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