

Technical Design Manual | Edition 5

CHANCETM



CHANCE TECHNICAL DESIGN MANUAL

Edition 5

Chance® Civil Construction | Hubbell Power Systems, Inc.
Centralia, Missouri 65240 | U.S.A.



Printed copies of this Manual are printed in the USA on recycled paper.

© Copyright 2023 Hubbell Incorporated. Chance® is a registered trademark of Hubbell, Inc.
All product names and brands are property of their respective owners.
CENT-CV-TECHNICAL-DESIGN-MANUAL-TECHP-EN-V5-00350

Bulletin 01-0605 | Revision 04-28-2023

Because Hubbell has a policy of continuous product improvement,
we reserve the right to change design and specifications without notice.

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 should be conducted and state and local building codes and authorities should be consulted 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 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.



ONLINE RESOURCES

- Chance® Websitewww.chancefoundationsolutions.com
- HeliCAP® Helical Capacity Design Software.www.hpsapps.com/helicap
- Select-A-Base Appwww.hpsapps.com/base
- Find a distributor.www.chanceexpert.com/distributor
- Blogblog.chancefoundationsolutions.com
- Warranty Registration.www.chanceexpert.com/warranty
- Video Libraryvideos.hubbellpowersystems.com (Helical Piles category in side menu)
- PDFs of Catalogs and Manualswww.hpslibrary.com (Chance Foundation Solutions category in side menu)

SYMBOLS & ACRONYMS

γ	Effective Unit Weight of Soil	DL	Design Load (Appendix B only)
γ'	Submerged Unit Weight (Submerged Density) (Section 2 only)	DMT	Dilatometer Test
γ''	Effective Unit Weight of the Soil (Section 5 only)	DS	Design Load
γ_d	Dry Unit Weight (Dry Density)	e	Void Ratio
γ_s	Saturated Unit Weight (Saturated Density)	E	Modulus of Elasticity
γ_t	Wet (Total) Unit Weight (Wet Density)	EI	Flexural Rigidity of the Foundation Shaft
ΔL_f	Incremental Pile Length	E_p	Modulus of Elasticity of Foundation Shaft
θ	Failure Plane Angle	EPA	Effective Projected Area
σ	Total Stress	E_s	Secant Modulus of the Soil Response Curve
σ'	Effective Stress	E_{sy}	Soil Reaction per Unit Length
σ_o	Mean Normal Stress	FHWA	Federal Highway Administration
τ_f	Shear Strength	FS	Factor of Safety
ϕ	Angle of Internal Friction	f_s	Sum of Friction and Adhesion Between Soil and Pile
A	Effective Cylinder Area	F_{sh}	Factor of Safety for Mechanical Strength of Hardware
AASHTO	American Association of State Highway and Transportation Officials	F_{Sp}	Proof Load Factor of Safety
ACI	American Concrete Institute	FVT	Field Vane Test
A_h	Projected Helix Area	G	Amount of Galvanized Coating
AISC	American Institute of Steel Construction	GWT	Ground Water Table
AL	Alignment Load	H	Height of Wall or Resisting Element
ASL	Allowable Steel Loss	H_d/S_d	Helix to Shaft Diameter Ratio
ASTM	American Society for Testing and Materials	HS	High Strength
AWS	American Welding Society	HSA	Hollow Stem Auger
B	Helix Diameter & Footing Width (Base)	I	Moment of Inertia (Section 5 only)
BOCA	Building Officials and Code Administrators International	I	Electrical Current (Appendix A only)
c	Cohesion of Soil	ICBO	International Conference of Building Officials
C_a	Adhesion Factor	ICC	International Code Council
CFA	Continuous Flight Auger	ICC-ES	ICC Evaluation Service, Inc.
CID	Cubic Inch Displacement	I_p	Moment of Inertia of Foundation Shaft
CL	Corrosion Weight Loss	K_0	Coefficient of Earth Pressure at Rest
CPT	Cone Penetration Test	K_2	Weight Loss by Corrosion
CPTU	Piezocone Penetration Test	K_a	Coefficient of Active Earth Pressure
D	Diameter	k_h	Modulus of Subgrade Reaction
DF	Driving Force	kip	Kilopound
DL	Dead Load	Kl/r	Slenderness Ratio
		K_p	Coefficient of Passive Earth Pressure



SYMBOLS & ACRONYMS

ksi	Kips (kilo-pounds) per square inch	Q_{ULT}	Ultimate Capacity of the Soil
K_t	Empirical Torque Factor	R	Resistance or Resistivity
L	Pin spacing	RF	Resisting Force
L	Foundation Shaft Length	Rh_{MAX}	Maximum Installation Force Based on Ultimate Capacity of Hardware
L.I.	Liquidity Index	Rh_{ULT}	Ultimate Hardware Installation Force
LI	Liquidity Index	R_{meter}	Resistivity Indication from Nillson Resistivity Meter
LL	Live Load	R_p	Installation Force to Achieve Proof Load
LL	Liquid Limit (Section 2 only)	RQD	Rock Quality Designation
L_p_{MAX}	Maximum Free Span Between Piers	RR	Round Rod
L_u	Unsupported Length	RS	Round Shaft
M	Mass	$R_{w_{ULT}}$	Ultimate Hardware Strength based on Structural Weight
n	Porosity	S	Degree of Saturation (Section 2 only)
N	Field Blowcount Value from Standard Penetration Test (SPT)	S	Average Friction Resistance on Pile Surface Area (Section 5 only)
$(N1)_{60}$	Normalized SPT N-value	SBCCI	Southern Building Code Congress International
NBS	National Bureau of Standards	S_K	Snow Load Factor
N_c	Bearing Capacity Factor for Cohesive Component of Soil	SL	Snow Load
N_q	Bearing Capacity Factor	SL	Shrinkage Limit (Section 2 only)
N_y	Bearing Capacity Factor for Soil Weight and Foundation Width	SL	Service Life (Appendix A only)
ϕ	Effective Friction Angle Between Soil & Pile Material	SLF	Street Light Foundation
OCR	Overconsolidation Ratio	SPT	Standard Penetration Test
P	Line Load on Footing	SS	Square Shaft
P_a	Active Earth Pressure	SS	Split Spoon (Section 2 only)
P_{cr}	Critical Buckling Load	SSCF	Soil Screw® Anchor Component Force
P_{crit}	Critical Compression Load	S_t	Soil Sensitivity
P_{des}	Design Load per Pier	ST	Shelby Tube
pH	Acidity or Alkalinity of a Solution	s_u	Undrained Shear Strength
PI	Plasticity Index	T	Tension Load (Section 4 only)
PIF	Power Installed Foundation	T	Average Installation Torque (Section 6 only)
PISA	Power Installed Screw Anchor	T	Indicated Force (Section 9 only)
PL	Plastic Limit (Section 2 only)	T/C	Tension/Compression
PL	Proof Load (Section 6 only)	T_{FN}	Critical Helical Anchor Head Load
P_o	Average Overburden Pressure	u	Pore Water Pressure
P_p	Passive Earth Pressure	UC	Unconfined Compression Test
ppm	Parts per Million	U_{cr}	Dimensionless Ratio
psf	Pounds per Square Foot	USCS	Unified Soil Classification System
PT	Test Pressure	V	Volume (Section 2 only)
P_w	Pier Working Load	V	Voltage (Appendix A only)
q	Effective Vertical Stress on Element	V_N	Punching Shear Strength
Q	Axial Compressive Load	VST	Vane Shear Test
q'	Effective Overburden Pressure	W	Soil Load
Q_{act}	Actual Capacity	w_n	Moisture Content
Q_{act}/Q_{calc}	Capacity Ratio	Ws	Weight of Steel Pile
Q_{calc}	Calculated Capacity	WSF	Wenner Spacing Factor
Q_h	Individual Helix Capacity	WWF	Welded Wire Fabric
Q_s	Capacity Upper Limit	x	Pier Spacing
Q_t	Total Ultimate Multi-Helix Anchor/Pile Capacity	x_{MAX}	Maximum Pier Spacing
q_u	Unconfined Compressive Strength	y	Lateral Deflection of Shaft at Point x



TABLE OF CONTENTS

SECTION 1: INTRODUCTION

Atlas Resistance® Piers.....1-2
 Helical Piles/Anchors..... 1-6
 Bibliography..... 1-13

SECTION 2: SOIL MECHANICS

Introduction2-2
 Soil Mechanics.....2-2
 Site Investigations.....2-7

SECTION 3: PRODUCT FEASIBILITY

Feasibility Of Using Chance® Helical or Atlas Resistance® Pier Products.....3-2
 Shaft Size Selection Based on Soil Parameters 3-4
 Preliminary Chance Helical Pile/Anchor and Atlas Resistance® Pier Design Guide..... 3-4

SECTION 4: LOAD DETERMINATION

Structural Loads 4-2
 Preliminary Tieback Design Guide..... 4-3
 Tables For Estimating Dead Line (DL) & Live Line (LL) Loads 4-6
 Tables For Estimating Free Spans Between Supports 4-8
 Preliminary Design Guidelines For Reinforced Concrete Grade Beams..... 4-12
 Preliminary Design Guidelines For Reinforced Concrete Pile Caps.....4-19

SECTION 5: DESIGN METHODOLOGY

5.1 Atlas Resistance Pier Capacity5-2
 5.2 Chance Helical Pile/Anchor Ultimate Bearing Capacity.....5-3
 5.3 Evaluating Soil Properties For Design 5-14
 5.4 Factor Of Safety5-20
 5.5 HeliCAP Helical Capacity Design Software 5-22
 5.6 Application Guidelines For Chance Helical Piles/Anchors..... 5-25
 5.7 Lateral Capacity Of Helical Piles 5-25
 5.8 Buckling/Bracing/Slenderness Considerations.....5-30
 5.9 Helical Pile Deflection At Working Load.....5-34

SECTION 6: INSTALLATION METHODOLOGY

Atlas Resistance® Piers..... 6-2
 Chance® Helical Piles/Anchors 6-4
 Installation Torque/Capacity Relationship 6-4
 Torque Indicator Calibration..... 6-12
 Installation Termination Criteria 6-12

SECTION 7: PRODUCT DRAWINGS AND RATINGS

Atlas Resistance Piers7-2
 Chance Helical Piles/Anchors..... 7-7
 Square Shaft Helical Piles and Anchors 7-11



TABLE OF CONTENTS

Round Shaft Helical Piles and Anchors	7-29
Chance Rock-It Helical Lead.....	7-75
Type SS/RS Combination Helical Piles	7-76
Chance Helical Pulldown Micropiles	7-77
Remedial Repair Brackets For Chance Helical Piles	7-81
New Construction Pile Caps	7-90

SECTION 8: DESIGN EXAMPLES

Design Example 1: Atlas Resistance Piers.....	8-2
Design Example 2: Atlas Resistance Piers with Integrated Tieback.....	8-3
Design Example 3: Helical Pile Foundation for New Construction	8-5
Design Example 4: Light Commercial Structure.....	8-6
Design Example 5: Helical Pulldown Micropiles for New Construction	8-10
Design Example 6: Helical Piles for Boardwalks	8-14
Design Example 7: Helical Piles for Boardwalks with Lateral Support.....	8-15
Design Example 8: Helical Tieback Anchors In Clay	8-16
Design Example 9: Helical Tieback Anchors In Sand.....	8-17
Design Example 10: Soil Screw Retention Wall System	8-19
Design Example 11: Helical Piles/Anchors for Telecommunication Towers	8-24
Design Example 12: Helical Anchors for Pipeline Buoyancy Control	8-31
Design Example 13: Type RS Helical Piles for Lateral Support.....	8-35
Design Example 14: Instant Foundations for Street Light Supports	8-36
Design Example 15: Foundation Earth Pressure Resistance.....	8-39
Design Example 16: Buckling Example Using the Davisson Method.....	8-40
Design Example 17: Buckling Example Using the Finite-Difference Method	8-41
Design Example 18: Buckling Example Using the Finite-Element Method	8-43
Design Example 19: Monopole Foundation with Steel Grillage & RS5500 Helical Piles....	8-44

SECTION 9: SOIL SCREW® RETENTION WALL SYSTEM

Introduction.....	9-2
Soil Screw® Retention Wall System	9-2
Preliminary Design Considerations	9-4
Geotechnical And Structural Engineering.....	9-5
Limiting Load Capacities	9-7
General Construction Considerations of Underpinning/Shoring Systems.....	9-7
Concepts And Applications Of Underpinning/Shoring Systems	9-8
Case Study 1 - High Foundation Line Load With Shallow Cut.....	9-9
Case Study 2 - Low Foundation Line With Deep Cut.....	9-12

SECTION 10: INSTANT FOUNDATION® SYSTEM

Introduction.....	10-2
Design.....	10-2
Laterally Loaded Foundations	10-6
Instant Foundations® System Specifications.....	10-7
Product Specifications.....	10-7
Pole Load Determination Data Sheet.....	10-9



TABLE OF CONTENTS

APPENDIX A: CORROSION - AN OVERVIEW

Introduction.....	A-2
Corrosion Theory	A-2
Soil Environments	A-3
Predicting Corrosion Loss	A-5
Corrosion Loss Rates.....	A-6
Field Measurement of Soil Resistivity	A-7
Corrosion Control Techniques.....	A-8
Design Examples.....	A-13

APPENDIX B: LOAD TESTS

Static Load Tests (Tiebacks).....	B-2
Static Axial Load Tests (Compression/Tension)	B-4
Static Load Tests (Lateral)	B-6
Acceptance Criteria	B-8

APPENDIX C: HELICAL PILES & ANCHORS - A BASIC GUIDELINE FOR DESIGNERS

I. Introduction	C-2
II. Helical Pile Capacity	C-2
III. Design Process	C-2
A. Data Gathering	C-3
B. Feasibility.....	C-3
C. P1, P2, P3 & P4	C-3
IV. P4 - Geotechnical Capacity	C-3
V. P1, P2 And P3 - Structural Strength	C-6
VI. Summary.....	C-11
VII. Reliability	C-11
VIII. Other Topics Related To Design.....	C-11
IX. How To Specify Helical Piles.....	C-13
X. Construction Documents.....	C-13

APPENDIX D: FORMS

Preliminary Design Request Form.....	D-2
Chance® Helical Pile/Anchor Axial Test Form	D-3
Atlas Resistance® Piers Installation Log	D-4
Chance® Helical Pile/Anchor Installation Log.....	D-5
Chance Helical Pulldown® Micropile Installation Log	D-6
Atlas Resistance® Piers - Project Summary Log.....	D-7
Pole Load Determination Data Sheet.....	D-8
Site Inspection Form.....	D-9



GLOSSARY

Alignment Load (AL) - A low magnitude load applied to a pile/anchor at the start of the load test to keep the testing equipment correctly positioned and to remove any slack in the reaction system.

Allowable Capacity - The geotechnical capacity of a pile/anchor or pier as determined by a reduction of the ultimate capacity with an appropriate factor of safety or resistance factor.

Anchor or Anchorage - A combination of anchor and the soil or deeply weathered rock into which it is installed that together resist tension loads applied to the anchor.

Atlas Resistance® Pier - An assembly of structural steel components that includes a foundation bracket assembly attached to the concrete foundation, which is then mounted to a steel pier that is installed to bedrock or dense bearing stratum via hydraulic jacking of the pipe shaft segments.

Axial Load (P) - An axially oriented compression or uplift (tension) load supported by an pile/anchor or pier resulting from dead, live and seismic loads.

Bearing Load - A load generally regarded as an axial compressive load on a pile or pier.

Bearing Stratum - Soil layers of sufficient strength to be capable of resisting the applied axial load transferred by a pile or pier.

Contractor - The person or firm responsible for performing the required construction, i.e., installation of Chance® Helical Piles/Anchors or Atlas Resistance Piers.

Coupling - A central steel shaft connection for Chance Type SS and RS helical piles. Couplings may be either separable sleeve couplings or integral forged sockets.

Coupling Bolts - High strength structural steel fasteners used to connect helical anchor/pile segments together. For Chance Type SS segments the coupling bolt transfers axial loads. For Chance Type RS segments the coupling bolt transfers both axial and torsional loads.

Coupling, Pier Sleeve - A steel tubing of suitable outside diameter to fit into a pier starter and extension section to provide a means for attaching the various pier sections together for Atlas Resistance Piers. It allows for extending the pier to the required depth.

Creep - The movement that occurs during the Creep Test of a pile/ anchor or pier under a constant load.

Dead Load (DL) - Generally, vertical loads comprised of the weight of the structure plus various fixed assets, such as equipment, machinery, walls and other permanent items.

Design Load (Pd) - The maximum anticipated service load applied to a pile or pier, comprised of calculated dead and live loads. Also known as Working Load.

Effective Stress - The total force on a cross section of a soil mass that is transmitted from grain to grain of the soil, divided by the area of the cross section. Also known as Intergranular Stress.

Elastic Movement - The recoverable movement measured during a pile/pier load test resulting from the elastic shortening or lengthening of the pile/pier shaft material.

End Bearing - The transfer of axial loads to the soil at the tip of a helical pile via helix plates or at the tip of a pier.

Evaluation Services Report (ESR) - The evaluation of a manufactured product or building component by the evaluation services of the various model code agencies (ICC). The report outlines the requirements that must be met to satisfy the intent of the Building Code.

Extension Pier Section - With reference to an Atlas Resistance Pier, the pipe sections following the starter pier section that extend the starter section to the load bearing stratum. The extension pier sections are equipped with a pier sleeve that allows for coupling the extensions to the starter section or other extensions.

Failure Criteria - A method used to determine the ultimate capacity of a pile/anchor based on a load test. A typical failure criteria for helical piles is the load where the pile head displacement is equal to 10% of the average helix diameter plus the elastic movement.

Foundation Soil Load - The load from soil overburden on the outstanding toe of a footing. This soil load is in addition to the existing structure weight supported by the footing. It increases the dead load used as a reaction to install a push pier and therefore aids the installation. However, it may work to defeat attempts to lift a structure and may require reduction or removal if a lift is required.

Friction Reduction Collar - The enlarged section at the bottom of the pipe starter section of an Atlas Resistance Pier. The collar diameter is larger than the following pipe shaft, thus forcing the displaced soil away from the pipe shaft.

Gunite - A dry concrete mixture that is carried to a nozzle in moving air where it is mixed with water. The operator controls the water-cement ratio.

Helical Extension - A helical pile/anchor component installed immediately following the lead section (if required) to increase the bearing area of the foundation. This component consists of one or more helical plates welded to a central steel shaft.

Helical Pile - A bearing type foundation consisting of a lead section, helical extension (if required by site conditions), plain extension section(s) and a pile cap. Also known as a screw pile or helical screw foundation.

Helical Pulldown® Micropile - A small diameter, soil displacement, cast-in-place helical pile in which the applied load is resisted by both end bearing and friction. The design was originally covered under United States Patent 5,707,180, Method and Apparatus for Forming Piles In-Situ.



Helix Plate - A round steel plate formed into a ramped spiral. The helical shape provides the downward force used to install a helical pile/anchor, plus the plate transfers the load to the soil in end bearing. Helical plates are available in various diameters and thicknesses.

Impact Driven - A pile driven with a pile hammer.

In-Situ - In the natural or original position. Used in soil mechanics to describe the original state of soil condition prior to disturbance from field testing or sampling methods.

Installation Torque - The resistance generated by a helical pile/anchor when installed into soil. The installation resistance is a function of the soil plus the size and shape of the various components of the helical pile/anchor. The installation energy must equal the resistance to penetrate the soil (penetration energy) plus the energy loss due to friction (friction energy).

Kip - one thousand pounds of force, or a “kilopound.”

Lagging - Horizontal members, usually of timber or concrete, spanning between soldier piles to retain the soil between pile locations. They transfer the load directly from the soil to the soldier piles.

Lateral Load (V) - A load applied perpendicular to the longitudinal axis of a pile or pier resulting from live and seismic loads. Also called a shear load.

Lead Section - The first helical pile/anchor component installed into the soil, consisting of single or multiple helix plates welded to a central steel shaft. The helical plates transfer the axial load to bearing stratum.

Live Load (LL) - A load comprised of roof, wind, floor, and in some cases, seismic loads. Floor loads include people, temporary or non-fixed equipment, furniture and machinery. Roof loads include ice and snow.

Load Bearing Stratum - See Bearing Stratum.

Net Settlement - The non-elastic (non-recoverable) movement or displacement of a pile/pier measured during load testing.

Open Specification - An arrangement in which the contractor is given the responsibility for the scope and design of the pile or pier installation. The construction, capacity and performance of the pile or pier are the sole responsibility of the contractor. This specification is most common for securing bids on temporary projects, and is not recommended for permanent applications. See also Performance Specification and Prescriptive Specification.

Overburden - Natural or placed material that overlies the load bearing stratum.

Performance Specification - An arrangement in which the contractor is given the responsibility for certain design and/or construction procedures, but must demonstrate to the owner through testing and/or mutually agreed upon acceptance criteria that the production piles/piers meet or exceed the specified performance parameters. The contractor and owner share responsibility for the work. See also open Specification and Prescriptive Specification.

Pier Head Assembly - An Atlas Resistance® Pier bracket or other termination device that allows attachment to an existing footing or floor slab.

Pile Cap - A means of connection through which structural loads are transferred to a pile or pier. The type of connection varies depending on the requirements of the project and the type of pile/pier material used. Note: Care must be used in the design of pile caps to ensure adequate structural load transfer. Design constraints such as expansive soils, compressible soils and seismic loads must be accounted for in pile cap design.

Pipe Shaft - A central shaft element made from hollow, steel, round pipe, ranging in diameter from 2” to 10”. Also known as Hollow Shaft, Round Shaft (Type RS), Type T/C and Type PIF for Chance® Helical Piles.

PISA® System - The acronym for Power Installed Screw Anchor. The PISA System was originally developed for the power utility industry in the late 1950’s.

Plain Extension - A central steel shaft segment without helical plates. It is installed following the installation of the lead section or helical extension (if used). The units are connected with separable sleeve couplings or integral forged couplings and bolts. Plain extensions are used to extend the helical plates beyond the specified minimum depth into competent load bearing stratum.

Pore Pressure - unit stress carried by the water in the soil pores in a cross section.

Post Tensioning - The stressing of a structure after all structural elements are in place (e.g., loading a tieback anchor to post tension a retaining wall).

Preloading - A load applied to a pile prior to connection to a structure to minimize structural movement in service. Also known as Prestressing.

Prescriptive Specification - An arrangement in which the owner has the sole responsibility for the scope and design of the pile or pier installation and specifies the procedures that must be followed. Prescriptive specifications mandate the owner to be responsible for the proper performance of the production piles/piers. The contractor is responsible for fulfilling the obligations/details as specified in the construction documents.

Pretensioning - The prestressing of an anchor or foundation prior to the service load being applied.

Proof Test - The incremental loading of a pile or pier, where the load is held for a period of time and the total movement is recorded at each load increment. The maximum applied load is generally 1.0 to 1.25 times the design load.

Rebound - Waste created by sprayed concrete falling to the floor or ground below the intended target location. Rebound is usually half for shotcrete compared to gunite.

Reinforced Earth - A soil mass whose overall shear strength has been increased via some reinforcing technique (e.g., Soil Screw® Anchor, soil nail, geofabric, etc.).



Round Shaft - Hollow steel, round pipe, central shaft elements ranging in diameter from 2" to 10". Also known as Hollow Shaft, Round Shaft (Type RS), Type T/C and Type PIF for Chance® Helical Piles.

Safety Factor (SF) - The ratio of the ultimate capacity to the working or design load used for the design of any structural element. Also referred to as a factor of safety.

Seismic Load - A load induced on a structure caused by ground motions resulting from a seismic event (earthquake). usually included as part of the live load.

Shaft - A steel or composite steel/grout shaft or rod used to transfer load from the surface to the bearing plates.

Shotcrete - A wet concrete mixture that is pumped to a nozzle where air is added to carry the concrete mix to the application. often used to quickly provide a facing on soil nail or Soil Screw® Anchor reinforced retaining walls.

Soil Nail - A steel rod driven or drilled and grouted into the ground to reinforce, stabilize, or strengthen soil such as the soil mass behind a retaining wall.

Soil Screw Anchor - A Chance® Helical Anchor with helices welded along the entire length of the shaft. A Soil Screw® Anchor is used to engage the soil and serves the same function as a soil nail, i.e., soil reinforcement.

Soldier Pile - An H or WF section normally driven (or placed in a drilled hole and backfilled with weak grout or concrete) vertically at intervals of several feet to resist the load on the lagging of a retaining wall. It is the main structural element of a retaining wall. Also known as an h-pile.

Square Shaft (SS) - A solid steel, round-cornered-Square central Shaft element ranging in size from 1-1/4" to 2-1/4". Also known as Type SS for Chance® Helical Anchors.

Starter Pier Section - With reference to an Atlas Resistance® Pier, the first pipe section to be placed in the ground. It is usually equipped with a friction reduction collar.

Starter Section - With reference to a Chance® Helical Pile, a lead section, but usually used in reference to a Soil Screw® Anchor.

Test Load - The maximum load applied to a pile or pier during testing.

Thread Bar Adapter - A section of central steel shaft that can be used to connect a tiedown or ground anchor to a new or existing concrete foundation/pile cap via a high tensile strength pre-stressing thread bar.

Tieback - A tension anchor used to resist the loads on a retaining wall due to the earth pressure and other loads at or near the top of a wall.

Tiedown - A device used to transfer tensile loads to soil. Tiedowns are used for seismic retrofit. They consist of a central steel shaft, helix bearing plates, coatings, corrosion protection, a means of connection, etc. Also known as a ground anchor.

Top Pier Platform - The top section of an Atlas Resistance Pier equipped with vertical stabilizers that facilitate attachment to the pier bracket.

Torque Rating - The maximum torque energy that can be applied to a helical anchor/pile during installation in soil. Also known as allowable torque or safe torque.

Ultimate Capacity (Qu) - The limit state based on the structural and/or geotechnical capacity of a pile or pier, defined as the point at which no additional capacity can be justified.

Ultimate Load (Pu) - The load determined by applying a safety factor to the working load. The ultimate load applied to a structural element must be less than the ultimate capacity of that same element or a failure limit state may occur.

Underpinning Bracket - A bracket used to connect an existing strip or spread foundation or footing to a Chance Helical Pile or Atlas Resistance Pier.

Uplift Load - Generally, an axial tensile load on an anchor.

Verification Test - Similar to the Proof Test except a cyclic loading method is used to analyze total, elastic and net movement of the pile. used for pre-contract or pre-production pile load tests.

Vertical Stabilizer - A steel plate element, welded to the side of the top pier platform, which prevents lateral movement within the pier bracket. Vertical stabilizers will allow the pier bracket to move vertically up from the top pier platform but prevent the bracket from moving below a previously set elevation.

Waler - A horizontal structural member placed along soldier piles to accept the load from the piles and transmit it to struts, shoring or tieback anchors.

Working Load - Another term for Design Load.



SECTION 1: INTRODUCTION

CONTENTS

Atlas Resistance Piers1-2

- Definition of Atlas Resistance® Piers
- History Of Pushed Steel Pile Systems
- Applied Research and Development
- Testing and Code Compliance
- Applications
- Advantages of Atlas Resistance Piers

Helical Piles/Anchors 1-6

- Definition Of Helical Piles/Anchors
- History And Science Of Chance® Helical Piles/Anchors
- Applied Research and Development
- Applications
- Advantages of Helical Piles/Anchors

Bibliography 1-13

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 should be conducted and state and local building codes and authorities should be consulted 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 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.



ATLAS RESISTANCE PIERS



DEFINITION OF ATLAS RESISTANCE® PIERS

The Atlas Resistance pier utilizes the weight of the structure as its reaction system to drive or push the pier pipe 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 a 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 loaded to a force greater than that 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.

Work space 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. It 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 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 ‘60s and ‘70s, further extending the work of these early pioneers.

APPLIED RESEARCH AND DEVELOPMENT

The development of the Atlas Resistance pier system early in the 1980s created new opportunities for building owners to reclaim the hard-earned equity of their structure’s previously devalued 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, thus reducing the driving force 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 the 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

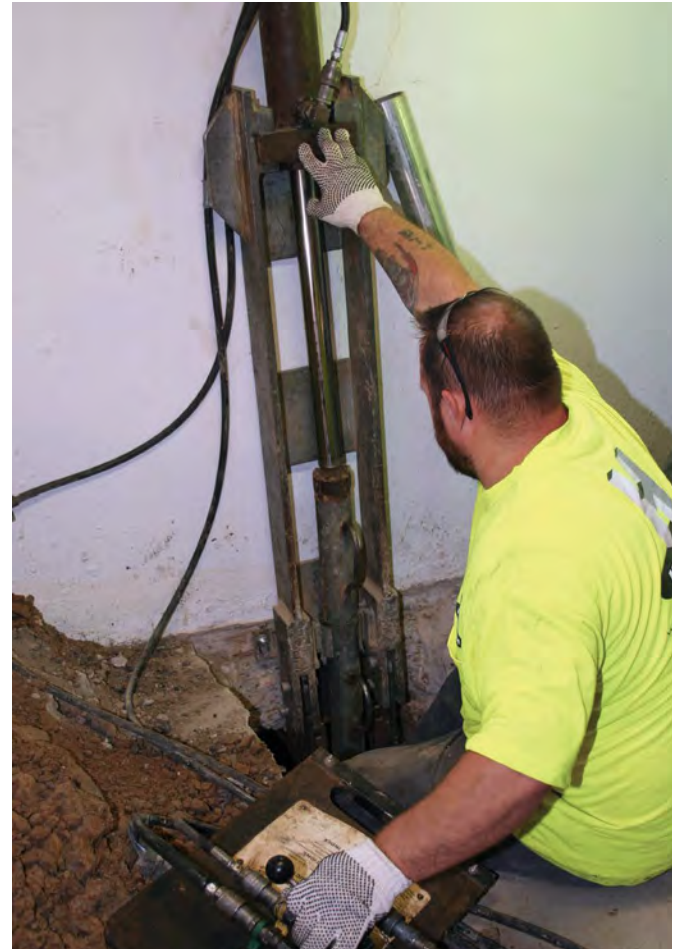


International (BOCA) National Code, International Conference of Building Officials (ICBO) Uniform Code, Southern Building Code Congress International (SBCCI) Standard Code, and the 2000 International Building and Residential Codes of the International Code Council (ICC) with the new 2002 Accumulative Supplement.

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 lightweight, 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.



APPLICATIONS

Atlas Resistance® piers are used primarily for underpinning and 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 structure. 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

ATLAS RESISTANCE® PIER ADVANTAGES, TABLE 1-1

SUMMARY OF ATLAS RESISTANCE PIER ADVANTAGES

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



INTRODUCTION



HELICAL PILES/ANCHORS

DEFINITION OF HELICAL PILES/ANCHORS

The helical pile/anchor is 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 Systems, Inc., defines "deep" as five helix diameters vertically below the surface, where the helical plate can develop the 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 size that results in the shaft material's allowable stress when the working load is applied. Realistically, the shaft also needs to be strong enough to resist the torque required for installation and large enough in section for the shaft 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. The termination's configuration is dictated 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.

Today, readily available hydraulic equipment, either small or large, can install helical piles/anchors almost anywhere. Backhoes,





skid-steer loaders, and mini-excavators are easily fitted with hydraulically driven torque motors to install helical piles/anchors in construction sites inaccessible by the larger equipment required for other deep foundation types. According to site conditions, installation equipment may be self-propelled, carrier-mounted, tracked, wheeled, or floating and may have a guided or articulated torque head.

The following is a summarized 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 for tension applications. It was installed by hand using a small tubular wrench. Other early developments include measurement devices for classifying soil.

PISA® (POWER INSTALLED SCREW ANCHORS)

In the late 1950s, 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 of electric and telephone utility poles.

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 eye nut for guy attachment, and a special installing wrench. The square-tube anchor wrench attaches to the Kelly bar of a digger truck, fits over the rod, and engages over the helical anchor hub. A PISA anchor can typically be installed in 8 to 10 minutes. Rod and wrench extensions may be added to reach soil layers which develop enough resistance to achieve the required capacity. PISA rods are offered 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 include 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 hollow socket hubs. Both use the standard PISA rods and 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 adequately 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 1-1/2" 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 was 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 1960s, Type HS anchors were first developed for high-torque guying requirements. They later were applied as foundation helical piles for utility substations and transmission towers. The HS anchor/pile family had 3-1/2" pipe shafts which could be lengthened by extensions with swaged couplings. HS anchors/piles now are used for a wide array of foundation applications. The Type HS anchors/piles are now referred to as Type RS piles. Hubbell now offers 2-7/8" (RS2875.203, RS2875.276, RS2875.276 HCP), 4-1/2" (RS4500.237, RS4500.337), 5-1/2" (RS5500.361), 6-5/8" (RS6625.280), 7" (RS7000.362), 8-5/8" (RS8625.250), and 9-5/8" (RS9625.395) pipe shafts in addition to the 3-1/2" (RS3500.300).

LARGE DIAMETER PIPE PILES (LDPP)

To meet an industry need for helical piles with higher tension/compression capacities and greater 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 90,000 ft-lb and compression capacities of 360 kip.

POWER INSTALLED FOUNDATION (PIF) PILES

Also launched in the 1960's were non-extendable piles termed Power Installed Foundations. PIF sizes and load capacities satisfy 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 3-1/2" shaft PIF, but with fence-type caps instead of base plates to serve as traffic barriers around booths, cabinets, doorways, etc.

STREET LIGHT FOUNDATION (SLF) PILES

In 1972, Chance Instant Foundations were introduced. Commonly referred to as Light Pole Bases or Street Light Foundations, piles with pipe shaft diameters of 6-5/8", 8-5/8", and 10-3/4" in fixed lengths of 5, 8, and 10 feet are available as standard designs. 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 used with Type SS helical piles, the 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 the installation of 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 resulting pile has 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.

APPLIED RESEARCH AND DEVELOPMENT

In addition to products developed for specific applications, significant contributions to the applied science of helical piles and

anchors have been made by Hubbell Power Systems, Inc. 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 anchor for a given load.

TORQUE-TO-CAPACITY RELATIONSHIPS

The relationship of installation torque to load capacity is an empirical method the A.B. Chance Company originally developed in the 1960s. 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. An analogy can be made 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 soft soil, and dense soil will generate higher load capacity compared to soft soil.

For the torque correlation method to work, torque must be measured. Hubbell engineers have developed both mechanical and electronic indicators over the years, some of which are commercially available for torque measurement in the field. The most recent addition to the product line is the C3031836 Torque Indicator, which features a continuous-reading digital display of installation torque up to 30,000 ft·lb. The Torque Indicator is used in conjunction with a wireless device app that displays real-time torque data and can log torque and other installation data for 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 via 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).

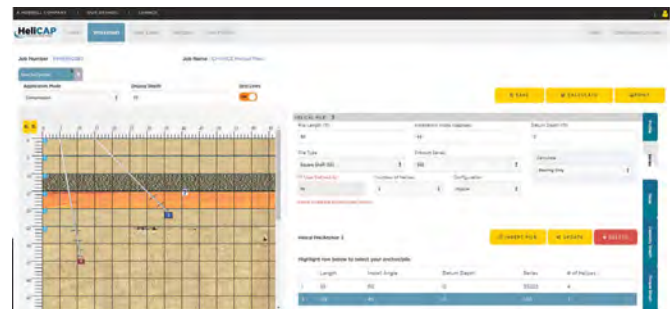
100+ YEARS OF FIELD TEST DATA

Hubbell has a long-standing practice of proving theory with load tests in the field. Hubbell engineers continue to build on the work of their predecessors, who conducted thousands of field tests throughout the decades. It has been said that soil occurs in infinite variety and 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 for typical capacities of helical piles/anchors in soil.

HELICAP® HELICAL CAPACITY DESIGN SOFTWARE

Hubbell engineers developed HeliCAP Helical Capacity Design Software that assists the designer in selecting the optimal helical lead configuration and overall pile/anchor length. It also estimates the installation torque. A proprietary engineering software for confident helical engineering, HeliCAP performs powerful calculations on site soil parameters to aid engineers designing foundations, tiebacks, soil nails, and anchors for heavy guy loads. The software gives prompts to maintain control over essential criteria and guides the user through the same process Hubbell application engineers employ daily to analyze problems and specify solutions.

Unlike previous versions of HeliCAP, version 3 is cloud based and can be instantly accessed from any web-connected device by visiting www.hpsapps.com/helicap.



CHANCE® CIVIL CONSTRUCTION SOIL CLASSIFICATION, TABLE 1-2

CLASS	COMMON SOIL TYPE DESCRIPTION	GEOLOGICAL SOIL CLASSIFICATION	PROBE VALUES* (ft-lb [in-lb] {N·m})	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)	63-134 [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	50-63 [600-750] {68-85}	45-60
3	Dense sands and gravel; hard silts and clays	Glacial till; weathered shale, schist, gneiss, and siltstone	42-50 [500-600] {57-68}	35-50
4	Medium-dense sand and gravel; very stiff to hard silts and clays	Glacial till; hardpan; marls	33-42 [400-500] {45-57}	24-40
5	Medium-dense coarse sands and sandy gravels; stiff to very stiff silts and clays	Saprolite; residual soil	25-33 [300-400] {34-45}	14-25
6	Loose to medium-dense fine to coarse sands; medium-stiff to stiff clays and silts	Dense hydraulic fill; compacted fill; residual soil	17-25 [200-300] {23-34}	7-14
7**	Loose fine sands; alluvium; loess; soft to medium-stiff clays; fill	Flood plain soil; lake clay; adobe; gumbo; fill	8-17 [100-200] {11-23}	4-8
8**	Peat; organic and inundated silts; fly ash; very loose sands; very soft to soft clays	Miscellaneous fill; swamp marsh	0-8 [0-100] {0-11}	0-5

Notes:

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

* Probe values are based on using the Chance Soil Test Probe.

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



SELECT-A BASE™ LIGHTING BASE PROGRAM

The SELECT-A BASE lighting base program is an online program developed in 2009 by Hubbell Power Systems, Inc., to assist engineers and designers in selecting appropriate Instant Foundations for their lighting and signage needs. 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 online at www.hpsapps.com/base.

INTERHELIX 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 engineers determined that the optimal 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 in ever-expanding roles in civil construction applications. However, you will probably not find helical piles/anchors mentioned in most foundation engineering textbooks, and familiarity with helical piles/anchors is still lacking among most civil and structural engineers with a foundation background. This situation is slowly changing. Helical piles are listed as a deep foundation system since the 2009 edition of the International Building Code (IBC). In addition, ICC-ES Acceptance Criteria AC358 for Helical Pile Systems and Devices was published in 2007 and has been

frequently revised since then. Hubbell was the first manufacturer of helical piles and anchors to obtain evaluation reports from all three model building code agencies: ICBO, BOCA, and SBCCI. ESR-2794 is an ICC-ES evaluation report that demonstrates code compliance with the IBC and the International Residential Code (IRC). ESR-2794 now includes seismic design categories D, E, and F. Copies of ICC-ES ESR-2794 Evaluation Reports are available on www.chancefoundationsolutions.com.

INSTRUCTOR'S CURRICULUM FOR FOUNDATION ENGINEERING COURSES

In 2012, Hubbell contracted with Dr. Alan Lutenecker 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. The Student Guide was updated in 2022.

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 kip (445 kN) in tension and 300 kip (1335 kN) 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, light commercial, and heavy commercial construction; machinery/equipment foundations; telecommunication and transmission towers; tie-downs for wind and/or seismic forces; and



CHANCE HELICAL PILE/ANCHOR ADVANTAGES, TABLE 1-3



SUMMARY OF CHANCE HELICAL PILE/ANCHOR ADVANTAGES	
<ul style="list-style-type: none"> • No need for concrete to cure • Quick, easy turnkey installation • Immediate loading • Small installation equipment • Pre-engineered system • Easily field modified • Torque-to-capacity relationship for production control • Install in any weather 	<ul style="list-style-type: none"> • Solution for: <ul style="list-style-type: none"> • Restricted-access sites • High water table • Weak surface soils • Environmentally friendly • No vibration • No spoils to remove

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 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 swell adhesion value and the surface area between the soil and the shaft. 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.





BIBLIOGRAPHY OF HELICAL PILE/ANCHOR TECHNICAL LITERATURE

Adams, J.I. and Hayes, D.C., 1967. The Uplift Capacity of Shallow Foundations. Ontario Hydro Research Quarterly, Vol. 19, No. 1, pp. 1-13.

Adams, J.I. and Klym, T.W., 1972. A Study of Anchors for Transmission Tower Foundations. Canadian Geotechnical Journal, Vol. 9, No. 1, pp. 89-104.

Black, D.R. and Pack, J.S., 2002. Design and Performance of Helical Screw Piles in Collapsible and Expansive Soils in Arid Regions of the United States. Proceedings of the 9th International Conference on Piling and Deep Foundations, pp. 469-476.

Bobbitt, D.W., and Clemence, S.P., 1987. Helical Anchors: Application and Design Criteria. Proceedings of the 9th Southeast Asian Geotechnical Conference, Vol. 2, pp. 6-105 - 6-120.

Bobbitt, D.E. and Thorsten, R., 1989. The Use of Helical Tieback Anchors for a Permanent Retaining Wall. Foundation Congress, ASCE.

Bradka, T.D., 1997. Vertical Capacity of Helical Screw Anchor Piles. M.S. Report, Geotechnical Group, Department of Civil Engineering, University of Alberta.

Bustamante, M. and Gianceselli, L., 1998. Installation Parameters and Capacity of Screwed Piles. Proceedings of the 3rd International Geotechnical Seminar on Deep Foundations on Bored and Auger Piles: BAP III, pp. 95-108.

Carville, C.A. and Walton, R.W., 1994. Design Guidelines for Screw Anchors. Proceedings of the International Conference on Design and Construction of Deep Foundations, Vol. 2, pp. 646-655.

Carville, C.A. and Walton, R.W., 1995. Foundation Repair Using Helical Screw Anchors. Foundation Upgrading and Repair for Infrastructure Improvement, ASCE, pp. 56-75.

Clemence, S.P., 1984. The Uplift and Bearing Capacity of Helix Anchors in Soil. Vols. 1, 2 & 3, Contract Report TT112-1 Niagra Mohawk Power Corporation, Syracuse, N.Y.

Clemence, S.P., 1994. Uplift Capacity of Helical Anchors in Soils. Proceedings of the 2nd Geotechnical Engineering Conference, Cairo, Vol. 1, pp. 332-343.

Clemence, S.P. and Pepe, F.D. Jr., 1984. Measurement of Lateral Stress

Around Multi-Helix Anchors in Sand. Geotechnical Testing Journal, Vol. 7, No. 3, pp. 145-152.

Clemence, S.P. and Smithling, A.P., 1984. Dynamic Uplift Capacity of Helical Anchors in Sand. Proceedings of the 4th Australia-New Zealand Conference on Geomechanics, Vol. 1, pp. 88-93.

Clemence, S.P., Thorsten, T.E., and Edwards, B., 1990. Helical Anchors: Overview of Application and Design. Foundation Drilling, Jan., pp. 8-12.

Clemence, S.P., Crouch, L.K., and Stephenson, R.W., 1994. Prediction of Uplift Capacity for Helical Anchors in Sand. Proceedings of the 2nd Geotechnical Engineering Conference, Cairo.

Cox, R., 1995. Alexander Mitchell and the Screw-Pile. Centre for Civil Engineering Heritage, Trinity College, Dublin, 14 pp.

Curle, R., 1995. Screw Anchors Economically Control Pipeline Bouyancy in Muskeg. Oil and Gas Journal, Vol. 93, No. 17.

Das, B.M., 1990. Earth Anchors. Elsevier Science Publishers, Amsterdam, 241 p.

Deardorff, D. A., 2007. Torque Correlation Factors for Round Shaft Helical Piles. Deep Foundations Institute Symposium on Helical Pile Foundations, Nov., 2007, 20 pp.

Deardorff, D. and Luna, R., 2009. LRFD for Helical Piles: An Overview. ASCE Geotechnical Special Publication No. 185, Contemporary Topics in Deep Foundations IFCEE09, March 2009, p. 480.

Downey, S., 2003. Helical Piles with Grouted Shafts - a Case History. Proceedings of 28th Annual Conference on Deep Foundations, Deep Foundations Institute, pp. 291-298.

Engineering News, 1903. The Pennsylvania Railroad Tunnel Under the North River, at New York City. Oct. 15, pp. 336-341.

Engineering News, 1915. A Submerged Pump Crib Pinned Down with Screw Piles. March 18, p. 529.

Engineering News Record, 1948. Screw Piles Support Turkish Pier. Jan. 8, p. 99.

The Engineering Record, 1906. The Cienfuegos Screw Pile Pier. Jan. 20, p. 80.

Engineering Record, 1912. Steel Screw Piles, Feb. 17, p. 181.

Fabre, R., 2005. Behavior of Helical Screw Piles in Clay and Sand, M.S. Thesis, University of Massachusetts, Amherst, Ma.

Feld, J., 1953. A Historical Chapter: British Royal Engineers' Papers on Soil Mechanics and Foundation Engineering, 1937-1974. Geotechnique, Vol.3, pp. 242-247.

Ghaly, A.M., 1995. Drivability and Pullout Resistance of Helical Units in Saturated Sands. Soils and Foundations, Vol. 35, No. 2, pp. 61-66.



- Ghaly, A.M., 1996. closure to Drivability and Pullout Resistance of Helical Units in Saturated Sands. *Soils and Foundations*, Vol. 36, No. 2, pp.139-141.
- Ghaly, A.M. and Clemence, S.P., 1998. Pullout Performance of Inclined Helical Screw Anchors in Sand. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 124, No. 7, pp. 617-627.
- Ghaly, A.M. and Clemence, S.P., 1999. closure to Pullout Performance of Inclined Helical Screw Anchors in Sand. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 125, No. 12, pp. 1102-1104.
- Ghaly, A.M. and Hanna, A.M., 1991. Experimental and Theoretical Studies on Installation Torque of Screw Anchors. *Canadian Geotechnical Journal*, Vol. 28, No. 3, pp. 353-364.
- Ghaly, A.M. and Hanna, A.M., 1991. Stress Development in Sand Due To Installation and Uplifting of Screw Anchors. *Proceedings of the 4th International Conference on Piling and Deep Foundations*, Vol. 1, pp. 565-570.
- Ghaly, A.M. and Hanna, A.M., 1992. Stress and Strains Around Helical Screw Anchors in Sand. *Soils and Foundations*, Vol. 32, No. 4, pp. 27-42.
- Ghaly, A.M. and Hanna, A.M., 1994. Model Investigation of the Performance of Single Anchors and Groups of Anchors. *Canadian Geotechnical Journal*, Vol. 31, No. 2, pp. 273-284.
- Ghaly, A.M. and Hanna, A., 1994. Ultimate Pullout Resistance of Single Vertical Anchors. *Canadian Geotechnical Journal*, Vol. 31, No. 5, pp. 661-672.
- Ghaly, A.M. and Hanna, A., 1994. Ultimate Pullout Resistance of Groups of Vertical Anchors. *Canadian Geotechnical Journal*, Vol. 31, No. 5, pp. 673-682.
- Ghaly, A.M. and Hanna, A., 1995. closure to Ultimate Pullout Resistance of Single Vertical Anchors. *Canadian Geotechnical Journal*, Vol. 32, No. 6, pp. 1093-1094.
- Ghaly, A.M. and Hanna, A., 2003. Response of Anchors to Variations in Displacement-Based Loading. *Canadian Geotechnical Journal*, Vol. 40, No. ?, pp. 694-701.
- Ghaly, A.M., Hanna, A.M. and Hanna, M.S., 1991. Uplift Behavior of Screw Anchors in Sand - I: Dry Sand. *Journal of Geotechnical Engineering*, ASCE, Vol. 117, No. 5, pp. 773-793.
- Ghaly, A.M., Hanna, A.M. and Hanna, M.S., 1991. Uplift Behavior of Screw Anchors in Sand - II: Hydrostatic and Flow Conditions. *Journal of Geotechnical Engineering*, ASCE, Vol. 117, No. 5, pp. 794-808.
- Ghaly, A., Hanna, A., and Hanna, M., 1991. Installation Torque of Screw Anchors in Dry Sand. *Soils and Foundations*, Vol. 31, No. 2, pp. 77-92.
- Ghaly, A.M., Hanna, A.M. and Hanna, M.S., 1991. Uplift Behavior of Screw Anchors in Sand - I: Dry Sand. *Journal of Geotechnical Engineering*, ASCE, Vol. 117, No. 5, pp. 773-793.
- Ghaly, A., Hanna, A., Ranjan, G. and Hanna, M., 1991. Helical Anchors in Dry and Submerged Sand Subjected to Surcharge. *Journal of Geotechnical Engineering*, ASCE, Vol. 117, No. 10, pp. 1463-1470.
- Ghaly, A., Hanna, A., Ranjan, G. and Hanna, M., 1993. closure to Helical Anchors in Dry and Submerged Sand Subjected to Surcharge. *Journal of Geotechnical Engineering*, ASCE, Vol. 119, No. 2, pp. 392-394.
- Gunnink, Brett; Gammon, Scott; Barker, Michael; Berry, Ron, 1995. A Finite Element Approach to the Buckling Behavior of Helical Soil Piers. *Journal of Engineering Mechanics*, ASCE.
- Hanna, A. and Ghaly, A., 1992. Effects of K_0 and Overconsolidation on Uplift Capacity. *Journal of Geotechnical Engineering*, ASCE, Vol. 118, No. 9, pp. 1449-1469.
- Hanna, A. and Ghaly, A., 1994. Ultimate Pullout Resistance of Groups of Vertical Anchors. *Canadian Geotechnical Journal*, Vol. 31, No. 5, pp. 673-682.
- Hargrave, R.L. and Thorsten, R.E., 1992. Helical Piers in Expansive Soils of Dallas, Texas. *Proceedings of the 7th International Conference on Expansive Soils*.
- Haskew, B.B., 1930. The Rebuilding of the Bassein Bridges on the Bombay, Baroda and Central India Railway. *Minutes of the Proceedings of the Institution of Civil Engineers*, Vol. 230, pp. 204-233.
- Hawkins, K. and Thorsten, R. 2009. Load Test Results-Large Diameter Helical Pipe Piles. *ASCE Geotechnical Special Publication No. 185, Contemporary Topics in Deep Foundations*, IFCEE09, March 2009, p. 488.
- Herrod, H., 1930. Screw-Piling, with Particular Reference to Screw-Piles Sewage Sea Outfall Works. *Selected Engineering Paper No. 94*, The Institution of Civil Engineers, 23 pp.
- Hovland, H.J., 1993. discussion of Helical Anchors in Dry and Submerged Sand Subjected to Surcharge. *Journal of Geotechnical Engineering*, ASCE, Vol. 119, No. 2, pp. 391-392.
- Hoyt, R.M. and Clemence, S.P., 1989. Uplift Capacity of Helical Anchors in Soil. *Proceedings of the 12th International Conference on Soil Mechanics and Foundation Engineering*, Vol. 2, pp. 1019-1022.
- Hoyt, R.M., Seider, G., Reese, L.C., and Wang, S.T., 1995. Buckling of Helical Anchors Used for Underpinning. *Foundation Upgrading and Repair for Infrastructure Improvement*, ASCE, pp. 89-108.
- Huang, F.C., Mohmood, I., Joolazadeh, M., and Axten, G.W., 1995. Design Considerations and Field Load Tests of a Helical Anchoring System for Foundation Renovation. *Foundation Upgrading and Repair for Infrastructure Improvement*, ASCE, pp. 76-88.
- Jacobs, C.M., 1910. The New York Tunnel Extension of the Pennsylvania Railroad. *Transactions of the American Society of Civil Engineers*, Vol. 68, pp. 40-56.
- Jennings, R. and Bobbitt, D., 2003. Helical Pulldown Micropiles Support Museum Celebrating the Bicentennial of the Lewis and Clark Expedition. *Proceedings of 28th Annual Conference on Deep Foundations*, DFI, pp. 285-290.
- Johnston, G. H. and Ladanyi, B., 1974. Field Tests of Deep Power-Installed Screw Anchors in Permafrost. *Canadian Geotechnical Journal*, Vol. 11, No. 3, pp. 348-358.
- Johnston, R.J., Swanston, D.N. Baxandall, F.W., 1999. Helical Piling Foundations in Juneau, Alaska. *Cold Regions Engineering: Putting Research into Practice 1999*.
- Khatri, D. and Stringer, S., 2003. Helical Pile Foundation Anchors as a Practical Alternative. *Proceedings of 28th Annual Conference on Deep Foundations*, DFI, pp. 299-308.
- Klosky, J.L., Sture, S., Hon-Yim Ko, H.Y. and Barnes, F., 1998. Helical Anchors for Combined Anchoring and Soil Testing in Lunar Operations. *Space 98* ASCE.
- Kennedy, D., 1930. Construction of Screw-Pile Jetty at Bhavnagar. *Selected Engineering Paper No. 95*, The Institution of Civil Engineers, 13 pp.
- Khatri, D. and Stringer, S., 2003. Helical Pile Foundation Anchors as a Practical Alternative. *Proceedings of 28th Annual Conference on Deep Foundations*, DFI, pp. 299-308.
- Klym, T.W., Radhakrishna, H.S., and Howard, K., 19???. Helical Plate Anchors for Tower Foundations. *Proceedings of the 25th Canadian Geotechnical Conference*, pp. 141-159.
- Kraft, D.C., Davis, J. And Raaf, D.B., 2003. Use of Helical Piles Set into Soft Rock for 1500-Ton Screw Press Foundation. *Proceedings of 28th Annual Conference on Deep Foundations*, DFI, pp. 209-218.
- Kumar, J., 1995. discussion of Ultimate Pullout Resistance of Single Vertical Anchors. *Canadian Geotechnical Journal*, Vol. 32, No. 6, p. 1093.
- Levesque, C.L., Wheaton, D.E. and Valsangkar, A.J., 2003. Centrifuge Modeling of Helical Anchors in Sand. *Proceedings of the 12th Panamerican Conference on Soil Mechanics and Foundation Engineering*, Vol. 2, pp. 1859-1863.
- Liu, H., Zubeck, H., and Baginski, S., 1999. Evaluation of Helical Piers in Frozen Ground. *Cold Regions Engineering: Putting Research into Practice 1999*.
- Livneh, B. and El Naggar, M.H., 2007. Axial Load Testing and Numerical Modeling of Square Shaft Helical Piles. *Canadian Geotechnical Journal*.
- Lutenegger, A.J., Smith, B.L. and Kabir, M.G., 1988. Use of In Situ Tests to Predict Uplift Performance of Multi-Helix Anchors. *Special Topics in Foundations*, ASCE, pp. 93-110.
- Lutenegger, A.J. and Kempker, J.H., 2008. Preservation of Historic Structures Using Screw-Pile Foundations. *Proceedings of the 6th International Conference on Structural Analysis of Historic Constructions*, Vol. 2, pp 1079-1086.
- Lutenegger, A.J. and Kempker, J.H., April 2009. History Repeats, Screw Piles Come of Age - Again, *Structural Engineer Magazine*.
- Lutenegger, A.J., 2008. Tension Tests on Single-Helix Screw-Piles in Clay. *Proceedings of the 2nd British Geotechnical Association International Conference on Foundations*, Dundee, Scotland.
- Lutenegger, A.J., 2009. Cylindrical Shear of Plate bearing? - Uplift Behavior of Multi-Helix Screw Anchors in Clay.
- Lutenegger, A.J., 2010. Using Helical Screw-Piles for Upgrading Existing Foundations for Urban Regeneration.
- Lutenegger, A.J., September 2010. Shaft Resistance of Grouted Helical Micropiles in Clay. *Proceedings of the International Workshop on Micropiles*, Washington, D.C.
- Lutenegger, A.J., January 2011. Historical Development of Iron Screw-Pile Foundations: 1836-1900. *International Journal for the History of Eng. & Tech.*, Vol. 81, No. 1, pp. 108-128.
- Lutenegger, A.J., June 2011. Behavior of Grouted Shaft Helical Anchors in Clay. *DFI Journal*, Vol. 5, No. 5.



- Lutenegeger, A.J., November 2012. Discussion of "Ultimate Uplift Capacity of Multiplate Helical Type Anchors in Clay" by R.S. Merifield, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE.
- McDonald, J.K., 1999. discussion of Pullout Performance of Inclined Helical Screw Anchors in Sand. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 125, No. 12, p. 1102.
- Mitsch, M.P. and Clemence, S.P., 1985. The Uplift Capacity of Helix Anchors and Sand. Uplift Behavior of Anchor Foundations in Soil, ASCE, pp. 26-47.
- Mooney, J.S., Adamczak, S.Jr., and Clemence, S.P., 1985. Uplift Capacity of Helix Anchors in Clay and Silt. Uplift Behavior of Anchor Foundations in Soil, ASCE, pp. 48-72.
- Morgan, H.D., 1944. The Design of Wharves on Soft Ground. *Journal of the Institution of Civil Engineers*, Vol. 22, pp. 5-25.
- (discussions by F.E. Wentworth-Shields, C.W. Knight, F.M.G. Du-Plat-Taylor, J.S. Wilson, L.F. Cooling, S. Packshaw, A.W. Skempton, G.P. Manning, J. Bickley, J.E.G. Palmer, and L.Turner, pp. 25-45.)
- Muiden, M.A., 1926. Screw-Pile Mooring-Berths. Selected Engineering Papers No. 37, The Institution of Civil Engineers, 14 pp.
- Narasimha Rao, .S., Prasad, Y.V.S.N., Shetty, M.D. and Joshi, V.V., 1989. Uplift Capacity of Screw Pile Anchors. *Geotechnical Engineering*, Vol. 20, No. 2, pp. 139-159.
- Narasimha Rao, S., Prasad, Y.V.S.N., and Prasad, C.V., 1990. Experimental Studies on Model Screw Pile Anchors. Proceedings of the Indian Geotechnical Conference, Bombay, pp. 465-468.
- Narasimha Rao, S., Prasad, Y.V.S.N. and Shetty, M.D., 1991. The Behavior of Model Screw Piles in Cohesive Soils. *Soil and Foundations*, Vol. 31, No. 2, pp. 35-50.
- Narasimha Rao, S. and Prasad, Y.V.S.N., 1993. Estimation of Uplift Capacity of Helical Anchors in Clays. *Journal of Geotechnical Engineering*, ASCE, Vol. 119, No. 2, pp. 352-357.
- Narasimha Rao, S., Prasad, Y.V.S.N. and Veeresh, C., 1993. Behavior of Embedded Model Screw Anchors in Soft Clays. *Geotechnique*, Vol. 43, No. 4, pp. 605-614.
- Narasimha Rao, S. and Prasad, Y.V.S.N., 1992. discussion of Uplift Behavior of Screw Anchors in Sand. I: Dry Sand. *Journal of Geotechnical Engineering*, ASCE, Vol. 118, No. 9, pp. 1474-1476.
- Nasr, M.H., 2004. Large Capacity Screw Piles. Proceedings of the International Conference on Future Vision and Challenges for Urban Development, Cairo, Egypt.
- Pack, J.S., 2000. Design of Helical Piles for Heavily Loaded Structures. *New Technological and Design Developments in Deep Foundations*, ASCE, pp. 353-367.
- Pack, J.S., 2003. Helical Foundation and Tiebacks: Quality Control, Inspection and Performance Monitoring. Proceedings of 28th Annual Conference on Deep Foundations, DFI, pp. 269 - 284.
- Pack, J.S. and McNeill, K.M., 2003. Square Shaft Helical Screw Piles in Expansive Clay Areas. Proceedings of the 12th Panamerican Conference on Soil Mechanics and Foundation Engineering, Vol. 2, pp. 1825-1832.
- Perko, H.A., 2000. Energy Method for Predicting the Installation Torque of Helical Foundations and Anchors. *New Technological and Design Developments in Deep Foundations*, ASCE, pp. 342-352.
- Perko, H.A., 2003. Lateral Capacity and Buckling Resistance of Helix Foundations. *Foundations Technology Seminar*, DFI, University of Cincinnati.
- Perko, H.A., 2004. Introduction to Corrosion and Galvanizing of Helix Foundations. *Deep Foundations Institute Specialty Seminar on Helical Foundations and Tiebacks*, Tampa, Florida, 7 pp.
- Prasad, Y.V.S.N. and Narasimha Rao, S., 1994. Pullout Behavior of Model Piles and Helical Pile Anchors Subjected to Lateral Cyclic Loading. *Canadian Geotechnical Journal*, Vol. 31, No. 1, pp. 110-119.
- Prasad, Y.V.S.N. and Narasimha Rao, S., 1996. Lateral Capacity of Helical Piles in Clays. *Journal of Geotechnical Engineering*, ASCE, Vol. 122, No. 11, pp. 938-941.
- Prasad, Y.V.S.N., 1996. discussion of Drivability and Pullout Resistance of Helical Units in Saturated Sands. *Soils and Foundations*, Vol. 36, No. 2, p. 139.
- Puri, V.K., Stephenson, R.W., Dziedzic, E. and Goen, L., 1984. Helical Anchor Piles Under Lateral Loading. *ASTM STP 835*, pp. 194-213.
- Rabaler, R.C., 1989. Soil Corrosion Evaluation of Screw Anchors. *ASTM STP 1013*, pp.
- Radhakrishna, H.S., 1975. Helix Anchor Tests in Stiff Fissured Clay. Ontario Hydro Research Division Research Report.
- Radhakrishna, H.S., 1976. Helix Anchor Tests in Sand. Ontario Hydro Research Division Research Report 76-130-K, pp. 1-33.
- Robinson, K.E. and Taylor, H., 1969. Selection and Performance of Anchors for Guyed Transmission Towers. *Canadian Geotechnical Journal*, Vol. 6, pp. 119-135.
- Rodgers, T.E. Jr., 1987. High Capacity Multi-Helix Screw Anchors for Transmission Line Foundations. *Foundation for Transmission Line Towers*, ASCE, pp. 81-95.
- Rupiper, S. and Edwards, W.G., 1989. Helical Bearing Plate Foundations for Underpinning. *Foundation Engineering: Current Principles and Practices*, ASCE, Vol. 1, pp. 221-230.
- Rupiper, S., 1994. Helical Plate Bearing Members, A Practical Solution to Deep Foundations. Proceedings of the International Conference on the Design and Construction of Deep Foundations, Vol. 2, pp. 980-991.
- Scientific American, 1904. Driving a Test Pile for the Hudson River Tunnel. April 23, p. 324.
- Schmidt, R. and Nasr, M., 2004. Screw Piles: Uses and Considerations. *Structure Magazine*, June, pp. 29-
- Seider, G.L., 1993. Eccentric Loading of Helical Piers for Underpinning. Proceedings of the 3rd International Conference on Case Histories in Geotechnical Engineering, Vol. 1, pp. 139-145.
- Seider, G. L., 2000. Versatile Steel Screw Anchors. *Structural Engineer*, March.
- Seider, G. L., 2004. Helical Foundations: What the Engineer Needs to Know. *Structure Magazine*, June, pp. 27-28.
- Seider, G.L. and Smith, W.P., 1995. Helical Tieback Anchors Help Reconstruct Failed Sheet Pile Wall. Proceedings of the 45th Highway Geology Symposium, Charleston, W.V.
- Seider, G.L., Thorsten, R. E., and Clemence, S.P., 2003. Helical Piles with Grouted Shafts - A Practical Overview. Proceedings of 28th Annual Conference on Deep Foundations, DFI, pp. 219-232.
- Shaheen, W.A., 1985. The Behavior of Helical Anchors in Soil. M.S. Thesis, Department of Civil Engineering, University of Massachusetts, Amherst, Ma.
- Shaheen, W.A. and Demars, K.R., 1995. Interaction of Multiple Helical Earth Anchors Embedded in Granular Soil. *Marine Georesources and Geotechnology*, Vol. 13, pp. 357-374. Tench, R., 1944. Cast Iron Piles Screw-Driven to Rock. *Engineering News-Record*, December 28, pp. 60-61.
- Trofimenkov, J.G. and Maruipolshii, L.G., 1964. Screw Piles as Foundations of Supports and Towers of Transmission Lines. *Soil Mechanics and Foundation Engineering*, (Osnovaniya Fundamenty I Mekhanika Gruntov), Vol. 1, No. 4, pp. 232-239.
- Trofimenkov, J.G. and Maruipolshii, L.G., 1965. Screw Piles Used for Mast and Tower Foundations. Proceedings of the 6th International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, pp. 328-332.
- Udwari, J.J., Rodgers, T.E., and Singh, H., 1979. A Rational Approach to the Design of High Capacity Multi-Helix Screw Anchors. Proceedings of the 7th Annual IEEE/PES, Transmission and Distribution Exposition, pp. 606-610.
- Vickars, R.A. and Clemence, S.P., 2000. Performance of Helical Piles with Grouted Shafts. *New Technological and Design Developments in Deep Foundations*, ASCE, pp. 327-341.
- Weech, C.N., 2002. Installation and Testing of Helical Piles in a Sensitive Fine-Grained Soil. M.S. Thesis, Dept. Of Civil Engineering, University of British Columbia.
- Weikart, A.M. and Clemence, S.P., 1987. Helix Anchor Foundations - Two Case Histories. *Foundations for Transmission Line Towers*, ASCE, pp. 72-80.
- White, B.G., 1949. The Construction of Military Ports in Gareloch and Loch Ryan. *Civil Engineering and Public Works Review*, Vol. 44, No. 514, pp. 212-216.
- Wilson, G., 1950. The Bearing Capacity of Screw Piles and Screwcrete Cylinders. *Journal of the Institution of Civil Engineers - London*, Vol. 34, No. 5, pp. 4-73.
- (discussions by H.D. Morgan, A.W. Skempton, J. Bickley, C.C. Marshall, G.G. Meyerhof, P.A. Scott, D.H. Little, N.S. Boulton, and G. Wood, pp. 74-93. also discussions by A.S.E. Ackermann, F.L. Cassel, W.T. Marshall, P.W. Rowe, G.P. Tschebotarioff, R.J.C. Tweed, R. Pavry, R.E. Gibson, and A.A. Yassin, *Journal of the Institution of Civil Engineers-London*, Vol. 34, pp. 374-386.)
- Yokel, F.Y., Chung, R.M., and Yancey, C.W.C., 1981. NBS Studies of Mobil Home Foundations. U.S. National Bureau of Standards Report NBSIR 81-2238.
- Zhang, D. J. Y., 1999. Predicting Capacity of Helical Screw Piles in Alberta Soils. M.S. Thesis University of Alberta, Edmonton, Canada.
- Zubeck, H. and Liu, H. 2000. Helical Piers in Frozen Ground. Proceedings of the 3rd International Workshop on Micropiles, Turku Finland, Tampere University of Technology, Geotechnical Laboratory Publication No. 4



SECTION 2: SOIL MECHANICS

CONTENTS

Introduction.....2-2

Soil Mechanics.....2-2

Site Investigations.....2-7

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 should be conducted and state and local building codes and authorities should be consulted 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 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.

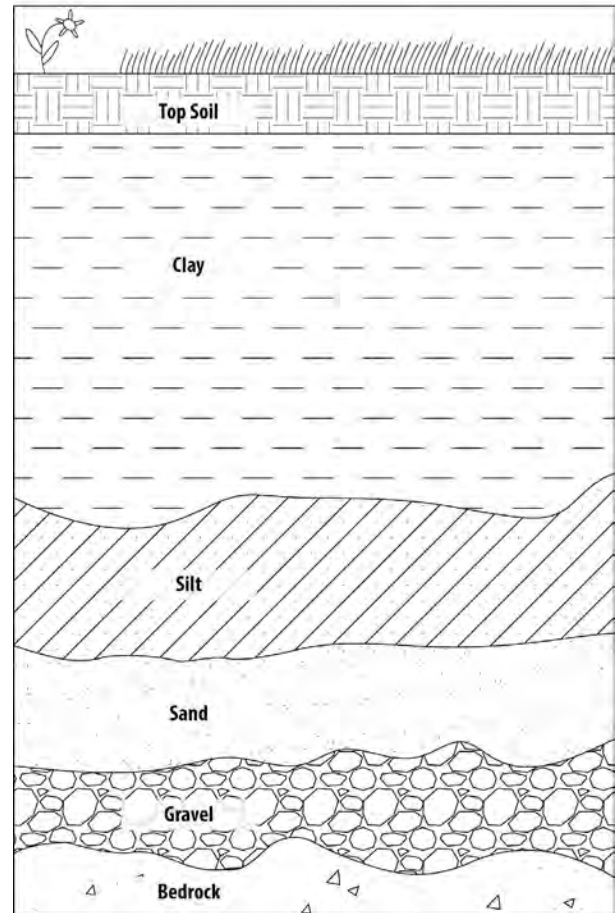
THE SOIL PROFILE

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.

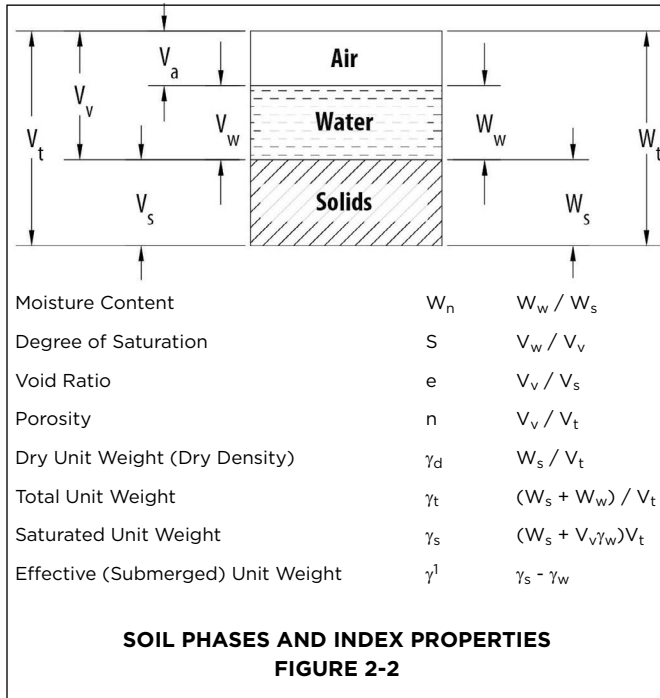
Deeper layers will have varying suitability depending on their properties and location. It is important to relate engineering properties to individual soil layers in order for the data to be meaningful. If data from several layers of varying strength are averaged, the result can be misleading and meaningless.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



**GENERALIZED SOIL PROFILE
FIGURE 2-1**

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.



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 as 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.

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



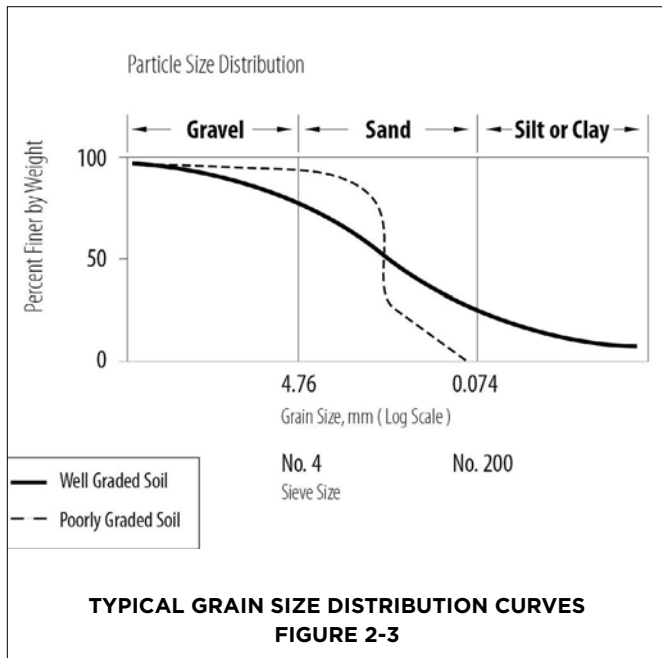
particles to be less than 0.001 mm (colloidal size). Fine-grained soils are sometimes referred to as cohesive soils. The particles of cohesive soils tend to stick together due to molecular attraction.

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

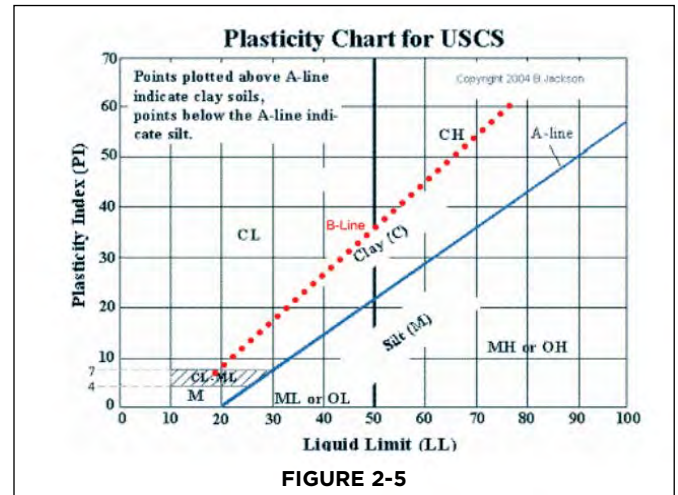
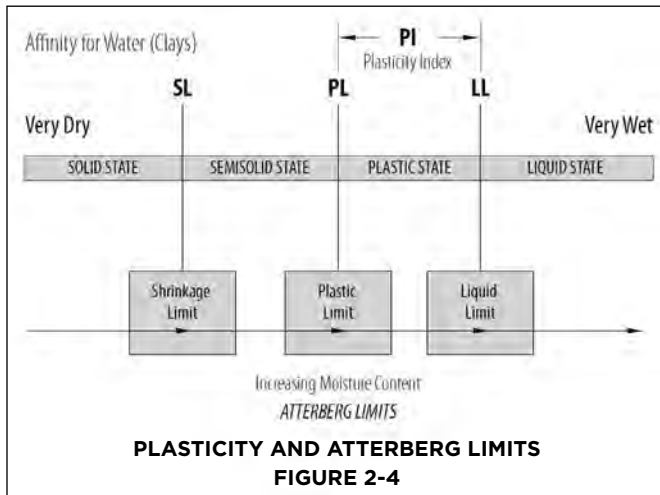
The consistency of fine-grained soils can range from a dry solid condition to a liquid form with successive addition of water and mixing as necessary to expand pore space for acceptance of water. The consistency passes from solid to semi-solid to plastic solid to viscous liquid as shown in Figure 2-4. 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.

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 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 K_t 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 $S_t = s_{u,und}/s_{u,rem}$. 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^o 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 deposits in a salt water environment and subsequently subjected to flushing by fresh water. Typical values of soil sensitivity are shown in Table 2-2.

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

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.

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 gravelly 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.

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



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 fine 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.

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:

EQUATION 2-1

$$\sigma' = \sigma - u$$

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:

EQUATION 2-2

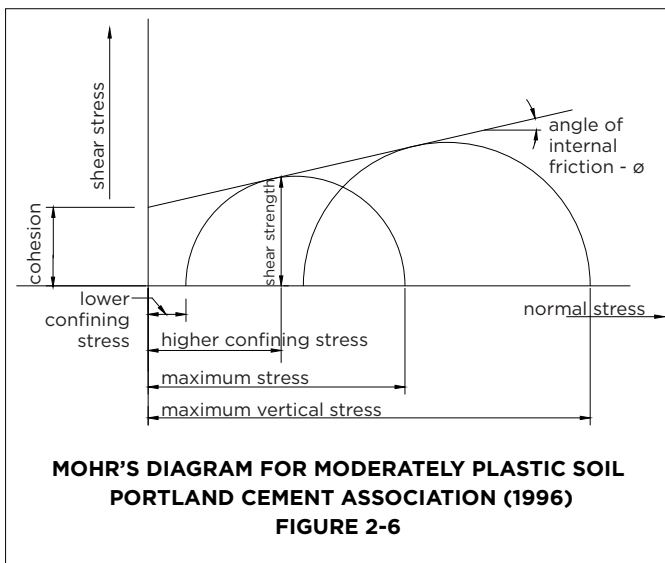
$$\tau = \sigma \tan \phi$$

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.

EQUATION 2-3

$$\tau_f = c + \sigma \tan \phi$$

In terms of effective stress:

EQUATION 2-4

$$\tau_f = c' + (\sigma - u) \tan \phi'$$

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.
- 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



**AUGER DRILLING OPERATION
FIGURE 2-7**

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 tie-backs (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 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

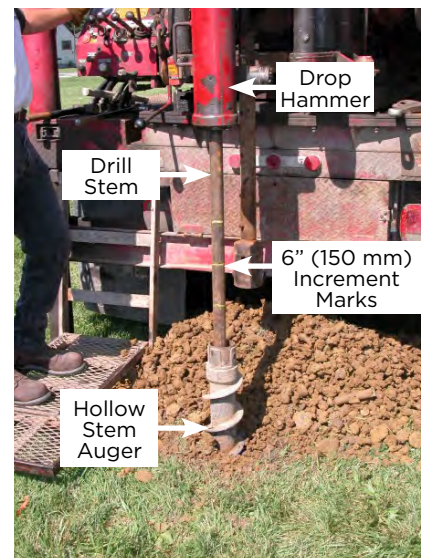


FIGURE 2-8



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.

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 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.

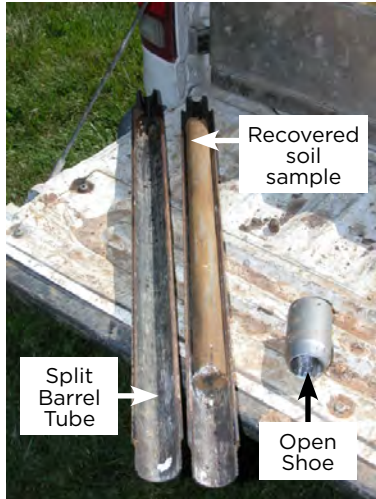
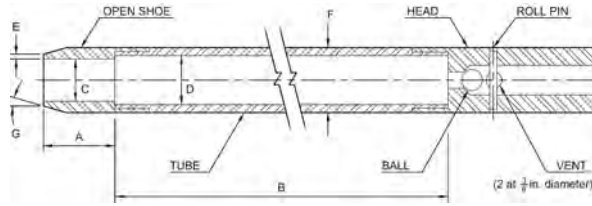
UNDISTURBED SAMPLES

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 cm². 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



- 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°

Split Spoon Sample

The $1\frac{1}{2}$ (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.

**GEOMETRY OF STANDARD PENETRATION TEST
SPLIT-BARREL SAMPLER (ASTM D 1586)
FIGURE 2-9**



**CPT/CPTU
FIGURE 2-10**



FIGURE 2-11



FIGURE 2-12

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.

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 by 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.

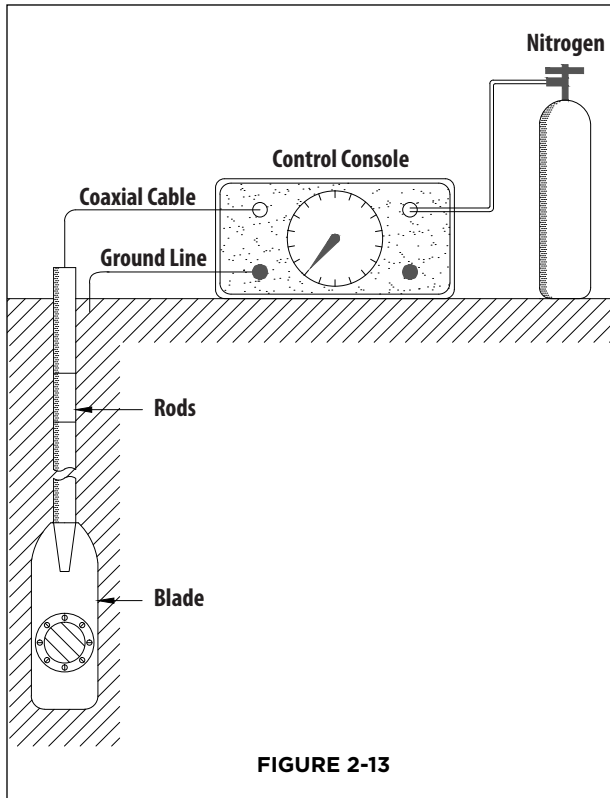
DILATOMETER TEST (DMT)

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 2-12. 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.


FIGURE 2-13
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 (%)	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:

EQUATION 2-5

$$s_u = (0.273T)/D^3$$

Vanes are available in 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:

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

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.
- 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 “ q_u ”. 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.

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.



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

SOIL MECHANICS

SUBSURFACE PROFILE				SAMPLE				Standard Penetration Test blows/ft.	Water Content % Wp ----- WI	
Depth (ft.)	Symbol	Description	Qp, T.S.F.	Dry Density, P.C.F.	Depth/Elev.	Number	Type			Blows/ft.
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		29
		(SP-SM), Trace Gravel, Fine to Coarse, Medium Dense, Dry Blow Sequence = 10-13-14 Recovery = 18"				2	SS	27		27
		(SP-SM), Fine to Medium, Medium Dense, Moist Blow Sequence = 8-8-7 Recovery = 16"				3	SS	15		15
		(SP-SM), Fine to Medium, Medium Dense, Moist Blow Sequence = 3-5-5 Recovery = 19"				4	SS	10		10
		(SP-SM), Fine to Medium, Loose, Moist Blow Sequence = 2-4-4 Recovery = 17"				5	SS	8		8
		(SP-SM), Fine to Medium, Medium Dense, Moist Blow Sequence = 3-6-6 Recovery = 18"				6	SS	12		12
		(SP-SM), Fine to Medium, Loose, Moist Blow Sequence = 2-3-4 Recovery = 15"				7	SS	7		7
		(SP-SM), Fine to Medium, Loose, Moist/Wet Blow Sequence = 1-2-3 Recovery = 15"				8	SS	5		5
		End of Boring @ 31½ Ft.			68.5 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



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



SAMPLE BORING LOG IN FINE-GRAINED SOIL, TABLE 2-7

Project No.: 09-1219	Boring Log	Rig: CME 55
Project: Mexico and Eaton Dam Drill Sites		Location: Mexico/Park Hills, Missouri
Client: Chance Civil Construction		Driller: MAS
Boring No.: Mexico		

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

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



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

SOIL MECHANICS



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 identified 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 $PI \geq 25$ 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.

REFERENCES

- Bowles, Joseph E., *Foundation Analysis and Design*, Fourth Edition, McGraw Hill, 1988.
- Chapel, Thomas A. (1998), *Field Investigation of Helical and Concrete Piers in Expansive Soil*, Proceedings of the Second International Conference on Unsaturated Soils (UNSAT 1998) Beijing, China.
- Hough, B.K., *Basic Soils Engineering*, Second Edition, Ronald Press Co., NY, 1969.
- Portland Cement Association, *PCA Soil Primer*, 1992.
- Spangler, Merlin G. and R.L. Handy, *Soil Engineering*, Fourth Edition, Harper and Row Publishers, NY, 1982.
- Terzaghi, Karl., *Theoretical Soil Mechanics*, John Wiley and Sons, NY, 1943.
- Terzaghi, Karl, R.B. Peck and G. Mesri, *Soil Mechanics in Engineering Practice*, Third Edition, John Wiley and Sons, NY, 1996.
- Weech, C. N., *Installation and Load Testing of Helical Piles in a Sensitive Fine-Grained Soil*, Thesis in Partial Fulfillment for Masters Degree, University of British Columbia, Vancouver, B.C., 2002.



SECTION 3: PRODUCT FEASIBILITY

CONTENTS

Feasibility Of Using Chance® Helical or Atlas Resistance® Pier Products.....3-2

Shaft Size Selection Based on Soil Parameters 3-4

Preliminary Chance Helical Pile/Anchor and Atlas Resistance® Pier Design Guide..... 3-4

PRODUCT FEASIBILITY

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 should be conducted and state and local building codes and authorities should be consulted 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 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.



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:

1. The most common selection of soil parameters for design is from field testing using the ASTM D1586 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.
2. 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.
3. 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 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 engineering department. The completed form can be sent to Hubbell or directly to your local Chance® Distributor.

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

Hubbell 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 compression 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 compression 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, riverine (terraces or delta) and lacustrine soil 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 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, SS175, RS2875.203, RS2875.276, RS3500.300, SS175/RS3500 Combo, and RS4500.377 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.chancefoundationsolutions.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 ₆₀ -VALUE Clay*	MAX N ₆₀ -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)	7,000 (9,491)	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)	25,000 (33,900)	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 D1586

Figure 3-1 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.

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

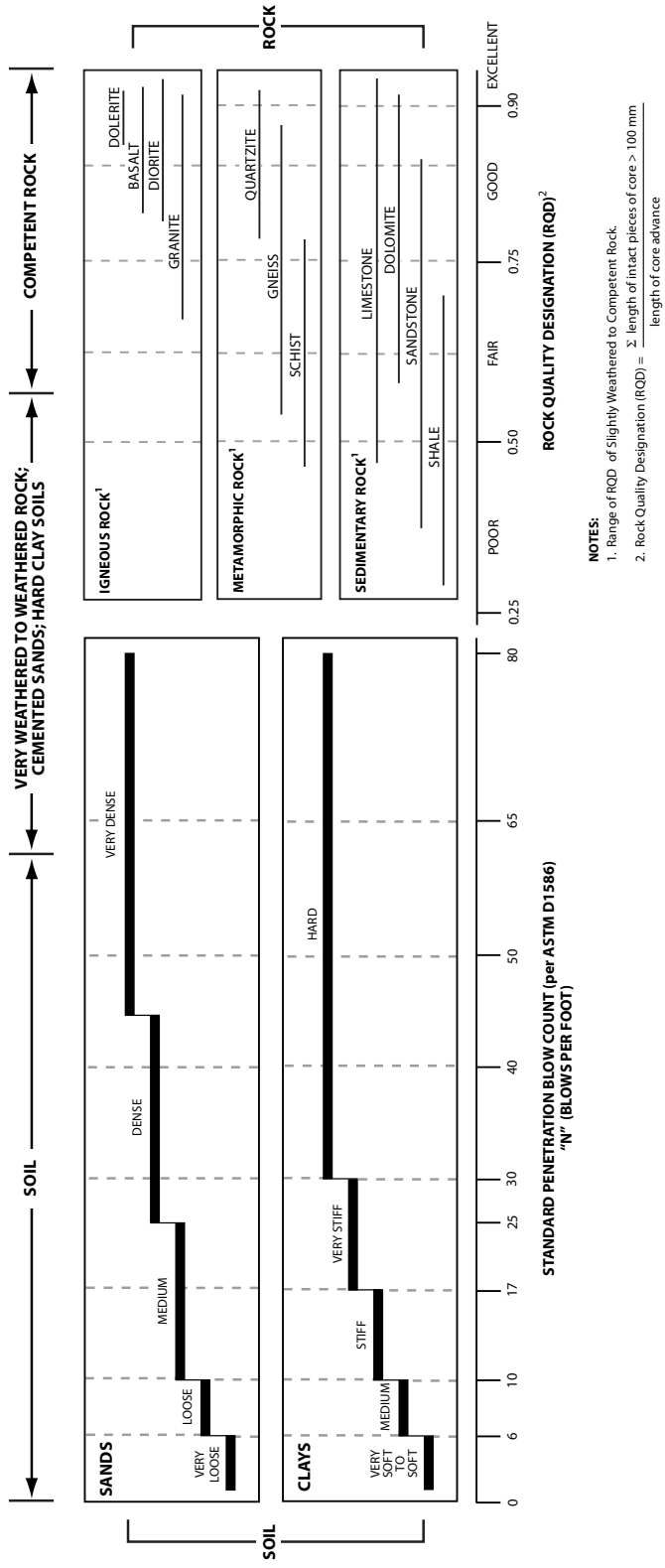
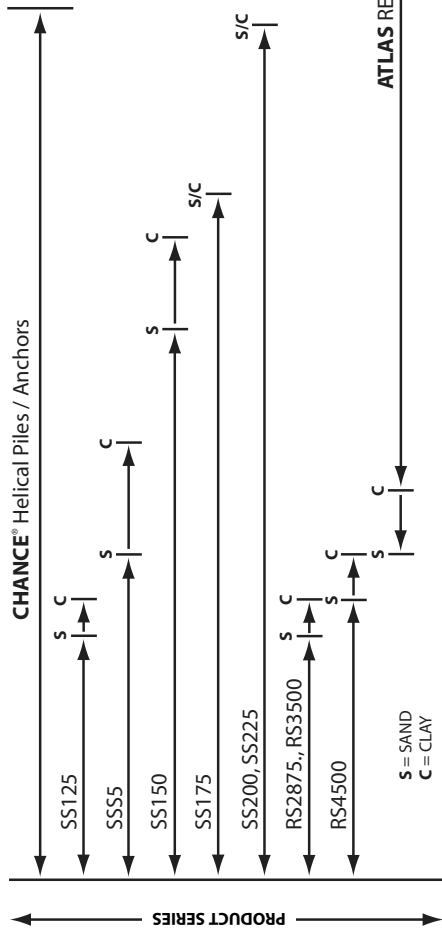
Hubbell Power Systems, Inc. manufactures Chance® helical piles/anchors and Atlas Resistance® pier products for use as tension anchors and/or compression piles for varied foundation support applications. There are many different applications for these end bearing piles and each application will require:

- 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.

HUBBELL POWER SYSTEMS, INC.
CHANCE® HELICAL PILE / ANCHOR
and ATLAS RESISTANCE® PIER
PRODUCT SELECTION GUIDE



NOTES:

1. Range of RQD of Slightly Weathered to Competent Rock.

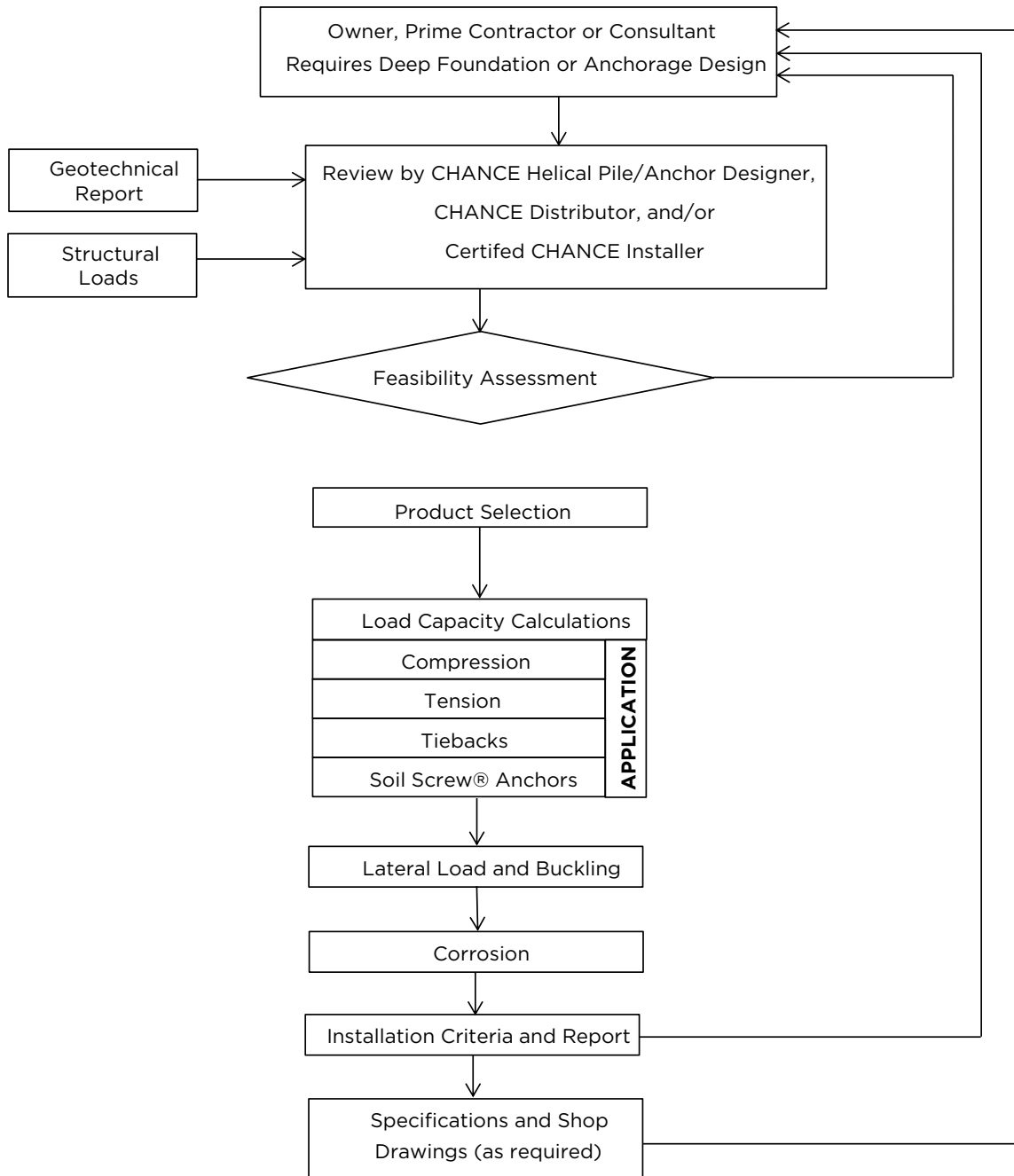
2. Rock Quality Designation (RQD) = $\frac{\sum \text{length of intact pieces of core} > 100 \text{ mm}}{\text{length of core advance}}$

PRODUCT FEASIBILITY

PRODUCT SELECTION GUIDE, FIGURE 3-1

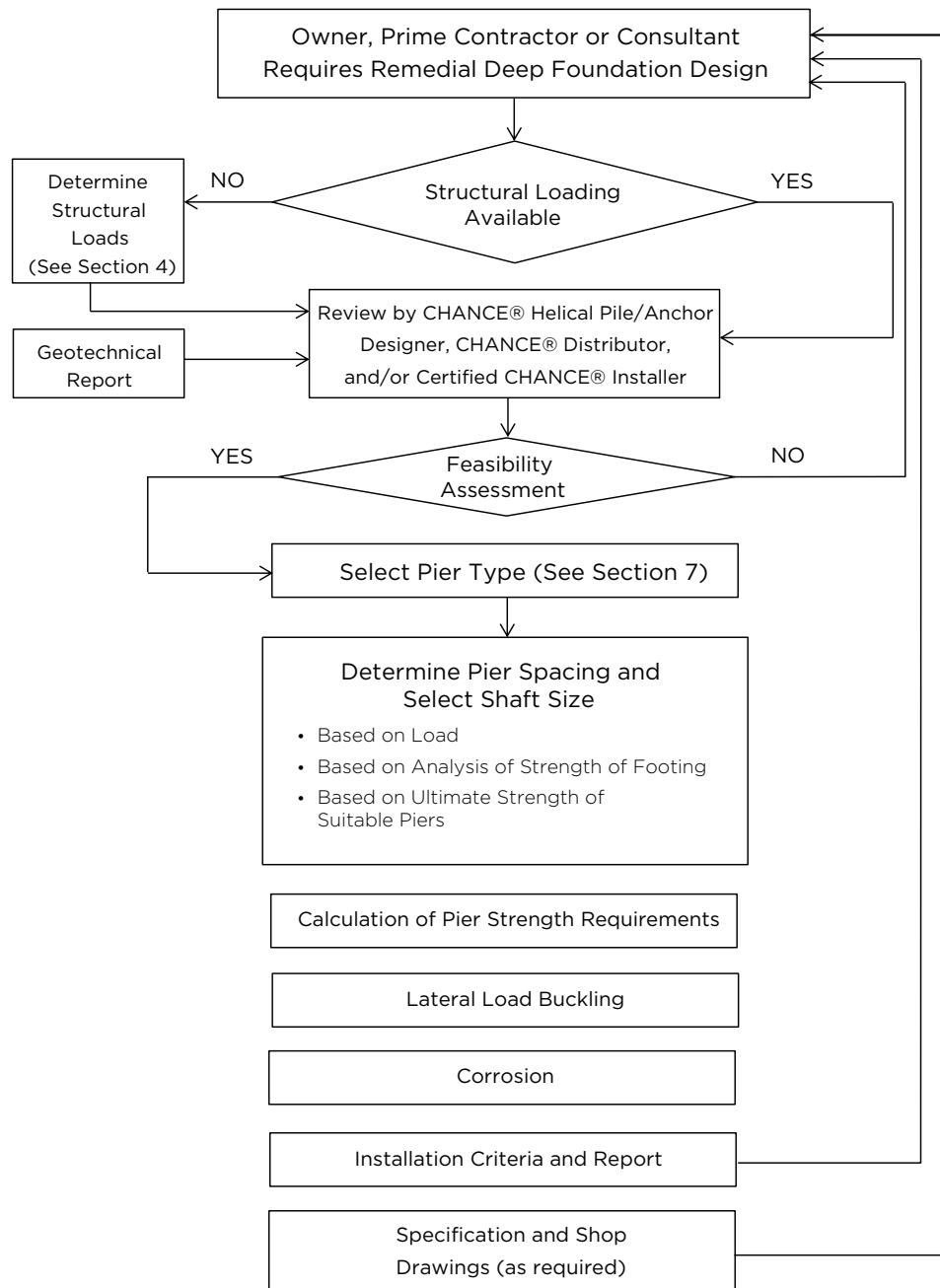


Preliminary Design Flowchart for New Construction CHANCE® Helical Piles/Anchors



DESIGN FLOWCHART FOR CHANCE® HELICAL PILES AND ANCHORS (NEW CONSTRUCTION), FIGURE 3-2

Design Steps Atlas Resistance® Piers



PRODUCT FEASIBILITY

DESIGN FLOWCHART FOR ATLAS RESISTANCE® PERS (REMEDIAL REPAIR APPLICATIONS), FIGURE 3-3



PRELIMINARY DESIGN REQUEST FORM

Contact at Chance Civil Construction: _____

Installing Contractor

Firm:	Contact:		
Phone:	Fax:	Cell:	

Project

Name:	Type:	<input type="checkbox"/> Foundation	<input type="checkbox"/> Underpinning/Shoring
Address:		<input type="checkbox"/> New Construction	<input type="checkbox"/> Rock
		<input type="checkbox"/> Tieback Retaining	<input type="checkbox"/> Other:
		<input type="checkbox"/> 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.



SECTION 4: LOAD DETERMINATION

CONTENTS

Structural Loads 4-2

Preliminary Tieback Design Guide..... 4-3

Tables For Estimating Dead Line (DL) & Live Line (LL) Loads 4-6

Tables For Estimating Free Spans Between Supports 4-8

Preliminary Design Guidelines For Reinforced Concrete Grade Beams 4-12

Preliminary Design Guidelines For Reinforced Concrete Pile Caps..... 4-19

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 should be conducted and state and local building codes and authorities should be consulted 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 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 five common loads that may be resisted by a given foundation element. These are compression, tension, lateral, moment, and torsion 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 bends the foundation about one of its transverse axes. Torsion tends to twist the foundation about its longitudinal axis.

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 ASD allowable strengths of Chance® helical piles/anchors. The designer can choose to use either limit states or allowable strength design for helical piles/anchors.

DESIGN OR WORKING LOAD

The design load or working load is typically considered to be the same load. This is a combination of dead and live loads. The dead load is simply the gravity load of the structure, equipment, etc. that exerts a constant force on the foundation. The live load takes into account seismic events; wind, snow, and ice loads; and occupancy activities. Live loads are transient loads that are dynamic in nature. Design or working loads are sometimes referred to as unfactored loads because 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.

FACTOR OF SAFETY

Before a foundation design is complete, a Factor of Safety (FS) must be selected and applied. In allowable strength design, the FS is the ratio between the ultimate capacity of the foundation and the design load. A Factor of Safety of 2 is typical 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 FS of 2 for permanent loading conditions and 1.5 for any temporary loading condition. See Section 5 for a discussion of Factors of Safety when using Atlas Resistance® piers for underpinning (remedial repair) applications.

ULTIMATE LOAD

The ultimate load is the greatest dead and live load combination multiplied by the factor of safety. This load may or may not be the load used for foundation design.

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 at which 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 in 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 are usually specified in the governing building codes. There are municipal, state, and regional codes 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 2021 IBC contains code sections for helical piles and 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 pile 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 feet.

When using an external waler system consisting of double


channels, W sections, or H 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.

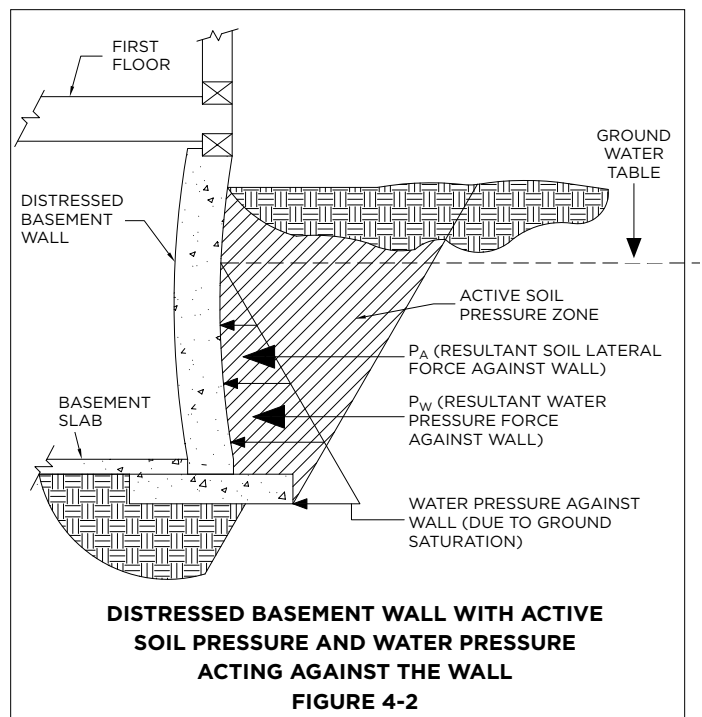
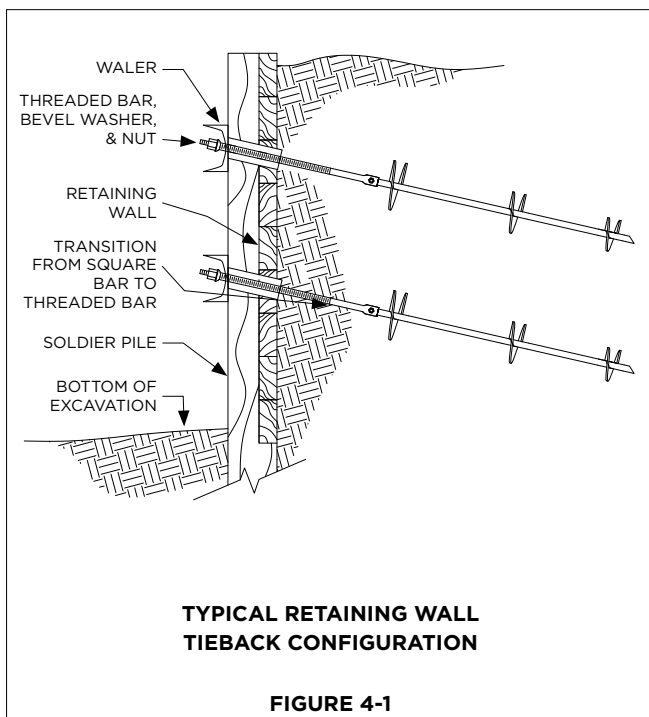
TIEBACK DESIGN CONSIDERATIONS FOR BASEMENT AND RETAINING WALL APPLICATIONS

In most regions of the United States, many 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 bulge inward, crack, or be subjected to other forms of distress. The Chance helical tieback can be an effective repair method for distressed basement walls (see Figures 4-2 through 4-4). There are some general guidelines that are important to understand and follow when specifying tiebacks for basement wall repair or retaining wall support.

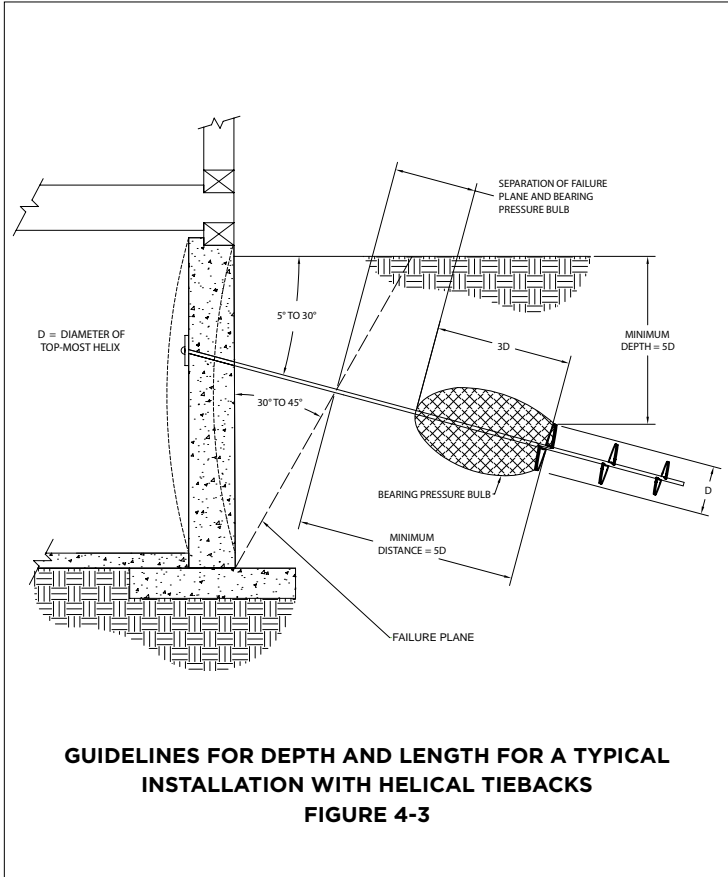
ACTIVE AND PASSIVE PRESSURE CONDITIONS

Figure 4-2 shows a distressed basement wall with active earth pressure pushing against the wall as well as water pressure due to the indicated soil saturation condition. Active earth pressure is defined as the pressure exerted by the earth on a structure that causes movement of the structure away from

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



LOAD DETERMINATION



LOAD DETERMINATION

the soil mass. Most often it is the combined effect of active earth pressure and water pressure that leads to basement wall bulges and cracks.

The installed helical tieback anchor capacity is developed through bearing pressure against the helix plate. The passive earth pressure against the wall aids the tieback anchor in supporting the wall. Passive earth pressure is defined as the pressure the soil on the inside (or lower elevation side) of the wall exerts in opposition to the active pressure of the soil outside the wall. The passive pressure occurs because the soil is in sufficient compression to develop internal shearing resistance. The helical tieback anchor must be installed properly to ensure the helical anchor capacity coupled with passive earth pressure is sufficient to resist the active pressures.

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 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 active pressure against the wall.

LOCATION AND PLACEMENT OF TIEBACKS

Every tieback wall situation is unique, but there are some aspects that merit special 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 or where 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 cracking induced by differential settlement, 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 that the recommended minimum height of soil cover is five times the diameter of the largest helix plate. Finally, the helical anchor must be installed to a sufficient distance away from the wall as shown in Figure 4-3 for the top-most helix plate to fully develop bearing capacity with adequate separation between the active failure plane and helix bearing pressure bulb. This requires the length of installation to be related to the height of soil backfill, also shown in Figure 4-3. The top-most helix must be located a minimum of five times its diameter beyond the assumed active failure plane.

ESTIMATING TIEBACK LOAD REQUIREMENTS

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

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

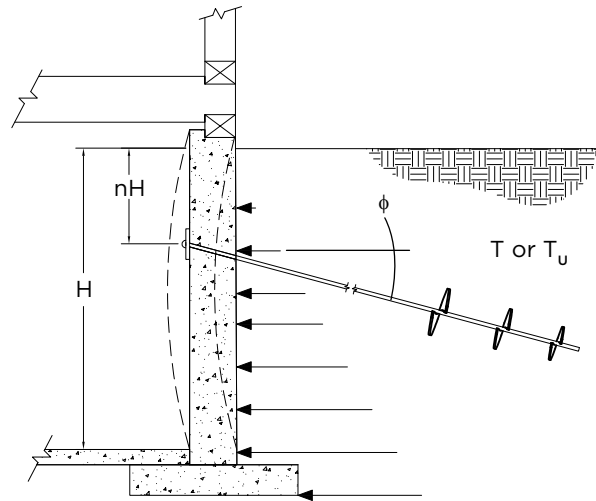
Figures 4-4, 4-5, and 4-6 were prepared for preliminary design assistance in estimating tieback load requirements. Figures 4-5 and 4-6 illustrate cases where no ground water table is present at the site. If hydrostatic water pressure is present, the magnitude of this pressure is determined and added to the earth pressure load requirement. 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. It is critical for basement wall repair to include remedial drainage work to prevent any future condition of soil saturation and resulting water pressure against the wall and/or to take into account the full effect of water pressure against the wall in the tieback design (see Figure 4-2).

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.

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)/ $\cos(\phi)$. Assumes tieback provides 80% of lateral support.
- T_U = $18(H^2)FS/\cos(\phi)$ (no water pressure present)
- T_U = $45(H^2)FS/\cos(\phi)$ (water pressure present)

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

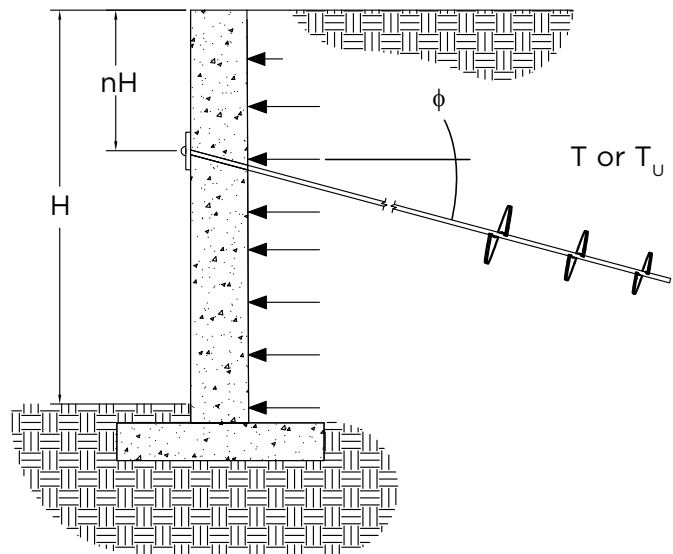


ESTIMATED TIEBACK FORCE REQUIRED FOR BASEMENT APPLICATIONS
FIGURE 4-4

RETAINING WALL

- 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)/ $\cos(\phi)$
- T_U = $25(H^2)FS/\cos(\phi)$

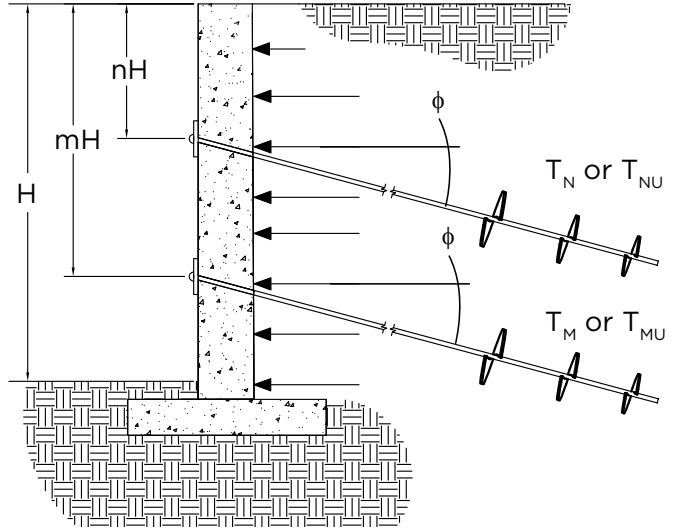
NOTE: Top of wall is assumed free to translate



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 = Upper 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(ϕ)
- T_{NU} = $12(H^2)FS/\cos(\phi)$
- T_{MU} = $18(H^2)FS/\cos(\phi)$
- 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 expertise in all 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 Systems, Inc., or will directly provide the complimentary service.

The Preliminary Design Initiation Form may be found on the last page of Section 3 in this manual. Please familiarize yourself with the information you will need before calling for assistance.

TABLES FOR ESTIMATING DEAD LINE (DL) & LIVE LINE (LL) LOADS

Tables 4-1 through 4-5 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 (DL) AT FOUNDATION (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

LOAD DETERMINATION



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 (DL) AT FOUNDATION (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 (DL) AT FOUNDATION (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

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 LIVE LOAD (LL) AT FOUNDATION (lb/ft)								
One Story - Residential on slab	N/A								
One Story - Residential on basement	N/A								
One Story - Residential over crawl space	250	300	333	300	346	375	400	461	500
Two Story - Residential on slab	N/A								
Two Story - Residential on basement	N/A								
Two Story - Residential over crawl space	500	600	667	600	692	750	800	923	1000
One Story - Commercial on slab	N/A								
One Story - Commercial on basement	N/A								
Two Story - Commercial on slab	450	540	600	540	623	675	720	831	900
Two Story - Commercial on basement	900	1080	1200	1080	1246	1350	1440	1662	1800

LOAD DETERMINATION



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 (SL) along the perimeter of the structure, use the following:

$$SL = S_K(w)L / 2(w + L)$$

where

- w = width of building
- L = length of building

ESTIMATING FOUNDATION SOIL LOAD (W), TABLE 4-5

LOAD FROM SOIL OVERBURDEN	FOOTING TOE WIDTH (B) (in)	HEIGHT OF SOIL OVERBURDEN (H) (ft)	SOIL TYPE			
			COHESIVE		GRANULAR	
			W_{b1} (lb/ft)	W_{b2} (lb/ft)	W_{b1} (lb/ft)	W_{b2} (lb/ft)
<p>NOTE: W_{b2} may be reduced or may not apply when only stabilizing the structure</p>	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.

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 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, the results were checked to ensure 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.

EQUATION 4-1

$$L_s = [F_y(d)A_s / 1.875(P)]^{1/2}$$

where

- L_s = Maximum footing free span (ft)
- F_y = Rebar yield strength = 24,000 psi
- d = Moment arm distance (in)
- A_s = Cross section area of steel (in²)
- P = Structural line load (lb/ft)

EQUATION 4-2

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

where

- x = Pile/pier spacing
- w_p = Width of foundation contact with pile/pier (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.

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

given

$$\begin{aligned} F_y &= 24,000 \text{ psi} \\ d &= (6" - 3") + 8" = 11" \\ A_s &= 2 \times 0.1963 = 0.3926 \text{ in}^2 \\ P &= 1020 + 250 = 1270 \text{ lb/ft} \end{aligned}$$

then

$$\begin{aligned} L_s &= [(24,000 \times 11 \times 0.3926) / (1.875 \times 1270)]^{1/2} \\ &= [43.526]^{1/2} \\ L_s &= 6.6 \text{ ft} = \text{maximum free span} \end{aligned}$$

$$x = \frac{(L_s + w_p / 12)}{FS_f} \quad \text{Equation 4-2}$$

given

$$\begin{aligned} w_p &= 10" \text{ (width of Atlas}^\circ \text{ AP-2-UFB-3500.165 pier} \\ &\text{ bracket or Chance}^\circ \text{ Underpinning helical pile} \\ &\text{ bracket C1500121)} \\ FS_f &= 1.2 \text{ (inspection revealed a well-built foundation)} \end{aligned}$$

then

$$\begin{aligned} x &= (6.6 + 10 / 12) / 1.2 \\ x &= 6.19 \text{ ft (specify pier spacing at 6 feet on center)} \end{aligned}$$

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.

When one wants to reduce the number of piles/piers on a project, it is important to keep in mind that 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.

$$x = \frac{(L_s + w_p / 12)}{FS_f} \quad \text{Equation 4-2}$$

given

$$\begin{aligned} w_p &= 24" \text{ (supplemental steel beam length)} \\ FS_f &= 1.2 \text{ (inspection revealed a well-built foundation)} \end{aligned}$$

then

$$\begin{aligned} x &= (6.6 + 24 / 12) / 1.2 \\ x &= 8.6 / 1.2 = 7.17 \text{ ft (pier spacing can be increased to} \\ &\text{ 7 ft on center)} \end{aligned}$$

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.

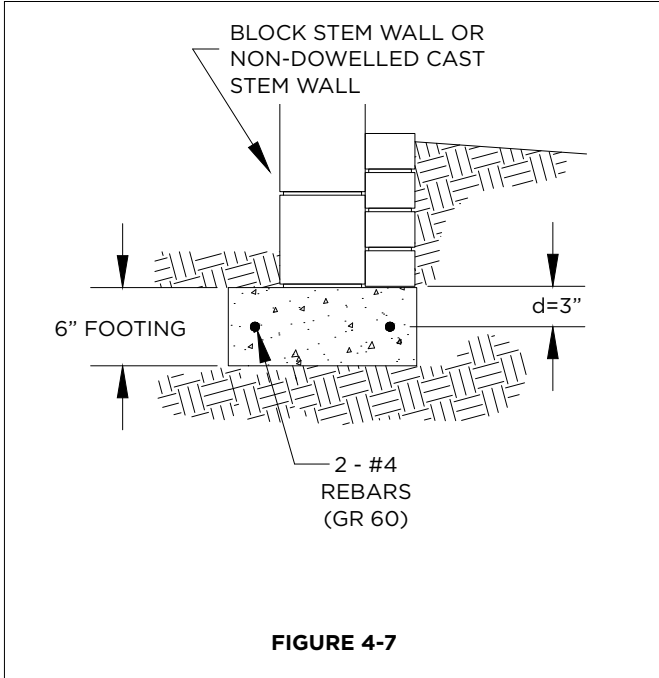


FIGURE 4-7

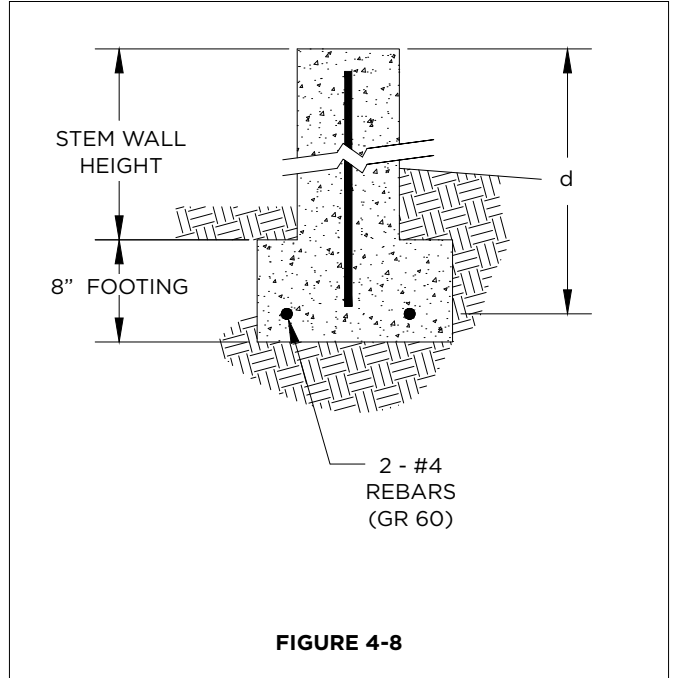


FIGURE 4-8

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

6" THICK REINFORCED CONCRETE SPREAD FOOTING 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'	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'	4'-9"	4'-7"
2 - #4 rebar (Gr 60); 6" x 48" tall cast stem wall (dowelled or monolithic) $d = 51"$	-	-	-	-	-	8'-7"	8'	7'-7"	7'-2"	6'-10"	6'-6"	6'-3"

LOAD DETERMINATION



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

8" THICK REINFORCED CONCRETE SPREAD FOOTING 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 rebars (Gr 60); concrete block or cast stem wall (not dowelled) $d = 5"$	4'-6"	3'-9"	3'-6"	-	-	-	-	-	-	-	-	-
2 - #4 rebars (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 rebars (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 rebars (Gr 60); 8" x 24" tall cast stem wall (dowelled or monolithic) $d = 29"$	-	8'-6"	7'-8"	7'	6'-5"	6'	5'-8"	5'-5"	5'-2"	4'-11"	4'-9"	4'-7"
2 - #4 rebars (Gr 60); 8" x 48" tall cast stem wall (dowelled or monolithic) $d = 53"$	-	-	-	-	-	8'-2"	7'-8"	7'-4"	7'	6'-8"	6'-5"	6'-2"

12" THICK REINFORCED CONCRETE SPREAD FOOTING 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 rebars (Gr 60); 10" x 12" tall cast stem wall (dowelled or monolithic) $d = 21"$	8'-4"	7'-10"	7'-2"	7'	6'-8"	6'-5"	6'-2"	5'-11"	5'-9"	5'-7"	5'-5"	5'-3"
3 - #5 rebars (Gr 60); 10" x 18" tall cast stem wall (dowelled or monolithic) $d = 27"$	-	-	8'-5"	8'	7'-7"	7'-3"	7'	6'-9"	6'-6"	6'-4"	6'-1"	5'-11"
3 - #5 rebars (Gr 60); 10" x 24" tall cast stem wall (dowelled or monolithic) $d = 33"$	-	-	-	-	-	8'	7'-9"	7'-5"	7'-2"	7'	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'

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

LOAD DETERMINATION

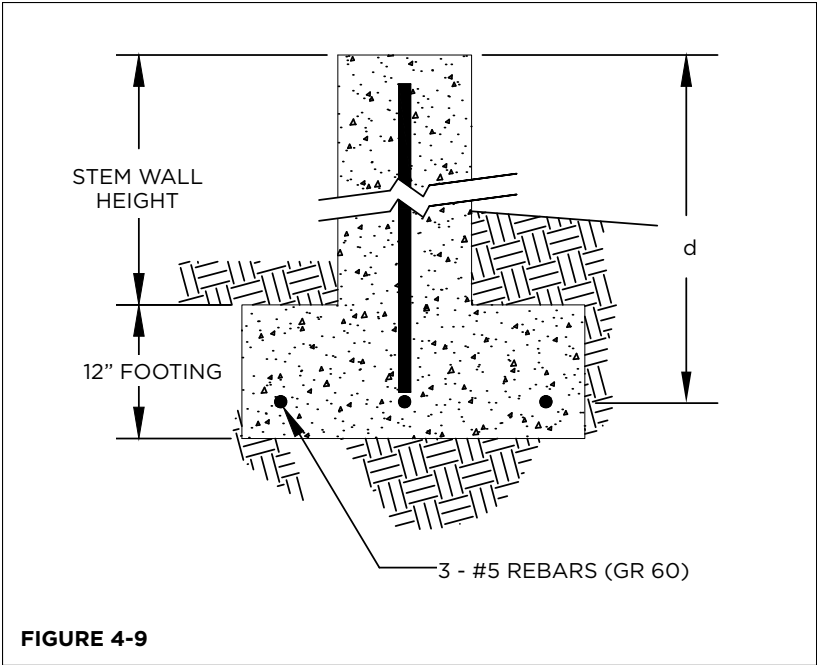


FIGURE 4-9

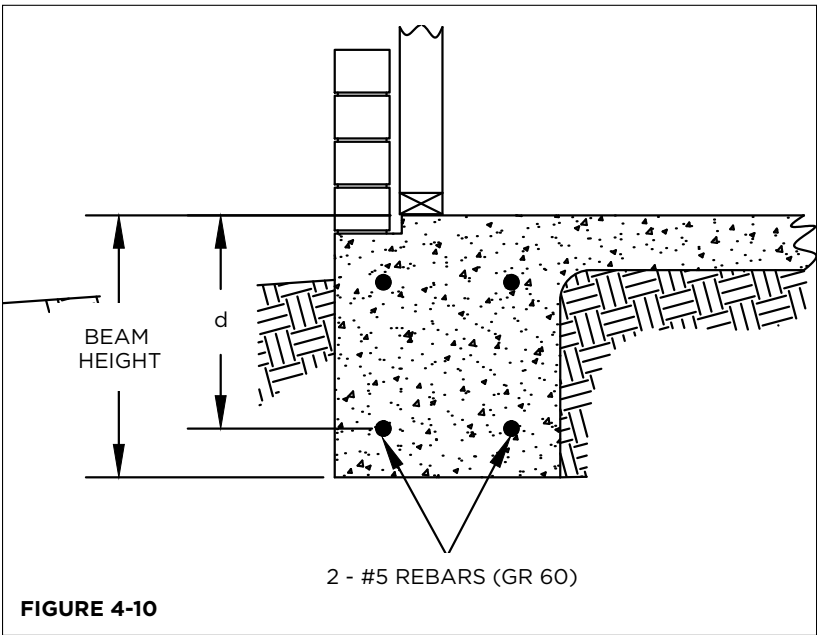


FIGURE 4-10

LOAD DETERMINATION

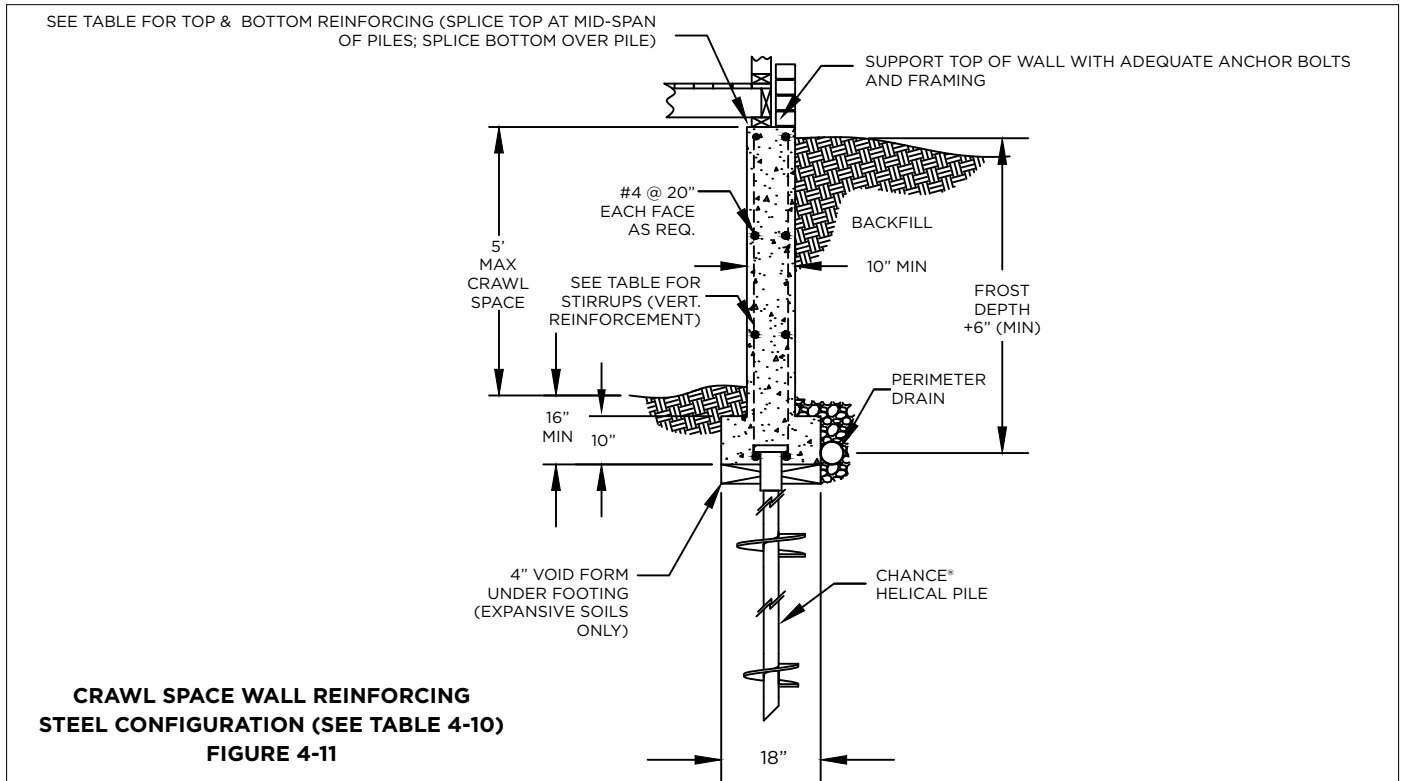
PRELIMINARY DESIGN GUIDELINES FOR REINFORCED CONCRETE GRADE BEAMS

Building loads are most commonly transferred to helical piles through concrete grade beams. Figures 4-11 through 4-15 and Tables 4-10 through 4-14 provide preliminary design guidance for grade beam sizing and steel reinforcement configuration. These tables 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.

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

CRAWL SPACE WALL



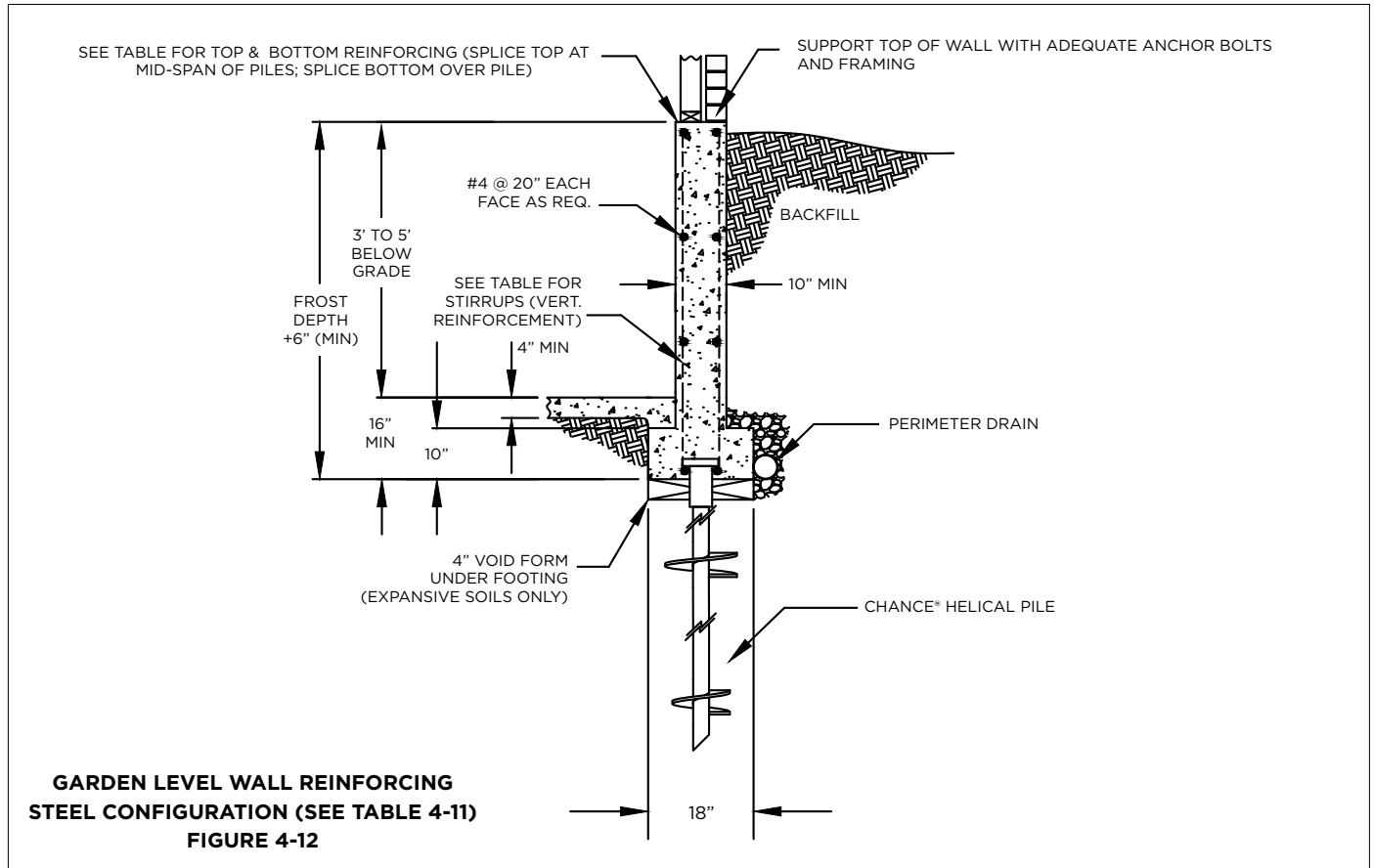
CRAWL SPACE WALL REINFORCING STEEL CONFIGURATION, TABLE 4-10

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)	
		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	

LOAD DETERMINATION



GARDEN LEVEL WALL

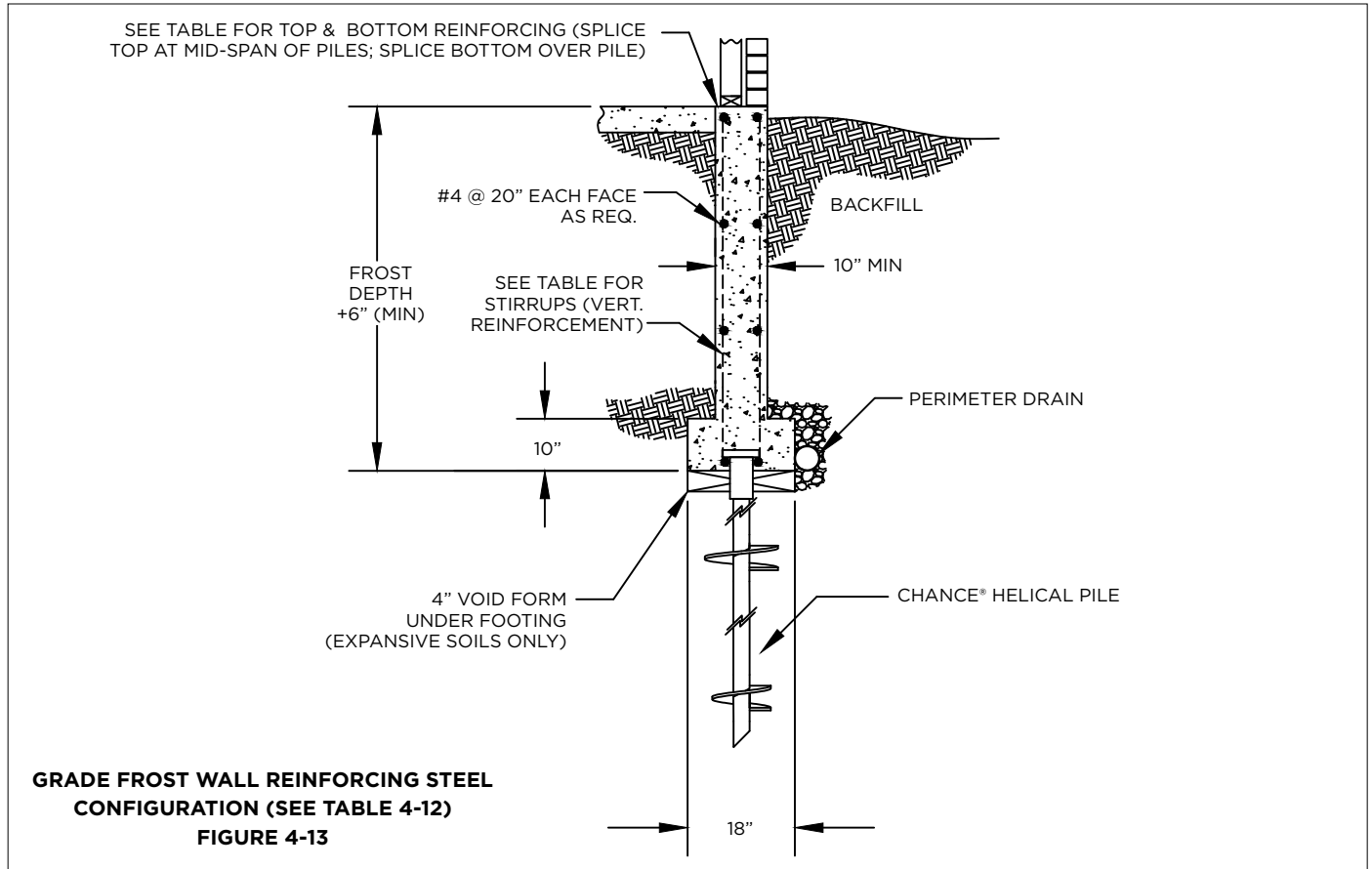


LOAD DETERMINATION

GARDEN LEVEL WALL REINFORCING STEEL CONFIGURATION, 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)	
		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		4 - #8	
	5'	2 - #7	#3 @ 12"	2 - #8	#3 @ 12"	4 - #7	#3 @ 12"	4 - #7	#3 @ 12"	4 - #7	#3 @ 12"

GRADE FROST WALL

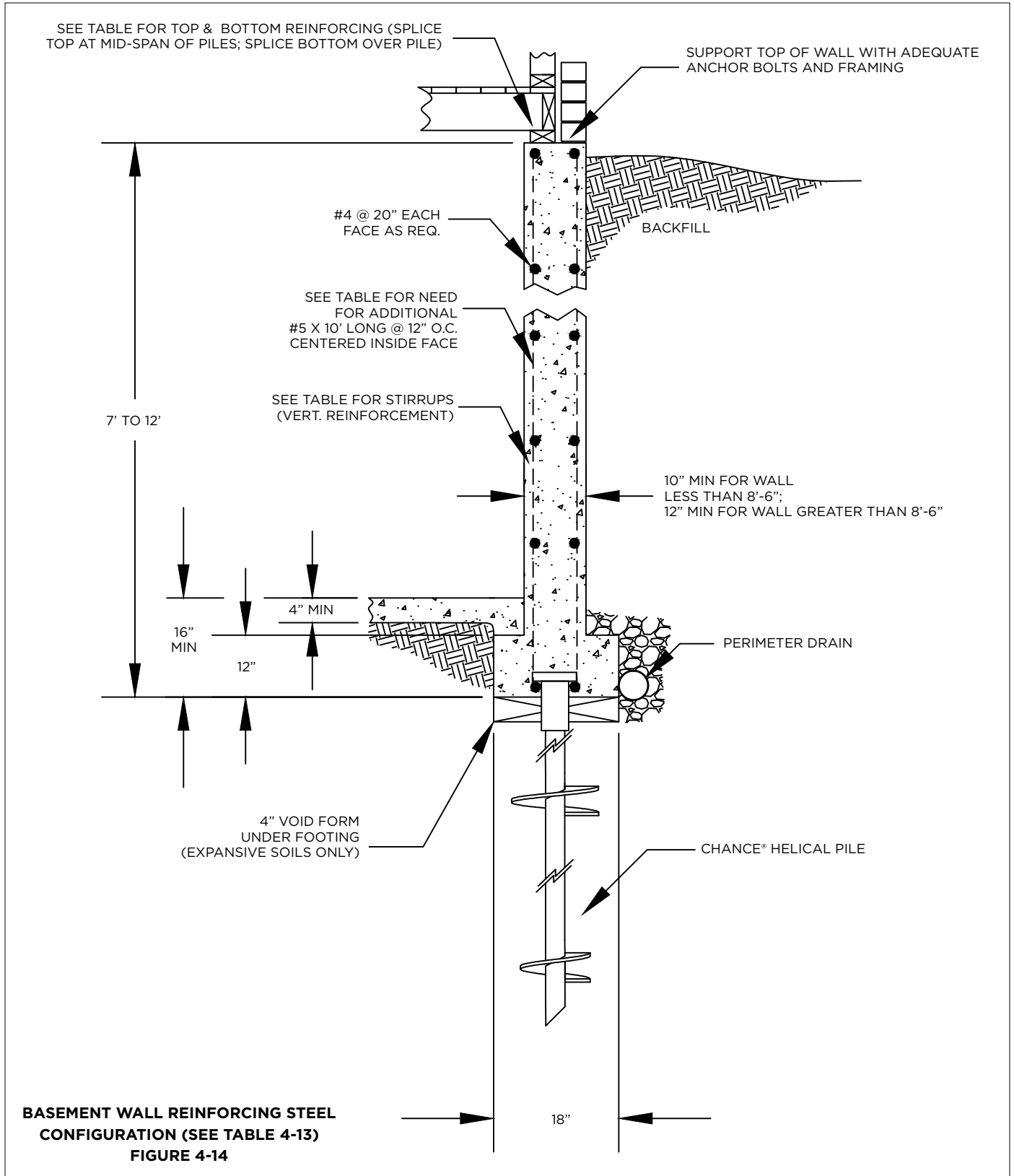


GRADE FROST WALL REINFORCING STEEL CONFIGURATION, TABLE 4-12

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)	
		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	

LOAD DETERMINATION

BASEMENT WALL



LOAD DETERMINATION



BASEMENT WALL REINFORCING STEEL CONFIGURATION, TABLE 4-13

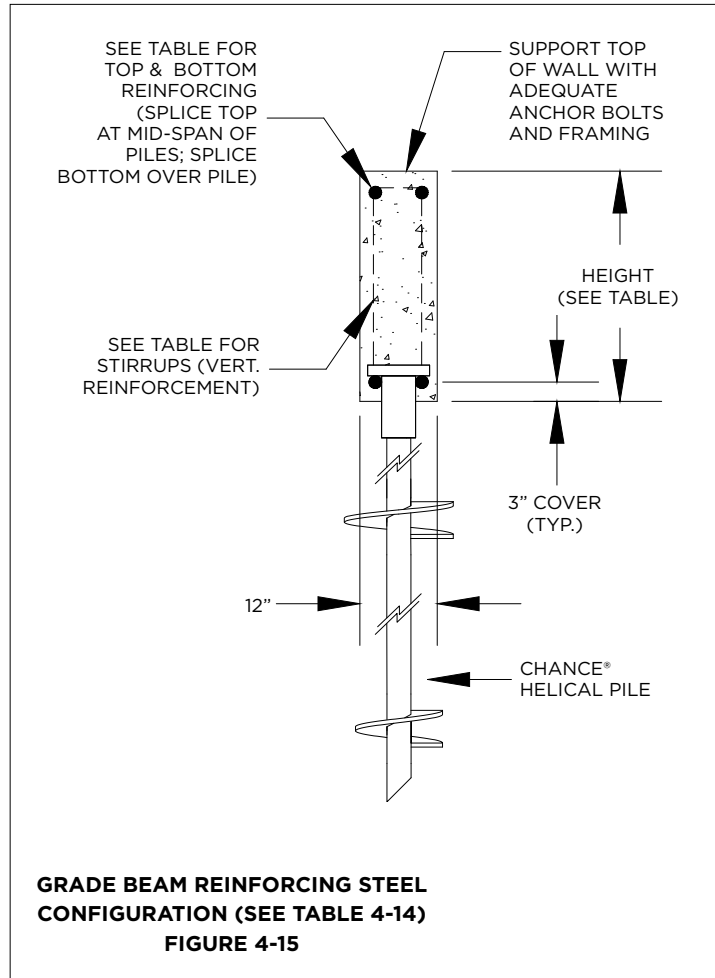
PILE SPACING	WALL HEIGHT	TOTAL FOUNDATION LINE LOAD									
		3,000 (lb/ft)		4,000 (lb/ft)		5,000 (lb/ft)		6,000 (lb/ft)		7,000 (lb/ft)	
		STEEL REINFORCING BARS REQUIRED									
		TOP & BOTTOM	STIRRUP (O.C.)	TOP & BOTTOM	STIRRUP (O.C.)	TOP & BOTTOM	STIRRUP (O.C.)	TOP & BOTTOM	STIRRUP (O.C.)	TOP & 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 10' long @ 12" O.C. bars centered vertically on inside wall face - See Figure 4-14.

LOAD DETERMINATION



GRADE BEAM



LOAD DETERMINATION

GRADE BEAM REINFORCING STEEL CONFIGURATION, 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 (O.C.)	HEIGHT	TOP & BOTTOM	STIRRUPS (O.C.)	HEIGHT	TOP & BOTTOM	STIRRUPS (O.C.)
8'	18"	2 - #5	#3 @ 12"	20"	3 - #5	#3 @ 12"	24"	4 - #5	#3 @ 12"
10'	18"	3 - #5	#3 @ 12"	22"	3 - #5	#3 @ 12"	30"	4 - #5	#3 @ 15"
12'	24"	3 - #5	#3 @ 12"	27"	4 - #5	#3 @ 15"	30"	4 - #5	#3 @ 15"
15'	24"	4 - #5	#3 @ 12"	30"	4 - #5	#3 @ 15"	36"	4 - #6	#3 @ 18"

PRELIMINARY DESIGN GUIDELINES FOR REINFORCED CONCRETE PILE CAPS

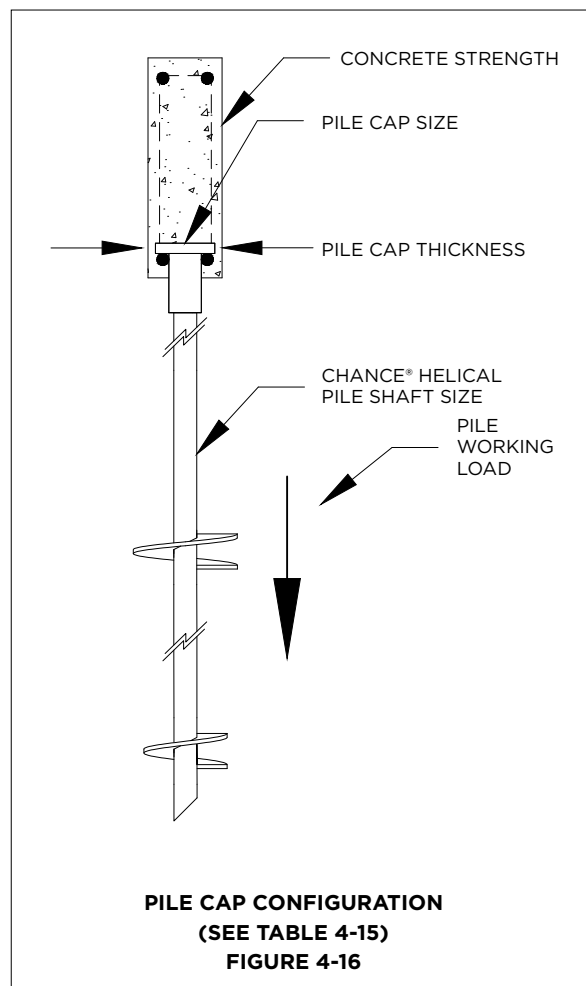
Pile cap configurations may be determined from Table 4-15. The table is based upon American Concrete Institute (ACI) criteria for concrete bearing stress from external bearing plates at working loads and from the American Institute of Steel Construction (AISC) criteria for bending stress in the steel 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 upper portion 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 helical pile shaft series. 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.



LOAD DETERMINATION



PILE CAP CONFIGURATION, 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 THICKNESS 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 RS2875.276	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

LOAD DETERMINATION



SECTION 5: DESIGN METHODOLOGY

CONTENTS

- 5.1 Atlas Resistance Pier Capacity5-2
- 5.2 Chance Helical Pile/Anchor Ultimate Bearing Capacity.....5-3
- 5.3 Evaluating Soil Properties For Design 5-14
- 5.4 Factor Of Safety5-20
- 5.5 HeliCAP Helical Capacity Design Software 5-22
- 5.6 Application Guidelines For Chance Helical Piles/Anchors..... 5-25
- 5.7 Lateral Capacity Of Helical Piles 5-25
- 5.8 Buckling/Bracing/Slenderness Considerations.....5-30
- 5.9 Helical Pile Deflection At Working Load..... 5-34

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 should be conducted and state and local building codes and authorities should be consulted 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 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.

5.1 ATLAS RESISTANCE PIER CAPACITY

Atlas Resistance® piers develop their capacity primarily through end bearing. The current accepted 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.

In general, the tip of the Atlas Resistance pier should be embedded in cohesionless soils with Standard Penetration Test (SPT) blow count (N_{60}) values above the 30 to 35 range and in cohesive soils with SPT N_{60} values above the 35 to 40 range. Past installation experience indicates that Atlas Resistance piers will provide foundation underpinning support via end bearing when positioned into these SPT N_{60} value ranges. 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 capacity. The first stage is to drive the pier pipe to a firm bearing stratum. In the second stage, the lift equipment is combined with a manifold system to lift the structure. The Atlas Resistance pier system procedure provides measured support capacity. Atlas Resistance piers are spaced at adequate centers so that each pier is driven to a suitable stratum and then tested to a force greater than the force 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.

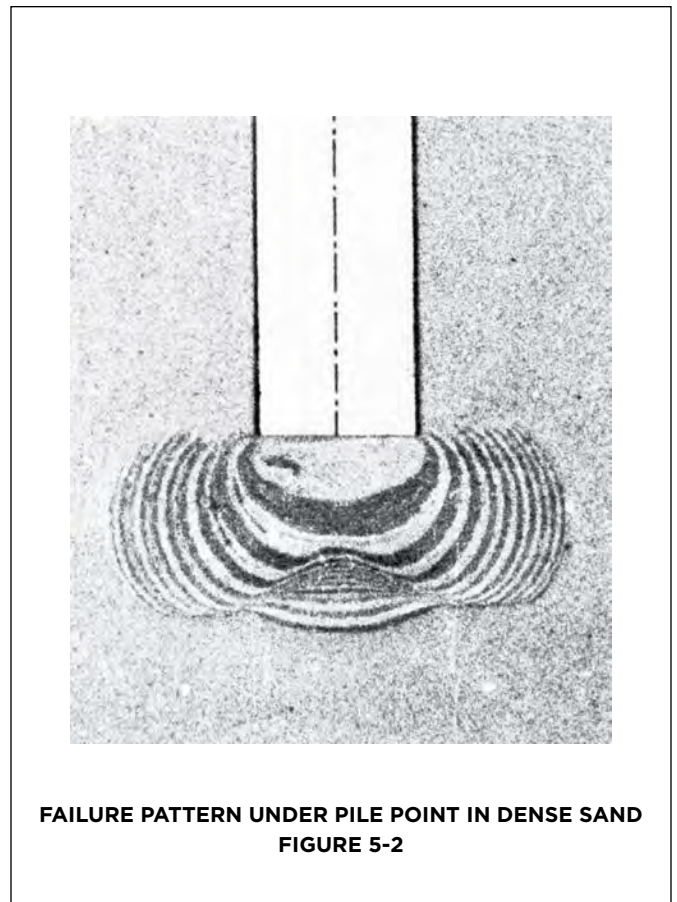
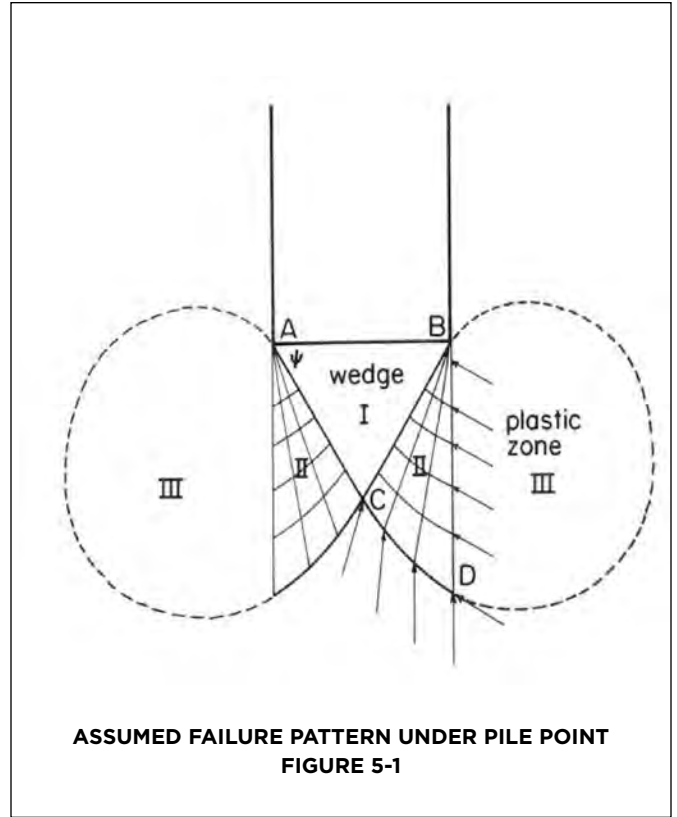
- Pier Working Load: The required working load per pier (P_w) is calculated based on the dead loads and live loads and the pier spacing that was selected considering the ability of the existing foundation to span between the proposed pier locations.

EQUATION 5-1

$$P_w = xP$$

where

- x = Selected pier spacing
 - P = Line load on footing = $DL + LL + SL + W$
 - DL = Dead load
 - LL = Live load
 - SL = Snow load
 - W = Soil load
- Hardware Factor of Safety: Hubbell Power Systems, Inc., recommends a minimum Factor of Safety (FS_h) of 2.0 for structural capacity of the hardware (pier pipe, bracket, etc.). The hardware Factor of Safety may be varied based on engineering judgement.
 - Hardware Structural Capacity: The required structural minimum ultimate capacity of the hardware ($R_{w\ ULT}$) is calculated from the pier working load and the hardware Factor of Safety.



**EQUATION 5-2**

$$R_{w\text{ ULT}} = P_w(FS_h)$$

where

- P_w = Pier working load
- FS_h = Hardware Factor of Safety = 2.0 (may be varied based on engineering judgment)

- Pier System Selection: Select a pier system with a published ultimate capacity ($R_{h\text{ ULT}}$) (found in Section 7 of this manual) equal to or greater than the required minimum ultimate hardware capacity.

EQUATION 5-3

$$R_{h\text{ ULT}} \geq R_{w\text{ ULT}}$$

where

- $R_{w\text{ ULT}}$ = Minimum ultimate hardware capacity based on pier working load

- Maximum Pier Spacing: Check the maximum pier spacing (x_{MAX}) based upon the selected pier system hardware capacity. The selected pier spacing must be less than or equal to the maximum pier spacing ($x \leq x_{\text{MAX}}$).

EQUATION 5-4

$$x_{\text{MAX}} = (R_{h\text{ ULT}}) / (FS_h)P$$

(foundation must be structurally capable of spanning this distance)

where

- $R_{h\text{ ULT}}$ = Pier system published ultimate capacity
- FS_h = Hardware Factor of Safety
- P = Line load on footing

- Proof Load: Atlas Resistance® piers are installed using a two-stage process as noted above. First, the pier is driven to a firm bearing stratum. The installation resistance force applied during this stage is called the proof load (R_p). The minimum proof load is calculated from the pier working load and the proof load Factor of Safety (FS_p). Hubbell recommends a minimum FS_p of 1.5 at installation unless structural lift occurs first. 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 (proof load) of up to 1.5 times that long-term working load. If footing damage occurs during installation, the free span between piers (L_p $_{\text{MAX}}$) may be excessive.

EQUATION 5-5

$$R_p = (FS_p)P_w = 1.5P_w$$

where

- FS_p = Proof load Factor of Safety = 1.5
- P_w = Pier working load

- Maximum Proof Load: The proof load (pier installation force) must be limited to avoid overloading the pier system hardware during installation. Hubbell Power Systems, Inc., recommends that the maximum proof load ($R_{p\text{ MAX}}$) not exceed 1.65 times the published maximum working capacity ($R_{h\text{ WORK}}$) of the pier system (found in Section 7 of this manual) without engineering approval. The maximum working capacity of the pier system is half of the ultimate capacity.

EQUATION 5-6

$$R_{p\text{ MAX}} \leq 1.65(R_{h\text{ WORK}}) \quad \text{or}$$

$$R_{p\text{ MAX}} \leq 1.65(R_{h\text{ ULT}})/2$$

where

- $R_{h\text{ WORK}}$ = Pier system published maximum working capacity = $(R_{h\text{ ULT}})/2$
- $R_{h\text{ ULT}}$ = Pier system published ultimate capacity

Additional notes:

Current practice by Hubbell 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 an SPT N_{60} value 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 N_{60} value is 4 or less. The sleeve must extend at least 36 inches beyond the unsupported exposure and/or the area of weak soil. If the anticipated lift is to exceed 4 inches, the Atlas Resistance Continuous Lift pier system should be used.

Atlas Resistance piers can be located as close as 12 inches (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 various field and lab test techniques. The projected area is controlled by the size and number of helix plates. Helical piles and anchors may be used for a variety of applications for compression loading (helical piles) and tension loading (helical anchors). Helical piles and 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 top 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 helical piles and 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 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 for helical piles is five times the diameter of the top-most helix (5B). For tension anchors it is five feet or 5B, whichever is greater. The embedment 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 are considered in design, depending on the relative spacing of the helix plates. For wide helix spacing (spacing $\geq 3B$), the individual plate bearing method is used; for close helix spacing (spacing $< 3B$), the perimeter shear method is used. These two methods are illustrated in Figures 5-3a & 5-3c (individual plate bearing) and Figures 5-3b & 5-3d (perimeter shear). With individual plate bearing, the helix capacity is determined by calculating the unit bearing capacity of the soil at the helix depth and multiplying the result by the helix projected area. The process is completed for each helix, and the individual helix capacities are added to yield the total pile/anchor capacity. Side resistance 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 occurs between helix plates. The perimeter shear method assumes that the close helix spacing causes a prism of soil to develop between the helix plates and that failure in this zone occurs along a plane as shown in Figures 5-3b & d. In reality, the perimeter shear method includes plate bearing and perimeter shear failure as illustrated.

The following is Terzaghi's general bearing capacity equation (Equation 5-7), which is used to determine the ultimate capacity of soil (Q_{ult} , a.k.a. Q_H). This equation and its use will be discussed in this section, as it forms the basis of determining helix capacity in soil.

EQUATION 5-7

$$Q_{ult} = A_h(cN_c + q'N_q + 0.5\gamma'BN_\gamma)$$

where

- 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 N_c , N_q , and N_γ are bearing capacity factors

Terzaghi's bearing capacity factors are shown in Table 5-2.

Following is based on Bowles (1988) concerning the use of Equation 5-7 for deep foundations where the various terms of the bearing capacity equation are distinguished.

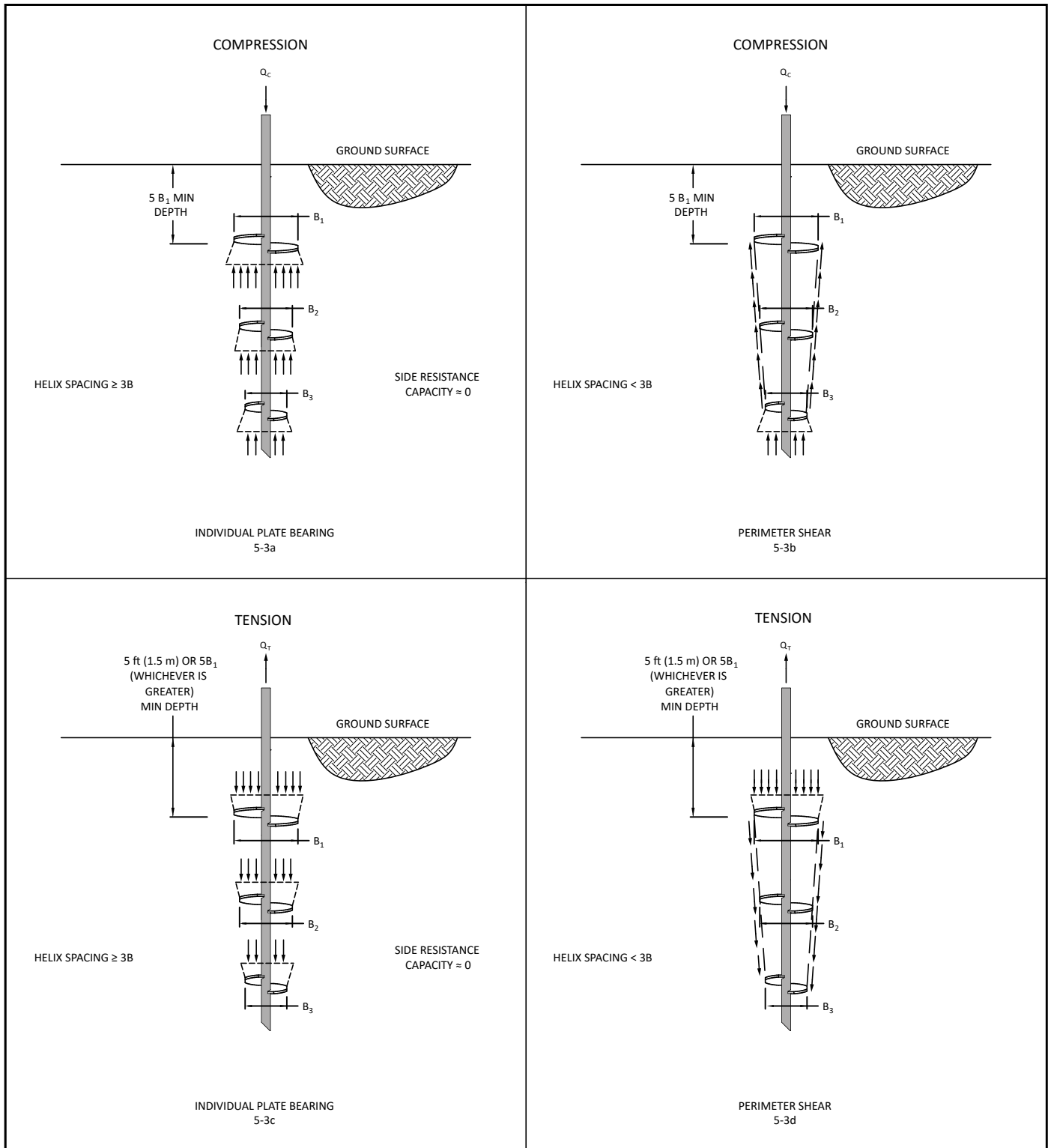
1. The cohesion term predominates in cohesive soil.
2. The depth term ($q'N_q$) predominates in cohesionless soils. Only a small increase in D (vertical depth to footing or helix plate) increases Q_{ult} substantially.
3. The base width term $0.5\gamma'BN_\gamma$ provides some increase in bearing capacity for cohesive and cohesionless soils. In cases where $B < 3$ to 4 m (9.8 to 13.1 ft), this term could be neglected with little error.

The base width term of the bearing capacity equation is not used when dealing with helical piles/anchors 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.

TYPICAL APPLICATIONS FOR SINGLE-HELIX AND MULTI-HELIX HELICAL PILES AND ANCHORS, TABLE 5-1

SOIL TYPE	SINGLE-HELIX				MULTI-HELIX			
	FAILURE CONDITION				FAILURE CONDITION			
	Shallow		Deep		Shallow		Deep	
	C	T	C	T	C	T	C	T
Clay	✓	✓	✓	✓	✗	✗	✓	✓
Sand	✓	✓	✓	✓	✗	✗	✓	✓
Mixed Soils	✓	✓	✓	✓	✗	✗	✓	✓

C = Compression T = Tension



**COMPARISON BETWEEN INDIVIDUAL PLATE BEARING AND PERIMETER SHEAR FOR COMPRESSION AND TENSION LOADING
FIGURE 5-3**

DESIGN METHODOLOGY



TERZAGHI'S SHALLOW FOUNDATION BEARING CAPACITY FACTORS [BOWLES (1988) AND ASCE (1993A)], TABLE 5-2

ϕ'	N_c	N_γ	N_q
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

NOTES ON USE OF TERZAGHI'S BEARING CAPACITY EQUATION:

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

EQUATION 5-8

$$Q_{ult} = A_h(1.3cN_c + q'N_q + 0.3\gamma'BN_\gamma)$$

2. Because B is considered very small for helical piles and 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:

EQUATION 5-9

$$N_c = 6.0(1 + 0.2D/B) \leq 9.0$$

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 piles/anchors, the side resistance is generally ignored. For round shaft piles/anchors there may be a component of side 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 structural capacity of the pile/anchor as provided by the manufacturer. See Section 7 of this manual for structural capacity ratings of Chance® helical piles/anchors.

There is cause for concern 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. This will typically produce higher load capacities and a more difficult installation, i.e., it will require more installation torque. In some cases, a larger helical pile/anchor product series, i.e., one with greater torque capacity, must be used to enable installation into the dry conditions.

5.2.1 SINGLE-HELIX HELICAL PILES AND 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 of the helix (D) to diameter of the helix (B) is less than 5, i.e., $D/B < 5$. In this case, the design is analogous to compression loading of a shallow foundation.

5.2.1.1.a Saturated Clays ($\phi' = 0$; $c > 0$)

In saturated clays with $\phi' = 0$, the term $N_\gamma = 0$ and $N_q = 1.0$. The bearing capacity equation becomes:

EQUATION 5-10

$$Q_{ult} = A_h(cN_c + \gamma'D)$$

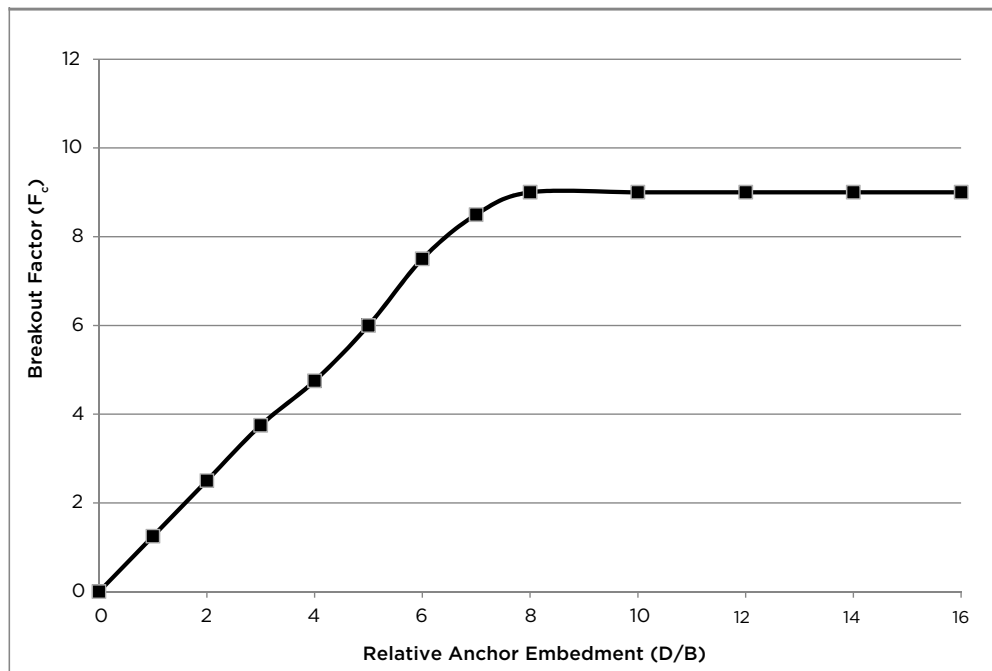
where

- Q_{ult} = Ultimate bearing capacity
- A_h = Projected helix area
- c = Cohesion; for $\phi' = 0$, c = undrained shear strength = s_u
- N_c = Bearing capacity factor; for $\phi' = 0$ for round plates, $N_c = 6.0(1 + 0.2D/B) \leq 9$
- γ' = Effective unit weight of soil above helical pile
- D = Depth

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

5.2.1.1.b Sands ($\phi' > 0$; $c = 0$)

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



VARIATION IN UPLIFT BREAKOUT FACTOR FOR SHALLOW SINGLE-HELIX ANCHORS IN CLAY
FIGURE 5-4

EQUATION 5-11

$$Q_{ult} = A_h(q'N_q + 0.5\gamma'BN_\gamma)$$

where

q' = Effective surcharge (overburden pressure) = $\gamma'D$

N_q and N_γ 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 ($\phi' > 0$; $c > 0$)

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

EQUATION 5-12

$$Q_{ult} = A_h(cN_c + q'N_q + 0.5\gamma'BN_\gamma)$$

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

5.2.1.2 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 feet or 5 times the diameter of the top-most helix, whichever is greater, 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 ($\phi' = 0$; $c > 0$)

Test results and analytical studies indicate that the breakout factor (F_c) 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. Figure 5-4 shows the variation in F_c vs. D/B for circular plates. This figure [from Das (1990)] shows that $F_c = 1.2D/B \leq 9$, 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 equation for ultimate uplift capacity (Q_{ultU}) is similar to Equation 5-10 and is given as:

EQUATION 5-13

$$Q_{ultU} = A_h(cF_c + \gamma'D)$$

where

c = Cohesion; for $\phi' = 0$, c = undrained shear strength = s_u

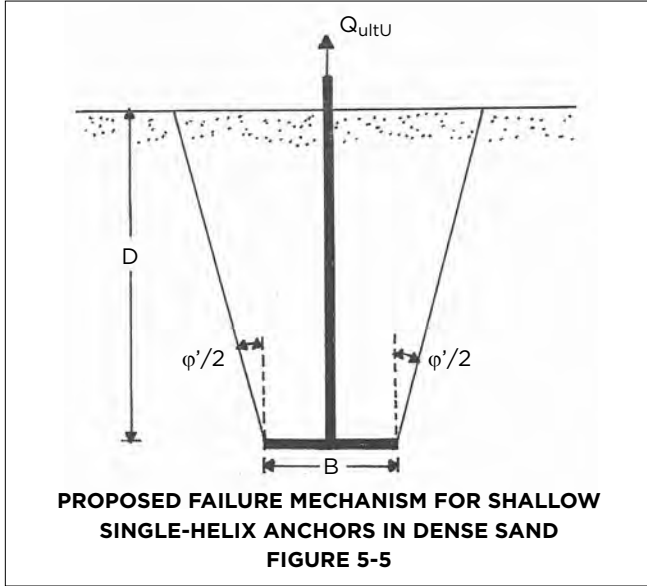
F_c = Breakout factor; for $\phi' = 0$, $F_c = 1.2D/B \leq 9$

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 soil disturbance above the helical plate as a result of installation. This depends on the sensitivity of the clay and is a matter of engineering judgment.



5.2.1.2.b Sands ($\phi' > 0$; $c = 0$)

In sands, uplift loading of shallow (generally $D/B < 5$) single-helix anchors develops a failure zone that looks similar to an inverted, truncated cone. The failure is assumed to take place by perimeter shear acting along this failure surface, which is inclined from the vertical at an angle of about $\phi'/2$ as shown in Figure 5-5. The uplift force must also lift the mass of the soil within the truncated cone. The ultimate uplift capacity (Q_{ultU}) is calculated from:

EQUATION 5-14

$$Q_{ultU} = W_s + \{\pi\gamma K_0 \tan(\phi') \cos^2(\phi'/2) [B(D)^2/2 + D^3 \tan(\phi'/2)/3]\}$$

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:

EQUATION 5-15

$$V = \pi D/3 \{2(B)^2 + [B + 2D \tan(\phi'/2)]^2 + 2BD \tan(\phi'/2)\}$$

The value of the at-rest lateral earth pressure coefficient for sands can reasonably be calculated as: $K_0 = 1 - \sin(\phi')$

5.2.1.2.c Mixed Soils ($\phi' > 0$; $c > 0$)

For shallow installations in mixed soils with frictional and cohesive components of shear strength, there is another component of the resisting force in uplift added to the components included in equation 5-14. This added component results from cohesion acting along the surface of the truncated cone failure zone between the helical plate and the ground surface. Adding a new term to equation 5-14 to account for the cohesion effect yields:

EQUATION 5-16

$$Q_{ultU} = W_s + \{\pi\gamma K_0 \tan(\phi') \cos^2(\phi'/2) [B(D)^2/2 + D^3 \tan(\phi'/2)/3]\} + cA_c$$

where

A_c = Surface area of truncated cone

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

EQUATION 5-17

$$A_c = \pi\{(R^2 + r^2) + [(R^2 - r^2) + (D(R + r))^2]^{0.5}\}$$

where

r = Radius of helical plate = $B/2$

R = Radius of cone failure surface at the ground surface = $B/2 + (D)\tan(\phi'/2)$

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

5.2.2 SINGLE-HELIX HELICAL PILES AND ANCHORS—DEEP INSTALLATION

Deep installations of helical piles and anchors are generally more common than shallow installations provided there is sufficient soil depth to 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 utilize a deep installation when possible. Figure 5-6 illustrates the single-helix plate capacity model, wherein the soil failure mechanism follows the theory of general plate bearing 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 or equal to 5, i.e., $D/B \geq 5$. In this case, the design is analogous to compression loading of a deep end-bearing foundation.

5.2.2.1.a Saturated Clays ($\phi' = 0$; $c > 0$)

Under compression loading, the ultimate capacity of a single-helix helical pile in clay is calculated from Equation 5-10 as:

$$Q_{ult} = A_h(s_u N_c + \gamma' D)$$

where

N_c = Bearing capacity factor for deep failure = 9

which yields

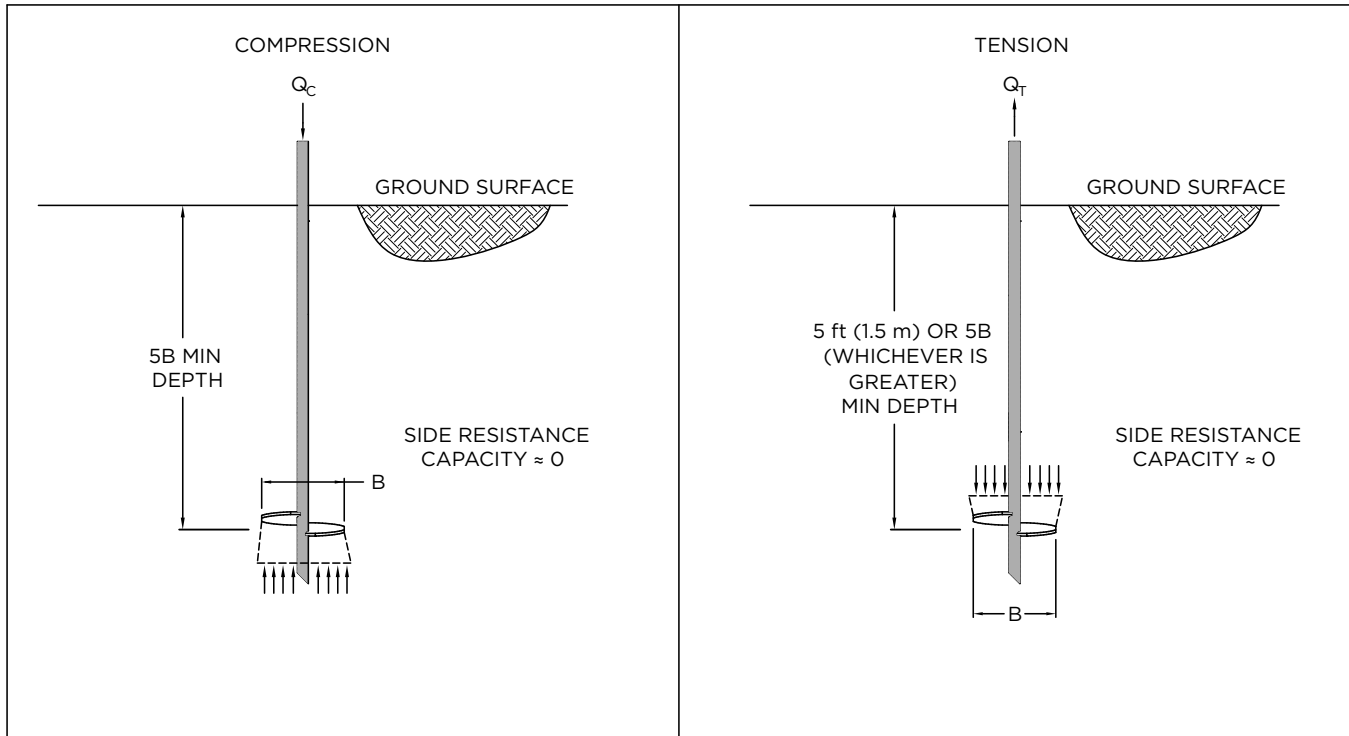
EQUATION 5-18

$$Q_{ult} = A_h(9s_u + \gamma' D)$$

5.2.2.1.b Sands ($\phi' > 0$; $c = 0$)

For clean, saturated sands, the cohesion is normally considered to be zero, and Equation 5-11 is used to calculate the ultimate capacity.

$$Q_{ult} = A_h(q' N_q + 0.5\gamma' B N_\gamma)$$



SINGLE-HELIX PLATE BEARING CAPACITY MODEL—HELICAL PILES WITH SLENDER SHAFTS
FIGURE 5-6

Even in sands with moisture or 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 very small and the width term is often ignored, leaving:

EQUATION 5-19

$$Q_{ult} = A_h q' N_q$$

For deep installations, the bearing capacity factor (N_q) is usually obtained from values used for determining the end-bearing capacity for deep pile foundations, which are different than the values used for shallow foundations. There are a number of recommendations for N_q available in foundation engineering textbooks as shown in Figure 5-7. The difference in N_q values shown in Figure 5-7 is largely related to the assumptions used in the failure mechanism. Figure 5-8 gives a reasonable chart of N_q values as a function of the friction angle of the soil (ϕ') that may be used for helical piles and anchors in cohesionless soils. The value of N_q in Figure 5-8 is obtained from:

EQUATION 5-20

$$N_q = 0.5(12\phi')^{\phi'/54}$$

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 helical piles and anchors, but engineers often use it with deep installation in saturated sands.

5.2.2.1.c Mixed Soils ($\phi' > 0$; $c > 0$)

The ultimate capacity of a deep single-helix helical pile in mixed-grain soils can be calculated from traditional bearing capacity theory using Equation 5-12:

$$Q_{ult} = A_h(cN_c + q'N_q + 0.5\gamma'BN_\gamma)$$

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 ($\phi' = 0$; $c > 0$)

Under tension loading, the ultimate uplift capacity (Q_{ultU}) of a single-helix helical anchor in clay 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. 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:

$$Q_{ultU} = A_h(9s_u + \gamma'D)$$

5.2.2.2.b Sands ($\phi' > 0$; $c = 0$)

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,

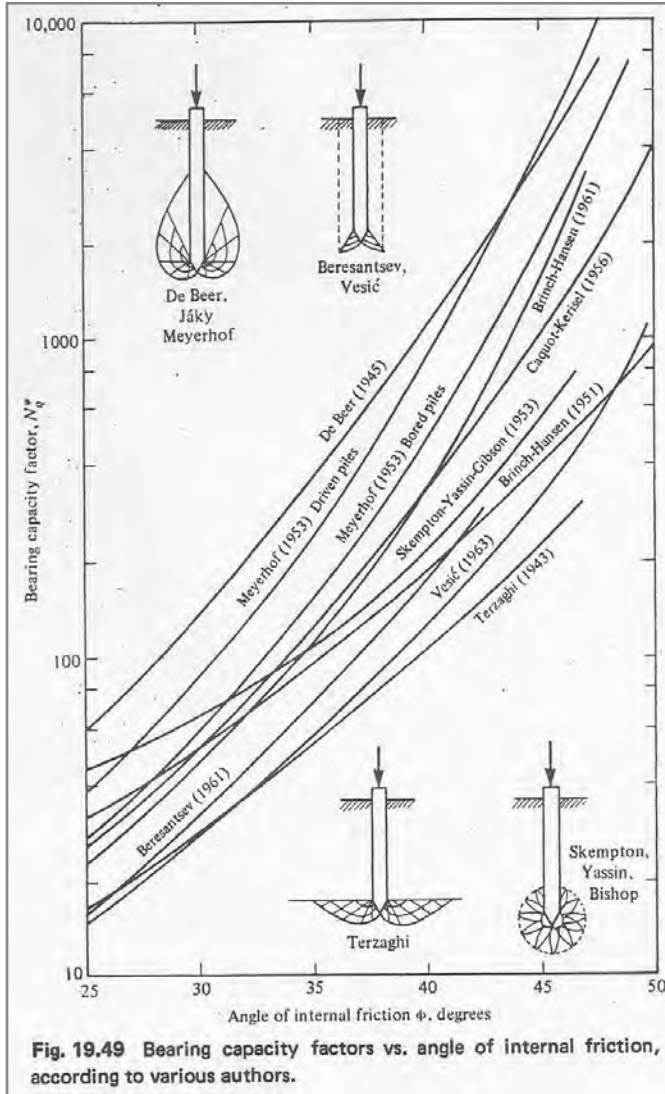


Fig. 19.49 Bearing capacity factors vs. angle of internal friction, according to various authors.

REPORTED VALUES OF BEARING CAPACITY FACTOR (N_q) FOR DEEP FOUNDATIONS IN SANDS [WINTERKORN & FANG (1983)]
FIGURE 5-7

saturated sands, the cohesion is normally considered to be zero, reducing the ultimate uplift capacity to:

$$Q_{ultU} = A_h(q'N_q + 0.5\gamma'BN_\gamma)$$

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

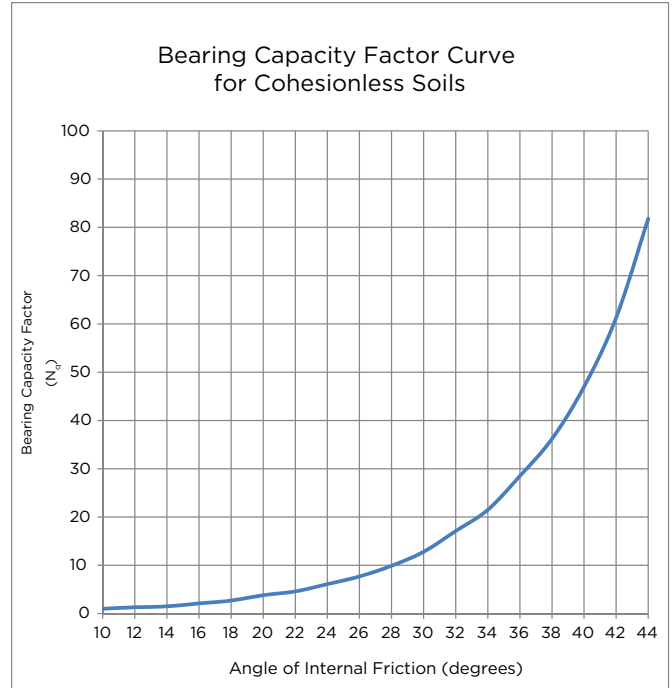
$$Q_{ultU} = A_hq'N_q$$

5.2.2.2.c Mixed Soils ($\phi' > 0$; $c > 0$)

The ultimate capacity of a deep helical anchor in mixed-grain soils can be calculated from traditional bearing capacity theory using Equation 5-12:

$$Q_{ultU} = A_h(cN_c + q'N_q + 0.5\gamma'BN_\gamma)$$

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



RECOMMENDED BEARING CAPACITY FACTOR (N_q) FOR DEEP HELICAL PILES/ANCHORS IN SAND
FIGURE 5-8

5.2.3 MULTI-HELIX HELICAL PILES AND ANCHORS—DEEP INSTALLATION

The ultimate capacity of deep multi-helix helical piles and 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 Figures 5-3b and 5-3d, if the spacing of helix plates is close, the helix plates interact with each other. The capacity is developed along the failure surface of the perimeter shear zone and by end bearing of the end helix plate (the bottom plate for compression loading or the top plate for tension loading). If the spacing of the helix plates is adequate, the capacity is the sum of the capacities developed by the individual helix plates, as shown in Figures 5-3a and 5-3c. There is no interaction between helix plates, and no capacity is developed along the shaft between the helix plates.

In the US, most manufacturers of helical piles and anchors use a standard helix spacing of 3 times the helix diameter. This spacing was originally used in Chance® products over 35 years ago and is assumed to allow individual helix plates to develop full capacity with no interaction between helix plates. Most Chance helical piles and anchors use interhelix 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. A lead section typically includes up to four helix plates. Additional helix plates can be added, if required, with the use of helical extensions. Standard helix sizes and projected

STANDARD HELIX SIZES, TABLE 5-3	
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]

areas are shown in Table 5-3. Comprehensive tables of helix projected areas, for the full plate area and the net plate area without the shaft, are included in Section 7 of this manual for square shaft and round shaft helical piles. The helix plates are usually arranged on the shaft such that their diameters remain constant or increase as the plates get farther from the pilot point (tip). The practical limit on the number of helix plates per pile/anchor is usually four to five in fine-grained soils and six in coarse-grained or granular soils.

5.2.3.1 COMPRESSION LOADING

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

EQUATION 5-21

$$Q_t = \sum Q_h$$

where

- Q_t = Total ultimate capacity of a multi-helix helical pile/anchor
- Q_h = Ultimate 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. This measure is to account for some disturbance of the clay due to anchor installation, and 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 calculated as the summation of the individual plate capacities using Equation 5-21:

$$Q_t = \sum Q_h$$

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

5.2.4 ROUND SHAFT HELICAL PILES AND ANCHORS

Helical piles and anchors are available with a square shaft or a round pipe shaft. Square shaft is used for tension applications and for compression applications when shaft buckling or bracing is not an issue. Round 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 piles. Typical round shaft pile diameters range from 2-7/8 inches (73 mm) to 12-3/4 inches (324 mm). Design for round shaft helical piles is essentially the same as previously described for square shaft piles with two simple modifications: 1) Some provision is usually made to include the additional load capacity developed via side resistance 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 5-12b. 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 side resistance due to skin friction. In addition, load capacity due to side resistance along the pile shaft is generally mobilized only if the shaft diameter is at least 3.5 inches (89 mm).

5.2.4.1 SIDE RESISTANCE IN CLAYS ($\phi' = 0$; $c > 0$)

In clays, the side resistance developed by round shaft helical piles and anchors is considered in much the same way as side resistance developed by driven piles. 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:

EQUATION 5-22

$$\alpha = f_s/s_u$$

where

- α = Adhesion factor
- f_s = Unit side resistance
- s_u = Undrained shear strength of the clay

The value of α is usually obtained from any one of several published charts and is typically related to the absolute value of the undrained shear strength of the clay. Figures 5-9 and 5-10 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 side resistance (Q_f) is then obtained from:

EQUATION 5-23

$$Q_f = \pi d(L)f_s$$

where

- d = Diameter of central shaft
- L = Length of round shaft in contact with soil

The design line given by the American Petroleum Institute (API) shown in Figure 5-11 may also be used in which:

- For $s_u < 500$ psf, $\alpha = 1.0$
- For $s_u > 1500$ psf, $\alpha = 0.5$
- For $500 \text{ psf} < s_u < 1500 \text{ psf}$, α varies linearly between 1.0 and 0.5

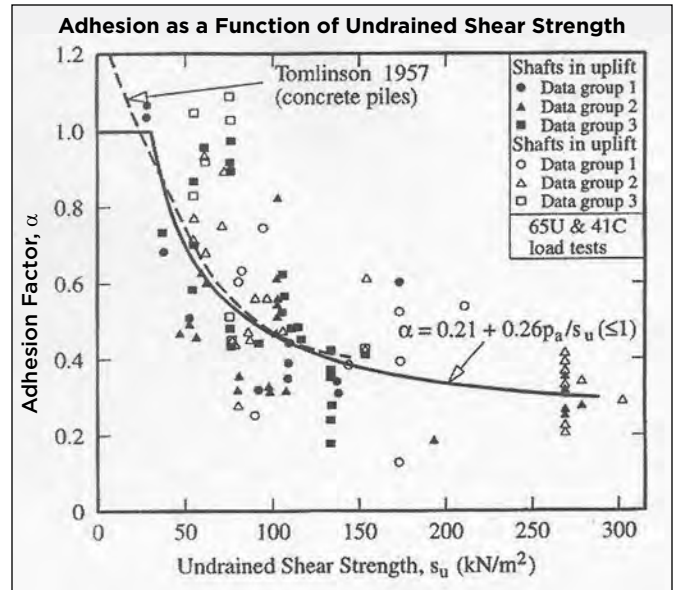
The side resistance should only be calculated for that portion of the shaft length that is in full contact with the soil. This will depend on the length of the lead section, the design of the shaft couplings that connect the pile sections, and the type of soil. In the case of flush connections between extension sections, the entire shaft is in full contact with the soil. On the other hand, flanged and bolted connections generally create an annulus between the shaft and soil as the pile or anchor is installed as shown in Figure 5-12c. 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 side resistance unless the disturbed soil moves back against the shaft or sufficient time is allowed for the soil to recover. If side resistance is calculated for shaft lengths where soil recovery has occurred between couplings, reduced shear strength should be used for the soil in those zones.

5.2.4.2 SIDE RESISTANCE IN SANDS AND MIXED SOILS ($\phi' > 0$; $c \geq 0$)

The side resistance of steel round shaft piles and anchors 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 the 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 5-4.

VALUES OF UNIT SIDE RESISTANCE FOR STEEL PILES IN SAND [NAVY MANUAL DM-7 (1974)], TABLE 5-4

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



VARIATION IN ADHESION FACTOR WITH UNDRAINED SHEAR STRENGTH OF CLAYS [CANADIAN FOUNDATION MANUAL (2006)]
FIGURE 5-9

5.2.5 HELICAL PILE/ANCHOR SPACING AND MINIMUM DEPTH

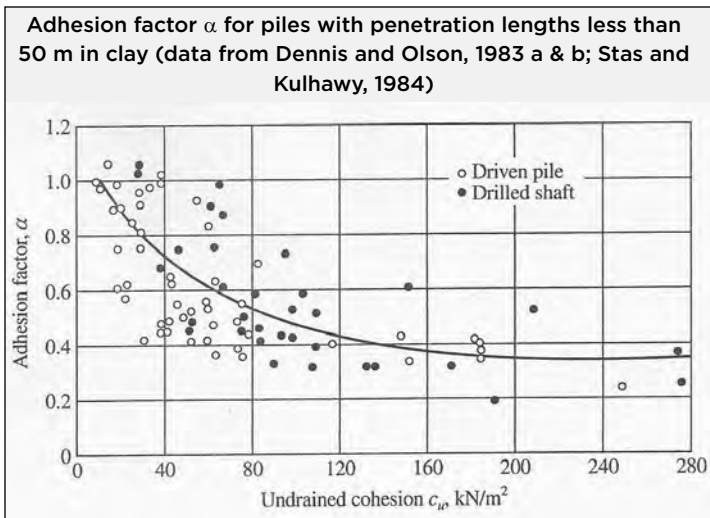
5.2.5.1 REASONABILITY CHECK

It is important to evaluate the validity of the values obtained when determining the bearing capacity and side 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 side resistance value. In situations where soil data is insufficient or not available, a helical trial probe pile/anchor can help determine data such as the location of bearing strata, pile/anchor capacity, the location of soft/loose soil, and the presence of obstructions such as cobbles, boulders, and debris.

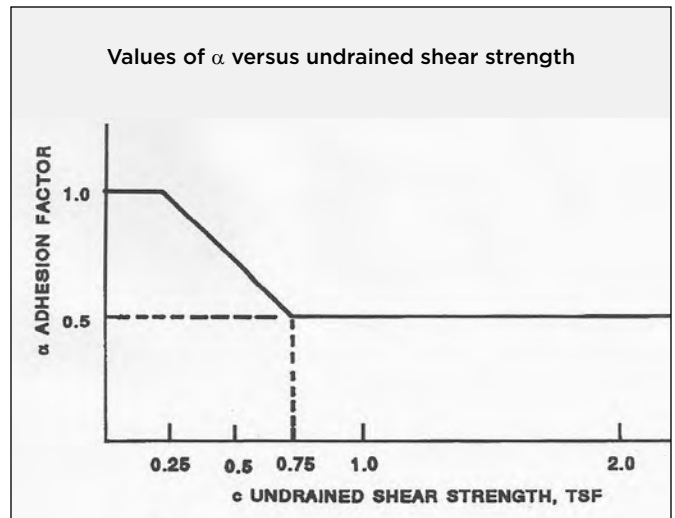
An important step in the process of determining the capacity of a helical pile/anchor is to conduct a reasonability check. The engineer should use the best engineering judgment to perform the reasonability check. This should be based on experience, historical test data, and consulting colleagues. This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

5.2.5.2 HELICAL PILE/ANCHOR SPACING

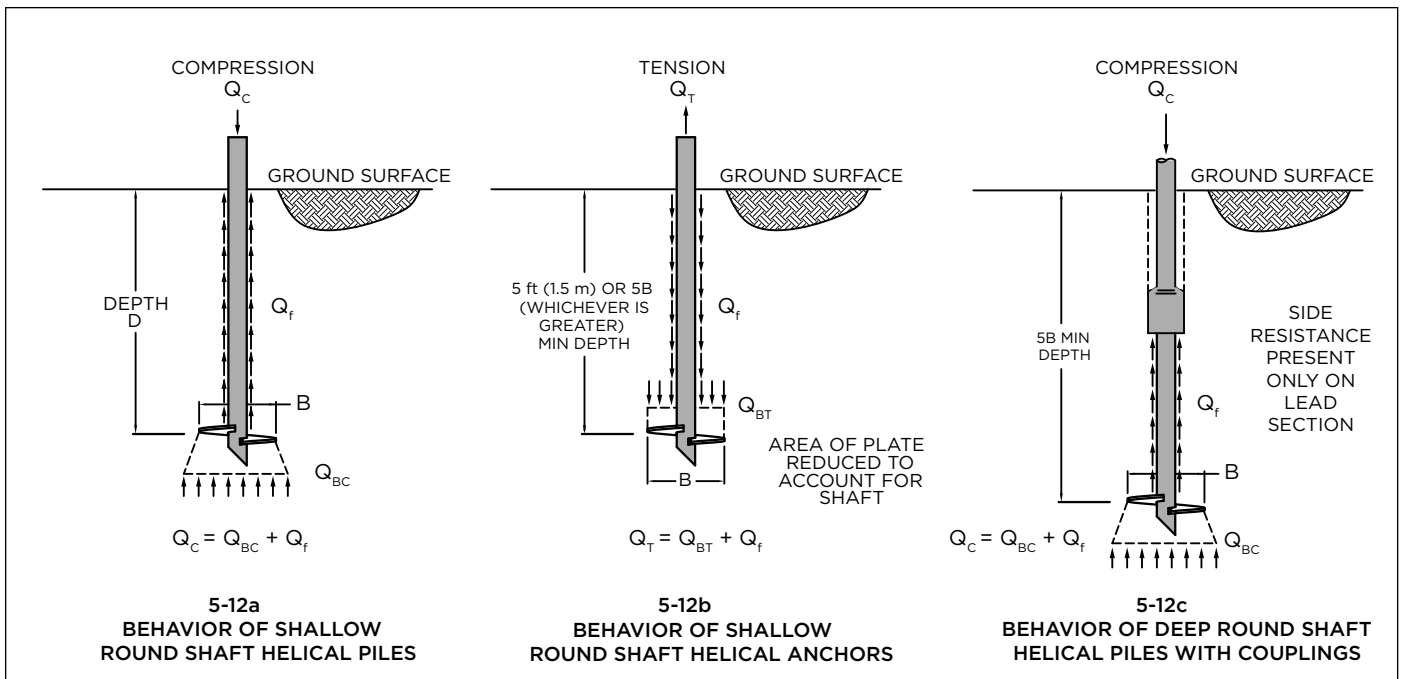
Once the capacity of the helical pile/anchor is determined, turn attention to the location of the foundation element with respect to the structure and to other helical piles/anchors. It is recommended that the center-to-center spacing between adjacent piles/anchors be no less than five times the diameter of the largest helix. The minimum spacing is three feet (0.91m).



VARIATION IN ADHESION FACTOR WITH UNDRAINED SHEAR STRENGTH OF CLAYS [MURTHY (2003)]
FIGURE 5-10



VARIATION IN ADHESION FACTOR FROM AMERICAN PETROLEUM INSTITUTE [ASCE (1993B)]
FIGURE 5-11



INDIVIDUAL BEARING AND SIDE RESISTANCE MODELS FOR HELICAL PILES AND ANCHORS WITH ROUND (PIPE) SHAFTS
FIGURE 5-12



This minimum 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 piles/anchors and other foundation elements, either existing or future, requires special consideration and is beyond the scope of this section.

Research into group effect, or the reduction of capacity due to close spacing, has recently been undertaken by Hubbell Power Systems, Inc., engineers. Bearing capacity theory indicates that capacity reduction due to group effect is possible. Current research indicates the critical horizontal spacing (no group effect) for helical anchors in stiff clay is greater than 2 diameters, but there is no group reduction effect in soft to firm clay. Research also indicates the critical horizontal spacing is greater than 5 diameters in dense sand but is greater than 3 diameters in loose to medium-dense sand. It is considered good practice to install helical piles/anchors into a dense bearing stratum to increase the bearing capacity beyond the required capacity when center-to-center spacing is less than 3 to 5 times the diameter of the largest helix.

5.2.5.3 MINIMUM DEPTH

As mentioned earlier, the minimum embedment depth recommended by Hubbell Power Systems, Inc., for a helical deep foundation is five helix diameters (5B), where B is the diameter of the top-most helix. The 5B depth is the vertical distance from the surface to the top-most helix. Standard practice is to locate the top-most helix 6B to 8B vertically 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 of 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 for a helical deep foundation subject to seasonal variations is three diameters (3B) below the depth of soil where these seasonal variations will occur. For example, frost depths may require embedment depths that exceed the 5B 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 12B, where B 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.

5.2.5.4 CRITICAL DEPTH

In granular soils, helical pile/anchor capacity is a function of the angle of internal friction (ϕ) and vertical effective overburden stress. Therefore, as a helical pile or anchor is extended deeper into soil, theoretical methods predict that the pile capacity will 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/anchor. Critical depth for helical piles is best determined by an experienced foundation engineer. Hubbell recommends the use of critical depths of 20B to 30B in loose, saturated soils at deep depth, where B is the diameter of the largest helix plate. The 20B to 30B critical depth is the depth into a suitable bearing stratum and is not necessarily measured from the ground surface.

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 side resistance capacity equations. Table 5-5 summarizes the required soil properties for different site conditions for design of 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 the Field Vane Test (FVT), Cone Penetration Test (CPT), Piezocone Test (CPTU), Dilatometer Test (DMT), Pressuremeter Test (PMT), or Borehole Shear Test (BST), are preferred. THERE IS NO SUBSTITUTE FOR A SITE-SPECIFIC GEOTECHNICAL INVESTIGATION.

5.3.1 ESTIMATING UNDRAINED SHEAR STRENGTH (s_u) IN CLAYS

The undrained shear strength of saturated clays, silty clays, and clayey silts is not an independent soil property like the liquid limit of clay content, but instead 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.

5.3.1.1 s_u FROM SPT

A number of correlations exist for estimating the undrained shear strength and unconfined compressive strength (q_u) of fine-grained soils from SPT results. Several of these correlations

SOIL PROPERTIES REQUIRED FOR HELICAL PILE/ANCHOR DESIGN FOR VARIOUS SITE CONDITIONS, TABLE 5-5

SOIL PROPERTY CATEGORY	REQUIRED SOIL PROPERTIES		
	SATURATED FINE GRAINED	COARSE GRAINED	UNSATURATED FINE GRAINED MIXED
Shear strength	s_u	ϕ'	c, ϕ'
Unit weight	γ_{sat}	γ_{wet} OR γ_{buoy}	γ_{wet}

are given in Tables 5-6 and 5-7. The undrained shear strength is generally considered to be 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.

5.3.1.2 s_u FROM CPT/CPTU

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

An estimate of s_u can be found from the CPT tip resistance by using an equation derived from the bearing capacity equation:

EQUATION 5-24

$$s_u = (q_c - \sigma_{vo})/N_k$$

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 in which 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). The undrained shear strength is estimated from:

EQUATION 5-25

$$s_u = (q_t - \sigma_{vo})/N_{kt}$$

where

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

REPORTED CORRELATIONS BETWEEN SPT N_{60} VALUE AND UNDRAINED SHEAR STRENGTH (s_u), TABLE 5-6

CORRELATION TO UNDRAINED SHEAR STRENGTH	UNITS OF s_u	SOIL TYPE	REFERENCE
$s_u = 29N_{60}^{0.72}$	kPa	Japanese cohesive soils	Hara et al. (1974)
$s_u = 4.5N_{60}$	tsf	Insensitive overconsolidated clays in U.K.	Stroud (1974)
$s_u = 8N_{60}$ for $N_{60} < 10$ $s_u = 7N_{60}$ for $10 < N_{60} < 20$ $s_u = 6N_{60}$ for $20 < N_{60} < 30$ $s_u = 5N_{60}$ for $30 < N_{60} < 40$	kPa	Guabiro tuba clay	Tavares (1988)
$s_u = 1.39N_{60} + 74.2$	tsf	Tropical soil	Ajayi & Balogun (1988)
$s_u = 12.5N_{60}$ $s_u = 10.5N_{60}$	kPa tsf	Sao Paulo overconsolidated clay	Decourt (1989)

Note: 1 kPa = 20.9 psf

REPORTED CORRELATIONS BETWEEN SPT N_{60} VALUE AND UNCONFINED COMPRESSIVE STRENGTH (q_u), TABLE 5-7

CORRELATION TO UNCONFINED COMPRESSIVE STRENGTH	UNITS OF q_u	SOIL TYPE	REFERENCE
$q_u = 12.5N_{60}$	kPa	Fine grained	Terzaghi & Peck (1967)
$q_u = N_{60}/8$	tsf	Clay	Golder (1961)
$q_u = 25N_{60}$ $q_u = 20N_{60}$	kPa kPa	Clay Silty clay	Sanglerat (1972)
$q_u = 25N_{60}$ $q_u = 15N_{60}$ $q_u = 7.5N_{60}$	kPa	Highly plastic clay Medium-plasticity clay Low-plasticity clay	Sowers (1979)
$q_u = 24N_{60}$	kPa	Clay	Nixon (1982)
$q_u = 62.5(N_{60} - 3.4)$	kPa		Sarac & Popovic (1982)
$q_u = 15N_{60}$	kPa	CL and CL-ML	Behpoor & Ghahramani (1989)
$q_u = 58N_{60}^{0.72}$	kPa	Fine grained	Kulhaway & Mayne (1990)
$q_u = 13.6N_{60}$ $q_u = 9.8N_{60}$ $q_u = 8.6N_{60}$ $q_u = (0.19\pi + 6.2)N_{60}$	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 CPTU pore pressure measurements or by first estimating the stress history from CPT/CPTU results and then converting to undrained shear strength, e.g., NCHRP (2007) and Schnaid (2009), both of which are viable approaches.

5.3.1.3 ESTIMATING SHEAR STRENGTH OF FINE-GRAINED SOILS—OTHER METHODS

5.3.1.3.a Vane Shear Test

Shear strength of fine-grained soils may be measured 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. The test device 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 type of portable vane shear tester 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 scan-



ning 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.

5.3.1.3.b 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 in 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 1/4" and a reading is taken from the sliding scale on the side. The scale is a direct reading of shear strength. Pocket penetrometer values should be used with caution and geotechnical reports should include correlations to unconfined compression strength or cohesion. It is not recommended for use in sands or gravel soils.

5.3.1.3.c 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 1/2 the unconfined compressive strength of undisturbed samples (see Laboratory Testing of Recovered Soil Samples in Section 2 of this manual).

5.3.1.3.d Empirical Correlations

The consistency of clays and other cohesive soils is usually described as very soft, soft, medium, stiff, very stiff, or hard. Values of consistency, overconsolidation ratio (OCR), and undrained shear strength (cohesion) empirically correlated to SPT N_{60} values per ASTM D1586 are given in Table 5-8 (Bowles, 1988). 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 5-8 should be used as a guide.

5.3.2 ESTIMATING FRICTION ANGLE (ϕ') IN SANDS

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

EMPIRICAL VALUES FOR SOIL CONSISTENCY, OVERCONSOLIDATION RATIO, AND UNDRAINED SHEAR STRENGTH VS. SPT N_{60} VALUE, TABLE 5-8

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 \approx 1 to 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 to 3	10-16	1.25 [59.9] to 2 [95.8]	Hard to deform by hand squeezing
Very stiff	Overconsolidated OCR = 4 to 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

5.3.2.1 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 is given in Table 5-9. The correlation of Hatanaka & Uchida (1996) is shown in Figure 5-13, taken from the FHWA Reference Manual on Subsurface Investigations (2002).

5.3.2.2 ϕ' FROM CPT/CPTU

An approach derived from bearing capacity theory, similar to the one used to estimate s_u from the CPT/CPTU tip resistance in clays, may be used to estimate the friction angle of sands. 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):

EQUATION 5-27

$$N_q = (q_c / \sigma'_{vo}) = 0.194 \exp[7.63 \tan(\phi')]$$

where

σ'_{vo} = Vertical effective (corrected for pore water pressure) stress at cone tip

This relationship is shown in Figure 5-15.

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

REPORTED CORRELATIONS BETWEEN SPT N_{60} VALUE AND ϕ' FOR COARSE-GRAINED SOILS, TABLE 5-9

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

EQUATION 5-28

$$\phi' = \arctan[0.1 + 0.38\log(q_t/\sigma'_{vo})]$$

Equation 5-28 is shown in Figure 5-17.

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

EQUATION 5-29

$$\phi' = 17.70 + 11.0\log(q_{t1})$$

where

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

$$\sigma_{atm} = \text{Atmospheric pressure (1 atm = 1 bar = 100 kPa = 1 tsf = 14.7 psi)}$$

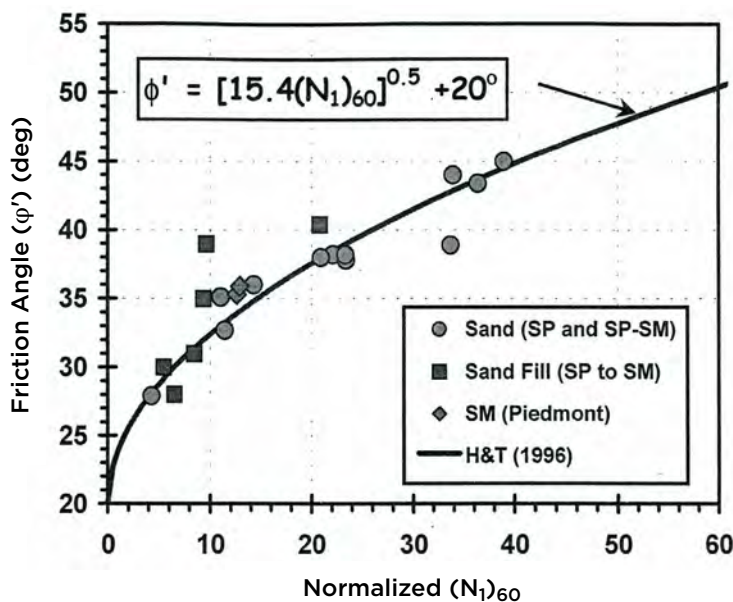
5.3.2.3 EMPIRICAL CORRELATIONS

The relative density of sands, gravels, and other granular soils is usually described as very loose, loose, medium dense, dense,

EMPIRICAL VALUES FOR RELATIVE DENSITY, FRICTION ANGLE, AND UNIT WEIGHT VS. SPT BLOW COUNT (ASSUMING A 20-FOOT (6-METER) DEPTH OF OVERBURDEN AND 70% ROD EFFICIENCY ON HAMMER), TABLE 5-10

DESCRIPTION	Very loose	Loose	Medium dense	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 (γ_{wet}) (pcf)	70-100	90-115	110-130	110-140	130-150	

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_{70} values per ASTM D1586 are given in Table 5-10 (Bowles, 1988). It should be noted that SPT values can be amplified in gravel because a 1" or larger 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.



**PEAK FRICTION ANGLE OF SANDS FROM SPT RESISTANCE—CORRELATION OF HATANAKA & UCHIDA (1996)
[FHWA REFERENCE MANUAL ON SUBSURFACE INVESTIGATIONS (2002)]**

FIGURE 5-13

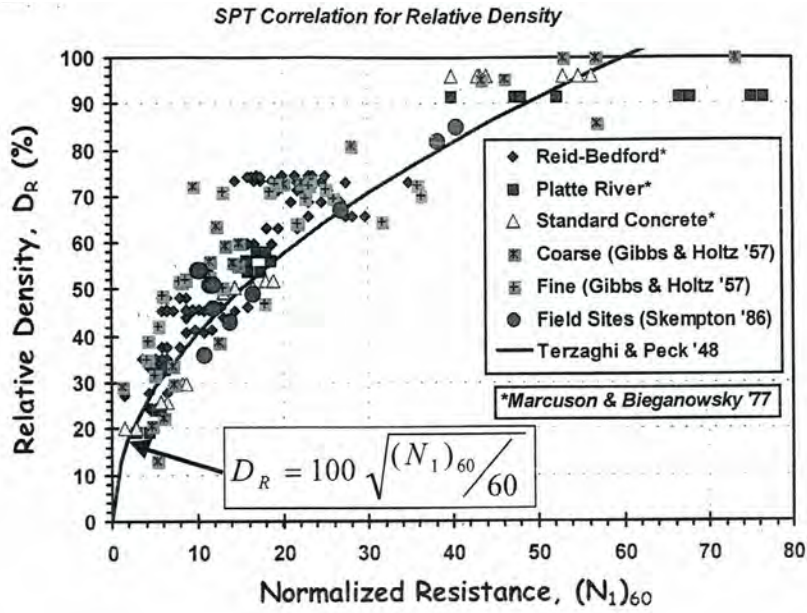


FIGURE 5-14

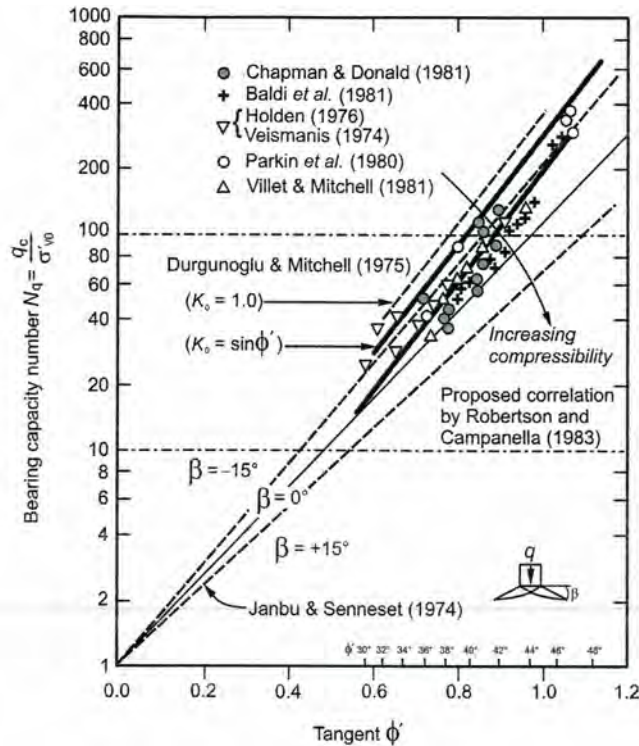
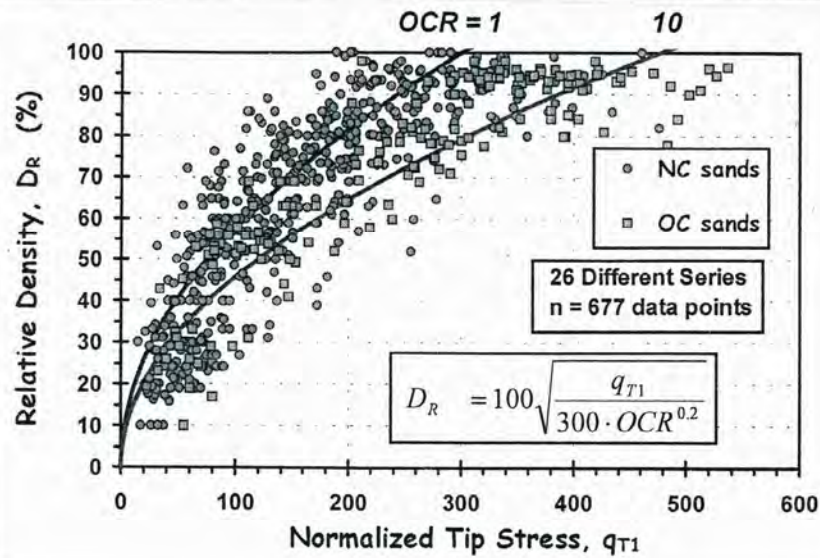
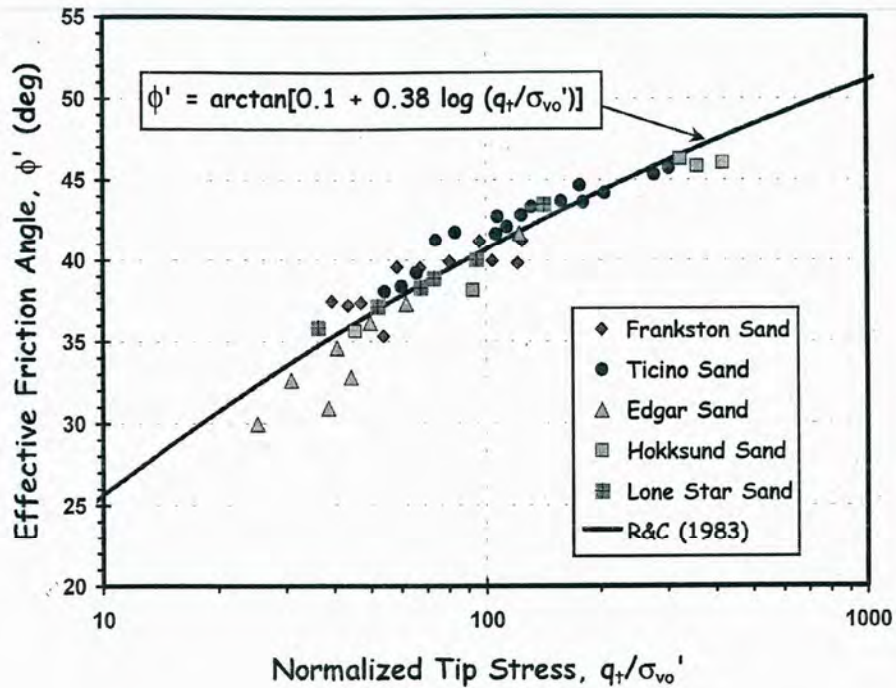


FIGURE 5-15

DESIGN METHODOLOGY



RELATIONSHIP BETWEEN RELATIVE DENSITY AND CPT NORMALIZED TIP STRESS FOR NORMALLY CONSOLIDATED (NC) AND OVERCONSOLIDATED (OC) SANDS
FIGURE 5-16



RELATIONSHIP BETWEEN FRICTION ANGLE AND CPTU EFFECTIVE TIP RESISTANCE
FIGURE 5-17



5.3.3 DIRECT ESTIMATE OF UNIT SIDE 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 by various authors. 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_{60} value and suggested that most of these correlations could be expressed using the general equation:

EQUATION 5-30

$$f_s = \beta + \alpha N$$

Lutenegger (2011) presented a summary, shown in Table 5-11, of more-or-less “global” reported correlations between SPT N_{60} values and unit side resistance for both driven and bored piles in a number of different soil materials.

Engineers might ask, “Why should the SPT N_{60} value correlate to unit 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 5-11 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 neglected with little effect on the correlation, which simplifies equation 5-30 to:

EQUATION 5-31

$$f_s = \alpha N$$

Note that equation 5-31 is similar to equation 5-22, suggesting a correlation between SPT N_{60} values and undrained shear strength (s_u) in fine-grained soils.

2. The value of α ranges from 0.3 to 12.5.
3. The observations presented in Table 5-11 generally suggest higher values of α for fine-grained soils as compared to coarse-grained soils.
4. Values of α are generally higher for driven piles as compared to bored piles.

The values of α vary considerably for several obvious reasons related to the pile data and the SPT data.

With 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 and H-Piles; construction practices such as dry bored and wet bored; 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 5-11 with caution.

5.4 FACTOR OF SAFETY

The equations discussed above are used to obtain the ultimate capacity of a helical pile/anchor. For allowable (working) 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 from earth pressure, water pressure, and surcharge loads (from buildings, etc.) for helical tieback or Soil Screw® earth retention anchor 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 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 (ASD). 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. These are typical geotechnical resistance factors for helical piles:

Compression: 0.65 to 0.75

Tension: 0.55 to 0.65



REPORTED CORRELATIONS BETWEEN SPT N_{60} VALUE AND UNIT SIDE RESISTANCE [LUTENEGER (2011)], TABLE 5-11

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	0	3	Bazaraa & Kurkur (1986)	
	Cohesionless	0	1.8		
	Sandy Clayey	29 34	2.0 4.0	Kanai & Yubuuchi (1989)	
Miscellaneous	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		
	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		
	Clayey soil Sandy soil	0 0	10.0 3.0	Matsui (1993)	
	Miscellaneous	17.3 18.2	1.18 0.65	Vrymoed (1994)	
	Miscellaneous	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)	30	2.0	Yamashita et al. (1987)
Cohesive ($F_s < 150$ kPa)		0	5.0		

Note: $f_s = \beta + \alpha N_{60}$ (f_s in units of kPa)



5.5 HELICAP HELICAL CAPACITY DESIGN SOFTWARE

Hubbell Power Systems, Inc., engineers developed HeliCAP® helical capacity design software to determine the bearing capacity of helical piles and anchors in soil. 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 (depth ≥ 5B). A key feature of HeliCAP software is that it is 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 manual. The most common information available from the soil boring is the soil profile, groundwater location, and SPT blow count data per ASTM D1586. To utilize this data, the software 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 equations, factors, empirical values, etc., presented in this section are used in the HeliCAP v3.0 helical capacity design software. Using this software makes the selection of a helical pile/anchor much quicker versus using 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 software are no better than the input data used to generate them.

The software will assist in determining an appropriate helical lead configuration and overall pile/anchor 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 pile/anchor 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.

HeliCAP® design software calculates ultimate capacity and must have an appropriate Factor of Safety applied to the results. The software 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 software. For additional as-

sistance, refer to the Help screen or contact Hubbell application engineers.

The image below is from HeliCAP v3.0 helical capacity design software. It shows a typical work page with the soil profile on the left and helical pile capacity on the right.

5.5.1 HELICAP SOFTWARE BEARING CAPACITY METHODOLOGY

As detailed earlier in this section, the individual plate bearing method states that the capacity of a single or multi-helix pile/anchor is determined by summing the bearing capacities of the individual helix plates. Thus:

$$Q_t = \sum Q_h$$

where

Q_t = Total ultimate multi-helix pile/anchor capacity

Q_h = Individual helix ultimate capacity

HeliCAP design software determines the ultimate bearing capacity of an individual helix with 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 manual for the mechanical strengths of helix plates.

EQUATION 5-32

$$Q_h = A_h(cN_c + q'N_q) \leq Q_s$$

where

A_h = Projected helix area

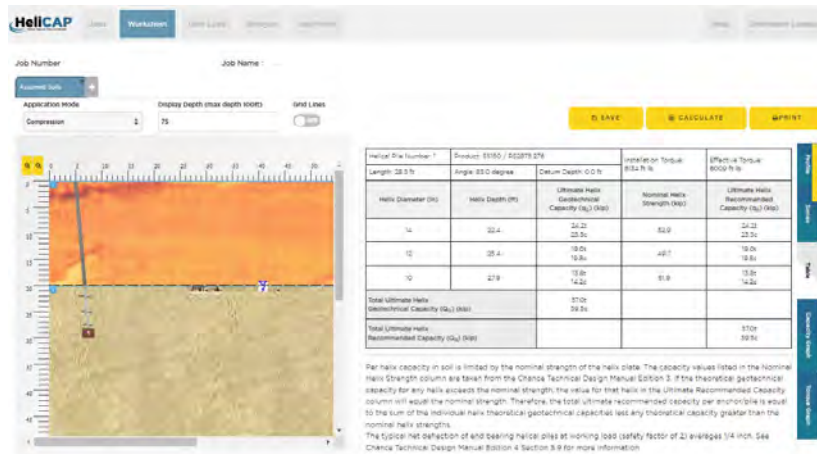
Q_s = Capacity upper limit = helix mechanical strength

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.

5.5.1.1 SANDS ($\phi' > 0$; $c = 0$)

HeliCAP design software determines the ultimate bearing capacity of a helix (Q_h) in a non-cohesive sand or gravel soil with Equation 5-33, which is derived from Equation 5-32 with the fine-grain (clay) term eliminated (also see Equation 5-19).

DESIGN METHODOLOGY





The bearing capacity factor (N_q) is dependent on the angle of internal friction (ϕ') of the soil. When a value is provided for the friction angle, HeliCAP software uses Figure 5-8 (N_q vs. ϕ') and Equation 5-20 to determine the value for N_q . The graph in Figure 5-8 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 5-20 was written for the curve shown in Figure 5-8, which is Myerhof's N_q values divided by 2 for long-term applications. When the angle of internal friction is not known, the software estimates it (and N_q) by using blow counts obtained from the Standard Penetration Test per ASTM D1586. Equation 5-34 provides 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. **Note: The correlated ϕ' and N_q values determined by HeliCAP software can be overridden. This is encouraged when more reliable soil data are available.**

EQUATION 5-33

$$Q_h = A_h q' N_q = A_h \gamma' D N_q$$

where

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

EQUATION 5-34

$$\phi' = 0.28 N_{60} + 27.4$$

where

- ϕ' = Angle of internal friction
- N_{60} = Blow count per ASTM D1586 Standard Penetration Test

In the event unit weight values are not available, HeliCAP software uses the following equations to obtain estimated unit weight values when blow counts from ASTM D1586 Standard Penetration Tests are available.

EQUATION 5-35

$$N_{60} = 0 \quad \gamma = 65 \text{ (lb/ft}^3\text{)}$$

EQUATION 5-36

$$0 < N_{60} \leq 7 \quad \gamma = 60 + 5N_{60} \text{ (lb/ft}^3\text{)}$$

EQUATION 5-37

$$8 \leq N_{60} \leq 10 \quad \gamma = 100 \text{ (lb/ft}^3\text{)}$$

EQUATION 5-38

$$11 \leq N_{60} < 50 \quad \gamma = 90 + N_{60} \text{ (lb/ft}^3\text{)}$$

EQUATION 5-39

$$N_{60} \geq 50 \quad \gamma = 140 \text{ (lb/ft}^3\text{)}$$

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 software can be overridden. This is encouraged when more reliable soil data are available.

5.5.1.2 FINE-GRAIN COHESIVE SOILS ($\phi' = 0$; $c > 0$)

HeliCAP® design software determines the ultimate bearing capacity of a helix (Q_h) in a cohesive or fine-grained soil with Equation 5-40, which is derived from Equation 5-32 with the coarse-grained (sand) term eliminated. Equation 40 is similar to Equation 5-18 with the overburden term neglected. The bearing capacity factor (N_c) is 9 provided the installation depth below grade is at least five times the diameter of the top-most helix.

EQUATION 5-40

$$Q_h = A_h c N_c = A_h (s_u) 9$$

where

- A_h = Projected helix area
- c = Cohesion; for $\phi' = 0$, c = undrained shear strength = s_u
- N_c = Bearing capacity factor for deep failure = 9 (depth $\geq 5B$)

In the event that cohesion or undrained shear strength values are not available, HeliCAP software uses the following equation to estimate undrained shear strength values when SPT blow counts 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. Results obtained using estimated shear strength values should be used with caution.

NOTE: The correlated undrained shear strength values determined by HeliCAP software can be overridden. This is encouraged when more reliable soil data are available.

EQUATION 5-41

$$c \text{ (ksf)} = N_{60}/8 = 0.125 N_{60}$$

$$c \text{ (kPa)} = 6 N_{60}$$

where

- c = Cohesion; for $\phi' = 0$; c = undrained shear strength = s_u
- N_{60} = Blow count value per ASTM D1586 Standard Penetration Test

If unit weight values are not available, the software uses the following equations to obtain estimated unit weight values when blow counts from ASTM D1586 Standard Penetration Tests are available.

EQUATION 5-42

$$0 < N_{60} \leq 19 \quad \gamma = 80 + 2N_{60} \text{ (lb/ft}^3\text{)}$$

EQUATION 5-43

$$20 \leq N_{60} \leq 40 \quad \gamma = 120 \text{ (lb/ft}^3\text{)}$$

**EQUATION 5-44**

$$41 \leq N_{60} < 50 \quad \gamma = 120 + 2(N_{60} - 40) \text{ (lb/ft}^3\text{)}$$

EQUATION 5-45

$$N_{60} \geq 50 \quad \gamma = 140 \text{ (lb/ft}^3\text{)}$$

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 software can be overridden. This is encouraged when more reliable soil data are available.**

5.5.1.3 MIXED SOILS ($\phi' > 0$; $c > 0$)

HeliCAP software determines the bearing capacity of a mixed soil, one that exhibits cohesion and friction properties, by use of Equation 5-32. This is straightforward when accurate values are available for the cohesion (undrained shear strength) and friction terms (ϕ' & γ') of the equation. It is not possible to use ASTM D1586 SPT blow count correlations to determine all soil strength variables in the bearing capacity equation. Therefore, the designer must take another approach 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.

5.5.2 HELICAP SOFTWARE SIDE RESISTANCE METHODOLOGY

As discussed earlier in this section, the side resistance (Q_f) developed by round shaft or grouted-shaft helical piles is considered similarly to side resistance developed by driven piles. HeliCAP design software uses the traditional approach presented in most foundation design textbooks.

The general equation is:

EQUATION 5-46

$$Q_f = \sum[\pi(B)f_s(\Delta L_f)]$$

where

- B = Diameter of steel or grout pile column
- f_s = Unit side resistance (sum of friction and adhesion between soil and pile)
- ΔL_f = Incremental pile length over which πB and f_s are considered to be constant

HeliCAP software uses two empirical methods to calculate side resistance: The Gouvenot method and the US Department of the 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. The software uses the Gouvenot method to calculate side

resistance for grouted columns only (Helical Pulldown® micropiles). The US Navy method uses the Department of the Navy Design Manual 7, Soil Mechanics, Foundations and Earth Structures (1974). The software uses the Navy method to calculate side resistance for both grouted columns and steel round shafts.

5.5.2.1 GOUVENOT METHOD

Gouvenot reported a range of values for unit side resistance 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 $35^\circ < \phi < 45^\circ$ and $c = 0$:

EQUATION 5-47

$$f_s = \sigma_o \tan(\phi)$$

where

- σ_o = Mean normal stress for the grout column

- Type II: Mixed soils; fine, loose silty sands with $20^\circ < \phi < 30^\circ$ and sandy clays with $205 \text{ psf} < c < 1024 \text{ psf}$ ($9.8 \text{ kPa} < c < 49 \text{ kPa}$)

EQUATION 5-48

$$f_s = \sigma_o \sin(\phi) + (c) \cos(\phi)$$

- Type III: Clays with $1024 \text{ psf} < c < 4096 \text{ psf}$ ($49 \text{ kPa} < c < 196 \text{ kPa}$)

EQUATION 5-49

$$f_s = c$$

for

$$1024 \text{ psf} < c < 2048 \text{ psf} \quad (49 \text{ kPa} < c < 98 \text{ kPa})$$

and:

EQUATION 5-50

$$f_s = 2048 \text{ psf (98 kPa)}$$

for

$$2048 \text{ psf} \leq c < 4096 \text{ psf} \quad (98 \text{ kPa} \leq c < 196 \text{ kPa})$$

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

5.5.2.2 DEPARTMENT OF THE NAVY DESIGN MANUAL 7 METHOD

- For cohesive soils (α method):

EQUATION 5-51

$$Q_f = \sum[\pi(B)C_a(\Delta L_f)]$$

where

- C_a = Adhesion factor (see Table 5-12)



- For cohesionless soils (α method):

EQUATION 5-52

$$Q_f = \Sigma[\pi B(q')K \tan(\phi')(\Delta L_f)]$$

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 pile shaft

- For cohesionless soils (alternate Navy method):

EQUATION 5-53

$$Q_f = \Sigma[\pi(B)S(\Delta L_f)]$$

where

- S = Average side resistance on pile surface area = $P_o \tan(\phi')$ (see Table 5-13)
- P_o = Average overburden pressure

For steel round shaft piles in sand, HeliCAP software uses the alternate Navy method to calculate side resistance with Equation 5-46 and f_s values from Table 5-4.

Tables 5-4, 5-12, and 5-13 are derived from graphs in the Department of the Navy Design Manual 7, Soil Mechanics, Foundations and Earth Structures (1974). Later editions of Design Manual 7 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 $D = 20B$.” (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 side resistance capacity of a Chance® Helical Pulldown® micropile.

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.

RECOMMENDED ADHESION VALUES IN CLAY [NAVY MANUAL DM-7 (1974)], TABLE 5-12

PILE TYPE	SOIL CONSISTENCY	COHESION (c) (psf)	ADHESION (C_a) (psf)
Grout	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

ROUTED PILES IN SAND [NAVY MANUAL DM-7 (1974)], TABLE 5-13

P_o (psf)	EFFECTIVE ANGLE OF INTERNAL FRICTION (ϕ') (degrees)				
	20	25	30	35	40
	S = AVERAGE SIDE 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

- 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 the relative economic feasibility of helical pile/anchor options if more than one combination of helix configuration and overall length can be used.

5.7 LATERAL CAPACITY OF HELICAL PILES

5.7.1 INTRODUCTION

The primary function of a deep foundation is to resist axial loads, but 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 the soil. 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/anchors do not provide any significant resistance to lateral loads. However, round shaft (RS) helical piles/anchors and Helical Pulldown® micropiles can provide significant resistance to lateral loads depending on the soil conditions.

In recent years, a considerable amount of research has been conducted 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-deflection lateral load.

5.7.2 LATERAL RESISTANCE—METHODS USED

Most helical piles/anchors have slender shafts [diameter less than 3 inches (89 mm)] that offer limited resistance to lateral loads when installed vertically. 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 battered 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 battered helical piles/anchors to resist only lateral loads. When battered piles are required to resist both vertical and lateral loads, it is good practice to limit the pile inclination angle to less than 15°. Figure 18 presents lateral resistance methods for helical piles.

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. Expansive soils, compressible strata, and liquefiable soils can result in a void under footings and pile caps.

5.7.2.1 FINITE-DIFFERENCE METHOD

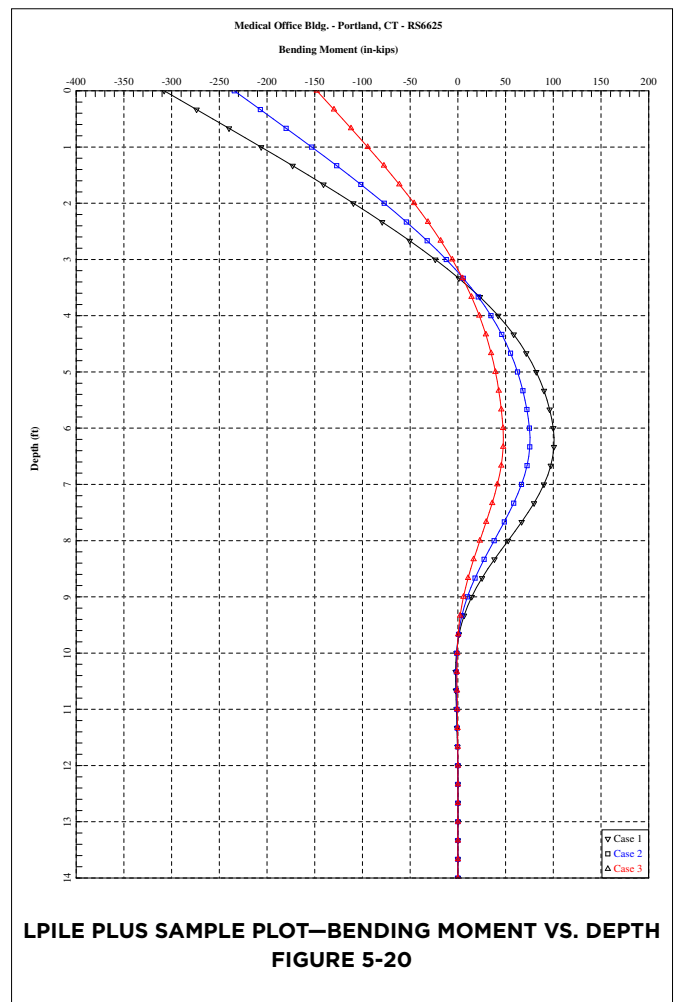
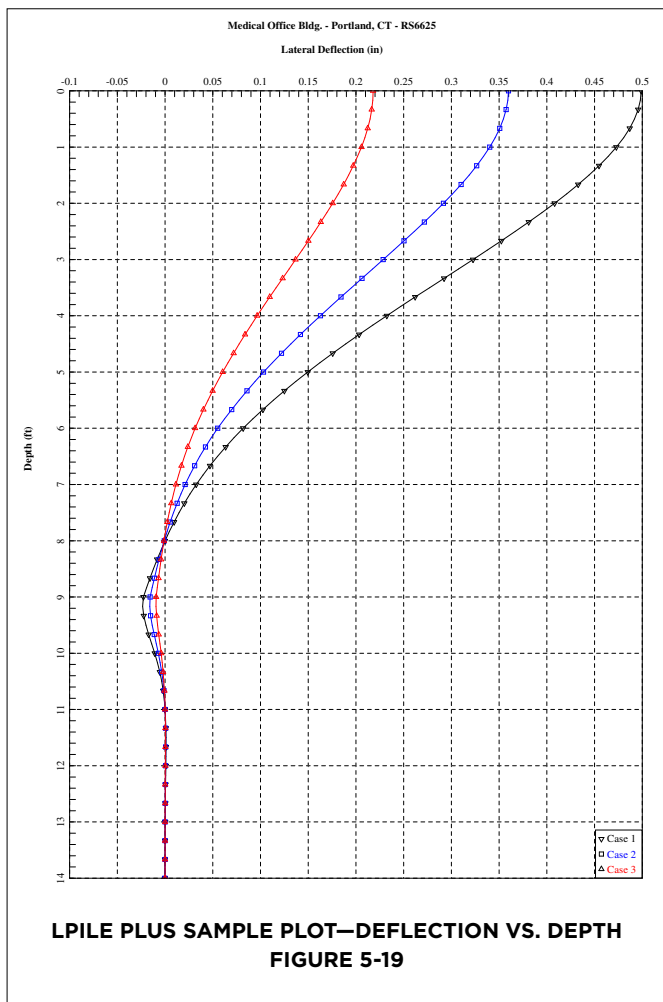
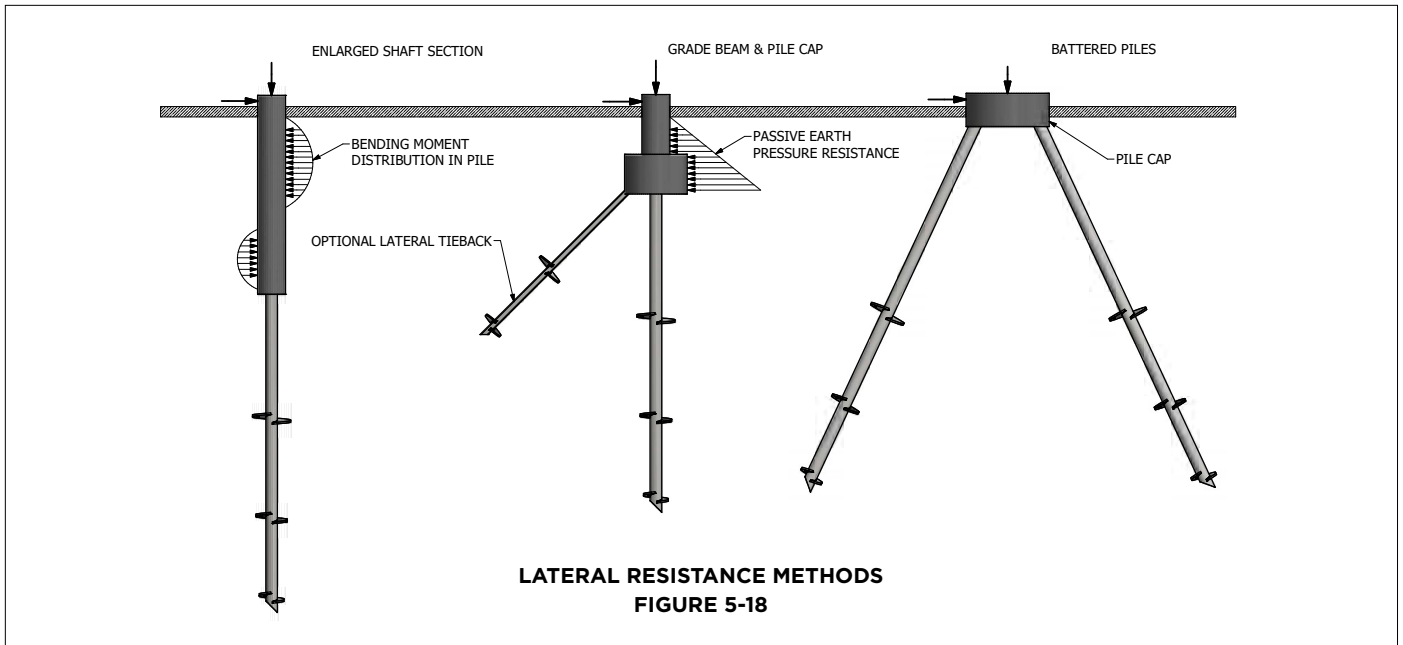
Several computer programs, such as LPILE (Ensoft, Austin, TX), are revisions of the COM624 program (Matlock and Reese) and its predecessor Beam-Column 28 (Matlock and Haliburton) that both use the p-y concept, i.e., soil resistance is a nonlinear function of pile deflection, which was further developed 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 nonlinear, fourth-order differential equation, which is explained in greater detail on page 5-32. 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 5-19 and 5-20 are sample LPILE Plus plots of lateral shaft deflection and bending moment vs. depth with the top of the pile 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 the pile's ability to withstand bending. Many geotechnical consultants use LPILE or other soil-structure interaction programs to predict soil-pile response to lateral loads.

5.7.2.2 BROMS' (1964A & 1964B) METHOD

Broms' method is best suited for applications where the top section of the helical pile/anchor 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 illustrates this. It uses Broms' method for short piles in cohesive soil. A short pile is one that is rigid enough that it will move by rotation or translation in the direction the load is tending. A long pile is one for which the top will rotate or translate without moving the bottom of the pile, i.e., a plastic hinge will form.

Broms developed lateral capacity methods for short and long piles in cohesive and non-cohesive soil. Broms theorized that a short, free-headed pile rotates about a center, above the lower end of the pile, without substantial deformation along its axis. The resistance is the sum of the net of the earth pressures above the center of rotation 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.



DESIGN METHODOLOGY



Figure 5-21 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 for the designer to obtain and review Broms' technical papers (see References at the end of this section) to learn about the various solution methods in 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.

5.7.2.3 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 (circa 1857) method, which assumes no soil cohesion or wall-soil friction. One can use known or assumed soil parameters to determine the magnitude of the passive earth pressure minus the active earth pressure on the other side of the foundation as shown in Figure 5-22. The following are general equations to calculate active and passive pressures on a wall for the simple case of a frictionless vertical face and a horizontal ground surface. Equations 5-57 and 5-58 are Rankine equations for sand, and Equations 5-59 and 5-60 are the previous equations modified to include cohesion in 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 the shear strength on the failure surface is completely mobilized.
3. The resisting element is vertical and resultant forces are horizontal.

EQUATION 5-54

$$K_0 = 1 - \sin(\phi')$$

EQUATION 5-55

$$K_a = \tan^2(45 - \phi'/2)$$

EQUATION 5-56

$$K_p = \tan^2(45 + \phi'/2)$$

For granular soil (sand):

EQUATION 5-57

$$P_a = K_a \rho H^2 / 2$$

EQUATION 5-58

$$P_p = K_p \phi' \rho H^2 / 2$$

For cohesive soil (clay):

EQUATION 5-59

$$P_a = K_a \rho H^2 / 2 - 2cH + 2c^2 / \phi' \rho$$

EQUATION 5-60

$$P_p = K_p \rho H^2 / 2 + 2cH$$

COEFFICIENTS OF EARTH PRESSURE [DAS (1987)],

TABLE 5-14

SOIL	K_0 , DRAINED	K_0 , TOTAL	K_a , TOTAL	K_p , TOTAL
Clay, soft *	0.6	1	1	1
Clay, hard *	0.5	0.8	1	1
Sand, loose	0.6	0.53	0.2	3
Sand, dense	0.4	0.35	0.3	4.6

* Assume saturated clays

where

- K_0 = Coefficient of earth pressure at rest
- ϕ' = Effective friction angle of soil
- K_a = Coefficient of active earth pressure
- K_p = Coefficient of passive earth pressure
- P_a = Active earth pressure
- ρ = Unit weight of soil
- H = Height of wall or resisting element
- c = Cohesion
- P_p = Passive earth pressure

Equations 5-54 through 5-60 are from Department of the Navy Design Manual 7.

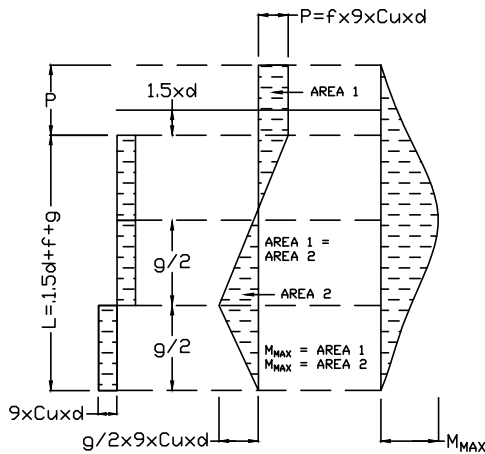
Table 5-14 is a tabulation of the coefficients for at-rest, active, and passive earth pressure for various soil types, relative densities, and consistencies.

Using the Rankine solution may be an over-simplification of the problem but tends to be conservative since the height of the projected area of the footing or pile cap is not large and the cohesion term will generally be small. Design Example 8-15 in Section 8 illustrates the use of the Passive Resistance method to determine the lateral capacity of a foundation.

5.7.2.4 BATTERED CHANCE® HELICAL PILES/ANCHORS FOR LATERAL LOADING

Lateral loads are commonly resolved with battered helical piles and anchors. The method is to statically resolve the axial load capacity into its vertical and horizontal components. As stated earlier, it is best to use vertically installed helical piles and anchors to resist only vertical loads and battered helical piles and anchors to resist only lateral loads.

Chance helical piles and anchors have been supplied to the seismic-prone areas of the west coast of the United States and Canada for over 35 years for civil construction projects. In tension applications, they have been in service for over 60 years. They have been subjected to many earthquakes and aftershocks with good experience. To date, there have been no ill effects observed using battered helical piles and anchors in seismic areas. These foundations, both vertically installed and battered, have been subjected to several earthquakes of magnitude 7+ on the Richter scale with no adverse effects. Anecdotal evidence indicates that the structures on helical piles experienced less earthquake-induced distress than their adjacent structures on other types of foundations. Their performances were documented anecdotally in technical literature, including the Engineering News Record.



SHEAR AT DEPTH $((1.5d)+f)=0$.

$$f \times 9 \times Cu \times d = P$$

$$f = P/9 \times Cu \times d$$

$$AREA 1 = (P \times (e + 1.5d)) + \frac{1}{2} \times f \times P$$

$$= P \times (e + 1.5d + \frac{1}{2} \times f)$$

$$AREA 2 = \frac{1}{2} \times g \times g/2 \times 9 \times Cu \times d$$

$$= (g^2/4) \times 9 \times Cu \times d$$

$$= 2.25 \times d \times g^2 \times Cu$$

$$g = ((P \times (e + 1.5d + .5f)) / (2.25 \times d \times Cu))^{\frac{1}{2}}$$

$$M_{MAX} = AREA 1 \text{ or } AREA 2$$

$$M_{MAX} = P \times (e + 1.5d + .5f)$$

L = REQUIRED DEPTH INTO SOIL WITH COHESION OF Cu

$$L = 1.5d + f + g$$

SOIL REACTION (KIPS/FT.)
PILE SHEAR (KIPS)
PILE MOMENT (FT. KIPS)

RECOMMENDED UNITS

- d = PILE DIAMETER (FT.)
- Cu = SOIL COHESION (KSF)
- P = LATERAL LOAD
- f = FT.
- g = FT.
- M_{MAX} = MAXIMUM PILE BENDING MOMENT (FT KPS)
- FB_{MAX} = MAXIMUM PILE BENDING STRESS (KSI)

ENERGY STRUCTURES INCORPORATED	
TITLE: BROM'S METHOD - COHESIVE SOILS	
DWG NO:	921102

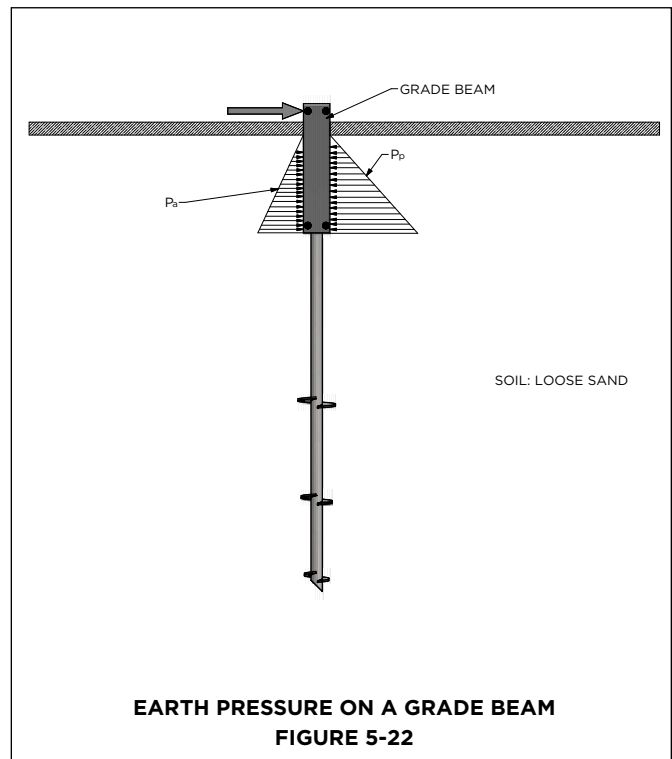
BROMS' METHOD FOR SHORT PILES IN CLAY (ENERGY STRUCTURES, INC., 1994)
FIGURE 5-21

Full scale seismic tests of helical piles were performed in 2016 at the University of California - San Diego shake table. Several different simulations were performed, including the Northridge and Takatori seismic events. Various helical pile configurations were installed in the test box, including square and round shaft piles of varying diameters. The reader is referred to numerous documents published about these studies (Cerato et al., 2017, Elsayw et al., 2017 and 2019).

5.7.3 ADDITIONAL COMMENTS

The lateral capacity of round shaft (RS) helical piles and anchors is greater than that of square shaft (SS) because of the larger section sizes. Typical pipe diameters of 2-7/8" (73mm), 3-1/2" (89 mm), and 4-1/2" (114 mm) OD are used for Chance® round shaft helical piles. As shown in Design Example 8-13 in Section 8, enlarged shaft sections are used for certain applications. From a practical standpoint, the largest diameter helical pile available from Hubbell Power Systems, Inc., is 10-3/4" diameter, but larger shaft diameters are available on a project-specific basis.

As previously noted, there are several other methods used to analyze the lateral capacity of pile shafts. Murthy (2003) also presented a direct method for evaluating the lateral behavior of battered (inclined) piles.



EARTH PRESSURE ON A GRADE BEAM
FIGURE 5-22

DESIGN METHODOLOGY

5.8 BUCKLING/BRACING/ SLENDERNESS CONSIDERATIONS

5.8.1 INTRODUCTION

Buckling of slender foundation elements is a common concern among designers and structural engineers. The literature shows that several researchers have addressed buckling of piles and micropiles over the years [Bjerrum (1957), Davisson (1963), Mascardi (1970), and Gouvenot (1975)]. Their results generally support the conclusion that buckling is likely to occur only in soils with very poor strength properties, such as peat, very loose sands, and soft clay.

However, it cannot be inferred that buckling of a helical pile will never occur. Buckling of helical piles in soil is a complex problem best analyzed using numerical methods on a computer. It involves parameters such as the shaft section and elastic properties, coupling strength and stiffness, soil strength and stiffness, and the eccentricity of the applied load. This section presents a description of the procedures available to evaluate buckling of helical piles and recommendations that aid the systematic performance of buckling analysis.

Buckling analysis of helical piles under compression loads, especially square shaft helical piles, may be important in three situations:

1. When an end-bearing pile is relatively long (>20 feet [>6 m]) and is installed through very soft clay into a very hard underlying layer.
2. When a pile is installed in loose, saturated clean sand that undergoes liquefaction during an earthquake event.
3. When a pile is subject to excessive eccentric load without adequate bracing.

5.8.2 BRACING

Designers and structural engineers must consider bracing of pile foundation elements, especially helical piles and resistance piers with slender shafts. Section 1810.2.2 of the 2021 Interna-

tional Building Code requires deep foundations to be braced to provide lateral stability in all directions. Bracing can be provided in various ways including pile groups of three or more; alternate lines of piles spaced apart; and using slabs, footings, grade beams, and other foundation elements to provide lateral stability. When Chance® helical piles and Atlas Resistance® piers are used for foundation repair, the piers must be braced as in situation 3 above. Figures 5-23 and 5-24 show two methods that are often used to ensure adequate bracing is achieved.

Figure 5-23 is a portion of a grade beam foundation underpinned with Atlas Resistance piers. The grade beam provides torsional stiffness based on its section properties and steel reinforcement. The 90° foundation element on the left end also provides torsional and shear stiffness. Figure 5-24 is a portion of a long, continuous grade beam foundation underpinned with Atlas Resistance piers. The piers are staggered and alternated on the inside and outside, which provides bracing.

5.8.3 BUCKLING BACKGROUND

Buckling of columns most often relates to the allowable compression load for a given unsupported length. The mathematician Leonhard Euler solved the question of critical compression load in the 18th century with a basic equation included in most strength of materials textbooks.

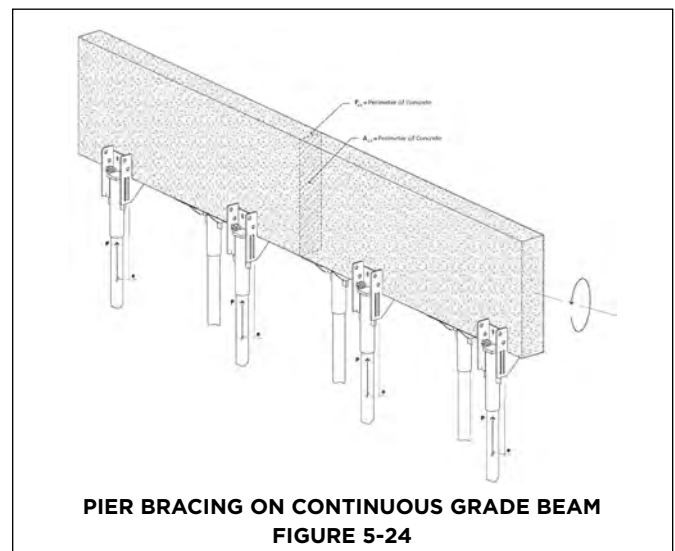
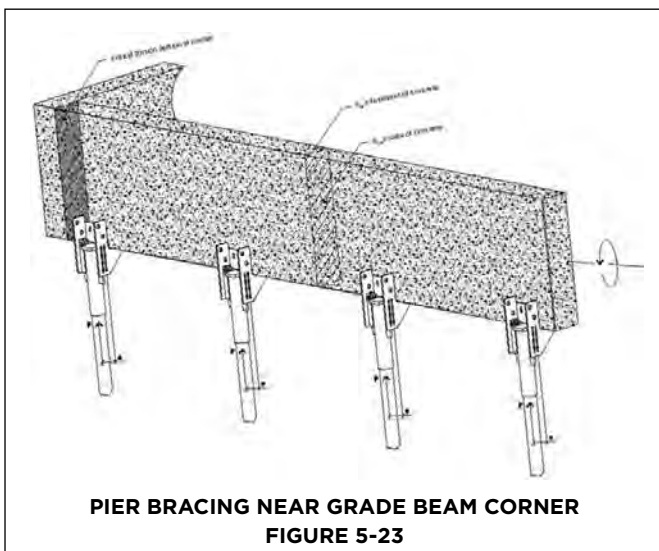
EQUATION 5-61

$$P_{crit} = \pi^2 EI / (KL_u)^2$$

where

- E = Modulus of elasticity
- I = Moment of inertia
- K = End condition parameter that depends on fixity
- L_u = Unsupported length

Most helical piles have slender shafts, which can lead to very high slenderness ratios (KL_u/r) depending on the length of the pile shaft. This condition would be a concern if the helical piles were in air or water and subjected to a compressive load. For this case, the critical buckling load could be estimated using the well-known Euler equation (Equation 5-61).



However, helical piles are not supported by air or water, but by soil. This is the reason helical piles can be loaded in compression well beyond the critical buckling loads predicted by Equation 5-61. As a practical guideline, soil with N_{60} SPT blow counts per ASTM D1586 greater than 4 along the entire embedded length of the helical pile shaft has been found to provide adequate support to resist buckling provided there are no horizontal (shear) loads or bending moments applied to the top of the pile. Only the very weak soils are of practical concern. For soils with N_{60} values of 4 blows/ft or less, buckling calculations can be done by hand using the Davisson Method (1963) or by computer solution using the finite-difference technique as implemented in the LPILE computer program (Ensoft, Austin, TX). In addition, the engineers at Hubbell Power Systems, Inc., have developed a macro-based computer solution using the finite-element technique with finite element analysis software from ANSYS, Inc. If required, application engineers can provide project-specific buckling calculations given sufficient data relating to the applied loads and the soil profile. If you need engineering assistance, please contact the Chance distributor in your area. Contact information for Chance distributors can be found at www.chancefoundationsolutions.com. These professionals will help you to collect the data required to perform a buckling analysis. The distributor will either send this data to Hubbell for a buckling analysis or directly provide this service.

5.8.4 BUCKLING/LATERAL STABILITY PER INTERNATIONAL BUILDING CODE (IBC) REQUIREMENTS

IBC 2021 Section 1810.2.1 Lateral Support states that any soil other than fluid soil shall be deemed to afford sufficient lateral support to prevent buckling of deep foundation elements in accordance with accepted engineering practice and the applicable provisions of this code. Per IBC 2021 section 1810.2.1, piers/piles can be considered fixed and laterally supported at 5 feet below the ground surface when driven into firm ground and at 10 feet below the ground surface when driven into soft material. The IBC does not specifically define fluid, soft, and firm soil. To remedy this, ICC-ES Acceptance Criteria AC358 defines these soil terms as follows:

- Firm soils are defined as any soil with a Standard Penetration Test (SPT) blow count (N_{60}) of five or greater.
- Soft soils are defined as any soil with an SPT blow count (N_{60}) greater than zero and less than five.
- Fluid soils are defined as any soil with an SPT blow count (N_{60}) of zero [weight of hammer (WOH) or weight of rods (WOR)].

Therefore, one method to check the effects of buckling and lateral stability of helical piles and resistance piers is to assume the depth to fixity is either 5 feet in firm soil or 10 feet in soft soil. The corresponding axial compression capacity of the pile shaft is determined based on either 5 feet or 10 feet of unsupported length. This is the method used to determine the nominal, LRFD design, and ASD allowable compression strengths of the helical pile product families provided in Section 7 of this manual.

5.8.5 BUCKLING ANALYSIS BY DAVISSON (1963) METHOD

A number of solutions have been developed for various combinations of pile head and tip boundary conditions and for the cases of constant modulus of subgrade reaction (k_h) with depth. One of these solutions is the Davisson (1963) method as described below. Solutions for various boundary conditions are presented by Davisson as shown in Figure 5-25. The solutions in Figure 5-25 are presented in dimensionless form as a plot of buckling load ratio (U_{cr}) versus length ratio (l_{max}). The axial load on the pile is assumed to be constant, i.e., no load transfer due to side resistance occurs, and the pile is assumed to be initially perfectly straight.

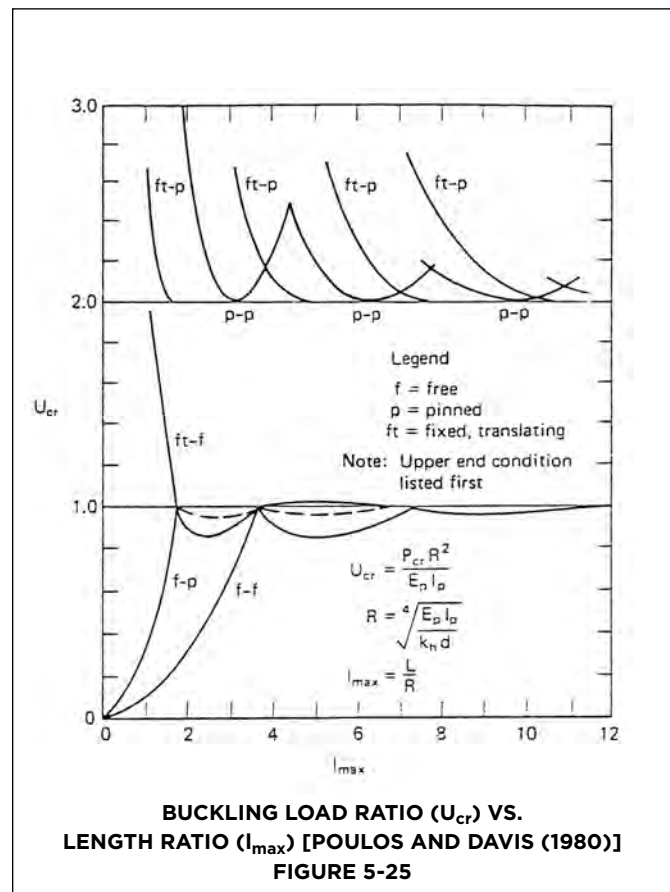
EQUATION 5-62

$$U_{cr} = P_{cr}R^2/E_pI_p \text{ or } P_{cr} = U_{cr}E_pI_p/R^2$$

where

- U_{cr} = Dimensionless buckling load ratio
- P_{cr} = Critical buckling load
- R = $4\sqrt{E_pI_p/k_hB}$
- E_p = Modulus of elasticity of pile shaft
- I_p = Moment of inertia of pile shaft
- k_h = Modulus of subgrade reaction
- B = Pile shaft diameter

EQUATION 5-63





EQUATION 5-64

$$l_{max} = L/R$$

where

- l_{max} = Dimensionless length ratio
- L = Pile shaft length over which k_h is considered to be constant

By assuming a constant modulus of subgrade reaction (k_h) for a given soil profile to determine R and l_{max} and using Figure 5-25 to determine U_{cr} , Equation 5-62 can be solved for the critical buckling load. Typical values for k_h are shown in Table 5-15.

Figure 5-25 shows that the boundary conditions at the pile head and tip exert a controlling influence on U_{cr} , with the lowest buckling loads occurring for piles with free (unrestrained) ends. Design Example 8-16 in Section 8 illustrates the use of the Davisson (1968) method to determine the critical buckling load.

5.8.6 BUCKLING ANALYSIS BY FINITE-DIFFERENCE METHOD

Another way to determine the buckling load of a helical pile in soil is to model it based on the classical Winkler (mathematician, circa 1867) concept of a beam-column on an elastic foundation. The finite-difference technique can then be used to solve the governing differential equation for successively greater loads until, at or near the buckling load, failure to converge to a solution occurs. The derivation for the differential equation for the beam-column on an elastic foundation was given by Hetenyi (1946). The assumption is made that a shaft on an elastic foundation is subjected not only to lateral loading, but also to compressive force acting at the center of gravity of the end cross-sections of the shaft, leading to the differential equation:

EQUATION 5-65

$$EI(d^4y/dx^4) + Q(d^2y/dx^2) + E_s y = 0$$

where

- EI = Flexural rigidity of the foundation shaft
- y = Lateral deflection of the shaft at a point x along the length of the shaft
- x = Distance along the axis, i.e., along the shaft
- Q = Axial compressive load on the helical pile
- $E_s y$ = Soil reaction per unit length
- E_s = Secant modulus of the soil response curve

MODULUS OF SUBGRADE REACTION—TYPICAL VALUES, TABLE 5-15

SOIL DESCRIPTION	MODULUS OF SUBGRADE REACTION (k_h) (pci)
Very soft clay	15-20
Soft clay	30-75
Loose sand	20

The first term of the equation corresponds to the equation for beams subject to transverse loading. The second term represents the effect of the axial compressive load. The third term represents the effect of the reaction from the soil. For soil properties varying with depth, it is convenient to solve this equation using numerical procedures such as the finite-element or finite-difference methods. Reese, et al. (1997) outlines the process to solve Equation 5-65 using a finite-difference approach. Several computer programs are commercially available that are applicable to piles subject to axial and lateral loads as well as bending moments. Such programs allow the introduction of soil and pile shaft properties that vary with depth and can be used advantageously for design of helical piles and micropiles subject to centered or eccentric loads.

To define the critical load for a particular structure using the finite-difference technique, it is necessary to analyze the structure under successively increasing loads. This is necessary because the solution algorithm becomes unstable at loads above the critical load. This instability may be seen as a convergence to a physically illogical configuration or a failure to converge to any solution. Since physically illogical configurations are not always easily recognized, it is best to build up a context of correct solutions at low loads with which any new solution can be compared. Design Example 8-17 in Section 8 illustrates the use of the finite-difference method to determine the critical buckling load.

5.8.7 BUCKLING ANALYSIS BY FINITE ELEMENTS

Hubbell Power Systems, Inc., has developed a design tool integrated with FEA software from ANSYS, Inc, to determine the load response and buckling of helical piles. The method uses a limited nonlinear model of the soil to simulate soil resistance response without requiring the solution time inherent in a full nonlinear model. The model is still more sophisticated than a simple elastic foundation model and allows for different soil layers and types.

The helical pile components are modeled as 3-D beam elements assumed to have elastic response. Couplings are modeled from actual test data, which includes an initial zero stiffness, elastic/rotation stiffness, and a final failed condition which includes some residual stiffness. Macros are used to create soil property data sets, helical pile component libraries, and load options with end conditions at the pile head.

After the helical pile has been configured and the soil and load conditions specified, the macros increment the load, solve for the current load, and update the lateral resistance based on the lateral deflection. After each solution, the FEA post-processor extracts the lateral deflection and recalculates the lateral stiffness of the soil for each element. The macro then restarts the analysis for the next load increment. This incremental process continues until buckling occurs. Various outputs such as deflection and bending moment plots can be generated from the results. Design Example 8-18 in Section 8 illustrates the use of the finite-element method to determine the critical buckling load.

DESIGN METHODOLOGY

5.8.8 PRACTICAL CONSIDERATIONS—BUCKLING

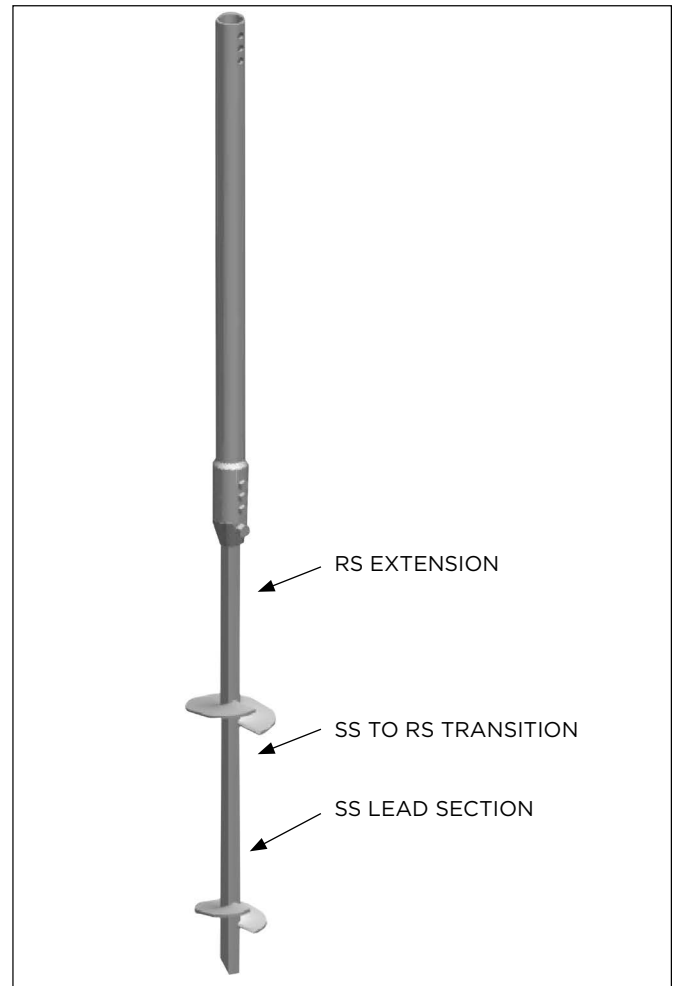
As stated previously, where soft and/or loose soils (SPT N_{60} blow count ≤ 4) overlie the bearing stratum, the possibility of shaft buckling must be considered. Buckling also becomes a potential limiting factor where lateral loads (bending and shear) are present in combination with compressive loads. Factors that determine the buckling load include the helical pile shaft diameter, length, flexural stiffness, and strength; the soil stiffness and strength; any lateral shear and/or moment applied at the pile head; and pile head fixity conditions (fixed, pinned, free, etc.). In addition, all extendable helical piles have couplings or joints used to connect succeeding sections together in order to install the helix plates into bearing soil. Bolted couplings or joints have a certain amount of rotational tolerance. This means the joint initially has no stiffness until it has rotated enough to act as a rigid element. This is analogous to saying the coupling or joint acts as a pin connection until it has rotated a specific amount, after which it acts as a rigid element with some flexural stiffness.

Concerns about slender shafts and joint stiffness, along with the fact that helical piles are routinely installed in soils with poor strength, are some of the reasons why helical piles are often installed with grouted shafts (Helical Pulldown[®] micropiles) and are available with larger diameter round pipe shafts. Round shaft (RS) helical piles have better buckling resistance than square shaft (SS) piles because they have greater section moduli (flexural resistance), plus they have greater resistance to lateral deflection in soil due to their larger lateral dimensions. See the specifications section of the helical pile product family pages in Section 7 for the section properties and dimensions of SS and RS helical piles/anchors.

SS helical piles/anchors provide the most efficient capacity-to-torque relationship (see Section 6, Installation Methodology). RS helical piles/anchors provide lateral capacity and better buckling resistance. A good compromise to address buckling in soft/loose soils is to use helical combination piles, or combo piles for short. A combo pile consists of a square shaft lead section and round shaft extension sections (see Figure 5-26). The combo pile provides the advantages of SS and RS piles, which enables the combo pile to penetrate dense/hard soils and provide a larger shaft section in the soft/loose soils above the bearing strata. See Section 7 for more information on combo piles.

The Chance[®] Helical Pulldown[®] micropile is a method for constructing a grout column around the shaft of either a square shaft or round shaft helical pile installed in soft/loose soil. The installation process displaces soil around the central steel shaft and replaces it with a gravity-fed neat cement grout mixture. Upon curing, the grout forms a column that increases the section modulus of the pile shaft to the point that buckling is not the limiting condition. In addition to buckling resistance, the grout column increases axial load capacity due to skin friction and/or adhesion along the column and stiffens the load-deflection response of the pile. See Section 7 for more information on Helical Pulldown micropiles.

Chance Helical Pulldown micropiles cannot be installed in every soil condition. To date, grouted-shaft helical piles have been successfully installed in overburden soil with SPT N_{60} blow counts greater than 10 blows/ft. In those cases, the grouted shaft is being used to develop greater load capacity and a stiffer response, not necessarily to prevent buckling. Contractors have successfully installed Helical Pulldown micropiles in glacial tills (SPT $N_{60} > 50$) using special soil-displacement methods. Increasingly dense soil makes installation more difficult for the displacement element, which must force soil laterally outward from the central steel shaft.



**SS TO RS COMBINATION PILE
FIGURE 5-26**



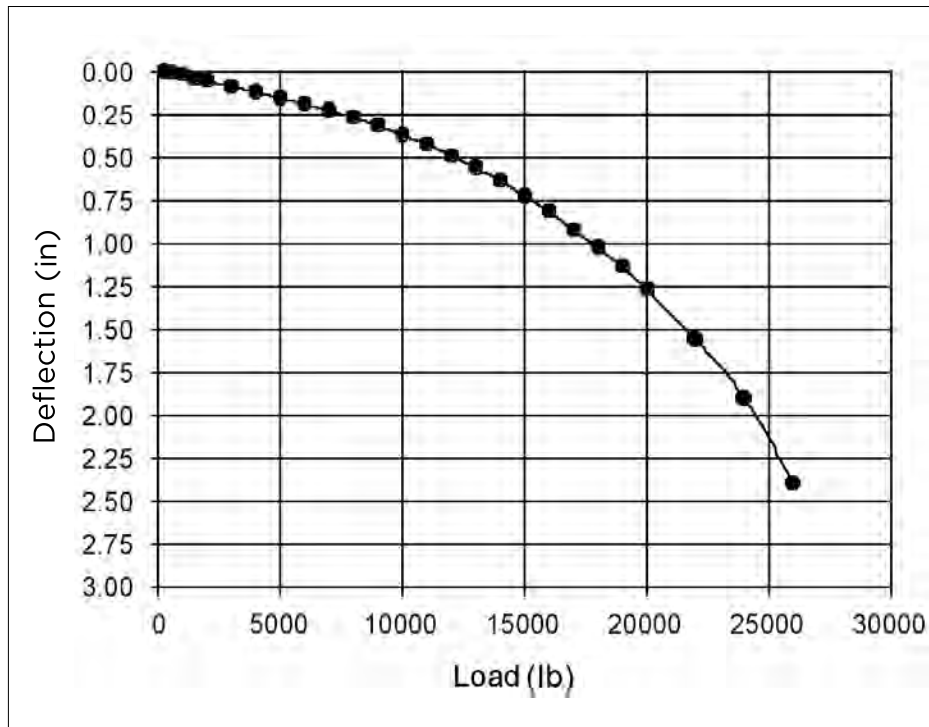
5.9 HELICAL PILE DEFLECTION AT WORKING LOAD

Most of the discussion thus far has focused on evaluating the ultimate load capacity of helical piles/anchors in axial compression or tension. This is considered as the load limit state and gives the upper bound on the load capacity. The deflections of the pile/anchor at this load state will be very large (> 2 inches [>51 mm]), and the pile/anchor deflection will continue to increase with no additional increase in load capacity. It is also of great interest to most engineers to consider the behavior of a helical pile/anchor at a lower working load or serviceability state which is well below the load limit state.

We can consider a typical load-deflection curve as shown in Figure 5-27. This plot shows the test results of a 1.5-inch square shaft helical anchor with a single 12-inch helix installed to a depth of 10 feet in a medium-dense silty sand. The test was performed in tension. According to the IBC, the ultimate capacity is the load producing a net deflection of 10% of the helix diameter. In this case, the ultimate capacity occurs at 1.20 inches of deflection, which corresponds to 19,500 lb. It is obvious that in this case, as in most cases, the anchor can actually hold load up to a deflection of as much as 20% of the helix diameter.

Using an ASD Factor of Safety of 2.0, the working load for this anchor is $(19,500 \text{ lb})/2.0 = 9,750 \text{ lb}$. Because the load-deflection curves of helical piles/anchors are generally nonlinear, it would be expected that the deflection at the working load would be less than one half of the deflection at the ultimate load (1.20 inches). In this case, the deflection at the working load of 9,750 lb is on the order of 0.36 inches. Using a lower Factor of Safety results in a higher deflection at the working load. For example, if a Factor of Safety of 1.5 is used, the working load becomes $(19,500 \text{ lb})/1.5 = 13,000 \text{ lb}$, and the deflection corresponding to this load is on the order of 0.55 inches.

Based on a review of a number of tests performed on single-helix piles/anchors in Colorado, Cherry and Perko (2012) suggested that for many piles/anchors, the deflection at the working loads ($FS = 2.0$) averaged about 0.25 inches. Additional work is needed to determine how this may vary for multi-helix piles/anchors and if other soils produce different behavior.



DEFLECTION VS. LOAD PLOT
FIGURE 5-27



REFERENCES:

1. Specification ASTM D1586, *Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils*, American Society for Testing and Materials.
2. Abdelghany, Y, and El Naggar (2010), Full-scale Experimental and Numerical Analysis of Instrumented Helical Screw Piles Under Axial and Lateral Monotonic and Cyclic Loadings - A Promising Solution for Seismic Retrofitting. Proceedings of the 6th International Engineering and Construction Conference, Cairo, Egypt.
3. Bjerrum, L., Norwegian Experiences with Steel Piles to Rock, *Geotechnique*, Vol 7, 1957.
4. Bowles, J.E., *Foundation Analysis and Design*, First Edition, McGraw-Hill, 1968.
5. Bowles, J.E., *Foundation Analysis and Design*, Fourth Edition, McGraw-Hill, 1988.
6. Brinch Hansen, J., The Ultimate Resistance of Rigid Piles Against Transversal Forces, *Geotechnik Institute Bulletin No. 12*, Copenhagen, 1961.
7. Broms, Bengt B., Lateral Resistance of Piles in Cohesive Soils, Proceedings of the American Society of Civil Engineers, Journal of the Soil Mechanics and Foundations Division, Vol. 90, SM2, 1964.
8. Broms, Bengt B., Lateral Resistance of Piles in Cohesionless Soils, Proceedings of the American Society of Civil Engineers, Journal of the Soil Mechanics and Foundations Division, vol. 90 SM3, 1964.
9. Cadden, Allen and Jesus Gomez, *Buckling of Micropiles*, ADSC-IAF Micropile Committee, Dallas, TX, 2002.
10. Clemence, Samuel P. and others, *Uplift Behavior of Anchor Foundations in Soil*, American Society of Civil Engineers, 1985.
11. Das, Braja M., *Theoretical Foundation Engineering*, Elsevier Science Publishing Company Inc., New York, NY, 1987.
12. Davis, E.H., *The Application of the Theory of Plasticity to Foundation Problems-Limit Analysis*, Post Graduate Course, University of Sydney, Australia, 1961.
13. Davisson, M.T., Estimating Buckling Loads for Piles, Proceedings of the Second Pan-American Conference on Soil Mechanics and Foundation Engineering, Brazil, Vol 1, 1963.
14. Davisson, M.T., Laterally Loaded Capacity of Piles, *Highway Research Record*, No. 333: 104-112, 1970.
15. Design Manual DM7, NAVFAC, Foundations and Earth Structures, Government Printing Office, 1986.
16. Design Manual DM7, NAVFAC, Soil Mechanics, Government Printing Office, 1986.
17. Gouvenot, D., *Essais en France et a l'Etranger sur le Frottement Lateral en Fondation: Amelioration par Injection*, Travaux, 464, Nov, Paris, France, 1973.
18. HeliCALC Micropile Design Assessment Program, Theoretical and User's Manual, Hubbell Power Systems/A.B. Chance Co., 2001.
19. Hetenyi, M., *Beams on Elastic Foundations*, The University of Michigan Press, Ann Arbor, MI, 1946.
20. Hoyt, Robert M., Gary L. Seider, Lymon C. Reese and Shin-Tower Wang, Buckling of Helical Anchors Used for Underpinning, Proceedings, ASCE National Convention, San Diego, CA, 1995.
21. Meyerhof, George Geoffrey, Bearing Capacity and Settlement of Pile Foundations, Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Volume 102, No GT3, 1976.
22. Poulos, H.G., Analysis of Piles in Soils Undergoing Lateral Movements, *JSMFD, ASCE*, Vol. 99, SM5, 1973.
23. Reese, L.C., The Analysis of Piles Under Lateral Loading, Proceedings, Symposium on the Interaction of Structure and Foundation, Midland Soil Mechanics and Foundation Engineering Society, University of Birmingham, England, 1971.
24. Reese, L.C. and S.J. Wright, *Drilled Shaft Design and Construction Guidelines Manual*, US Department of Transportation, Federal Highway Administration, 1977.
25. Reese, L.C., W.M. Wang, J.A. Arrellaga, and J. Hendrix, *Computer Program LPILEPLUS Technical Manual*, Version 3.0, Ensoft, Inc., AUSTIN, TX, 1997.
26. Sharnouby and El Naggar (2011), Monotonic and Cyclic Lateral Full-scale Testing of Reinforced Helical Pulldown Micropiles, Proceedings of the DFI Annual Conference 2011, Boston, MA.
27. Terzaghi, K. and R.B. Peck, *Soil Mechanics in Engineering Practice*, John Wiley and Sons, Inc., 1967.
28. Deep Foundations Institute (DFI), *Helical Pile Foundation Design Guide*. 2019.



SECTION 6: INSTALLATION METHODOLOGY

CONTENTS

Atlas Resistance® Piers.....	6-2
Chance® Helical Piles/Anchors.....	6-4
Installation Torque/Capacity Relationship.....	6-4
Torque Indicator Calibration.....	6-12
Installation Termination Criteria.....	6-12

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

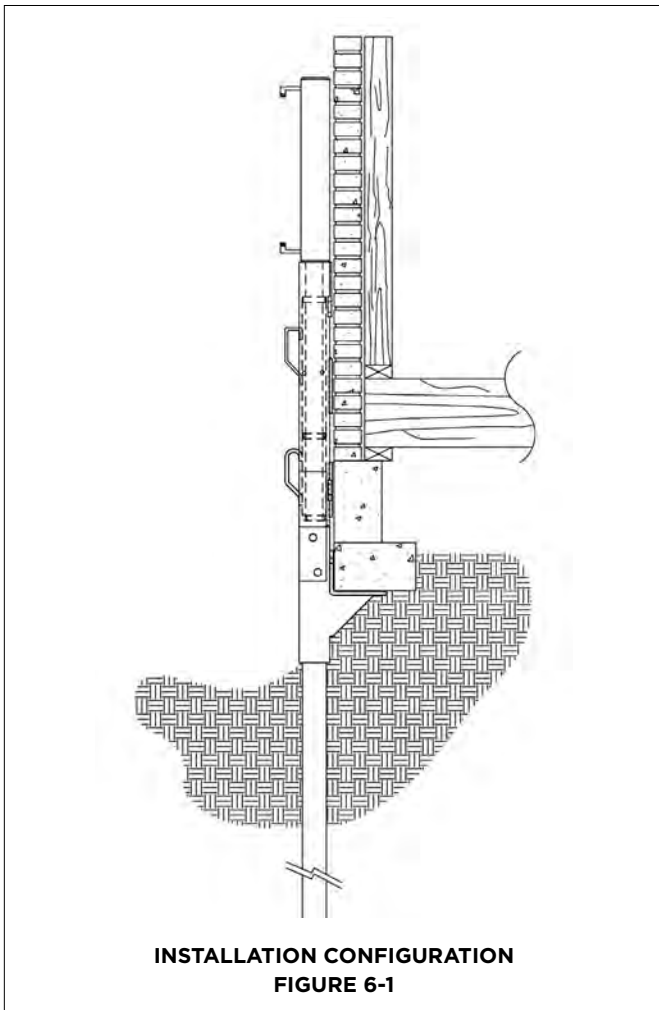
Hubbell Power Systems, Inc. shall not be responsible for or liable to you and/or your customers for the adoption, revision, implementation, use, or misuse of this information. Hubbell, Inc. takes great pride and has every confidence in its network of installing contractors and dealers.

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



ATLAS RESISTANCE® PIERS

Atlas Resistance® Piers develop their capacity as a result of a pile tip or end bearing reaction in soil or rock that is achieved by hydraulically driving hollow pier sections to suitable strata utilizing the reaction weight of an existing structure. The friction reduction collar on the initial or starter section allows for an end bearing pile. Most Atlas Resistance® Piers are installed to a force equal to a minimum of 150% of the calculated total load at each pier placement. The total load condition is a sum of the structure Dead Loads (DL) and all known potential Live Loads (LL). In addition to the usual calculated loads, it is extremely important to include loads imposed from soil overburden over a projected area, primarily outside of the foundation wall footprint (toe or heel) of the footing. The area of the projection plus the height of soils above it produce a loading condition that is quite often in excess of the structure load itself. When lifting the structure is required, an additional “soil wedge” area and/or volume should be considered relative to the soil type and its characteristics. To be conservative in design calculations it is prudent to consider the long-term loading effect from soils outside of the vertical and horizontal plane of the soil overburden even when stabilization only is required.



LOAD VERIFICATION

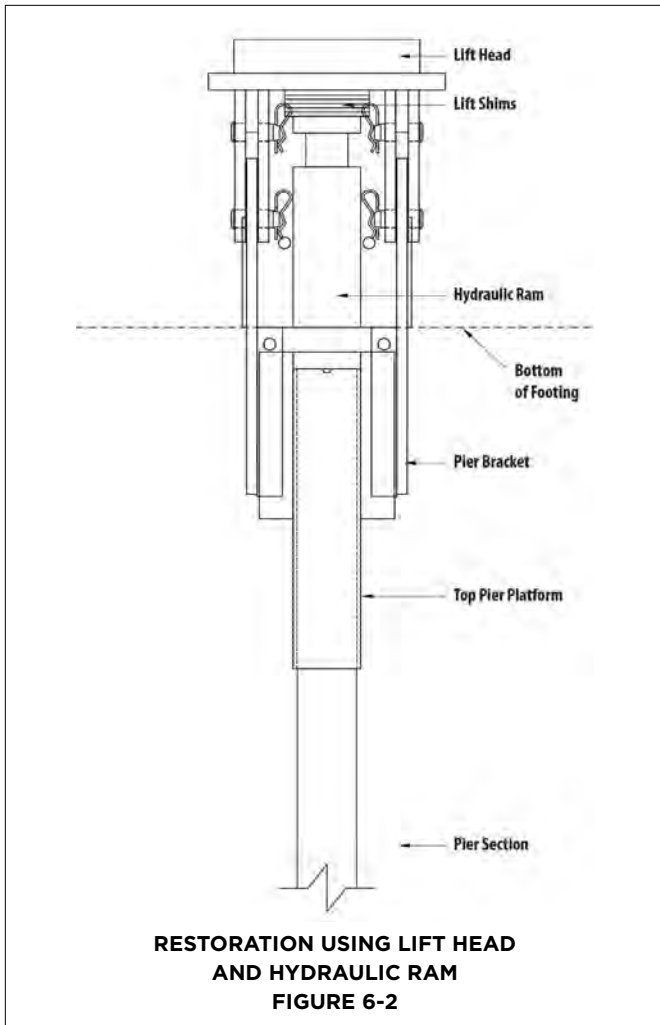
Atlas Resistance® Piers are installed using hydraulic cylinders with known effective areas. Although larger cylinders are available for extreme load conditions, the standard installation cylinders have an effective area of 8.29 in². The effective area of the cylinder is multiplied by the hydraulic pressure monitored by a gauge mounted between the hydraulic pump and the cylinder. The net result of this number is the actual force (in lbs) achieved as the pier sections are driven against the reaction weight of the structure until the required load is achieved or structure lift occurs. Additional pier sections are added as necessary until a competent bearing stratum is reached. The force is logged at the end of each pier section driven on the field installation log.

TWO STAGE SYSTEM METHODOLOGY

Atlas Resistance® Piers incorporate a two-stage installation procedure that consists of first driving each pier individually, using the structure as reaction to install the pier, followed by the second stage of installation that transfers the structure loads to all the installed piers simultaneously during lifting or stabilizing. This two-stage process is required to obtain an adequate Factor of Safety (FS). During the first stage of installing the piers, the integrity of the foundation and the dead weight of the structure determines the extent to which additional Factors of Safety can be achieved between the installation force and final lift loads. Figure 6-1 provides a schematic drawing that illustrates the installation of pier sections. The second stage occurs when all or most of the piers are loaded simultaneously using a manifold or series of manifolds and hydraulic cylinders placed at each pier. The manifolds and cylinders are connected to a pump or series of pumps depending on the number of piers being lifted. During the lifting stage the hydraulic pressure is monitored on each manifold system gauge. Typical 25 ton lifting cylinders have an effective area of 5.15 in². The load at each pier is monitored at the final lock off and noted on the field installation logs. The actual lift or lock off load at each pier can then be compared to the installation loads at each pier to determine the actual Factor of Safety developed between installation loads and actual loads required to produce structural lift and support. Figure 6-2 provides a schematic drawing illustrating the lift stage.

BEARING CAPACITY

The compressive bearing capacity of Atlas Resistance® Piers is developed predominantly by end-bearing due to the friction reduction collar at the lead end of the initial or starter section. Friction calculations do not normally enter into design steps unless required to comply with some older municipal codes. Increased tip areas (larger diameter pipe) will typically increase load resistance during installation of the pile. Standard pier section diameters are 2-7/8", 3-1/2", and 4-1/2". The selection of pier size is determined through consideration of pile load requirement, column stability (buckling concerns) structure integrity and the ability to drive the pile past seasonal zones of influence relative to available reaction forces. Bracket assemblies are coupled with the appropriate pier section size to service both the geotechnical and structural requirements.



CLAY SOILS

In clay soil conditions defined as very stiff to hard, i.e., Standard Penetration Test (SPT) “N” values in excess of 35-40 blows/foot, it has been shown empirically that an Atlas Resistance® Pier can generate substantial end-bearing capacity, often in excess of 50,000-60,000 lbs of bearing resistance. While the high capacities defy absolute calculation for both very dense sand and hard clay, empirical data developed over the last several decades gives evidence to this phenomenon. Data developed by A.S. Vesic (1972) for the Transportation Research Board suggests that hard/dense soil develops very high capacities due to the formation of a larger pile bulb at the base of an end-bearing foundation. This phenomenon results in higher values for the bearing capacity factor (N_q), especially for driven piles. Figure 6-3 is an excerpt from Patent 1.217.128 issued to L. White. It is a graphical rendition of the assumed large stress bulb formed under a pile tip.

SAND SOILS

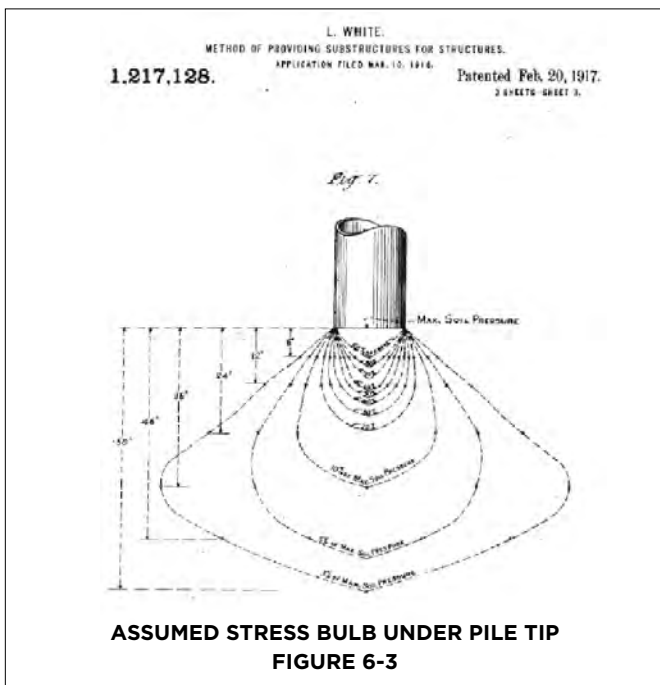
Atlas Resistance® Piers also develop substantial end-bearing capacities in granular soils, but the sand or gravel must typically exhibit a high relative density with “N” values in excess of 30-35 blows/ft. The same pile bulb described above for clay soils will form at the base of an Atlas Resistance® Pier in sand soils. In granular soils, the overburden pressure (effective vertical confining stress) has a large influence on bearing capacity, so the deeper the pier tip is embedded, the higher the bearing capacity will be for a given sand deposit of uniform density. A design condition consisting of a shallow ground water table (GWT) will require Atlas Resistance® Piers to be installed to a sufficient depth to counteract the reduction in confining stress caused by the buoyancy effect of the water.

BEDROCK

The presence of an intact bedrock surface represents an ideal ground condition for a totally end-bearing load transfer for any type of foundation. In this case the Atlas Resistance® Pier is installed to the rigid bearing surface represented by the bedrock layer, with load confirmation being verified by monitoring of the hydraulic pressure and effective area of the installation cylinder. The design capacity in this case is directly related to the structural strength of the pier shaft and bracket assembly.

INSTALLATION OVERVIEW

When the loading, structural and geotechnical conditions have been determined, the proper pier brackets and pier sections can be selected. Following excavation for the installation, the footing (if present) is notched to a point flush with the wall to be underpinned. Should steel reinforcement be encountered, notify the Engineer of Record prior to cutting. This procedure is performed to minimize the eccentricity of the pier assembly. In situations where notching the footing is prohibited, consideration needs to be given to the published pier capacity ratings if the footing extension from the wall is excessive, possibly increasing the eccentric load on the pier assembly resulting in a lower capacity. The bottom of the footing should be prepped





and/or a load bearing grout added between the pier bracket and footing to provide a uniform bearing connection. This is a critical point, especially in high load conditions. Failure to comply with this step could result in a point load on the bracket and cause an early bracket failure.

When the bracket and installation equipment are properly positioned and anchored to the foundation or wall, the starter section can be placed in a vertical and plumb position. Activate the hydraulic pump to advance and retract the installation cylinder as necessary to drive the pier sections (see photo at top right). The pressure is recorded at the end of each 42" pier section. Continue driving pier sections until reaching strata capable of resisting the estimated Proof Load (PL) or until structure lift occurs. When approaching the end of the drive, a good rule of thumb is to drive pipe until either the structure begins to lift and/or the pressure continues to build. If a small amount of movement has occurred but the pressure remains constant, an experienced installer will continue to drive pipe until either a more significant movement is noted or a consistent build in pressure occurs. Depending on the integrity of the foundation and the comfort level of the installer, this will often result in substantial Factors of Safety in excess of 1.5. When driving the pier pipe is completed, the final pier section is removed and cut or cut in place to the required elevation. When the cut pier section has been reinstalled on the pier, the installation equipment is removed and moved to the next pier location to be installed. When all the piers have been installed, lift heads and lift cylinders are placed on all the piers. The cylinders, manifolds and hydraulic pump are connected.

Load is applied uniformly, as much as possible (see photo at bottom right). Upon transfer of load to the entire pier assembly, lift pressures are noted at each pier and recorded on the field log. When all the lift cylinders have been locked off, shims and pins can be installed on the 2-Piece brackets or the nuts on the Continuous Lift brackets can be tightened to lock the load on the brackets. The Lift Heads and Lift Cylinders can be removed. The actual verified Factor of Safety between installation pre-load and final lock off load can then be confirmed. Table 6-1 is an example of the driving (installation) and lift forces that could be involved in the installation of Atlas Resistance® Piers.

Refer to the Atlas Resistance® Standard, Heavy Duty and Modified 2-Piece Pier Systems Model Specification found under the Resources tab on www.chancefoundationsolutions.com for detailed installation instructions.

CHANCE® HELICAL PILE/ANCHORS

A helical pile/anchor is a low soil displacement foundation element specifically designed to minimize disturbance during installation. In their simplest forms, helical piles/anchors consist of at least one helix plate and a central steel shaft (see Figure 6-4). The helix geometry is very important in that it provides the downward force or thrust that pulls a helical pile/anchor into the ground. The helix plate(s) must be a true ramped spiral with a uniform pitch to maximize efficiency during installation. If the helix is not formed properly, it will disturb the soil more than if a true helix advances at a rate of one pitch per revolution. The central steel shaft transmits the rotational energy or



torque from the machine to the helix plate(s). Most helical piles in North America use a low displacement (less than 4.5 inch (114 mm) diameter shaft in order to reduce friction and soil displacement during installation. A helical pile/anchor functions similar to a wood screw except that it has a discontinuous thread-form and is made to a much larger scale.

INSTALLATION TORQUE/CAPACITY RELATIONSHIP

When installed into soil, a helical pile/anchors functions as an axially loaded end-bearing deep foundation. The helix plates serve a two-fold purpose. The first purpose is to provide the means to install the helical pile/anchor. The second purpose is to provide the bearing element for load transfer to soil. As such, helical pile/anchor design is keyed to these two purposes, both of which can be used to predict the ultimate capacity.

Section 5 detailed how helix plates act as bearing elements. The capacity is determined by multiplying the unit bearing ca-

INSTALLATION LOAD VS LIFT LOAD, TABLE 6-1

First Stage			Second Stage					
DRIVE/PUSH			LIFT					
Installation Load Summary	STD Drive Cylinder Effective Area (sq. in.)		Pier Lift/Lock Summary	STD Lift Ram Effective Area (sq. in.)	Safety Factor Drive vs Lift			
8.29		←	→	5.15				
Pier #	PSI	CAPACITY	Pier #	PSI	LOAD	SF		
1	4,200	34.8	1	4,000	20.6	1.7		
2	4,600	38.1	2	4,000	20.6	1.9		
3	4,600	38.1	3	4,500	23.2	1.6		
4	4,800	39.8	←	→	4	4,500	23.2	1.7
5	5,000	41.5	5	4,800	24.7	1.7		

capacity of the soil at each helix location by the projected area of each helix. This capacity is generally defined as the ultimate theoretical capacity because it is based on soil parameters either directly measured or empirically derived from soil exploration sounding data.

The purpose of this section is to provide a basic understanding of how installation torque (or installation energy) provides a simple, reliable means to predict the capacity of a helical pile/anchor. More importantly, this prediction method is independent of the bearing capacity method detailed in Section 5, so it can be used as a “field production control” method to verify capacity during installation. The installation torque-to-capacity relationship is an empirical method originally developed by the A.B. Chance Company in the late 1950’s and early 1960’s. Hubbell Power Systems, Inc. has long promoted the concept that the torsional energy required to install a helical pile/anchor can be related to the ultimate capacity of a pile/anchor. Precise definition of the relationship for all possible variables remains to be achieved. However, simple empirical relationships, originally derived for tension loads but also valid for compression loads, have been used for a number of years. The principle is that as a helical pile/anchor is installed (screwed) into increasingly denser/harder soil, the resistance to installation (called installation energy or torque) will increase. Likewise, the higher the installation torque, the higher the axial capacity of the installed pile/anchor. Per the Deep Foundations Institute (DFI) Helical Pile Foundation Design Guide (2019), capacity-to-torque correlation factors, K_t , have been statistically established based on a large database of installations, and the method has been used successfully in helical pile applications. Hoyt and Clemence (1989) presented a landmark paper on this topic at the 12th International Conference on Soil Mechanics and Foundation Engineering. They proposed the following formula that relates the ultimate capacity of a helical pile/anchor to its installation torque:

EQUATION 6-1

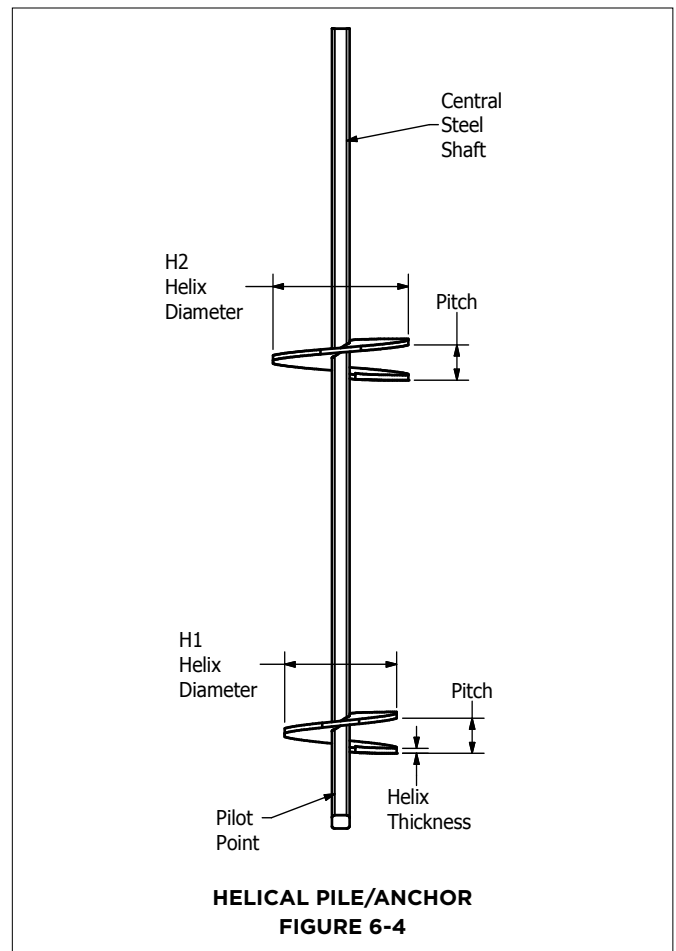
$$Q_{ult} = K_t \times T$$

where

- Q_{ult} = Ultimate uplift capacity [lb (kN)]
- K_t = Empirical torque factor [ft⁻¹ (m⁻¹)]
- T = Average installation torque [lb-ft (kN-m)]

Hoyt and Clemence recommended $K_t = 10$ ft⁻¹ (33 m⁻¹) for square shaft (SS) and round shaft (RS) helical anchors less than 3.5” (89 mm) in diameter, 7 ft⁻¹ (23 m⁻¹) for 3.5” diameter round shafts, and 3 ft⁻¹ (9.8 m⁻¹) for 8-5/8” (219 mm) diameter round shafts. The value of K_t is not a constant - it may range from 3 to 20 ft⁻¹ (10 to 66 m⁻¹), depending on soil conditions, shaft size and shape, helix thickness, and application (tension or compression). For Chance® Type SS Square Shaft Helical Piles/Anchors, K_t typically ranges from 10 to 13 ft⁻¹ (33 to 43 m⁻¹), with 10 ft⁻¹ (33 m⁻¹) being the recommended default value. For Chance® Type RS Pipe Shaft Helical Piles/Anchors, K_t typically ranges from 3 to 10 ft⁻¹ (10 to 33 m⁻¹), with 9 ft⁻¹ (30 m⁻¹) being the recommended default for Type RS2875; 7 ft⁻¹ (23 m⁻¹) being the recommended default for Type RS3500.300; and 6 ft⁻¹ (20 m⁻¹) being the recommended default for Type RS4500.337.

The Canadian Foundation Engineering Manual (2006) recom-





mends values of $K_t = 7 \text{ ft}^{-1}$ for pipe shaft helical piles with 90 mm OD, and $K_t = 3 \text{ ft}^{-1}$ for pipe shaft helical piles approaching 200 mm OD.

The correlation between installation torque (T), and the ultimate capacity (Q_{ult}) of a helical pile/anchor, is a simple concept but a complicated reality. This is partly because there are a large number of factors that can influence the determination of the empirical torque factor K_t . A number of these factors (not including soil), are summarized in Table 6.2.

It is important to understand that torque correlation is valid when the helical pile/anchor is advancing at a rate of penetration nearly equal to one helix pitch per revolution. Large displacement shafts [$>8\text{-}5/8$ " (219mm)] are less likely to advance at this rate, which means torque correlation cannot be used as a means to determine capacity.

FACTORS INFLUENCING K_t , TABLE 6-2

Factors Affecting Installation Torque (T)	Factors Affecting Ultimate Capacity (Q_{ult})
Method of Measuring Installation Torque (T)	Number and Size of Helix Plates
Installed Depth Used to Determine "Average" Torque	Direction of Loading (Tension or Compression)
Applied Down-Force or "Crowd"	Geometry of Couplings
Rate of Rotation	Spacing of Helix Plates
Alignment of Pile/Anchor	Shape and Size of Shaft
Rate of Advance	Time between Installation and Loading
Geometry of Couplings	
Shape and Size of Shaft	
Shape and Size of Shaft	
Number & Size of Helix Plates	
Pitch of Helix Plates	

The factors listed in Table 6-2 are some of the reasons why Hubbell Power Systems, Inc. has a dealer certification program. Contractors who install helical piles/anchors are trained in the proper methods and techniques before they are certified. In order for Equation 6-1 to be useful, installation torque must be measured. There are a variety of methods used to measure torque. Hubbell Power Systems, Inc. offers two in-line torque indicators; in-line indicators are the best method to determine torque for capacity prediction. Other useful methods to measure torque are presented later in this section. For torque correlation to be valid, the rate of penetration should be between 2.5" to 3" per revolution. The rotation speed should be consistent and in the range of 5 to 20 RPM. And, the minimum effective torsional resistance criterion (the average installation torque) should be taken over the last 3 feet of penetration at 1-foot intervals, unless a single helix pile is used for compression

load, where it is appropriate to use the final (last) installation torque.

ICC-ES Acceptance Criteria AC358 for Helical Pile Systems and Devices Section 3.13.2 provides prequalified torque correlation (K_t) values for conforming helical pile systems based on shaft size and shape. They are the same as recommended by Hubbell Power Systems, Inc. and by Hoyt and Clemence. Hubbell Power Systems, Inc. helical piles are conforming per AC358. The AC358 K_t values are the same for both tension and compression axial loads.

The International Building Code (IBC) 2021 Section 1810.3.3.1.9 states there are three ways to determine the capacity of helical piles—including well documented correlations with installation torque.

SOIL FACTORS INFLUENCING K_t

Locating helix bearing plates in very soft, loose, or sensitive soils will typically result in K_t values less than the recommended default. This is because some soils, such as salt leached marine clays and lacustrine clays, are very sensitive and lose considerable shear strength when disturbed. It is better to extend the helical pile/anchor beyond sensitive soils into competent bearing strata. If it's not practical to extend the helical pile/anchor beyond sensitive soils, testing is required to determine the appropriate K_t .

Full-scale load testing has shown that helical anchors/piles typically have at least the same capacity in compression as in tension. In practice, compression capacity is generally higher than tension capacity because the pile/anchor bears on soil below rather than above the helix plates, plus at least one helix plate is bearing on undisturbed soil. Soil above the bearing plates is disturbed by the slicing action of the helix, but not overly disturbed by being "augured" and removed. Typically, the same values of K_t are used for both tension and compression applications. This generally results in conservative results for compression applications. A poorly formed helix shape will disturb soil enough to adversely affect the torque-to-capacity relationship, i.e., K_t is reduced. To prevent this, Hubbell Power Systems, Inc. uses matching metal dies to form helix plates which are as near to a true helical shape as is practically possible. To understand all the factors that K_t is a function of, one must first understand how helical piles/anchors interact with the soil during installation.

TORQUE RESISTANCE FACTORS

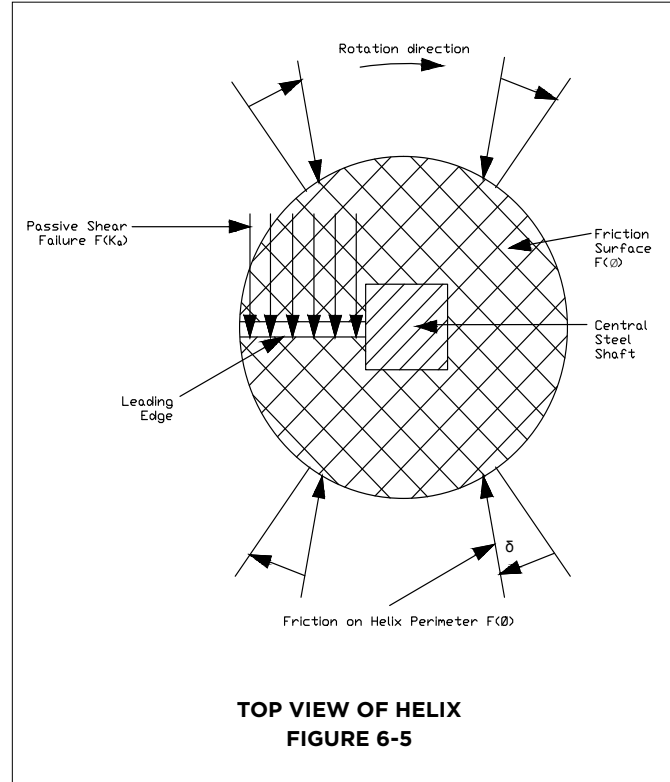
There are two main factors that contribute to the torque resistance generated during a pile/anchor installation, friction and penetration resistance. Of the two factors, friction is the larger component of torque resistance.

FRICION HAS TWO BASIC PARTS:

(1) Friction on the helix plate and friction along the central steel shaft. Friction resistance increases with helix size because the surface area of the helix in contact with the soil increases with the square of the diameter (see Figure 6-5). Likewise, friction

resistance increases with pitch size, i.e., the larger the pitch, the greater the resistance. This is analogous to the difference between a coarse thread and a fine thread bolt. Basic physics tells us that “work” is defined as force times distance. A larger pitch causes the helix to travel a greater distance per revolution, thus more work is required.

(2) Friction along the central steel shaft is similar to friction on the helix plate. Friction resistance increases with shaft size because the surface area of the shaft in contact with the soil increases as the diameter increases. An important performance factor for helical pile/anchors is the helix to shaft diameter ratio (H_d/S_d). The higher the H_d/S_d ratio, the more efficient a given helical pile/anchor will be during installation. Friction resistance also varies with shaft shape (see Figure 6-6). A round shaft may be the most efficient section to transmit torque energy, but it has the disadvantage of full surface contact with the soil during installation. When the central steel shaft is large ($> 3"$ [76 mm] in diameter) the shaft frictional resistance contributes significantly to the total frictional resistance. However, a square shaft ($< 3"$ [76 mm] in diameter) has only the corners in full surface contact with the soil during installation, thus less shaft frictional resistance. Frictional energy (energy loss) required to install a helical pile/anchor is related to the helix and shaft size. The total energy loss due to friction is equal to the sum of the friction loss of all the individual helix plates plus the length of shaft subjected to friction via contact with the soil.



PENETRATION RESISTANCE HAS TWO BASIC PARTS:

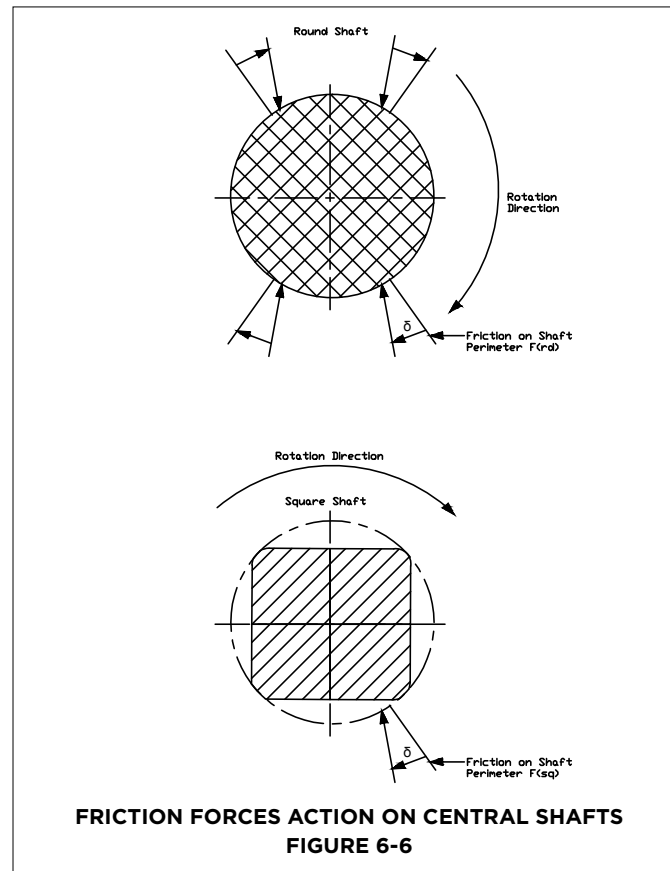
(1) Shearing resistance along the leading edge of the helix plate to allow passage of the helix plate and penetration resistance of the shaft/pilot point. Shearing resistance increases with helix size because leading edge length increases as the diameter increases. Shearing resistance also increases with helix thickness because more soil has to be displaced with a thick helix than with a thin helix (see Figure 6-7). The average distance the soil is displaced is equal to approximately 1/2 the helix thickness, so as the thickness increases the more work (i.e., energy) is required to pass the helix through the soil.

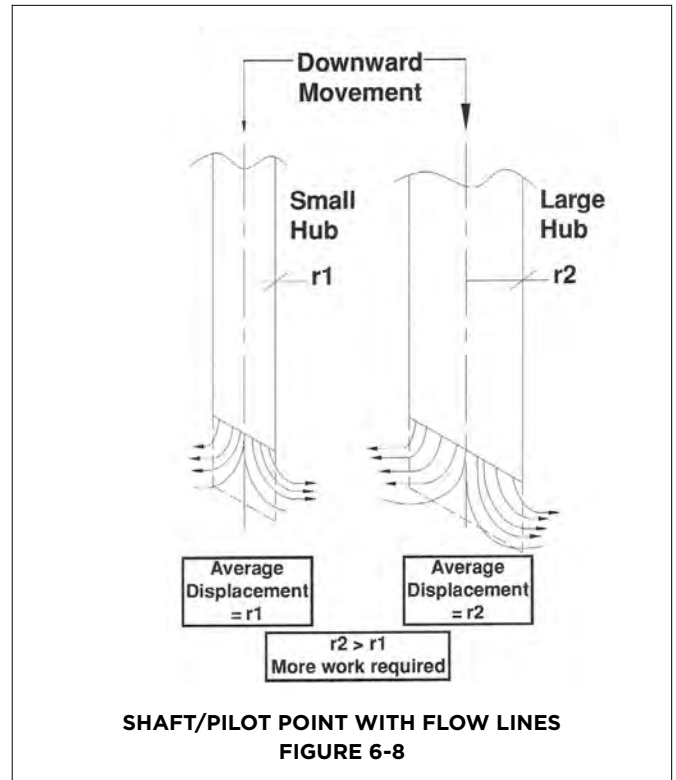
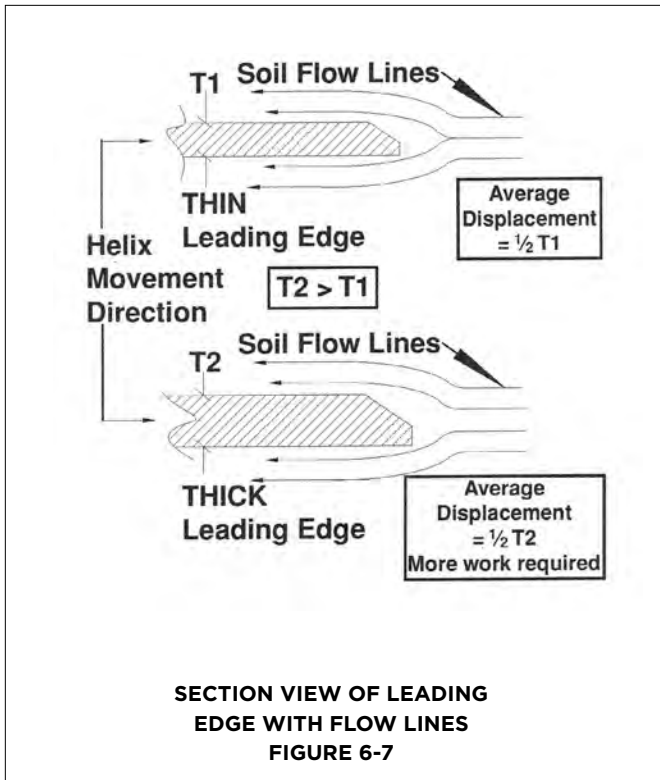
(2) Penetration resistance increases with shaft size because the projected area of the hub/pilot point increases with the square of the shaft radius (see Figure 6-8). The average distance the soil is displaced is approximately equal to the radius of the shaft, so as the shaft size increases, the more work (i.e., energy) is required to pass the hub/pilot point through the soil.

The penetration energy required to install a helical pile/anchor is proportional to the volume of soil displaced times the distance traveled. The volume of soil displaced by the pile/anchor is equal to the sum of the volumes of all the individual helix plates plus the volume of the soil displaced by the hub/pilot point in moving downward with every revolution.

ENERGY RELATIONSHIPS

Installation energy must equal the energy required to penetrate the soil (penetration resistance) plus the energy loss due to friction (frictional resistance). The installation energy is provided by the machine and consists of two components,





rotation energy supplied by the torque motor and downward force (or crowd) provided by the machine. The rotational energy provided by the motor along with the inclined plane of a true helical form generates the thrust necessary to overcome the penetration and friction resistance. The rotational energy is what is termed “installation torque.” The downward force also overcomes penetration resistance, but its contribution is usually required only at the start of the installation, or when the lead helix is transitioning from a soft soil to a hard soil.

From an installation energy standpoint, the perfect helical pile/anchor would consist of an infinitely thin helix plate attached to an infinitely strong, infinitely small diameter central steel shaft. This configuration would be energy efficient because penetration resistance and frictional resistance is low. Installation torque to capacity relationships would be high. However, infinitely thin helix plates and infinitely small shafts are not realistically possible, so a balanced design of size, shape, and material is required to achieve consistent, reliable torque to capacity relationships.

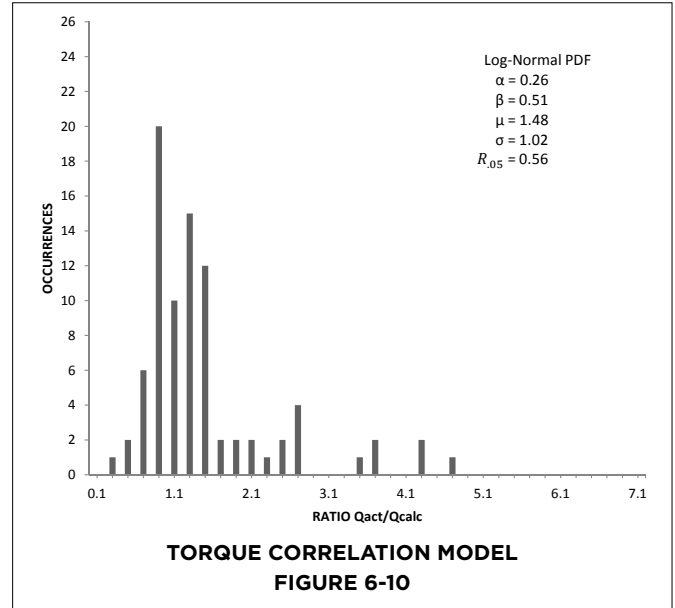
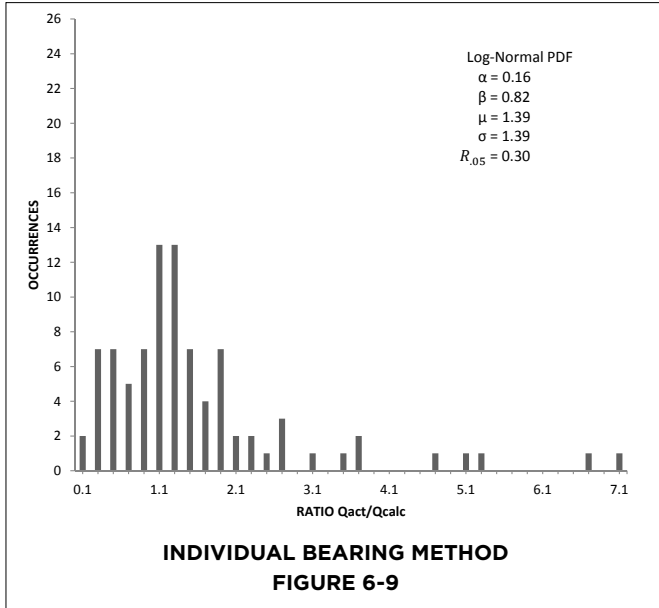
As stated previously, the empirical relationship between installation torque and ultimate capacity is well known, but not precisely defined. As one method of explanation, a theoretical model based on energy exerted during installation has been proposed [Perko (2000)]. The energy model is based on equating the energy exerted during installation with the penetration and frictional resistance. Perko showed how the capacity of an installed helical pile/anchor can be expressed in terms of installation torque, applied downward force, soil displacement, and the geometry of the pile/anchor. The model indicates that K_t is weakly dependent on crowd, final installation torque, number of helix plates, and helix pitch. The model also indicates that

K_t is moderately affected by helix plate radius and strongly affected by shaft diameter and helix plate thickness.

The important issue is energy efficiency. Note that a large shaft helical anchor/pile takes more energy to install into the soil than a small shaft pile/anchor. Likewise, a large diameter, thick helix takes more energy to install into the soil than a smaller diameter, thinner helix. The importance of energy efficiency is realized when one considers that the additional energy required to install a large displacement helical pile/anchor contributes little to the load capacity of the pile/anchor. In other words, the return on the energy “investment” is not as good. This concept is what is meant when Hubbell engineers say large shaft diameter and/ or large helix diameter (>16” diameter) pile/anchors are not efficient “torque-wise.” This doesn’t mean large diameter or large helix plate piles are not capable of producing high capacity, it just means the installation energy, i.e. machine, must be larger in order to install the pile.

If one considers an energy balance between the energy exerted during loading and the appropriate penetration energy of each of the helix plates, then it can be realized that any installation energy not specifically related to helix penetration is wasted. This fact leads to several useful observations. For a given helix configuration and the same available installation energy (i.e., machine):

1. Small displacement shafts will disturb less soil than large displacement shafts.
2. Small displacement shafts result in less pore pressure buildup than large displacement shafts.
3. Small displacement shafts will penetrate farther into a given bearing strata than large displacement shafts.



4. Small displacement shafts will penetrate soils with higher SPT “N” values than large displacement shafts.
5. Small displacement shafts will generate more axial capacity with less deflection than large displacement shafts.
6. K_t varies inversely with shaft diameter.

RELIABILITY OF TORQUE/CAPACITY MODEL

Hoyt and Clemence (1989) analyzed 91 tension load tests at 24 different sites with sand, silt and clay soils all represented. All of the tests used in the study were short term; most were strain controlled and included a final loading step of imposing continuous deflection at a rate of approximately 4 inches (102 mm) per minute. This final load was taken as the ultimate capacity. The capacity ratio Q_{act}/Q_{calc} was obtained for each test by dividing the actual capacity (Q_{act}) by the calculated capacity (Q_{calc}). Q_{calc} was calculated by using three different capacity models: (1) Cylindrical shear, (2) Individual bearing, and (3) Torque correlation. These data were then compared and plotted on separate histograms (see Figures 6-9 and 6-10, cylindrical shear histogram not shown).

All three capacity models exhibited the capability of over-predicting pile/anchor capacity. This would suggest the use of appropriate Factors of Safety. However, the authors did not discriminate between “good” and “poor” bearing soils when analyzing the results. In other words, some of the test data analyzed were in areas where the helix plates were located in soils typically not suitable for end bearing, (i.e., sensitive) clays and loose sands.

All three capacity models’ mean values were quite close, but the range and standard deviation were significantly lower for the torque correlation method than for the other two. This improved consistency is probably due to the removal of several

random variables from the capacity model. Therefore, the installation torque correlation method yields more consistent results than either of the other two methods. The installation torque method does have one disadvantage, however, in that it cannot be used until after the helical pile/anchor has been installed. Therefore, it is better suited to on-site production control and termination criteria than design in the office.

Perko (2012) suggested that if both individual bearing capacity and torque correlation are used to determine the bearing capacity of a helical pile/anchor, the resulting capacity will be accurate to within 97.7% reliability.

MEASURING INSTALLATION TORQUE

The torque correlation method requires the installation torque to be measured and recorded in the field. There are several





methods that can be used to measure torque, and Hubbell Power Systems, Inc. has a complete line of torque indicators to choose from. Each one is described below along with its advantages and disadvantages:

SHAFT TWIST

A.B. Chance Company stated in early editions of the Encyclopedia of Anchoring (1977) that for standard SS5 Anchors, “the most secure anchoring will result when the shaft has a 1 to 1-1/2 twist per 5-foot section.” Shaft twist is not a true torque-indicating device. It has been used as an indication of “good bearing soil” since Type SS anchors were first introduced in the mid-1960s. Shaft twist should not be used exclusively as a true torque-indicating device. Some of the reasons for this are listed below.

Advantages:

- Simple, cheap, easy to use.
- Doesn't require any additional tooling.
- Visible indication of torque.

Disadvantages:

- Qualitative, not quantitative torque relationship.
- Not very accurate.
- Shaft twist can't be correlated to installation torque on a consistent basis.
- Type SS5, SS150, SS175, SS200, and SS225 shafts twist, or wrap-up, at different torque levels.
- Shaft twist for a round shaft is not obvious without other means of reference.

SHEAR PIN TORQUE LIMITER

A shear pin torque limiter is a mechanical device consisting of two shear halves mounted to a central pin such that the shear halves are free to rotate (see Figure 6-11). Shear pins inserted into perimeter holes prevent the shear halves from rotating and are rated to shear at 500 ft-lb of torque per pin. Required torque can be achieved by loading the shear halves with the appropriate number of pins, i.e., 4000 ft-lb = 8 pins. The shear pin torque limiter is mounted in line with the torque motor and pile/anchor tooling.

Advantages:

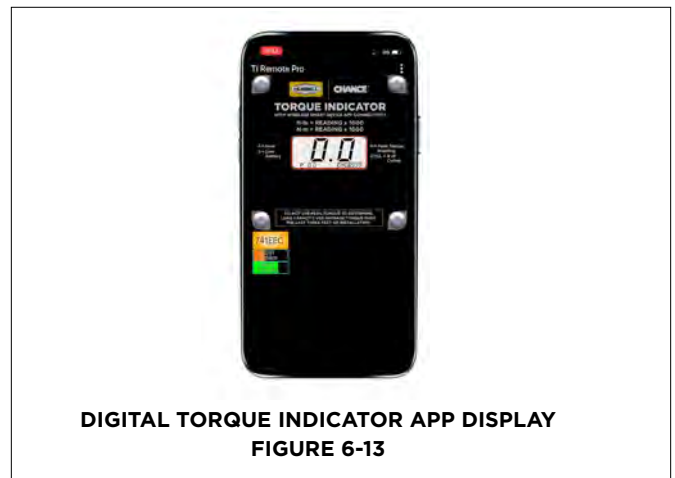
- Simple design, easy to use.
- Tough and durable, will take a lot of abuse and keep working.
- Accurate within $\pm 5\%$ if kept in good working condition.
- Torque limiter - used to prevent exceeding a specified torque.
- Relatively inexpensive to buy and maintain.
• Easy interchange from one machine to another.

Disadvantages:

- Point-wise torque indicator, i.e., indicates torque at separate points, not continuously.



**DIGITAL TORQUE INDICATOR
FIGURE 6-12**



**DIGITAL TORQUE INDICATOR APP DISPLAY
FIGURE 6-13**



**DIGITAL TORQUE INDICATOR WITH INCLUDED
CASE AND HARDWARE, FIGURE 6-14**

- Requires constant unloading and reloading of shear pins.
- Limited to 10,000 ft-lb.
- Sudden release of torsional (back-lash) energy when pins shear.
- Fits tools with 5-1/4" bolt circle only.

DIGITAL TORQUE INDICATOR

A digital torque indicator is a device consisting of strain gauges mounted to a torsion bar located between two bolt flanges (see Figure 6-12). This tool measures installation torque by measuring the shear strain of the torsion bar. The digital display reads torque directly. The digital torque indicator is mounted in-line with the torque motor and pile/anchor tooling.

Advantages:

- Simple torsion bar & strain gauge design, easy to use.
- Continuous reading torque indicator.
- Digital display reads torque directly.
- Accurate within $\pm 2\%$ if kept in good working condition.
- Fits tools with 5-1/4" and 7-5/8" bolt circles.
- Calibrated with equipment traceable to US Bureau of Standards before leaving plant.
- Can be used as a calibration tool for other types of torque indicators.
- Easy interchange from one machine to another.
- Reliable, continuous duty torque indicator.
- Bluetooth technology
- Torque displays on base unit and transmits to Bluetooth device (Bluetooth device not supplied with Torque Indicator)
- Torque Indicator Remote Pro App is available in Android and IOS versions for free download from Google Play™ or Apple® App Store

Disadvantages:

- Drive tools must be switched out when installing different types of helical pile/anchor.

DP-1 DIFFERENTIAL PRESSURE TORQUE INDICATOR

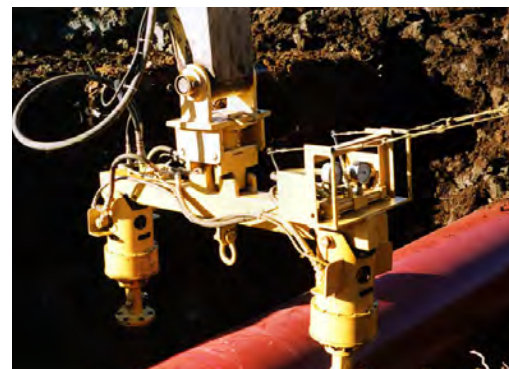
A differential pressure torque indicator is a hydraulic device consisting of back-to-back hydraulic pistons; hoses, couplings, and a gauge (see Figure 6-15). Its operation is based on the principle that the work output of a hydraulic torque motor is directly related to the pressure drop across the motor. The DP-1 hydraulically or mechanically "subtracts" the low pressure from the high to obtain the "differential" pressure. Installation torque is calculated using the cubic inch displacement and gear ratio of the torque motor. The DP-1 piston block and gauge can be mounted anywhere on the machine. Hydraulic hoses must be connected to the high and low pressure lines at the torque motor.

Advantages:

- Indicates torque by measuring pressure drop across hydraulic torque motor.
- No moving parts.
- Continuous reading torque indicator.
- Very durable - the unit is not in the tool string.
- Pressure gauge can be located anywhere on the machine.
- Analog type gauge eliminates "transient" torque peaks.
- Pressure gauge can be overlaid to read torque (ft-lb) instead of pressure (psi).
- Accurate within $\pm 5\%$ if kept in good working condition.
- After mounting, it is always ready for use.
- Can be provided with multiple readout gauges.

Disadvantages:

- Requires significant initial installation setup time and material, i.e., hydraulic fittings, hoses, oil.
- Requires a hydraulic pressure-to-torque correlation based on the torque motor's cubic inch displacement (CID) and gear ratio.
- For two-speed torque motors, pressure-to-torque correlation changes depending on which speed the motor is in (high or low).
- Requires periodic recalibration against a known standard, such as the digital torque indicator, or shear pin torque limiter.
- Sensitive to hydraulic leaks in the lines that connect the indicator to the torque motor.
- Relatively expensive.
- Difficult interchange from one machine to another.



**DIFFERENTIAL PRESSURE
TORQUE INDICATOR
FIGURE 6-15**



TORQUE INDICATOR CALIBRATION

All torque indicators require periodic calibration. Hubbell Power Systems, Inc. recommends that torque indicators be calibrated at least once per year. The digital torque indicator can be used in the field to calibrate other indicators, such as hydraulic pressure gauges and the DP-1. As torque motors age, the relationship between hydraulic pressure and installation torque will change. Therefore, it is recommended that hydraulic torque motors be periodically checked for pressure/torque relationship throughout their service life.

INSTALLATION TERMINATION CRITERIA

The Engineer of Record can use the relationship between installation torque and ultimate capacity to establish minimum torque criteria for the installation of production helical piles/anchor. The recommended default values for K_t of [10ft⁻¹ (33m⁻¹)] for Chance® Type SS, [9ft⁻¹ (30m⁻¹)] for Type RS2875, [7ft⁻¹ (23m⁻¹)] for Type RS3500 and [6ft⁻¹ (20m⁻¹)] for Type RS4500 will typically provide conservative results.

For large projects that merit the additional effort, a pre-production test program can be used to establish the appropriate torque correlation factor (K_t) for the existing project soils. It is recommended that K_t be determined by dividing the ultimate capacity determined by load test by the average installation (effective) torque taken over the last 3 feet (1 meter) of penetration into the bearing strata. The minimum effective torsional resistance criterion applies to the “background” resistance; torque spikes resulting from encounters with obstacles in the ground must be ignored in determining whether the torsional resistance criterion has been satisfied. The minimum effective torsional resistance criterion (the average installation torque taken over the last 3 feet of penetration) may not be applicable in certain soil profiles, such as, a relatively soft stratum overlying a very hard stratum. Engineering judgment must be exercised. See Appendix B for more detailed explanation of full-scale load tests. Large-scale projects warrant more than one pre-production test.

Whatever method is used to determine K_t , the production helical piles/anchors should be installed to a specified minimum torque and overall minimum depth. These termination criteria should be written into the construction documents. See www.chancefoundationsolutions.com for model specifications that contain sections on recommended termination criteria for helical piles/anchors.

ICC-Evaluation Services ESR-2794 requires the following installation termination criteria:

- When installing single-helix anchors/piles that will be loaded in tension and all multi-helix anchors/piles, torsional resistance must be recorded at the final tip embedment minus 2 feet (710 mm) and final embedment minus 1 foot (305 mm), in addition to the resistance at final embedment.

- For single-helix compression piles, the final torsional resistance reading must be equal to or exceed the specified minimum.
- For multi-helix anchors and piles, the average of the final three torsional resistance readings must be equal to or exceed the specified minimum.
- The tip embedment and torsional resistance readings must be verified to meet or exceed the specified termination criteria before terminating installation.

MINIMUM BEARING DEPTH OF TOP-MOST HELIX

For deep foundation behavior, Hubbell Power Systems, Inc. recommends the minimum vertical depth of the top-most helix plate should be at least five times the diameter of the top-most helix. Natural factors such as frost depth and active zones (expansive soil) can also affect minimum depth. Hubbell Power Systems, Inc. recommends the minimum vertical depth of the top-most helix plate should be at least three times the diameter of the top most helix below the maximum frost depth or depth of active zone. For example, if the frost depth is 4 feet and the top-most helix plate is 12 in (305 mm), then the minimum depth to the top-most helix is $4 + 3 \times (12 \text{ in}) = 7 \text{ ft}$ (2.1 m).

TOLERANCES

It is possible to install helical piles/anchors within reasonable tolerance ranges. For example, it is common to locate and install an pile/anchor within 1 inch (25 mm) of the staked location. Plumbness can usually be held within $\pm 1^\circ$ of design alignment. For vertical installations a visual plumbness check is typically all that's required. For battered installations, an inclinometer can be used to establish the required angle. See www.chancefoundationsolutions.com for model specifications that contain sections on recommended termination criteria for helical piles/anchors.

TORSIONAL STRENGTH RATING

Torsional strength is important when choosing the correct helical pile/anchor for a given project. It is a practical limit since the torque strength must be greater than the resistance generated during installation. In fact, the central steel shaft is stressed more during installation than at any other time during the life of the helical pile/anchor. This is why it is important to control both material strength variation and process capability in the fabrication process. Hubbell Power Systems, Inc. designs and manufactures helical piles/anchors to achieve the torque ratings published in the product family sections in Section 7. The ratings are listed based on product series, such as SS5, SS175, RS3500, etc.

The torque rating is defined as the maximum torsional energy that should be applied to the helical pile/anchor during installation in soil. It is not the ultimate torque strength, defined as the point where the central shaft experiences torsion fracture. It is best described as an allowable limit, or “safe torque” that can be applied to the helical pile/anchor. Some other manufacturers publish torque ratings based on ultimate torque strength.



The designer should select the product series that provides a torque strength rating that meets or exceeds the anticipated torsion resistance expected during the installation. HeliCAP® Helical Capacity Design Software (see Section 5) generates installation torque vs. depth plots that estimate the torque resistance of the defined soil profile. The plotted torque values are based on a K_t of 10 for Type SS and 9, 7 or 6 for Type RS. The torque ratings published in the product family sections in Section 7 are superimposed on the HeliCAP® Torque vs Depth plot, so the user can see at a glance when the estimated torque resistance equals or exceeds the torque rating of a given product series.

In some instances, it may be necessary to exceed the torque rating in order to achieve the minimum specified depth, or to install the helical pile/anchor slightly deeper to locate the helix plates farther into bearing stratum. This “finishing torque limit” should never exceed the published torque rating by more than 10%. To avoid fracture under impact loading due to obstruction laden soils, choose a helical product series with at least 30% more torque strength rating than the expected torque resistance. Note that the possibility of torsion fracture increases significantly as the applied torque increases beyond the published ratings. The need to install helical pile/anchors deeper is better accomplished by reducing the size and/or number of helix plates, or by choosing a helical product series with a higher torque rating.

REFERENCES:

1. A.B. Chance Company, Encyclopedia of Anchoring, Bulletin 01-9401UA, 1977, A.B. Chance Company, Centralia, MO
2. A.B. Chance, a Division of Hubbell Power Systems, Inc., Product Selection Guide - Resistance Piers and Helical Piles for Remedial (Underpinning) Applications, Bulletin 01-0601, Hubbell, Inc., Centralia, MO, 2006.
3. Clemence, S. P., L.K. Crouch and R.W. Stephenson, Prediction of Uplift Capacity for Helical Anchors in Sand, Conference Proceedings from the Second Geotechnical Engineering Conference - Cairo University, Cairo, Egypt, 1994.
4. Crouch, L.K. and R.W. Stephenson (1991), Installation Torque Requirements and Uplift Capacity of Helical Soil Anchors Using Measured Geotechnical Properties of Soil, Doctoral Dissertation, University of Missouri-Rolla, Rolla, MO, 1994.
5. Hargrave, R.L., and R.E. Thorsten, Helical Piers in Expansive Soils of Dallas, Texas, Proceedings of the 7th International Conference on Expansive Soils, 1992.
6. Hoyt, R.M. and S.P. Clemence, Uplift Capacity of Helical Anchors in Soil, Proceedings of the 12th International Conference on Soil Mechanics and Foundation Engineering, Rio de Janeiro, Brazil, 1989.
7. Perko, Howard A., Energy Method for Predicting Installation Torque of Helical Foundations and Anchors, Proceedings of Sessions of Geo-Denver 2000, ASCE Geotechnical Special Publication NO. 100, 2000.
8. Helical Pile Design Guide, by the Deep Foundations Institute's (DFI) Helical Piles and Tiebacks Committee (2019).



SECTION 7: DRAWINGS AND RATINGS

CONTENTS

Atlas Resistance Piers	7-2
Chance Helical Piles/Anchors.....	7-7
Square Shaft Helical Piles and Anchors	
SS125	7-11
SS5	7-14
SS150	7-17
SS175	7-20
SS200	7-23
SS225	7-26
Round Shaft Helical Piles and Anchors	
RS2875.203.....	7-29
RS2875.203 Building Code.....	7-32
RS2875.276	7-36
RS2875.276 Building Code	7-39
RS2875.276 High Capacity	7-43
RS3500.300	7-46
RS3500.300 Building Code	7-49
RS4500.237	7-53
RS4500.337	7-56
RS4500.337 Building Code	7-59
RS5500.361.....	7-63
RS6625.280	7-66
RS7000.362.....	7-68
RS8625.250	7-71
RS9625.395.....	7-73
Chance Rock-It Helical Lead.....	7-75
Type SS/RS Combination Helical Piles	7-76
Chance Helical Pulldown Micropiles	7-77
Remedial Repair Brackets For Chance Helical Piles.....	7-81
New Construction Pile Caps	7-90

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 should be conducted and state and local building codes and authorities should be consulted 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 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.

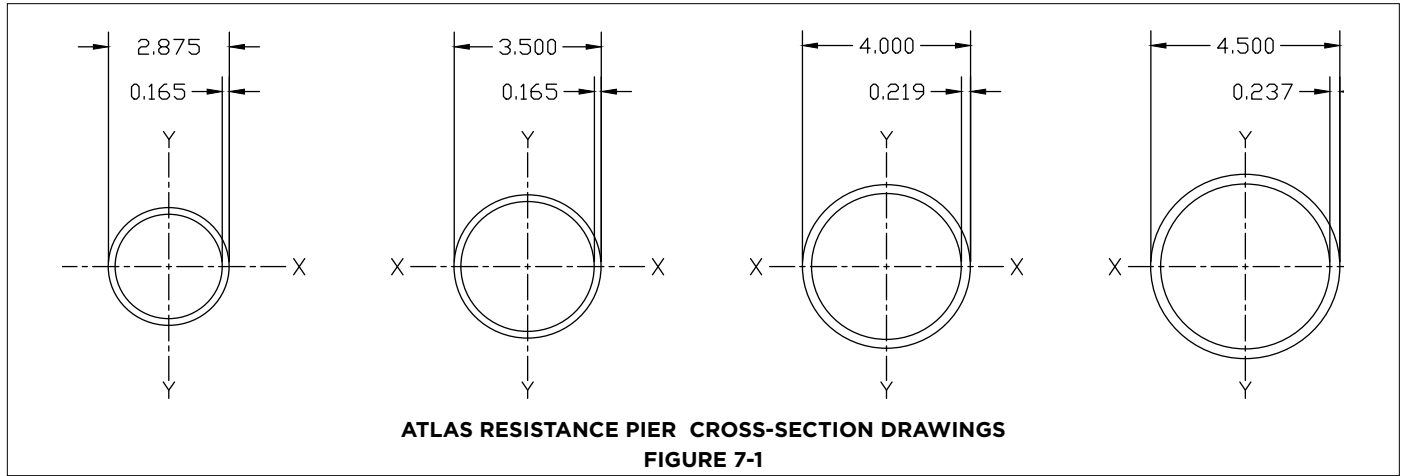


ATLAS RESISTANCE PIERS

The Atlas Resistance® pier is an assembly of structural steel components that includes a steel bracket attached to the foundation or slab, which is then mounted on a steel pier that is installed to bedrock or firm bearing stratum. The lead pier starter section includes a unique friction-reduction collar that reduces

skin friction on the pier pipe during installation. Hubbell Power Systems, Inc., offers a broad range of pier pipe sizes and remedial repair brackets for both foundation and slab underpinning applications. This section will discuss those products in detail along with their capacity ratings.

PIER PIPE SHAFTS



ATLAS RESISTANCE PIER SECTION PROPERTIES, TABLE 7-1

PRODUCT SERIES	SHAFT SIZE (in) [mm]	WALL THICKNESS (in) [mm]	METAL AREA (in ²) [cm ²]	PERIMETER (in) [cm]	MOMENT OF INERTIA (in ⁴) [cm ⁴] <i>I_{x-x}</i> , <i>I_{y-y}</i> , <i>I_{x-y}</i>	SECTION MODULUS (in ³) [cm ³]	
						<i>S_{x-x}</i> , <i>S_{y-y}</i>	<i>S_{x-y}</i>
RS2875.165	2.875 [73]	0.165 [4.2]	1.4 [9.0]	9.0 [22.9]	1.29 [53.7]	0.90 [14.7]	0.90 [14.7]
RS3500.165	3.5 [89]	0.165 [4.2]	1.7 [11.0]	11.0 [27.9]	2.41 [100.3]	1.38 [22.6]	1.38 [22.6]
RS4000.219	4.0 [101]	0.219 [5.6]	2.6 [16.8]	12.6 [32.0]	4.66 [194.0]	2.33 [38.2]	2.33 [38.2]
RS4500.237	4.5 [114]	0.237 [6.0]	3.2 [20.6]	14.1 [35.9]	7.23 [301.0]	3.21 [52.6]	3.21 [52.6]



REMEDIAL REPAIR BRACKETS FOR ATLAS RESISTANCE® PIERS

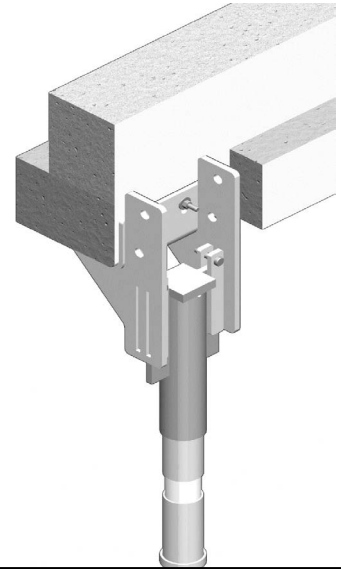
ATLAS RESISTANCE STANDARD AND MODIFIED 2-PIECE SYSTEMS

- Use for lifts up to 4”
- All 2-piece pier systems include:
 - Pier bracket
 - Top pier platform
 - Pier starter with friction reduction collar
 - Pier section
 - “M” designates one modified sleeve included

ORDER SEPARATELY: Two pier pins (two Grade 8 bolts for 4-1/2” pier) and pier shims. Each pier requires a minimum of four anchor bolts.

NOTE: Anchor bolts not supplied by Hubbell Power Systems, Inc.

See Note 3 at bottom of table for available finishes.



ATLAS RESISTANCE STANDARD AND MODIFIED 2-PIECE PIER DESIGNATORS				
PIER DESIGNATION	ULTIMATE CAPACITY ¹ (lb)	MAX WORKING CAPACITY ¹ (lb)	PIER DIA (in)	FEATURES
AP-2-UF-2875.165	60,000	30,000	2-7/8	Lowest cost
AP-2-UF-2875.165M	70,000	35,000	2-7/8	Lowest cost; Increased rotational stiffness; Recommended for weak surface soils
AP-2-UF-3500.165	85,000	42,500	3-1/2	“Flow Coat” pier pipe standard (NER579) ²
AP-2-UFVL-3500.165	86,000	43,000	3-1/2	Has additional mounting plate for two additional anchor bolts
AP-2-UF-3500.165M	91,000	45,500	3-1/2	“Flow Coat” pier pipe standard (NER579) ² ; Increased rotational stiffness; Recommended for weak surface soils
AP-2-UFVL-3500.165M	91,000	45,500	3-1/2	Has additional mounting plate for two additional anchor bolts; Increased rotational stiffness
AP-2-UF-4000.219	98,000	49,000	4	Higher capacity; Easier installation than AP2-3500M
AP-2-UFVL-4000.219	110,000	55,000	4	Has additional mounting plate for two additional anchor bolts
AP-2-UF-4500.237	141,000	70,500	4-1/2	Highest capacity

Notes:

- 1 Capacities based upon maximum pipe exposure of 2 feet and soil strength having a minimum Standard Penetration Test (SPT) blow count (N) of 4. The capacities are based on a pier depth to fixity of 5'-6”.
- 2 Complies with the structural provisions of the most recent editions of BOCA National Code, ICBO Uniform Code, SBCCI Standard Code, and 2000 International Building and Residential Code (2002 Accumulative Supplement).
- 3 Available finishes: P = Entire product supplied mill finish steel. G = Entire product supplied galvanized. PA = Plain steel bracket assy; “Flow Coat” corrosion protection on pier pipe. GA = HDG bracket assy; “Flow Coat” corrosion protection on pier pipe.

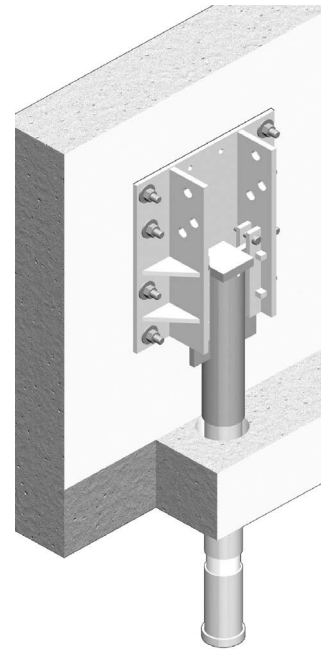


ATLAS RESISTANCE® 2-PIECE PLATE PIER SYSTEMS

- Easy surface mount installation
- May be used for round columns (custom manufactured - see information below)
 - Use for lifts up to 4"
- All plate pier systems include:
 - Pier bracket
 - Top pier platform
 - Pier starter with friction reduction collar
 - Pier section

ORDER SEPARATELY: Two pier pins (two Grade 8 bolts for 4-1/2" pier) and pier shims. Six or eight anchor bolts per pier are required (consult specification drawings on www.chancefoundationsolutions.com for anchor bolt specifications).

NOTE: Anchor bolts not supplied by Hubbell Power Systems, Inc. See Note 3 at bottom of table for available finishes.



ATLAS RESISTANCE 2-PIECE PIER PLATE PIER DESIGNATORS				
PIER DESIGNATION	ULTIMATE CAPACITY ¹ (lb)	MAX WORKING CAPACITY ¹ (lb)	PIER DIA (in)	FEATURES
AP-2-PP-2875.165	60,000	30,000	2-7/8	Lowest cost
AP-2-PP-2875.165M	70,000	35,000	2-7/8	Lowest cost; Increased rotational stiffness; Recommended for weak surface soils
AP-2-PP-3500.165	86,000	43,000	3-1/2	Standard pier for most applications; "Flow Coat" pier pipe standard
AP-2-PP-3500.165M	90,000	45,000	3-1/2	"Flow Coat" pier pipe standard; Increased rotational stiffness; Recommended for weak surface soils
AP-2-PP-4000.219	103,000	51,500	4	Larger pier pipe; Higher capacity
AP-2-PP-4500.237	112,000	56,000	4-1/2	Commercial and Industrial applications; Greater pier pipe diameter; Highest capacity

Notes:

1. Capacities based upon maximum pipe exposure of 2 feet and soil strength having a minimum Standard Penetration Test (SPT) blow count (N) of 4. The capacities are based on a pier depth to fixity of 5'-6".
2. Mounting distance from bottom of stem wall to bottom of plate pier bracket must be greater than 5".
3. Available Finishes: P = Entire product supplied mill finish steel. G = Entire product supplied galvanized. PA = Plain steel bracket assy; "Flow Coat" corrosion protection on pier pipe. GA = HDG bracket assy; "Flow Coat" corrosion protection on pier pipe.

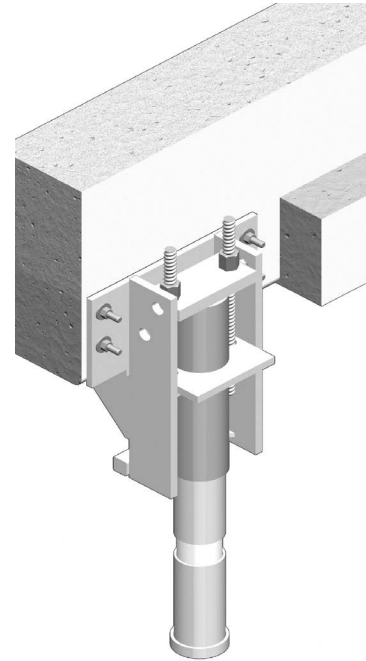
ROUND COLUMN APPLICATIONS

Where a plate pier must be attached to a round column, the pier bracket can be custom manufactured at extra cost to match the radius of the column and the side rail width will be extended for clearance. Please provide diameter of column when ordering. Specify: AP-2-PPRC-2875.165, AP-2-PPRC-3500.165M, or AP-2-PPRC-4000.219.



ATLAS RESISTANCE® CONTINUOUS LIFT PIER SYSTEMS

- Use for lifts exceeding 4”
- Exceptional extended lift capabilities
- All continuous lift pier systems include:
 - Continuous lift pier bracket assembly
 - Cap plate assembly
 - Top pier sleeve (not applicable on AP-CL-UF-4000.219)
 - Pier starter with friction reduction collar
 - Pier section



ORDER SEPARATELY: Reuseable lift head, continuous thread rebar, nuts, and 6 anchor bolts.

See Note 2 at bottom of table for available finishes.

ATLAS RESISTANCE CONTINUOUS LIFT PIER DESIGNATORS				
PIER DESIGNATION	ULTIMATE CAPACITY ¹ (lb)	MAX WORKING CAPACITY ¹ (lb)	PIER DIA (in)	FEATURES
AP-CL-UF-2875.165	40,000	20,000	2-7/8	Lowest cost
AP-CL-UF-3500.165	61,000	30,500	3-1/2	“Flow Coat” pier pipe standard
AP-CL-UF-4000.219 (Similar to illustration)	100,000	50,000	4	Higher capacity

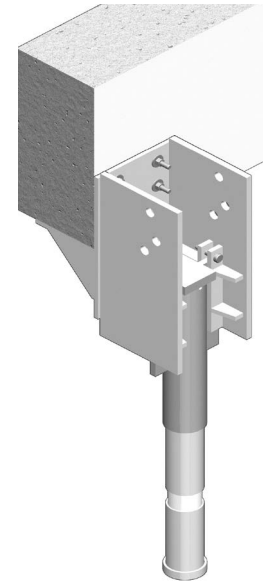
Notes:

1. Capacities based upon maximum pipe exposure of 2 feet and soil strength having a minimum Standard Penetration Test (SPT) blow count (N) of 4. The capacities are based on a pier depth to fixity of 5'-6".
2. Available Finishes: P = Entire product supplied mill finish steel. G = Entire product supplied galvanized. PA = Plain steel bracket assy; “Flow Coat” corrosion protection on pier pipe. GA = HDG bracket assy; “Flow Coat” corrosion protection on pier pipe.



ATLAS RESISTANCE® 2-PIECE PREDRILLED PIER SYSTEMS

- Use for lifts up to 4”
- Drilled pier access hole required where unsuitable rock is near surface
- Use where designer requires penetration into bearing rock
- Eccentricity from wall to centerline of pipe is 6-3/4”
- All pre-drilled piers include:
 - Pier bracket
 - Top pier platform
 - Pier starter with friction reduction collar
 - Pier section



ORDER SEPARATELY: Two pier pins and four anchor bolts per pier and shims as required.

See Note 2 at bottom of table for available finishes.

ATLAS RESISTANCE 2-PIECE PREDRILLED PIER DESIGNATORS				
PIER DESIGNATION	ULTIMATE CAPACITY ¹ (lb)	MAX WORKING CAPACITY ¹ (lb)	PIER DIA (in)	FEATURES
AP-2-UFPDVL-2875.165M	58,000	29,000	2-7/8	Lowest cost
AP-2-UFPDVL-3500.165M	62,000	31,000	3-1/2	Low cost; Corrosion resistant; “Flow Coat” pier pipe standard
AP-2-UFPDVL-4000.219	76,000	38,000	4	Higher capacity
AP-2-UFPD-4500.237	92,000	46,000	4-1/2	Highest capacity; Commercial and Industrial applications

Notes:
 1 Capacities based upon maximum pipe exposure of 2 feet and soil strength having a minimum Standard Penetration Test (SPT) blow count (N) of 4. The capacities are based on a pier depth to fixity of 5'-6".
 2 Available Finishes: P = Entire product supplied mill finish steel. G = Entire product supplied galvanized. PA = Plain steel bracket assy; “Flow Coat” corrosion protection on pier pipe. GA = HDG bracket assy; “Flow Coat” corrosion protection on pier pipe.

ATLAS RESISTANCE 2-PIECE PREDRILLED PLATE PIER DESIGNATORS (SPECIAL ORDER; NOT SHOWN)				
PIER DESIGNATION	ULTIMATE CAPACITY ¹ (lb)	MAX WORKING CAPACITY ¹ (lb)	PIER DIA (in)	FEATURES
AP-2-PPPD-3500.165	76,000	38,000	3-1/2	Lowest cost; Corrosion resistant; “Flow Coat” pier pipe standard
AP-2-PPPD-3500.165M	80,000	40,000	3-1/2	Low cost; Corrosion resistant; “Flow Coat” pier pipe standard
AP-2-PPPD-4000.219	83,000	41,500	4	Higher capacity
AP-2-PPPD-4500.237	95,000	47,500	4-1/2	Highest capacity; Commercial and Industrial applications

Notes:
 1 Capacities based upon maximum pipe exposure of 2 feet and soil strength having a minimum Standard Penetration Test (SPT) blow count (N) of 4. The capacities are based on a pier depth to fixity of 5'-6".
 2 Mounting distance from bottom of stem wall to bottom of plate pier bracket must be greater than 5".

CHANCE HELICAL PILES/ANCHORS

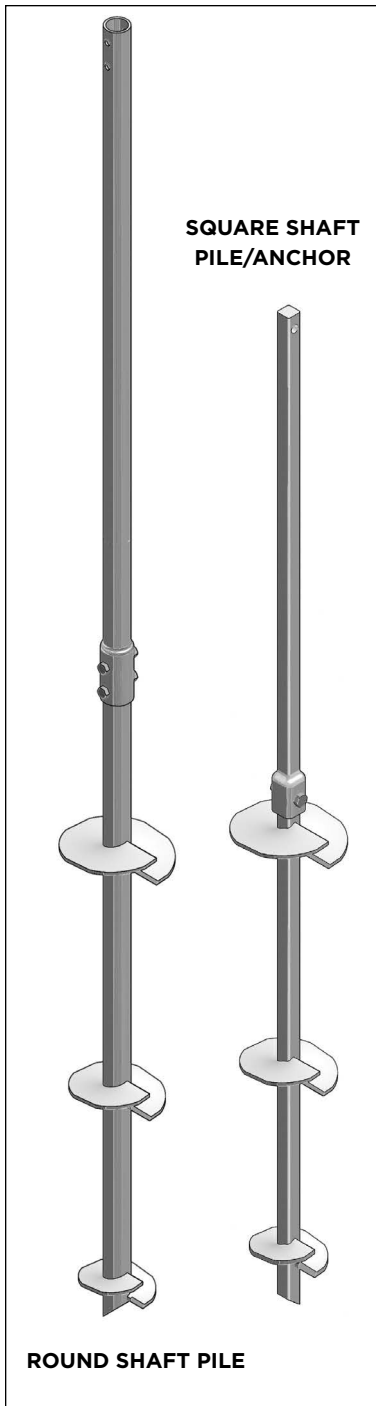
INTRODUCTION

A helical pile/anchor is a factory-manufactured steel deep foundation system designed to resist axial compression, axial tension, and/or lateral loads from structures. It consists of a central steel shaft with one or more helical-shaped bearing plates welded to the central steel shaft. The central steel shaft can be one-piece (non-extendable) or fully extendable with one or more extension shafts, couplings, and a bracket/termination that allows for connection to building structures. A helical pile/anchor is screwed into the ground by application of torsion and can be extended until a required depth or a suitable bearing soil stratum is reached. Load is transferred to the soil through the helix bearing plates. Central steel shafts are available in either Type SS (Square Shaft) series or Type RS (Round Shaft) series. The Type SS series are available in 1-1/4" to 2-1/4" solid square shaft sizes. The Type RS series are available in 2-7/8" to 8" diameter pipe shaft sizes. Type SS/RS Combo Piles are available for compression applications in soil conditions where dense/hard soils must be penetrated with soft/loose soils above the bearing strata. The grouted-shaft Chance® Helical Pulldown® micropile series is also used in applications similar to those requiring the use of the Type SS/RS Combo Piles, but have the additional benefit of generating capacity via skin friction along the grout-soil interface in a suitable bond zone stratum. For a complete list of mechanical ratings and section properties of the central steel shafts, see the tables found in each helical pile/anchor Product Family in this section. Refer to Section 3, Product Feasibility and Section 6, Installation Methodology for guidelines on the proper shaft selection based on application, soil conditions, site accessibility, etc.

Helical pile/anchor sections are joined with bolted couplings. Installation depth is limited only by soil density and practicality based on economics. A helical bearing plate or helix plate is one pitch of a screw thread. Most helical piles include more than one helix plate, and the plates are arranged in a tapered configuration with the smallest helix being on the bottom and the largest helix being on the top. The large majority of Chance helix plates, regardless of their diameter, have a standard 3" pitch. Being a true helical shape, the helix plates do not auger into the soil but rather screw into it with minimal soil disturbance. Chance helix plates are "pre-qualified" per the requirements of Table 3 in ICC-ES AC358 Acceptance Criteria for Helical Pile Systems and Devices, meaning they are generally circular in plan, have a true helix shape, and are attached perpendicular to the central steel shaft with the leading and trailing edges parallel. Helix plates are spaced at distances far enough apart that they function independently as individual bearing elements. Consequently, the capacity of a particular helix on a helical pile/anchor shaft is not influenced by the helix above or below it.

LEAD SECTION AND EXTENSIONS

The starter section or lead section contains the helix plates. This lead section can include a single helix or up to four helices. Additional helix plates can be added, if required, with the use of helical extensions. Standard helix sizes and projected areas are shown in Tables 7-2 and 7-3 below. Table 7-2 provides helix areas for Type Round Shaft (RS) helical piles, and Table 7-3 provides helix areas for Type Square Shaft (SS) helical piles. The full plate projected area includes the area occupied by the central steel shaft. The "area w/o hole" is the projected area of the helix plate less the area occupied by the center shaft. Most Chance helix plates are provided with a sharp leading edge, which is the front edge of the helix that penetrates the soils as the helical pile/anchor is advanced clockwise through soil. The sharp leading edge enables the helix to better slice through tough soils, roots, and seasonally frozen ground. Hubbell Power Systems, Inc., offers several helix plates with seashell leading edges as special options to the product series. Our standard configuration that works best in most tough soils conditions is the 90° design as shown below. The seashell cut is a leading edge with a spiral cut that is very effective when installing helical piles/anchors in debris-laden soils, cobbles, and weathered rock.

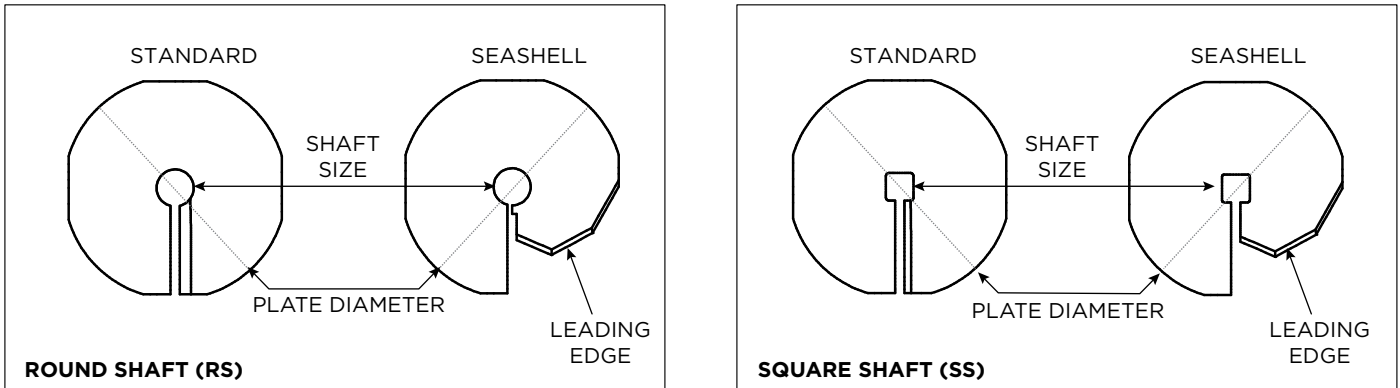




However, it is important to remember that the bearing capacity of the helical pile/anchor is reduced because the bearing surface area is reduced. Therefore, larger helix diameters or additional helix plates may be required when using seashell cut plates. Tables 7-2 and 7-3 include the projected areas of helix plates offered with the seashell cut. The helix plates are arranged on the shaft such that their diameters increase as they get farther from the pilot point. The practical limits on the

number of helices per pile/anchor is four to five if placed in a cohesive soil and six if placed in a cohesionless or granular soil.

Plain extensions are then added in standard lengths of 3, 5, 7, or 10 feet until the lead section penetrates into the bearing strata. Standard helix configurations are provided in the product series tables in this section. Note that lead time will be significantly reduced if a standard helix configuration is selected.



PROJECTED VIEWS OF ROUND SHAFT AND SQUARE SHAFT HELIX PLATES
FIGURE 7-2

CHANCE® ROUND SHAFT HELIX PLATE SIZES AND PROJECTED AREAS BY PRODUCT FAMILY, TABLE 7-2

		ROUND SHAFTS			
		STANDARD		SEASHELL	
	DIAMETER (in) [mm]	AREA w/o HOLE (ft ²) [m ²]	FULL PLATE AREA (ft ²) [m ²]	AREA w/o HOLE (ft ²) [m ²]	FULL PLATE AREA (ft ²) [m ²]
RS2875	8 [200]	0.290 [0.0269]	0.336 [0.0312]	0.270 [0.0251]	0.316 [0.0294]
	10 [250]	0.485 [0.0451]	0.531 [0.0493]	0.433 [0.0402]	0.479 [0.0445]
	12 [300]	0.725 [0.0674]	0.771 [0.0716]	0.633 [0.0588]	0.680 [0.0632]
	14 [350]	1.003 [0.0932]	1.049 [0.0975]	0.869 [0.0807]	0.915 [0.0850]
	16 [406]	1.31 [0.122]	1.378 [0.128]	N/A	N/A
RS3500	8 [200]	0.268 [0.0249]	0.336 [0.0312]	N/A	N/A
	10 [250]	0.463 [0.0430]	0.531 [0.0493]	N/A	N/A
	12 [300]	0.703 [0.0653]	0.771 [0.0716]	0.612 [0.0569]	0.680 [0.0632]
	14 [350]	0.981 [0.0911]	1.049 [0.0975]	N/A	N/A
	16 [406]	1.312 [0.122]	1.378 [0.128]	N/A	N/A
RS4500	8 [200]	0.224 [0.0208]	0.336 [0.0312]	N/A	N/A
	10 [250]	0.419 [0.0389]	0.531 [0.0493]	0.367 [0.0341]	0.479 [0.0445]
	12 [300]	0.659 [0.0612]	0.771 [0.0716]	N/A	N/A
	14 [350]	0.937 [0.0871]	1.049 [0.0975]	N/A	N/A
	16 [406]	1.266 [0.1176]	1.378 [0.128]	N/A	N/A
	20 [508]	2.034 [0.1889]	2.146 [0.1994]	N/A	N/A



CHANCE® SQUARE SHAFT HELIX PLATE SIZES AND PROJECTED AREAS BY PRODUCT FAMILY, TABLE 7-3

DRAWINGS & RATINGS

		SQUARE SHAFTS			
		STANDARD		SEASHELL	
	DIAMETER (in) [mm]	AREA w/o HOLE (ft ²) [m ²]	FULL PLATE AREA (ft ²) [m ²]	AREA w/o HOLE (ft ²) [m ²]	FULL PLATE AREA (ft ²) [m ²]
SS125	6 [150]	0.174 [0.0162]	0.185 [0.0172]	N/A	N/A
	8 [200]	0.324 [0.0301]	0.336 [0.0312]	0.304 [0.0282]	0.316 [0.0294]
	10 [250]	0.519 [0.0482]	0.531 [0.0493]	0.468 [0.0435]	0.479 [0.0445]
	12 [300]	0.759 [0.0705]	0.771 [0.0716]	0.668 [0.0621]	0.679 [0.0631]
	14 [350]	1.037 [0.0963]	1.049 [0.0975]	0.903 [0.0839]	0.915 [0.0850]
	16 [406]	1.366 [0.1269]	1.378 [0.128]	N/A	N/A
SS5/ SS150	6 [150]	0.169 [0.0157]	0.185 [0.0172]	0.156 [0.0145]	0.172 [0.0160]
	8 [200]	0.320 [0.0297]	0.336 [0.0312]	0.300 [0.0279]	0.316 [0.0294]
	10 [250]	0.515 [0.048]	0.531 [0.0493]	0.463 [0.0430]	0.479 [0.0445]
	12 [300]	0.755 [0.0701]	0.771 [0.0716]	0.663 [0.0616]	0.679 [0.0631]
	14 [350]	1.033 [0.0960]	1.049 [0.0975]	0.899 [0.0835]	0.915 [0.0850]
	16 [406]	1.362 [0.1265]	1.378 [0.128]	N/A	N/A
SS175	6 [150]	0.163 [0.151]	0.185 [0.0172]	N/A	N/A
	8 [200]	0.314 [0.0292]	0.336 [0.0312]	0.293 [0.0272]	0.316 [0.0294]
	10 [250]	0.509 [0.0473]	0.531 [0.0493]	0.457 [0.0425]	0.479 [0.0445]
	12 [300]	0.749 [0.0696]	0.771 [0.0716]	0.658 [0.0611]	0.679 [0.0631]
	14 [350]	1.027 [0.0954]	1.049 [0.0975]	N/A	N/A
	16 [406]	1.356 [0.126]	1.378 [0.128]	N/A	N/A
SS200	6 [150]	0.154 [0.0143]	0.185 [0.0172]	0.143 [0.0133]	0.172 [0.0160]
	8 [200]	0.305 [0.0283]	0.336 [0.0312]	N/A	N/A
	10 [250]	0.500 [0.0465]	0.531 [0.0493]	0.450 [0.0418]	0.479 [0.0445]
	12 [300]	0.740 [0.0687]	0.771 [0.0716]	N/A	N/A
	14 [350]	1.018 [0.0946]	1.049 [0.0975]	N/A	N/A
	16 [406]	1.349 [0.1253]	1.378 [0.128]	N/A	N/A
SS225	6 [150]	0.149 [0.0138]	0.185 [0.0172]	N/A	N/A
	8 [200]	0.300 [0.0279]	0.336 [0.0312]	N/A	N/A
	10 [250]	0.495 [0.0460]	0.531 [0.0493]	N/A	N/A
	12 [300]	0.735 [0.0683]	0.771 [0.0716]	N/A	N/A
	14 [350]	1.013 [0.0941]	1.049 [0.0975]	N/A	N/A
	16 [406]	1.341 [0.125]	1.378 [0.128]	N/A	N/A



Table 7-4 is a quick reference guide for the design professional. It relates ASTM D1586 SPT N_{60} values for cohesive and non-cohesive soils to the expected load capacity of various Chance® Type Square Shaft (SS) and Round Shaft (RS) helical piles. It is intended to be used as a reference guide to enable the designer to quickly determine which helical pile systems to use for project-specific soil conditions and load requirements.

CHANCE HELICAL PILE/ANCHOR LOAD CAPACITY TABLE, TABLE 7-4

SOIL TYPE		PRODUCT FAMILY			AXIAL COMPRESSION / TENSION CAPACITY*	
N_{60} VALUE** COHESIVE	N_{60} VALUE** NON-COHESIVE	PILE/ ANCHOR TYPE	SHAFT SIZE (in) [mm]	TORQUE RATING (ft-lb) [N·m]	ULTIMATE CAPACITY (P_u) (kip) [kN]	ALLOWABLE CAPACITY ($P_a = 0.5 P_u$) (kip) [kN]
25-35	25-30	SS5	1-1/2 [38]	5,700 [7,730]	57 [254]	28.5 [127]
25-40	25-35	SS150	1-1/2 [38]	7,000 [9,500]	70 [312]	35 [156]
35-50	35-40	SS175	1-3/4 [44]	10,500 [14,200]	105 [467]	52.5 [234]
50-70	40-60	SS200	2 [51]	16,000 [21,700]	160 [712]	80 [356]
70-90	60-80	SS225	2-1/4 [57]	21,000 [28,475]	210 [934]	105 [467]
20-25	15-20	RS2875.203	2-7/8 [73]	7,000 [9,490]	63 [280]	31.5 [140]
25-35	20-30	RS2875.276	2-7/8 [73]	8,000 [10,850]	72 [320]	36 [160]
35-40	30-35	RS3500.300	3-1/2 [89]	13,000 [17,600]	91 [405]	45.5 [202]
35-40	30-35	RS4500.337	4-1/2 [114]	25,000 [33,900]	150 [667]	76 [334]

* Based on Torque Rating - Axial Compression / Tension Capacity = Torque Rating x K_t . Well-documented correlations with installation torque are recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. "Default" K_t for Type SS = 10 ft⁻¹ (33 m⁻¹). "Default" K_t for Type RS2875 Series = 9 ft⁻¹ (30 m⁻¹); for Type RS3500.300 = 7 ft⁻¹ (23 m⁻¹); for Type RS4500.337 = 6 ft⁻¹ (20 m⁻¹).

** N_{60} values or blow count from the Standard Penetration Test per ASTM D1586.

Notes:

1. The table above is given as a guideline only. The capacity of Chance helical piles/anchors may vary depending on factors including, but not limited to, water table elevation and changes to that elevation, changes in soil conditions, and soil layer thicknesses.
2. Achievable capacities could be higher or lower than stated in the table depending on:
 - a. Site-specific conditions
 - b. On-site testing verification
 - c. Helical Pulldown® micropiles can achieve higher capacities in compression. On-site testing should be performed to verify additional pile capacity.
 - d. This table is to be used for preliminary design assessment only. Capacities should be verified on a per project, site-specific basis by a registered design professional.
3. The above table represents the hardest or densest soil conditions that the helical pile can be installed into. The helical pile will likely achieve its torque rating quickly upon encountering the highest N values indicated above.

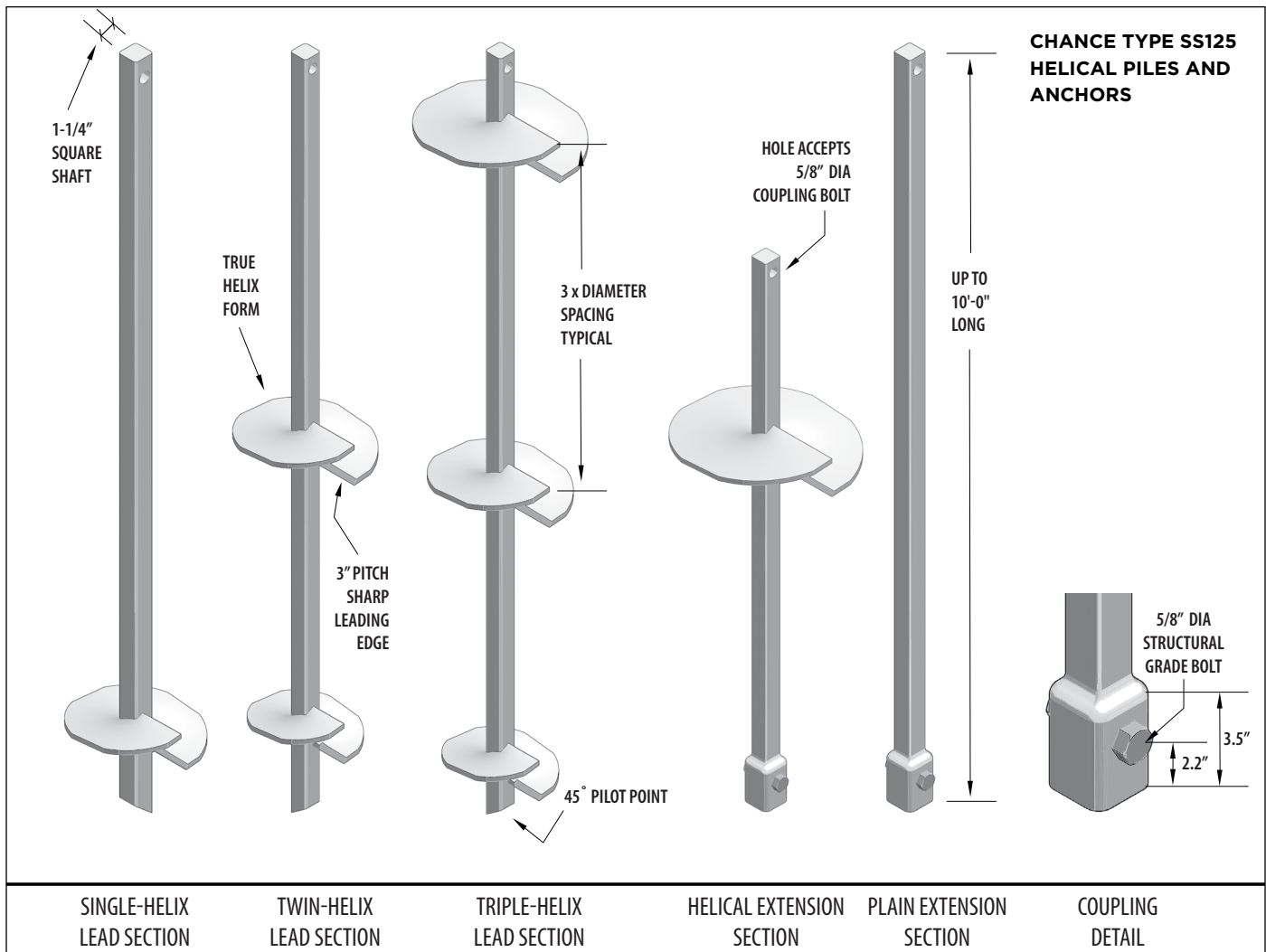
CHANCE TYPE SS125 HELICAL PILES AND ANCHORS

- 40 kip ULTIMATE - 20 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING - 4,000 ft-lb
- MULTI-PURPOSE 1-1/4 INCH SOLID ROUND-CORNERED-SQUARE STEEL SHAFT WITH INTEGRALLY FORMED SQUARE UPSET SOCKETS

DESCRIPTION:

Chance® Type SS125 helical piles and anchors have 40 kip ultimate capacity and 20 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Solid square shaft helical piles and anchors provide greater penetration into bearing soils and increased axial capacity in firm soils compared to round shaft helical piles with similar torque strength. Strength calculations are based on a design corrosion level of 50 years for most soil

conditions. Chance Type SS helical piles and anchors have a longer service life than do round shaft piles because of their reduced surface area. Chance Type SS helical piles and anchors feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are **MADE IN THE U.S.A.**



SS125 HELICAL PILE AND ANCHOR SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: Round-cornered-square (RCS) 1-1/4 inch solid steel shaft produced exclusively for Chance® products.

COUPLING: Forged as a deep socket from the steel shaft material as an integral part of the extension, connected with structural bolts.

HELIX - 3/8 INCH THICK: ASTM A572 or A1018 or A656, with minimum yield strength of 50 ksi.

3 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 6, 8, 10, 12, or 14 inches.

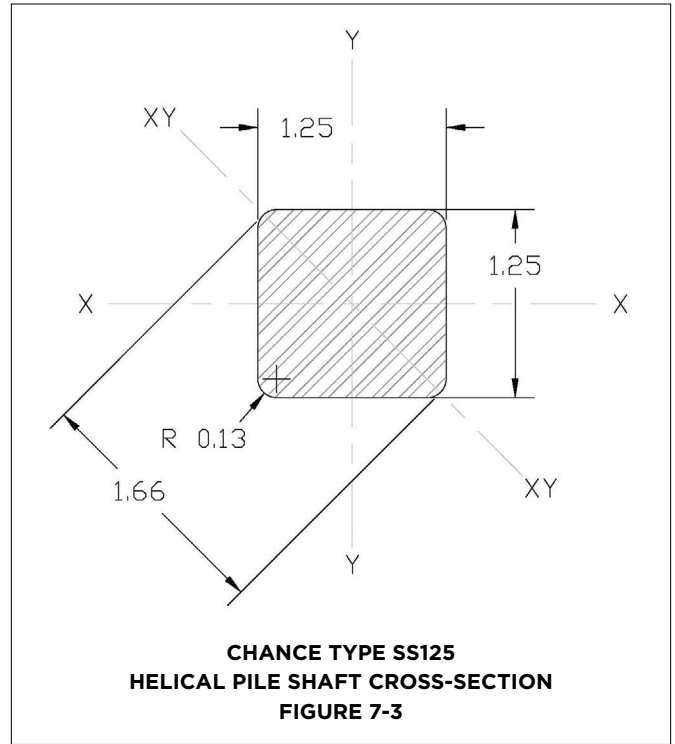
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight sharpened leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

- Single-, double-, and triple-helix lead sections, 1, 5, and 7 feet long
- Plain extensions, 3-1/2, 5, 7, and 10 feet long
- Extensions with helix plates, 3-1/2 feet long, single and double helix

Helical products are hot dip galvanized per ASTM A153 Class B-1.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.

NOMINAL, LRFD DESIGN, & ASD ALLOWABLE STRENGTHS OF SS125 HELIX PLATES FOR SHAFT AXIAL TENSION & COMPRESSION¹

HELIX DIAMETER (in) [mm]	THICKNESS (in) [mm]	NOMINAL STRENGTH (kip) [kN]	LRFD DESIGN STRENGTH (kip) [kN]	ASD ALLOWABLE STRENGTH (kip) [kN]
6 [150]	0.375 [9.5]	37.4 [166.3]	28.05 [124.7]	18.7 [83.2]
8 [200]	0.375 [9.5]	37.4 [166.3]	28.05 [124.7]	18.7 [83.2]
10 [250]	0.375 [9.5]	46.6 [207.3]	34.9 [155.5]	23.3 [103.6]
12 [300]	0.375 [9.5]	44.1 [196.2]	33.1 [147.2]	22.1 [98.3]
14 [350]	0.375 [9.5]	36.0 [160.1]	27.0 [120.1]	18.0 [80.1]

Notes:

For SI: 1 kip = 4.448 kN.

¹Capacities based on a design corrosion level of 50 years.

NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS OF CHANCE TYPE SS125 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS (kip) [kN]							
	FIRM SOIL				SOFT SOIL			
	FIXED		PINNED		FIXED		PINNED	
	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN
Lead, Single Helix	See Helix Strength Table Above		27.3 [121.4]	24.6 [109.4]	13.4 [59.6]	12.0 [53.4]	6.8 [30.2]	6.2 [27.6]
Lead, Multi-Helix	53.6 [238.4]	48.2 [214.4]	27.3 [121.4]	24.6 [109.4]	13.4 [59.6]	12.0 [53.4]	6.8 [30.2]	6.2 [27.6]
Extension	53.6 [238.4]	48.2 [214.4]	27.3 [121.4]	24.6 [109.4]				

Notes:

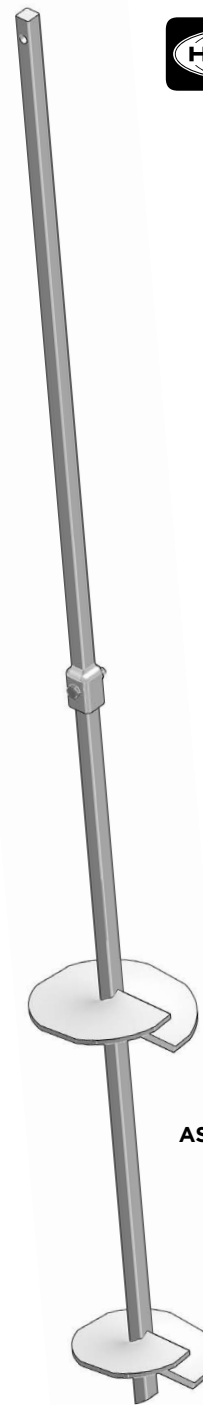
For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.



SS125 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled round-cornered-square (RCS) solid steel bars per ASTM A29; modified AISI 1530 with 90 ksi minimum yield strength			
SHAFT SIZE	1.25 in	32 mm	CORRODED	
			1.237 in	31.4 mm
MOMENT OF INERTIA (I)	0.20 in ⁴	8.3 cm ⁴	CORRODED	
			0.191 in ⁴	7.95 cm ⁴
SHAFT AREA (A)	1.55 in ²	10.0 cm ²	CORRODED	
			1.52 in ²	9.81 cm ²
SECTION MODULUS (S _{x-x})	0.32 in ³	5.3 cm ³	CORRODED	
			0.31 in ³	5.1 cm ³
PERIMETER	4.79 in	12.17 cm	CORRODED	
			4.74 in	12.0 cm
COUPLING	Integral forged square deep socket			
COUPLING BOLTS	One 5/8 inch diameter ASTM A325 Type 1 hex head bolt with threads excluded from shear planes			
HELIX PLATES	0.375 inch thick, formed on matching metal dies, ASTM A572 Grade 50 or better			
COATINGS	Hot dip galvanized per ASTM A153 Class B-1, 3.1 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	10 ft ⁻¹		33 m ⁻¹	
TORQUE RATING	4,000 ft-lb		5,400 N-m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	50 kip	222 kN	37.5 kip	167 kN
ALLOWABLE TENSION STRENGTH	25 kip		111 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	40 kip	178 kN	20 kip	89 kN



**ASSEMBLY OF SS125
FIGURE 7-4**

ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE SS125 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	ASD ALLOWABLE AXIAL COMPRESSION STRENGTH (kip) [kN]			
	FIRM SOIL		SOFT SOIL	
	FIXED	PINNED	FIXED	PINNED
Lead, Single Helix	See Helix Strength Table Above		8.0 [35.6]	4.1 [18.2]
Lead, Multi-Helix	32.1 [142.8]	16.4 [72.9]	8.0 [35.6]	4.1 [18.2]
Extension	32.1 [142.8]	16.4 [72.9]	8.0 [35.6]	4.1 [18.2]

Notes:

For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.



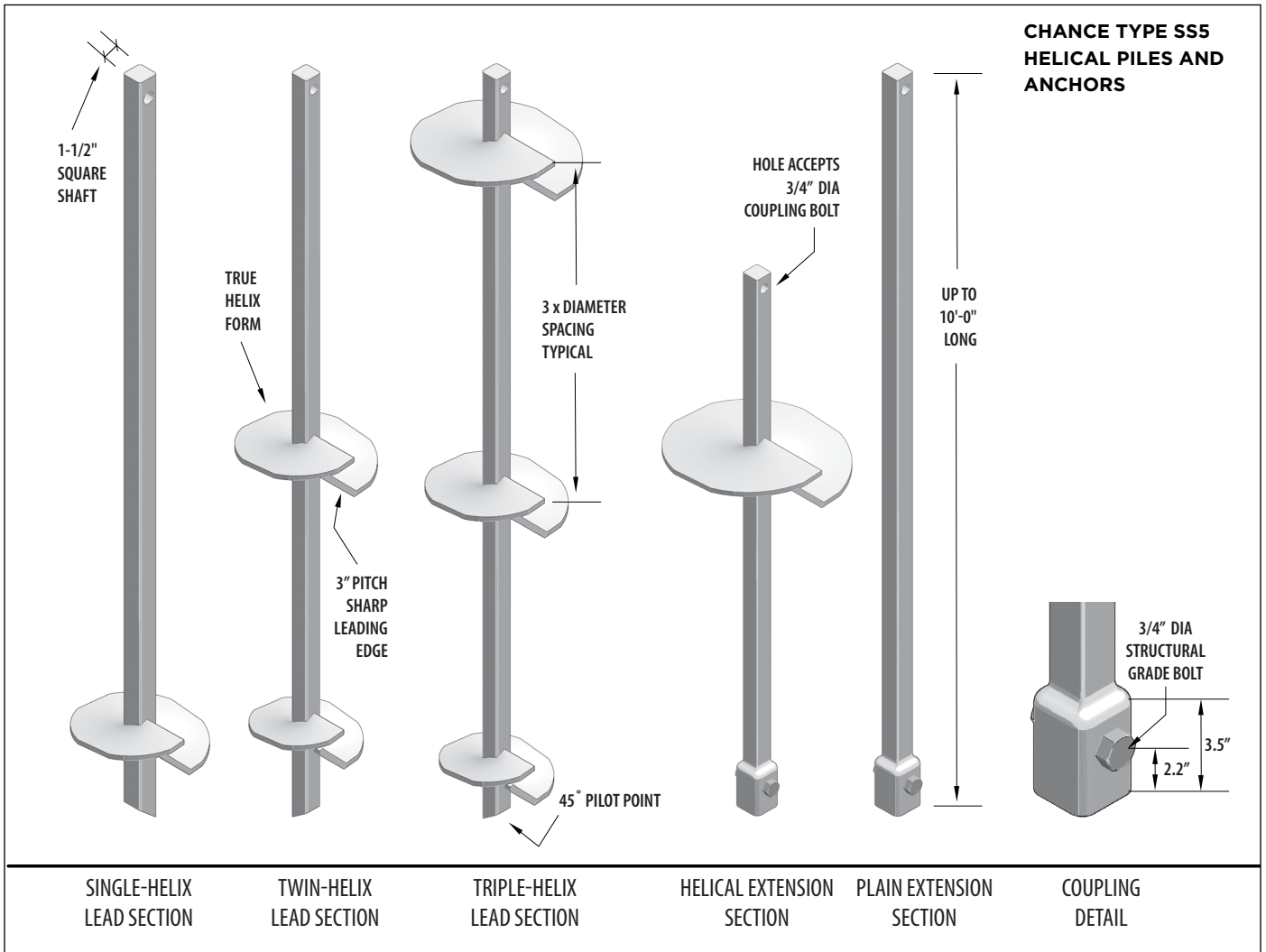
CHANCE TYPE SS5 HELICAL PILES AND ANCHORS

- 57 kip ULTIMATE – 28.5 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING – 5,700 ft-lb
- MULTI-PURPOSE 1-1/2 INCH SOLID ROUND-CORNERED-SQUARE STEEL SHAFT WITH INTEGRALLY FORMED SQUARE UPSET SOCKETS

DESCRIPTION:

Chance® Type SS5 helical piles and anchors have 57 kip ultimate capacity and 28.5 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Solid square shaft helical piles and anchors provide greater penetration into bearing soils and increased axial capacity in firm soils compared to round shaft helical piles with similar torque strength. Strength calculations are based on a design corrosion level of 50 years for most soil

conditions. Chance Type SS helical piles and anchors have a longer service life than do round shaft piles because of their reduced surface area. Chance Type SS helical piles and anchors feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.

SS5 HELICAL PILE AND ANCHOR SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: Round-cornered-square (RCS) 1-1/2 inch solid steel shaft produced exclusively for Chance® products.

COUPLING: Forged as a deep socket from the steel shaft material as an integral part of the extension, connected with structural bolts.

HELIX - 3/8 INCH THICK: ASTM A572 or A1018 or A656, with minimum yield strength of 50 ksi.

3 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for helical piles and anchors.

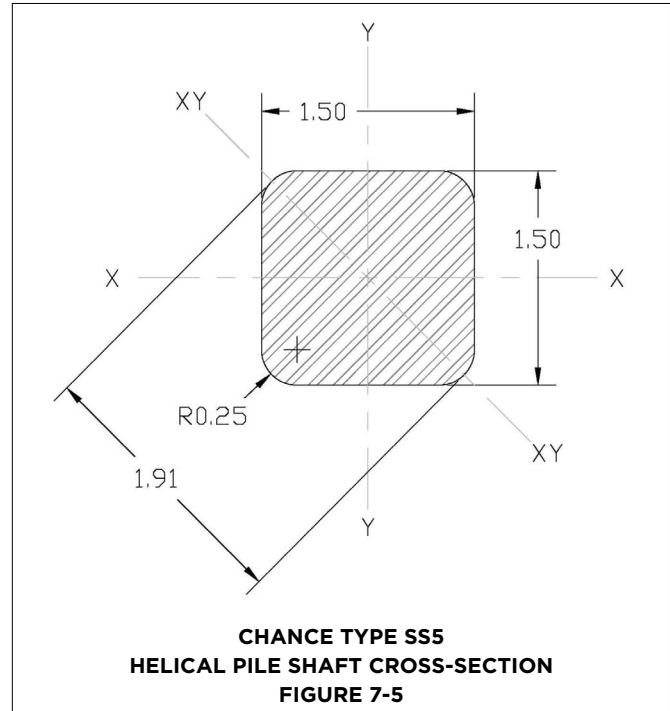
AVAILABLE HELIX DIAMETERS: 6, 8, 10, 12, 14, or 16 inches.

All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight sharpened leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

- Single-, double-, triple-, and quad-helix lead sections, 3, 3-1/2, 5, 7, and 10 feet long
- Plain extensions, 3-1/2, 5, 7, and 10 feet long
- Extensions with helix plates, 3-1/2 and 5 feet long, single helix



Helical products are hot dip galvanized per ASTM A153 Class B-1.

NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.

NOMINAL, LRFD DESIGN, & ASD ALLOWABLE STRENGTHS OF SS5 HELIX PLATES FOR SHAFT AXIAL TENSION & COMPRESSION¹

HELIX DIAMETER (in) [mm]	THICKNESS (in) [mm]	NOMINAL STRENGTH (kip) [kN]	LRFD DESIGN STRENGTH (kip) [kN]	ASD ALLOWABLE STRENGTH (kip) [kN]
6 [150]	0.375 [9.5]	57.3 [254.9]	43.0 [191.2]	28.7 [127.7]
8 [200]	0.375 [9.5]	57.3 [254.9]	43.0 [191.2]	28.7 [127.7]
10 [250]	0.375 [9.5]	47.7 [212.2]	35.8 [159.2]	23.8 [105.6]
12 [300]	0.375 [9.5]	44.2 [196.6]	33.2 [147.5]	22.1 [98.3]
14 [350]	0.375 [9.5]	54.1 [240.7]	40.6 [180.5]	27.1 [120.6]

Notes:

For SI: 1 kip = 4.448 kN.

¹ Capacities based on a design corrosion level of 50 years.

NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS OF CHANCE TYPE SS5 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS kip[kN]							
	FIRM SOIL				SOFT SOIL			
	FIXED		PINNED		FIXED		PINNED	
	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN
Lead, Single Helix	See Helix Strength Table Above		Single 6 & 8 inch - 54.4 [242.0]	Single 6 & 8 inch - 48.9 [217.5]	26.6 [118.3]	24.0 [106.8]	13.6 [60.5]	12.2 [54.3]
			For Other Helix Diameters, See Helix Strength Table Above					
Lead, Multi-Helix	89.8 [399.5]	80.8 [359.4]	54.4 [242.0]	48.9 [219.5]	26.6 [118.3]	24.0 [106.8]	13.6 [60.5]	12.2 [54.3]
Extension	89.8 [399.5]	80.8 [359.4]	54.4 [242.0]	48.9 [219.5]				

Notes:

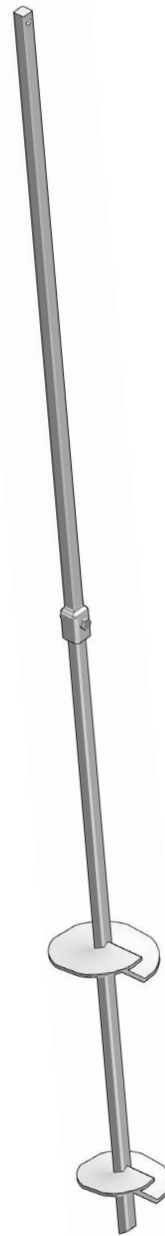
For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5, and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.



SS5 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled round-cornered-square (RCS) solid steel bars per ASTM A29; modified AISI 1044 with 70 ksi minimum yield strength			
SHAFT SIZE	1.50 in	38 mm	CORRODED	
			1.487 in	37.8 mm
MOMENT OF INERTIA (I)	0.40 in ⁴	16.5 cm ⁴	CORRODED	
			0.38 in ⁴	15.6 cm ⁴
SHAFT AREA (A)	2.2 in ²	14.2 cm ²	CORRODED	
			2.16 in ²	13.94 cm ²
SECTION MODULUS (S _{x-x})	0.53 in ³	8.7 cm ³	CORRODED	
			0.40 in ³	6.6 cm ³
PERIMETER	5.6 in	14.2 cm	CORRODED	
			5.5 in	14 cm
COUPLING	Integral forged square deep socket			
COUPLING BOLTS	One 3/4 inch diameter ASTM A193 Grade B7 hex head bolt with threads excluded from shear planes			
HELIX PLATES	0.375 inch thick, formed on matching metal dies, ASTM A572 Grade 50 or better			
COATINGS	Hot dip galvanized per ASTM A153 Class B-1, 3.1 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	10 ft ⁻¹		33 m ⁻¹	
TORQUE RATING	5,700 ft·lb		7,730 N·m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	70 kip	312 kN	52.5 kip	234 kN
ALLOWABLE TENSION STRENGTH	35 kip		156 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	57 kip	254 kN	28.5 kip	127 kN



**ASSEMBLY OF SS5
FIGURE 7-6**

ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE SS5 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	ASD ALLOWABLE AXIAL COMPRESSION STRENGTH (kip) [kN]			
	FIRM SOIL		SOFT SOIL	
	FIXED	PINNED	FIXED	PINNED
Lead, Single Helix	See Helix Strength Table Above		16 [71.2]	8.1 [36.0]
Lead, 2-Helix 8"-10"	52.5 [233.5]	32.6 [145.0]	16 [71.2]	8.1 [36.0]
Lead, 2-Helix 10"-12"	45.9 [204.2]			
Lead, 2-Helix 12"-14"	49.9 [222.0]			
Lead, 2-Helix 14"-14"	53.8 [239.3]			
Lead, Multi-Helix	53.8 [239.3]	32.6 [145.0]	16 [71.2]	8.1 [36.0]
Extension	53.8 [239.3]	32.6 [145.0]	16 [71.2]	8.1 [36.0]

Notes:

For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.

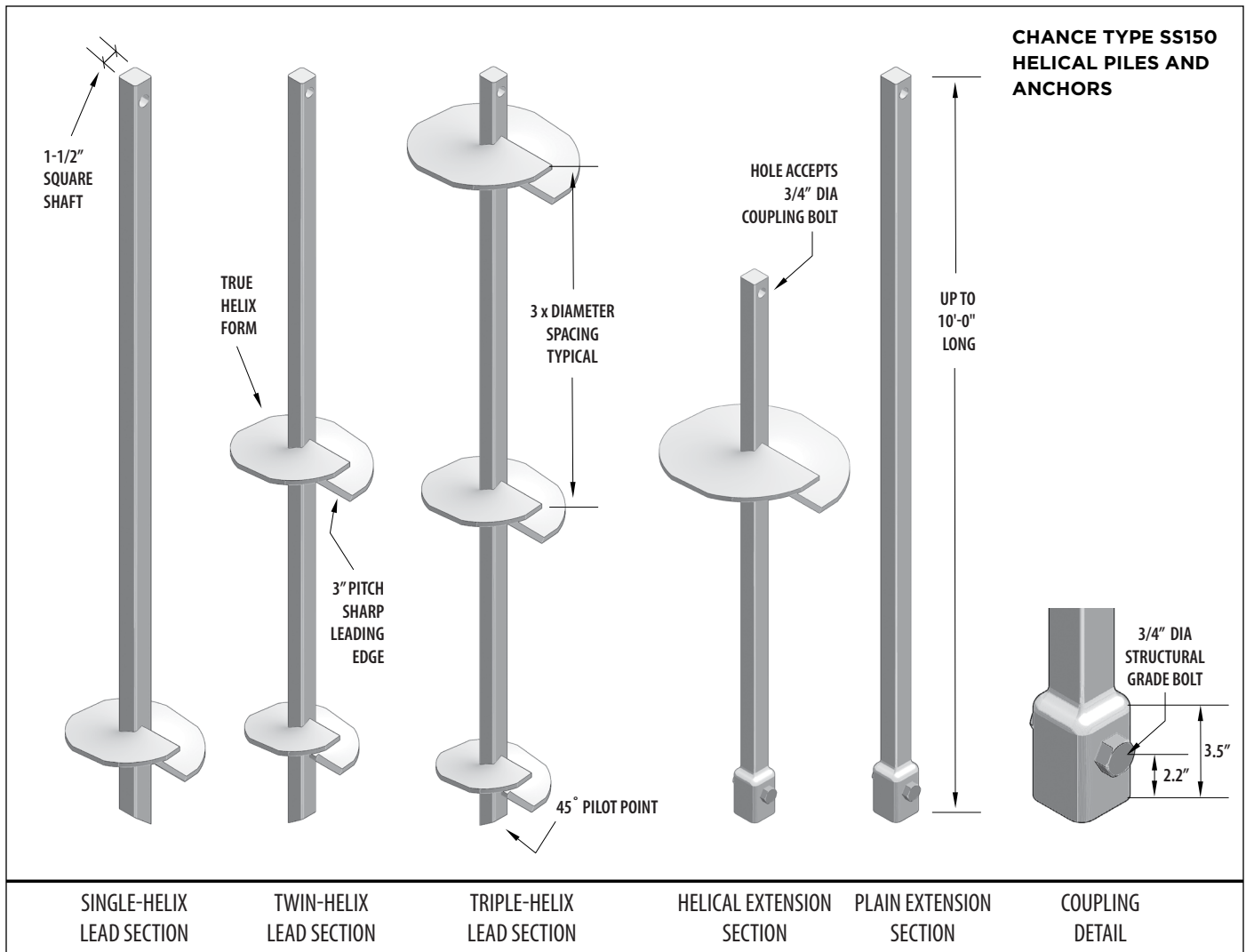
CHANCE TYPE SS150 HELICAL PILES AND ANCHORS

- 70 kip ULTIMATE – 35 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING – 7,000 ft-lb
- MULTI-PURPOSE 1-1/2 INCH SOLID ROUND-CORNERED-SQUARE STEEL SHAFT WITH INTEGRALLY FORMED SQUARE UPSET SOCKETS

DESCRIPTION:

Chance® Type SS150 helical piles and anchors have 70 kip ultimate capacity and 35 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Solid square shaft helical piles and anchors provide greater penetration into bearing soils and increased axial capacity in firm soils compared to round shaft helical piles with similar torque strength. Strength calculations are based on a design corrosion level of 50 years for most

soil conditions. Chance Type SS helical piles and anchors have a longer service life than do round shaft piles because of their reduced surface area. Chance Type SS helical piles and anchors feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.



SS150 HELICAL PILE AND ANCHOR SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: Round-cornered-square (RCS) 1-1/2 inch solid steel shaft produced exclusively for Chance® products.

COUPLING: Forged as a deep socket from the steel shaft material as an integral part of the extension, connected with structural bolts.

HELIX - 3/8 INCH THICK: ASTM A656 or A1018, with minimum yield strength of 80 ksi.

3 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 6, 8, 10, 12, and 14 inches.

All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

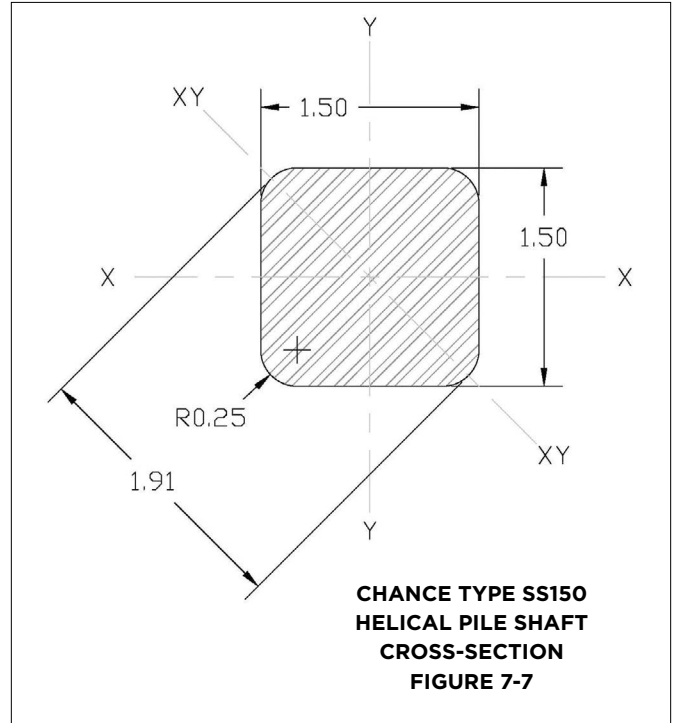
The standard helix plate has a straight sharpened leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

- Single-, double-, triple-, and quad-helix lead sections, 3, 5, 7, and 10 feet long
- Plain extensions 3-1/2, 5, 7, and 10 feet long
- Extensions with helix plates, 4, 5, 7, and 10 feet long, single and multi-helix

Helical products are hot dip galvanized per ASTM A153 Class B-1.

NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Axial Deflections of 0.25 to 0.50 inches are typical at allowable capacity.



NOMINAL, LRFD DESIGN, & ASD ALLOWABLE STRENGTHS OF SS150 HELIX PLATES FOR SHAFT AXIAL TENSION & COMPRESSION¹

HELIX DIAMETER (in) [mm]	THICKNESS (in) [mm]	NOMINAL STRENGTH (kip) [kN]	LRFD DESIGN STRENGTH (kip) [kN]	ASD ALLOWABLE STRENGTH (kip) [kN]
6 [150]	0.375 [9.5]	57.7 [257]	43.3 [192.8]	28.8 [128]
8 [200]	0.375 [9.5]	57.7 [257]	43.3 [192.8]	28.8 [128]
10 [250]	0.375 [9.5]	61.9 [275]	46.4 [206.3]	30.9 [137]
12 [300]	0.375 [9.5]	49.7 [221]	37.3 [165.8]	24.8 [110]
14 [350]	0.375 [9.5]	52.9 [235]	39.7 [176.3]	26.5 [118]

Notes:

For SI: 1 kip = 4.448 kN.

¹Capacities based on a design corrosion level of 50 years.

NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS OF CHANCE TYPE SS150 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS (kip) [kN]							
	FIRM SOIL				SOFT SOIL			
	FIXED		PINNED		FIXED		PINNED	
	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN
Lead, Single Helix	See Helix Strength Table Above		Single 6, 8, or 10 inch - 54.4 [242]	Single 6, 8, or 10 inch - 48.9 [218]	26.6 [118]	24.0 [107]	13.6 [60.5]	12.2 [54]
			For Other Helix Diameters, See Helix Strength Table Above					
Lead, Multi-Helix Extension	99.5 [443]	89.5 [398]	54.4 [242]	48.9 [218]	26.6 [118]	24.0 [107]	13.6 [60.5]	12.2 [54]

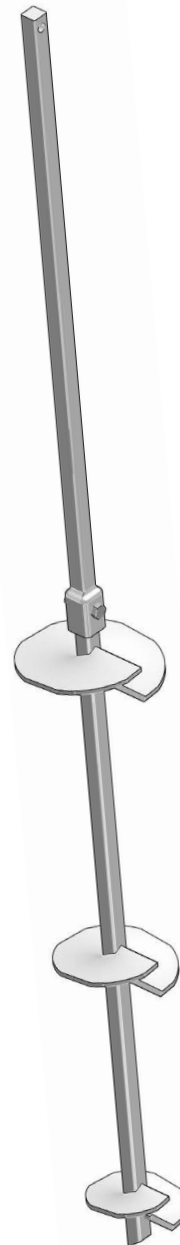
Notes:

For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.

SS150 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled round-cornered-square (RCS) solid steel bars per ASTM A29; modified AISI 1530 with 90 ksi minimum yield strength			
SHAFT SIZE	1.50 in	38 mm	CORRODED	
			1.487 in	37.8 mm
MOMENT OF INERTIA (I)	0.40 in ⁴	16.5 cm ⁴	CORRODED	
			0.38 in ⁴	15.6 cm ⁴
SHAFT AREA (A)	2.2 in ²	14.2 cm ²	CORRODED	
			2.16 in ²	13.94 cm ²
SECTION MODULUS (S _{x-x})	0.53 in ³	8.7 cm ³	CORRODED	
			0.40 in ³	6.6 cm ³
PERIMETER	5.6 in	14.2 cm	CORRODED	
			5.5 in	14.0 cm
COUPLING	Integral forged square deep socket			
COUPLING BOLTS	One 3/4 inch diameter ASTM A193 Grade B7 hex head bolt with threads excluded from shear planes			
HELIX PLATES	0.375 inch thick, formed on matching metal dies, ASTM A656 Grade 80 or better			
COATINGS	Hot dip galvanized per ASTM A153 Class B-1, 3.1 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	10 ft ⁻¹		33 m ⁻¹	
TORQUE RATING	7,000 ft·lb		9,500 N·m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	70 kip	312 kN	52.5 kip	234 kN
ALLOWABLE TENSION STRENGTH	35 kip		156 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	70 kip	312 kN	35 kip	156 kN



ASSEMBLY OF SS150
FIGURE 7-8

ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE SS150 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	ASD ALLOWABLE AXIAL COMPRESSION STRENGTH (kip) [kN]			
	FIRM SOIL		SOFT SOIL	
	FIXED	PINNED	FIXED	PINNED
Lead, Single Helix	See Helix Strength Table Above		16 [71]	8.1 [36]
Lead, 2-Helix 8"-10"	59.6 [265]	32.6 [145]	16 [71]	8.1 [36]
Lead, 2-Helix 10"-12"	55.7 [248]			
Lead, 2-Helix 12"-14"	51.3 [228]			
Lead, 2-Helix 14"-14"	53.0 [236]			
Lead, Multi-Helix	59.6 [265]			
Extension	59.6 [265]			

Notes:

For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.



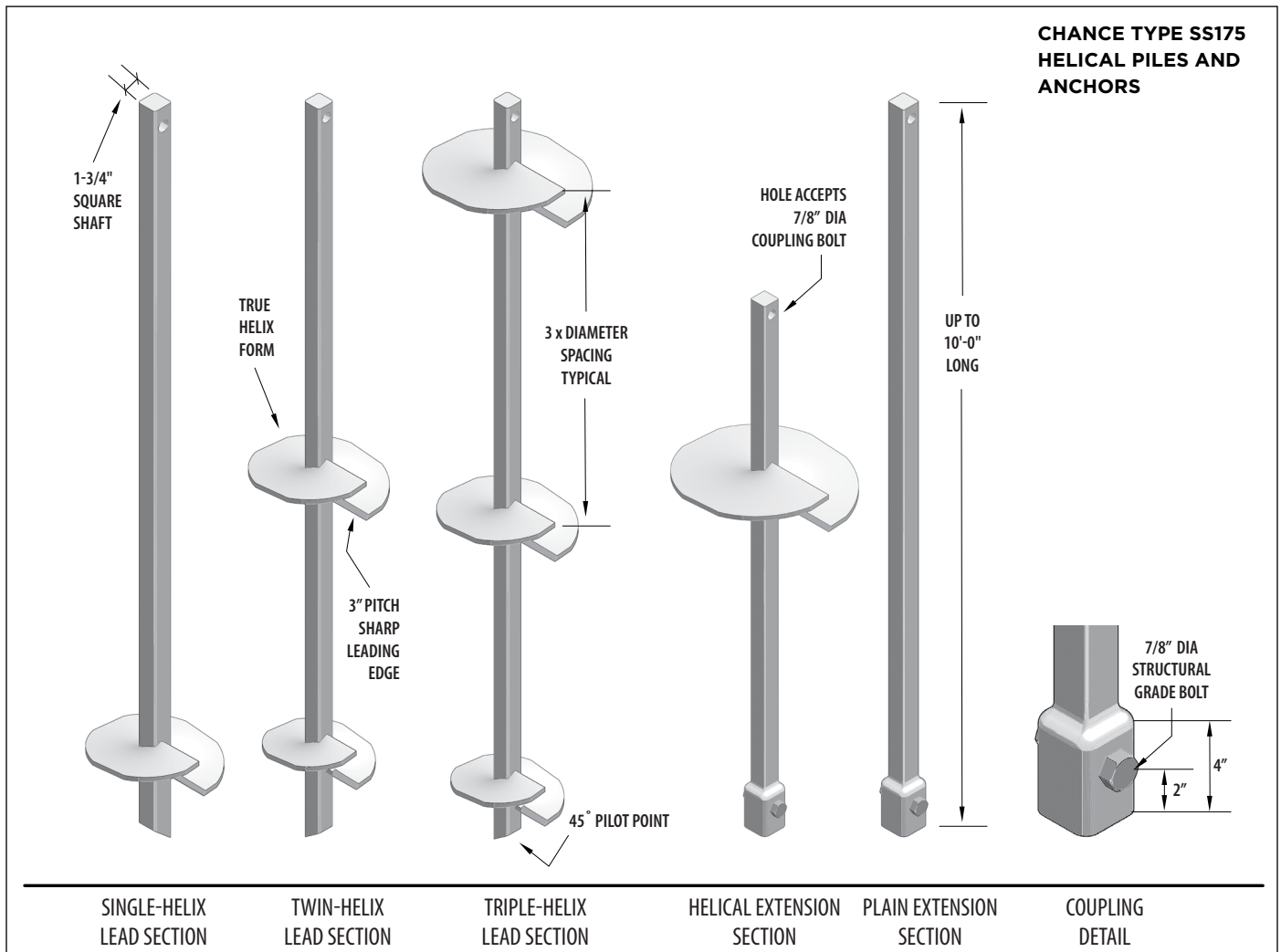
CHANCE TYPE SS175 HELICAL PILES AND ANCHORS

- 105 kip ULTIMATE - 52.5 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING - 10,500 ft-lb
- MULTI-PURPOSE 1-3/4 INCH SOLID ROUND-CORNERED-SQUARE STEEL SHAFT WITH INTEGRALLY FORMED SQUARE UPSET SOCKETS

DESCRIPTION:

Chance® Type SS175 helical piles and anchors have 105 kip ultimate capacity and 52.5 kip working or allowable capacity in compression and 100 kip ultimate capacity and 50 kip working or allowable capacity in tension. This capacity is based on structural strength ratings and well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Solid square shaft helical piles and anchors provide greater penetration into bearing soils and increased axial capacity in firm soils compared to round shaft helical piles with similar torque strength.

Strength calculations are based on a design corrosion level of 50 years for most soil conditions. Chance Type SS helical piles and anchors have a longer service life than do round shaft piles and anchors because of their reduced surface area. Chance Type SS helical piles and anchors feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.

SS175 HELICAL PILE AND ANCHOR SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: Round-cornered-square (RCS) 1-3/4 inch solid steel shaft produced exclusively for Chance® products.

COUPLING: Forged as a deep socket from the steel shaft material as an integral part of the extension, connected with structural bolts.

HELIX - 3/8 & 1/2 INCH THICK: ASTM A656 or A1018, with minimum yield strength of 80 ksi.

3 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 6, 8, 10, 12, or 14 inches.

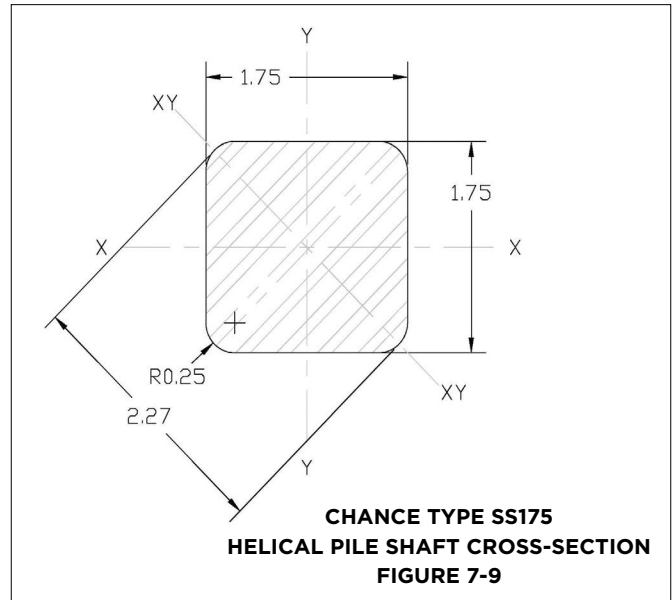
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight sharpened leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

- Single-, double-, triple-, and quad-helix lead sections, 3, 3-1/2, 5, 7, and 10 feet long
- Plain extensions, 3-1/2, 5, 7, and 10 feet long
- Extensions with helix plates, 4, 5, 7, and 10 feet long, single and multi-helix

Helical products are hot dip galvanized per ASTM A153 Class B-1.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.

NOMINAL, LRFD DESIGN, & ASD ALLOWABLE STRENGTHS OF SS175 HELIX PLATES FOR SHAFT AXIAL TENSION & COMPRESSION¹

HELIX DIAMETER (in) [mm]	THICKNESS (in) [mm]	NOMINAL STRENGTH (kip) [kN]	LRFD DESIGN STRENGTH (kip) [kN]	ASD ALLOWABLE STRENGTH (kip) [kN]
6 [150]	0.5 [13]	123.3 [548.5]	92.5 [411.4]	61.6 [274]
8 [200]	0.375 [9.5]	84.5 [375.9]	63.4 [282]	42.3 [188.2]
10 [250]	0.375 [9.5]	66.1 [294]	49.6 [220.5]	33.1 [147.2]
12 [300]	0.375 [9.5]	57.5 [255.8]	43.1 [191.9]	28.7 [127.7]
14 [350]	0.375 [9.5]	51.8 [230.4]	38.9 [172.8]	25.9 [115.2]

Notes:

For SI: 1 kip = 4.448 kN.

¹Capacities based on a design corrosion level of 50 years.

NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS OF CHANCE TYPE SS175 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS (kip) [kN]							
	FIRM SOIL				SOFT SOIL			
	FIXED		PINNED		FIXED		PINNED	
	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN
Lead, Single Helix	See Helix Strength Table Above		See Helix Strength Table Above		50.5 [224.6]	45.4 [201.9]	25.8 [114.8]	23.2 [103.2]
Lead, 2-Helix 8"-10"	164.3 [730.8]	147.8 [657.4]	103.0 [458.2]	92.7 [412.4]	50.5 [224.6]	45.4 [201.9]	25.8 [114.8]	23.2 [103.2]
Lead, 2-Helix 10"-12"	123.6 [549.8]	111.2 [494.6]						
Lead, 2-Helix 12"-14"	109.3 [486.2]	98.4 [437.7]						
Lead, 2-Helix 14"-14"	103.6 [460.8]	93.4 [415.5]						
Lead, Multi-Helix	164.3 [730.8]	147.8 [657.4]						
Extension	164.3 [730.8]	147.8 [657.4]						

Notes:

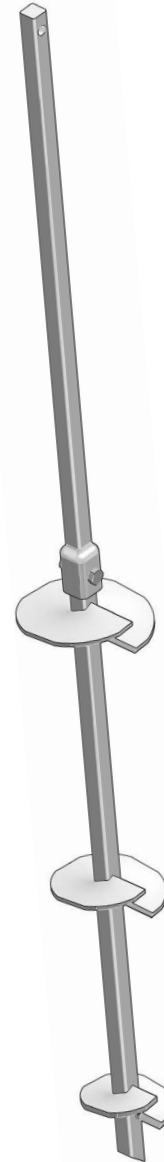
For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.



SS175 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled round-cornered-square (RCS) solid steel bars per ASTM A29; modified AISI 1530 with 90 ksi minimum yield strength			
SHAFT SIZE	1.75 in	44.4 mm	CORRODED	
			1.737 in	44 mm
MOMENT OF INERTIA (I)	0.75 in ⁴	31.1 cm ⁴	CORRODED	
			0.725 in ⁴	30.1 cm ⁴
SHAFT AREA (A)	3.1 in ²	19.4 cm ²	CORRODED	
			2.97 in ²	19.16 cm ²
SECTION MODULUS (S _{x-x})	0.85 in ³	13.9 cm ³	CORRODED	
			0.835 in ³	13.65 cm ³
PERIMETER	6.6 in	16.7 cm	CORRODED	
			6.5 in	16.5 cm
COUPLING	Integral forged square deep socket			
COUPLING BOLTS	One 7/8 inch diameter ASTM A193 Grade B7 hex head bolt with threads excluded from shear planes			
HELIX PLATES	0.375 & 0.5 inch thick, formed on matching metal dies, ASTM A656 Grade 80 or better			
COATINGS	Hot dip galvanized per ASTM A153 Class B-1, 3.1 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	10 ft ⁻¹		33 m ⁻¹	
TORQUE RATING	10,500 ft-lb		14,240 N-m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	100 kip	445 kN	75 kip	334 kN
ALLOWABLE TENSION STRENGTH	50 kip		222 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	105 kip	467 kN	52.5 kip	234 kN



ASSEMBLY OF SS175
FIGURE 7-10

ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE SS175 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	ASD ALLOWABLE AXIAL COMPRESSION STRENGTH (kip) [kN]			
	FIRM SOIL		SOFT SOIL	
	FIXED	PINNED	FIXED	PINNED
Lead, Single Helix	See Helix Strength Table Above	See Helix Strength Table Above	30.2 [134.3]	15.4 [68.5]
Lead, Single 12" Helix			28.7 [127.7]	
Lead, Single 14" Helix			25.9 [115.2]	
Lead, 2-Helix 8"-10"	94.7 [421.2]	61.7 [274.5]	30.2 [134.3]	15.4 [68.5]
Lead, 2-Helix 10"-12"	61.8 [274.9]	61.7 [274.5]		
Lead, 2-Helix 12"-14"	54.6 [242.9]	54.6 [242.9]		
Lead, 2-Helix 14"-14"	51.8 [230.4]	51.8 [230.4]		
Lead, Multi-Helix	98.4 [437.7]	61.7 [274.5]	30.2 [134.3]	15.4 [68.5]
Extension	98.4 [437.7]	61.7 [274.5]	30.2 [134.3]	15.4 [68.5]

Notes:

For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.

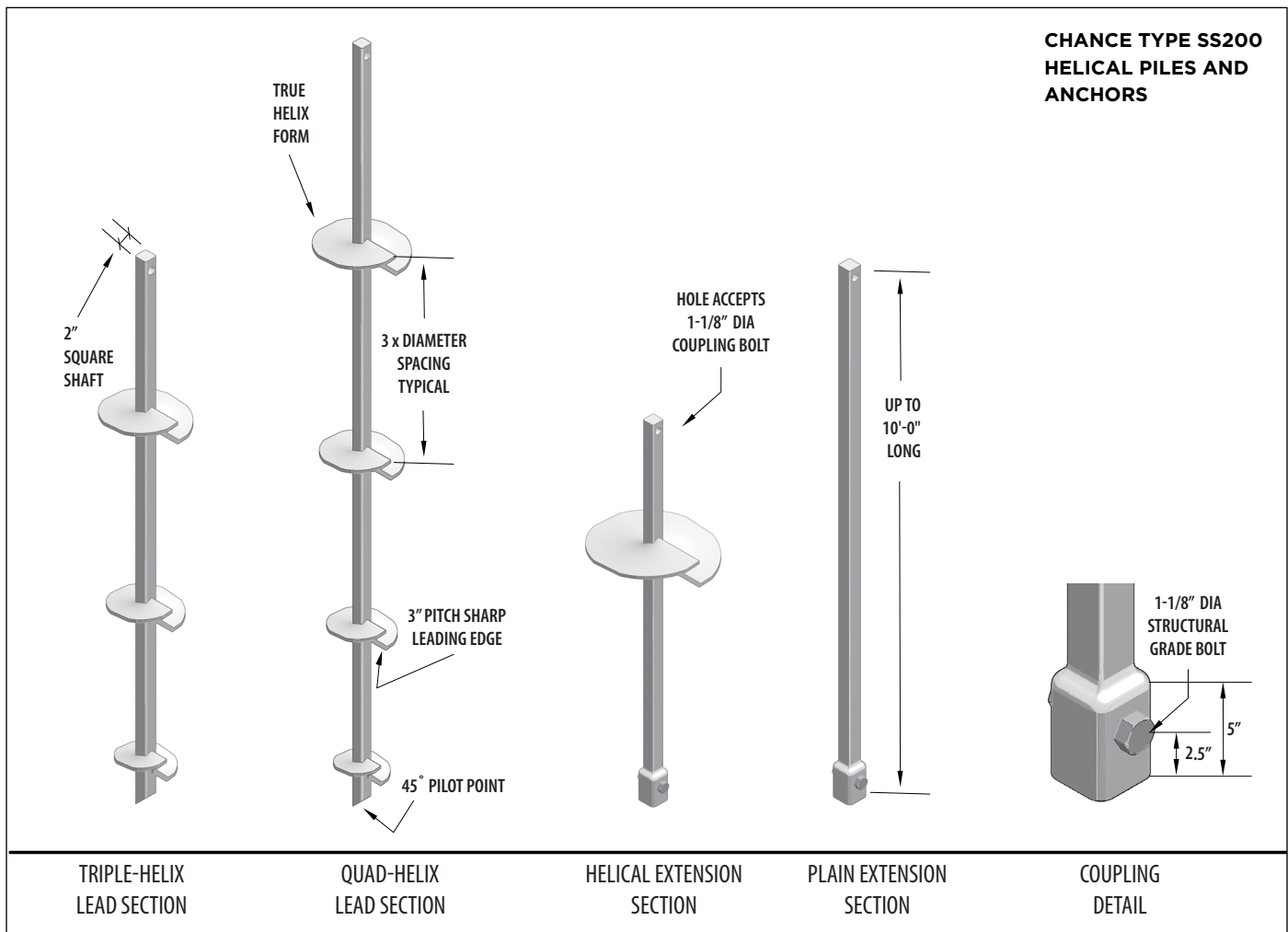
CHANCE TYPE SS200 HELICAL PILES AND ANCHORS

- 160 kip ULTIMATE - 80 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING - 16,000 ft·lb
- MULTI-PURPOSE 2 INCH SOLID ROUND-CORNERED-SQUARE STEEL SHAFT WITH INTEGRALLY FORMED SQUARE UPSET SOCKETS

DESCRIPTION:

Chance® Type SS200 helical piles and anchors have 160 kip ultimate capacity and 80 kip working or allowable capacity in compression and 150 kip ultimate capacity and 75 kip working or allowable capacity in tension. This capacity is based on structural strength ratings and well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Solid square shaft helical piles and anchors provide greater penetration into bearing soils and increased axial capacity in firm soils compared to round shaft helical piles with similar torque strength.

Strength calculations are based on a design corrosion level of 50 years for most soil conditions. Chance Type SS helical piles and anchors have a longer service life than do round shaft piles because of their reduced surface area. Chance Type SS helical piles and anchors feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.



SS200 HELICAL PILE AND ANCHOR SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: Round-cornered-square (RCS) 2 inch solid steel shaft produced exclusively for Chance® products.

COUPLING: Forged as a deep socket from the steel shaft material as an integral part of the extension, connected with structural bolts.

HELIX - 1/2 INCH THICK: ASTM A656 or A1018, with minimum yield strength of 80 ksi.

3 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 6, 8, 10, 12, and 14 inches.

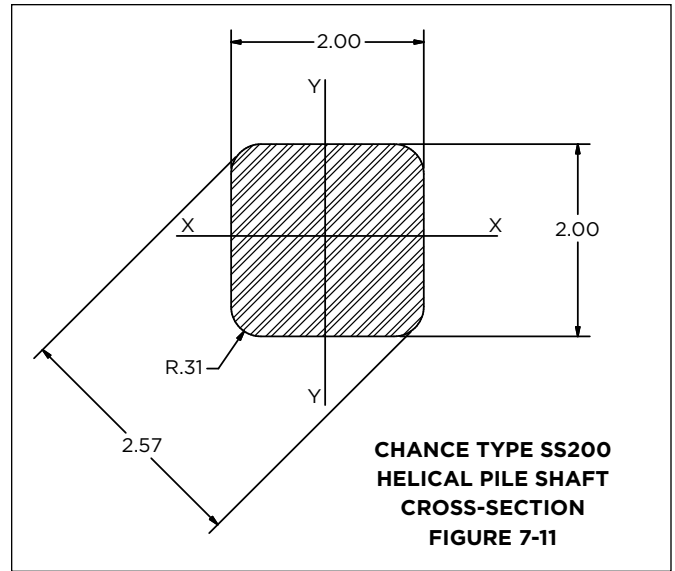
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight sharpened leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

- Triple- and quad-helix lead sections, 5, 7, & 10 feet long
- Plain extensions, 3-1/2, 5, 7, and 10 feet long
- Extensions with helix plates, 4, 7, and 10 feet long, single and multi-helix

Helical products are hot dip galvanized per ASTM A153 Class B-1.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Axial deflections of 0.25 to 0.50 inches are typical at allowable capacity.

NOMINAL, LRFD DESIGN, & ASD ALLOWABLE STRENGTHS OF SS200 HELIX PLATES FOR SHAFT AXIAL TENSION & COMPRESSION¹

HELIX DIAMETER (in) [mm]	THICKNESS (in) [mm]	NOMINAL STRENGTH (kip) [kN]	LRFD DESIGN STRENGTH (kip) [kN]	ASD ALLOWABLE STRENGTH (kip) [kN]
6 [150]	0.5 [13]	154 [685]	115.5 [513.8]	77 [342.5]
8 [200]	0.5 [13]	154 [685]	115.5 [513.8]	77 [342.5]
10 [250]	0.5 [13]	122.8 [546.2]	92.1 [409.7]	61.4 [273.1]
12 [300]	0.5 [13]	131.3 [584]	98.5 [438]	65.6 [291.8]
14 [350]	0.5 [13]	115.3 [512.9]	86.5 [384.7]	57.6 [256.2]

Notes:

For SI: 1 kip = 4.448 kN.

¹Capacities based on a design corrosion level of 50 years.

NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS OF CHANCE TYPE SS200 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS (kip) [kN]							
	FIRM SOIL				SOFT SOIL			
	FIXED		PINNED		FIXED		PINNED	
	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN
Lead, Single Helix	See Helix Strength Table Above		See Helix Strength Table Above		85.6 [380.8]	77.1 [342.9]	43.7 [194.4]	39.3 [174.8]
Lead, 2-Helix 8"-10"	239.6 [1065.8]	215.6 [959]	167.5 [745]	150.8 [670.8]	86.6 [385.2]	77.1 [342.9]	43.7 [194.4]	39.3 [174.8]
Lead, 2-Helix 10"-12"	239.6 [1065.8]	215.6 [959]						
Lead, 2-Helix 12"-14"	239.6 [1065.8]	215.6 [959]						
Lead, 2-Helix 14"-14"	230.6 [1025.8]	207.6 [923.5]						
Lead, Multi-Helix	239.6 [1065.8]	215.6 [959]						
Extension	239.6 [1065.8]	215.6 [959]						

Notes:

For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.

SS200 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled round-cornered-square (RCS) solid steel bars per ASTM A29; modified AISI 1530 with 90 ksi minimum yield strength			
SHAFT SIZE	2 in	51 mm	CORRODED	
			1.971 in	50 mm
MOMENT OF INERTIA (I)	1.26 in ⁴	52.4 cm ⁴	CORRODED	
			1.19 in ⁴	49.53 cm ⁴
SHAFT AREA (A)	3.9 in ²	25.3 cm ²	CORRODED	
			3.81 in ²	24.58 cm ²
SECTION MODULUS (S _{x-x})	1.26 in ³	20.6 cm ³	CORRODED	
			1.21 in ³	19.83 cm ³
PERIMETER	7.5 in	18.9 cm	CORRODED	
			7.36 in	18.69 cm
COUPLING	Integral forged square deep socket			
COUPLING BOLTS	One 1-1/8 inch diameter ASTM A193 Grade B7 hex head bolt with threads excluded from shear planes			
HELIX PLATES	0.5 inch thick, formed on matching metal dies, ASTM A656 or A1018 Grade 80			
COATINGS	Hot dip galvanized per ASTM A153 Class B-1, 3.1 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	10 ft ⁻¹		33 m ⁻¹	
TORQUE RATING	16,000 ft-lb		21,700 N-m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	150 kip	668 kN	112.5 kip	500 kN
ALLOWABLE TENSION STRENGTH	75 kip		334 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	160 kip	712 kN	80 kip	356 kN



ASSEMBLY OF SS200
FIGURE 7-12

ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE SS200 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	ASD ALLOWABLE AXIAL COMPRESSION STRENGTH (kip) [kN]			
	FIRM SOIL		SOFT SOIL	
	FIXED	PINNED	FIXED	PINNED
Lead, Single Helix	See Helix Strength Table Above		See Helix Strength Table Above	
Lead, 2-Helix 8"-10"	138.4 [615.6]	100.3 [446.1]	51.3 [228.2]	26.2 [116.5]
Lead, 2-Helix 10"-12"	127.0 [765.1]			
Lead, 2-Helix 12"-14"	123.2 [548]			
Lead, 2-Helix 14"-14"	115.2 [512.4]			
Lead, Multi-Helix	143.5 [638.3]	100.3 [446.1]	51.3 [228.2]	26.2 [116.5]
Extension	143.5 [638.3]	100.3 [446.1]	51.3 [228.2]	26.2 [116.5]
Notes:				
For SI: 1 kip = 4.448 kN.				
¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.				
² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.				



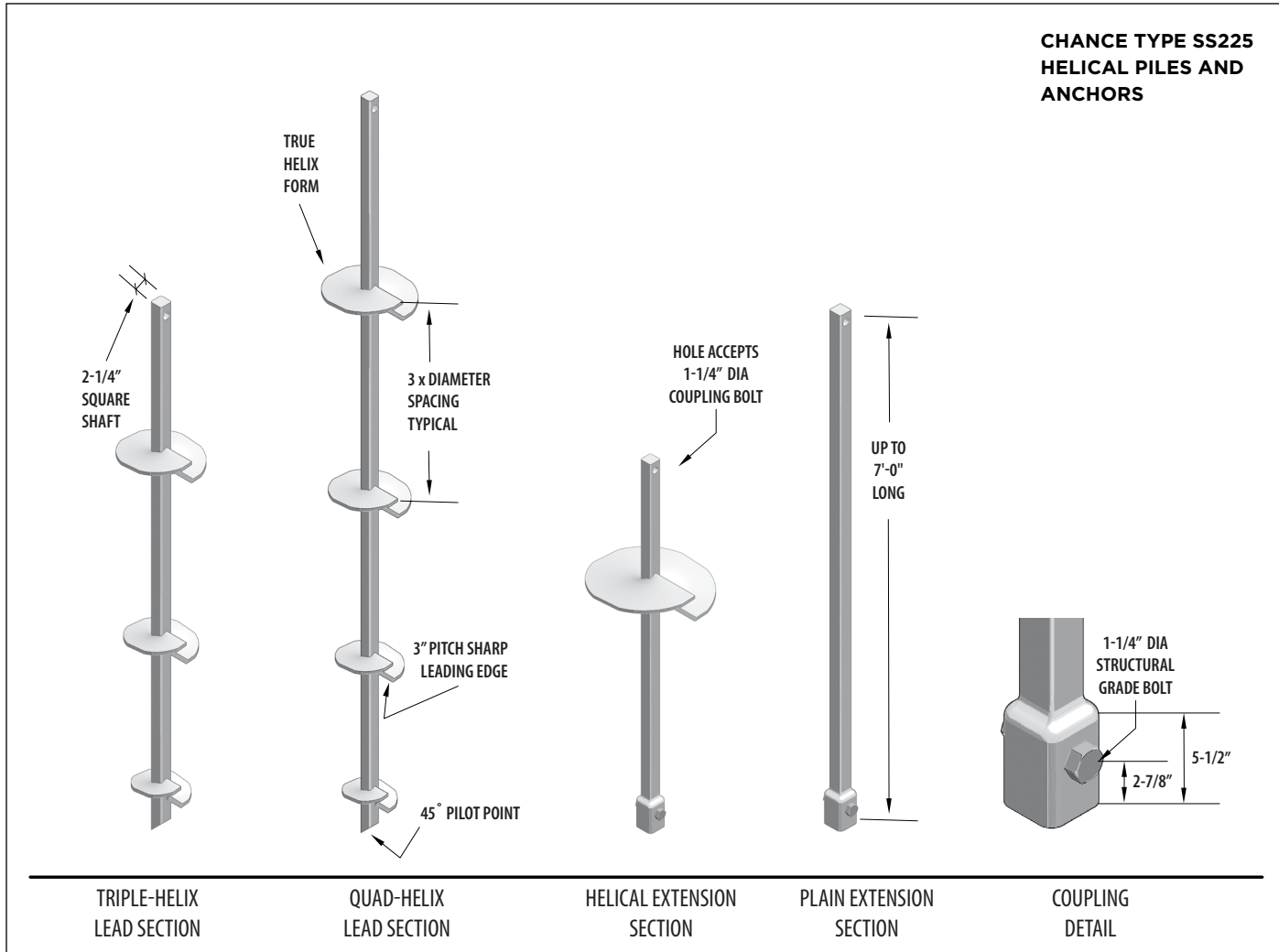
CHANCE TYPE SS225 HELICAL PILES AND ANCHORS

- 210 kip ULTIMATE - 105 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING - 21,000 ft-lb
- MULTI-PURPOSE 2-1/4 INCH SOLID ROUND-CORNERED-SQUARE STEEL SHAFT WITH INTEGRALLY FORMED SQUARE UPSET SOCKETS

DESCRIPTION:

Chance® Type SS225 helical piles and anchors have 210 kip ultimate capacity and 105 kip working or allowable capacity in compression and 200 kip ultimate capacity and 100 kip working or allowable capacity in tension. This capacity is based on structural strength ratings and well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Solid square shaft helical piles and anchors provide greater penetration into bearing soils and increased axial capacity in firm soils compared to round shaft helical piles with similar

torque strength. Strength calculations are based on a design corrosion level of 50 years for most soil conditions. Chance Type SS helical piles and anchors have a longer service life than do round shaft piles because of their reduced surface area. Chance Type SS helical piles and anchors feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.

SS225 HELICAL PILE AND ANCHOR SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: Round-cornered-square (RCS) 2-1/4 inch solid steel shaft produced exclusively for Chance® products.

COUPLING: Forged as a deep socket from the steel shaft material as an integral part of the extension, connected with structural bolts.

HELIX - 1/2 INCH THICK: ASTM A656 or A1018, with minimum yield strength of 80 ksi.

3 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 6, 8, 10, 12, and 14 inches.

All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight sharpened leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

- Triple- and quad-helix lead sections, 5, 6-1/2, and 10 feet long
- Plain extensions, 3-1/2, 5, and 7 feet long
- Extensions with helix plates, 3-1/2 and 7 feet long, single and multi-helix

Helical products are hot dip galvanized per ASTM A153 Class B-1.

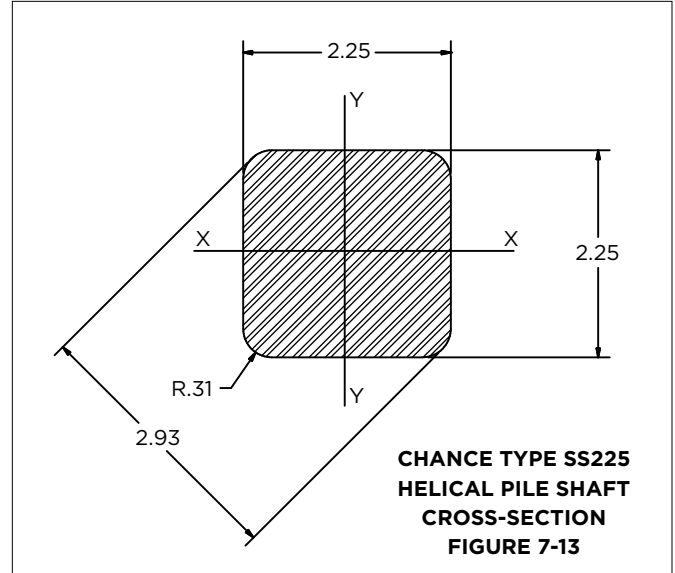
NOMINAL, LRFD DESIGN, & ASD ALLOWABLE STRENGTHS OF SS225 HELIX PLATES FOR SHAFT AXIAL TENSION & COMPRESSION¹

HELIX DIAMETER (in) [mm]	THICKNESS (in) [mm]	NOMINAL STRENGTH (kip) [kN]	LRFD DESIGN STRENGTH (kip) [kN]	ASD ALLOWABLE STRENGTH (kip) [kN]
6 [150]	0.5 [13]	188 [836.3]	141 [627.2]	94 [418.1]
8 [200]	0.5 [13]	188 [836.3]	141 [627.2]	94 [418.1]
10 [250]	0.5 [13]	151.8 [675.2]	113.9 [506.4]	75.9 [337.6]
12 [300]	0.5 [13]	141.3 [628.5]	106 [471.4]	70.6 [314]
14 [350]	0.5 [13]	126.3 [561.8]	94.7 [421.4]	63.2 [281.1]

Notes:

For SI: 1 kip = 4.448 kN.

¹Capacities based on a design corrosion level of 50 years.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Axial deflections of 0.25 to 0.50 inches are typical at allowable capacity.

NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS OF CHANCE TYPE SS225 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS (kip) [kN]							
	FIRM SOIL				SOFT SOIL			
	FIXED		PINNED		FIXED		PINNED	
	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN
Lead, Single Helix	See Helix Strength Table Above		See Helix Strength Table Above		139.0 [618.3]	125.1 [556.5]	70.9 [315.4]	63.8 [283.8]
					Single 14 inch - 126.3 [561.8]	Single 14 inch - 113.7 [505.8]		
Lead, 2-Helix 8"-10"	331.6 [1475]	298.4 [1327.3]	250.1 [1112.5]	225.1 [1001.3]	139.0 [618.3]	125.1 [556.5]	70.9 [315.4]	63.8 [283.8]
Lead, 2-Helix 10"-12"	293.1 [1303.8]	263.8 [1173.4]						
Lead, 2-Helix 12"-14"	267.6 [1190.3]	240.9 [1071.6]						
Lead, 2-Helix 14"-14"	252.6 [1123.6]	227.4 [1011.5]						
Lead, Multi-Helix	331.6 [1475]	298.4 [1327.3]						
Extension	331.6 [1475]	298.4 [1327.3]						

Notes:

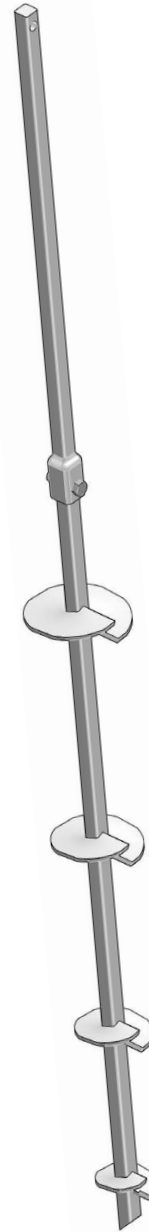
For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.



SS225 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled round-cornered-square (RCS) solid steel bars per ASTM A29; modified AISI 1530 with 90 ksi minimum yield strength			
SHAFT SIZE	2.25 in	57 mm	CORRODED	
			2.237 in	56.8 mm
MOMENT OF INERTIA (I)	2.04 in ⁴	84.9 cm ⁴	CORRODED	
			1.99 in ⁴	82.83 cm ⁴
SHAFT AREA (A)	5.0 in ²	32.1 cm ²	CORRODED	
			4.93 in ²	31.81 cm ²
SECTION MODULUS (S _{x-x})	1.81 in ³	29.7 cm ³	CORRODED	
			1.79 in ³	29.37 cm ³
PERIMETER	8.5 in	21.5 cm	CORRODED	
			8.43 in	21.41 cm
COUPLING	Integral forged square deep socket			
COUPLING BOLTS	One 1-1/4 inch diameter ASTM A193 Grade B7 hex head bolt with threads excluded from shear planes			
HELIX PLATES	0.5 inch thick, formed on matching metal dies, ASTM A656 or A1018 Grade 80			
COATINGS	Hot dip galvanized per ASTM A153 Class B-1, 3.1 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	10 ft ⁻¹		33 m ⁻¹	
TORQUE RATING	21,000 ft·lb		28,475 N·m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	200 kip	890 kN	150 kip	667 kN
ALLOWABLE TENSION STRENGTH	100 kip		445 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	210 kip	934 kN	105 kip	467 kN



**ASSEMBLY OF SS225
FIGURE 7-14**

ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE SS225 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	ASD ALLOWABLE AXIAL COMPRESSION STRENGTH (kip) [kN]			
	FIRM SOIL		SOFT SOIL	
	FIXED	PINNED	FIXED	PINNED
Lead, Single Helix	See Helix Strength Table Above	See Helix Strength Table Above	See Helix Strength Table Above, except single 6 & 8 inch - 83.2 [370.1]	42.5 [189]
Lead, 2-Helix 8"-10"	169.9 [755.8]	149.8 [666.3]	83.2 [370.1]	42.5 [189]
Lead, 2-Helix 10"-12"	146.5 [651.6]	146.5 [650.7]		
Lead, 2-Helix 12"-14"	133.8 [595.1]	133.8 [595.1]		
Lead, 2-Helix 14"-14"	126.4 [562.2]	126.4 [562.3]		
Lead, Multi-Helix	198.6 [883.4]	149.8 [666.3]	83.2 [370.1]	42.5 [189]
Extension	198.6 [883.4]	149.8 [666.3]	83.2 [370.1]	42.5 [189]

Notes:

For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.

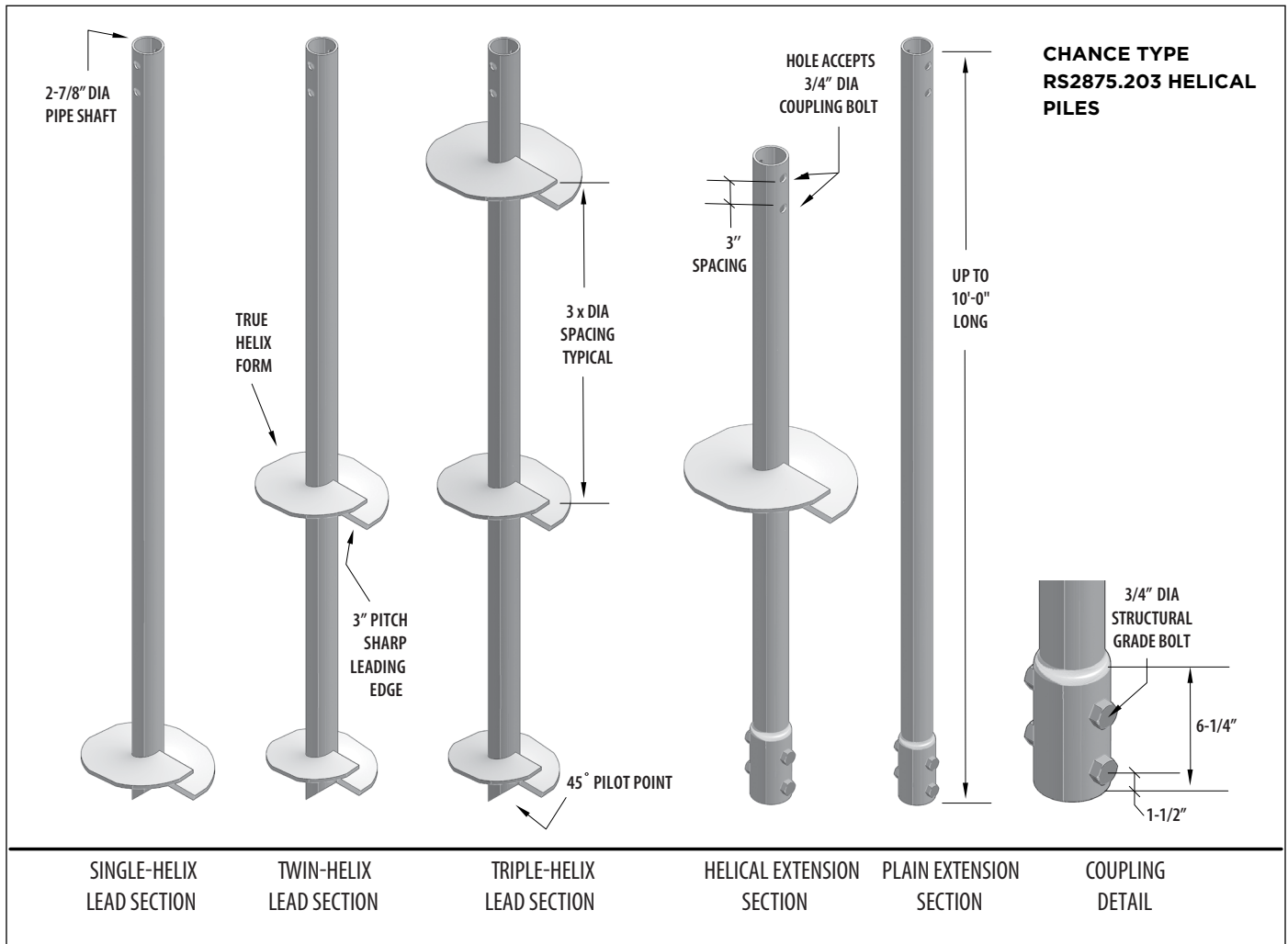
CHANCE TYPE RS2875.203 HELICAL PILES

- 63 kip ULTIMATE – 31.5 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING – 7,000 ft-lb
- MULTI-PURPOSE 2-7/8" DIAMETER, 0.203" WALL, ROUND HSS SHAFT WITH INTEGRALLY FORMED UPSET SOCKETS

DESCRIPTION:

Chance® Type RS2875.203 helical piles have 63 kip ultimate capacity and 31.5 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Round shaft helical piles offer increased lateral and buckling resistance compared to solid square shafts with similar torque strength. Strength calculations are based on a design corrosion level of 50 years for most soil conditions.

Chance Type RS helical piles can be coupled with square shaft lead sections (Combo Piles) to provide greater penetration into bearing soils. Chance Type RS helical piles feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.



RS2875.203 HELICAL PILE SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: HSS 2-7/8 inch OD x 0.203 inch (schedule 40) wall steel shaft produced exclusively for Chance® products.

COUPLING: Forged as an integral part of the plain and helical extension material as round deep sockets, connected with multiple structural bolts.

HELIX - 3/8 INCH THICK: ASTM A572 or A1018 or A656, with minimum yield strength of 50 ksi.

3 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 8, 10, 12, or 14 inches.

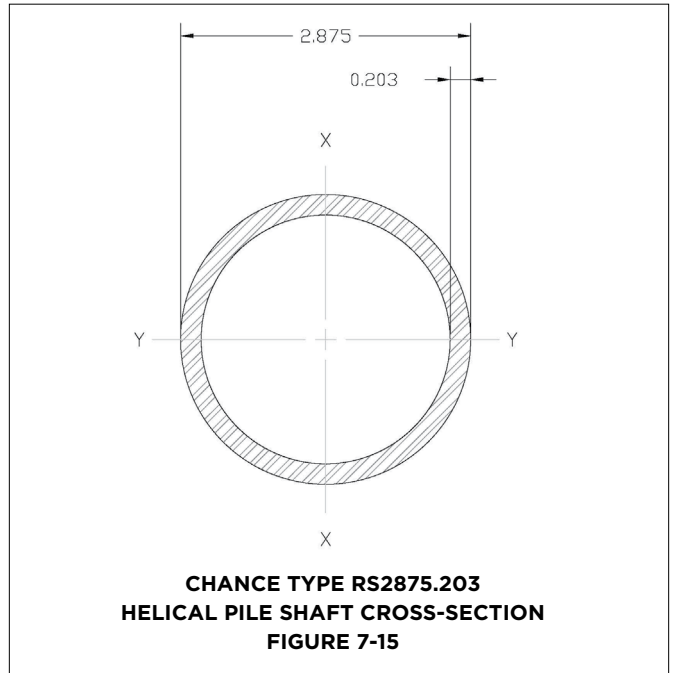
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight sharpened leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

- Single-, double-, and triple-helix lead sections, 5, 7, and 10 feet long
- Plain extensions, 3-1/2, 5, 7, and 10 feet long
- Extensions with helix plates, 5 and 7 feet long

Helical products are hot dip galvanized per ASTM A153 Class B-1.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.

NOMINAL, LRFD DESIGN, & ASD ALLOWABLE STRENGTHS OF RS2875.203 HELIX PLATES FOR SHAFT AXIAL TENSION & COMPRESSION¹

HELIX DIAMETER (in) [mm]	THICKNESS (in) [mm]	NOMINAL STRENGTH (kip) [kN]	LRFD DESIGN STRENGTH (kip) [kN]	ASD ALLOWABLE STRENGTH (kip) [kN]
8 [200]	0.375 [9.5]	85.8 [381.7]	64.4 [286.3]	42.9 [190.8]
10 [250]	0.375 [9.5]	73.6 [327.4]	55.2 [245.6]	36.8 [163.7]
12 [300]	0.375 [9.5]	75.6 [336.3]	56.7 [252.2]	37.8 [168.1]
14 [350]	0.375 [9.5]	61.0 [271.3]	45.8 [203.5]	30.5 [135.7]

Notes:

For SI: 1 kip = 4.448 kN.

¹Capacities based on a design corrosion level of 50 years.

NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS OF CHANCE TYPE RS2875.203 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS (kip) [kN]							
	FIRM SOIL				SOFT SOIL			
	FIXED		PINNED		FIXED		PINNED	
	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN
Lead, Single Helix	69.0 [306.9]	62.1 [276.2]	64.3 [286.0]	57.9 [257.6]	55.5 [246.9]	49.9 [222.0]	42.0 [186.8]	37.8 [168.1]
	For Single 14"- 61 [271.3]	For Single 14"- 54.9 [244.2]	For Single 14"- 61.0 [271.3]	For Single 14"- 57.9 [257.6]				
Lead, Multi-Helix	69.0 [306.9]	62.1 [276.2]	64.3 [286.0]	57.9 [257.6]	55.5 [246.9]	49.9 [222.0]	42.0 [186.8]	37.8 [168.1]
Extension	69.0 [306.9]	62.1 [276.2]	64.3 [286.0]	57.9 [257.6]	55.5 [246.9]	49.9 [222.0]	42.0 [186.8]	37.8 [168.1]

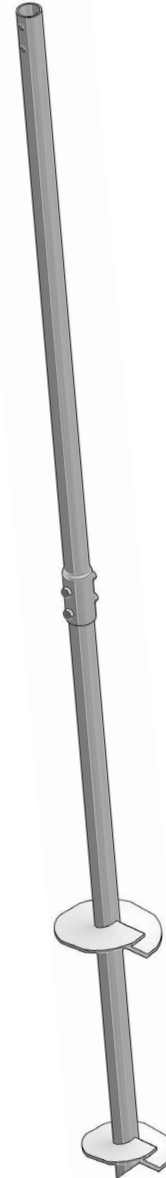
Notes:

For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.

RS2875.203 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled HSS 2-1/2 inch nominal Schedule 40 (0.203 inch nominal wall) per ASTM A500 Grade B/C with 65 ksi minimum yield strength			
SHAFT SIZE, OD	2.875 in	73 mm	CORRODED	
			2.862 in	72.7 mm
SHAFT SIZE, ID*	2.497 in	63.4 mm	CORRODED	
			2.510 in	63.75 mm
MOMENT OF INERTIA (I)*	1.44 in ⁴	59.9 cm ⁴	CORRODED	
			1.344 in ⁴	55.9 cm ⁴
SHAFT AREA (A)*	1.59 in ²	10.3 cm ²	CORRODED	
			1.48 in ²	9.57 cm ²
SECTION MODULUS (S _{x-x})*	1.0 in ³	16.4 cm ³	CORRODED	
			0.939 in ³	15.4 cm ³
PERIMETER	9.0 in	22.8 cm	CORRODED	
			8.99 in	22.8 cm
COUPLING	Integral forged round deep socket sleeve			
COUPLING BOLTS	Two 3/4 inch diameter SAE J429 Grade 5 hex head bolts with threads excluded from shear planes			
HELIX PLATES	0.375 inch thick, formed on matching metal dies, ASTM A572 Grade 50 or better			
COATINGS	Hot dip galvanized per ASTM A153 Class B-1, 3.1 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	9 ft ⁻¹		30 m ⁻¹	
TORQUE RATING	7,000 ft-lb		9,491 N-m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	60 kip	267 kN	45 kip	200 kN
ALLOWABLE TENSION STRENGTH	30 kip		133 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	63 kip	280 kN	31.5 kip	140 kN
Note: * Computed with 93% of wall thickness per AISC 360-10, B4.2				



ASSEMBLY OF RS2875.203
FIGURE 7-16

ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE RS2875.203 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	ASD ALLOWABLE AXIAL COMPRESSION STRENGTH (kip) [kN]			
	FIRM SOIL		SOFT SOIL	
	FIXED	PINNED	FIXED	PINNED
Lead, Single Helix	For Single 8" - 41.3 [183.7]	For Single 8" - 38.5 [171.3]	33.2 [147.7]	25.1 [111.7]
	See Helix Strength Table Above for 10", 12", & 14"	See Helix Strength Table Above for 10", 12", & 14"	For Single 14" - 30.5 [135.7]	
Lead, 2-Helix 8"-10"	41.3 [183.7]	38.5 [171.3]	33.2 [147.7]	25.1 [111.7]
Lead, 2-Helix 10"-12"				
Lead, 2-Helix 12"-14"				
Lead, 2-Helix 14"-14"				
Lead, Multi-Helix	41.3 [183.7]	38.5 [171.3]	33.2 [147.7]	25.1 [111.7]
Extension	41.3 [183.7]	38.5 [171.3]	33.2 [147.7]	25.1 [111.7]
Note: For SI: 1 kip = 4.448 kN. 1 Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil. 2 Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.				



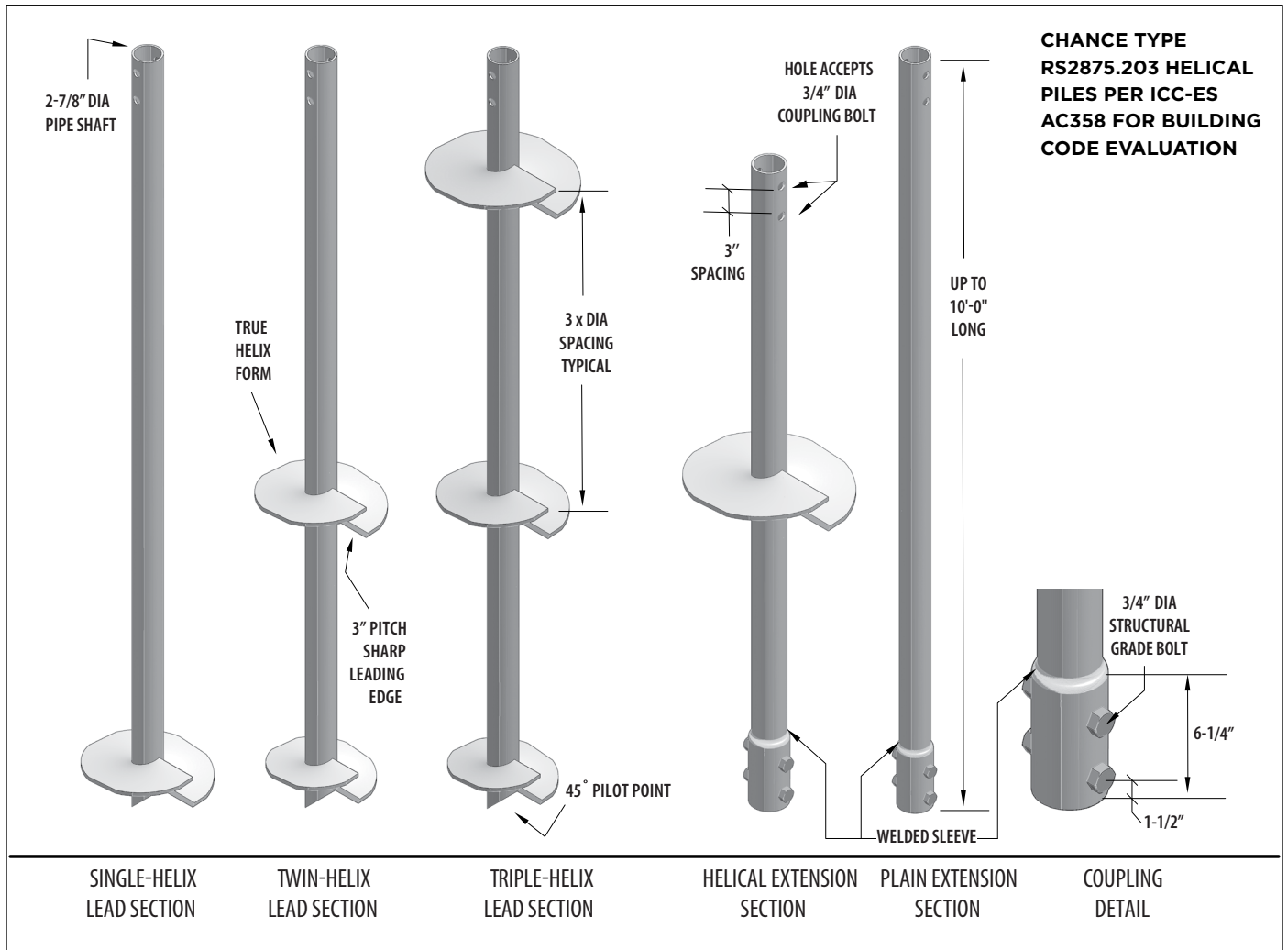
CHANCE TYPE RS2875.203 HELICAL PILES PER ICC-ES AC358 FOR BUILDING CODE EVALUATION

- 60.4 kip ULTIMATE - 30.2 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING - 6,710 ft-lb
- MULTI-PURPOSE 2-7/8" DIAMETER, 0.203" WALL, ROUND HSS SHAFT WITH SLEEVE COUPLINGS

DESCRIPTION:

Chance® Type RS2875.203 helical piles have 60.4 kip ultimate capacity and 30.2 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Round shaft helical piles offer increased lateral and buckling resistance compared to solid square shafts with similar torque strength. Strength calculations are based on a design corrosion level of 50 years for most soil conditions. Chance Type RS helical piles can be coupled with square shaft

lead sections (Combo Piles) to provide greater penetration into bearing soils. Chance Type RS helical piles feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.

RS2875.203 BUILDING CODE HELICAL PILE SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: HSS 2-7/8 inch OD x 0.203 inch (schedule 40) wall steel shaft produced exclusively for Chance® products.

COUPLING: Welded sleeve forming a socket, connected with multiple structural bolts.

HELIX - 3/8 INCH THICK: ASTM A572 or A1018 or A656, with minimum yield strength of 50 ksi.

3 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 8, 10, 12, or 14 inches.

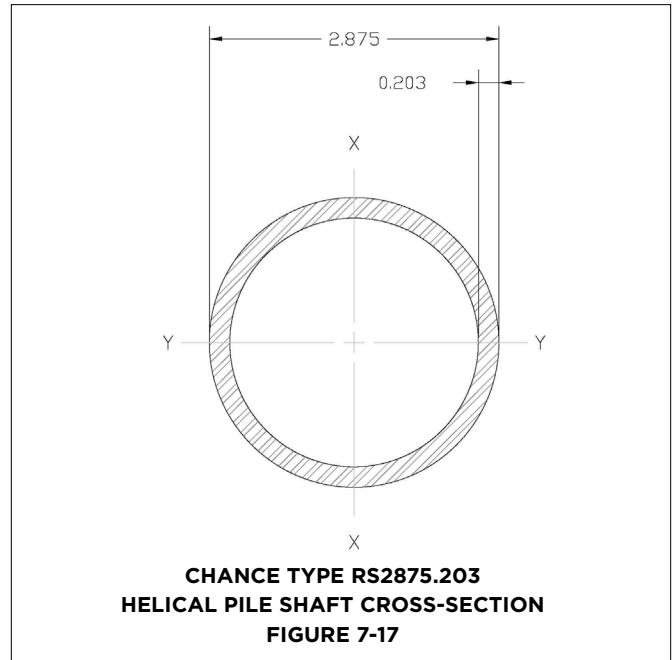
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight sharpened leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

- Single-, double-, and triple-helix lead sections, 5, 7, and 10 feet long
- Plain extensions, 3-1/2, 5, 7, and 10 feet long
- Extensions with helix plates, 5 and 7 feet long

Helical products are hot dip galvanized per ASTM A153 Class B-1.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.

NOMINAL, LRFD DESIGN, & ASD ALLOWABLE STRENGTHS OF RS2875.203 HELIX PLATES FOR SHAFT AXIAL TENSION & COMPRESSION¹

HELIX DIAMETER (in) {mm}	THICKNESS (in) {mm}	NOMINAL STRENGTH (kip) {kN}	LRFD DESIGN STRENGTH (kip) {kN}	ASD ALLOWABLE STRENGTH (kip) {kN}
8 {200}	0.375 {9.5}	135.0 {600.5}	101.3 {450.6}	67.5 {300.3}
10 {250}	0.375 {9.5}	122.7 {545.8}	92.0 {409.2}	61.4 {273.1}
12 {300}	0.375 {9.5}	127.1 {565.4}	95.3 {423.9}	63.6 {282.9}
14 {350}	0.375 {9.5}	124.9 {555.6}	93.7 {416.8}	62.4 {277.6}

Notes:

For SI: 1 kip = 4.448 kN.

¹Capacities based on a design corrosion level of 50 years.

NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS OF CHANCE TYPE RS2875.203 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2,3}

SECTION TYPE & HELIX COUNT	NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS (kip) {kN}							
	FIRM SOIL				SOFT SOIL			
	FIXED		PINNED		FIXED		PINNED	
	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN
Lead, Single Helix	87.1 {387.4}	65.3 {290.5}	80.1 {356.3}	65.3 {290.5}	66.1 {294.0}	59.5 {264.7}	45.2 {201.1}	41.4 {184.2}
Lead, Multi-Helix	[84.2	[65.3	[73.9	[65.3	[56.0	[50.4	[33.2	[29.9
Extension	{374.5}]	{290.5}]	{328.7}]	{290.5}]	{249.1}]	{224.2}]	{147.7}]	{133.0}]

Notes:

For SI: 1 kip = 4.448 kN.

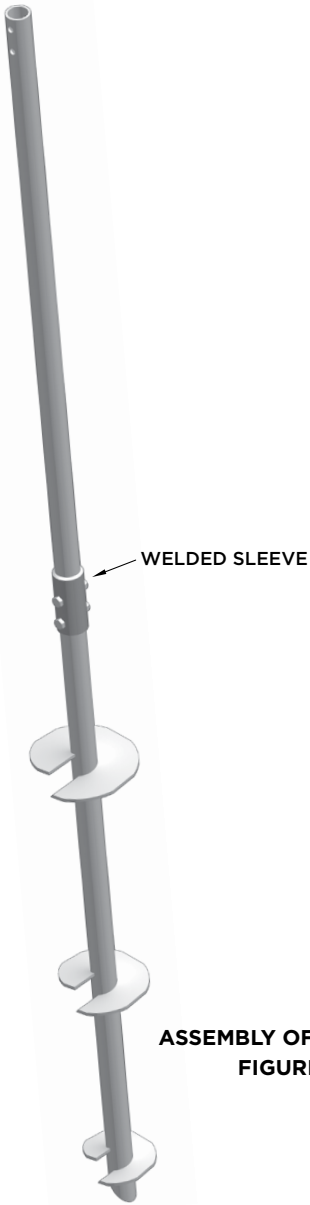
¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings include an allowance for corrosion over a 50-year service life and presume the supported structure is braced in accordance with IBC Section 1810.2.2 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.

³ Values in brackets [] are for use in Seismic Design Categories D, E, and F.



RS2875.203 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled HSS 2-1/2 inch nominal Schedule 40 (0.203 inch nominal wall) per ASTM A500 Grade B/C with 65 ksi minimum yield strength			
SHAFT SIZE, OD	2.875 in	73 mm	CORRODED	
			2.862 in	72.7 mm
SHAFT SIZE, ID*	2.497 in	63.4 mm	CORRODED	
			2.510 in	63.75 mm
MOMENT OF INERTIA (I)*	1.44 in ⁴	59.9 cm ⁴	CORRODED	
			1.344 in ⁴	55.9 cm ⁴
SHAFT AREA (A)*	1.59 in ²	10.3 cm ²	CORRODED	
			1.48 in ²	9.57 cm ²
SECTION MODULUS (S _{x-x})*	1.0 in ³	16.4 cm ³	CORRODED	
			0.939 in ³	15.4 cm ³
PERIMETER	9.0 in	22.8 cm	CORRODED	
			8.99 in	22.8 cm
COUPLING	Welded round deep socket sleeve			
COUPLING BOLTS	Two 3/4 inch diameter SAE J429 Grade 5 hex head bolts with threads excluded from shear planes			
HELIX PLATES	0.375 inch thick, formed on matching metal dies, ASTM A572 Grade 50 or better			
COATINGS	Hot dip galvanized per ASTM A153 Class B-1, 3.1 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	9 ft ⁻¹		30 m ⁻¹	
TORQUE RATING	6,710 ft-lb		9,100 N-m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	87 kip	387 kN	65.3 kip	290.5 kN
ALLOWABLE TENSION STRENGTH	43.5 kip		193.5 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	60.4 kip	269 kN	30.2 kip	134 kN
Note: * Computed with 93% of wall thickness per AISC 360-10, B4.2				

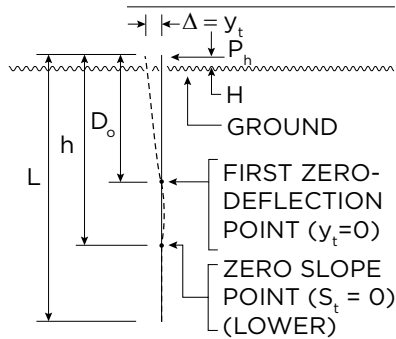


**ASSEMBLY OF RS2875.203
FIGURE 7-18**

ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE RS2875.203 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2,3}

SECTION TYPE & HELIX COUNT	ASD ALLOWABLE AXIAL COMPRESSION STRENGTH (kip) {kN}			
	FIRM SOIL		SOFT SOIL	
	FIXED	PINNED	FIXED	PINNED
Lead, Single Helix	43.5 {193.5} [43.5 {193.5}]	43.5 {193.5} [43.5 {193.5}]	39.6 {176.1} [33.5 {149.0}]	27.5 {122.3} [19.9 {88.5}]
Lead, 2-Helix 8"-10"	43.5 {193.5} [43.5 {193.5}]	43.5 {193.5} [43.5 {193.5}]	39.6 {176.1} [33.5 {149.0}]	27.5 {122.3} [19.9 {88.5}]
Lead, 2-Helix 10"-12"				
Lead, 2-Helix 12"-14"				
Lead, 2-Helix 14"-14"				
Lead, Multi-Helix	43.5 {193.5} [43.5 {193.5}]	43.5 {193.5} [43.5 {193.5}]	39.6 {176.1} [33.5 {149.0}]	27.5 {122.3} [19.9 {88.5}]
Extension	43.5 {193.5} [43.5 {193.5}]	43.5 {193.5} [43.5 {193.5}]	39.6 {176.1} [33.5 {149.0}]	27.5 {122.3} [19.9 {88.5}]
Notes: For SI: 1 kip = 4.448 kN.				
¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.				
² Strength ratings include an allowance for corrosion over a 50-year service life and presume the supported structure is braced in accordance with IBC Section 1810.2.2 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.				
³ Values in brackets [] are for use in Seismic Design Categories D, E, and F.				

RS2875.203 LATERAL DEFLECTION ANALYSIS FOR SEISMIC DESIGN CATEGORIES D, E, & F: The free-headed pile lateral movements at allowable loads (lateral only - no vertical) of Chance® Type RS2875.203 helical piles can be determined from the following table.



- P_h = Allowable lateral load
- L = Pile length
- $y_t = \Delta$ = Lateral drift of pile head
- H = Length from pile head to groundline
- D_o = Depth along pile shaft to point of first zero-deflection point ($y_t = 0$), a.k.a. the flexural length of pile
- h = Depth along pile shaft to point of zero slope ($S_t = 0$)
- M = Bending moment in pile shaft

For lateral resistance in non-seismic areas, special analysis and tests of RS2875.203 helical piles indicate allowable lateral load with less than 1/2" lateral deflection in stiff, firm, and soft soils. Test and analysis data are listed in the table. Per AC358 1.4.7, the Shaft Flexural Length (D_o) is measured from the top of the pile down to the first point of zero lateral deflection in the soil. AC358 1.4.8 defines the Shaft Seismic Flexural Length (Seismic D_o) as 120% of the Shaft Flexural Length, which for RS2875.203 helical piles embedded in stiff, firm, and soft soils is 4.3 ft, 4.7 ft, and 7.0 ft respectively.

SOIL TYPE	NON-SEISMIC LATERAL LOADING (SEISMIC DESIGN CATEGORIES A, B, AND C)							SEISMIC LATERAL LOADING (SEISMIC DESIGN CATEGORIES D, E, AND F)						
	ALLOWABLE LATERAL LOAD (P_h) (kip)	DRIFT AT PILE HEAD (y_t) (in)	DEPTH TO SOIL (H) (ft)	DEPTH TO $y_t = 0$ (D_o) (ft)	DEPTH TO $S_t = 0$ (h) (ft)	ALLOWABLE STORY DRIFT (Δ) AT PILE HEAD	BENDING MOMENT IN PILE (kip-in)	ALLOWABLE LATERAL LOAD (P_h) (kip)	DRIFT AT PILE HEAD (y_t) (in)	DEPTH TO SOIL (H) (ft)	DEPTH TO $y_t = 0$ (D_o) (ft)	DEPTH TO $S_t = 0$ (h) (ft)	ALLOWABLE STORY DRIFT (Δ) AT PILE HEAD	BENDING MOMENT IN PILE (kip-in)
Stiff Soil: N = 9 to 20	1.61	0.46	0	3.6	4.4	0.009h	21.6	1.61	0.94	0.7	4.3	7.0	0.011h	32.4
Firm Soil: N = 5 to 8	1.30	0.46	0	3.9	4.9	0.008h	18.0	1.39	1.19	0.8	4.7	5.9	0.017h	32.4
Soft Soil: N = 1 to 4	0.40	0.46	0	5.8	7.2	0.005h	8.4	N/A	9.0	1.2	7.0	8.5	0.088h	N/A

All shaft, coupler, and bracket-tube shapes and materials have been checked for adequate ductility per AC358 3.14.3; AISC 341 Table D1.1 (including applicable footnotes); and IBC 1810.3.6.1.

Lateral deflections during seismic loading should be checked against the ASCE-7 Table 12.12-1 (reproduced below) showing Allowable Story Drift values for various structure lateral load resistance system types that are presented as ratios of the story height in Seismic Design Categories D, E, and F. The story height (h) assumed for the helical pile is the depth from the pile top down to the point of zero pile slope-from-vertical in the soil (S_t). The Allowable Story Drift in the seismic loading table above must be less than or equal to the Allowable Story Drift in Table 12.12.1 in order to determine if RS2875.203 helical piles can be used in that Structure Type and Risk Category.

Structure	Risk Category		
	I or II	III	IV
Structures, other than masonry shear walls, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	0.025h	0.020h	0.015h
Masonry cantilever shear wall structures	0.010h	0.010h	0.010h
Other masonry shear wall structures	0.007h	0.007h	0.007h
All other structures	0.020h	0.015h	0.010h



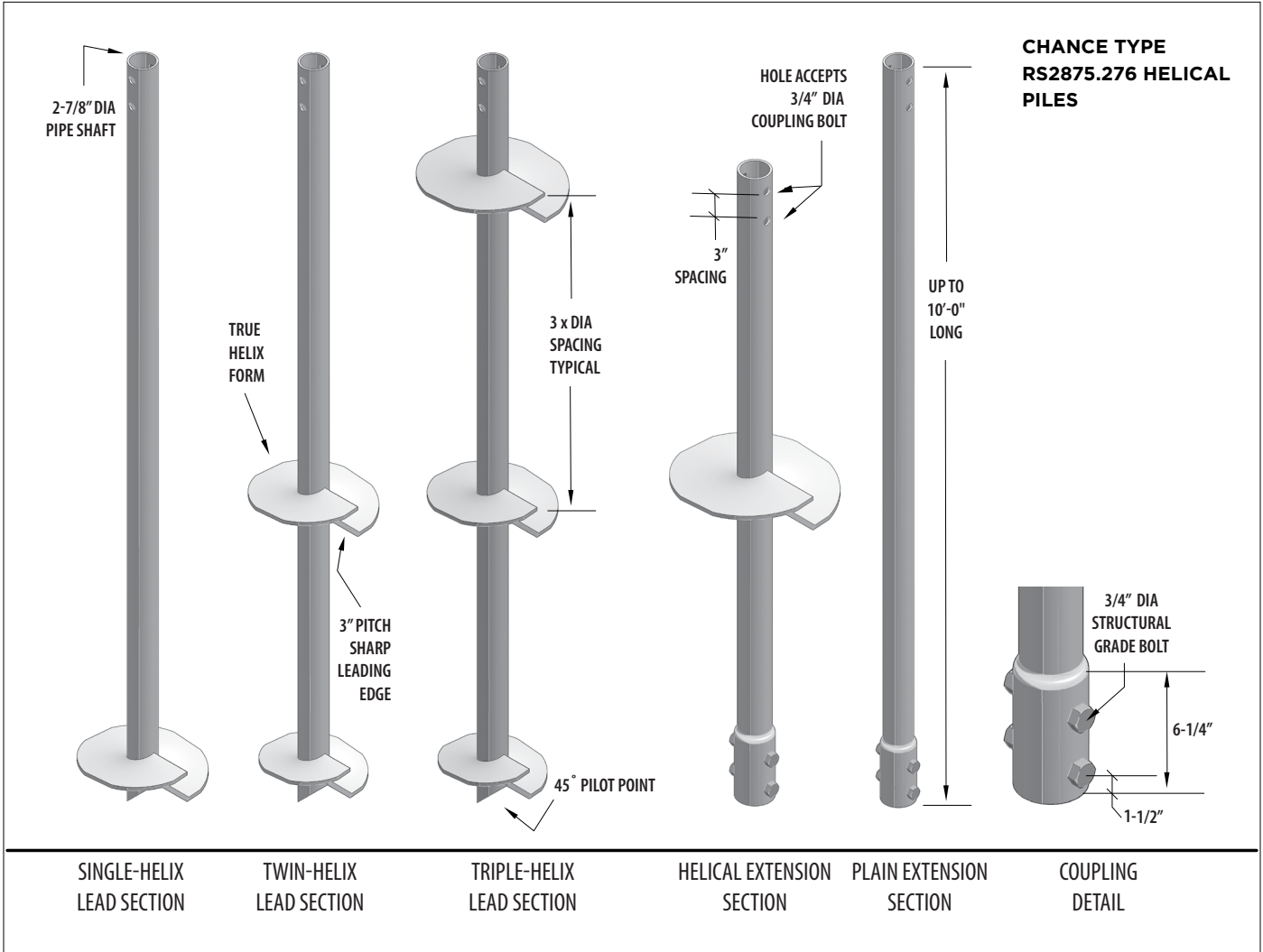
CHANCE TYPE RS2875.276 HELICAL PILES

- 72 kip ULTIMATE – 36 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING – 8,000 ft-lb
- MULTI-PURPOSE 2-7/8" DIAMETER, 0.276" WALL, ROUND HSS SHAFT WITH INTEGRALLY FORMED UPSET SOCKETS

DESCRIPTION:

Chance® Type RS2875.276 helical piles have 72 kip ultimate capacity and 36 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Round shaft helical piles offer increased lateral and buckling resistance compared to solid square shafts with similar torque strength. Strength calculations are based on a design corrosion level of 50 years for most soil conditions. Chance Type

RS helical piles can be coupled with square shaft lead sections (Combo Piles) to provide greater penetration into bearing soils. Chance Type RS helical piles feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.

RS2875.276 HELICAL PILE SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: HSS 2-7/8 inch OD x 0.276 inch (schedule 80) wall steel shaft produced exclusively for Chance® products.

COUPLING: Forged as an integral part of the plain and helical extension material as round deep sockets, connected with multiple structural bolts.

HELIX - 3/8 INCH THICK: ASTM A656 or A1018, with minimum yield strength of 80 ksi.

3 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 8, 10, 12, or 14 inches.

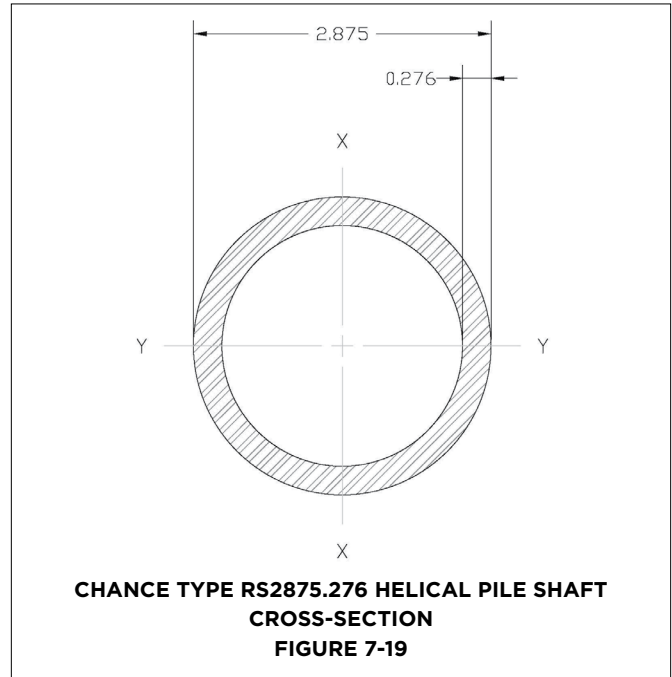
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight sharpened leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

- Single-, double-, triple-, and quad-helix lead sections, 3-1/2, 5, 7, and 10 feet long
- Plain extensions, 3-1/2, 5, 7, and 10 feet long
- Extensions with helix plates, 3-1/2 feet long

Helical products are hot dip galvanized per ASTM A153 Class B-1.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.

NOMINAL, LRFD DESIGN, & ASD ALLOWABLE STRENGTHS OF RS2875.276 HELIX PLATES FOR SHAFT AXIAL TENSION & COMPRESSION¹

HELIX DIAMETER (in) [mm]	THICKNESS (in) [mm]	NOMINAL STRENGTH (kip) [kN]	LRFD DESIGN STRENGTH (kip) [kN]	ASD ALLOWABLE STRENGTH (kip) [kN]
8 [200]	0.375 [9.5]	121.4 [540.0]	91.1 [378]	60.7 [270.0]
10 [250]	0.375 [9.5]	98.9 [439.9]	74.2 [330]	49.5 [220.2]
12 [300]	0.375 [9.5]	85.3 [379.4]	63.9 [284.6]	42.7 [189.9]
14 [350]	0.375 [9.5]	53.7 [238.9]	40.3 [179.2]	26.9 [119.7]

Notes:

For SI: 1 kip = 4.448 kN.

¹Capacities based on a design corrosion level of 50 years.

NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS OF CHANCE TYPE RS2875.276 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS (kip) [kN]							
	FIRM SOIL				SOFT SOIL			
	FIXED		PINNED		FIXED		PINNED	
	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN
Lead, Single Helix	92.9 [413.2]	83.6 [371.9]	86.3 [383.9]	77.7 [345.6]	73.9 [328.7]	66.5 [295.8]	55.2 [245.5]	49.7 [221.1]
	See Helix Table Above for Single 12" & 14"				See Helix Table Above for Single 14"			
Lead, Multi-Helix	92.9 [413.2]	83.6 [371.9]	86.3 [383.9]	77.7 [345.6]	73.9 [328.7]	66.5 [295.8]	55.2 [245.5]	49.7 [221.1]
Extension	92.9 [413.2]	83.6 [371.9]	86.3 [383.9]	77.7 [345.6]				

Notes:

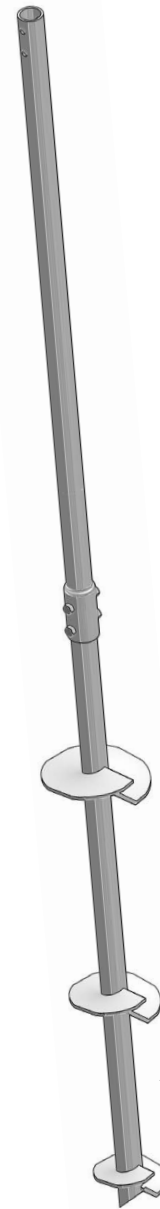
For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.



RS2875.276 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled HSS 2-1/2 inch nominal Schedule 80 (0.276 inch nominal wall) per ASTM A500 Grade B/C with 50 ksi minimum yield strength			
SHAFT SIZE, OD	2.875 in	73 mm	CORRODED	
			2.862 in	72.7 mm
SHAFT SIZE, ID*	2.36 in	60 mm	CORRODED	
			2.375 in	60.3 mm
MOMENT OF INERTIA (I)*	1.83 in ⁴	76.2 cm ⁴	CORRODED	
			1.733 in ⁴	72.1 cm ⁴
SHAFT AREA (A)*	2.11 in ²	13.6 cm ²	CORRODED	
			2.0 in ²	12.9 cm ²
SECTION MODULUS (S _{x-x})*	1.27 in ³	20.8 cm ³	CORRODED	
			1.21 in ³	19.8 cm ³
PERIMETER	9.0 in	22.8 cm	CORRODED	
			8.99 in	22.8 cm
COUPLING	Integral forged round deep socket sleeve			
COUPLING BOLTS	Two 3/4 inch diameter SAE J429 Grade 5 hex head bolts with threads excluded from shear planes			
HELIX PLATES	0.375 inch thick, formed on matching metal dies, ASTM A656 Grade 80 or better			
COATINGS	Hot dip galvanized per ASTM A153 Class B-1, 3.1 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	9 ft ⁻¹		30 m ⁻¹	
TORQUE RATING	8,000 ft-lb		10,846 N·m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	90 kip	400 kN	67.5 kip	300 kN
ALLOWABLE TENSION STRENGTH	45 kip		200 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	72 kip	320 kN	36 kip	160 kN
Note: * Computed with 93% of wall thickness per AISC 360-10, B4.2				



**ASSEMBLY OF RS2875.276
FIGURE 7-20**

ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE RS2875.276 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	ASD ALLOWABLE AXIAL COMPRESSION STRENGTH (kip) [kN]			
	FIRM SOIL		SOFT SOIL	
	FIXED	PINNED	FIXED	PINNED
Lead, Single Helix	For Single 8" - 55.6 [247.3]	For Single 8" - 51.7 [230.0]	44.3 [197.1]	33.0 [146.8]
	See Helix Strength Table Above for 10", 12", & 14"	See Helix Strength Table Above for 10", 12", & 14"	See Helix Strength Table Above for 12", & 14"	For Single 14" - 26.9 [119.7]
Lead, 2-Helix 8"-10"	55.6 [247.3]	51.7 [230.0]	44.3 [197.1]	33.0 [146.8]
Lead, 2-Helix 10"-12"				
Lead, 2-Helix 12"-14"				
Lead, 2-Helix 14"-14"	55.6 [247.3]	51.7 [230.0]	44.3 [197.1]	33.0 [146.8]
Lead, Multi-Helix	55.6 [247.3]	51.7 [230.0]	44.3 [197.1]	33.0 [146.8]
Extension	55.6 [247.3]	51.7 [230.0]	44.3 [197.1]	33.0 [146.8]

Notes:
 For SI: 1 kip = 4.448 kN.
¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.
² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.

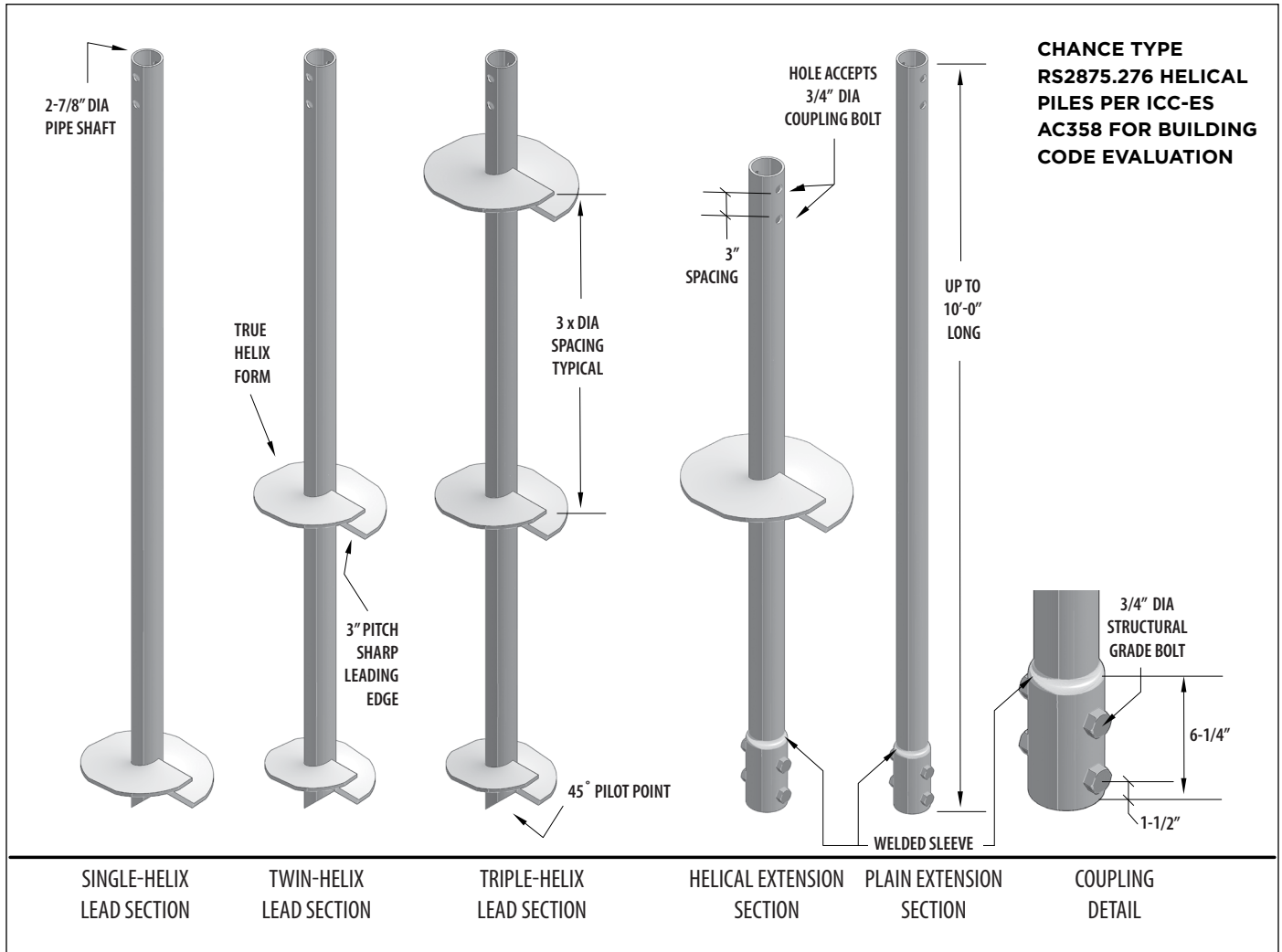
CHANCE TYPE RS2875.276 HELICAL PILES PER ICC-ES AC358 FOR BUILDING CODE EVALUATION

- 80.1 kip ULTIMATE – 40.05 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING – 8,900 ft-lb
- MULTI-PURPOSE 2-7/8" DIAMETER, 0.276" WALL, ROUND HSS SHAFT WITH SLEEVE COUPLINGS

DESCRIPTION:

Chance® Type RS2875.276 helical piles have 80.1 kip ultimate capacity and 40.05 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Round shaft helical piles offer increased lateral and buckling resistance compared to solid square shafts with similar torque strength. Strength calculations are based on a design corrosion level of 50 years for most soil conditions.

Chance Type RS helical piles can be coupled with square shaft lead sections (Combo Piles) to provide greater penetration into bearing soils. Chance Type RS helical piles feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.



RS2875.276 BUILDING CODE HELICAL PILE SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: HSS 2-7/8 inch OD x 0.276 inch (schedule 80) wall steel shaft produced exclusively for Chance® products.

COUPLING: Welded sleeve forming a socket connected with multiple structural bolts.

HELIX - 3/8 INCH THICK: ASTM A656 or A1018, with minimum yield strength of 80 ksi.

3 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 8, 10, 12, or 14 inches.

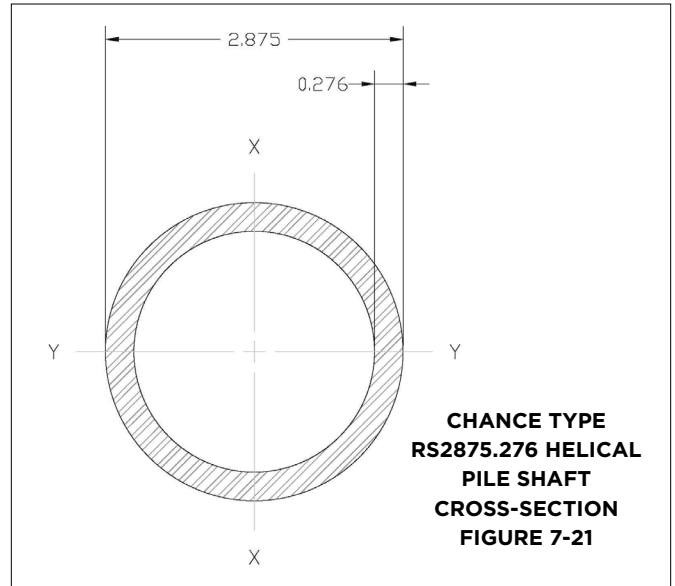
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight sharpened leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

- Single-, double-, triple-, and quad-helix lead sections, 3-1/2, 5, 7, and 10 feet long
- Plain extensions, 3-1/2, 5, 7, and 10 feet long
- Extensions with helix plates, 3-1/2 feet long

Helical products are hot dip galvanized per ASTM A153 Class B-1.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.

NOMINAL, LRFD DESIGN, & ASD ALLOWABLE STRENGTHS OF RS2875.276 HELIX PLATES FOR SHAFT AXIAL TENSION & COMPRESSION¹

HELIX DIAMETER (in) {mm}	THICKNESS (in) {mm}	NOMINAL STRENGTH (kip) {kN}	LRFD DESIGN STRENGTH (kip) {kN}	ASD ALLOWABLE STRENGTH (kip) {kN}
8 {200}	0.375 {9.5}	113.9 {504.4}	85.4 {378.3}	56.9 {253.1}
10 {250}	0.375 {9.5}	94.5 {420.4}	70.9 {315.3}	47.3 {210.4}
12 {300}	0.375 {9.5}	93.0 {413.7}	69.8 {310.3}	46.5 {206.8}
14 {350}	0.375 {9.5}	100.3 {446.2}	75.2 {334.7}	50.2 {223.3}

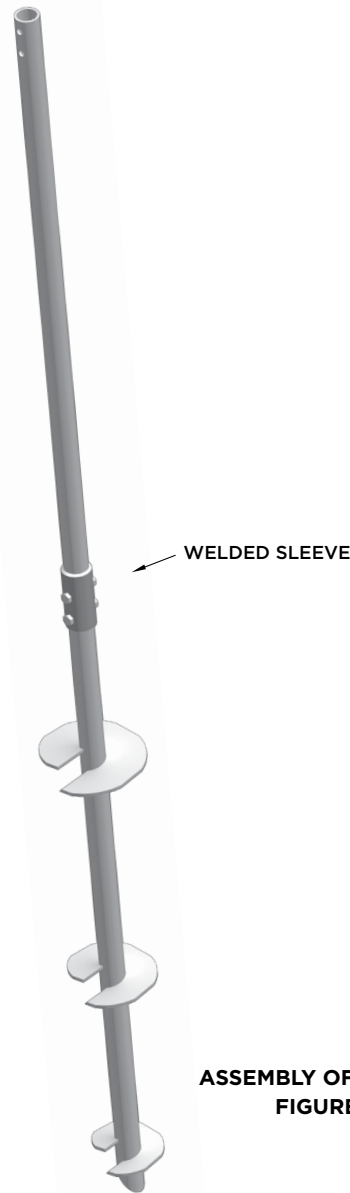
Notes:
For SI: 1 kip = 4.448 kN.
¹Capacities based on a design corrosion level of 50 years.

NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS OF CHANCE TYPE RS2875.276 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2,3}

SECTION TYPE & HELIX COUNT	NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS (kip) {kN}							
	FIRM SOIL				SOFT SOIL			
	FIXED		PINNED		FIXED		PINNED	
	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN
Lead, Single Helix	92.9 {413.2} [89.8 {399.5}]	83.6 {371.9} [80.8 {359.4}] See Helix Strength Table Above for Single 10", 12", & 14"	86.3 {383.9} [80.8 {359.4}]	77.7 {345.6} [72.8 {323.8}] See Helix Strength Table Above for Single 10", 12", & 14"	73.9 {328.7} [64.6 {287.4}]	66.5 {295.8} [58.2 {258.9}]	55.2 {245.5} [42.4 {188.6}]	49.7 {221.1} [38.2 {170}]
Lead, Multi-Helix	92.9 {413.2} [89.8 {399.5}]	83.6 {371.9} [80.8 {359.4}]	86.3 {383.9} [80.8 {359.4}]	77.7 {345.6} [72.8 {323.8}]	73.9 {328.7} [64.6 {287.4}]	66.5 {295.8} [58.2 {258.9}]	55.2 {245.5} [42.4 {188.6}]	49.7 {221.1} [38.2 {170}]
Extension	92.9 {413.2} [89.8 {399.5}]	83.6 {371.9} [80.8 {359.4}]	86.3 {383.9} [80.8 {359.4}]	77.7 {345.6} [72.8 {323.8}]	[64.6 {287.4}]	[58.2 {258.9}]	[42.4 {188.6}]	[38.2 {170}]

Notes:
For SI: 1 kip = 4.448 kN.
¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.
² Strength ratings include an allowance for corrosion over a 50-year service life and presume the supported structure is braced in accordance with IBC Section 1810.2.2 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.
³ Values in brackets [] are for use in Seismic Design Categories D, E, and F.

RS2875.276 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled HSS 2-1/2 inch nominal Schedule 80 (0.276 inch nominal wall) per ASTM A500 Grade B/C with 50 ksi minimum yield strength			
SHAFT SIZE, OD	2.875 in	73 mm	CORRODED	
			2.862 in	72.7 mm
SHAFT SIZE, ID*	2.36 in	60 mm	CORRODED	
			2.375 in	60.3 mm
MOMENT OF INERTIA (I)*	1.83 in ⁴	76.2 cm ⁴	CORRODED	
			1.733 in ⁴	72.1 cm ⁴
SHAFT AREA (A)*	2.11 in ²	13.6 cm ²	CORRODED	
			2.0 in ²	12.9 cm ²
SECTION MODULUS (S _{x-x})*	1.27 in ³	20.8 cm ³	CORRODED	
			1.21 in ³	19.8 cm ³
PERIMETER	9.0 in	22.8 cm	CORRODED	
			8.99 in	22.8 cm
COUPLING	Welded round deep socket sleeve			
COUPLING BOLTS	Two 3/4 inch diameter SAE J429 Grade 5 hex head bolts with threads excluded from shear planes			
HELIX PLATES	0.375 inch thick, formed on matching metal dies, ASTM A656 Grade 80 or better			
COATINGS	Hot dip galvanized per ASTM A153 Class B-1, 3.1 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	9 ft ⁻¹		30 m ⁻¹	
TORQUE RATING	8,900 ft·lb		12,067 N·m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	97.9 kip	453.3 kN	73.4 kip	326.5 kN
ALLOWABLE TENSION STRENGTH	48.9 kip		217.5 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	80.1 kip	356.3 kN	40.05 kip	178.2 kN
Note: * Computed with 93% of wall thickness per AISC 360-10, B4.2				



**ASSEMBLY OF RS2875.276
FIGURE 7-22**

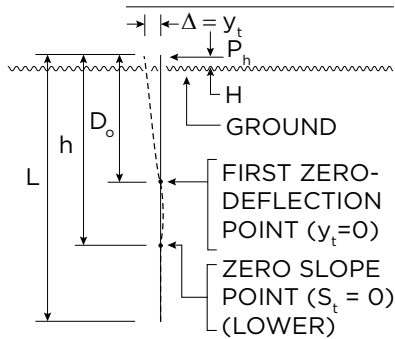
ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE RS2875.276 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2,3}

SECTION TYPE & HELIX COUNT	ASD ALLOWABLE AXIAL COMPRESSION STRENGTH (kip) {kN}			
	FIRM SOIL		SOFT SOIL	
	FIXED	PINNED	FIXED	PINNED
Lead, Single Helix	For Single 8" - 55.6 {247.3} [53.8 {249.3}]	For Single 8" - 51.7 {230.0} [48.4 {215.3}]	44.3 {197.1} [38.7 {172.1}]	33.0 {146.8} [25.4 {113}]
	See Helix Strength Table Above for 10", 12", & 14"	See Helix Strength Table Above for 10", 12", & 14"		
Lead, 2-Helix 8"-10"	55.6 {247.3} [53.8 {249.3}]	51.7 {230.0} [48.4 {215.3}]	44.3 {197.1} [38.7 {172.1}]	33.0 {146.8} [25.4 {113}]
Lead, 2-Helix 10"-12"				
Lead, 2-Helix 12"-14"				
Lead, 2-Helix 14"-14"				
Lead, Multi-Helix	55.6 {247.3} [53.8 {249.3}]	51.7 {230.0} [48.4 {215.3}]	44.3 {197.1} [38.7 {172.1}]	33.0 {146.8} [25.4 {113}]
Extension	55.6 {247.3} [53.8 {249.3}]	51.7 {230.0} [48.4 {215.3}]		

Notes:
 For SI: 1 kip = 4.448 kN.
¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.
² Strength ratings include an allowance for corrosion over a 50-year service life and presume the supported structure is braced in accordance with IBC Section 1810.2.2 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.
³ Values in brackets [] are for use in Seismic Design Categories D, E, and F.



RS2875.276 LATERAL DEFLECTION ANALYSIS FOR SEISMIC DESIGN CATEGORIES D, E, & F: The free-headed pile lateral movements at allowable loads (lateral only - no vertical) of Chance® Type RS2875.276 helical piles can be determined from the following table.



- P_h = Allowable lateral load
- L = Pile length
- $y_t = \Delta$ = Lateral drift of pile head
- H = Length from pile head to groundline
- D_o = Depth along pile shaft to point of first zero-deflection point ($y_t = 0$), a.k.a. the flexural length of pile
- h = Depth along pile shaft to point of zero slope ($S_t = 0$)
- M = Bending moment in pile shaft

For lateral resistance in non-seismic areas, special analysis and tests of RS2875.276 helical piles indicate allowable lateral load with less than 1/2" lateral deflection in stiff, firm, and soft soils. Test and analysis data are listed in the table. Per AC358 1.4.7, the Shaft Flexural Length (D_o) is measured from the top of the pile down to the first point of zero lateral deflection in the soil. AC358 1.4.8 defines the Shaft Seismic Flexural Length (Seismic D_o) as 120% of the Shaft Flexural Length, which for RS2875.276 helical piles embedded in stiff, firm, and soft soils is 4.8 ft, 5.0 ft, and 7.6 ft respectively.

SOIL TYPE	NON-SEISMIC LATERAL LOADING (SEISMIC DESIGN CATEGORIES A, B, AND C)							SEISMIC LATERAL LOADING (SEISMIC DESIGN CATEGORIES D, E, AND F)						
	ALLOWABLE LATERAL LOAD (P_h) (kip)	DRIFT AT PILE HEAD (y_t) (in)	DEPTH TO SOIL (H) (ft)	DEPTH TO $y_t = 0$ (D_o) (ft)	DEPTH TO $S_t = 0$ (h) (ft)	ALLOWABLE STORY DRIFT (Δ) AT PILE HEAD	BENDING MOMENT IN PILE (kip-in)	ALLOWABLE LATERAL LOAD (P_h) (kip)	DRIFT AT PILE HEAD (y_t) (in)	DEPTH TO SOIL (H) (ft)	DEPTH TO $y_t = 0$ (D_o) (ft)	DEPTH TO $S_t = 0$ (h) (ft)	ALLOWABLE STORY DRIFT (Δ) AT PILE HEAD	BENDING MOMENT IN PILE (kip-in)
Stiff Soil: N = 9 to 20	1.85	0.54	0	4.0	4.8	0.009h	27.6	1.85	1.17	0.8	4.8	5.6	0.017h	42.0
Firm Soil: N = 5 to 8	1.48	0.54	0	4.2	5.3	0.008h	24.0	1.5	1.14	0.8	5.0	6.2	0.015h	36.0
Soft Soil: N = 1 to 4	0.47	0.54	0	6.3	7.8	0.006h	10.8	0.50	1.29	1.3	7.6	9.2	0.012h	18.0

All shaft, coupler, and bracket-tube shapes and materials have been checked for adequate ductility per AC358 3.14.3; AISC 341 Table D1.1 (including applicable footnotes); and IBC 1810.3.6.1.

Lateral deflections during seismic loading should be checked against the ASCE-7 Table 12.12-1 (reproduced below) showing Allowable Story Drift values for various structure lateral load resistance system types that are presented as ratios of the story height in Seismic Design Categories D, E, and F. The story height (h) assumed for the helical pile is the depth from the pile top down to the point of zero pile slope-from-vertical in the soil (S_t). The Allowable Story Drift in the seismic loading table above must be less than or equal to the Allowable Story Drift in Table 12.12.1 in order to determine if RS2875.276 helical piles can be used in that Structure Type and Risk Category.

Structure	Risk Category		
	I or II	III	IV
Structures, other than masonry shear walls, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	0.025h	0.020h	0.015h
Masonry cantilever shear wall structures	0.010h	0.010h	0.010h
Other masonry shear wall structures	0.007h	0.007h	0.007h
All other structures	0.020h	0.015h	0.010h

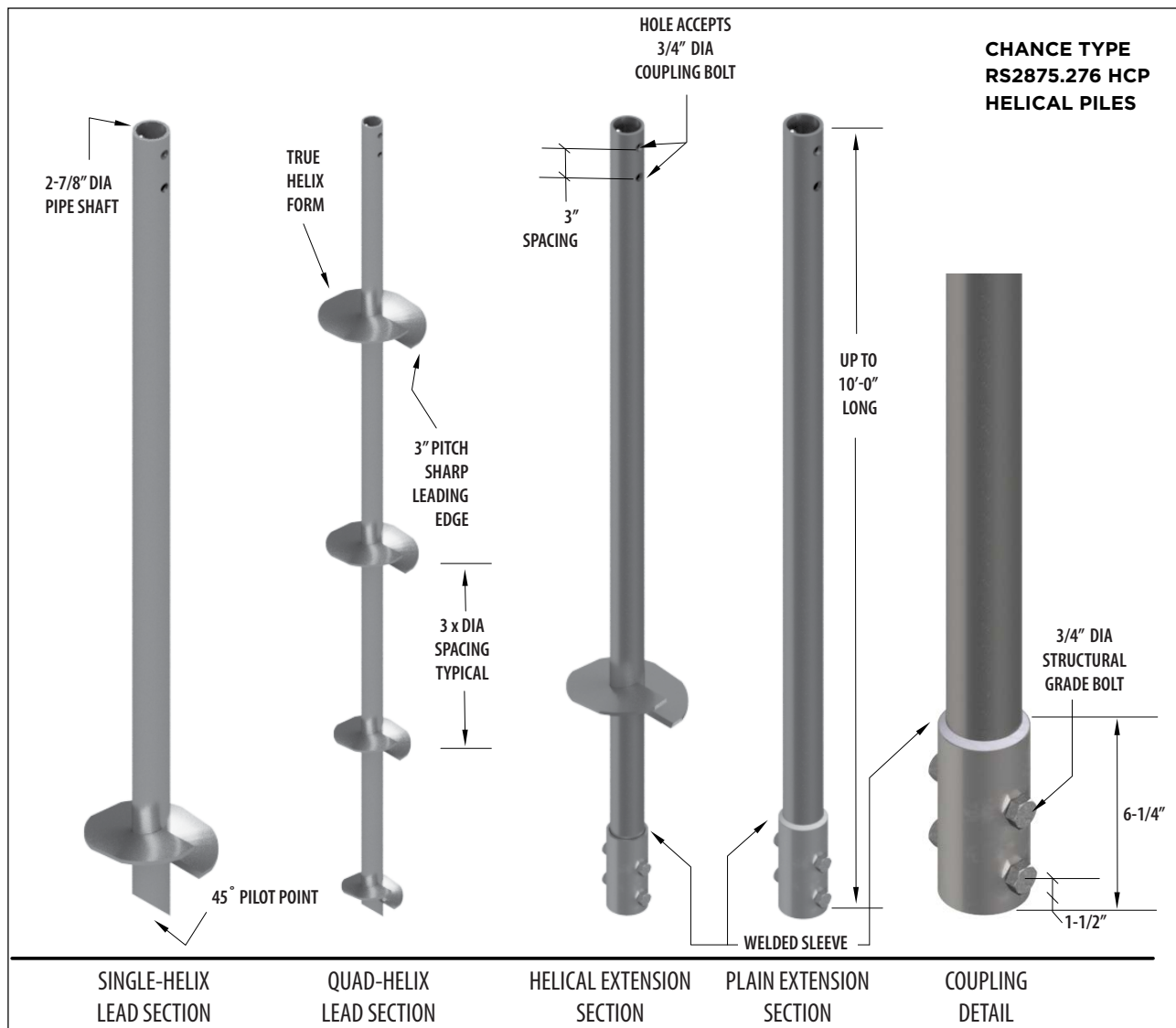
CHANCE TYPE RS2875.276 HCP HELICAL PILES

- 94.5 kip ULTIMATE - 47.25 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING - 10,500 ft-lb
- MULTI-PURPOSE 2-7/8" DIAMETER, 0.276" WALL, ROUND HSS SHAFT WITH SLEEVE COUPLINGS

DESCRIPTION:

Chance® Type RS2875.276 HCP helical piles have 94.5 kip ultimate capacity and 47.25 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Round shaft helical piles offer increased lateral and buckling resistance compared to solid square shafts with similar torque strength. Strength calculations are based on a design corrosion level of 50 years for most soil conditions.

Chance Type RS helical piles can be coupled with square shaft lead sections (Combo Piles) to provide greater penetration into bearing soils. Chance Type RS helical piles feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.



RS2875.276 HCP HELICAL PILE SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: HSS 2-7/8 inch OD x 0.276 inch wall steel shaft produced exclusively for Chance® products.

COUPLING: Welded sleeve forming a socket, connected with multiple structural grade bolts.

HELIX - 3/8 INCH THICK: ASTM A656 or A1018, with minimum yield strength of 80 ksi.

3 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 8, 10, 12, and 14 inches.

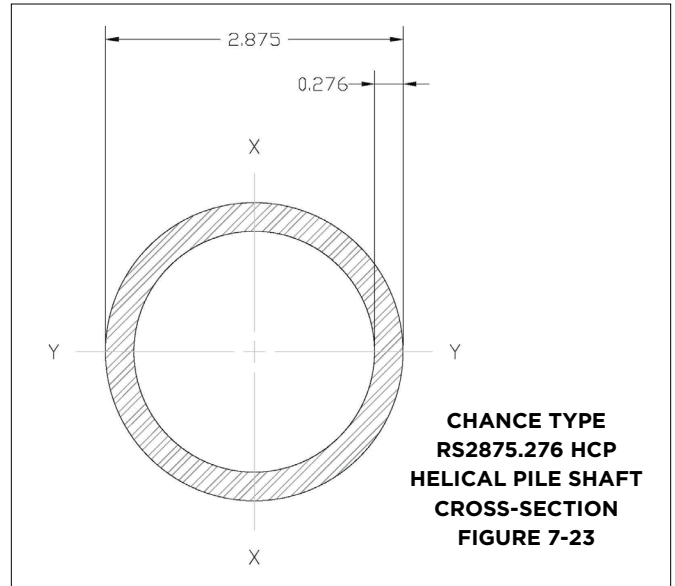
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight sharp leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

- Single-, double-, triple-, and quad-helix lead sections, up to 10 feet long
- Plain extensions, 3-½, 5, 7, and 10 feet long
- Extensions with helix plates, 5 feet long

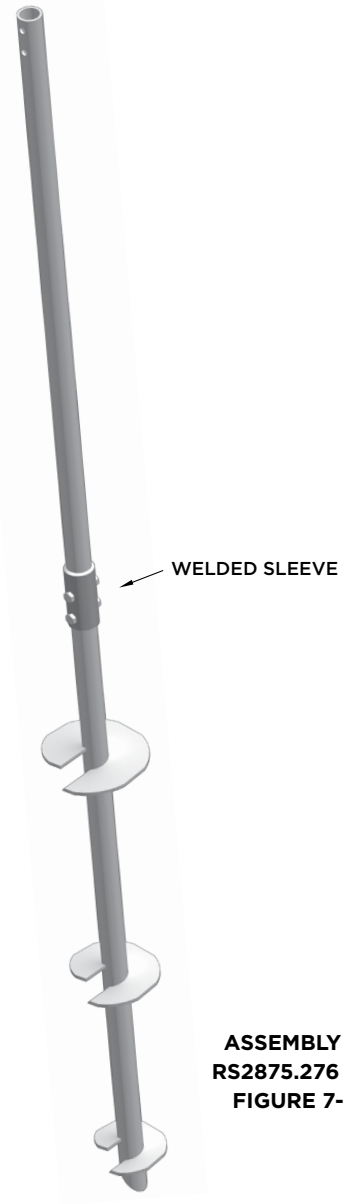
helical products are hot dip galvanized per ASTM A123 Grade 75 or are available black.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.

RS2875.276 HCP HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS

SHAFT	Hot-rolled HSS 2.875 inch, 0.276 inch nominal wall with 80 ksi minimum yield strength			
SHAFT SIZE, OD	2.875 in	73 mm	CORRODED	
			2.862 in	72.7 mm
SHAFT SIZE, ID*	2.36 in	60 mm	CORRODED	
			2.375 in	60.3 mm
MOMENT OF INERTIA (I)*	1.83 in ⁴	76.2 cm ⁴	CORRODED	
			1.733 in ⁴	72.1 cm ⁴
SHAFT AREA (A)*	2.11 in ²	13.6 cm ²	CORRODED	
			2.0 in ²	12.9 cm ²
SECTION MODULUS (S _{x-x})*	1.27 in ³	20.8 cm ³	CORRODED	
			1.21 in ³	19.8 cm ³
PERIMETER	9.0 in	22.8 cm	CORRODED	
			8.99 in	22.8 cm
COUPLING	Welded round deep socket			
COUPLING BOLTS	Two 3/4 inch diameter SAE J429 Grade 5 hex head bolts with threads excluded from shear planes			
HELIX PLATES	0.375-inch thick, formed on matching metal dies, ASTM A656 Grade 80 or better			
COATINGS	Hot dip galvanized per ASTM A123 Grade 75, 3.0 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	9 ft ⁻¹		30 m ⁻¹	
TORQUE RATING	10,500 ft-lb		14,236 N-m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	100 kip	445 kN	75 kip	334 kN
ALLOWABLE TENSION STRENGTH	50 kip		222 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	94.5 kip	420 kN	47.25 kip	210 kN
Note: * Computed with 93% of wall thickness per AISC 360-10, B4.2				



**ASSEMBLY OF RS2875.276 HCP
FIGURE 7-24**



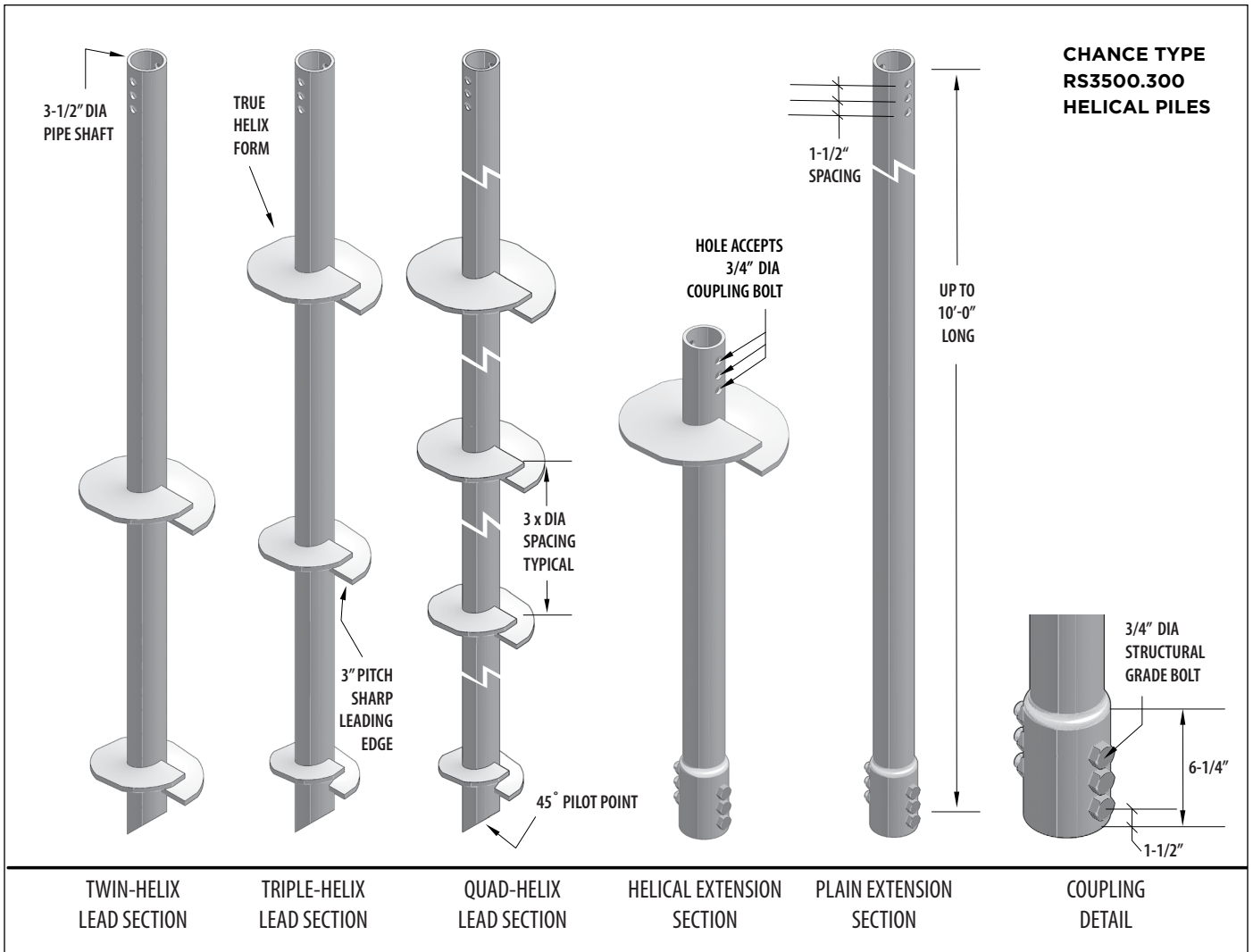
CHANCE TYPE RS3500.300 HELICAL PILES

- 91 kip ULTIMATE - 45.5 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING - 13,000 ft-lb
- MULTI-PURPOSE 3-1/2" DIAMETER, 0.300" WALL, ROUND HSS SHAFT WITH INTEGRALLY FORMED UPSET SOCKETS

DESCRIPTION:

Chance® Type RS3500.300 helical piles have 91 kip ultimate capacity and 45.5 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Round shaft helical piles offer increased lateral and buckling resistance compared to solid square shafts with similar torque strength. Strength calculations are based on a design corrosion level of 50 years for most soil conditions.

Chance Type RS helical piles can be coupled with square shaft lead sections (Combo Piles) to provide greater penetration into bearing soils. Chance Type RS helical piles feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.

RS3500.300 HELICAL PILE SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: HSS 3-1/2 inch OD x 0.300 inch (schedule 80) wall steel shaft produced exclusively for Chance® products.

COUPLING: Forged as an integral part of the plain and helical extension material as round deep sockets, connected with multiple structural bolts.

HELIX - 1/2 INCH THICK: ASTM A572 or A1018 or A656, with minimum yield strength of 50 ksi.

3 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 8, 10, 12, 14, or 16 inches.

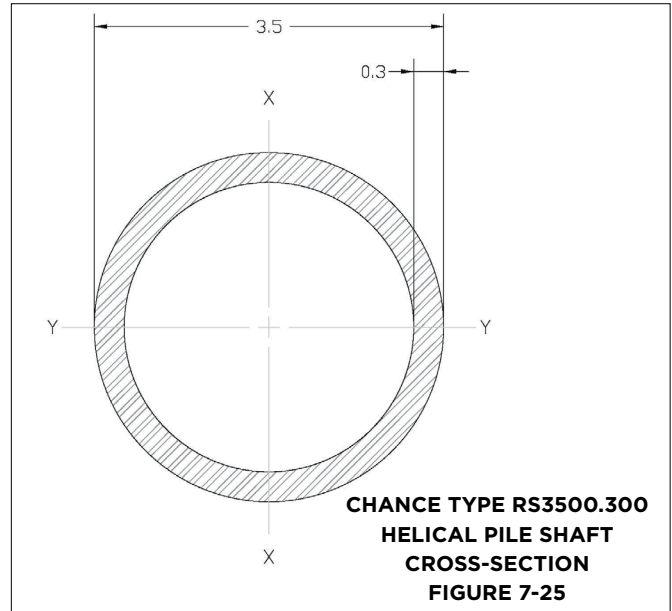
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight sharpened leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

- Single-, double-, triple-, and quad-helix lead sections, 3-1/2, 4, 5, 7, and 10 feet long
- Plain extensions, 3-1/2, 5, 7, and 10 feet long
- Extensions with helix plates, 3-1/2, 4, 7, and 10 feet long

Helical products are hot dip galvanized per ASTM A123 Grade 75.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.

NOMINAL, LRFD DESIGN, & ASD ALLOWABLE STRENGTHS OF RS3500.300 HELIX PLATES FOR SHAFT AXIAL TENSION & COMPRESSION¹

HELIX DIAMETER (in) [mm]	THICKNESS (in) [mm]	NOMINAL STRENGTH (kip) [kN]	LRFD DESIGN STRENGTH (kip) [kN]	ASD ALLOWABLE STRENGTH (kip) [kN]
8 [200]	0.5 [13]	158.3 [704.2]	118.7 [528.2]	79.1 [351.9]
10 [250]	0.5 [13]	132.5 [589.3]	99.4 [442]	66.3 [294.9]
12 [300]	0.5 [13]	98.4 [437.7]	73.8 [328.3]	49.2 [187.7]
14 [350]	0.5 [13]	132.3 [588.5]	99.2 [441.4]	66.2 [294.5]

Notes:

For SI: 1 kip = 4.448 kN.

¹Capacities based on a design corrosion level of 50 years.

NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS OF CHANCE TYPE RS3500.300 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS (kip) [kN]							
	FIRM SOIL				SOFT SOIL			
	FIXED		PINNED		FIXED		PINNED	
	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN
Lead, Single Helix	128.0 [569.4]	115.2 [512.4]	121.9 [542.2]	109.7 [488.0]	110.0 [489.3]	99.0 [440.3]	90.7 [403.5]	81.6 [363.0]
	For Single 12" - 98.4 [437.7]	See Helix Strength Table Above for Single 10", 12", & 14"	For Single 12" - 98.4 [437.7]	See Helix Strength Table Above for Single 10", 12", & 14"	For Single 12" - 98.4 [437.7]	For Single 12" - 73.8 [328.3]		For Single 12" - 73.8 [328.3]
Lead, Multi-Helix	128 [569.4]	115.2 [512.4]	121.9 [542.2]	109.7 [488.0]	110.0 [489.3]	99.0 [440.4]	90.7 [403.5]	81.6 [363.0]
Extension	128.0 [569.4]	115.2 [512.4]	121.9 [542.2]	109.7 [488.0]				

Notes:

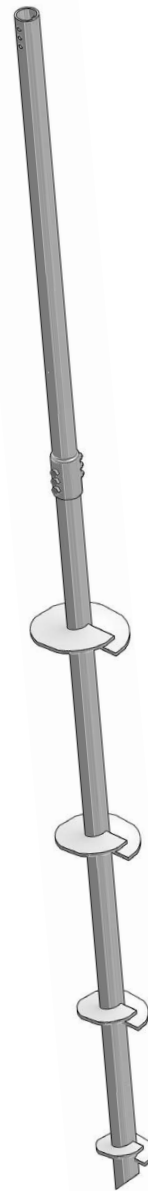
For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.



RS3500.300 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled HSS 3 inch nominal Schedule 80 (0.300 inch nominal wall) per ASTM A500 Grade B/C with 50 ksi minimum yield strength			
SHAFT SIZE, OD	3.5 in	89 mm	CORRODED	
			3.487 in	63.2 mm
SHAFT SIZE, ID*	2.942 in	74.7 mm	CORRODED	
			2.955 in	75.1 mm
MOMENT OF INERTIA (I)*	3.69 in ⁴	153.6 cm ⁴	CORRODED	
			3.514 in ⁴	146.3 cm ⁴
SHAFT AREA (A)*	2.82 in ²	18.2 cm ²	CORRODED	
			2.692 in ²	17.4 cm ²
SECTION MODULUS (S _{x-x})*	2.11 in ³	34.5 cm ³	CORRODED	
			2.016 in ³	33.0 cm ³
PERIMETER	11.0 in	27.9 cm	CORRODED	
			10.95 in	27.8 cm
COUPLING	Integral forged round deep socket sleeve			
COUPLING BOLTS	Three 3/4 in diameter SAE J429 Grade 5 hex head bolts with threads excluded from shear planes			
HELIX PLATES	0.5 inch thick, formed on matching metal dies, ASTM A572 Grade 50 or better			
COATINGS	Hot dip galvanized per ASTM A123 Grade 75, 3.0 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	7 ft ⁻¹		23 m ⁻¹	
TORQUE RATING	13,000 ft-lb		17,600 N-m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	120 kip	534 kN	90 kip	400 kN
ALLOWABLE TENSION STRENGTH	60 kip		261 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	91 kip	405 kN	45.5 kip	202.5 kN
Note: * Computed with 93% of wall thickness per AISC 360-10, B4.2				



**ASSEMBLY OF RS3500.300
FIGURE 7-26**

ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE RS3500.300 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	ASD ALLOWABLE AXIAL COMPRESSION STRENGTH (KIP) (KN)			
	FIRM SOIL		SOFT SOIL	
	FIXED	PINNED	FIXED	PINNED
Lead, Single Helix	For Single 8" - 76.6 [340.7]	For Single 8" - 73.0 [324.7]	65.9 [293.1]	54.3 [241.5]
	See Helix Strength Table Above for 10", 12", & 14"	See Helix Strength Table Above for 10", 12", & 14"	For Single 12" - 49.2 [218.9]	For Single 12" - 49.2 [218.9]
Lead, 2-Helix 8"-10"	76.6 [340.7]	73.0 [324.7]	65.9 [293.1]	54.3 [241.5]
Lead, 2-Helix 10"-12"				
Lead, 2-Helix 12"-14"				
Lead, 2-Helix 14"-14"				
Lead, Multi-Helix	76.6 [340.7]	73.0 [324.7]	65.9 [293.1]	54.3 [241.5]
Extension	76.6 [340.7]	73.0 [324.7]	65.9 [293.1]	54.3 [241.5]

Notes:

For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.

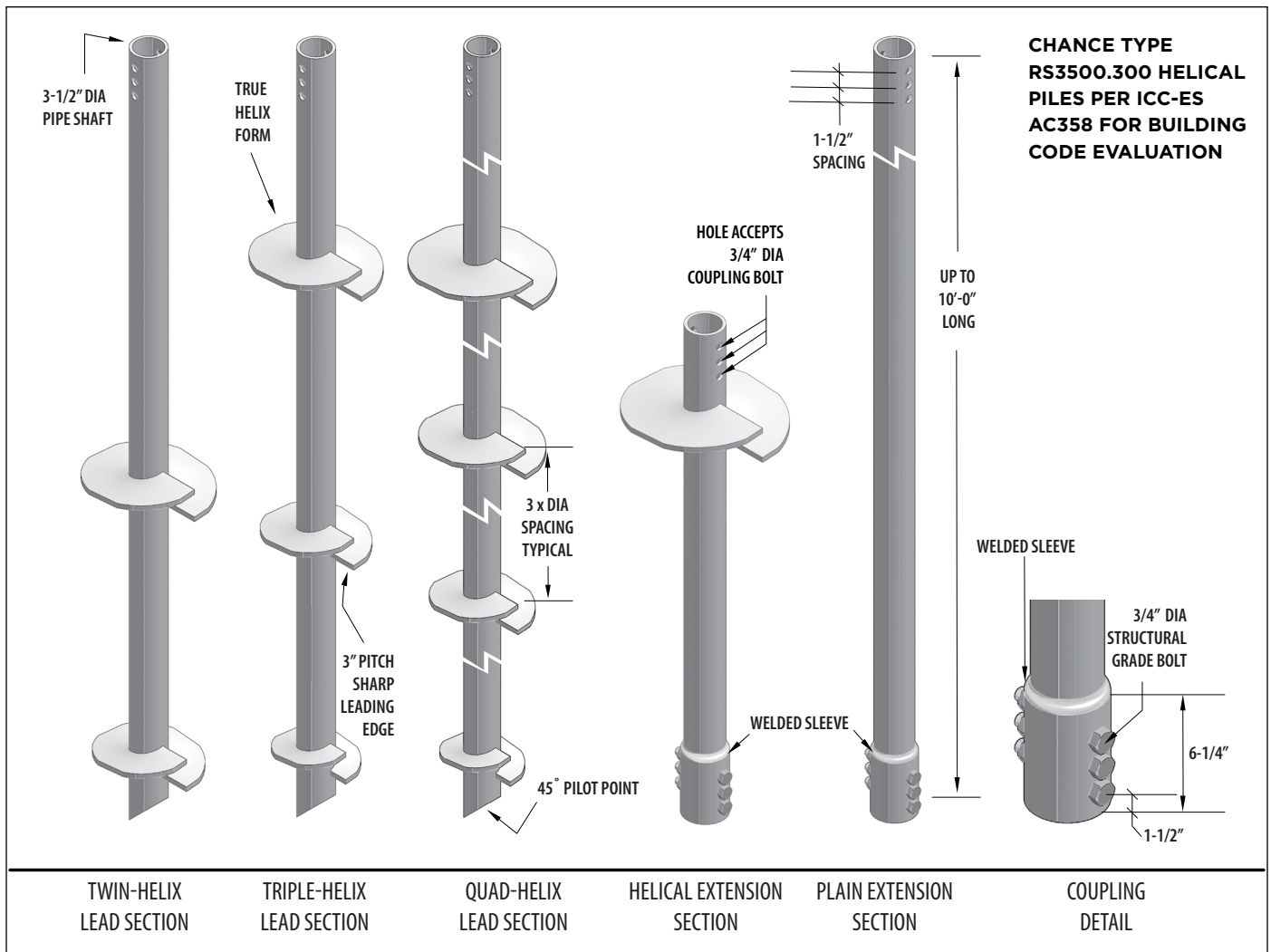
CHANCE TYPE RS3500.300 HELICAL PILES PER ICC-ES AC358 FOR BUILDING CODE EVALUATION

- 87.5 kip ULTIMATE – 43.75 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING – 12,500 ft-lb
- MULTI-PURPOSE 3-1/2" DIAMETER, 0.300" WALL, ROUND HSS SHAFT WITH WELDED SLEEVE COUPLING

DESCRIPTION:

Chance® Type RS3500.300 helical piles have 87.5 kip ultimate capacity and 43.75 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Round shaft helical piles offer increased lateral and buckling resistance compared to solid square shafts with similar torque strength. Strength calculations are based on a design corrosion level of 50 years for most soil conditions.

Chance Type RS helical piles can be coupled with square shaft lead sections (Combo Piles) to provide greater penetration into bearing soils. Chance Type RS helical piles feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.



RS3500.300 BUILDING CODE HELICAL PILE SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: HSS 3-1/2 inch OD x 0.300 inch (schedule 80) wall steel shaft produced exclusively for Chance® products.

COUPLING: Welded sleeve forming a socket, connected with multiple structural bolts.

HELIX - 1/2 INCH THICK: ASTM A572 or A1018 or A656, with minimum yield strength of 50 ksi.

3 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 8, 10, 12, or 14 inches.

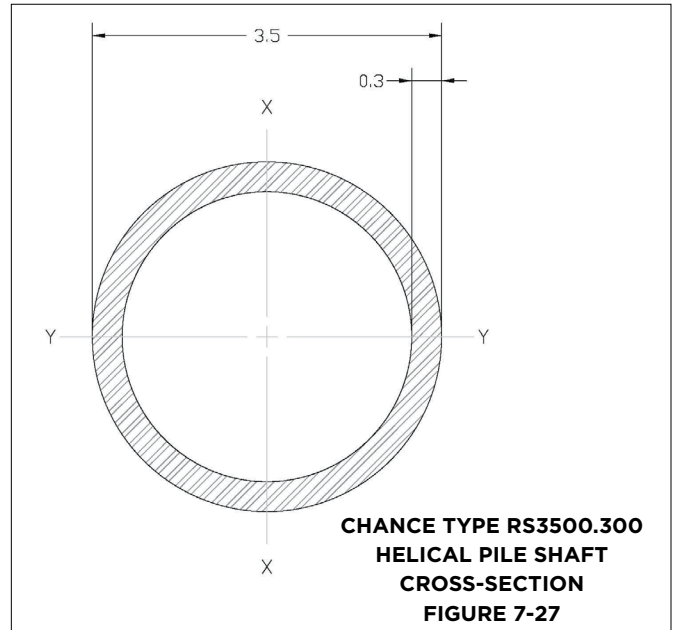
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight sharpened leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

- Single-, double-, triple-, and quad-helix lead sections, 3-1/2, 4, 5, 7, and 10 feet long
- Plain extensions, 3-1/2, 5, 7, and 10 feet long
- Extensions with helix plates, 3-1/2, 7, and 10 feet long

Helical products are hot dip galvanized per ASTM A123 Grade 75.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.

NOMINAL, LRFD DESIGN, & ASD ALLOWABLE STRENGTHS OF RS3500.300 HELIX PLATES FOR SHAFT AXIAL TENSION & COMPRESSION¹

HELIX DIAMETER (in) {mm}	THICKNESS (in) {mm}	NOMINAL STRENGTH (kip) {kN}	LRFD DESIGN STRENGTH (kip) {kN}	ASD ALLOWABLE STRENGTH (kip) {kN}
8 {200}	0.5 {13}	141.1 {627.6}	105.8 {470.7}	70.6 {314.0}
10 {250}	0.5 {13}	155.1 {689.9}	116.3 {517.4}	77.6 {345.2}
12 {300}	0.5 {13}	159.6 {709.9}	119.7 {532.4}	79.8 {354.9}
14 {350}	0.5 {13}	139.4 {620.1}	104.6 {465.1}	69.7 {301.1}

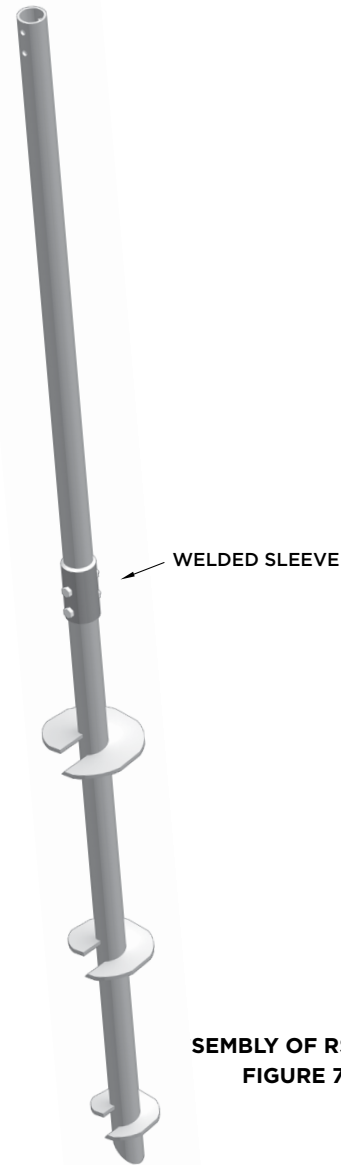
Notes:
For SI: 1 kip = 4.448 kN.
¹Capacities based on a design corrosion level of 50 years.

NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS OF CHANCE TYPE RS3500.300 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2,3}

SECTION TYPE & HELIX COUNT	NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS (kip) {kN}							
	FIRM SOIL				SOFT SOIL			
	FIXED		PINNED		FIXED		PINNED	
	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN
Lead, Single Helix	128.0 {569.4}	115.2 {512.4} [112.7 {501.3}]	121.9 {542.2} [116.7 {519.1}]	109.7 {488.0} [105.1 {467.5}]	110 {489.3} [100.7 {448}]	99.0 {440.3} [90.6 {403.0}]	90.7 {403.5} [76.2 {339.0}]	81.6 {363.0} [68.6 {305.1}]
	[125.2 {556.9}]	See Helix Strength Table Above for Single 8" & 14"		See Helix Strength Table Above for Single 8" & 14"				
Lead, Multi-Helix	128 {569.4} [125.2 {557}]	115.2 {512.4} [112.7 {501.3}]	121.9 {542.2} [116.7 {519.1}]	109.7 {488.0} [105.1 {467.5}]	110.0 {489.3} [100.7 {448}]	99.0 {440.3} [90.6 {403.0}]	90.7 {403.5} [76.2 {339.0}]	81.6 {363.0} [68.6 {305.1}]
Extension	128 {569.4} [125.2 {557}]	115.2 {512.4} [112.7 {501.3}]	121.9 {542.2} [116.7 {519.1}]	109.7 {488.0} [105.1 {467.5}]	110.0 {489.3} [100.7 {448}]	99.0 {440.3} [90.6 {403.0}]	90.7 {403.5} [76.2 {339.0}]	81.6 {363.0} [68.6 {305.1}]

Notes:
For SI: 1 kip = 4.448 kN.
¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.
² Strength ratings include an allowance for corrosion over a 50-year service life and presume the supported structure is braced in accordance with IBC Section 1810.2.2 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.
³ Values in brackets [] are for use in Seismic Design Categories D, E, and F.

RS3500.300 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled HSS 3 inch nominal Schedule 80 (0.300 inch nominal wall) per ASTM A500 Grade B/C with 50 ksi minimum yield strength			
SHAFT SIZE, OD	3.5 in	89 mm	CORRODED	
			3.487 in	63.2 mm
SHAFT SIZE, ID*	2.942 in	74.7 mm	CORRODED	
			2.955 in	75.1 mm
MOMENT OF INERTIA (I)*	3.69 in ⁴	153.6 cm ⁴	CORRODED	
			3.514 in ⁴	146.3 cm ⁴
SHAFT AREA (A)*	2.82 in ²	18.2 cm ²	CORRODED	
			2.692 in ²	17.4 cm ²
SECTION MODULUS (S _{x-x})*	2.11 in ³	34.5 cm ³	CORRODED	
			2.016 in ³	33.0 cm ³
PERIMETER	11.0 in	27.9 cm	CORRODED	
			10.95 in	27.8 cm
COUPLING	Welded round deep socket			
COUPLING BOLTS	Two 3/4 inch diameter SAE J429 Grade 5 hex head bolts with threads excluded from shear planes			
HELIX PLATES	0.5 inch thick, formed on matching metal dies, ASTM A572 Grade 50 or better			
COATINGS	Hot dip galvanized per ASTM A123 Grade 75, 3.0 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	7 ft ⁻¹		23 m ⁻¹	
TORQUE RATING	12,500 ft-lb		16,947.7 N-m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	100 kip	444.8 kN	86.1 kip	383.0 kN
ALLOWABLE TENSION STRENGTH	57.4 kip		255.3 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	87.5 kip	389.2 kN	43.75 kip	194.6 kN
Note: * Computed with 93% of wall thickness per AISC 360-10, B4.2				



**SEMBLY OF RS3500.300
FIGURE 7-28**

ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE RS3500.300 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2,3}

SECTION TYPE & HELIX COUNT	ASD ALLOWABLE AXIAL COMPRESSION STRENGTH (kip) {kN}			
	FIRM SOIL		SOFT SOIL	
	FIXED	PINNED	FIXED	PINNED
Lead, Single Helix	See Helix Strength Table Above for Single 8" & 14"	See Helix Strength Table Above for Single 8" & 14"	65.9 {293.1}	54.3 {241.5}
	76.6 {340.7} [75.0 {333.6}]	73.0 {324.7} [70.0 {311.4}]	[60.3 {268.2}]	[45.6 {202.8}]
Lead, 2-Helix 8"-10"	76.6 {340.7} [75.0 {333.6}]	73.0 {324.7} [70.0 {311.4}]	65.9 {293.1} [60.3 {268.2}]	54.3 {241.5} [45.6 {202.8}]
Lead, 2-Helix 10"-12"				
Lead, 2-Helix 12"-14"				
Lead, 2-Helix 14"-14"				
Lead, Multi-Helix	76.6 {340.7} [75.0 {333.6}]	73.0 {324.7} [70.0 {311.4}]	65.9 {293.1} [60.3 {268.2}]	54.3 {241.5} [45.6 {202.8}]
Extension	76.6 {340.7} [75.0 {333.6}]	73.0 {324.7} [70.0 {311.4}]	65.9 {293.1} [60.3 {268.2}]	54.3 {241.5} [45.6 {202.8}]

Notes:

For SI: 1 kip = 4.448 kN.

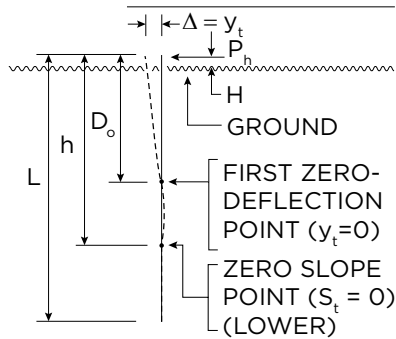
¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings include an allowance for corrosion over a 50-year service life and presume the supported structure is braced in accordance with IBC Section 1810.2.2 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.

³ Values in brackets [] are for use in Seismic Design Categories D, E, and F.



RS3500.300 LATERAL DEFLECTION ANALYSIS FOR SEISMIC DESIGN CATEGORIES D, E, & F: The free-headed pile lateral movements at allowable loads (lateral only - no vertical) of Chance® Type RS3500.300 helical piles can be determined from the following table.



- P_h = Allowable lateral load
- L = Pile length
- $y_t = \Delta$ = Lateral drift of pile head
- H = Length from pile head to groundline
- D_o = Depth along pile shaft to point of first zero-deflection point ($y_t = 0$), a.k.a. the flexural length of pile
- h = Depth along pile shaft to point of zero slope ($S_t = 0$)
- M = Bending moment in pile shaft

For lateral resistance in non-seismic areas, special analysis and tests of RS3500.300 helical piles indicate allowable lateral load with less than 1/2" lateral deflection in stiff, firm, and soft soils. Test and analysis data are listed in the table. Per AC358 1.4.7, the Shaft Flexural Length (D_o) is measured from the top of the pile down to the first point of zero lateral deflection in the soil. AC358 1.4.8 defines the Shaft Seismic Flexural Length (Seismic D_o) as 120% of the Shaft Flexural Length, which for RS3500.300 helical piles embedded in stiff, firm, and soft soils is 5.8 ft, 5.8 ft, and 8.6 ft respectively.

SOIL TYPE	NON-SEISMIC LATERAL LOADING (SEISMIC DESIGN CATEGORIES A, B, AND C)							SEISMIC LATERAL LOADING (SEISMIC DESIGN CATEGORIES D, E, AND F)						
	ALLOWABLE LATERAL LOAD (P_h) (kip)	DRIFT AT PILE HEAD (y_t) (in)	DEPTH TO SOIL (H) (ft)	DEPTH TO $y_t = 0$ (D_o) (ft)	DEPTH TO $S_t = 0$ (h) (ft)	ALLOWABLE STORY DRIFT (Δ) AT PILE HEAD	BENDING MOMENT IN PILE (kip-in)	ALLOWABLE LATERAL LOAD (P_h) (kip)	DRIFT AT PILE HEAD (y_t) (in)	DEPTH TO SOIL (H) (ft)	DEPTH TO $y_t = 0$ (D_o) (ft)	DEPTH TO $S_t = 0$ (h) (ft)	ALLOWABLE STORY DRIFT (Δ) AT PILE HEAD	BENDING MOMENT IN PILE (kip-in)
Stiff Soil: N = 9 to 20	2.42	0.52	0	4.8	5.1	0.008h	40.8	2.42	1.17	1.0	5.8	6.6	0.015h	64.8
Firm Soil: N = 5 to 8	1.90	0.52	0	4.8	6.1	0.007h	34.8	1.90	1.12	1.0	5.8	7.1	0.013h	52.8
Soft Soil: N = 1 to 4	0.61	0.52	0	7.2	8.8	0.005h	15.6	0.62	1.12	1.4	8.6	10.3	0.009h	25.2

All shaft, coupler, and bracket-tube shapes and materials have been checked for adequate ductility per AC358 3.14.3; AISC 341 Table D1.1 (including applicable footnotes); and IBC 1810.3.6.1.

Lateral deflections during seismic loading should be checked against the ASCE-7 Table 12.12-1 (reproduced below) showing Allowable Story Drift values for various structure lateral load resistance system types that are presented as ratios of the story height in Seismic Design Categories D, E, and F. The story height (h) assumed for the helical pile is the depth from the pile top down to the point of zero pile slope-from-vertical in the soil (S_t). The Allowable Story Drift in the seismic loading table above must be less than or equal to the Allowable Story Drift in Table 12.12.1 in order to determine if RS3500.300 helical piles can be used in that Structure Type and Risk Category.

Structure	Risk Category		
	I or II	III	IV
Structures, other than masonry shear walls, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	0.025h	0.020h	0.015h
Masonry cantilever shear wall structures	0.010h	0.010h	0.010h
Other masonry shear wall structures	0.007h	0.007h	0.007h
All other structures	0.020h	0.015h	0.010h

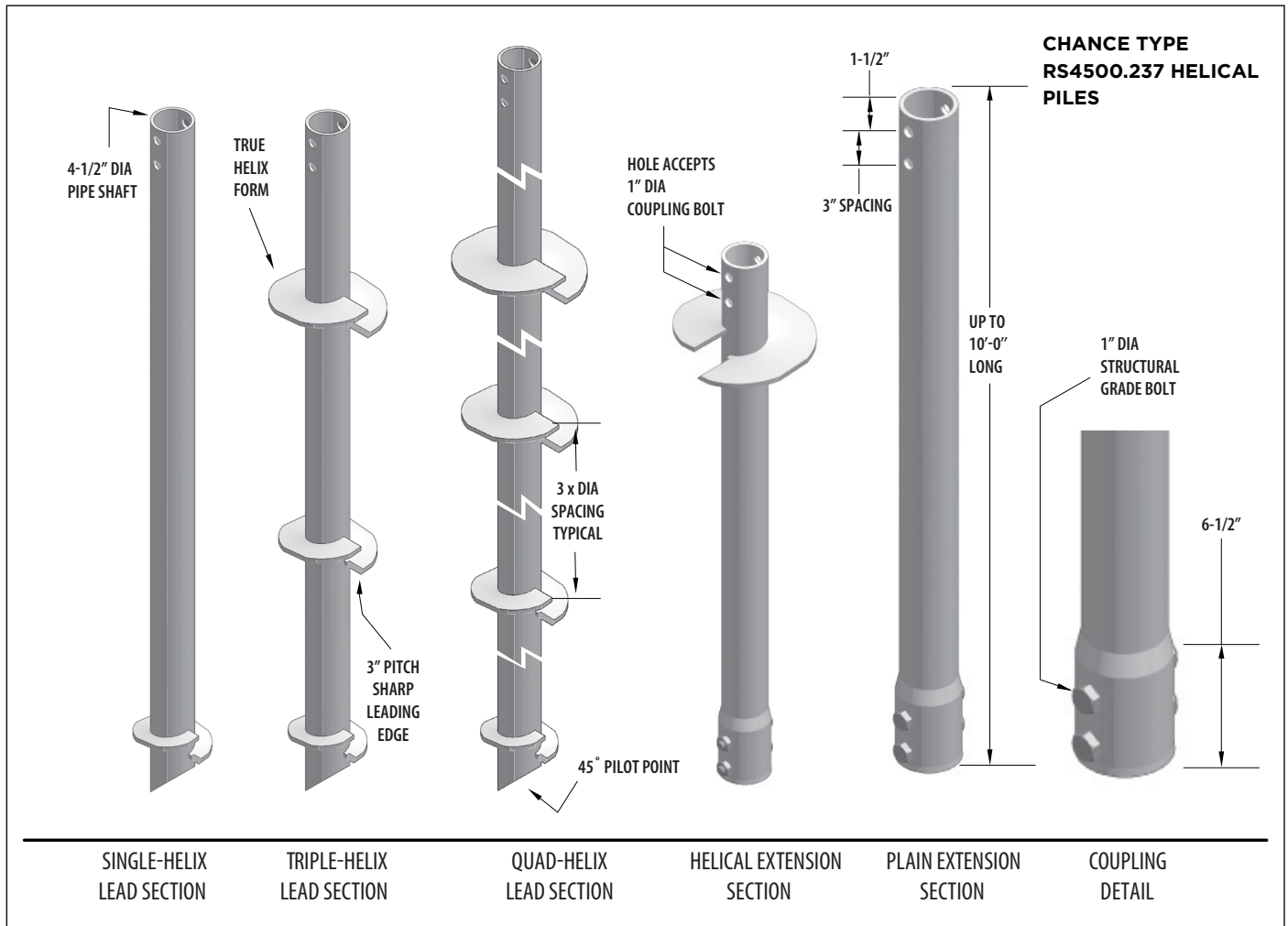
CHANCE TYPE RS4500.237 HELICAL PILES

- 108 kip ULTIMATE - 54 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING - 18,000 ft·lb
- MULTI-PURPOSE 4-1/2" DIAMETER, 0.237" WALL, ROUND HSS SHAFT WITH INTEGRALLY FORMED UPSET SOCKETS

DESCRIPTION:

Chance® Type RS4500.237 helical piles have 108 kip ultimate capacity and 54 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Round shaft helical piles offer increased lateral and buckling resistance compared to solid square shafts with similar torque strength. Strength calculations are based on a design corrosion level of 50 years for most soil conditions.

Chance Type RS helical piles can be coupled with square shaft lead sections (Combo Piles) to provide greater penetration into bearing soils. Chance Type RS helical piles feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.



RS4500.237 HELICAL PILE SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: HSS 4-1/2 inch OD x 0.237 inch (schedule 40) wall steel shaft produced exclusively for Chance® products.

COUPLING: Forged as an integral part of the plain and helical extension material as round deep sockets, connected with multiple structural bolts.

HELIX - 1/2 INCH THICK: ASTM A1018 or A656, with minimum yield strength of 80 ksi.

3 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 8, 10, 12, 14, 16, or 20 inches.

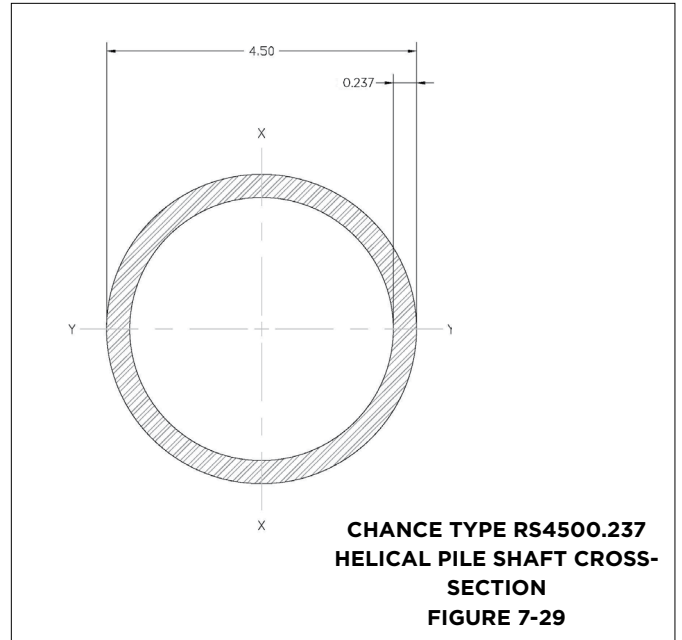
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight sharpened leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

- Single-, double-, triple-, and quad-helix lead sections, 7 and 10 feet long
- Plain extensions, 3-1/2, 5, 7, and 10 feet long
- Extensions with helix plates, 5, 7, and 10 feet long

Helical products are hot dip galvanized per ASTM A123 Grade 75.



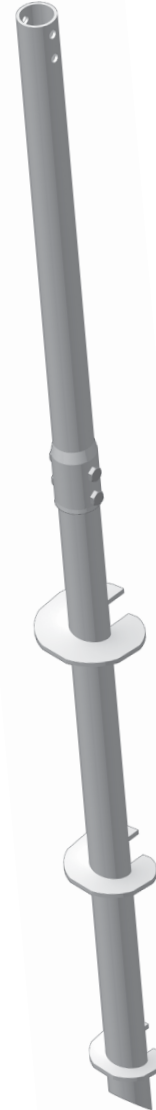
NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.

NOMINAL, LRFD DESIGN, & ASD ALLOWABLE STRENGTHS OF RS4500.237 HELIX PLATES FOR SHAFT AXIAL TENSION & COMPRESSION¹

HELIX DIAMETER (in) [mm]	THICKNESS (in) [mm]	NOMINAL STRENGTH (kip) [kN]	LRFD DESIGN STRENGTH (kip) [kN]	ASD ALLOWABLE STRENGTH (kip) [kN]
8 [200]	0.5 [13]	244.5 [1087.6]	183.4 [815.7]	122.3 [499.5]
10 [250]	0.5 [13]	200.3 [891.0]	150.2 [608.3]	100.2 [445.7]
12 [300]	0.5 [13]	168.5 [749.5]	126.4 [562.1]	84.3 [375.0]
14 [350]	0.5 [13]	133.0 [591.6]	99.8 [443.7]	66.5 [295.8]

Notes:
 For SI: 1 kip = 4.448 kN.
¹Capacities based on a design corrosion level of 50 years.

RS4500.237 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled HSS 4 inch nominal Schedule 40 (0.237 inch nominal wall) per ASTM A500 Grade B/C with 50 ksi minimum yield strength			
SHAFT SIZE, OD	4.5 in	114 mm	CORRODED	
			4.487 in	114 mm
SHAFT SIZE, ID*	4.059 in	103.4 mm	CORRODED	
			4.071 in	103.4 mm
MOMENT OF INERTIA (I)*	6.79 in ⁴	282.6 cm ⁴	CORRODED	
			6.415 in ⁴	267.9 cm ⁴
SHAFT AREA (A)*	2.96 in ²	19.1 cm ²	CORRODED	
			2.786 in ²	18.09 cm ²
SECTION MODULUS (S _{x-x})*	3.02 in ³	49.6 cm ³	CORRODED	
			2.859 in ³	47.0 cm ³
PERIMETER	14.1 in	35.9 cm	CORRODED	
			14.09 in	35.8 cm
COUPLING	Integral forged round deep socket sleeve			
COUPLING BOLTS	Two 1 inch diameter ASTM A193 Grade B7 hex head bolts			
HELIX PLATES	0.5 inch thick, formed on matching metal dies, ASTM A572 Grade 80 or better			
COATINGS	Hot dip galvanized per ASTM A123 Grade 75, 3.0 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	6 ft ⁻¹		20 m ⁻¹	
TORQUE RATING	18,000 ft-lb		31,200 N-m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	120 kip	712 kN	90 kip	534 kN
ALLOWABLE TENSION STRENGTH	60 kip		356 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	108 kip	614 kN	54 kip	307 kN
Note: * Computed with 93% of wall thickness per AISC 360-10, B4.2				


 ASSEMBLY OF RS4500.237
 FIGURE 7-30



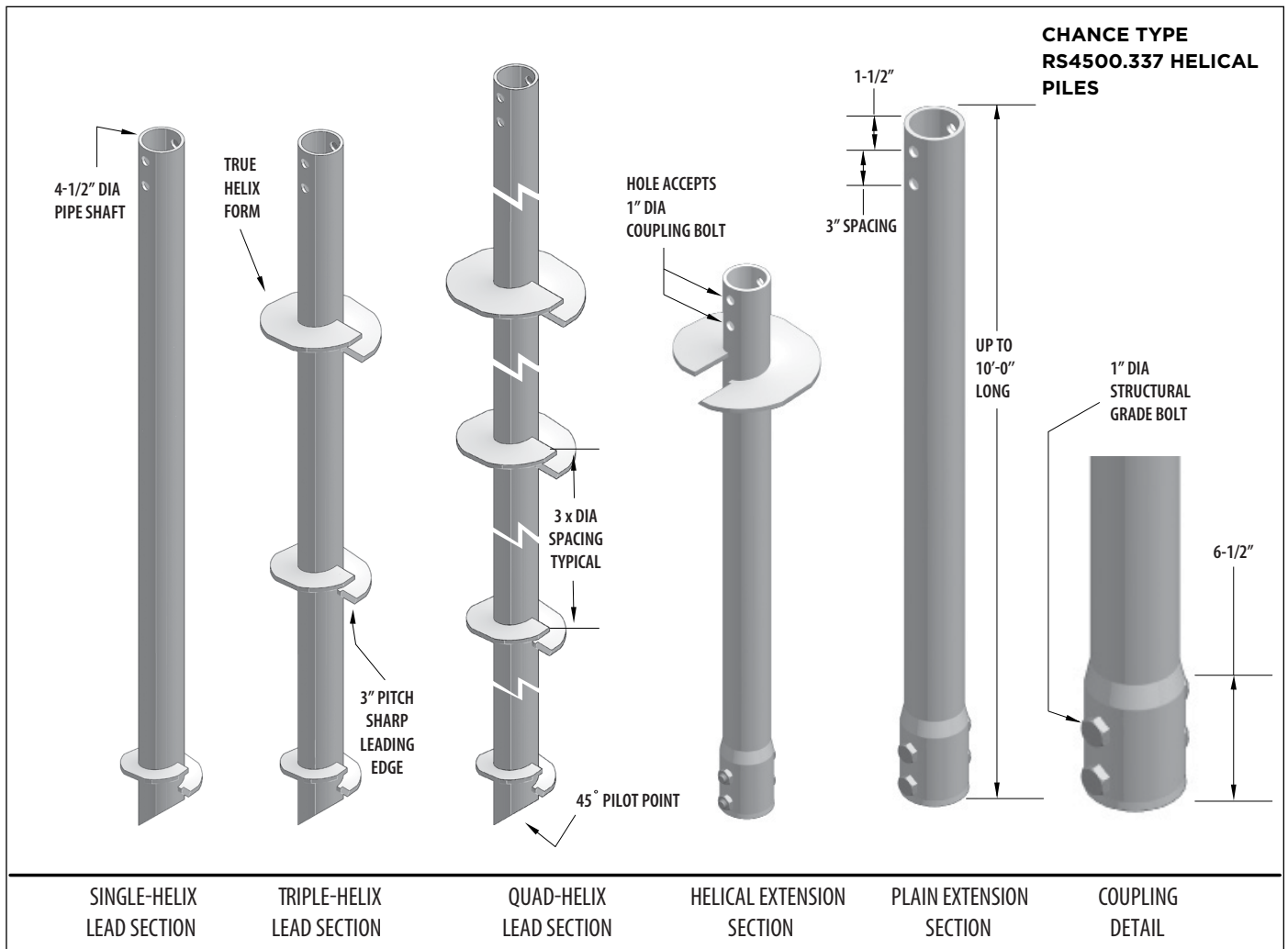
CHANCE TYPE RS4500.337 HELICAL PILES

- 150 kip ULTIMATE - 75 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING - 25,000 ft-lb
- MULTI-PURPOSE 4-1/2" DIAMETER, 0.337" WALL, ROUND HSS SHAFT WITH INTEGRALLY FORMED UPSET SOCKETS

DESCRIPTION:

Chance® Type RS4500.337 helical piles have 150 kip ultimate capacity and 75 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Round shaft helical piles offer increased lateral and buckling resistance compared to solid square shafts with similar torque strength. Strength calculations are based on a design corrosion level of 50 years for most soil conditions. Chance Type

RS helical piles can be coupled with square shaft lead sections (Combo Piles) to provide greater penetration into bearing soils. Chance Type RS helical piles feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.

RS4500.337 HELICAL PILE SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: HSS 4-1/2 inch OD x 0.337 inch (schedule 80) wall steel shaft produced exclusively for Chance® products.

COUPLING: Forged as an integral part of the plain and helical extension material as round deep sockets, connected with multiple structural bolts.

HELIX - 1/2 INCH THICK: ASTM A1018 or A656, with minimum yield strength of 80 ksi.

3 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 8, 10, 12, 14, 16, or 20 inches.

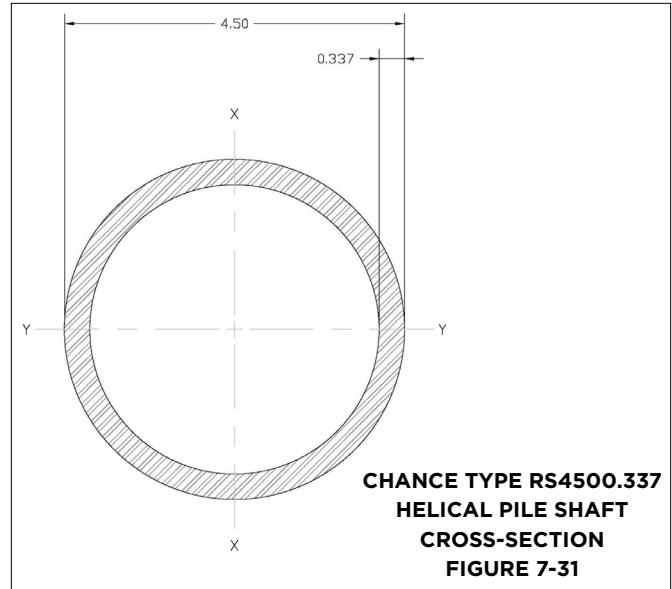
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight sharpened leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

- Single-, double-, triple-, and quad-helix lead sections, 7 and 10 feet long
- Plain extensions, 3-1/2, 5, 7, and 10 feet long
- Extensions with helix plates, 5, 7, and 10 feet long

Helical products are hot dip galvanized per ASTM A123 Grade 75.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.

NOMINAL, LRFD DESIGN, & ASD ALLOWABLE STRENGTHS OF RS4500.337 HELIX PLATES FOR SHAFT AXIAL TENSION & COMPRESSION¹

HELIX DIAMETER (in) [mm]	THICKNESS (in) [mm]	NOMINAL STRENGTH (kip) [kN]	LRFD DESIGN STRENGTH (kip) [kN]	ASD ALLOWABLE STRENGTH (kip) [kN]
8 [200]	0.5 [13]	244.5 [1087.6]	183.4 [815.7]	122.3 [499.5]
10 [250]	0.5 [13]	200.3 [891.0]	150.2 [668.3]	100.2 [445.7]
12 [300]	0.5 [13]	168.5 [749.5]	126.4 [562.1]	84.3 [375.0]
14 [350]	0.5 [13]	133.0 [591.6]	99.75 [443.7]	66.5 [295.8]

Notes:
For SI: 1 kip = 4.448 kN.
¹Capacities based on a design corrosion level of 50 years.

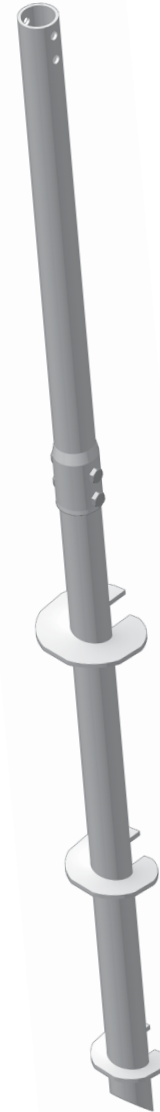
NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS OF CHANCE TYPE RS4500.337 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS (kip) [kN]							
	FIRM SOIL				SOFT SOIL			
	FIXED		PINNED		FIXED		PINNED	
	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN	NOMINAL	DESIGN
Lead, Single Helix	191.7 [852.7]	172.6 [767.7]	186.3 [828.7]	167.7 [746.0]	175.3 [779.8]	157.8 [701.9]	156.3 [695.3]	140.7 [625.9]
	For Nominal, see Helix Strength Table Above for Single 12", & 14" For LRFD design, see Helix Strength Table Above for 10", 12", & 14"						For Single 14" - 133.0 [591.6]	For Single 12" & 14" - see Helix Table Above
Lead, Multi-Helix	191.7 [852.7]	172.6 [767.8]	186.3 [828.7]	167.7 [746.0]	175.3 [779.8]	157.8 [701.9]	156.3 [695.3]	140.7 [625.9]
Extension	191.7 [852.7]	172.6 [767.8]	186.3 [828.7]	167.7 [746.0]				

Notes:
For SI: 1 kip = 4.448 kN.
¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.
² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.



RS4500.337 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled HSS 4 inch nominal Schedule 80 (0.337 inch nominal wall) per ASTM A500 Grade B/C with 50 ksi minimum yield strength			
SHAFT SIZE, OD	4.5 in	114 mm	CORRODED	
			4.487 in	114 mm
SHAFT SIZE, ID*	3.874 in	98.4 mm	CORRODED	
			3.886 in	98.7 mm
MOMENT OF INERTIA (I)*	9.07 in ⁴	377.5 cm ⁴	CORRODED	
			8.701 in ⁴	362.2 cm ⁴
SHAFT AREA (A)*	4.12 in ²	26.6 cm ²	CORRODED	
			3.951 in ²	25.5 cm ²
SECTION MODULUS (S _{x-x})*	4.03 in ³	66.1 cm ³	CORRODED	
			3.878 in ³	63.6 cm ³
PERIMETER	14.1 in	35.9 cm	CORRODED	
			14.09 in	35.8 cm
COUPLING	Integral forged round deep socket sleeve			
COUPLING BOLTS	Two 1 inch diameter ASTM A193 Grade B7 hex head bolts			
HELIX PLATES	0.5 inch thick, formed on matching metal dies, ASTM A572 Grade 80 or better			
COATINGS	Hot dip galvanized per ASTM A123 Grade 75, 3.0 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	6 ft ⁻¹		20 m ⁻¹	
TORQUE RATING	25,000 ft-lb		33,900 N-m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	160 kip	712 kN	120 kip	534 kN
ALLOWABLE TENSION STRENGTH	80 kip		356 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	150 kip	667 kN	75 kip	334 kN
Note: *Computed with 93% of wall thickness per AISC 360-10, B4.2				



**ASSEMBLY OF RS4500.337
FIGURE 7-32**

ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE RS4500.337 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

SECTION TYPE & HELIX COUNT	ASD ALLOWABLE AXIAL COMPRESSION STRENGTH (kip) [kN]			
	FIRM SOIL		SOFT SOIL	
	FIXED	PINNED	FIXED	PINNED
Lead, Single Helix	For Single 8" - 114.8 [551.7]	For Single 8" - 111.6 [496.4]	105.0 [467.1]	93.6 [416.4]
	See Helix Strength Table Above for 10", 12", & 14"	See Helix Strength Table Above for 10", 12", & 14"	See Helix Strength Table Above for 10", 12", & 14"	See Helix Strength Table Above for 12" & 14"
Lead, 2-Helix 8"-10"	114.8 [551.7]	111.6 [496.4]	105.0 [467.1]	93.6 [416.4]
Lead, 2-Helix 10"-12"				
Lead, 2-Helix 12"-14"				
Lead, 2-Helix 14"-14"	114.8 [551.7]	111.6 [496.4]	105.0 [467.1]	93.6 [416.4]
Lead, Multi-Helix	114.8 [551.7]	111.6 [496.4]	105.0 [467.1]	93.6 [416.4]
Extension	114.8 [551.7]	111.6 [496.4]	105.0 [467.1]	93.6 [416.4]

Notes:
 For SI: 1 kip = 4.448 kN.
¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.
² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.

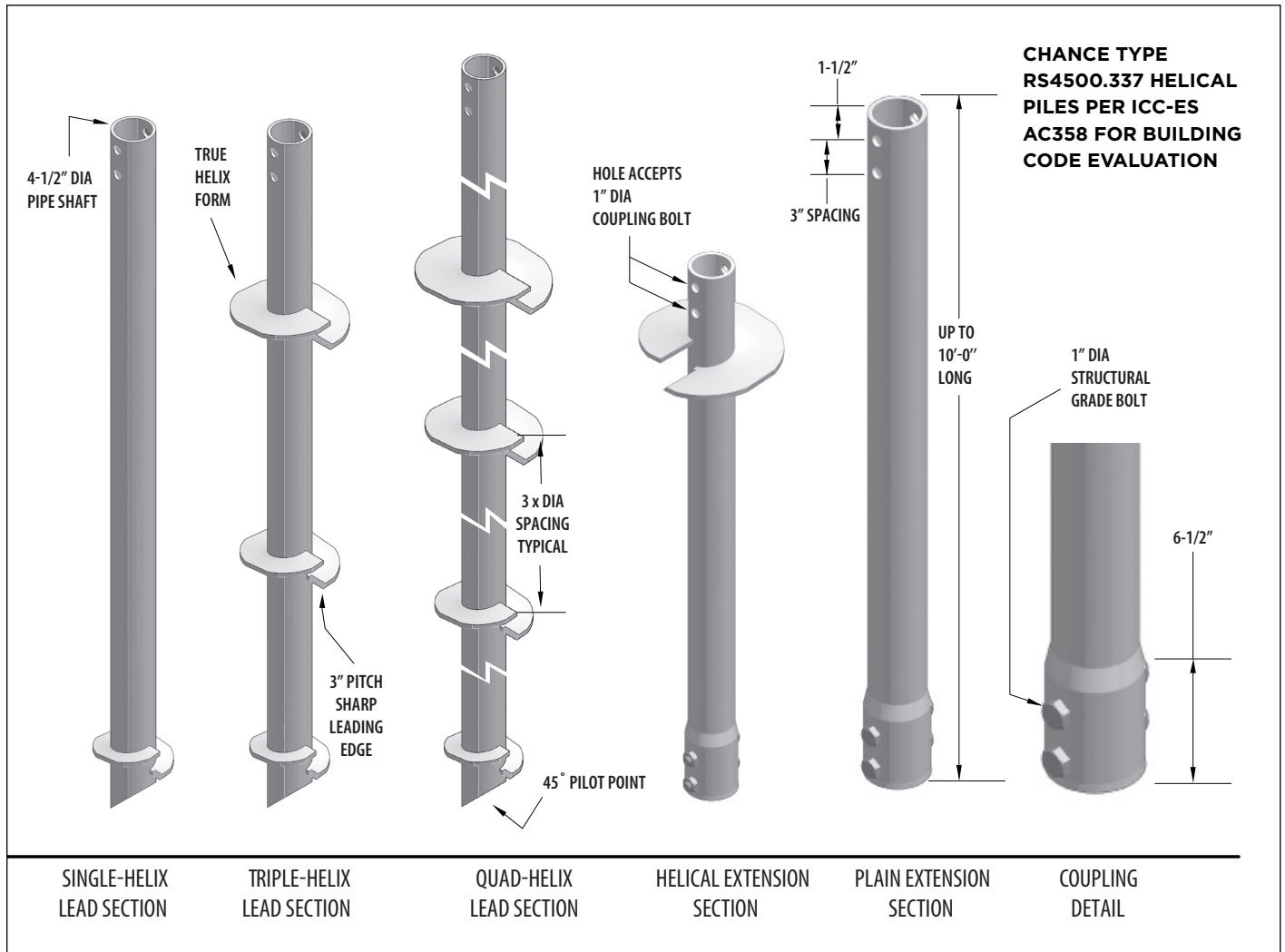
CHANCE TYPE RS4500.337 HELICAL PILES PER ICC-ES AC358 FOR BUILDING CODE EVALUATION

- 136.1 kip ULTIMATE - 68.1 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING - 24,300 ft-lb
- MULTI-PURPOSE 4-1/2" DIAMETER, 0.337" WALL, ROUND HSS SHAFT WITH INTEGRALLY FORMED UPSET SOCKETS

DESCRIPTION:

Chance® Type RS4500.337 helical piles have 136.1 kip ultimate capacity and 68.1 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Round shaft helical piles offer increased lateral and buckling resistance compared to solid square shafts with similar torque strength. Strength calculations are based on a design corrosion level of 50 years for most soil conditions.

Chance Type RS helical piles can be coupled with square shaft lead sections (Combo Piles) to provide greater penetration into bearing soils. Chance Type RS helical piles feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.



RS4500.337 BUILDING CODE HELICAL PILE SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: HSS 4-1/2 inch OD x 0.337 inch (schedule 80) wall steel shaft produced exclusively for Chance® products.

COUPLING: Forged as an integral part of the plain and helical extension material as round deep sockets, connected with multiple structural bolts.

HELIX - 1/2 INCH THICK: ASTM A1018 or A656, with minimum yield strength of 80 ksi.

3 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 8, 10, 12, 14, 16, or 20 inches.

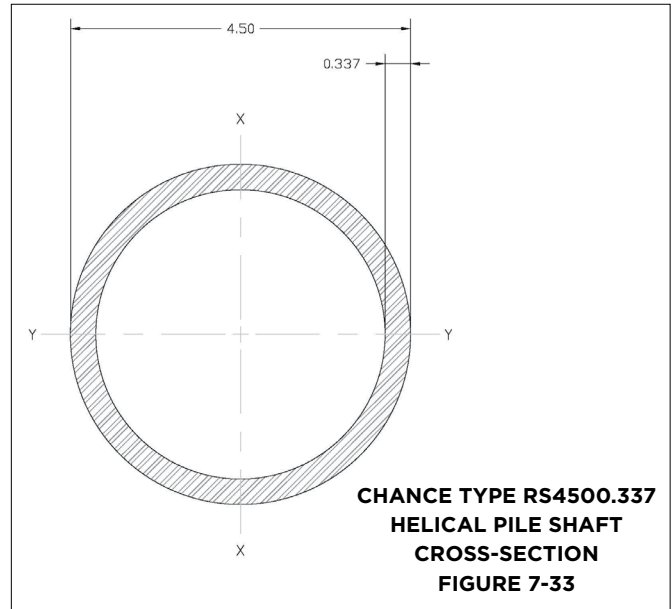
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight sharpened leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

- Single-, double-, triple-, and quad-helix lead sections, 7 and 10 feet long
- Plain extensions, 3-1/2, 5, 7, and 10 feet long
- Extensions with helix plates, 5, 7, and 10 feet long

Helical products are hot dip galvanized per ASTM A123 Grade 75.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.

NOMINAL, LRFD DESIGN, & ASD ALLOWABLE STRENGTHS OF RS4500.337 HELIX PLATES FOR SHAFT AXIAL TENSION & COMPRESSION¹

HELIX DIAMETER (in) {mm}	THICKNESS (in) {mm}	NOMINAL STRENGTH (kip) {kN}	LRFD DESIGN STRENGTH (kip) {kN}	ASD ALLOWABLE STRENGTH (kip) {kN}
8 {200}	0.5 {13}	180.2 {801.6}	135.2 {601.4}	90.2 {401.2}
10 {250}	0.5 {13}	180.2 {801.6}	135.2 {601.4}	90.2 {401.2}
12 {300}	0.5 {13}	180.2 {801.6}	135.2 {601.4}	90.2 {401.2}
14 {350}	0.5 {13}	180.2 {801.6}	135.2 {601.4}	90.2 {401.2}

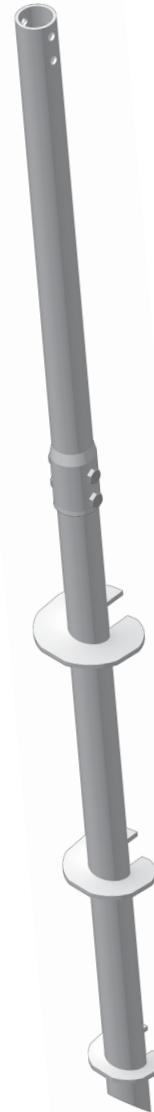
Notes:
 For SI: 1 kip = 4.448 kN.
¹Capacities based on a design corrosion level of 50 years.

NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS OF CHANCE TYPE RS4500.337 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2,3,4}

SECTION TYPE & HELIX COUNT	NOMINAL & LRFD DESIGN COMPRESSION STRENGTHS (kip) {kN}							
	FIRM SOIL				SOFT SOIL			
	FIXED		PINNED		FIXED		PINNED	
	NOMINAL	DESIGN ³	NOMINAL	DESIGN ³	NOMINAL	DESIGN ³	NOMINAL	DESIGN
Lead, Single Helix	See Helix Strength Table Above				175.3 {779.8}	135.2 {601.4}	156.3 {695.3}	135.2 {601.4}
Lead, Multi-Helix	191.7 {852.7}	143.33 {646.5}	186.3 {828.7}	145.33 {646.57}	175.3 {779.8}	145.33 {646.57}	156.3 {695.3}	140.67 {625.7}
Extension	[189.2 {841.6}]	[145.3 {646.3}]	[181.6 {807.8}]	[145.3 {646.3}]	[166.3 {739.7}]	[145.3 {646.3}]	[141.0 {627.2}]	[126.9 {564.5}]

Notes:
 For SI: 1 kip = 4.448 kN.
¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.
² Strength ratings include an allowance for corrosion over a 50-year service life and presume the supported structure is braced in accordance with IBC Section 1810.2.2 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.
³ Limited by coupling bolt shear.
⁴ Values in brackets [] are for use in Seismic Design Categories D, E, and F.

RS4500.337 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled HSS 4 inch nominal Schedule 80 (0.337 inch nominal wall) per ASTM A500 Grade B/C with 50 ksi minimum yield strength			
SHAFT SIZE, OD	4.5 in	114 mm	CORRODED	
			4.487 in	114 mm
SHAFT SIZE, ID*	3.874 in	98.4 mm	CORRODED	
			3.886 in	98.7 mm
MOMENT OF INERTIA (I)*	9.07 in ⁴	377.5 cm ⁴	CORRODED	
			8.701 in ⁴	362.2 cm ⁴
SHAFT AREA (A)*	4.12 in ²	26.6 cm ²	CORRODED	
			3.951 in ²	25.5 cm ²
SECTION MODULUS (S _{x-x})*	4.03 in ³	66.1 cm ³	CORRODED	
			3.878 in ³	63.6 cm ³
PERIMETER	14.1 in	35.9 cm	CORRODED	
			14.09 in	35.8 cm
COUPLING	Integral forged round deep socket sleeve			
COUPLING BOLTS	Two 1 inch diameter ASTM A193 Grade B7 hex head bolts			
HELIX PLATES	0.5 inch thick, formed on matching metal dies, ASTM A572 Grade 80 or better			
COATINGS	Hot dip galvanized per ASTM A123 Grade 75, 3.0 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR [†]	5.6 ft ⁻¹		18.5 m ⁻¹	
TORQUE RATING	24,300 ft-lb		32,946 N-m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	150 kip	667.2 kN	112.5 kip	500.2 kN
ALLOWABLE TENSION STRENGTH	75 kip		333.6 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	136.1 kip	605.4 kN	68.1 kip	302.9 kN
Notes: * Computed with 93% of wall thickness per AISC 360-10, B4.2 † Per ICC-ES AC308 Section 3.13.1.1				



ASSEMBLY OF RS4500.337
FIGURE 7-34

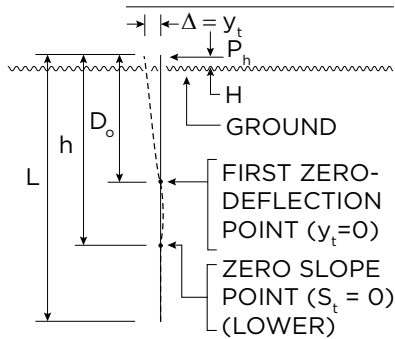
ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE RS4500.337 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2,3,4}

SECTION TYPE & HELIX COUNT	ASD ALLOWABLE AXIAL COMPRESSION STRENGTH (kip) {kN}			
	FIRM SOIL		SOFT SOIL	
	FIXED	PINNED	FIXED	PINNED
Lead, Single Helix	See Helix Strength Table Above for 8", 10", 12", & 14"	See Helix Strength Table Above for 8", 10", 12", & 14"	See Helix Strength Table Above for 8", 10", 12", & 14"	90.2 {401.2} [84.4 {375.5}]
Lead, 2-Helix 8"-10"	96.9 {431} [96.9 {431.0}]	96.9 {431} [96.9 {431.0}]	96.9 {431} [96.9 {431.0}]	93.6 {416.4} [84.4 {375.5}]
Lead, 2-Helix 10"-12"				
Lead, 2-Helix 12"-14"				
Lead, 2-Helix 14"-14"				
Lead, Multi-Helix	96.9 {431} [96.9 {431.0}]	96.9 {431} [96.9 {431.0}]	96.9 {431} [96.9 {431.0}]	93.6 {416.4} [84.4 {375.5}]
Extension	96.9 {431} [96.9 {431.0}]	96.9 {431} [96.9 {431.0}]	96.9 {431} [96.9 {431.0}]	93.6 {416.4} [84.4 {375.5}]

Notes: For SI: 1 kip = 4.448 kN.
¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.
² Strength ratings include an allowance for corrosion over a 50-year service life and presume the supported structure is braced in accordance with IBC Section 1810.2.2 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.
³ Limited by coupling bolt shear.
⁴ Values in brackets [] are for use in Seismic Design Categories D, E, and F.



RS4500.337 LATERAL DEFLECTION ANALYSIS FOR SEISMIC DESIGN CATEGORIES D, E, & F: The free-headed pile lateral movements at allowable loads (lateral only – no vertical) of Chance® Type RS4500.337 helical piles can be determined from the following table.



- P_h = Allowable lateral load
- L = Pile length
- $y_t = \Delta$ = Lateral drift of pile head
- H = Length from pile head to groundline
- D_o = Depth along pile shaft to point of first zero-deflection point.
- $(y_t = 0)$, a.k.a. the flexural length of pile
- h = Depth along pile shaft to point of zero slope ($S_t = 0$)
- M = Bending moment in pile shaft

For lateral resistance in non-seismic areas, special analysis and tests of RS4500.337 helical piles indicate allowable lateral load with less than 1/2" lateral deflection in stiff, firm, and soft soils. Test and analysis data are listed in the table. Per AC358 1.4.7, the Shaft Flexural Length (D_o) is measured from the top of the pile down to the first point of zero lateral deflection in the soil. AC358 1.4.8 defines the Shaft Seismic Flexural Length (Seismic D_o) as 120% of the Shaft Flexural Length, which for RS4500.337 helical piles embedded in stiff, firm, and soft soils is 6.4 ft, 6.7 ft, and 10 ft respectively.

SOIL TYPE	NON-SEISMIC LATERAL LOADING (SEISMIC DESIGN CATEGORIES A, B, AND C)				SEISMIC LATERAL LOADING (SEISMIC DESIGN CATEGORIES D, E, and F)									
	ALLOWABLE LATERAL LOAD (P_h) (kip)	DRIFT AT PILE HEAD (y_t) (in)	DEPTH TO SOIL (H) (ft)	DEPTH TO $y_t = 0$ (D_o) (ft)	DEPTH TO $S_t = 0$ (h) (ft)	ALLOWABLE STORY DRIFT (Δ) AT PILE HEAD	BENDING MOMENT IN PILE (kip-in)	ALLOWABLE LATERAL LOAD (P_h) (kip)	DRIFT AT PILE HEAD (y_t) (in)	DEPTH TO SOIL (H) (ft)	DEPTH TO $y_t = 0$ (D_o) (ft)	DEPTH TO $S_t = 0$ (h) (ft)	ALLOWABLE STORY DRIFT (Δ) AT PILE HEAD	BENDING MOMENT IN PILE (kip-in)
Stiff Soil: N = 9 to 20	3.66	0.42	0	5.3	6.1	0.006h	67.2	3.66	0.97	1.1	6.4	7.2	0.011h	108.0
Firm Soil: N = 5 to 8	2.36	0.42	0	5.6	7.1	0.005h	50.4	2.36	0.87	1.1	6.7	8.3	0.009h	76.8
Soft Soil: N = 1 to 4	0.78	0.42	0	8.3	10.1	0.003h	24.0	0.78	0.90	1.7	10.0	11.9	0.006h	37.2

All shaft, coupler, and bracket-tube shapes and materials have been checked for adequate ductility per AC358 3.14.3; AISC 341 Table D1.1 (including applicable footnotes); and IBC 1810.3.6.1.

Lateral deflections during seismic loading should be checked against the ASCE-7 Table 12.12-1 (reproduced below) showing Allowable Story Drift values for various structure lateral load resistance system types that are presented as ratios of the story height in Seismic Design Categories D, E, and F. The story height (h) assumed for the helical pile is the depth from the pile top down to the point of zero pile slope-from-vertical in the soil (S_t). The Allowable Story Drift in the seismic loading table above must be less than or equal to the Allowable Story Drift in Table 12.12.1 in order to determine if RS4500.337 helical piles can be used in that Structure Type and Risk Category.

Structure	Risk Category		
	I or II	III	IV
Structures, other than masonry shear walls, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	0.025h	0.020h	0.015h
Masonry cantilever shear wall structures	0.010h	0.010h	0.010h
Other masonry shear wall structures	0.007h	0.007h	0.007h
All other structures	0.020h	0.015h	0.010h

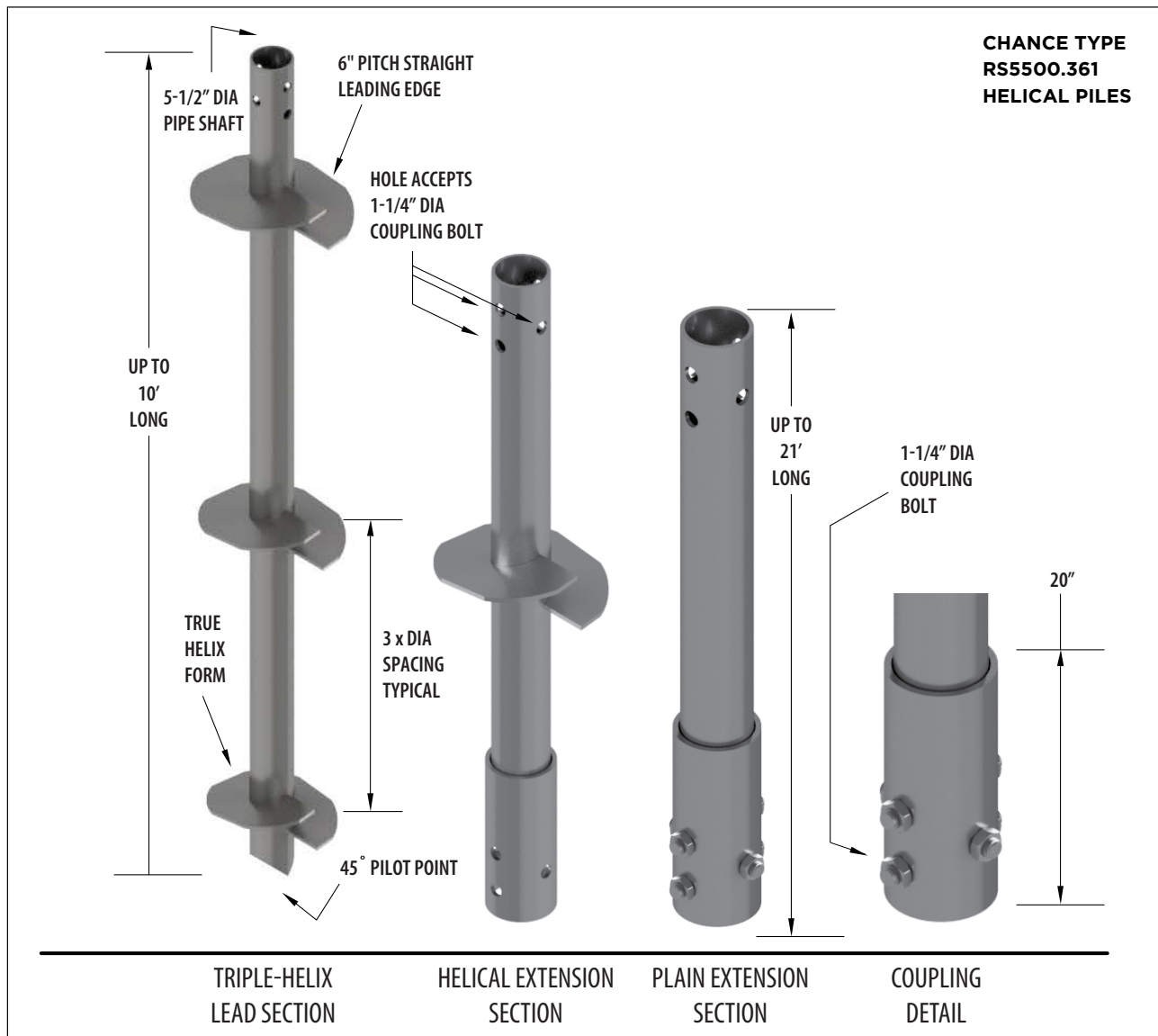
CHANCE TYPE RS5500.361 HELICAL PILES

- 280 kip ULTIMATE - 140 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING - 56,000 ft-lb
- MULTI-PURPOSE 5-1/2" DIA, 0.361" WALL, ROUND HSS SHAFT WITH SLEEVE COUPLINGS

DESCRIPTION:

Chance® Type RS5500.361 helical piles have 280 kip ultimate capacity and 140 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Round shaft helical piles offer increased lateral and buckling resistance compared to solid square

shafts with similar torque strength. Strength calculations are based on a design corrosion level of 50 years for most soil conditions. Chance Type RS helical piles feature straight-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.



RS5500.361 HELICAL PILE SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: HSS 5-1/2 inch OD x 0.361 inch wall steel shaft produced exclusively for CHANCE products.

COUPLING: Welded sleeve forming a socket, connected with multiple structural grade bolts.

HELIX - 1/2, 5/8, AND 3/4 INCH THICK: ASTM A572 with minimum yield strength of 50 ksi.

6 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for larger diameter helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 12, 14, 16, and 18 inches.

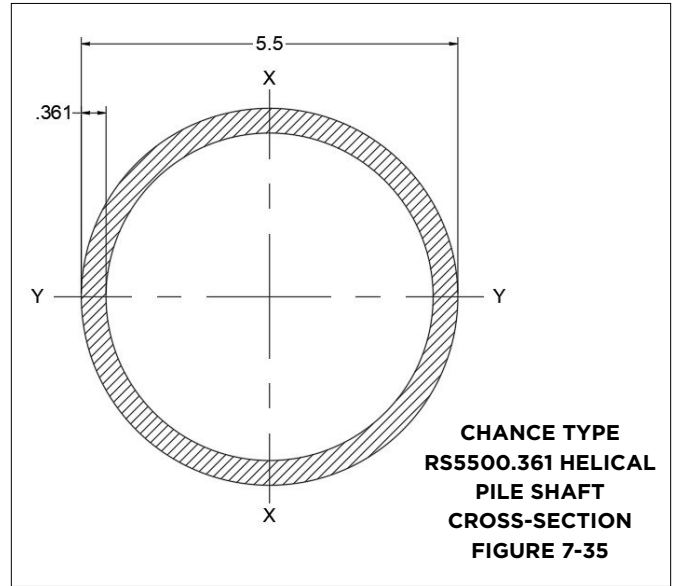
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight leading edge.

CONFIGURATIONS:

- Single-, double-, and triple-helix lead sections, 10 feet long
- Plain extensions, 5, 7, 10, 15, and 21 feet long
- Extensions with helix plates, 5 feet long

Helical products are hot dip galvanized per ASTM A123 Grade 75 or are available black.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.

RS5500.361 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled HSS 5.5 inch, 0.361 inch nominal wall with 80 ksi minimum yield strength			
SHAFT SIZE, OD	5.5 in	140 mm	CORRODED	
			5.487 in	139.4 mm
SHAFT SIZE, ID*	4.83 in	122.7 mm	CORRODED	
			4.842 in	122.9 mm
MOMENT OF INERTIA (I)*	18.23 in ⁴	758.79 cm ⁴	CORRODED	
			17.51 in ⁴	728.82 cm ⁴
SHAFT AREA (A)*	5.45 in ²	35.2 cm ²	CORRODED	
			5.23 in ²	33.74 cm ²
SECTION MODULUS (S _{x-x})*	6.63 in ³	108.6 cm ³	CORRODED	
			6.38 in ³	104.5 cm ³
PERIMETER	17.3 in	44 cm	CORRODED	
			17.24 in	43.8 cm
COUPLING	Welded round deep socket			
COUPLING BOLTS	Three 1-1/4 inch diameter SAE J429 Grade 5 hex head bolts			
HELIX PLATES	0.5 - 0.625-inch thick, formed on matching metal dies, ASTM A572 Grade 50 or better			
COATINGS	Hot dip galvanized per ASTM A123 Grade 75, 3.0 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	5 ft ⁻¹		16.5 m ⁻¹	
TORQUE RATING	56,000 ft-lb		76,000 N-m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	280 kip	1245 kN	210 kip	934 kN
ALLOWABLE TENSION STRENGTH	140 kip		623 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	280 kip	1245 kN	140 kip	623 kN
Note: * Computed with 93% of wall thickness per AISC 360-10, B4.2				



ASSEMBLY OF
RS5500.361
FIGURE 7-36



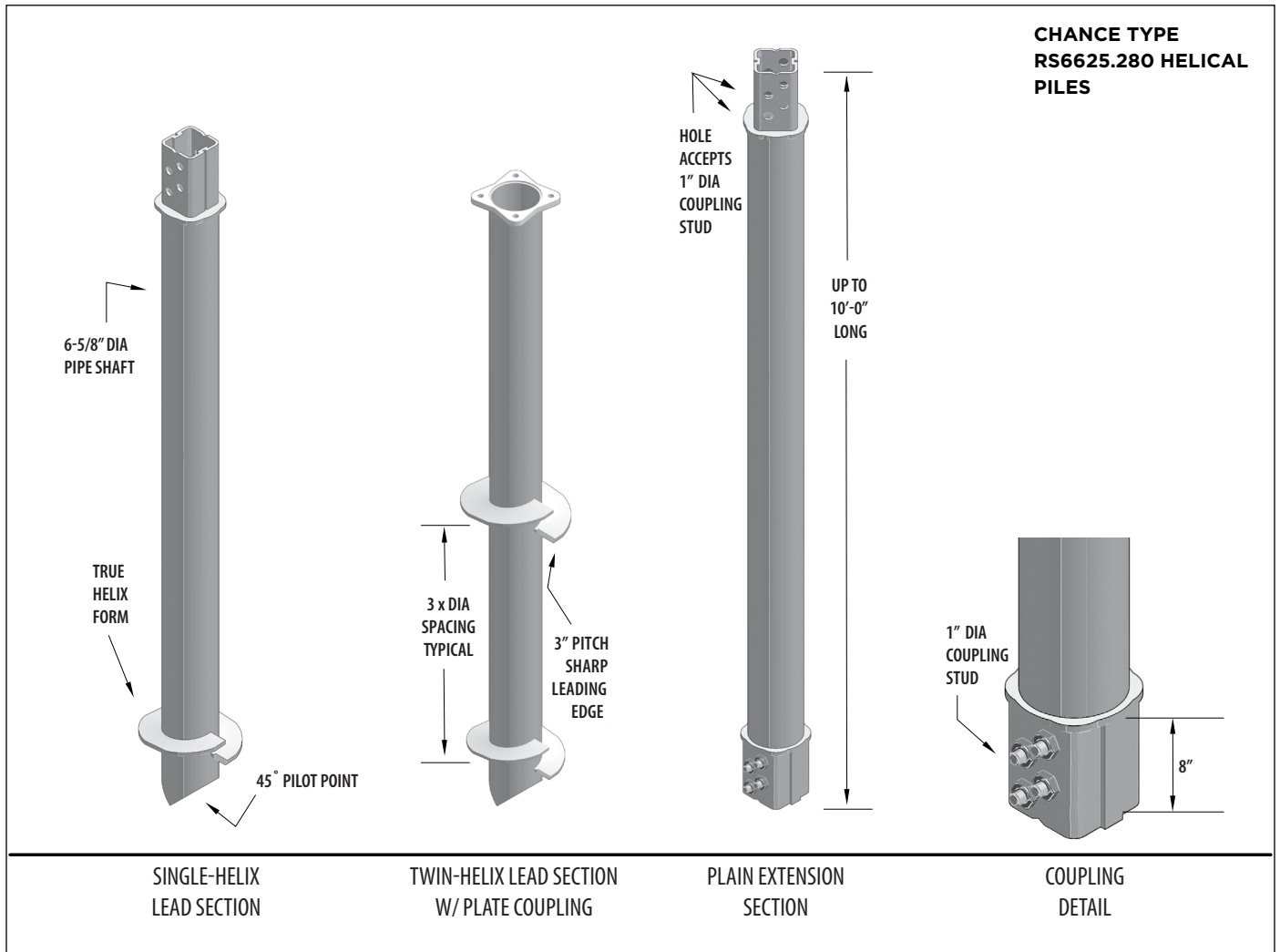
CHANCE TYPE RS6625.280 HELICAL PILES

- 200 kip ULTIMATE - 100 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING - 40,000 ft-lb
- MULTI-PURPOSE 6-5/8" DIAMETER, 0.280" WALL, ROUND HSS SHAFT WITH WELDED SQUARE FORMED COUPLINGS

DESCRIPTION:

Chance® Type RS6625.280 helical piles have 200 kip ultimate capacity and 100 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Round shaft helical piles offer increased lateral and buckling resistance compared to solid square shafts with similar torque strength. Strength calculations are based on a design corrosion level of 50 years for most soil conditions.

Chance Type RS helical piles can be coupled with square shaft lead sections (Combo Piles) to provide greater penetration into bearing soils. Chance Type RS helical piles feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.

RS6625.280 HELICAL PILE SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: HSS 6-5/8 inch OD x 0.280 inch (schedule 40) wall steel shaft produced exclusively for Chance® products.

COUPLING: Formed and welded as a deep square socket, connected with multiple threaded studs & nuts.

HELIX - 1/2 INCH THICK: ASTM A1018 or A656, with minimum yield strength of 80 ksi.

3 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 12, 14, and 16 inches.

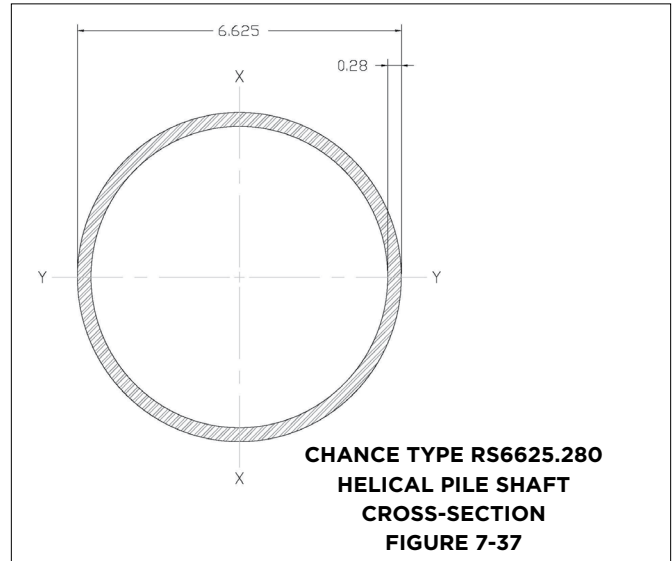
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight sharpened leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

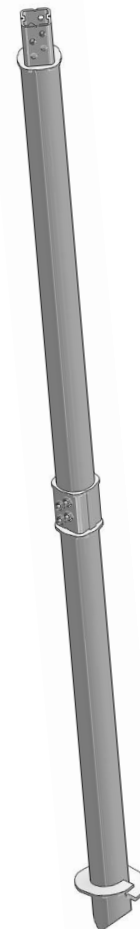
- Single-, double-, and triple-helix, lead sections, 7, 10, and 15 feet long
- Plain extensions, 5, 7, and 10 feet long
- Extensions with helix plates, 5, 7 and 10 feet long

Helical products are hot dip galvanized per ASTM A123 Grade 75.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.

RS6625.280 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled HSS 6 inch nominal Schedule 40 (0.280 inch nominal wall) per ASTM A500 Grade B/C with 50 ksi minimum yield strength			
SHAFT SIZE, OD	6.625 in	168 mm	CORRODED	
			6.612 in	167.95 mm
SHAFT SIZE, ID*	6.10 in	155.1 mm	CORRODED	
			6.118 in	155.4 mm
MOMENT OF INERTIA (I)*	26.37 in ⁴	1096.1 cm ⁴	CORRODED	
			25.05 in ⁴	1041.2 cm ⁴
SHAFT AREA (A)*	5.2 in ²	33.55 cm ²	CORRODED	
			4.94 in ²	31.9 cm ²
SECTION MODULUS (S _{x-x})*	7.96 in ³	130.2 cm ³	CORRODED	
			7.58 in ³	124 cm ³
PERIMETER	20.8 in	52.8 cm	CORRODED	
			20.77 in	52.7 cm
COUPLING	Formed and welded square socket			
COUPLING BOLTS	Four 1 inch diameter Grade 2 studs			
HELIX PLATES	0.5 inch thick, formed on matching metal dies, ASTM A572 Grade 80 or better			
COATINGS	Hot dip galvanized per ASTM A123 Grade 75, 3.0 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	5 ft ⁻¹		16.5 m ⁻¹	
TORQUE RATING	40,000 ft-lb		54,233 N-m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	200 kip	890 kN	150 kip	667 kN
ALLOWABLE TENSION STRENGTH	100 kip		445 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	200 kip	890 kN	100 kip	445 kN
Note: * Computed with 93% of wall thickness per AISC 360-10, B4.2				



ASSEMBLY OF RS6625.280 FIGURE 7-38



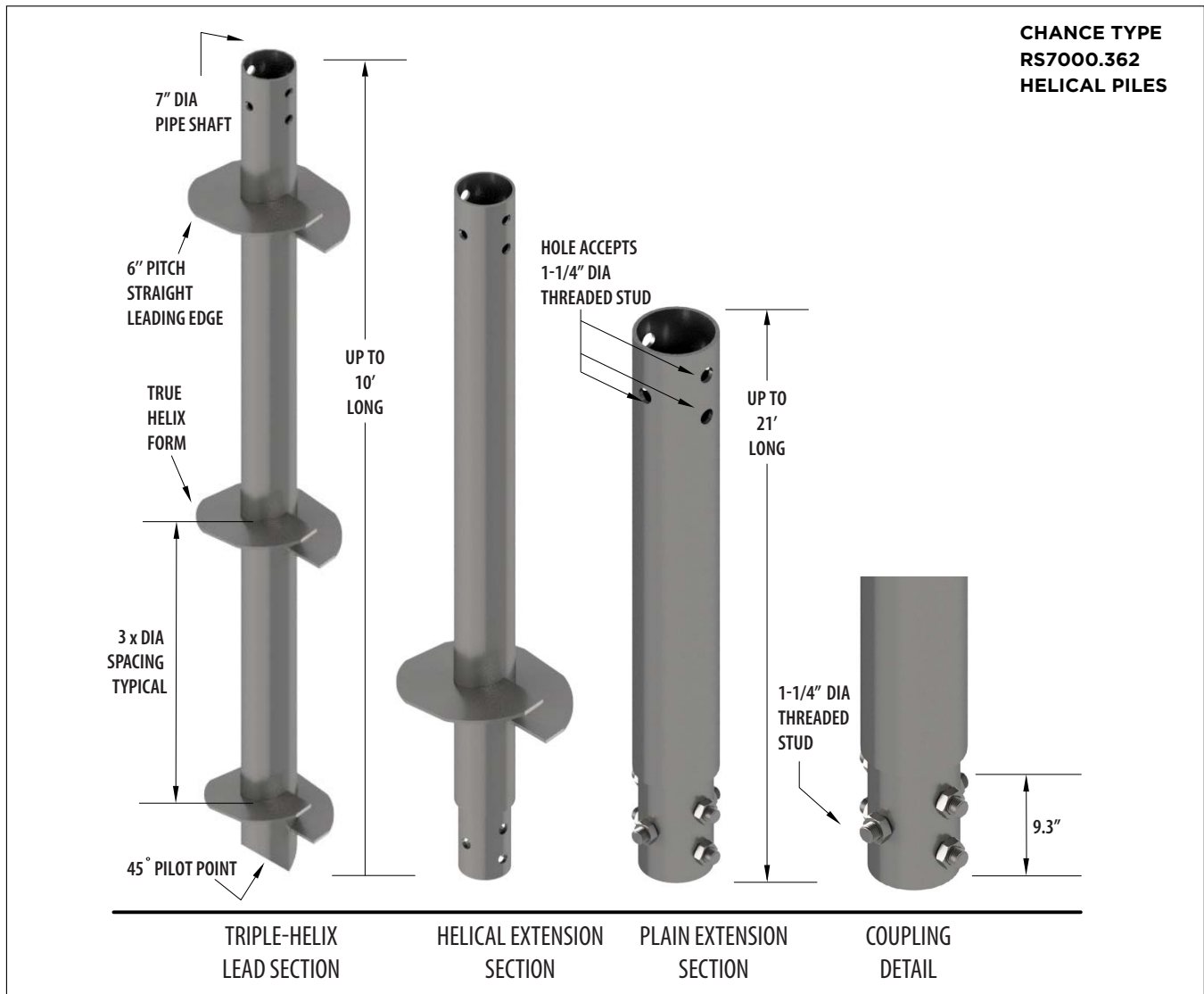
CHANCE TYPE RS7000.362 HELICAL PILES

- 360 kip ULTIMATE - 180 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING - 90,000 ft-lb
- MULTI-PURPOSE 7" DIAMETER, 0.362" WALL, ROUND HSS SHAFT WITH SLEEVE COUPLINGS

DESCRIPTION:

Chance® Type RS7000.362 helical piles have 360 kip ultimate capacity and 180 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Round shaft helical piles offer increased lateral and buckling resistance compared to solid square

shafts with similar torque strength. Strength calculations are based on a design corrosion level of 50 years for most soil conditions. Chance Type RS helical piles feature straight-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.

RS7000.362 HELICAL PILE SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: HSS 7 inch OD x 0.362 inch wall steel shaft produced exclusively for Chance® products.

COUPLING: Welded internal sleeve forming a flush fit joint, connected with multiple structural grade bolts.

HELIX - 1/2 AND 5/8 INCH THICK: ASTM A656 with minimum yield strength of 80 ksi, and ASTM A572 with minimum yield strength of 50 ksi, depending on helix diameter.

6 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for larger diameter helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 16, 18, 20, 22, 24 and 26 inches.

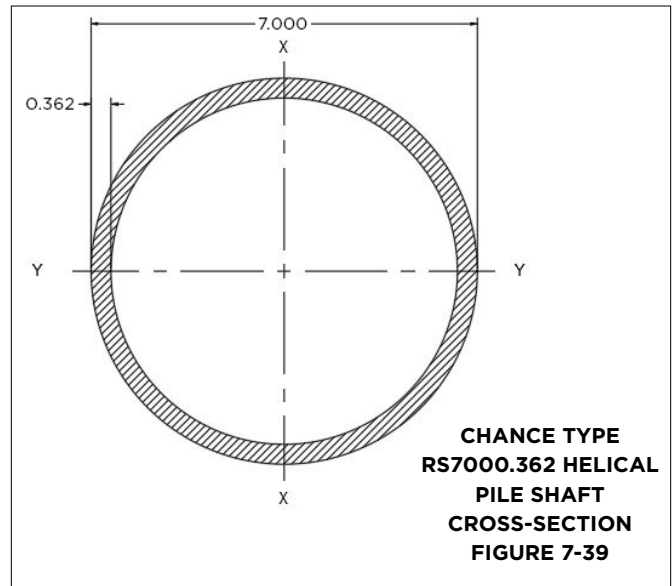
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight leading edge.

CONFIGURATIONS:

- Single-, double-, and triple-helix, lead sections, 10 feet long
- Plain extensions, 3-1/2, 5, 7, 10, 15, and 21 feet long
- Extensions with helix plates, 5 and 7 feet long

Helical products are hot dip galvanized per ASTM A123 Grade 75 or are available black.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.



RS7000.362 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled HSS 7 inch, 0.362 inch nominal wall with 80 ksi minimum yield strength			
SHAFT SIZE, OD	7 in	177.8 mm	CORRODED	
			6.987 in	177.5 mm
SHAFT SIZE, ID*	6.33 in	160.8 mm	CORRODED	
			6.342 in	161.1 mm
MOMENT OF INERTIA (I)*	39.2 in ⁴	1631.6 cm ⁴	CORRODED	
			37.58 in ⁴	1564.2 cm ⁴
SHAFT AREA (A)*	7.05 in ²	45.5 cm ²	CORRODED	
			6.75 in ²	43.5 cm ²
SECTION MODULUS (S _{x-x})*	11.2 in ³	183.5 cm ³	CORRODED	
			10.76 in ³	176.3 cm ³
PERIMETER	22 in	55.9 cm	CORRODED	
			21.95 in	55.7 cm
COUPLING	Welded internal sleeve flush fit joint			
COUPLING BOLTS	Three 1-1/4 inch diameter ASTM A354 Grade BD threaded studs			
HELIX PLATES	0.5 - 0.625-inch thick, formed on matching metal dies, ASTM A572 Grade 50 & A656 Grade 80 or better			
COATINGS	Hot dip galvanized per ASTM A123 Grade 75, 3.0 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	4 ft ⁻¹		13.2 m ⁻¹	
TORQUE RATING	90,000 ft-lb		122,000 N-m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	360 kip	1601 kN	270 kip	1201 kN
ALLOWABLE TENSION STRENGTH	180 kip		801 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	360 kip	1601 kN	180 kip	801 kN
Note: * Computed with 93% of wall thickness per AISC 360-10, B4.2				



**ASSEMBLY OF
RS7000.362
FIGURE 7-40**

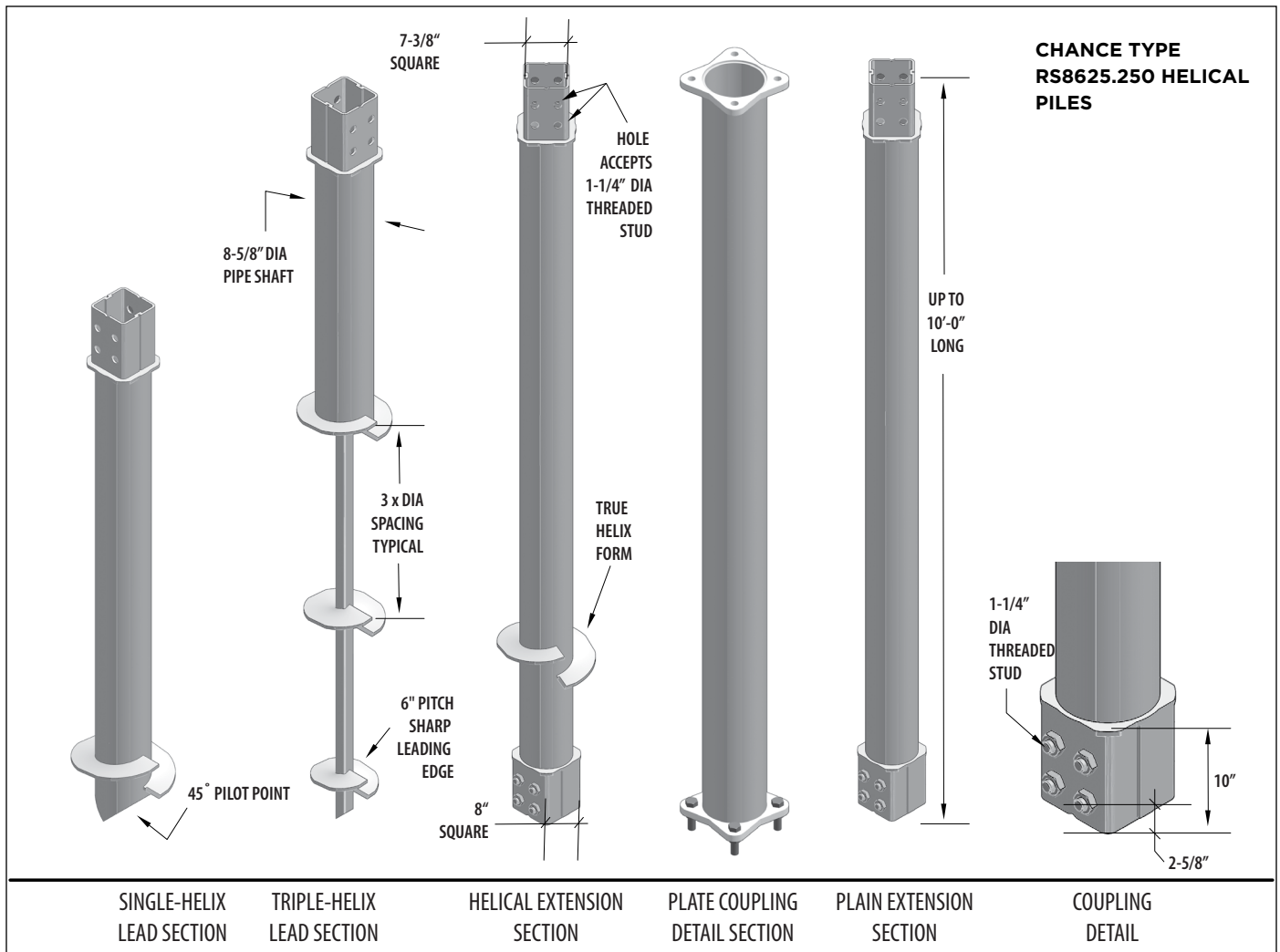
CHANCE TYPE RS8625.250 HELICAL PILES

- 300 kip ULTIMATE – 150 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING – 60,000 ft-lb
- MULTI-PURPOSE 8-5/8" DIAMETER, 0.250" WALL, ROUND HSS SHAFT WITH WELDED SQUARE FORMED COUPLINGS

DESCRIPTION:

Chance® Type RS8625.250 helical piles have 300 kip ultimate capacity and 150 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Round shaft helical piles offer increased lateral and buckling resistance compared to solid square shafts with similar torque strength. Strength calculations are based on a design corrosion level of 50 years for most soil conditions.

Chance Type RS helical piles can be coupled with square shaft lead sections (Combo Piles) to provide greater penetration into bearing soils. Chance Type RS helical piles feature sharpened-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Helix plates can be equipped with seashell cuts on the leading edge to enhance penetration through dense soils with occasional cobbles and debris. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.



RS8625.250 HELICAL PILE SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: HSS 8-5/8 inch OD x 0.250 inch (schedule 20) wall steel shaft produced exclusively for Chance® products.

COUPLING: Formed and welded as a deep square socket, connected with multiple threaded studs and nuts.

HELIX - 1/2, 5/8, AND 3/4 INCH THICK: ASTM A572 with minimum yield strength of 50 ksi, and A1018 or A656, with minimum yield strength of 80 ksi, depending on helix diameter.

6 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for larger diameter helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 16, 18, and 24 inches.

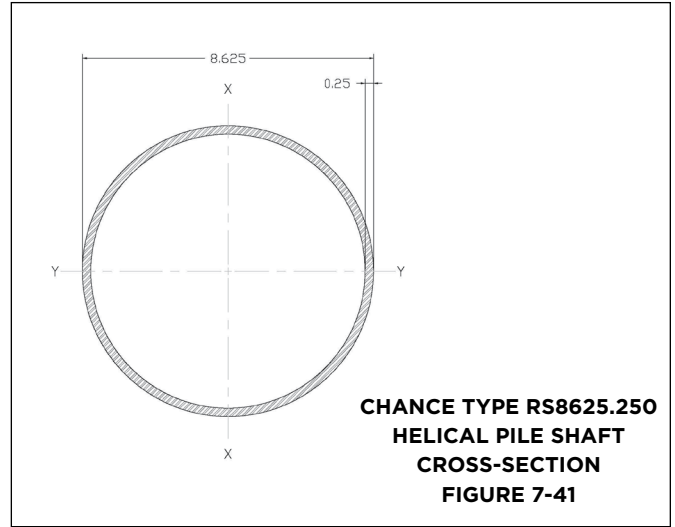
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight sharpened leading edge or can be ordered with a seashell cut (see Fig. 7-2). The seashell cut is best suited when it is necessary to penetrate soils with fill debris, cobbles, or fractured rock.

CONFIGURATIONS:

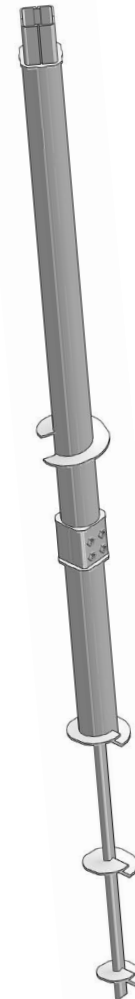
- Single-, double-, and triple-helix, lead sections, 5, 7, 10, 15, and 20 feet long
- Plain extensions, 5, 7, 10, 15, and 10 feet long
- Extensions with helix plates, 10 feet long

Helical products are hot dip galvanized per ASTM A123 Grade 75.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.

RS8625.250 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled HSS 8 inch nominal Schedule 20 (0.250 inch nominal wall) per ASTM A500 Grade B/C with 50 ksi minimum yield strength			
SHAFT SIZE, OD	8.625 in	219 mm	CORRODED	
			8.612 in	218.7 mm
SHAFT SIZE, ID*	8.16 in	207.3 mm	CORRODED	
			8.172 in	207.5 mm
MOMENT OF INERTIA (I)*	54.12 in ⁴	2249.5 cm ⁴	CORRODED	
			51.09 in ⁴	2123.6 cm ⁴
SHAFT AREA (A)*	6.14 in ²	39.6 cm ²	CORRODED	
			5.80 in ²	37.4 cm ²
SECTION MODULUS (S _{x-x})*	12.55 in ³	205.2 cm ³	CORRODED	
			11.87 in ³	194.1 cm ³
PERIMETER	27.1 in	68.8 cm	CORRODED	
			27.05 in	68.1 cm
COUPLING	Formed and welded square socket			
COUPLING BOLTS	Four 1-1/4 inch diameter Grade 2 studs			
HELIX PLATES	0.5 - 0.75 inch thick, formed on matching metal dies, ASTM A572 Grade 80 or better			
COATINGS	Hot dip galvanized per ASTM A123 Grade 75, 3.0 mil minimum thickness or bare steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	5 ft ⁻¹		13 m ⁻¹	
TORQUE RATING	60,000 ft·lb		81,349 N·m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	300 kip	1334 kN	225 kip	1001 kN
ALLOWABLE TENSION STRENGTH	150 kip		667 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	300 kip	1334 kN	150 kip	667 kN
Note: * Computed with 93% of wall thickness per AISC 360-10, B4.2				



ASSEMBLY OF RS8625.250 FIGURE 7-42

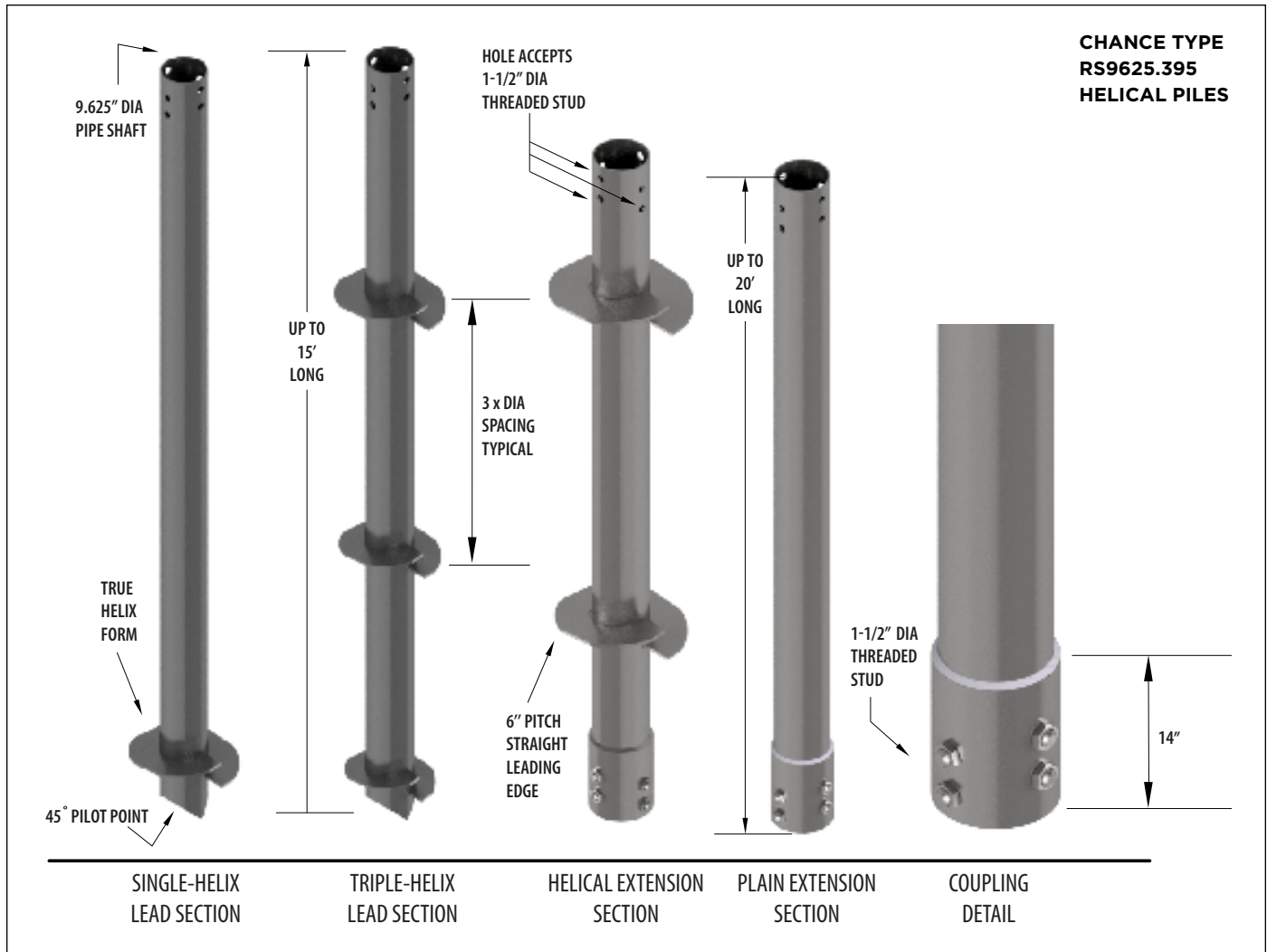
CHANCE TYPE RS9625.395 HELICAL PILES

- 600 kip ULTIMATE - 300 kip ALLOWABLE CAPACITY
- INSTALLATION TORQUE RATING - 200,000 ft-lb
- MULTI-PURPOSE 9-5/8" DIAMETER, 0.395" WALL, ROUND HSS SHAFT WITH SLEEVE COUPLINGS

DESCRIPTION:

Chance® Type RS9625.395 helical piles have 600 kip ultimate capacity and 300 kip working or allowable capacity in compression or tension. This capacity is based on well-documented correlations with installation torque, which is recognized as one method to determine capacity per IBC Section 1810.3.3.1.9. Lead sections and extensions couple together to extend the helix bearing plates to the required load-bearing stratum. Round shaft helical piles offer increased lateral and buckling

resistance compared to solid square shafts with similar torque-strength. Strength calculations are based on a design corrosion level of 50 years for most soil conditions. Chance Type RS helical piles feature straight-leading-edge helix plates that are circular in plan to provide uniform load bearing in most soil conditions. Custom lengths and helix configurations are available upon request. See below for additional information and other sections of this manual for specifications and design details.



All Hubbell Power Systems, Inc., Chance helical products are MADE IN THE U.S.A.



RS9625.395 HELICAL PILE SPECIFICATIONS & AVAILABLE CONFIGURATIONS

SHAFT: HSS 9-5/8 inch OD x 0.395 inch wall steel shaft produced exclusively for Chance® products.

COUPLING: Welded external sleeve forming a flush fit joint, connected with multiple structural grade bolts.

HELIX - 5/8 INCH THICK: ASTM A572 Grade 50 with minimum yield strength of 50 ksi.

6 INCH HELIX PITCH: A standard established by Hubbell Power Systems, Inc., for larger diameter helical piles and anchors.

AVAILABLE HELIX DIAMETERS: 18, 20, 22, and 24 inches.

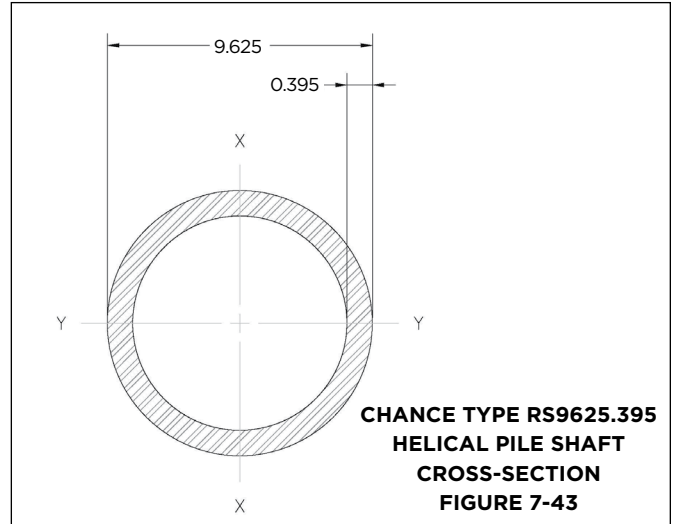
All helix plates are spaced 3 times the diameter of the preceding (lower) helix unless otherwise specified.

The standard helix plate has a straight leading edge.

CONFIGURATIONS:

- Single- and double-helix lead sections, 10 feet long; triple-helix lead sections, 15 feet long
- Plain extensions, 10 and 20 feet long
- Extensions with helix plates, 5 and 7 feet long

Helical products are hot dip galvanized per ASTM A123 Grade 75 or are available black.



NOTE: Helical piles shall be installed to appropriate depth in suitable bearing stratum as determined by the geotechnical engineer or local jurisdictional authority. Torque-correlated capacities are based on installing the pile to its torque rating using consistent rate of advance and RPM. A minimum Factor of Safety of 2 is recommended for determining allowable capacity from correlations. Deflections of 0.25 to 0.50 inches are typical at allowable capacity.

RS9625.395 HELICAL PILE AND ANCHOR PRODUCT SPECIFICATIONS				
SHAFT	Hot-rolled HSS 9-5/8-inch, 0.395 inch nominal wall with 80 ksi minimum yield strength			
SHAFT SIZE, OD	9.625 in	244.5 mm	CORRODED	
			9.612 in	244.1 mm
SHAFT SIZE, ID*	8.89 in	225.8 mm	CORRODED	
			8.903 in	226.1 mm
MOMENT OF INERTIA (I)*	114.65 in ⁴	4772.1 cm ⁴	CORRODED	
			110.47 in ⁴	4598.1 cm ⁴
SHAFT AREA (A)*	10.69 in ²	68.9 cm ²	CORRODED	
			10.30 in ²	66.45 cm ²
SECTION MODULUS (S _{x-x})*	23.8 in ³	390.4 cm ³	CORRODED	
			22.95 in ³	376.2 cm ³
PERIMETER	30.2 in	76.7 cm	CORRODED	
			30.1 in	76.4 cm
COUPLING	Welded external sleeve flush fit joint			
COUPLING BOLTS	Four 1-1/2 inch diameter ASTM F1554 Grade 105 threaded studs			
HELIX PLATES	0.625-inch thick, formed on matching metal dies, ASTM A572 Grade 50 or better			
COATINGS	Hot dip galvanized per ASTM A123 Grade 75, 3.0 mil minimum thickness or Bare Steel			
TORQUE PROPERTIES				
TORQUE CORRELATION FACTOR	3 ft ⁻¹		10 m ⁻¹	
TORQUE RATING	200,000 ft-lb		271,164 N-m	
STRUCTURAL CAPACITY				
TENSION STRENGTH	NOMINAL		LRFD DESIGN	
	600 kip	2669 kN	450 kip	2002 kN
ALLOWABLE TENSION STRENGTH	300 kip		1334.5 kN	
TORQUE-CORRELATED CAPACITY				
CAPACITY LIMIT BASED ON TORQUE CORRELATION, TENSION / COMPRESSION	ULTIMATE		ALLOWABLE	
	600 kip	2669 kN	300 kip	1334.5 kN
Note: * Computed with 93% of wall thickness per AISC 360-10, B4.2				



ASSEMBLY OF RS9625.395 FIGURE 7-44

CHANCE ROCK-IT HELICAL LEAD

DESCRIPTION:

The ROCK-IT™ lead section is an innovative solution to penetrate rocky or high-blow-count soils without pre-drilling or field modification. The single-carbide-tip, patent-pending design was developed after site testing of several rock anchor configurations to provide an economical yet proven solution to reach load-bearing depths in high-blow-count material.

KEY BENEFITS:

- Wear-resistant, offset carbide tip designed to break through rocky soil
- Reduced installation time to save time and money
- Reduced spiking of torque and chatter during installation for better pile performance and a safer alternative

CATALOG NO.	DESCRIPTION*
C1501488	Lead, SS150, 6/8 X 3 ft, ROCK-IT
C1501489	Lead, SS150, 8/10 X 3 ft, ROCK-IT
C1501505	Lead, SS150, 8/10 X 5 ft, ROCK-IT
C1501507	Lead, SS175, 8/10 X 5 ft, ROCK-IT
C1501509	Lead, SS175, 8/10/12 X 5 ft, ROCK-IT
C1501511	Lead, SS175, 6/8 X 3 ft, ROCK-IT
C1501513	Lead, SS175, 8/10 X 3 ft, ROCK-IT
C1501532	Lead, SS200, 8/10/12 X 7 ft, ROCK-IT
C1501612	Lead, SS200, 6/8/10 X 5 ft, ROCK-IT
C1501544	Lead, SS225, 8/10/12 X 7 ft, ROCK-IT
C1501715	Lead, SS225, 6/8/10 X 5 ft, ROCK-IT
C1501756	Lead, SS225, 6/8/10/12 X 10 ft, ROCK-IT

*See helical pile and anchor specifications of the product family for torque rating, helix strengths, and pile capacities.



**ROCK-IT LEAD SECTION
FIGURE 7-45**



TYPE SS/RS COMBINATION HELICAL PILES

CHANCE® HELICAL TRANSITION COUPLER

ADAPTS TYPE SS TO TYPE RS PILE SHAFTS

The Type SS/RS Combination Pile is used mainly in compression applications in areas where soft/loose soils are located above the bearing strata (hard/dense soils) for the helices. The Type RS material with its much greater section modulus will resist columnar buckling in the soft/loose soil. Its larger shaft diameter also provides for lateral load resistance. Due to its slender size, the Type SS material provides the means for the helix plates to penetrate deeper into hard/dense soil strata than if the helical pile shaft was round shaft only. For a given helix configuration and equal available installation energy (i.e., machine), a small-displacement shaft will penetrate farther into a soil bearing stratum than a large-displacement shaft and will disturb less soil.

It is recommended that a Chance SS/RS Combination Pile be used in all projects where round shaft is being used. The square shaft lead section will provide better load capacity and less settlement than a comparable straight round shaft pile.

The transition coupler (see Figure 7-46) adapts Type SS helical lead sections to Type RS plain extensions. Installation of this combination pile is the same as a standard helical pile. Table 7-5 provides the various standard transition couplers that are available along with their ratings. Special transition couplers, such as RS2875 to RS4500, are also available. Please contact your area Chance distributor for availability and delivery times.



PILE ASSEMBLY WITH TRANSITION COUPLER
FIGURE 7-46

TRANSITION COUPLERS, TABLE 7-5

CATALOG NUMBER	DESCRIPTION	TORQUE RATINGS	K_t
C1500896	SS5 square shaft to RS2875.203 round shaft	5,700 ft·lb	9.5
C1500896	SS150 square shaft to RS2875.203 round shaft	7,000 ft·lb	9.5
C1500896	SS150 square shaft to RS2875.276 round shaft	7,000 ft·lb	9.5
C1501365	SS175 square shaft to RS2875.276 round shaft	8,000 ft·lb	9.5
C1500895	SS175 square shaft to RS3500.300 dia round shaft	10,500 ft·lb	8.5
C1500937	SS200 square shaft to RS3500.300 dia round shaft	13,000 ft·lb	8.5
C1101443	SS200 square shaft to RS4500 round shaft	16,000 ft·lb	7
C1101418	SS225 square shaft to RS4500 round shaft	21,000 ft·lb	7

CHANCE HELICAL PULLDOWN MICROPILES

The Chance® Helical Pulldown® Micropile (HPM) is a patented (U.S. patent 5,707,180) method used to form a grout column around the shaft of a standard square shaft or round shaft helical pile. The installation process can employ grout only (see Figure 7-47) or grout in combination with either steel or PVC casing (see Figure 7-48). The result is a helical pile with a grouted shaft similar, in terms of installation, to drilled and grouted anchors or auger cast-in-place piles using gravity grouting.

The initial reason for developing the HPM was to design a helical pile with sufficient shaft size to resist buckling. However, since its inception, the method has demonstrated more advantages than simply buckling resistance. The advantages and limitations, based on the results of field tests, are summarized below:

- Increases buckling capacity of a helical pile shaft in soft/ loose overburden soils to the point that end bearing controls failure.
- Increased compression capacity due to the mobilization of skin friction at the grout/soil interface. Total capacity is a function of both skin friction and end bearing.
- The grout column provides additional corrosion protection to the steel pile shaft from naturally occurring aggressive soils with high metal-loss rates, organic soils such as peat, or other corrosive environments like slag, ash, swamp, chemical waste, or other man-made material.
- Stiffens the load/deflection response of helical piles. Axial deflection per unit load is typically less than with ungrouted shafts.

The installation procedure for Chance Helical Pulldown Micropiles is rather unique in that the soil along the sides of the shaft is displaced laterally and then replaced and continuously supported by the flowable grout as the pile is installed. To begin the installation process, a helical pile is placed into the soil by applying torque to the shaft. The helical shape of the bearing plates creates a significant downward force that keeps the pile advancing into the soil. After the lead section with the helical plates penetrates the soil, a lead displacement plate and extension are placed onto the shaft. Resuming torque on the assembly advances the helical plates and pulls the displacement plate downward, forcing soil outward to create a cylindrical void around the shaft. From a reservoir at the surface, a flowable grout is gravity fed and immediately fills the void surrounding the shaft. Additional extensions and displacement plates are added until the helical bearing plates reach the minimum depth required or competent load-bearing soil. This displacement pile system does not require removing spoils from the site.

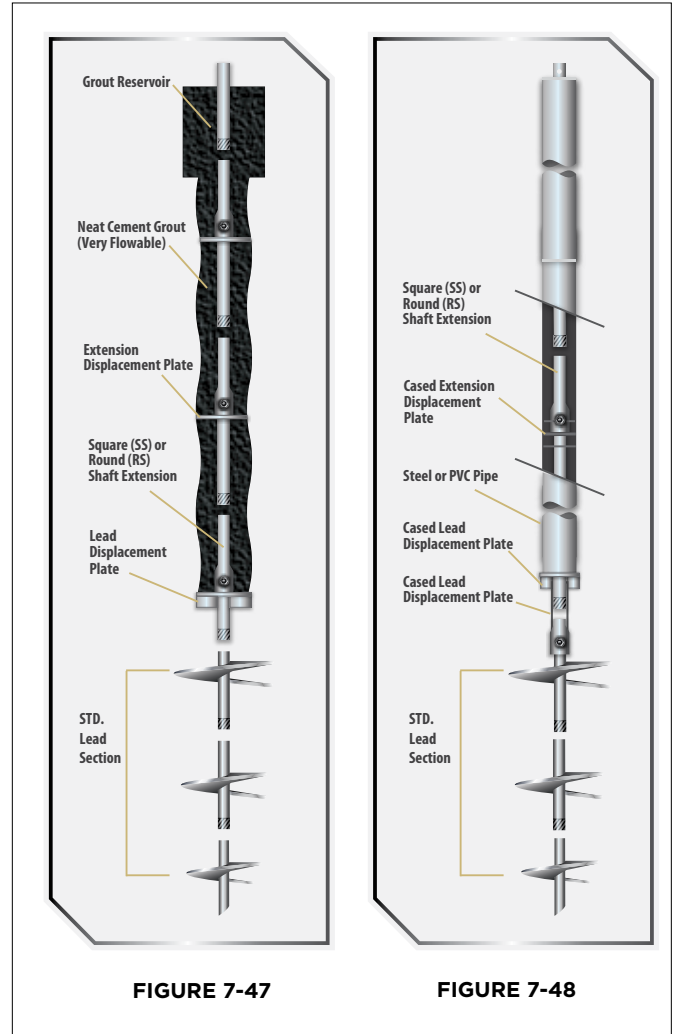


FIGURE 7-47

FIGURE 7-48



THEORETICAL GROUT VOLUME PER UNIT DEPTH, TABLE 7-6

GROUT COLUMN DIAMETER (in) [mm]	PILE SHAFT SIZE (in) [mm]	GROUT VOLUME (ft ³ /ft) [m ³ /m]
4 [102]	1-1/2 [38] solid square	0.071 [0.007]
5 [127]	1-1/2 [38] solid square	0.120 [0.011]
	1-3/4 [44] solid square	0.115 [0.011]
6 [152]	1-1/2 [38] solid square	0.181 [0.017]
	1-3/4 [44] solid square	0.175 [0.016]
	2 [51] solid square	0.169 [0.016]
	2-1/4 [57] solid square	0.161 [0.015]
	2-7/8 x 0.203 [73 x 5.2] pipe shaft	0.185 [0.017]
	2-7/8 x 0.276 [73 x 7] pipe shaft	0.181 [0.017]
	3-1/2 x 0.300 [89 x 7.6] pipe shaft	0.176 [0.016]
	4-1/2 x 0.337 [114 x 8.6] pipe shaft	0.166 [0.015]
7 [178]	1-1/2 [38] solid square	0.249 [0.023]
	1-3/4 [44] solid square	0.246 [0.023]
	2 [51] solid square	0.240 [0.022]
	2-1/4 [57] solid square	0.232 [0.022]
	3-1/2 x 0.300 [89 x 7.6] pipe shaft	0.246 [0.023]
8 [203]	4-1/2 x 0.337 [114 x 8.6] pipe shaft	0.237 [0.022]
	1-3/4 [44] solid square	0.328 [0.030]
	2-7/8 x 0.203 [73 x 5.2] pipe shaft	0.337 [0.031]
	2-7/8 x 0.276 [73 x 7] pipe shaft	0.333 [0.031]
	3-1/2 x 0.300 [89 x 7.6] pipe shaft	0.328 [0.030]
8.5 [216]	4-1/2 x 0.337 [114 x 8.6] pipe shaft	0.319 [0.029]
	2 [51] solid square	0.367 [0.034]
	2-1/4 [57] solid square	0.359 [0.033]
10 [254]	1-3/4 [44] solid square	0.524 [0.049]
	2 [51] solid square	0.517 [0.048]
	2-1/4 [57] solid square	0.511 [0.047]
	3-1/2 x 0.300 [89 x 7.6] pipe shaft	0.525 [0.049]
	4-1/2 x 0.337 [114 x 8.6] pipe shaft	0.515 [0.048]

Multiply the volume per unit depth in the table by the grout column length to calculate the total grout volume. Be sure to use the appropriate length units of feet or meters for the grout column length.

Note that if the piles are uncased, more grout may be required due to irregularities in the column and subsurface voids. Also, don't forget to account for the grout reservoir and waste when bidding the job.

strengths of shafts without grout for comparison.

Per the International Building Code (IBC) 2006 Section 1808.2.9.2 & IBC 2021 Section 1810.2.1, the depth to fixity of piles driven into soft ground can be considered fixed and laterally supported at 10 feet below the ground surface.

HIGHER COMPRESSION STRENGTHS WITH GROUTED SHAFTS

The following tables provide the nominal, LRFD design, and ASD allowable compression strengths of helical piles with various diameter grouted shafts. The strengths listed are based on an unsupported shaft length of 10 feet (3 meters) with either a fixed or pinned end condition at the pile head. The grout column diameters listed are the most common used per each helical product family. Each table includes the compression



NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE SS5 GROUDED-SHAFT PILES IN SOFT SOIL^{1,2,3}

GROUT COLUMN DIAMETER	NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS (kip) [kN]					
	SOFT SOIL					
	PINNED			FIXED		
	NOMINAL	DESIGN	ALLOWABLE	NOMINAL	DESIGN	ALLOWABLE
No grout	13.6 [60]	12.2 [54]	8.1 [36]	26.6 [118]	24.0 [107]	16.0 [71]
4" OD	30.2 [134]	22.6 [101]	15.1 [67]	59.2 [263]	44.4 [198]	29.6 [132]
5" OD	54.9 [244]	41.2 [183]	27.4 [122]	104.5 [465]	78.3 [348]	52.2 [232]
6" OD	86.2 [383]	64.6 [287]	43.1 [192]	148.3 [660]	111.2 [495]	74.1 [330]
7" OD	126.2 [561]	94.6 [421]	63.1 [281]	194.6 [866]	145.9 [649]	97.3 [433]

NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE TYPE SS150 GROUDED-SHAFT PILES IN SOFT SOIL^{1,2,3}

GROUT COLUMN DIAMETER	NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS (kip) [kN]					
	SOFT SOIL					
	PINNED			FIXED		
	NOMINAL	DESIGN	ALLOWABLE	NOMINAL	DESIGN	ALLOWABLE
No grout	13.6 [60]	12.2 [54]	8.1 [36]	26.6 [118]	24.0 [107]	16.0 [71]
4" OD	30.2 [134]	22.6 [101]	15.1 [67]	59.2 [263]	44.4 [198]	29.6 [132]
5" OD	54.9 [244]	41.2 [183]	27.4 [122]	104.5 [465]	78.3 [348]	52.2 [232]
6" OD	86.2 [383]	64.6 [287]	43.1 [192]	148.3 [660]	111.2 [495]	74.1 [330]
7" OD	126.8 [564]	95.1 [423]	63.4 [282]	208.4 [927]	156.3 [695]	104.2 [464]

NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE TYPE SS175 GROUDED-SHAFT PILES IN SOFT SOIL^{1,2,3}

GROUT COLUMN DIAMETER	NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS (kip) [kN]					
	SOFT SOIL					
	PINNED			FIXED		
	NOMINAL	DESIGN	ALLOWABLE	NOMINAL	DESIGN	ALLOWABLE
No grout	25.8 [115]	23.2 [103]	15.4 [69]	50.5 [225]	45.4 [202]	30.2 [134]
5" OD	66.6 [296]	49.9 [222]	33.3 [148]	127.2 [566]	95.4 [424]	63.6 [283]
6" OD	111.5 [496]	83.6 [372]	55.7 [248]	185.6 [826]	139.2 [619]	92.8 [413]
7" OD	158.3 [704]	118.7 [528]	79.1 [352]	236.2 [1051]	177.2 [788]	118.1 [525]
8" OD	209.2 [931]	156.9 [698]	104.6 [465]	290.4 [1292]	217.8 [969]	145.2 [646]

NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE TYPE SS200 GROUDED-SHAFT PILES IN SOFT SOIL^{1,2,3}

GROUT COLUMN DIAMETER	NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS (kip) [kN]					
	SOFT SOIL					
	PINNED			FIXED		
	NOMINAL	DESIGN	ALLOWABLE	NOMINAL	DESIGN	ALLOWABLE
No grout	43.7 [194]	39.3 [175]	26.2 [117]	85.6 [381]	77.1 [343]	51.3 [228]
6" OD	128.7 [572]	96.6 [430]	64.4 [286]	233.9 [1040]	175.4 [780]	116.9 [520]
7" OD	201.9 [898]	151.4 [673]	101.0 [449]	312.9 [1392]	234.6 [1044]	156.4 [696]
8.5" OD	294.7 [1311]	221.0 [983]	147.4 [656]	407.6 [1813]	305.7 [1360]	203.8 [907]
10" OD	401.4 [1786]	301.1 [1339]	200.7 [893]	513.6 [2285]	385.2 [1713]	256.8 [1142]

Notes:

For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, and soft soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.

³ Column length to "fixity" of shaft in soil = 10 feet (3 meters).



NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE SS225 GROUTED-SHAFT PILES IN SOFT SOIL^{1,2,3}

GROUT COLUMN DIAMETER	NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS (kip) [kN]					
	SOFT SOIL					
	PINNED			FIXED		
	NOMINAL	DESIGN	ALLOWABLE	NOMINAL	DESIGN	ALLOWABLE
No grout	70.9 [315]	63.8 [284]	42.5 [189]	139.0 [618]	125.1 [556]	83.2 [370]
6" OD	154.9 [689]	116.2 [517]	77.5 [345]	281.8 [1254]	211.4 [940]	140.9 [627]
7" OD	228.8 [1018]	171.6 [763]	114.4 [509]	363.2 [1171]	272.4 [1212]	181.6 [808]
8.5" OD	354.3 [1576]	265.7 [1182]	177.1 [788]	482.3 [2145]	361.7 [1609]	241.1 [1072]
10" OD	466.1 [2073]	349.6 [1555]	233.1 [1037]	591.3 [2630]	443.5 [1973]	295.7 [1315]

NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE TYPE RS2875.203 GROUTED-SHAFT PILES IN SOFT SOIL^{1,2,3}

GROUT COLUMN DIAMETER	NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS (kip) [kN]					
	SOFT SOIL					
	PINNED			FIXED		
	NOMINAL	DESIGN	ALLOWABLE	NOMINAL	DESIGN	ALLOWABLE
No grout	42.0 [187]	37.8 [168]	25.1 [112]	55.5 [247]	49.9 [222]	33.2 [148]
6" OD	95.7 [426]	71.8 [319]	47.8 [213]	125.7 [559]	94.3 [419]	62.8 [279]
8" OD	160.1 [712]	120.1 [534]	80.1 [356]	203.2 [904]	152.4 [678]	101.6 [452]

NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE TYPE RS2875.276 GROUTED-SHAFT PILES IN SOFT SOIL^{1,2,3}

GROUT COLUMN DIAMETER	NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS (kip) [kN]					
	SOFT SOIL					
	PINNED			FIXED		
	NOMINAL	DESIGN	ALLOWABLE	NOMINAL	DESIGN	ALLOWABLE
No grout	55.2 [246]	49.7 [221]	33.0 [147]	73.9 [329]	66.5 [296]	44.3 [197]
6" OD	114.3 [508]	85.7 [381]	57.1 [254]	147.7 [657]	110.8 [493]	73.9 [329]
8" OD	181.4 [807]	136.0 [605]	90.7 [403]	226.9 [1009]	170.2 [757]	113.5 [505]

NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE TYPE RS3500.300 GROUTED-SHAFT PILES IN SOFT SOIL^{1,2,3}

GROUT COLUMN DIAMETER	NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS (kip) [kN]					
	SOFT SOIL					
	PINNED			FIXED		
	NOMINAL	DESIGN	ALLOWABLE	NOMINAL	DESIGN	ALLOWABLE
No grout	90.7 [403]	81.6 [363]	54.3 [242]	110.0 [49]	99.0 [440]	65.9 [293]
6" OD	145.1 [645]	108.8 [484]	72.5 [322]	175.6 [781]	131.7 [586]	87.8 [391]
7" OD	179.3 [798]	134.4 [598]	89.6 [399]	214.1 [952]	160.6 [714]	107.0 [476]
8" OD	216.7 [964]	162.5 [723]	108.4 [482]	257.3 [1145]	193.0 [859]	128.6 [572]
10" OD	314.4 [1399]	235.8 [1049]	157.2 [699]	365.6 [1626]	274.2 [1220]	182.8 [813]

NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE TYPE RS4500.337 GROUTED-SHAFT PILES IN SOFT SOIL^{1,2,3}

GROUT COLUMN DIAMETER	NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS (kip) [kN]					
	SOFT SOIL					
	PINNED			FIXED		
	NOMINAL	DESIGN	ALLOWABLE	NOMINAL	DESIGN	ALLOWABLE
No grout	156.3 [695]	140.7 [626]	93.6 [416]	175.3 [780]	157.8 [702]	105.0 [467]
6" OD	195.3 [869]	146.5 [652]	97.6 [434]	220.6 [981]	165.5 [736]	110.3 [491]
7" OD	230.4 [1025]	172.8 [769]	115.2 [512]	259.6 [1155]	194.7 [866]	129.8 [577]
8" OD	274.2 [1220]	205.6 [915]	137.1 [610]	306.4 [1363]	229.8 [1022]	153.2 [681]
10" OD	372.8 [1658]	279.6 [1244]	186.4 [829]	415.0 [1846]	311.3 [1385]	207.5 [923]

Notes:

For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ESR-2794 for descriptions of fixed condition, pinned condition, and soft soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with IBC Section 1808.2.5 and the lead section with which the extension is used will provide sufficient helix capacity to develop the full shaft capacity.

³ Column length to "fixity" of shaft in soil = 10 feet (3 meters).

REMEDIAL REPAIR BRACKETS FOR CHANCE HELICAL PILES

CHANCE® HELICAL C1500121 STANDARD BRACKET AND T-PIPE SYSTEM

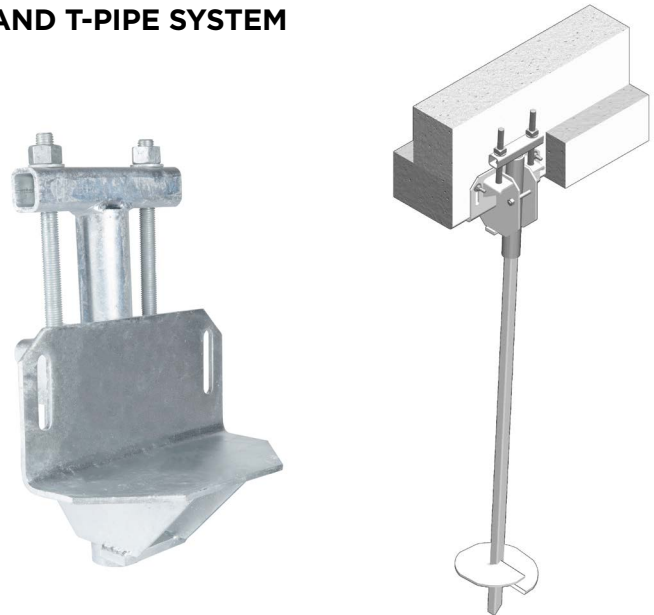
- Used with Chance Type SS5 & SS150 1-1/2" square shaft helical piles and Type RS2875.203 and RS2875.276 2-7/8" OD round shaft helical piles
- Use for lifts up to 4" (10 cm)
- All C1500121 standard systems include:
 - Foundation bracket
 - T-pipe
 - Hardware

ORDER SEPARATELY: Two 5/8" (16 mm) diameter concrete anchor bolts per pile as required.

Standard finish is hot-dip galvanized per ASTM A153.

Ultimate mechanical strength of bracket body is 80,000 lb (356 kN). Working mechanical strength of bracket body is 40,000 lb (178kN).

See table below for system (bracket/pile shaft) ratings.



DRAWINGS & RATINGS

CHANCE HELICAL C1500121 STANDARD BRACKET AND T-PIPE RATINGS					
T-PIPE DESIGNATIONS FOR THE C1500121 BRACKET	ULTIMATE MECHANICAL STRENGTH ^{1,3} (lb) [kN]	PILE SIZE (in) [mm]	PRODUCT SERIES	MAX WORKING CAPACITY ^{2,3} BASED ON PRODUCT SERIES (lb) [kN]	FEATURES
C1500486	40,000 [178]	1-1/2 [38] square	SS5 SS150	20,000 [89] 20,000 [89]	Lowest cost with square shaft
C1500487	80,000 [356]	1-1/2 [38] square	SS5 SS150	20,000 [89] 25,000 [111]	Higher capacity with SS150
C2780001	40,000 [178]	2-7/8 [73] round	RS2875.203	20,000 [89]	Lowest cost with round shaft
C2780002	80,000 [356]	2-7/8 [73] round	RS2875.203	25,000 [111]	Higher capacity with stronger T-pipe
C2788012	40,000 [178]	2-7/8 [73] round	RS2875.276	20,000 [89]	Lowest cost with RS2875.276
C2788011	80,000 [356]	2-7/8 [73] round	RS2875.276	30,000 [133]	Higher capacity with RS2875.276

Notes:

1 Ultimate mechanical strength is for the bracket body and T-pipe combination.

2 The capacity of Chance helical pile systems is a function of many individual elements including the capacity of the foundation, bracket, pile shaft, helix plate, and bearing stratum; the strength of the foundation-to-bracket connection; and the quality of the helical pile installation. The fifth column shows typical working capacities of the Chance helical pile system based upon maximum shaft exposure of 2 feet and soil strength having a minimum Standard Penetration Test (SPT) blow count (N₆₀) of 4. Actual capacities could be higher or lower depending on the above factors.

3 The ultimate capacity of the system, i.e., bracket, T-pipe, and pile shaft, can be increased to the pile shaft compression capacity limit as shown on the product data pages provided the pile shaft is reinforced using a pipe sleeve or grout column. The maximum working capacity shall not be greater than one half the ultimate mechanical strength of the bracket and T-pipe combination given above.



BUILDING CODE COMPLIANCE PER ICC-ES ESR-2794

The following tables provide the nominal, LRFD design, and ASD allowable compression strengths of C1500121 foundation repair brackets, T-pipes, and Type SS5, SS150, RS2875.203, and RS2875.276 helical piles as evaluated per ICC-ES Acceptance Criteria AC358. These strengths are published in ICC-ES ESR-2794. The strengths listed are based on three different concrete foundation strengths and two different soils conditions—firm

and soft. The pile head is assumed to be fixed within the bracket assembly, and the piles are assumed to be braced.

Per the International Building Code (IBC) 2006 Section 1808.2.9.2 & IBC 2021 Section 1810.2.1, piles can be considered fixed and laterally supported at 5 feet below the ground surface in firm soil and at 10 feet in soft soil.

NOMINAL STRENGTHS OF C1500121 REMEDIAL REPAIR BRACKETS & HELICAL PILES^{1,2,3,4,5}

BRACKET CATALOG NUMBER	T-PIPE CATALOG NUMBER	PILE MODEL	NOMINAL STRENGTH IN AXIAL COMPRESSION (kip) [kN]					
			2500 psi CONCRETE ⁵		3000 psi CONCRETE ⁵		4000 psi CONCRETE ⁵	
			FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL
C1500121	C1500486	SS5/150	36.3 [161]	26.6 [118]	36.3 [161]	26.6 [118]	36.3 [161]	26.6 [118]
C1500121	C1500487	SS5	70.3 [313]	26.6 [118]	77.8 [346]	26.6 [118]	89.8 [399]	26.6 [118]
C1500121	C1500487	SS150	78.7 [350]	26.6 [118]	87.1 [387]	26.6 [118]	99.5 [443]	26.6 [118]
C1500121	C2780001	RS2875.203	38.8 [173]	38.8 [173]	38.8 [173]	38.8 [173]	38.8 [173]	38.8 [173]
C1050121	C2780002	RS2875.203	73.8 [328]	66.1 [294]	77.8 [346]	66.1 [294]	77.8 [346]	66.1 [294]
C1050121	C2780011	RS2875.276	75.1 [334]	70 [311]	83.4 [371]	73.9 [329]	83.4 [371]	73.9 [329]
C1050121	C2780012	RS2875.276	38.8 [173]	38.8 [173]	38.8 [173]	38.8 [173]	38.8 [173]	38.8 [173]

LRFD DESIGN STRENGTHS OF C1500121 REMEDIAL REPAIR BRACKETS & HELICAL PILES^{1,2,3,4,5}

BRACKET CATALOG NUMBER	T-PIPE CATALOG NUMBER	PILE MODEL	LRFD DESIGN STRENGTH IN AXIAL COMPRESSION (kip) [kN]					
			2500 psi CONCRETE ⁵		3000 psi CONCRETE ⁵		4000 psi CONCRETE ⁵	
			FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL
C1500121	C1500486	SS5/150	32.6 [145]	24.0 [107]	32.6 [145]	24.0 [107]	32.6 [145]	24.0 [107]
C1500121	C1500487	SS5	49.2 [219]	24.0 [107]	54.4 [242]	24.0 [107]	62.8 [279]	24.0 [107]
C1500121	C1500487	SS150	55.1 [245]	24.0 [107]	60.9 [271]	24.0 [107]	69.6 [310]	24.0 [107]
C1500121	C2780001	RS2875.203	34.9 [155]	34.9 [155]	34.9 [155]	34.9 [155]	34.9 [155]	34.9 [155]
C1050121	C2780002	RS2875.203	51.6 [229.5]	46.3 [206]	54.5 [242]	46.3 [206]	54.5 [242]	53.3 [237]
C1050121	C2780011	RS2875.276	52.6 [234]	49.0 [218]	58.4 [260]	51.7 [230]	58.4 [260]	53.3 [237]
C1050121	C2780012	RS2875.276	34.9 [155]	34.9 [155]	34.9 [155]	34.9 [155]	34.9 [155]	34.9 [155]

ASD ALLOWABLE STRENGTHS OF C1500121 REMEDIAL REPAIR BRACKETS & HELICAL PILES^{1,2,3,4,5}

BRACKET CATALOG NUMBER	T-PIPE CATALOG NUMBER	PILE MODEL	ASD ALLOWABLE STRENGTH IN AXIAL COMPRESSION (kip) [kN]					
			2500 psi CONCRETE ⁵		3000 psi CONCRETE ⁵		4000 psi CONCRETE ⁵	
			FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL
C1500121	C1500486	SS5/150	21.7 [97]	16.0 [71]	21.7 [97]	16.0 [71]	21.7 [97]	16.0 [71]
C1500121	C1500487	SS5	30.9 [137]	16.0 [71]	34.2 [152]	16.0 [71]	39.4 [175]	16.0 [71]
C1500121	C1500487	SS150	34.6 [154]	16.0 [71]	38.2 [170]	16.0 [71]	43.7 [194]	16.0 [71]
C1500121	C2780001	RS2875.203	23.2 [103]	23.2 [103]	23.2 [103]	23.2 [103]	23.2 [103]	23.2 [103]
C1050121	C2780002	RS2875.203	32.4 [144.1]	29.0 [129]	34.2 [152]	29.0 [129]	34.3 [152.5]	34.3 [152.5]
C1050121	C2788011	RS2875.276	33.0 [147]	30.8 [137]	36.6 [163]	32.5 [145]	36.6 [163]	34.3 [152.5]
C1050121	C2788012	RS2875.276	23.2 [103]	23.2 [103]	23.2 [103]	23.2 [103]	23.2 [103]	23.2 [103]

Notes:

For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ICC-ES ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with Section 1810.2.2 of the 2021 IBC (Section 1808.2.5 of the 2006 IBC).

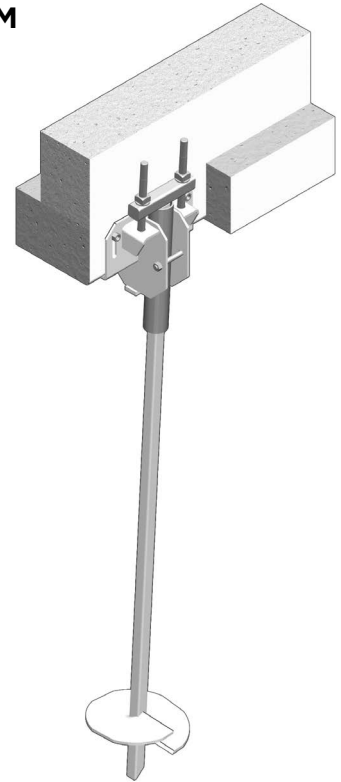
³ Strength ratings apply to the specific bracket, T-pipe, and pile/anchor models listed.

⁴ See Section 4.1.2 of ICC-ES ESR-2794 for applicable limit states that must be evaluated by a registered design professional.

⁵ Refer to the specified compressive strength of concrete at 28 days.

CHANCE® HELICAL C1500299 STANDARD BRACKET AND T-PIPE SYSTEM

- Used with Chance Type SS175 1-3/4" square shaft helical piles
- Use for lifts up to 4" (10 cm)
- All C1500299 standard systems include:
 - Foundation bracket
 - T-pipe
 - Hardware



ORDER SEPARATELY: Two 5/8" (16 mm) diameter concrete anchor bolts per pile as required.

Standard finish is hot-dip galvanized per ASTM A153.

Ultimate mechanical strength of bracket body is 80,000 lb (356 kN). Working mechanical strength of bracket body is 40,000 lb (178kN).

See table below for system (bracket/pile shaft) ratings.

CHANCE HELICAL C1500299 STANDARD BRACKET AND T-PIPE RATINGS					
T-PIPE DESIGNATIONS FOR THE C1500299 BRACKET	ULTIMATE MECHANICAL STRENGTH ^{1,3} (lb) [kN]	PILE SIZE (in) [mm]	PRODUCT SERIES	MAX WORKING CAPACITY ^{2,3} BASED ON PRODUCT SERIES (lb) [kN]	FEATURES
C1500488	80,000 [356]	1-3/4 [44] square	SS175	30,000 [133]	Lowest cost with SS175

Notes:

1 Ultimate mechanical strength is for the bracket body and T-pipe combination.

2 The capacity of Chance helical pile systems is a function of many individual elements including the capacity of the foundation, bracket, pile shaft, helix plate, and bearing stratum; the strength of the foundation-to-bracket connection; and the quality of the helical pile installation. The fifth column shows typical working capacities of the Chance helical pile system based upon maximum shaft exposure of 2 feet and soil strength having a minimum Standard Penetration Test (SPT) blow count (N_{60}) of 4. Actual capacities could be higher or lower depending on the above factors.

3 The ultimate capacity of the system, i.e., bracket, T-pipe, and pile shaft, can be increased to the pile shaft compression capacity limit as shown on the product data pages provided the pile shaft is reinforced using a pipe sleeve or grout column. The maximum working capacity shall not be greater than one half the ultimate mechanical strength of the bracket and T-pipe combination given above.



BUILDING CODE COMPLIANCE PER ICC-ES ESR-2794

The following tables provide the nominal, LRFD design, and ASD allowable compression strengths of C1500299 foundation repair brackets, T-pipes, and Type SS175 helical piles as evaluated per ICC-ES Acceptance Criteria AC358. These strengths are published in ICC-ES ESR-2794. The strengths listed are based on three different concrete foundation strengths and two different soils conditions—firm and soft. The pile head is assumed to be fixed within the bracket assembly, and the piles are assumed to be braced.

Per the International Building Code (IBC) 2006 Section 1808.2.9.2 & IBC 2021 Section 1810.2.1, piles can be considered fixed and laterally supported at 5 feet below the ground surface in firm soil and at 10 feet in soft soil.

NOMINAL STRENGTHS OF C1500299 REMEDIAL REPAIR BRACKETS & HELICAL PILES^{1,2,3,4,5}

BRACKET CATALOG NUMBER	T-PIPE CATALOG NUMBER	PILE MODEL	NOMINAL STRENGTH IN AXIAL COMPRESSION (kip) [kN]					
			2500 psi CONCRETE ⁵		3000 psi CONCRETE ⁵		4000 psi CONCRETE ⁵	
			FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL
C1500299	C1500488	SS175	79.0 [351]	50.5 [225]	89.4 [398]	50.5 [225]	99.0 [440]	50.5 [225]

LRFD DESIGN STRENGTHS OF C1500299 REMEDIAL REPAIR BRACKETS & HELICAL PILES^{1,2,3,4,5}

BRACKET CATALOG NUMBER	T-PIPE CATALOG NUMBER	PILE MODEL	LRFD DESIGN STRENGTH IN AXIAL COMPRESSION (kip) [kN]					
			2500 PSI CONCRETE ⁵		3000 PSI CONCRETE ⁵		4000 PSI CONCRETE ⁵	
			FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL
C1500299	C1500488	SS175	55.3 [246]	42.9 [191]	62.6 [278]	45.4 [202]	74.2 [330]	45.4 [202]

ASD ALLOWABLE STRENGTHS OF C1500299 REMEDIAL REPAIR BRACKETS & HELICAL PILES^{1,2,3,4,5}

BRACKET CATALOG NUMBER	T-PIPE CATALOG NUMBER	PILE MODEL	ASD ALLOWABLE STRENGTH IN AXIAL COMPRESSION (kip) [kN]					
			2500 psi CONCRETE ⁵		3000 psi CONCRETE ⁵		4000 psi CONCRETE ⁵	
			FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL
C1500299	C1500488	SS175	34.7 [154]	27.7 [123]	39.3 [175]	27.7 [123]	47.9 [213]	30.2 [134]

Notes:

For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ICC-ES ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with Section 1810.2.2 of the 2021 IBC (Section 1808.2.5 of the 2006 IBC).

³ Strength ratings apply to the specific bracket, T-pipe, and pile/anchor models listed.

⁴ See Section 4.1.2 of ICC-ES ESR-2794 for applicable limit states that must be evaluated by a registered design professional.

⁵ Refer to the specified compressive strength of concrete at 28 days.



CHANCE® HELICAL C1500147 HEAVY DUTY BRACKET AND T-PIPE SYSTEM

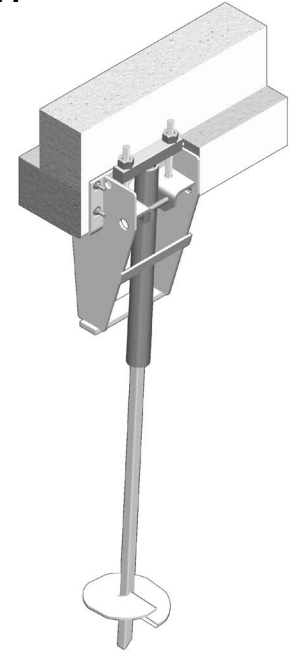
- Used with Chance Type SS175 1-3/4" square shaft helical piles
- Use for lifts up to 4" (10 cm)
- All C1500299 standard systems include:
 - Foundation bracket
 - T-pipe
 - Hardware

ORDER SEPARATELY: Four 5/8" (16 mm) diameter concrete anchor bolts per pile as required.

Standard finish is hot-dip galvanized per ASTM A153.

Ultimate mechanical strength of bracket body is 120,000 lb (534 kN). Working mechanical strength of bracket body is 60,000 lb (267kN).

See table below for system (bracket/pile shaft) ratings.



DRAWINGS & RATINGS

CHANCE HELICAL C1500147 STANDARD BRACKET AND T-PIPE RATINGS					
T-PIPE DESIGNATIONS FOR THE C1500147 BRACKET	ULTIMATE MECHANICAL STRENGTH ^{1,3} (lb) [kN]	PILE SIZE (in) [mm]	PRODUCT SERIES	MAX WORKING CAPACITY ^{2,3} BASED ON PRODUCT SERIES (lb) [kN]	FEATURES
C1500474	120,000 [534]	1-3/4 [44] square	SS175	40,000 [178]	Lowest cost with square shaft
C1500475	120,000 [534]	3-1/2 [89] round	RS3500.300	50,000 [222]	Higher capacity with RS3500.300
C1500508	120,000 [534]	2 [51] square	SS200	50,000 [222]	Highest capacity with square shaft

Notes:

1 Ultimate mechanical strength is for the bracket body and T-pipe combination.

2 The capacity of Chance helical pile systems is a function of many individual elements including the capacity of the foundation, bracket, pile shaft, helix plate, and bearing stratum; the strength of the foundation-to-bracket connection; and the quality of the helical pile installation. The fifth column shows typical working capacities of the Chance helical pile system based upon maximum shaft exposure of 2 feet and soil strength having a minimum Standard Penetration Test (SPT) blow count (N_{60}) of 4. Actual capacities could be higher or lower depending on the above factors.

3 The ultimate capacity of the system, i.e., bracket, T-pipe, and pile shaft, can be increased to the pile shaft compression capacity limit as shown on the product data pages provided the pile shaft is reinforced using a pipe sleeve or grout column. The maximum working capacity shall not be greater than one half the ultimate mechanical strength of the bracket and T-pipe combination given above.



BUILDING CODE COMPLIANCE PER ICC-ES ESR-2794

The following tables provide the nominal, LRFD design, and ASD allowable compression strengths of C1500147 foundation repair brackets, T-pipes, and both Type SS175 and RS3500.300 helical piles as evaluated per ICC-ES Acceptance Criteria AC358. These strengths are published in ICC-ES ESR-2794. The strengths listed are based on three different concrete foundation strengths and two different soils conditions—firm and soft. The pile head is assumed to be fixed within the bracket assembly, and the piles are assumed to be braced.

Per the International Building Code (IBC) 2006 Section 1808.2.9.2 & IBC 2021 Section 1810.2.1, piles can be considered fixed and laterally supported at 5 feet below the ground surface in firm soil and at 10 feet in soft soil.

NOMINAL STRENGTHS OF C1500147 REMEDIAL REPAIR BRACKETS & HELICAL PILES^{1,2,3,4,5}

BRACKET CATALOG NUMBER	T-PIPE CATALOG NUMBER	PILE MODEL	NOMINAL STRENGTH IN AXIAL COMPRESSION (kip) [kN]					
			2500 psi CONCRETE ⁵		3000 psi CONCRETE ⁵		4000 psi CONCRETE ⁵	
			FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL
C1500147	C1500474	SS175	100 [445]	50.5 [225]	100 [445]	50.5 [225]	100 [445]	50.5 [225]
C1500147	C1500475	RS3500	100 [445]	100 [445]	100 [445]	100 [445]	100 [445]	100 [445]

LRFD DESIGN STRENGTHS OF C1500147 REMEDIAL REPAIR BRACKETS & HELICAL PILES^{1,2,3,4,5}

BRACKET CATALOG NUMBER	T-PIPE CATALOG NUMBER	PILE MODEL	LRFD DESIGN STRENGTH IN AXIAL COMPRESSION (kip) [kN]					
			2500 psi CONCRETE ⁵		3000 psi CONCRETE ⁵		4000 psi CONCRETE ⁵	
			FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL
C1500147	C1500474	SS175	86.7 [386]	45.4 [202]	88.4 [393]	45.4 [202]	90 [400]	45.4 [202]
C1500147	C1500475	RS3500	71.8 [319]	71.8 [319]	77 [343]	77 [343]	77 [343]	77 [343]

ASD ALLOWABLE STRENGTHS OF C1500147 REMEDIAL REPAIR BRACKETS & HELICAL PILES^{1,2,3,4,5}

BRACKET CATALOG NUMBER	T-PIPE CATALOG NUMBER	PILE MODEL	ASD ALLOWABLE STRENGTH IN AXIAL COMPRESSION (kip) [kN]					
			2500 psi CONCRETE ⁵		3000 psi CONCRETE ⁵		4000 psi CONCRETE ⁵	
			FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL
C1500147	C1500474	SS175	54.4 [242]	30.2 [134]	57.0 [254]	30.2 [134]	60.0 [267]	30.2 [134]
C1500147	C1500475	RS3500	45.1 [201]	45.1 [201]	51.3 [228]	51.3 [228]	51.3 [228]	51.3 [228]

Notes:

For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ICC-ES ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with Section 1810.2.2 of the 2021 IBC (Section 1808.2.5 of the 2006 IBC).

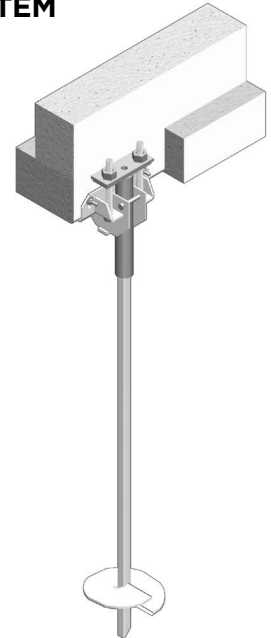
³ Strength ratings apply to the specific bracket, T-pipe, and pile/anchor models listed.

⁴ See Section 4.1.2 of ICC-ES ESR-2794 for applicable limit states that must be evaluated by a registered design professional.

⁵ Refer to the specified compressive strength of concrete at 28 days.

CHANCE® HELICAL PSAC1500499 LOW PROFILE BRACKET AND T-PIPE SYSTEM

- Used with Chance Type SS5 & SS150 1-1/2" square shaft helical piles and Type RS2875.203 and RS2875.276 2-7/8" OD round shaft helical piles
- Use for lifts up to 4" (10 cm)
- All PSAC1501500499 low profile systems include:
 - Foundation bracket
 - T-pipe
 - Hardware



ORDER SEPARATELY: Two 1/2" (13 mm) diameter concrete anchor bolts per pile as required.

Standard finish is hot-dip galvanized per ASTM A153.

Ultimate mechanical strength of bracket body is 30,000 lb (133 kN). Working mechanical strength of bracket body is 15,000 lb (67 kN).

See table below for system (bracket/pile shaft) ratings.

CHANCE HELICAL PSAC1500499 LOW PROFILE BRACKET AND T-PIPE RATINGS					
T-PIPE DESIGNATIONS FOR THE PSA1500499 BRACKET	ULTIMATE MECHANICAL STRENGTH ^{1,3} (lb) [kN]	PILE SIZE (in) [mm]	PRODUCT SERIES	MAX WORKING CAPACITY ^{2,3} BASED ON PRODUCT SERIES (lb) [kN]	FEATURES
PSAC1500503	30,000 [133]	1-1/2 [38] square	SS5 SS150	15,000 [67]	Lowest cost with SS5
PSAC2780003	30,000 [133]	2-7/8 [73] round	RS2875.203	15,000 [67]	Lowest cost with RS2875.203

Notes:

1 Ultimate mechanical strength is for the bracket body and T-pipe combination.

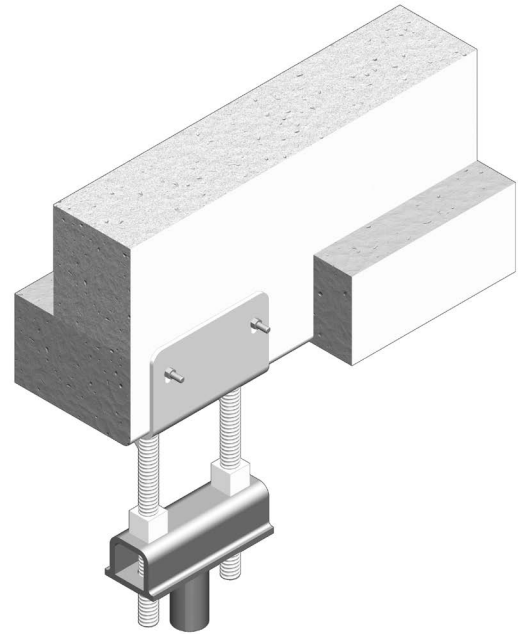
2 The capacity of Chance helical pile systems is a function of many individual elements including the capacity of the foundation, bracket, pile shaft, helix plate, and bearing stratum; the strength of the foundation-to-bracket connection; and the quality of the helical pile installation. The fifth column shows typical working capacities of the Chance helical pile system based upon maximum shaft exposure of 2 feet and soil strength having a minimum Standard Penetration Test (SPT) blow count (N_{60}) of 4. Actual capacities could be higher or lower depending on the above factors.

3 The ultimate capacity of the system, i.e., bracket, T-pipe, and pile shaft, can be increased to the pile shaft compression capacity limit as shown on the product data pages provided the pile shaft is reinforced using a pipe sleeve or grout column. The maximum working capacity shall not be greater than one half the ultimate mechanical strength of the bracket and T-pipe combination given above.



CHANCE® HELICAL DIRECT JACK UNDERPINNING BRACKETS

- Used with Chance Type SS5 & SS150 1-1/2" and SS175 1-3/4" square shaft helical piles; and Type RS2875.276 2-7/8" OD and Type RS3500.300 3-1/2" OD round shaft helical piles
- Use for lifts up to 4" (10 cm)
- All direct jack underpinning brackets include:
 - Foundation bracket
 - T-pipe
 - Two thread bar nuts



ORDER SEPARATELY: Two 1/2" (13 mm) diameter concrete anchor bolts per pile as required.

Standard finish is hot-dip galvanized per ASTM A153.

The bracket body and T-pipe are packaged together.

See table below for system (bracket/pile shaft) ratings.

CHANCE HELICAL DIRECT JACK UNDERPINNING BRACKETS					
DIRECT JACK CATALOG NUMBER	ULTIMATE MECHANICAL STRENGTH ^{1,3} (lb) [kN]	PILE SIZE (in) [mm]	PRODUCT SERIES	MAX WORKING CAPACITY ^{2,3} BASED ON PRODUCT SERIES (lb) [kN]	FEATURES
C1500738	70,000 [311]	1-1/2 [38] square	SS5 SS150	35,000 [156]	Lowest cost
C1500733	100,000 [445]	1-3/4 [44] square	SS175	50,000 [222]	Highest capacity
C1500840	72,000 [320]	2-7/8 [73] round	RS2875.276	36,000 [160]	
C1500841	91,000 [405]	3-1/2 [89] round	RS3500.300	45,500 [202]	

Notes:
 1 Ultimate mechanical strength is for the bracket body and T-pipe combination.
 2 The capacity of Chance helical pile systems is a function of many individual elements including the capacity of the foundation, bracket, pile shaft, helix plate, and bearing stratum; the strength of the foundation-to-bracket connection; and the quality of the helical pile installation. The fifth column shows typical working capacities of the Chance helical pile system based upon maximum shaft exposure of 2 feet and soil strength having a minimum Standard Penetration Test (SPT) blow count (N_{60}) of 4. Actual capacities could be higher or lower depending on the above factors.
 3 The ultimate capacity of the system, i.e., bracket, T-pipe, and pile shaft, can be increased to the pile shaft compression capacity limit as shown on the product data pages provided the pile shaft is reinforced using a pipe sleeve or grout column. The maximum working capacity shall not be greater than one half the ultimate mechanical strength of the bracket and T-pipe combination given above.



BUILDING CODE COMPLIANCE PER ICC-ES ESR-2794

The following tables provide the nominal, LRFD design, and ASD allowable compression strengths of C1500738, C1500840, and C1500841 foundation repair brackets, T-pipes, and Type SS5, RS2875, and RS3500 helical piles as evaluated per ICC-ES Acceptance Criteria AC358. These strengths are published in ICC-ES ESR-2794. The strengths listed are based on three different concrete foundation strengths and two different soils

conditions—firm and soft. The pile head is assumed to be fixed within the bracket assembly, and the piles are assumed to be braced.

Per the International Building Code (IBC) 2006 Section 1808.2.9.2 & IBC 2021 Section 1810.2.1, piles can be considered fixed and laterally supported at 5 feet below the ground surface in firm soil and at 10 feet in soft soil.

NOMINAL STRENGTHS OF C1500738 REMEDIAL REPAIR BRACKETS & HELICAL PILES^{1,2,3,4,5}

BRACKET CATALOG NUMBER	T-PIPE CATALOG NUMBER	PILE MODEL	NOMINAL STRENGTH IN AXIAL COMPRESSION (kip) [kN]					
			2500 psi CONCRETE ⁵		3000 psi CONCRETE ⁵		4000 psi CONCRETE ⁵	
			FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL
C1500738	Incl. w/ bracket	SS5	79.4 [353]	25.9 [115]	79.4 [353]	25.9 [115]	79.4 [353]	25.9 [115]
C1500840	Incl. w/ bracket	RS2875.203	80.7 [359]	63.0 [280]	80.7 [359]	63.0 [280]	80.7 [359]	63.0 [280]
C1500840	Incl. w/ bracket	RS2875.276	85.1 [379]	70.2 [312]	85.1 [379]	70.2 [312]	85.1 [379]	70.2 [312]
C1500841	Incl. w/ bracket	RS3500	100 [445]	95.3 [424]	100 [445]	100 [445]	100 [445]	100 [445]

LRFD DESIGN STRENGTHS OF C1500738 REMEDIAL REPAIR BRACKETS & HELICAL PILES^{1,2,3,4,5}

BRACKET CATALOG NUMBER	T-PIPE CATALOG NUMBER	PILE MODEL	LRFD DESIGN STRENGTH IN AXIAL COMPRESSION (kip) [kN]					
			2500 psi CONCRETE ⁵		3000 psi CONCRETE ⁵		4000 psi CONCRETE ⁵	
			FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL
C1500738	Incl. w/ bracket	SS5	71.4 [318]	23.3 [104]	71.4 [318]	23.3 [104]	71.4 [318]	23.3 [104]
C1500840	Incl. w/ bracket	RS2875.203	72.6 [323]	56.7 [252]	72.6 [323]	56.7 [252]	72.6 [323]	56.7 [252]
C1500840	Incl. w/ bracket	RS2875.276	76.6 [341]	63.1 [281]	76.6 [341]	63.1 [281]	76.6 [341]	63.1 [281]
C1500841	Incl. w/ bracket	RS3500	85.8 [382]	85.8 [382]	90 [400]	90 [400]	90 [400]	90 [400]

ASD ALLOWABLE STRENGTHS OF C1500738 REMEDIAL REPAIR BRACKETS & HELICAL PILES^{1,2,3,4,5}

BRACKET CATALOG NUMBER	T-PIPE CATALOG NUMBER	PILE MODEL	ASD ALLOWABLE STRENGTH IN AXIAL COMPRESSION (kip) [kN]					
			2500 psi CONCRETE ⁵		3000 psi CONCRETE ⁵		4000 psi CONCRETE ⁵	
			FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL	FIRM SOIL	SOFT SOIL
C1500738	Incl. w/ bracket	SS5	47.5 [211]	15.5 [69]	47.5 [211]	15.5 [69]	47.5 [211]	15.5 [69]
C1500840	Incl. w/ bracket	RS2875.203	48.3 [215]	37.7 [168]	48.3 [215]	37.7 [168]	48.3 [215]	37.7 [168]
C1500840	Incl. w/ bracket	RS2875.276	51 [227]	42 [187]	51 [227]	42 [187]	51 [227]	42 [187]
C1500841	Incl. w/ bracket	RS3500	60 [267]	60 [267]	60 [267]	60 [267]	60 [267]	60 [267]

Notes:

For SI: 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ICC-ES ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings are based on a design corrosion level of 50 years and presume the supported structure is braced in accordance with Section 1810.2.2 of the 2021 IBC (Section 1808.2.5 of the 2006 IBC).

³ Strength ratings apply to the specific bracket, T-pipe, and pile/anchor models listed.

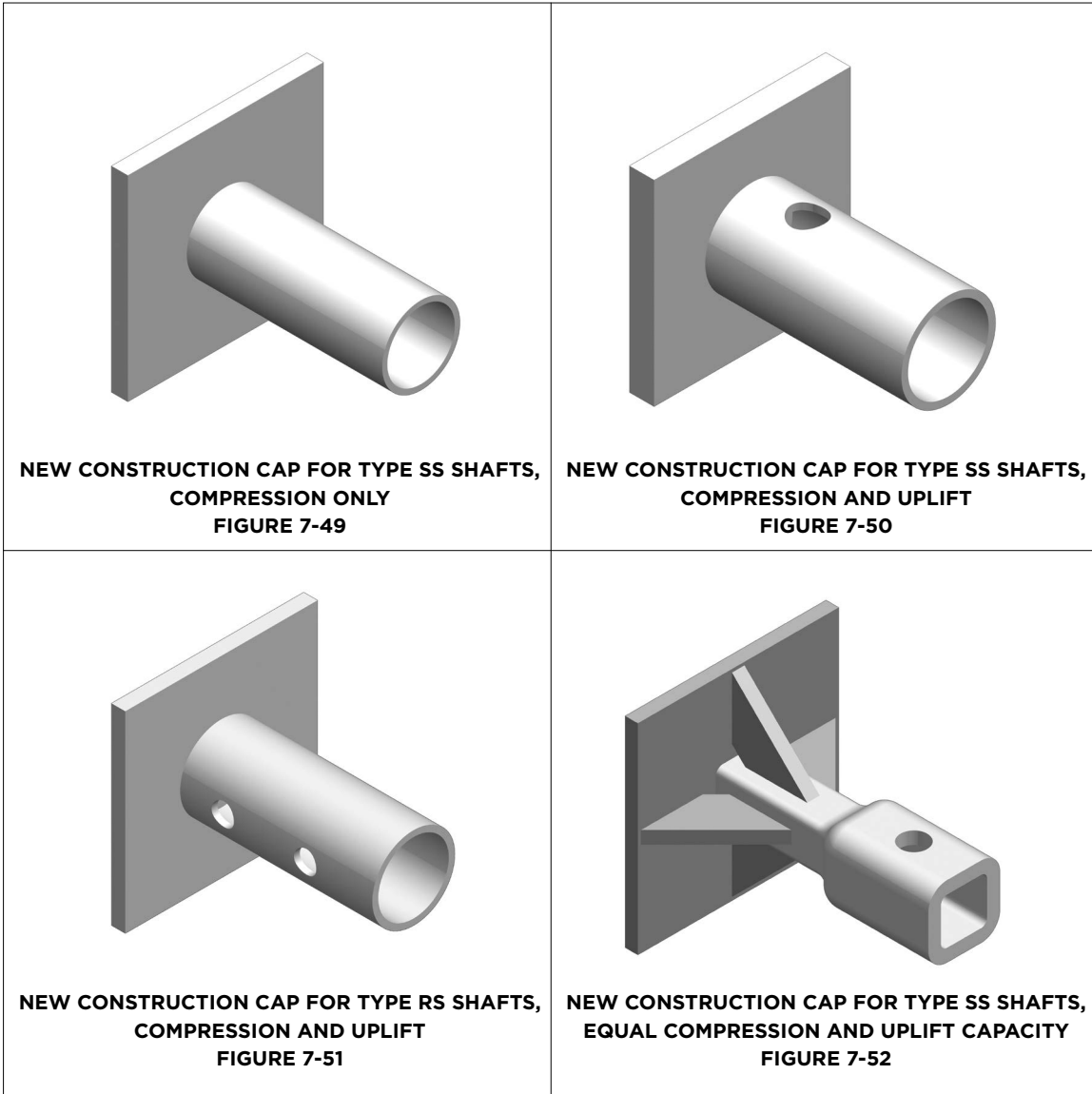
⁴ See Section 4.1.2 of ICC-ES ESR-2794 for applicable limit states that must be evaluated by a registered design professional.

⁵ Refer to the specified compressive strength of concrete at 28 days.

NEW CONSTRUCTION PILE CAPS

The Chance® new construction pile caps are designed for use with the Chance Type SS square shaft and Type RS round shaft helical piles for embedment in cast-in-place concrete foundations. Each new construction pile cap consists of either one bearing plate and one steel tube sleeve that are factory-welded together to form the cap, or one bearing plate, two rebars and one steel tube sleeve that are factory-welded together. The plate-type pile caps are designed to be used in spread footings, grade beams, structural slabs, and reinforced

concrete pile caps. The rebar cap is designed to be used in grade beams and reinforced pile caps. The concrete foundation and the interaction of the pile shaft, new construction pile cap, and concrete footing for moment transfer, as applicable, must be designed and justified with due consideration to all applicable limit states and the direction and eccentricity of applied loads, including reactions provided by the brackets, acting on the concrete foundation. For preliminary design guidelines for reinforced pile caps refer to Section 4.





CHANCE® HELICAL NEW CONSTRUCTION PILE CAPS, TABLE 7-7				
PILE CAP DESIGNATION	DESIGN (WORKING) LOAD (kip) [kN]	PLATE SIZE (SQUARE)	PIPE OD & LENGTH	DESCRIPTION
C1500458	40 [178] compression	6" x 6" x 1/2"	2-1/2" x 6"	Fits SS5/SS150 and RS2875.203; use for compression only
C1500459	60 [267] compression	6" x 6" x 3/4"	3" x 6"	Fits SS175; use for compression only
C1500465	40 [178] compression 20 [89] uplift	6" x 6" x 1/2"	2-1/2" x 6"	Fits SS5/SS150; use for uplift and compression
C1500467	60 [267] compression 30 [133] uplift	6" x 6" x 3/4"	3" x 6"	Fits SS175; use for uplift and compression
C1500777	35 [156] compression	7" x 7" x 1/2"	2-1/2" x 6"	Fits SS5/SS150; use for compression only
C1500778	52.5 [234] compression	8" x 8" x 1/2"	2-7/8" x 6"	Fits SS175; use for compression only
C1500779	75 [334] compression	12" x 12" x 1/2"	3-1/2" x 6"	Fits SS200; use for compression only
C1500780	100 [445] compression	12" x 12" x 1/2"	3-1/2" x 6"	Fits SS225; use for compression only
C1500781	36 [160] compression	7" x 7" x 1/2"	3-1/2" x 6"	Fits RS2875; use for compression only
C1500782	50 [222] compression	10" x 10" x 1/2"	4-1/2" x 6"	Fits RS3500; use for compression only
C1500783	70 [311] compression	12" x 12" x 1/2"	5-9/16" x 6"	Fits RS4500; use for compression only
C1500793	(35) [156] compression (23) [102] uplift	7" x 7" x 1/2"	2-1/2" x 6"	Fits SS5/SS150; use for uplift and compression
C1500794	52.5 [234] compression 37 [165] uplift	8" x 8" x 1/2"	2-7/8" x 6"	Fits SS175; use for uplift and compression
C1500795	75 [334] compression 45 [200] uplift	12" x 12" x 1/2"	3-1/2" x 6"	Fits SS200; use for uplift and compression
C1500796	100 [445] compression 40 [178] uplift	12" x 12" x 1/2"	3-1/2" x 6"	Fits SS225; use for uplift and compression
C1500797	36 [160] compression 36 [160] uplift	7" x 7" x 1/2"	3-1/2" x 7"	Fits RS2875; use for uplift and compression
C1500798	50 [222] compression 50 [222] uplift	10" x 10" x 1/2"	4-1/2" x 7"	Fits RS3500; use for uplift and compression
C1500799	70 [311] compression 70 [311] uplift	12" x 12" x 1/2"	5-9/16" x 7"	Fits RS4500; use for uplift and compression
C1502108	50 [222] compression	8" x 8" x 1/2"	3-1/2" x 7"	Fits RS2875.276 HCP; use for compression only
C1503165	140 [623] compression 140 [623] uplift	12" x 12" x 1"	6-5/8" x 10-1/2"	Fits RS5500.361; use for uplift and compression
C1503166	140 [623] compression	12" x 12" x 1"	6-5/8" x 10-1/2"	Fits RS5500.361; use for compression only
C1073119	180 [800] compression (130) [578] uplift	14" x 14" x 1-1/4"	6" x 9-1/4"	Fits RS7000.362; use for uplift and compression
C1073431	300 [1334] compression 235 [1045] tension	20" x 20" x 1-1/2"	10-3/4" x 12"	Fits RS9625.395; use for uplift and compression



BUILDING CODE COMPLIANCE PER ICC-ES ESR-2794

The following tables provide the nominal, LRFD design, and ASD allowable compression strengths of new construction pile caps used with Type SS5 and SS175 square shaft helical piles, and Type RS2875.203, RS2875.276, RS3500.300, and RS4500.337 round shaft helical piles as evaluated per ICC-ES Acceptance Criteria AC358. The last table on page 7-95 provides the nominal, LRFD design, and ASD allowable tension strengths of new construction pile caps used with Type SS5 and SS175 square shaft helical piles, and Type RS2875.203, RS2875.276, RS3500.300, and RS4500.337 round shaft helical piles as evaluated per ICC-ES Acceptance Criteria AC358. These strengths are published in ICC-ES ESR-2794. The

strengths listed are based on three different concrete foundation strengths and two different soils conditions—firm and soft. The pile head is assumed to be either pinned or fixed within the concrete foundation depending on cover and reinforcing, and the piles are assumed to be braced. The helical pile must be embedded at least 7.5 inches into the concrete foundation when designed as fixed end condition.

Per the International Building Code (IBC) 2006 Section 1808.2.9.2 & IBC 2021 Section 1810.2.1, piles can be considered fixed and laterally supported at 5 feet below the ground surface in firm soil and at 10 feet in soft soil.

NOMINAL STRENGTHS OF NEW CONSTRUCTION PILE CAPS LOADED IN COMPRESSION^{1,2,3,4,5,6,7,8,9}

CATALOG NUMBER ⁹	PILE MODEL	NOMINAL COMPRESSION STRENGTH (kip) {kN}											
		2500 psi CONCRETE ⁶				3000 psi CONCRETE ⁶				4000 psi CONCRETE ⁶			
		FIRM SOIL		SOFT SOIL		FIRM SOIL		SOFT SOIL		FIRM SOIL		SOFT SOIL	
		PINNED	FIXED	PINNED	FIXED	PINNED	FIXED	PINNED	FIXED	PINNED	FIXED	PINNED	FIXED
C1500458(G)	SS5	54.4 {242}	60.0 {267}	13.6 {60}	26.6 {118}	54.4 {242}	62.3 {277}	13.6 {60}	26.6 {118}	54.4 {242}	66.9 {298}	13.6 {60}	26.6 {118}
C1500459(G)	SS175	100 {445}	100 {445}	25.8 {115}	50.5 {225}	100 {445}	100 {445}	25.8 {115}	50.5 {225}	100 {445}	100 {445}	25.8 {115}	50.5 {225}
C1500465(G)	SS5	54.4 {242}	60.0 {267}	13.6 {60}	26.6 {118}	54.4 {242}	62.3 {277}	13.6 {60}	26.6 {118}	54.4 {242}	66.9 {298}	13.6 {60}	26.6 {118}
C1500467(G)	SS175	100 {445}	100 {445}	25.8 {115}	50.5 {225}	100 {445}	100 {445}	25.8 {115}	50.5 {225}	100 {445}	100 {445}	25.8 {115}	50.5 {225}
C1500781(G) C1500797(G)	RS2875.276	71.5 {318} [71.5 {318}]	71.5 {318} [71.5 {318}]	55.2 {245} [42.4 {188.6}]	71.5 {318} [64.6 {287.4}]	80.1 {356.3} [80.1 {356.3}]	80.1 {356.3} [80.1 {356.3}]	55.2 {245} [42.4 {188.6}]	73.9 {328} [64.6 {287.4}]	86.3 {383} [80.8 {359.4}]	92.9 {413} [89.8 {399.5}]	55.2 {245} [42.4 {188.6}]	73.9 {328} [64.6 {287.4}]
C1501356(G) C1501357(G)	RS3500.300	121.9 {542.2} [116.7 {519.1}]	128.0 {569.4} [125.2 {557}]	90.7 {403} [76.2 {339}]	110 {489.3} [100.7 {448}]	121.9 {542.2} [116.7 {519.1}]	128.0 {569.4} [125.2 {557}]	90.7 {403} [76.2 {339}]	110 {489.3} [100.7 {448}]	121.9 {542.2} [116.7 {519.1}]	128 {569.4} [125.2 {557}]	90.7 {403} [76.2 {339}]	110 {489.3} [100.7 {448}]
C1500781(G) C1501977(G)	RS2875.203	71.5 {318} [71.5 {318}]	71.5 {318} [71.5 {318}]	45.2 {201.0} [33.2 {147.7}]	66.1 {294.0} [56.0 {249.1}]	80.1 {356.3} [73.9 {328.7}]	80.1 {356.3} [80.1 {356.3}]	45.1 {201.0} [33.2 {147.7}]	66.1 {294.0} [56.0 {249.1}]	80.1 {356.3} [73.9 {328.7}]	87.1 {387.4} [84.2 {374.5}]	45.2 {201.0} [33.2 {147.7}]	66.1 {294.0} [56.0 {249.1}]
C1072726(G) C1072727(G)	RS4500.337	166.1 {739} [166.1 {739}]	166.1 {739} [166.1 {739}]	156.3 {695.3} [141.0 {627.2}]	166.1 {739} [166.1 {739}]	186.2 {828.3} [181.6 {807.8}]	186.2 {828.3} [186.2 {828.3}]	156.3 {695.3} [141.0 {627.2}]	175.3 {779.8} [166.3 {739.7}]	186.3 {828.7} [181.6 {807.8}]	191.7 {852.7} [189.2 {841.6}]	156.3 {695.3} [141.0 {627.2}]	175.3 {779.8} [166.3 {739.7}]

Notes:

For SI: 1 inch = 25.4 mm, 1 kip = 4.448 kN, 1 kip-in = 113 N-m.

¹ Refer to Section 4.1.3 of ICC-ES ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings include allowance for corrosion of zinc-coated new construction pile caps over a 50-year service life and presume the supported structure is braced in accordance with IBC Section 1810.2.2.

³ Capacities apply to the specific pile cap and pile models listed.

⁴ The fixed end condition requires that the foundation itself be fixed and that the pile and pile cap be embedded in the foundation with adequate concrete cover and reinforcing to resist 56.4 kip-in, 116 kip-in, 71.66 kip-in, 138.3 kip-in, 138.3 kip-in, and 263.72 kip-in nominal bending moment for SS5, SS175, RS2875 (including RS2875.203 and RS2875.276), RS3500, RS3500/SS175 combo, and RS4500 pile models, respectively. The center of the shaft must be at least 6 inches away from the end/corner of the concrete footing.

⁵ See Section 4.1.2 of ICC-ES ESR-2794 for applicable limit states that must be evaluated by a registered design professional.

⁶ Refer to the specified compressive strength of concrete at 28 days [minimum of 24 MPa is required under ADIBC Appendix L, Section 5.5.1].

⁷ Values in brackets [] are for use in Seismic Design Categories D, E, and F.

⁸ The concrete footing design and reinforcement design, including the concrete thickness above the new construction pile cap, must be determined by a registered design professional.

⁹ (G): The cap is available as galvanized and nongalvanized (bare steel). Catalog items with "G" suffix are galvanized and without "G" are bare steel.



LRFD DESIGN STRENGTHS OF NEW CONSTRUCTION PILE CAPS LOADED IN COMPRESSION^{1,2,3,4,5,6,7,8,9}

CATALOG NUMBER ⁹	PILE MODEL	LRFD DESIGN COMPRESSION STRENGTH (kip) {kN}											
		2500 psi CONCRETE ⁶				3000 psi CONCRETE ⁶				4000 psi CONCRETE ⁶			
		FIRM SOIL		SOFT SOIL		FIRM SOIL		SOFT SOIL		FIRM SOIL		SOFT SOIL	
		PINNED	FIXED	PINNED	FIXED	PINNED	FIXED	PINNED	FIXED	PINNED	FIXED	PINNED	FIXED
C1500458(G)	SS5	48.9 {218}	50.6 {225}	12.2 {54}	24.0 {107}	48.9 {218}	52.0 {231}	12.2 {54}	24.0 {107}	48.9 {218}	54.7 {243}	12.2 {54}	24.0 {107}
C1500459(G)	SS175	79.2 {352}	79.2 {352}	23.2 {103}	45.4 {202}	90.0 {400}	90.0 {400}	23.2 {103}	45.4 {202}	90.0 {400}	90.0 {400}	23.2 {103}	45.4 {202}
C1500465(G)	SS5	48.9 {218}	50.6 {225}	12.2 {54}	24.0 {107}	48.9 {218}	52.0 {231}	12.2 {54}	24.0 {107}	48.9 {218}	54.7 {243}	12.2 {54}	24.0 {107}
C1500467(G)	SS175	79.2 {352}	79.2 {352}	23.2 {103}	45.4 {202}	90.0 {400}	90.0 {400}	23.2 {103}	45.4 {202}	90.0 {400}	90.0 {400}	23.2 {103}	45.4 {202}
C1500781(G) C1500797(G)	RS2875.276	58.9 {262} [58.9 {262}]	58.9 {262} [58.9 {262}]	49.7 {221} [38.2 {169.9}]	58.9 {262} [58.2 {258.9}]	65.0 {289} [65.0 {289.1}]	65.0 {289} [65.0 {289.1}]	49.7 {221} [38.2 {169.9}]	65.0 {289} [58.2 {258.9}]	76.3 {339} [72.8 {323.8}]	76.3 {339} [76.3 {339}]	49.7 {221} [38.2 {169.9}]	66.5 {295} [58.2 {258.9}]
C1501356(G) C1501357(G)	RS3500.300	108.8 {484} [105.1 {467.5}]	108.8 {484} [108.8 {484}]	81.6 {362} [68.6 {305.1}]	99.0 {440.4} [90.6 {403}]	109.7 {488} [105.1 {467.5}]	115.2 {512.4} [112.7 {501.3}]	81.6 {362} [68.6 {305.1}]	99.0 {440.4} [90.6 {403}]	109.7 {488} [105.1 {467.5}]	115.2 {512.4} [112.7 {501.3}]	81.6 {362} [68.6 {305.1}]	99.0 {440.4} [90.6 {403}]
C1500781(G) C1501977(G)	RS2875.203	58.9 {262} [58.9 {262}]	58.9 {262.0} [58.9 {262}]	41.4 {184.2} [29.9 {133}]	58.9 {262.0} [50.4 {224.2}]	65.0 {289.1} [65.0 {289.1}]	65.0 {289.1} [65.0 {289.1}]	41.4 {184.2} [29.9 {133}]	59.5 {264.7} [50.4 {224.2}]	65.3 {290.5} [65.3 {290.5}]	65.3 {290.5} [65.3 {290.5}]	41.4 {184.2} [29.9 {133}]	59.5 {264.7} [50.4 {224.2}]
C1072726(G) C1072727(G)	RS4500.337	130.4 {580} [130.4 {580}]	130.4 {580} [130.4 {580}]	130.4 {580} [126.9 {564.5}]	130.4 {580} [130.4 {580}]	142.4 {633.4} [142.4 {633.4}]	142.4 {633.4} [142.4 {633.4}]	140.7 {625.9} [126.9 {564.5}]	142.4 {633.4} [142.4 {633.4}]	145.3 {646.3} [145.3 {646.3}]	145.3 {646.3} [145.3 {646.3}]	140.7 {625.9} [126.9 {564.5}]	145.3 {646.3} [145.3 {646.3}]

Notes:

For SI: 1 inch = 25.4 mm, 1 kip = 4.448 kN, 1 kip-in = 113 N-m.

¹ Refer to Section 4.1.3 of ICC-ES ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings include allowance for corrosion of zinc-coated new construction pile caps over a 50-year service life and presume the supported structure is braced in accordance with IBC Section 1810.2.2.

³ Capacities apply to the specific pile cap and pile models listed.

⁴ The fixed end condition requires that the foundation itself be fixed and that the pile and pile cap be embedded in the foundation with adequate concrete cover and reinforcing to resist 56.4 kip-in, 116 kip-in, 71.66 kip-in, 138.3 kip-in, 138.3 kip-in, and 263.72 kip-in nominal bending moment for SS5, SS175, RS2875 (including RS2875.203 and RS2875.276), RS3500, RS3500/SS175 combo, and RS4500 pile models, respectively. The center of the shaft must be at least 6 inches away from the end/corner of the concrete footing.

⁵ See Section 4.1.2 of ICC-ES ESR-2794 for applicable limit states that must be evaluated by a registered design professional.

⁶ Refer to the specified compressive strength of concrete at 28 days [minimum of 24 MPa is required under ADIBC Appendix L, Section 5.5.1].

⁷ Values in brackets [] are for use in Seismic Design Categories D, E, and F.

⁸ The concrete footing design and reinforcement design, including the concrete thickness above the new construction pile cap, must be determined by a registered design professional.

⁹ (G): The cap is available as galvanized and nongalvanized (bare steel). Catalog items with "G" suffix are galvanized and without "G" are bare steel.



ASD ALLOWABLE STRENGTHS OF NEW CONSTRUCTION PILE CAPS LOADED IN COMPRESSION^{1,2,3,4,5,6,7,8,9}

CATALOG NUMBER ⁹	PILE MODEL	ASD ALLOWABLE COMPRESSION STRENGTH (kip) {kN}											
		2500 psi CONCRETE ⁶				3000 psi CONCRETE ⁶				4000 psi CONCRETE ⁶			
		FIRM SOIL		SOFT SOIL		FIRM SOIL		SOFT SOIL		FIRM SOIL		SOFT SOIL	
		PINNED	FIXED	PINNED	FIXED	PINNED	FIXED	PINNED	FIXED	PINNED	FIXED	PINNED	FIXED
C1500458(G)	SS5	(32.6) {145}	(33.7) {150}	(8.1) {36}	(16.0) {71}	(32.6) {145}	(34.6) {154}	(8.1) {36}	(16.0) {71}	(32.6) {145}	(36.4) {162}	(8.1) {36}	(16.0) {71}
C1500459(G)	SS175	(52.7) {234}	(52.7) {234}	(15.4) {69}	(30.2) {134}	(60.0) {267}	(60.0) {267}	(15.4) {69}	(30.2) {134}	(60.0) {267}	(60.0) {267}	(15.4) {69}	(30.2) {134}
C1500465(G)	SS5	(32.6) {145}	(33.7) {150}	(8.1) {36}	(16.0) {71}	(32.6) {145}	(34.6) {154}	(8.1) {36}	(16.0) {71}	(32.6) {145}	(36.4) {162}	(8.1) {36}	(16.0) {71}
C1500467(G)	SS175	(52.7) {234}	(52.7) {234}	(15.4) {69}	(30.2) {134}	(60.0) {267}	(60.0) {267}	(15.4) {69}	(30.2) {134}	(60.0) {267}	(60.0) {267}	(15.4) {69}	(30.2) {134}
C1500781(G) C1500797(G)	RS2875.276	37.6 {167}	37.6 {167}	33.0 {146}	37.6 {167}	41.8 {186}	41.8 {186}	33.0 {146}	41.8 {186}	49.5 {220}	49.5 {220}	33.0 {146}	44.3 {197}
C1501356(G) C1501357(G)	RS3500.300	68.7 {305.6}	68.7 {305.6}	54.3 {241}	65.9 {293.1}	73.0 {324.7}	75.9 {337.6}	54.3 {241}	65.9 {293.1}	73.0 {324.7}	76.6 {340.7}	54.3 {241}	65.9 {293.1}
C1500781(G) C1501977(G)	RS2875.203	37.6 {167.3}	37.6 {167.3}	27.5 {122.3}	37.6 {167.3}	41.8 {185.9}	41.8 {185.9}	27.5 {122.3}	39.6 {176.1}	43.5 {195.5}	43.5 {195.5}	27.5 {122.3}	39.6 {176.1}
C1072726(G) C1072727(G)	RS4500.337	88.7 {394.6}	88.7 {394.6}	88.7 {394.6}	88.7 {394.6}	96.9 {431}	96.9 {431}	93.6 {416.4}	96.9 {431}	96.9 {431}	96.9 {431}	93.6 {416.4}	96.9 {431}

Notes:

For SI: 1 inch = 25.4 mm, 1 kip = 4.448 kN, 1 kip-in = 113 N-m.

¹ Refer to Section 4.1.3 of ICC-ES ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings include allowance for corrosion of zinc-coated new construction pile caps over a 50-year service life and presume the supported structure is braced in accordance with IBC Section 1810.2.2.

³ Capacities apply to the specific pile cap and pile models listed.

⁴ The fixed end condition requires that the foundation itself be fixed and that the pile and pile cap be embedded in the foundation with adequate concrete cover and reinforcing to resist 56.4 kip-in, 116 kip-in, 71.66 kip-in, 138.3 kip-in, 138.3 kip-in, and 263.72 kip-in nominal bending moment for SS5, SS175, RS2875 (including RS2875.203 and RS2875.276), RS3500, RS3500/SS175 combo, and RS4500 pile models, respectively. The center of the shaft must be at least 6 inches away from the end/corner of the concrete footing.

⁵ See Section 4.1.2 of ICC-ES ESR-2794 for applicable limit states that must be evaluated by a registered design professional.

⁶ Refer to the specified compressive strength of concrete at 28 days [minimum of 24 MPa is required under ADIBC Appendix L, Section 5.5.1].

⁷ Values in brackets [] are for use in Seismic Design Categories D, E, and F.

⁸ The concrete footing design and reinforcement design, including the concrete thickness above the new construction pile cap, must be determined by a registered design professional.

⁹ (G): The cap is available as galvanized and nongalvanized (bare steel). Catalog items with "G" suffix are galvanized and without "G" are bare steel.



NOMINAL, LRFD DESIGN, & ASD ALLOWABLE STRENGTHS OF NEW CONSTRUCTION PILE CAPS LOADED IN TENSION^{1,2,3,4,5,6,7,8}

CATALOG NUMBER ⁸	PILE MODEL	NOMINAL, LRFD DESIGN, & ASD ALLOWABLE STRENGTHS IN TENSION (kip) [kN]								
		2500 psi CONCRETE ⁵			3000 psi CONCRETE ⁵			4000 psi CONCRETE ⁵		
		NOMINAL STRENGTH	DESIGN STRENGTH	ALLOWABLE STRENGTH	NOMINAL STRENGTH	DESIGN STRENGTH	ALLOWABLE STRENGTH	NOMINAL STRENGTH	DESIGN STRENGTH	ALLOWABLE STRENGTH
C1500465(G)	SS5	56.2 [250]	42.1 [187]	28.1 [125]	56.2 [250]	42.1 [187]	28.1 [125]	56.2 [250]	42.1 [187]	28.1 [125]
C1500467(G)	SS175	78.9 [351]	59.2 [263]	39.5 [176]	78.9 [351]	59.2 [263]	39.5 [176]	78.9 [351]	59.2 [263]	39.5 [176]
C150797(G)	RS2875.276	95.0 [422]	70.4 [313]	47.5 [211]	95.0 [422]	71.3 [317]	47.5 [211]	95.0 [422]	71.3 [317]	47.5 [211]
C1501357(G)	RS3500.300	100 [444]	76.9 [342]	51.9 [231]	100 [444]	77.9 [346]	51.9 [231]	100 [444]	77.9 [346]	51.9 [231]
C1500797(G)	RS2875.203	87 [387]	65.3 [290.5]	43.5 [193.5]	87 [387]	65.3 [290.5]	43.5 [193.5]	87 [387]	65.3 [290.5]	43.5 [193.5]
C1072727(G)	RS4500.337	140.9 [626.8]	105.7 [470.2]	70.5 [313.6]	140.9 [626.8]	105.7 [470.2]	70.5 [313.6]	140.9 [626.8]	105.7 [470.2]	70.5 [313.6]
C1500793(G)	SS5	45.2 [201.1]	33.9 [150.8]	22.6 [100.5]	45.2 [201.1]	33.9 [150.8]	22.6 [100.5]	45.2 [201.1]	33.9 [150.8]	22.6 [100.5]
C1500794(G)	SS175	60.8 [270.5]	45.6 [202.8]	30.4 [135.2]	60.8 [270.5]	45.6 [202.8]	30.4 [135.2]	60.8 [270.5]	45.6 [202.8]	30.4 [135.2]

Notes:

For SI: 1 inch = 25.4 mm, 1 kip = 4.448 kN.

¹ Refer to Section 4.1.3 of ICC-ES ESR-2794 for descriptions of fixed condition, pinned condition, soft soil, and firm soil.

² Strength ratings include allowance for corrosion of zinc-coated new construction pile caps over a 50-year service life.

³ Capacities apply to the specific pile cap and pile models listed.

⁴ See Sections 4.1.2 and 5.7 of ICC-ES ESR-2794 for applicable limit states that must be evaluated by a registered design professional.

⁵ The specified compressive strength of concrete at 28 days must not be less than 2,500 psi (17.2 MPa) [minimum of 24 MPa is required under ADIBC Appendix L, Section 5.5.1].

⁶ Anchorage design must comply with the requirements of Section 1810.3.11.2 of the IBC for Seismic Design Categories D, E, and F.

⁷ The concrete footing design, reinforcement design, and anchorage capacity between the bracket and concrete must be determined by a registered design professional. The bracket anchorage with concrete may control the capacity.

⁸ (G): The cap is available as galvanized and nongalvanized (bare steel). Catalog items with "G" suffix are galvanized and without "G" are bare steel.



SECTION 8: DESIGN EXAMPLES

CONTENTS

Design Example 1: Atlas Resistance Piers.....8-2

Design Example 2: Atlas Resistance Piers with Integrated Tieback.....8-3

Design Example 3: Helical Pile Foundation for New Construction 8-5

Design Example 4: Light Commercial Structure..... 8-6

Design Example 5: Helical Pulldown Micropiles for New Construction8-10

Design Example 6: Helical Piles for Boardwalks8-14

Design Example 7: Helical Piles for Boardwalks with Lateral Support..... 8-15

Design Example 8: Helical Tieback Anchors In Clay 8-16

Design Example 9: Helical Tieback Anchors In Sand..... 8-17

Design Example 10: Soil Screw Retention Wall System 8-19

Design Example 11: Helical Piles/Anchors for Telecommunication Towers8-24

Design Example 12: Helical Anchors for Pipeline Buoyancy Control 8-31

Design Example 13: Type RS Helical Piles for Lateral Support..... 8-35

Design Example 14: Instant Foundations for Street Light Supports8-36

Design Example 15: Foundation Earth Pressure Resistance8-39

Design Example 16: Buckling Example Using the Davisson Method..... 8-40

Design Example 17: Buckling Example Using the Finite-Difference Method8-41

Design Example 18: Buckling Example Using the Finite-Element Method8-43

Design Example 19: Monopole Foundation with Steel Grillage & RS5500 Helical Piles....8-44

DESIGN EXAMPLES

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 should be conducted and state and local building codes and authorities should be consulted 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 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.



DESIGN EXAMPLE 1: ATLAS RESISTANCE PIERS

TYPE OF STRUCTURE

The structure is a two-story, 20' x 40' frame residence with full brick veneer siding located in the Midwest. The house sits on 8" thick by 8' high cast concrete basement walls with steel reinforced concrete footings 1'-8" wide by 1' thick. The roof is composition shingles over 1/2" plywood decking and felt underlayment. There is six feet of peaty clay soil overburden present.

PRELIMINARY INVESTIGATION

Settlement of up to 2-1/2" is evident in portions of the structure. Checking with local building officials reveals no special controlling codes for underpinning existing structures that must be observed. Preliminary geotechnical information indicates the footing is situated in peaty clay type soil with Standard Penetration Test (SPT) blow count (N) values of six and higher. This soil extends to a depth of 15 feet where a dense glacial till exists. It is determined that the glacial till layer will serve as an adequate bearing stratum for the Atlas Resistance® piers.

PRELIMINARY ESTIMATE OF TOTAL LOAD ON FOOTING

EQUATION 8-1

$$\begin{aligned}
 P &= DL + LL + SL + W \\
 &= (1,890 + 667 + 120 + 2,310) \\
 &= 4,987 \text{ lb/ft}
 \end{aligned}$$

where

- P = Total live load on perimeter footing
- DL = Dead load (1,890 lb/ft)
- LL = Live load (667 lb/ft)
- SL = Snow load
 - = $S_K(w)L / 2(w + L)$
 - where L and w are the building dimensions
 - = $18 \text{ lb/ft}^2 \times (800 / 120) \text{ ft} = 120 \text{ lb/ft}$
- S_K = Snow load requirement factor
 - = 18 lb/ft^2 (for this example)
- W = Soil load
 - = $W_{b1} + W_{b2} = (330 + 1,980) \text{ lb/ft} = 2,310 \text{ lb/ft}$
- W_{b1} = Soil load directly above footing (see Table 4-5)
- W_{b2} = Soil load from soil wedge (see Table 4-5)

(See Tables 4-2, 4-4, and 4-5 in Section 4 for DL, LL, and W.)

ATLAS RESISTANCE PIER SELECTION

While the Atlas Resistance continuous lift pier could be used for this application, the small lift required makes it unnecessary. The Atlas Resistance predrilled pier is not a good choice here due to the absence of a hard, impenetrable layer above the intended bearing stratum. Therefore, the Atlas Resistance 2-piece standard pier is selected for strength and economy. The more expensive Atlas Resistance plate pier system could

also be attached to the concrete basement wall and used for this application. Since there are suitable soils with N counts above four, there is no need to sleeve the pier pipe for added stiffness.

PIER SPACING

Using the information obtained about the stem wall and footing to be supported and applying sound engineering judgment, the nominal pier spacing based on the foundation system's ability to span between piers is estimated at about eight feet. This results in a nominal working pier load (P_w) of:

EQUATION 8-2

$$\begin{aligned}
 P_w &= xP \\
 &= 8 \text{ ft} \times 4,987 \text{ lb/ft} \\
 &= 39,896 \text{ lb}
 \end{aligned}$$

where

- P_w = Pier working load
- x = Selected pier spacing = 8 ft
- P = Line load on footing = 4,987 lb/ft

FACTOR OF SAFETY

Hubbell recommends a minimum Factor of Safety (FS) for the mechanical strength of the hardware of 2.0.

EQUATION 8-3

$$\begin{aligned}
 R_{w \text{ ULT}} &= P_w(FS_h) \\
 &= (39,896 \text{ lb}) \times 2 \\
 &= 79,792 \text{ lb}
 \end{aligned}$$

where

- P_w = Pier working load
- FS_h = Hardware Factor of Safety = 2.0 (may be varied based on engineering judgment)
- $R_{w \text{ ULT}}$ = Minimum ultimate hardware strength requirement based on structural weight

Select a pier system with an adequate minimum ultimate strength rating:

EQUATION 8-4

$$\begin{aligned}
 x_{\text{MAX}} &= (R_{h \text{ ULT}}) / (FS_h)P \\
 &= 86,000 \text{ lb} / (2 \times 4,987) \\
 &= 8.6 \text{ ft} \text{ (Wall and footing are judged able to span this distance)}
 \end{aligned}$$

where

- FS_h = Hardware Factor of Safety
- $R_{h \text{ ULT}}$ = 86,000 lb - Choose AP-2-UFVL3500.165 [14'-0"] modified 2-piece pier system
- x_{MAX} = Maximum pier spacing based on hardware capacity

PROOF LOAD

Hubbell recommends a minimum Factor of Safety of 1.5 at installation unless structural lift occurs first.



EQUATION 8-5

$$\begin{aligned}
 R_p &= (FS_p)P_w \\
 &= (1.5)(8.6 \times 4987) \\
 &= 64,332 \text{ lb}
 \end{aligned}$$

where

FS_p = Proof load Factor of Safety¹ = 1.5

R_p = Installation force based on weight of structure to achieve proof load verification

$R_{h \text{ MAX}}$ = Maximum installation force based on hardware ultimate capacity²

$$= (R_{h \text{ ULT}}/2) (1.65) = (86,000/2) (1.65) = 70,950 \text{ lb}$$

$R_{w \text{ MIN}} < R_{h \text{ MAX}} = \text{OK}$, where $R_{w \text{ MIN}} = R_p$

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 ($L_{p \text{ MAX}}$) may be excessive.

² It is recommended that $R_{h \text{ MAX}}$ not exceed $(R_{h \text{ ULT}} / 2) \times (1.65)$ during installation without engineering approval.

DESIGN RECOMMENDATIONS

The result of the analysis provides the following design specifications:

- Underpinning product: Atlas Resistance® modified 2-piece pier AP-2-UF-3500.165 [14'-0"]
- Pier spacing: 8.6' on center
- Installation proof load: 64,332 lb ± (unless lift of the structure occurs first)
- Working load is anticipated to be 42,900 lb ± (4,987 lb/ft x 8.6 ft)
- Anticipated pier depths: 15 ft ±

DESIGN EXAMPLE 2: ATLAS RESISTANCE PIERS WITH INTEGRATED TIEBACK

PROJECT INFORMATION

An existing three-story commercial building located within a hurricane prone region requires foundation retrofitting for potential scour activity and lateral load forces from hurricane force winds. The structure sits on a shallow foundation system consisting of a 4' high 10" thick stem wall and a 4' wide 12" thick spread footing with three #5 reinforcement bars (Grade 60). The structural Engineer of Record has requested a new foundation system capable of withstanding 2 kip per lineal foot design lateral forces and temporary scour depths to 1' below the existing spread footing. The estimated design compression loading is 5 kip per lineal ft for the existing structure. The structural engineer has determined that the existing foundation system can handle underpinning support spans of 8' or less.

GEOTECHNICAL INVESTIGATION

A geotechnical investigation was performed to determine the soil types and strengths at the project location. The soil borings advanced near the project location show medium dense silty sand with SPT "N" values ranging from 15 to 25 bpf to a depth of 20 ft below ground surface (bgs). This medium dense silty sand layer is underlain by dense sand and weathered limestone bedrock with SPT "N" values greater than 40 bpf. Groundwater was observed at 18' bgs during the investigation.

UNDERPINNING SYSTEM SELECTION

The availability of a dense stratum with "N" values greater than 40 bpf allows the use of the Atlas Resistance pier. The additional lateral loading can be designed for using a helical tieback anchor and the integrated Atlas Resistance pier bracket. Based on the design compression loading (P) of 5 kip per lineal ft and the allowable pier spacing (x) of 8' the required minimum design capacity of the Atlas Resistance pier ($P_{w \text{ MIN}}$) is $(x) \times (P) = 8.0 \times 5.0$ or 40 kip.

The AP-2-UF-3500.165 system could be used since it has a maximum working (design) capacity of 42.5 kip. However, due to the possibility of scour and subsequent lack of soil support the modified pier with a working capacity of 45.5 kip is recommended (AP-2-UF-3500.165M) with at least three modified pier sections to increase the rotational stiffness of the bracket.

HELICAL TIEBACK DESIGN AND INSTALLATION

With a maximum spacing of 8' and an estimated design lateral line load of 2 klf, the horizontal design load (DL_H) at the tieback anchor location is 16 kip. The tieback anchors are typically installed between 15° to 25° from horizontal. An installation angle of 20° was chosen after determining that there are no underground structures/conduits that may interfere with the tieback installation. The tieback must be designed with a minimum embedment depth of 5D (distance from the last helical plate to the ground surface) where D = diameter of the helical



plate. The tieback will be designed to bear in the silty sand with “N” values of 20 bpf observed at 5 to 10 feet bgs. Based on the SPT “N” values and soil descriptions, the following parameters are used in the design:

- Cohesion (c) = 0
- Friction angle (ϕ) = 34°
- Bearing capacity factor (N_q) = 21
- Unit weight of soil (γ) = 115 pcf

Using a Factor of Safety (FS) = 2 on the design load and an installation angle of 20° , the required ultimate tension capacity of the tieback (UC_r) is $(FS \times DL_h) / \cos(20^\circ) = (2 \times 16) / \cos(20^\circ) = 34$ kip. The ultimate bearing capacity (Q_t) of a helical tieback can be determined from:

EQUATION 8-6

$$Q_t = A_n (cN_c + qN_q)$$

Try a Type SS5 series (12”-14” Lead) with a length of 15 ft:

Check depth criteria based on:

- A starting depth of 4 ft below the ground surface
- Tieback length of 15 ft
- An installation angle of 20°

The length to the top of the lead helix is $15 \text{ ft} - 3(12/12) - 4/12 = 11.7$ ft. The depth of embedment would be $4 + 11.7\sin(20^\circ) = 4 \text{ ft} + 4 \text{ ft} = 8$ ft which is greater than $5D$ (6 ft), so the depth criteria is met.

Check the ultimate capacity of the helical tieback (T_u) using: $N_q = 21$

EQUATION 8-7

$$d_{\text{avg}} = 4 \text{ ft} + \frac{[15 \text{ ft} - 1 \frac{(3 (12 \text{ in}) + 4 \text{ in})\sin(20^\circ)}{(12 \text{ in/ft})}]}{2} = 8.6 \text{ ft}$$

$$\gamma = 115 \text{ pcf}$$

$$\begin{aligned} \Sigma A_h &= A_{12} + A_{14} \\ &= 0.77 \text{ ft}^2 + 1.05 \text{ ft}^2 \\ &= 1.82 \text{ ft}^2 \end{aligned}$$

EQUATION 8-8

$$\begin{aligned} Q_t &= 1.82 \text{ ft}^2 (8.6 \text{ ft})(115 \text{ pcf})(21) \\ &= 37.8 \text{ kip} \end{aligned}$$

Since the ultimate bearing capacity (37.8 kip) is greater than the required ultimate capacity of 34 kip, the Type SS5 (12”-14”) tieback is acceptable. The average minimum installation torque would be UC_r/K_t or $34,000/10 = 3400$ ft-lb. This minimum installation torque is less than the torque rating of the SS5 and SS125 bar; therefore, either shaft size would be acceptable. K_t = empirical torque factor (default value = 10 for the SS series).

The distance from the assumed “active” failure plane to the 14” helix must be at least 5 times its diameter or 6’-0”. Both the minimum length and estimated installation torque must be satisfied prior to the termination of tieback installation.

ATLAS RESISTANCE® PIER UNDERPINNING INSTALLATION

Given a design load of 40 kip and the potential for 1 ft of temporary exposed pier section due to scour, use the AP-2-UF-3500.165M:

- The AP-2-UF-3500.165M pier has a working (design) load capacity of 45.5 kip. The estimated line load (P) is 5 klf, therefore with a maximum pier c-to-c spacing (x) of 8 ft, the piers will experience a design load (P_w) of 40 kip. The spacing may need to be decreased based upon field conditions.
- Use a minimum 3 modified pier sections (10.5 ft) offset halfway from the inner sleeve sections
- The depth to a suitable stratum for Atlas Resistance pier placement is approximately 20 ft bgs
- Install each pier to a minimum installing force, (Proof Load) $R_p = 1.50 \times P_w$ (estimated Factor of Safety (FS_p) of 1.5 on the design load) which makes the minimum installing force $R_{w \text{ MIN}} = 60,000$ lb (based on an 8 ft spacing) or imminent lift, whichever occurs first. The maximum installation force ($R_{h \text{ MAX}}$) shall not exceed $R_{h \text{ ULT}}/2 \times FS_h$ or $(91,000/2) \times 1.65 = 75,000$ lb (estimated Factor of Safety (FS_h) of 1.65 of the design load for hardware).

DESIGN EXAMPLE 3: HELICAL PILE FOUNDATION FOR NEW CONSTRUCTION

BUILDING TYPE

- Two story residence
- Slab on grade
- Masonry wall, wood frame
- Width = 30 ft, L/W = 1-1/2

STRUCTURAL LOADS

EQUATION 8-9

$$P = DL + LL + SL$$

$$P = 1540 + 346 + 162 = 2,048 \text{ lb/ft}$$

where

P = Total Load on perimeter footing

DL = Dead Load (1,540 lb/ft)

LL = Live Load (346 lb/ft)

SL = Snow Load (162 lb/ft)

(See Tables 4-1 and 4-4 in Section 4 for DL and LL)

Factor of Safety (FS) = 2.0 (minimum)

PILE SPACING

EQUATION 8-10

$$\text{Estimated working load} = P_w = xP$$

$$\text{Estimated pile spacing (x)} = 6.0 \text{ ft}$$

$$P_w = 6.0 \times 2,048 = 12,288 \text{ lb}$$

CHANCE® HELICAL PILE SELECTION

RS2875.203 with 8-10-12 helix configuration

ULTIMATE PILE CAPACITY

EQUATION 8-11

$$Q_t = (A_8 + A_{10} + A_{12}) c N_c$$

where

A_8, A_{10}, A_{12} = Projected area of helical plates

$$A_8 = 0.34 \text{ ft}^2$$

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

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

$$c = 2,000 \text{ psf (based on } N = 16 \text{ - Equation 5-41)}$$

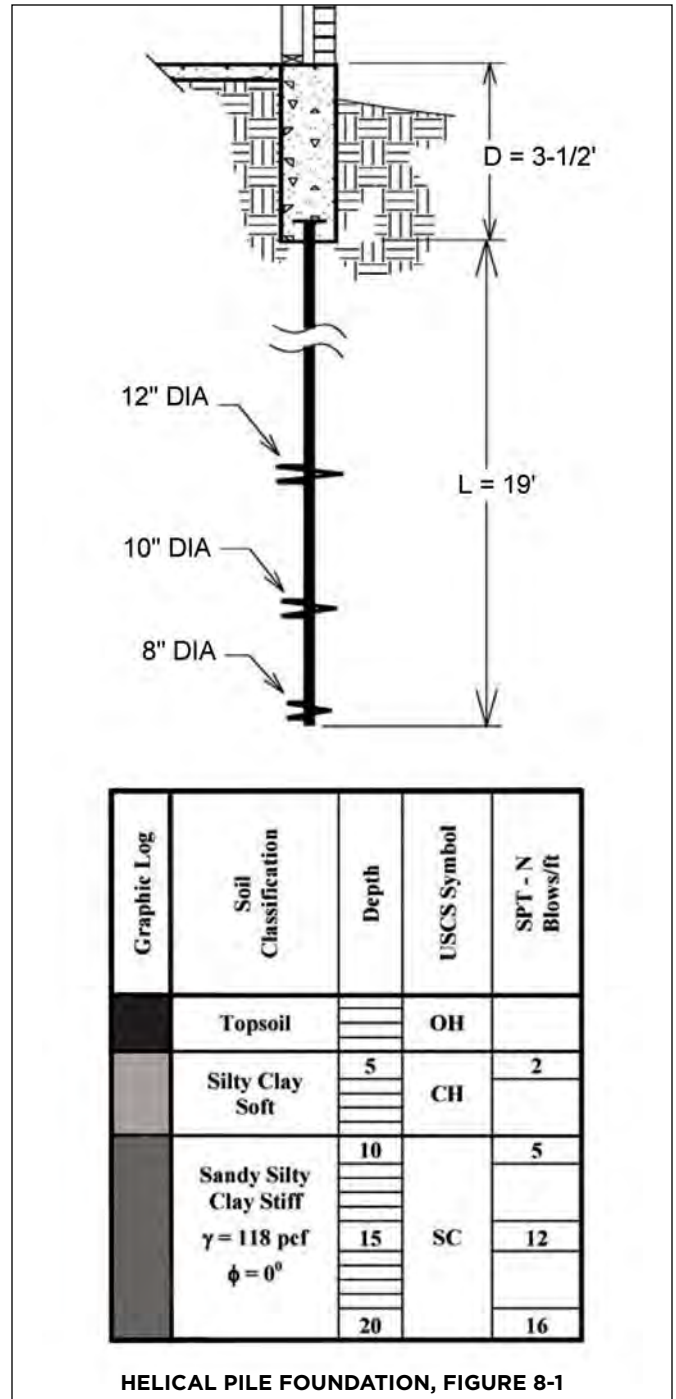
$$N_c = \text{Bearing capacity} = 9.0$$

$$Q_t = (1.64) (2,000) (9.0)$$

$$Q_t = 29,520 \text{ lb (installation depth is over 20 ft)}$$

CHECK Q_t

- Conduct Field Load Test (if required per specifications)





ESTIMATE INSTALLATION TORQUE

EQUATION 8-12

$$T = (P_w \times FS) / K_t$$

$$= (12,288 \times 2) / 9$$

$$= 2,750 \text{ ft}\cdot\text{lb}$$

where

K_t = Empirical torque factor (default value = 9 for the R2875 series)

The rated installation torque of the RS2875.203 series is 6,710 ft·lb, which is greater than the required estimated installation torque of 2,750 ft·lb. (OK)

NOTE: If during installation $T = 2,750$ ft·lb is not achieved, then two options are available: (1) reduce pile spacing (x), or (2) change helix configuration to a larger combination, i.e., (10"-12"-14")

FACTOR OF SAFETY

EQUATION 8-13

Theoretical Ultimate Capacity:

$$FS = (Q_t / P_w)$$

$$= 29,520 / 12,288$$

$$= 2.4 \text{ (OK)}$$

Torque Correlation:

$$FS = (T \times K_t) / P_w$$

$$= (2,750 \times 9) / 12,288$$

$$= 2.01 \text{ (OK)}$$

DESIGN EXAMPLE 4: LIGHT COMMERCIAL STRUCTURE

PROBLEM

Build a new (lightly loaded single story) commercial building on a typical clay soil profile as given on a single boring. The profile consists of the upper 10'-0" of highly plastic clay (CH), Plasticity Index (PI) = 35; cohesion (c) = 2000 psf; unit weight (γ) of 105 pcf. The swell potential of this layer is estimated to be 2". The top 10'-0" layer is underlain by 20' of stiff to very stiff low plasticity clay (CL) that has a Standard Penetration Test (SPT) blow count "N" = 20. The boring was terminated at 30 feet without encountering the water table. No further soil parameters or lab data given.

POSSIBLE SOLUTION

Support the structure on a grade beam and structural slab, which is in turn supported by helical piles. Isolate the foundation and slab from the expansive subgrade by forming a 2" void, using a cardboard void form. Assume the water table is at the soil boring termination depth. This is typically a conservative design assumption when the water table is not encountered. The stiff to very stiff clay soil in the 20-foot thick layer is probably at or near 100% saturation (volume of water is the same as the volume of the voids).

STEP 1: FEASIBILITY

- Site Access - The site is road accessible, with no overhead or underground obstructions, but the owner is concerned about potential damage to neighboring sites due to vibration and noise.
- Working Loads - The structure is single story, so the working loads are probably considerably less than 100 kip per pile.
- Soils - Boulders, large cobbles, or other major obstructions are not present in the bearing stratum. The clay soil does not appear to be too hard to penetrate with helical piles. See Table 3-1 (Helical Shaft Series Selection) or Figure 3-1 (Product Selection Guide) in Section 3 to determine

if helical piles are feasible, and if so, which product series to use.

- Qualified Installers - Local Certified Chance Installers are available and can get competitive bids from a second certified installer 20 miles away.
- Codes - Local building codes allow both shallow and deep foundations.

Cost-bid must be competitive with other systems. Owner may pay a small premium to "protect" the investment in the structure.

STEP 2: SOIL MECHANICS

See Problem section above.

STEP 3: LOADS

- Exterior Grade Beam - The dead and live loads result in a total load (P) of 3 kip per lineal foot on the perimeter grade beam (12" wide x 18" deep). The grade beam is designed to span between piles on 8'-0" centers. Therefore, the design or working load per pile (P_w) is 3 kip/ft x 8 ft = 24 kip. A Factor of Safety (FS) of 2.0 is recommended. Therefore, the required ultimate capacity (UC_r) per exterior pile is 24 x 2 = 48 kip compression.
- Interior Columns - The dead load results in 9 kip per column. The live load results in 20 kip per column. The total dead and live load per column is 9 + 20 = 29 kip/column design or working load. A Factor of Safety of 2 is recommended. Therefore, the required ultimate capacity per interior pile is 29 x 2 = 58 kip compression. The required ultimate loads for both the exterior grade beam and interior columns are well within the load ratings of the Hubbell Power Systems, Inc., Chance® product series.
- Lateral Loads - The piles are not required to resist any lateral loads.



STEP 4: BEARING CAPACITY

Find the ultimate bearing capacity in the stiff to very stiff clay using hand calculations.

EQUATION 8-14

$$\text{Bearing Capacity: } Q_{ult} = A_h (cN_c + q'N_q + 0.5\gamma'BN_\gamma)$$

For saturated clay soils, the second term of Equation 8-14 becomes zero since the angle of internal friction (ϕ) is assumed to be zero for saturated clays, thus $N_q = 0$. The third term (base term) may be dropped because B is relatively small. The simplified equation becomes:

EQUATION 8-15

$$Q_{ult} = A_h c N_c = A_h c 9$$

EQUATION 8-16

$$c \text{ (ksf)} = N/8$$

From Equation 5-41, $c \text{ (ksf)} = 20/8 = 2.5 \text{ ksf}$. At this point, an iterative process is required. Select a helix configuration that can develop the required ultimate capacity. Try a 10"-12" twin helix with a minimum of 5'-0" embedded into the bearing stratum which is the stiff low plasticity clay starting 10 ft below grade. From Table 8-1, the helix area of a 10" helix is 76.4 in² or 0.531 ft²; the helix area of a 12" helix is 111 in² or 0.771 ft².

Substituting:

EQUATION 8-17

$$Q_{10} = 0.531 \text{ ft}^2 \times 2.5 \text{ ksf} \times 9 = 11.95 \text{ kip}$$

$$Q_{12} = 0.771 \text{ ft}^2 \times 2.5 \text{ ksf} \times 9 = 17.35 \text{ kip}$$

$$Q_t = \Sigma Q_h = 11.95 + 17.35 = 29.3 \text{ kip}$$

STANDARD HELIX SIZES, TABLE 8-1

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)

Another trial is required because the total ultimate capacity ($Q_t = 29.3 \text{ kip}$) is less than required. Try a three-helix configuration (10"-12"-14") with a minimum of 5'-0" embedded in the bearing stratum. From Table 8-1, the helix area of a 14" helix is 151 in² or 1.05 ft².

EQUATION 8-18

$$Q_{14} = 1.05 \text{ ft}^2 \times 2.5 \text{ ksf} \times 9 = 23.63 \text{ kip}$$

$$Q_t = \Sigma Q_h = 11.95 + 17.35 + 23.63 = 52.93 \text{ kip}$$

To achieve the necessary Factor of Safety of 2, two helical piles with a 10"-12" helical configuration can be used under the interior columns ($29.3 \times 2 = 58.6 \approx 59 \text{ kip}$ ultimate capacity) and a single helical pile with a 10"-12"-14" helical configuration can be used under the perimeter grade beam. The termination of the helical pile in a concrete cap or grade beam should be made with an appropriately designed pile cap or an available "new

construction" bracket from Hubbell Power Systems, Inc. This will allow the foundation to rise up, should the swell ever exceed the 2" void allowance, but to shrink back and rest on the pile tops.

CHECKING BEARING CAPACITY USING HELICAP® ENGINEERING SOFTWARE

A sample tabular data printout is shown in Figure 8-2, where the twin helix (10"-12") $Q_{ult} = 29.2 \text{ kip} \approx 29.3 \text{ kip}$, OK; and the triple helix (10"-12"-14") $Q_{ult} = 52.8 \text{ kip} \approx 52.93 \text{ kip}$, OK

STEPS 5 AND 6: LATERAL CAPACITY AND BUCKLING

- Lateral Capacity - None is required in the statement of the problem. In reality, horizontal loads due to wind will be resisted by net earth pressure (passive-active) on the grade beam and/or caps. See Section 5 for an explanation of earth pressure resistance.
- Buckling Concerns - The soil density and shear strength is sufficient to provide lateral confinement to the central steel shaft. This is supported by the fact that the SPT blow count is greater than four for the top clay layer. Should analysis be required, the Davisson method described in Section 5 may be used to determine the critical load.

STEP 7: CORROSION

No electrochemical properties were given for the clay soil. Generally, undisturbed, i.e., non-fill, material tends to be benign as little oxygen is present and the ions that are present in solution are not washed away due to flowing water or fluctuating water level. In the absence of soil data, a useful guide is to observe the use of corrugated metal pipe (CMP) by the local Department of Transportation (DOT). If the DOT uses CMP, the likelihood is that the local soils are not very aggressive.

STEP 8: PRODUCT SELECTION

Ultimate capacity for a 10"-12" configuration per Step 4 above was 29 kip, and the ultimate capacity for a 10"-12"-14" configuration was 53 kip. Table 8-2 shows that both Chance® Type SS5 and Type RS2875.276 product series can be used, since 53 kip is within their allowable load range. Note that Table 8-2 assumes a K_t of 10 ft⁻¹ for the Type SS product series and K_t of 9 ft⁻¹ for the Type RS2875 product series. In this case, use the Type SS5 product series because shaft buckling is not a practical concern and the required capacity can be achieved with less installation torque.



HeliCAP SUMMARY REPORT

Job Name: Design Manual for New Construction

Job Number: Example 4

Boring No: B-1

Anchor Use: Compression

Water Table Depth: None

Capacity Summary

Anchor Number	Anchor Family	Helix Depth (ft)	Helix Capacity (kips)	Total Anchor Capacity (kips)	Recommended Ultimate Capacity (kips)	Torque (ft-lbs)
Anchor 1	Angle: 90 Datum Depth: 0 Length: 18					
12" helix	SS 5	15	17.3t 17.3c	29.2t	29.2t	2925
10" helix	SS 5	17.5	11.9t 11.9c	29.2c	29.2c	
Anchor 2	Angle: 90 Datum Depth: 0 Length: 21					
14" helix	SS 5	15	23.6t 23.6c	52.8t 52.8c	52.8t 52.8c	5287
12" helix	SS 5	18	17.3t 17.3c			
10" helix	SS 5	20.5	11.9t 11.9c			

Soil Profile

Top of Layer Depth (ft)	Soil Type	Cohesion (lb/ft ²)	N	Angle of Internal Friction (Degrees)	Unit Weight (lb/ft ³)
0	Clay	2000	0	0	105
10	Clay	2500	20	0	120

HELICAP® SOFTWARE SUMMARY REPORT
FIGURE 8-2

DESIGN EXAMPLES



PRACTICAL GUIDELINES FOR FOUNDATION SELECTION, TABLE 8-2

INSTALLATION TORQUE	ULTIMATE LOAD ¹		DESIGN LOAD ²		HELICAL PILE PRODUCT SERIES
	kip	kN	kip	kN	
5,500	55	244	27.5	110	SS5
5,500	49.5	202	24.75	110	RS2875.203
7,000	70	312	35	156	SS150
8,000	72	320	36	160	RS2875.276

¹ Based on a torque factor $K_t = 10$ for SS series and $K_t = 9$ for RS2875 series.
² Based on a Factor of Safety of 2.

For the 10"-12" configuration, the minimum depth of 18'-0" can be achieved by using a lead section, which is the first pile segment installed and includes the helix plates, followed by two or three plain extensions. For the 10"-12"-14" configuration, the minimum depth of 21'-0" can be achieved by using a lead section followed by three or four plain extensions. The exact catalog items to use for a specific project are usually the domain of the contractor. Your Certified Chance Installer is familiar with the standard catalog items and is best able to determine which ones to use based on availability and project constraints. For your reference, helical piles with product descriptions are provided in Section 7 of this manual.

The head of the helical pile is to be approximately 1'-0" below grade in the grade beam or cap excavation, which will put the twin-helix pile tip 18'-0" below the original ground level and the three-helix pile tip 21'-0". These are minimum depths, required to locate the helix plates at least 5'-0" into the bearing stratum. On large projects, it is advisable to add 3% to 5% extra extensions in case the soil borings vary considerably or if widely spaced borings fail to indicate differences in bearing depths.

STEP 9: FIELD PRODUCTION CONTROL

Use $K_t = 10 \text{ ft}^{-1}$ for Chance® Type SS material if verification testing is not done prior to production work. The minimum depth and minimum installing torque must both be achieved. If the minimum torque requirement is not achieved, the contractor

should have the right to load test the helical pile to determine if K_t is greater than 10 ft^{-1} . Verification testing is often done in tension since it's simpler and less costly to do than compression testing, and the compressive capacity is generally higher than tension capacity, which results in a conservative site-specific K_t value.

Estimate installing torque for field production control and specifying the minimum allowable without testing.

EQUATION 8-19

$$Q_{ult} = K_t T, \text{ or } T = Q_{ult} / K_t$$

where

$Q_{ult} = UC_r$ in this example

Interior columns: $T = Q_{ult} / K_t = (58,000 \text{ lb} / 2 \text{ piles}) / 10 \text{ ft}^{-1} = 2,900 \text{ ft}\cdot\text{lb} \approx 3,000 \text{ ft}\cdot\text{lb}$ for the minimum average torque taken over the last three readings.

Perimeter grade beam: $T = Q_{ult} / K_t = 48,000 \text{ lb} / 10 \text{ ft}^{-1} = 4,800 \text{ ft}\cdot\text{lb}$ for the minimum average torque taken over the last three readings.

Note that the torque rating for the Chance Type SS5 product series is 5,700 ft·lb – OK.

STEP 10: PRODUCT SPECIFICATIONS

See Section 7, Product Drawings and Ratings and Appendix C for Hubbell model specifications.

STEP 11: LOAD TEST

Since this is a small project with low loads in "normal" soils, it is acceptable to use the torque correlation method as the driving criteria and omit the "optional" load test.

DESIGN EXAMPLES



DESIGN EXAMPLE 5: HELICAL PULLDOWN MICROPILES FOR NEW CONSTRUCTION

PROBLEM

Determine the capacity of the following Chance® Helical Pulldown® micropile (HPM) installed into the soil described in Figure 8-4.

- SS5 1-1/2" x 1-1/2" square shaft
- Helix configuration: 8"-10"-12"
- Total depth: 40 ft
- Grout column: 5" dia x 31 ft

CALCULATIONS

End bearing calculations from the HeliCAP® engineering software. See Table 8-3 below for the ultimate end bearing capacity of the proposed 8"-10"-12" lead configuration.

- Summary: Compression Capacity (ΣQ_h) = 44.7 kip
- Summary: Friction Capacity (Q_f) = 22.1 kip (see Table 8-4)
- Total Capacity (Q_t) = $\Sigma Q_h + Q_f = 44.7 + 22.1 = 66.8$ kip

REVIEW OF COMPRESSION TEST

Figure 8-3 is a load deflection plot from the actual compression test on the HPM installed into the soil described in Figure 8-4. From the plotted data, the ultimate capacity (based on $0.08D_h + PL/AE$) was 80 kip, compared to the calculated total capacity of 66.8 kip. This calculated value provides a conservative approach to determining the ultimate capacity of an HPM.

DESIGN EXAMPLES

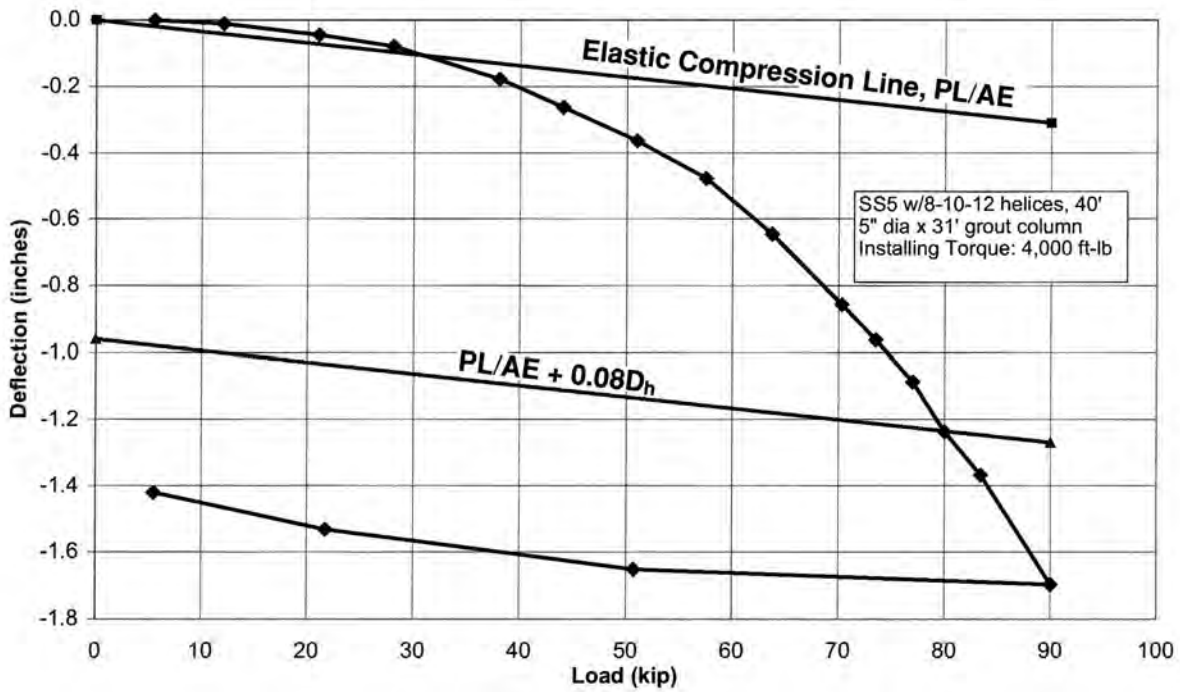
HELICAP SOFTWARE SUMMARY REPORT, TABLE 8-3

Job Name: Medina, MN Demonstration						
Job Number: Stannard Soil Anchor Systems			Water Table Depth: 15 ft			
Boring No: B-1						
Anchor Use: Compression						
Capacity Summary						
Anchor Number	Anchor Family	Helix Depth (ft)	Helix Capacity (kips)	Total Anchor Capacity (kips)	Recommended Ultimate Capacity (kips)	Torque (ft-lbs)
Anchor 1	Angle: 90 Datum Depth: 0 Length: 40					
12" helix	SS 5	35	17.9t			
			19.9c			
10" helix	SS 5	37.5	14.3t			
			14.8c	41.9t	41.9t	4263
8" helix	SS 5	39.5	9.6t	44.7c	44.7c	
			9.8c			

FRICTION CALCULATION (SEE SOIL BORING LOG IN FIGURE 8-4), TABLE 8-4

DEPTH (ft)	SOIL	"N"	ESTIMATED		EFFECTIVE UNIT WEIGHT (lb/ft ³)	AVERAGE OVERBURDEN (lb/ft ²)	ADHESION/FRICTION (lb/ft ²)	SIDE FRICTION (lb)
			COHESION (lb/ft ²)	ϕ				
0 - 9	Clay	6	750	-	92	-	682	8040
9 - 15	Clay	2	250	-	84	-	250	1965
15 - 18	Clay	1	125	-	20	-	125	491
18 - 22	Sand	5	-	29	23	1438	798	3192
22 - 28	Clay	7	875	-	32	-	682	5364
28 - 31	Sand	8	-	30	38	1733	1001	3003
TOTAL								22055

Notes: (1) $\phi = 0.28N + 27.4$ (2) $c = (N \times 1000) / 8$ (3) Area/ft of pile = $\pi \times d = \pi (5/12) = 1.31\text{ft}^2/\text{ft}$



**HELICAL PULLDOWN[®] MICROPILE COMPRESSION TEST
FIGURE 8-3**



MINNESOTA DEPARTMENT OF TRANSPORTATION - GEOTECHNICAL SECTION
 LABORATORY LOG & TEST RESULTS - SUBSURFACE EXPLORATION



UNIQUE NUMBER
 U.S. Customary Units

State Project		Bridge No. or Job Desc.		Trunk Highway/Location		Boring No.	Ground Elevation		
				Anchor Demonstration - Medina, MN		1			
Location					Drill Machine		SHEET 1 of 2		
Co. Coordinate: X= Y= (ft.)					91		7/16/01		
Latitude (North)= Longitude (West)=					CME Automatic Calibrated		Drilling Completed		
DEPTH	Depth	Lithology	Classification	Drilling Operation	SPT	MC	COH	γ	Other Tests Or Remarks
	Elev.				N ₆₀	(%)	(psf)	(pcf)	
					REC (%)	POD (%)	AGL (ft)	Core Breaks	Formation or Member
	5		FILL, mixture of sandy lean clay, clayey sand, a little gravel, brown and gray						
	9.5		SAPRIC PEAT, black, soft (PT)						
	12.0		ORGANIC CLAY, grayish brown, very soft (OH)						
	15			WH					
	18.0		SILTY SAND, fine grained, gray, waterbearing, loose, lenses of lean clay (SM)						
	22.0		LEAN CLAY, gray, firm (CL)						
	28.0		CLAYEY SAND, a little gravel, gray, firm to stiff to very stiff, lenses of sand (SC/CL)						

DESIGN EXAMPLES

SOIL BORING LOG
FIGURE 8-4
(SHEET 1 OF 2)



MINNESOTA DEPARTMENT OF TRANSPORTATION - GEOTECHNICAL SECTION
 LABORATORY LOG & TEST RESULTS - SUBSURFACE EXPLORATION



UNIQUE NUMBER
 U.S. Customary Units

Mn/DOT GEOTECHNICAL SECTION - LOG & TEST RESULTS							SHEET 2 of 2			
State Project		Bridge No. or Job Desc.		Trunk Highway/Location		Boring No.		Ground Elevation:		
				Anchor Demonstration - Medina, MN		1				
DEPTH	Depth	Lithology	Classification	Drilling Operation	SPT	MC	COH	γ	Soil	Other Tests
	Elev.				N ₆₀	(%)	(psf)	(pcf)		Or Remarks
					REG (%)	RQD (%)	ACL (%)	Core Breaks	Rock	Formation or Member
					11					
					11					
	40		CLAYEY SAND, a little gravel, gray, firm to stiff to very stiff, lenses of sand (SC/CL)							
	45									
	50									
	55									
	60									
	64.0		Bottom of Hole - 64'							

Soil Class. LR Rock Class. LR Edit Date 7/19/07
 R:\DATA\INT\W\2001\01-7068.GPJ

SOIL BORING LOG
FIGURE 8-4
(SHEET 2 OF 2)

DESIGN EXAMPLES

DESIGN EXAMPLE 6: HELICAL PILES FOR BOARDWALKS

SOILS

A helical pile foundation is proposed to support a pedestrian walkway. The soil profile consists of 7'-0 (2.1 m) of very soft clay with a reported Standard Penetration Test (SPT) blow count "N" equal to weight of hammer (WOH) and a unit weight of 65 lb/ft³ (10.2 kN/m³). Below the very soft clay is a thick layer of medium-dense sand with a SPT blow count value of 17. The correlated friction angle is 32° and the unit weight is 107 lb/ft³ (16.8 kN/m³). The water table is located at the surface. The proposed helical pile is connected to the walkway with a Chance® Walkway Support Bracket. The helical piles must be checked for lateral stability in the very soft clay.

WALKWAY

- The helical piles are spaced 5 ft (1.5 m) apart and are exposed 2 ft (0.61 m) above grade as shown in Figure 8-5.
- The walkway is 7 ft (2.1 m) wide; each pile group or "bent" is spaced 10'-0 apart.

STRUCTURAL LOADS

- The dead and live vertical load is 100 lb/ft² (4.8 kN/m²). Lateral loads are negligible.
- The required compression load per helical pile (P_w) is 100 lb/ft² x 7'-0 x 10'-0 = 7000 lb/2 helical piles = 3500 lb (15.6 kN) per pile.
- Using a Factor of Safety (FS) of 2, the required ultimate capacity (UC_r) per helical pile is 3500 lb x 2 = 7000 lb (31.1 kN).

CHANCE HELICAL PILE SELECTION

- Try a twin-helix configuration with 10" (254 mm) and 12" (305 mm) diameters.
- Try either Type SS5 1-1/2" (38 mm) Square Shaft or Type RS2875.203 2-7/8" (73 mm) Round Shaft material.

ULTIMATE PILE CAPACITY

The top-most helix should be at least three diameters into a suitable bearing soil; which in this example is the medium-dense sand starting 7 ft (2.1 m) below grade. The spacing between helix plates is also three diameters; which is 3 x 10" = 2.5 ft (0.8 m) for a 10"-12" (254 mm - 305 mm) configuration. Finally, the distance from the bottom-most helix to the pile tip is 0.5 ft (0.15 m). Therefore, the minimum overall length for a 10"-12" helix configuration in this soil profile is 7 ft + (3 x 12 inch) + 2.5 ft + 0.5 ft = 13 ft (4 m). The effective unit weight is the submerged unit weight in this case, because the water table is at the ground surface. The general bearing capacity equation (simplified for cohesionless soils) is:

EQUATION 8-20

$$Q_h = AD\gamma'N_q$$

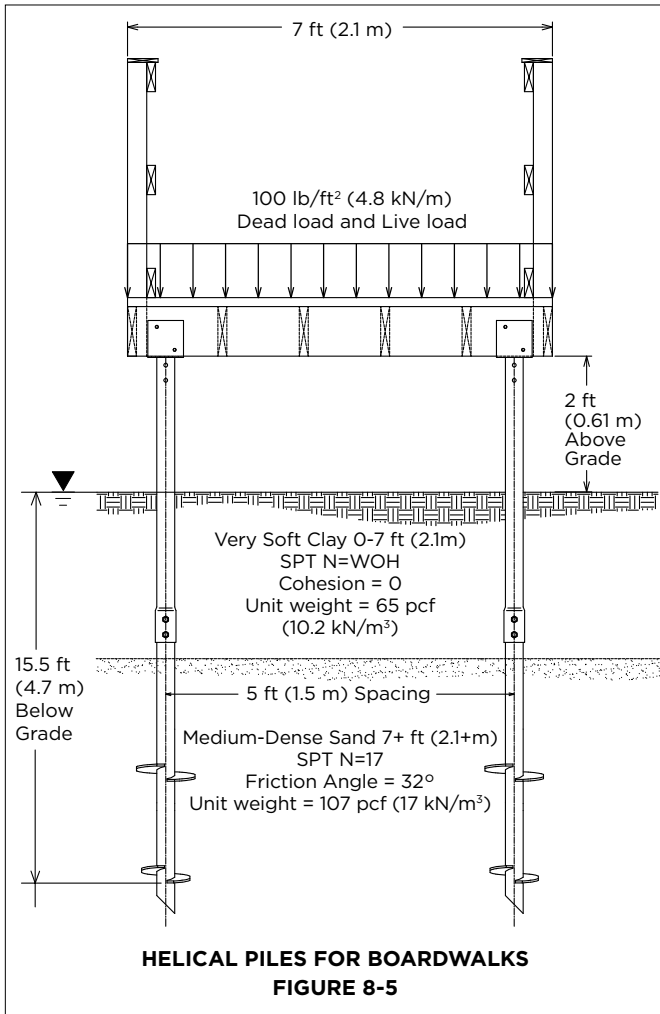
where

- Q_h = Ultimate capacity of helix plate
- A = Projected area of helix plate
- D = Vertical depth to helix plate
- γ' = Effective unit weight of soil = 2.6 lb/ft³ (0.4 kN/m³) for the very soft clay and 44.6 lb/ft³ (7.1 kN/m³) for the medium-dense sand
- N_q = Bearing capacity factor for cohesionless soils = 17 for 32° sand

For a 10"-12" configuration, the bearing capacity equation is:

EQUATION 8-21

$$\begin{aligned} \Sigma Q_h &= A_{10}D_{10}\gamma'N_q + A_{12}D_{12}\gamma'N_q \\ \Sigma Q_h &= 0.531 \text{ ft}^2 [(7 \text{ ft} \times 2.6 \text{ lb/ft}^3) + (5.5 \text{ ft} \times 44.6 \text{ lb/ft}^3)]17 + 0.771 \text{ ft}^2 [(7 \text{ ft} \times 2.6 \text{ lb/ft}^3) + (3 \text{ ft} \times 44.6 \text{ lb/ft}^3)]17 \\ \Sigma Q_h &= 4371 \text{ lb (19.4 kN)} \end{aligned}$$





4371 lb is less than the required ultimate capacity (7000 lb) needed for the vertical piles. Greater capacity can be obtained by extending the helix plates deeper into the medium-dense sand. Try extending the pile length 3 ft (0.9 m) deeper so that the tip is 16 ft (4.9 m).

EQUATION 8-22

$$\Sigma Q_h = 0.531 \text{ ft}^2[(7 \text{ ft} \times 2.6 \text{ lb/ft}^3) + (8.5 \text{ ft} \times 44.6 \text{ lb/ft}^3)]17 + 0.771 \text{ ft}^2[(7 \text{ ft} \times 2.6 \text{ lb/ft}^3) + (6 \text{ ft} \times 44.6 \text{ lb/ft}^3)]17$$

$$\Sigma Q_h = 7332 \text{ lb (32.6 kN)}$$

7332 lb is greater than the required ultimate capacity needed for the vertical piles, so 16 ft (4.9 m) pile length will work.

BUCKLING

Check for buckling on Type SS5 1-1/2" (38 mm) square shaft and Type RS2875.203 2-7/8" (73 mm) OD pipe shaft material with 2 ft (0.61 m) of exposed shaft above grade. Assume a free-fixed ($K = 2$) end-condition. Assume the very soft clay provides no lateral support, i.e., the pile shaft is unsupported above the sand, so the unsupported (effective) length (L_u) of the "column" is 2 ft + 7 ft = 9 ft (2.7 m).

FOR TYPE SS5 SQUARE SHAFT MATERIAL:

EQUATION 8-23

$$\text{Euler's Equation: } P_{crit} = \pi^2 EI / [KL_u]^2$$

$$P_{crit} = \pi^2 [30 \times 10^6 \text{ lb/in}^2] [.396 \text{ in}^4] / [2 \times 108 \text{ in}]^2$$

$$P_{crit} = 2513 \text{ lb (11.2 kN)}$$

The critical load for the Type SS5 series is less than the required 7000 lb (31.1 kN) ultimate capacity, so a shaft with greater stiffness is required.

FOR TYPE RS2875.203 PIPE SHAFT MATERIAL:

EQUATION 8-24

$$P_{crit} = p^2 [30 \times 10^6 \text{ lb/in}^2] [1.53 \text{ in}^4] / [2 \times 108 \text{ in}]^2$$

$$P_{crit} = 9710 \text{ lb (42.2 kN)}$$

The critical load for Type RS2875.203 pipe shaft is greater than the required 7000 lb (31.1 kN) ultimate capacity. Use the RS2875.203 series (2-7/8 inch (73 mm) OD pipe shaft material).

TORQUE

EQUATION 8-25

$$\text{Torque required} = \text{Required ultimate capacity} / K_t$$

where

$$K_t = 9 \text{ (26) for RS2875 round shaft}$$

$$\text{Torque required} = 7000 \text{ lb} / 9$$

$$\text{Torque required} = 778 \text{ ft}\cdot\text{lb (1186 N}\cdot\text{m)}$$

The torque strength rating for building code evaluated RS2875.203 material is 6,710 ft·lb (9,100 N·m) - OK.

DESIGN EXAMPLE 7: HELICAL PILES FOR BOARDWALKS WITH LATERAL SUPPORT

A Chance® Type SS5 helical pile is proposed as the foundation for a pedestrian walkway. The pile is connected to the walkway with a Chance Walkway Support Bracket with lateral support. The soil is a soft to medium clay with a Standard Penetration Test (SPT) "N" value of 6, cohesion of 750 psf (36.0 kN/m²) and unit weight of 92 lb/ft³ (14 kN/m³). The ground water table (GWT) is 15 ft (4.5 m) below grade.

WALKWAY:

- The piles are spaced 5 ft (1.5 m) apart and are exposed 2 ft (0.61 m) above grade.
- The walkway is 7 ft (2.1 m) wide and pier sets are 5 ft (1.5 m) apart.
- The battered pile is at an angle of 22°.

STRUCTURAL LOADS:

- Using a Factor of Safety (FS) of 2, the required ultimate capacity (UC_v) per vertical pile is 4550 lb (20 kN).
- Using a Factor of Safety of 2, the required ultimate capacity (UC_b) per battered pile is 2646 lb (12 kN).

CHANCE HELICAL PILE SELECTION:

- Try a Type SS5 square shaft with a 12" (305 mm) diameter helix.

ULTIMATE PILE CAPACITY:

The pile depth needs to be at least 5 diameters into the soft to medium clay layer. Therefore the vertical pile length should be at least 5 ft (1.5 m) below grade.

EQUATION 8-26

$$Q_t = AcN_c$$

$$Q_t = [.771 \text{ ft}^2][750 \text{ psf}][9]$$

$$= 5,204 \text{ lb (23 kN)}$$

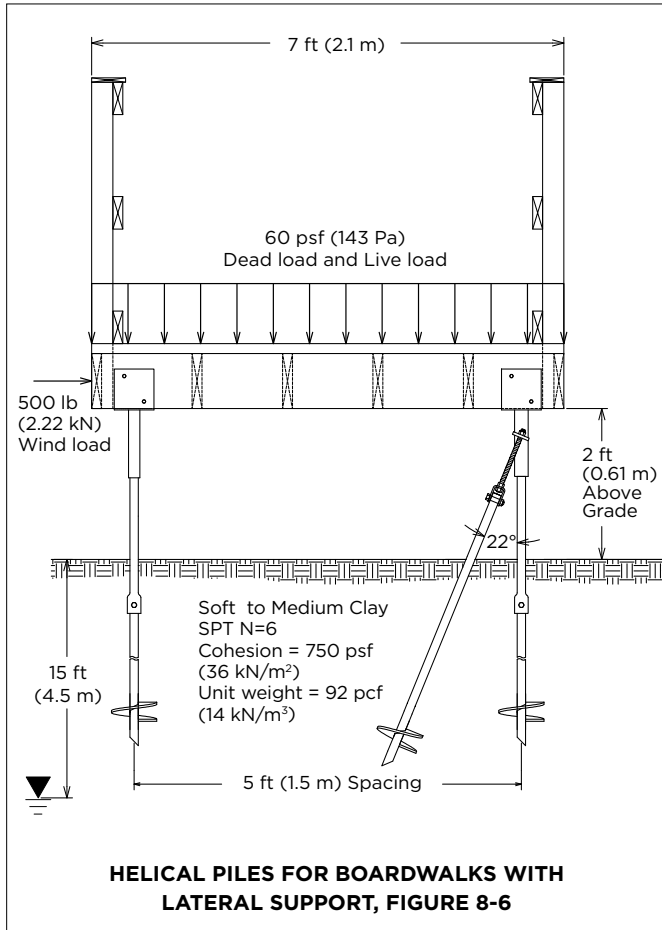
where

$$A = \text{Projected area of helical plates}$$

$$c = \text{Cohesion of soil}$$

$$N_c = \text{Bearing capacity}$$

5,204 lb is greater than UC_v for the vertical pile. The battered pile depth needs to be at least 5 diameters below grade. Therefore the battered pile length should be 6 ft (1.8 m) below grade.


BUCKLING:

Check for buckling on the SS5 square shaft with 2 ft (0.61 m) of exposed shaft above grade. Assume a pin-pin ($K = 1$) connection.

EULER'S EQUATION:
EQUATION 8-27

$$P_{crit} = \pi^2 EI / [KL_u]^2$$

$$P_{crit} = \pi^2 [30 \times 10^6] [.396] / [1 \times 24]^2$$

$$P_{crit} = 203,354 \text{ lb (904 kN)}$$

The critical load is greater than the ultimate vertical load so buckling is not a concern.

TORQUE:
EQUATION 8-28

$$\text{Torque required} = \text{Required load} / K_t$$

where

$$K_t = 10 \text{ (33) for square shaft}$$

$$\text{Torque required} = 5,204 \text{ lb} / 10$$

$$\text{Torque required} = 520 \text{ ft} \cdot \text{lb (705 N} \cdot \text{m)}$$

This does not exceed the SS5 torque rating of 5,700 ft·lb (7,730 N·m).

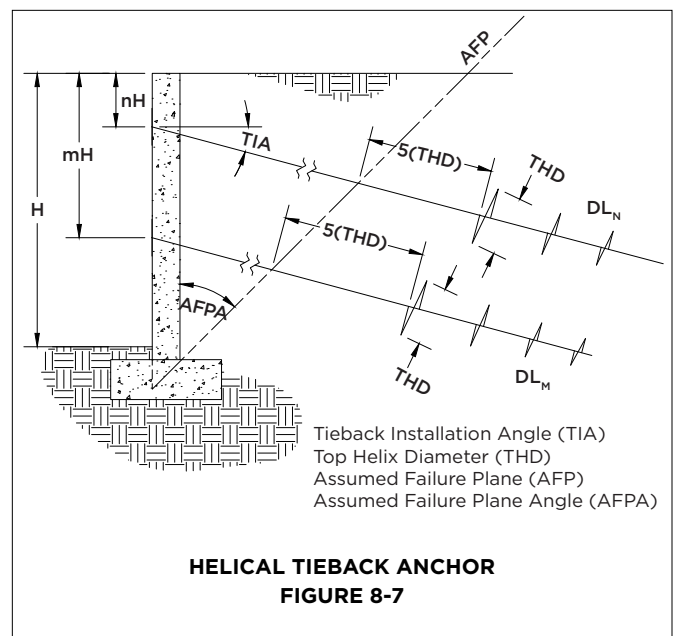
DESIGN EXAMPLE 8: HELICAL TIEBACK ANCHORS IN CLAY

STRUCTURE TYPE

- Cast concrete retaining wall
- Height (H) = 18 ft, thickness = 2'-0"
- $nH = 0.25H = 4.5$ ft, $mH = 0.63H = 11.3$ ft
- Residual soils: stiff clay with $N = 28$. No ground water table (GWT) present.
- Tieback installation angle = 15°

Structural Design Loads (See Figure 4-6 in Section 4)

- $DL_N / \text{ft} = (12 \times H^2) / \cos(15^\circ)$
- $DL_N / \text{ft} = (12 \times 18^2) / \cos(15^\circ)$
- $DL_N / \text{ft} = 4,025 \text{ lb/lin ft}$
- $DL_M / \text{ft} = (18 \times H^2) / \cos(15^\circ)$
- $DL_M / \text{ft} = (18 \times 18^2) / \cos(15^\circ)$
- $DL_M / \text{ft} = 6,040 \text{ lb/lin ft}$





CHANCE® HELICAL ANCHOR SELECTION

- Wall height ≥ 15 ft; use two rows of tiebacks
- Try Type SS150 series, C1500169 (8"-10"-12" lead) for DL_N .
- Try Type SS175 series, C1100247 (8"-10"-12"-14" lead) for DL_M .

ULTIMATE TENSION CAPACITY (USING BEARING CAPACITY APPROACH)

EQUATION 8-29

$$Q_{tN} = (A_8 + A_{10} + A_{12}) \times (c N_c)$$

$$Q_{tN} = (0.336 + 0.531 + 0.771) \times 3,500 \times 9$$

$$Q_{tN} = 51,600 \text{ lb}$$

where

$$A_8, A_{10}, A_{12} = \text{Projected area of helical plates (8", 10", and 12")}$$

$$N_c = \text{Bearing capacity factor related to the residual soil, clay}$$

$$A_8 = 0.336 \text{ ft}^2$$

$$A_{10} = 0.531 \text{ ft}^2$$

$$A_{12} = 0.771 \text{ ft}^2$$

$$N_c = 9$$

$$c = N / 8 = 28 / 8 = 3.5 \text{ ksf or } 3,500 \text{ psf}$$

(see Equation 5-41)

EQUATION 8-30

$$Q_{tM} = (A_8 + A_{10} + A_{12} + A_{14}) \times (c N_c)$$

$$Q_{tM} = (0.336 + 0.531 + 0.771 + 1.049) \times 3,500 \times 9$$

$$Q_{tM} = 84,640 \text{ lb}$$

where

$$A_8, A_{10}, A_{12}, A_{14} = \text{Projected area of helical plates (8", 10", 12", and 14")}$$

$$A_{14} = 1.049 \text{ ft}^2$$

CHECK ULTIMATE ANCHOR CAPACITY (T_U)

Compare Q_{tN} and Q_{tM} to field load tension tests if required by specifications.

DESIGN EXAMPLE 9: HELICAL TIEBACK ANCHORS IN SAND

STRUCTURE TYPE

- Cast concrete retaining wall
- Granular backfill for wall $\phi = 35^\circ$ $\gamma = 120$ pcf
- Height = 15 ft, thickness = 1-1/2 ft
- Anchor Height = $H/3 = 5$ ft
- Residual soils: silty coarse sand; medium to dense $\phi = 31^\circ$ $\gamma = 118$ pcf. No ground water table present.
- Tieback installation angle = 25°

STRUCTURAL DESIGN LOADS

- Use backfill $\phi = 35^\circ$

TIEBACK SPACING

$$\text{Spacing}_N = (Q_{tN} / FS) / DL_N = (51,600 / 2) / (4,025) = 6.4 \text{ ft}$$

$$\text{Spacing}_M = (Q_{tM} / FS) / DL_M = (84,640 / 2) / (6,040) = 7.0 \text{ ft}$$

(use 6'-6" center to center spacing for both rows of tiebacks) where $FS = 2.0$

ESTIMATE INSTALLATION TORQUE

EQUATION 8-31

$$T = (DL \times \text{Spacing} \times FS) / K_t$$

$$T_N = (DL_N \times \text{Spacing}_N \times FS) / K_t$$

$$= (4,025 \times 6.5 \times 2) / 10 = 5,300 \text{ ft}\cdot\text{lb}$$

$$T_M = (DL_M \times \text{Spacing}_M \times FS) / K_t$$

$$= (6,040 \times 6.5 \times 2) / 10 = 7,850 \text{ ft}\cdot\text{lb}$$

where

$$K_t = \text{Empirical torque factor (default value = 10 for Type SS series)}$$

CHECK INSTALLATION TORQUE RATINGS

The rated installation torque of the Type SS150 series is 7,000 ft·lb, which is greater than the required installation torque (T_N) of 5,300 ft·lb.

The rated installation torque of the Type SS175 series is 10,500 ft·lb, which is greater than the required installation torque (T_M) of 7,850 ft·lb.

MINIMUM TIEBACK LENGTH

The distance from the assumed "active" failure plane to the 12" helix must be at least 5 x its diameter or 5'-0". The distance from the assumed "active" failure plane to the 14" helix must be at least 5 x its diameter or 6'-0". Both the minimum length and estimated installation torque must be satisfied prior to the termination of tieback installation.

- $K_a = (1 - \sin(\phi)) / (1 + \sin(\phi)) = 0.27$
- $DL/\text{ft} = (1/2 \gamma H^2 K_a) / \cos(25^\circ)$
- $= [1/2 (120) (15)^2 (0.27)] / \cos(25^\circ)$
- $= 4,000 \text{ lb/lin ft}$
- Assume tieback carries 80%; therefore, $DL_t/\text{ft} = 0.80 \times 4,000 = 3,200 \text{ lb/lin ft}$

CHANCE HELICAL ANCHOR SELECTION

- Wall height ≤ 15 ft; use single row of tiebacks
- Try Type SS5 series, C1500007 (8"-10"-12" Lead)



ULTIMATE TENSION CAPACITY (USING BEARING CAPACITY APPROACH)

EQUATION 8-32

$$Q_t = (A_8 \times q_8 + A_{10} \times q_{10} + A_{12} \times q_{12}) \times (N_q)$$

A_8, A_{10}, A_{12} = Projected area of helical plates (8", 10" and 12")
 N_q = Bearing capacity factor related to ϕ of residual soil (31°)
 $A_8 = 0.336 \text{ ft}^2$
 $A_{10} = 0.531 \text{ ft}^2$
 $A_{12} = 0.771 \text{ ft}^2$
 $N_q = 15$ (from Equation 5-20)
 $q_h = \gamma \times D_h$ (depth of helix below ground line, ft)
 $q_8 = 118 \text{ pcf} (5' + 25' \sin(25^\circ)) = 1836 \text{ psf}$
 $q_{10} = 118 \text{ pcf} (5' + 23' \sin(25^\circ)) = 1736 \text{ psf}$
 $q_{12} = 118 \text{ pcf} (5' + 20.5' \sin(25^\circ)) = 1612 \text{ psf}$
 $Q_t = [(0.336 \times 1836) + (0.531 \times 1736) + (0.771 \times 1612)] \times 15$
 $Q_t = 41,725 \text{ lb}$

CHECK ULTIMATE ANCHOR CAPACITY (T_U)

Compare Q_t to field load tension tests if required by specifications.

TIEBACK SPACING

EQUATION 8-33

$$\text{Spacing}_N = (Q_t / FS) / DL_t$$

$$= (41,725 / 2) / (3,200) = 6.5 \text{ ft}$$

(use 6'-6" center to center spacing)

where $FS = 2.0$

ESTIMATE INSTALLATION TORQUE

EQUATION 8-34

$$T = (DL_t \times \text{Spacing} \times FS) / K_t$$

$$= (3,200 \times 6.5 \times 2.0) / 10$$

$$= 4,200 \text{ ft}\cdot\text{lb}$$

where

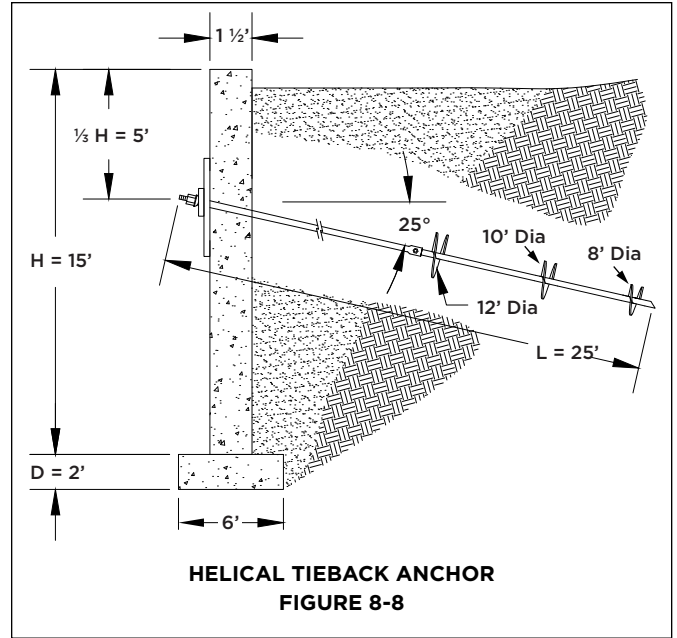
K_t = Empirical torque factor (default value = 10 for Type SS series)

CHECK INSTALLATION TORQUE RATINGS

The rated installation torque of the Type SS5 series is 5,700 ft·lb, which is greater than the required installation torque (T) of 4,200 ft·lb.

MINIMUM TIEBACK LENGTH

The distance from the assumed "active" failure plane to the 12" helix must be at least 5 times its diameter or 5'-0". Both the minimum length and estimated installation torque must be satisfied prior to the termination of tieback installation.



SOIL BORING LOG				
Graphic Log	Soil Classification	Depth	USCS Symbol	SPT - N Blows/ft
	Topsoil		OH	
		5		17
	Silty Sand		SM	
		10		30
		15		32
	Silty Coarse Sand $\gamma = 118 \text{ pcf}$ $\phi = 31^\circ$	20		34

SOIL BORING LOG
FIGURE 8-9

DESIGN EXAMPLE 10: SOIL SCREW RETENTION WALL SYSTEM

PROBLEM

Determine the Soil Screw anchor spacing (S_V, S_H), Soil Screw anchor length (L) and facing requirements for an excavation support system for a 23 foot deep excavation in a silty sand. The required design Factor of Safety (FS) for internal stability is 1.5, and for global stability is 1.3.

STEP 1 - DEFINE DESIGN PARAMETERS

Given: The unit weight (γ) and friction angle (ϕ) of the silty sand is 120 pcf and 30° respectively. The allowable bearing capacity of the silty sand at the bottom of the excavation is 4000 psf. The electrochemical properties of the silty sand are listed below:

- Resistivity 4000 Ω -cm
- pH 7
- Chlorides 50 ppm
- Sulfates 100 ppm

A design live surcharge load of 100 psf is considered to be applied uniformly across the ground surface at the top of the wall. The wall face is vertical. Groundwater is located 60 feet below the ground surface.

Chance® Type SS5 Helical Soil Screw anchors, for which lead sections and extensions are available in 5' and 7' lengths, are to be used for the Soil Screw anchors. The design life of the structure is one year. Design Soil Screw anchor lengths will be governed by the lead and extension pieces and thus will be 10', 12', 14', 15', 17', 19', etc.

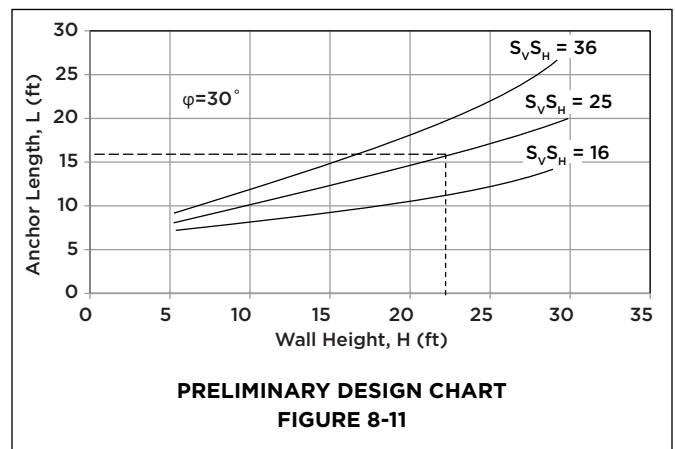
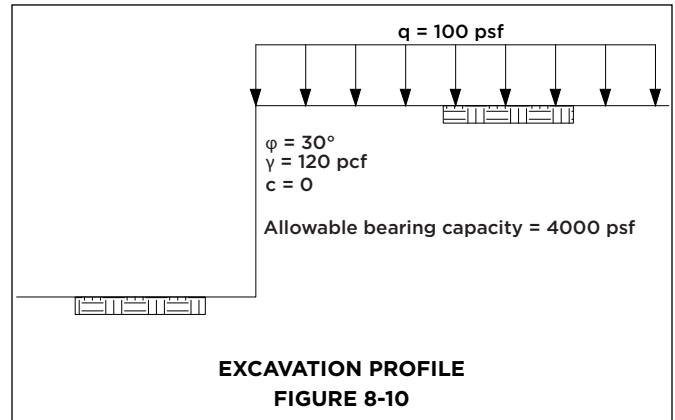
STEP 2 - CHECK THE PRELIMINARY FEASIBILITY OF THE SOIL SCREW RETENTION WALL SYSTEM

The medium dense, silty sands at this site are well suited for the Soil Screw retention wall system (i.e., good stand up time). The ground water table (GWT) is well below the bottom of the excavation. The conditions at the site are therefore favorable for the Soil Screw retention wall system.

Design charts are used to determine preliminary Soil Screw anchor spacing and lengths for the given wall geometry, loading and soil conditions. For the soil conditions, $\phi = 30^\circ$, enter the Preliminary Design Chart (Figure 8-11) along the x-axis at a wall height (H) = 23 ft. A typical Soil Screw anchor spacing for soils with "good" stand up time is 5 ft x 5 ft. Therefore, use the $S_V S_H = 25$ curve to determine the preliminary Soil Screw anchor length (L) = 16 ft.

STEP 3 - DETERMINE EXTERNAL EARTH PRESSURES

Use Equation 8-35 to determine the active earth pressure (K_a) at the back of the reinforced soil mass.



EQUATION 8-35

$$K_a = \tan^2[45 - (\phi/2)]$$

$$K_a = \tan^2[45 - (30/2)]$$

$$= 0.33$$

STEP 4 - CHECK PRELIMINARY SOIL SCREW ANCHOR LENGTH WITH RESPECT TO SLIDING

Available Soil Screw anchor lengths for Chance Type SS5 anchors are 10', 12', 14', 15', 17', 19', etc. The 16 foot preliminary length determined in Step 2 does not account for surcharge loading, which tends to increase Soil Screw anchor lengths. Try 19' Soil Screw anchors (length to height ratio of 0.83). For preliminary designs for walls with the given soil and loading conditions, a length to height ratio of 0.8 to 1.0 is a starting point for the analysis and appears to be conservative.

The horizontal force from the retained soil (F_1) is determined using Equation 8-36.



EQUATION 8-36

$$F_1 = 1/2 K_a \gamma H^2$$

$$F_1 = 1/2 (0.33) (120) 23^2 = 10474 \text{ lb/lf of wall}$$

The horizontal force from the surcharge load (F_2) is determined using Equation 8-37.

EQUATION 8-37

$$F_2 = K_a qH$$

$$= 0.33 (100) 23 = 759 \text{ lb/lf of wall}$$

Using 19' Soil Screw® anchors installed at a 15° angle, the horizontal length (L_x) of the Soil Screw anchor is determined using Equation 8-38.

EQUATION 8-38

$$L_x = L \cos(15^\circ)$$

$$L_x = 19 \cos(15^\circ) = 18.4 \text{ ft}$$

The Factor of Safety against sliding is determined using Equation 8-39.

EQUATION 8-39

$$FS = \frac{\gamma HL_x \tan(\phi)}{F_1 + F_2}$$

$$= \frac{120 (23) 18.4 \tan(30^\circ)}{10474 + 759}$$

$$FS = 2.61$$

STEP 5 - CHECK REQUIRED BEARING CAPACITY AT THE BASE OF THE WALL

Determine the eccentricity (e) of the resultant vertical force using Equation 8-40.

EQUATION 8-40

$$e = \frac{[F_1 (H/3)] + [F_2 (H/2)]}{\gamma HL_x}$$

$$= \frac{[10474 (23/3)] + [759 (23/2)]}{120 (23) 18.4}$$

$$= 1.75 < (L_x/6) = (18.4/6) = 3.06$$

The vertical stress (σ_v) of the bottom of the wall is determined using Equation 8-41.

EQUATION 8-41

$$\sigma_v = \frac{\gamma HL_x + qL_x}{L_x - 2e}$$

$$= \frac{120 (23) 18.4 + 100 (18.4)}{18.4 - 2 (1.75)}$$

$$= 3532 \text{ psf}$$

Given the allowable bearing capacity (Q_{allow}) is 4000 psf:

EQUATION 8-42

$$Q_{allow} = 4000 \text{ psf} > \sigma_v = 3532 \text{ psf}$$

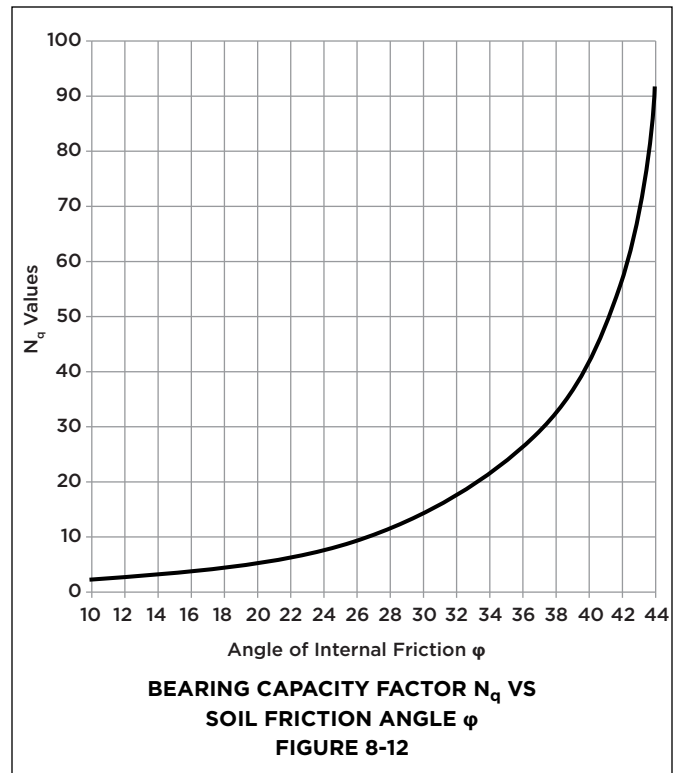
STEP 6 - DETERMINE THE ALLOWABLE HELICAL ANCHOR STRENGTH

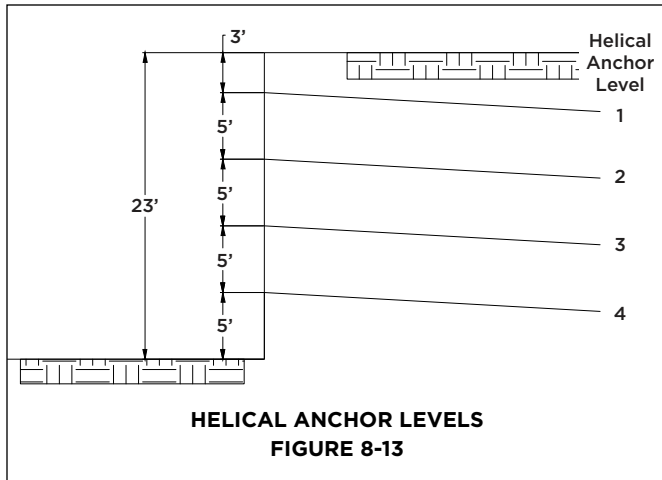
ALLOWABLE DESIGN STRENGTH OF TYPE SS5 HELICAL ANCHOR (SERVICE LIFE = 75 YEARS), TABLE 8-5

Ta 75 yrs (kip)	V 75 yrs (kip)	ALLOWABLE DESIGN STRENGTH (TEMPORARY STRUCTURES) (kip)	ALLOWABLE DESIGN STRENGTH 75 yrs (kip)
50	37	45	37

The Soil Screw Anchor wall is a temporary structure with a design life of one year. From Table 8-5, the allowable design strength of the Chance® SS5 anchor is 45 kip. This table is based on the following electrochemical properties of soil:

- Resistivity: >3000 Ω-cm
- pH: >5 <10
- Chlorides: 100 ppm
- Sulfates: 200 ppm
- Organic content: 1% max





STEP 7 - ESTIMATE THE TENSION CAPACITY OF THE SOIL SCREW® ANCHORS

Determine the bearing capacity factor (N_q) for helical anchors for a sand with an effective friction angle, $\phi = 30^\circ$. From Figure 8-12, $N_q = 14$. Assumed vertical spacing is 5 feet (see Figure 8-13). Nail pattern is as shown in Figure 8-13. There are eight helices per anchor, as shown in Figure 8-14.

The ultimate tension capacity (P) of the helical Soil Screw anchor at Level 1 is determined using Equation 8-43.

EQUATION 8-43

$$P = \sum_{i=1}^8 A_i q_i N_q$$

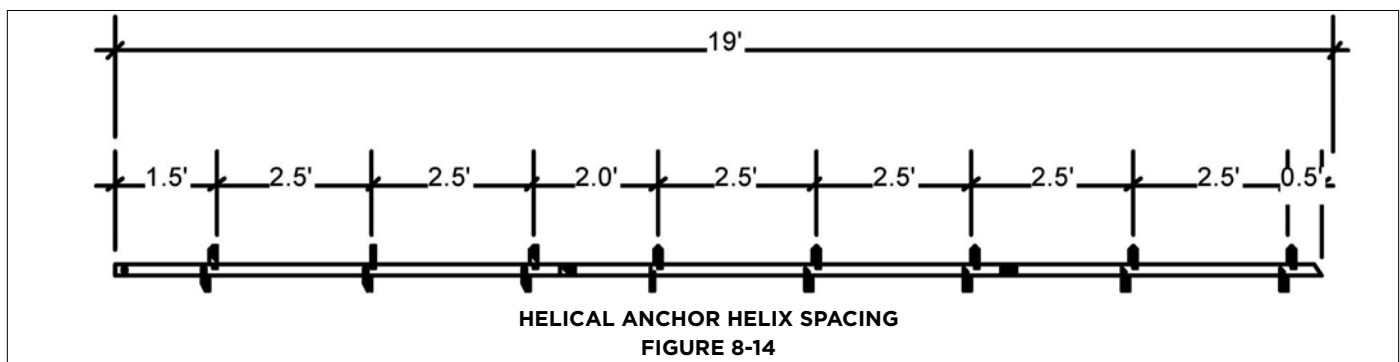
Helical anchors have 8" diameter helices. The helix area (A) can be calculated using Equation 8-44.

EQUATION 8-44

$$A = \pi(0.33)^2$$

$$= 0.336 \text{ ft}^2 \text{ (use } 0.34 \text{ ft}^2\text{)}$$

The ultimate tension capacities for the helical anchors at the various levels are determined using Equation 8-45.



EQUATION 8-45

$$y = L \sin(\theta)$$

$$= 19 \sin(15^\circ)$$

$$= 4.9 \text{ ft}$$

where

- L = Length of Soil Screw anchor
- θ = Installation angle (from horizontal)

Average Overburden Depth = $3 + (y/2) = 5.5 \text{ ft}$ at Level 1

$$P_{\text{LEVEL1}} = 8 (0.34) 5.5 (120) 14 = 25 \text{ kip}$$

$$P_{\text{LEVEL2}} = 8 (0.34) 10.5 (120) 14 = 48 \text{ kip}$$

$$P_{\text{LEVEL3}} = 8 (0.34) 15.5 (120) 14 = 71 \text{ kip}$$

$$P_{\text{LEVEL4}} = 8 (0.34) 20.5 (120) 14 = 94 \text{ kip}$$

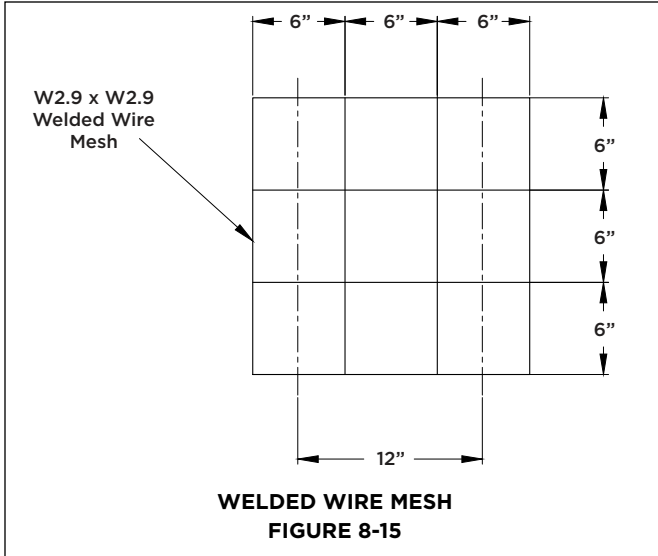
STEP 8 - DEFINE A TRIAL FACING SYSTEM

Try a 4" thick, 4000 psi shotcrete face with 6 x 6, W2.9 x W2.9 welded wire mesh reinforcing and two #4 vertical rebars at the helical anchor locations. Try a helical anchor spacing of 5 feet vertically and horizontally and an 8" square by 3/4" thick bearing plate with a steel yield stress of 36 ksi.

STEP 9 - DETERMINE THE ALLOWABLE FLEXURAL STRENGTH OF THE FACING

For typical helical anchor wall construction practice, the facing is analyzed using vertical strips of width equal to the horizontal anchor spacing. For facing systems involving horizontal nail spacings that are larger than the vertical spacing or unit horizontal moment capacities that are less than the vertical unit moment capacities, horizontal strips of width equal to the vertical anchor spacing should be used.

The area of steel (A_s) for a vertical beam of width 5 feet ($S_H = 5$ feet) with the anchor on the beam's centerline is determined using Equation 8-46. Diameter (d) of the welded fabric wire is 0.192". Diameter (D) of the rebar is 0.500". For a 5 foot wide vertical beam centered between the anchors, the rebars are located at the beam edges and should be ignored. A_s is calculated using Equation 8-47. The corresponding average nominal unit moment resistances are determined using Equation 8-48.


EQUATION 8-46

$$\begin{aligned}
 A_{s,neg} &= \left(\frac{\pi d^2}{4} \right) \left(\frac{\text{in}^2}{\text{wire}} \right) \times \left(\frac{2 \text{ wires}}{\text{ft}} \right) (5 \text{ ft}) + \left(\frac{\pi D^2}{4} \right) \left(\frac{\text{in}^2}{\text{rebar}} \right) \times \left(\frac{2 \text{ rebars}}{5 \text{ ft}} \right) (5 \text{ ft}) \\
 &= \left(\frac{\pi (0.192^2)}{4} \right) (2) (5) + \left(\frac{\pi (0.500^2)}{4} \right) (2) \\
 &= 0.682 \text{ in}^2
 \end{aligned}$$

EQUATION 8-47

$$\begin{aligned}
 A_{s,pos} &= \left(\frac{\pi d^2}{4} \right) \left(\frac{\text{in}^2}{\text{wire}} \right) \times \left(\frac{2 \text{ wires}}{\text{ft}} \right) (5 \text{ ft}) \\
 &= \frac{\pi (0.192^2)}{4} (2) (5) \\
 &= 0.289 \text{ in}^2
 \end{aligned}$$

EQUATION 8-48

$$\begin{aligned}
 m_v &= \frac{A_s F_y \left(d - \frac{A_s F_y}{1.7 f_{cb}} \right)}{b} \\
 m_{v,neg} &= \frac{0.682 (60) \left(2 - \frac{0.682 (60)}{1.7 (4) (5 \times 12)} \right)}{5 (12)} \\
 &= 1.30 \text{ k in/in} \\
 &= 1.30 \text{ k ft/ft} \\
 m_{v,pos} &= \frac{0.289 (60) \left(2 - \frac{0.289 (60)}{1.7 (4) (5 \times 12)} \right)}{5 (12)} \\
 &= 566 \text{ ft/ft}
 \end{aligned}$$

STEP 10 - DETERMINE THE MAXIMUM HELICAL ANCHOR HEAD LOAD

Determine the maximum helical anchor head load that will produce the allowable moments determined in Step 9 using Equation 8-49. Using Table 8-6, determine the facing pressure factor (C_F) for temporary shotcrete facing 4" thick.

EQUATION 8-49

$$\begin{aligned}
 T_{FN, \text{flexure}} &= C_F (m_{v,neg} + m_{v,pos}) 8 (S_H/S_V) \\
 T_{FN, \text{flexure}} &= 2.0 (1.30 + 0.57) 8 (5 \text{ ft}/5 \text{ ft}) = 29.8 \text{ kip}
 \end{aligned}$$

FACING PRESSURE FACTOR, TABLE 8-6

NOMINAL FACING THICKNESS (in)	TEMPORARY FACING C_F	PERMANENT FACING C_F
4	2.0	1.0
6	1.5	1.0
8	1.0	1.0

STEP 11 - DETERMINE THE ALLOWABLE PUNCHING SHEAR STRENGTH OF THE FACING

The punching shear strength (V_N) is determined using Equation 8-50.

EQUATION 8-50

$$\begin{aligned}
 V_N &= 0.125 \sqrt{f'_c} (\pi) (D'_c) (h_c) \\
 V_N &= 0.125 \sqrt{4} (\pi) (12) (4) = 38 \text{ kip}
 \end{aligned}$$

where

$$\begin{aligned}
 f'_c &= 4,000 \text{ psi} = 4 \text{ ksi} \\
 h_c &= 4 \text{ in} \\
 D'_c &= 8 + 4 = 12 \text{ in}
 \end{aligned}$$

STEP 12 - DETERMINE CRITICAL HELICAL ANCHOR HEAD LOAD FOR PUNCHING

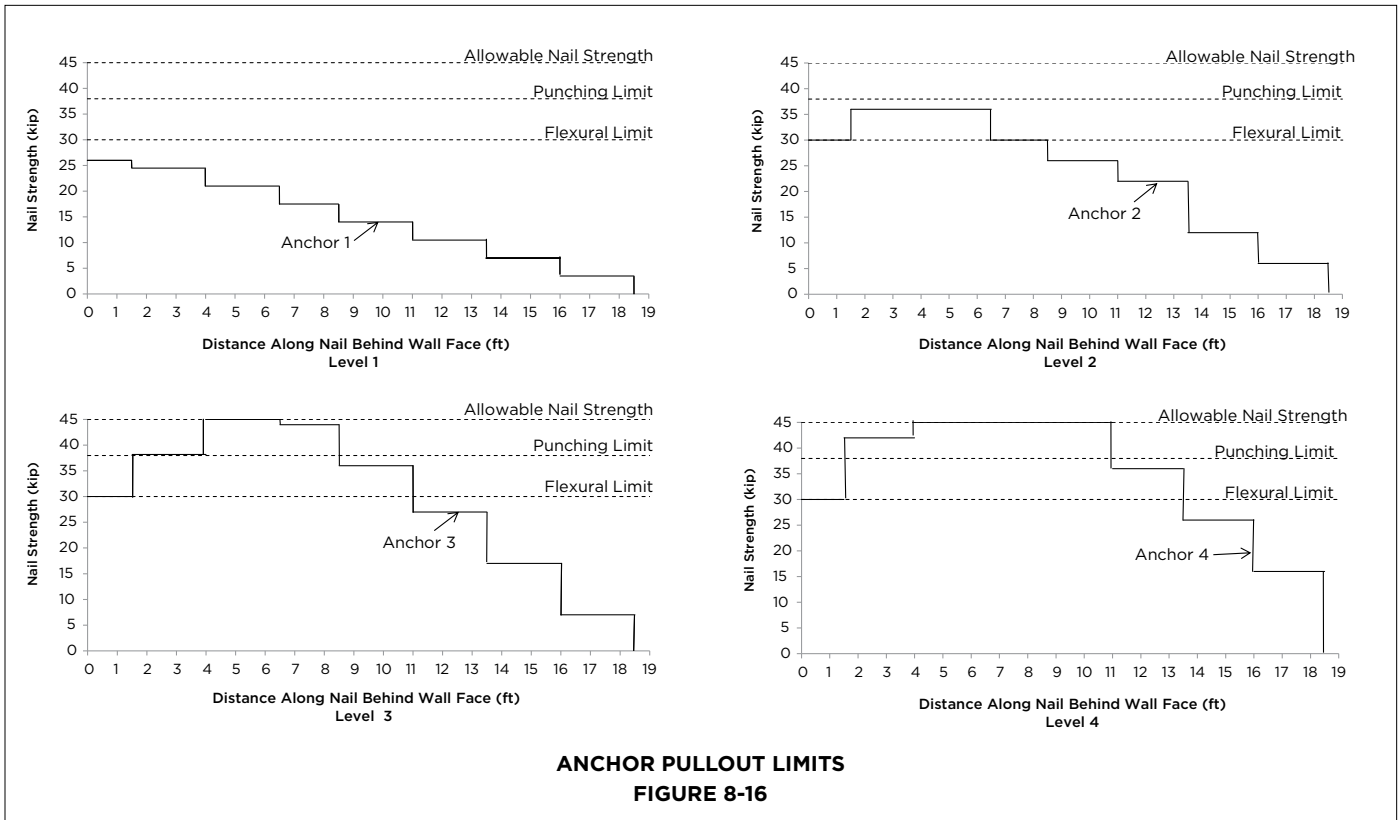
Determine the critical helical anchor head load (T_{FN}) for punching using Equation 8-51.

EQUATION 8-51

$$T_{FN, \text{punching}} = V_N = 38 \text{ kip}$$

STEP 13 - CONSTRUCT SOIL SCREW® ANCHOR STRENGTH ENVELOPE

Construct the strength envelope at each anchor level as shown in Figure 8-16. At the wall face, the anchor head flexural strength is less than the anchor head punching strength and therefore controls. There are eight helices per anchor. Each step in strength equals the single-helix bearing capacity for the anchor layer (Step 7). From the last helix (working from right to left) increase the pullout capacity in a stepwise fashion. If the pullout envelope working from the back of the nail does not intersect the flexural limit line, the strength envelope will



look like that shown for Anchor 1. If the pullout envelope working from the back of the nail exceeds the flexural limit, then construct a pullout envelope working from the flexural limit at the head of the nail.

STEP 14 - EVALUATE INTERNAL AND COMPOUND STABILITY

GoldNail 3.11, “A Stability Analysis Computer Program for Soil Nail Wall Design,” developed by Golder and Associates, was used to perform the internal and compound stability analysis. Refer to Attachment EX1 in the Chance® Soil Screw® Retention Wall System Design Manual for printout result of this stability analysis. The following discussion is based on these results.

The anchor strength envelope developed in Step 13 needs to be modified for GoldNail. The increase in pullout capacity along the length of the nail is estimated for GoldNail as straight lines, not step functions. An example of this modification for Anchor Level 2 is shown in Figure 8-17.

Within GoldNail there are several analysis options. The option used for this example is “Factor of Safety.” Using this option, the Internal Factor of Safety (FS_{internal}) = 2.11 for the anchor pattern defined in Step 7. The GoldNail output printout lists “Global Stability” not “Internal Stability.” However, the location of the critical failure surface (Circle #13) indicates an internal mode of failure, as shown on the GoldNail geometry printout.

STEP 15 - CHECK GLOBAL STABILITY

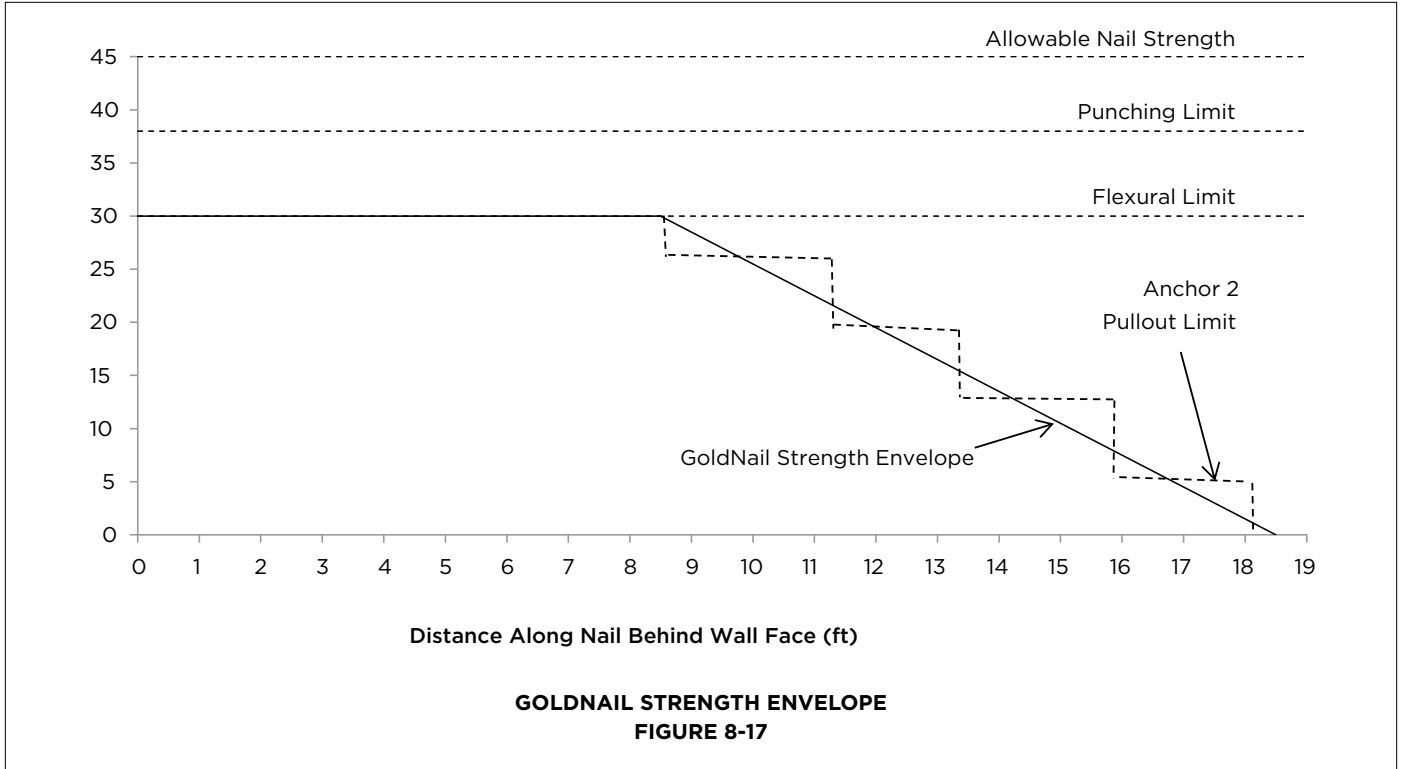
Analysis was performed for the given slope geometry by the computer program PCSTABL6H, developed by Purdue University and modified by Harald Van Aller, and the pre-processor STED, developed by Harald Van Aller. The resulting Global Factor of Safety (FS_{global}) = 1.93. Refer to Attachment EX2 in the Chance Soil Screw Retention Wall System Design Manual for printout results of this global stability analysis.

STEP 16 - CHECK CANTILEVER AT TOP OF WALL

In Step 7 the layout of anchors was assumed. The cantilever at the top of the wall from Step 7 is 3 feet. Check cantilever moment (M_c) using Equation 8-52.

EQUATION 8-52

$$\begin{aligned}
 M_c &= K_a \gamma \left[\left(\frac{H_1^2}{2} \right) \left(\frac{H_1}{3} \right) + q \left(\frac{H_1^2}{2} \right) \right] \\
 &= 0.33 (120) \left[\left(\frac{3^2}{2} \right) \left(\frac{3}{3} \right) + 100 \left(\frac{3^2}{2} \right) \right] \\
 &= 326.7 \text{ ft}\cdot\text{lb}/\text{ft}
 \end{aligned}$$



Maximum allowable moment at midspan (Step 9) is 566 ft·lb/ft, therefore:

EQUATION 8-53

$$FS_{Mc} = (566 / 327) = 1.73 \quad \text{OK}$$

Check shear force at cantilever (S_c) using Equation 8-54.

EQUATION 8-54

$$\begin{aligned} S_c &= K_a [\gamma (H_1^2 / 2) + qH_1] \\ &= 0.33 [120 (3^2 / 2) + 100 (3)] \\ &= 277 \text{ lb/ft} \end{aligned}$$

Determine allowable shear using Equation 8-55

EQUATION 8-55

$$\begin{aligned} V_N &= 0.125 \sqrt{f'_c} h_c \\ &= 0.125 \sqrt{4} (4) = 1000 \text{ lb/ft} \end{aligned}$$

EQUATION 8-56

$$FS_{\text{shear}} = (1000 / 277) = 3.6 \quad \text{OK}$$

DESIGN EXAMPLE 11: HELICAL PILES/ANCHORS FOR TELECOMMUNICATION TOWERS

PURPOSE

This Design Example provides an aid in the selection of appropriate helical guywire anchors and center mast helical piles for telecommunication towers.

The guywire loads are to be resisted by a helical tension anchor. When the vertical and horizontal components are provided the resultant must be determined as well as the angle between the resultant load and the horizontal, (this is the angle the helical anchor should be installed at to properly resist the guywire load(s)). There may be one or more guywires that come to the ground to be restrained by one or more helical anchors depending on the magnitude of the load and/or the soil strength. Helical piles can be used to resist the loads from the structure

mast. These loads will generally be composed of a vertical load and a lateral load at the base of the mast or pole.

If the structure is a self supporting tower (SST), the loads from each leg of the tower must be resisted. These generally consist of vertical uplift and compression loads and a horizontal shear load at the ground line. These three loads can be dealt with in a number of ways. Typically one or more helical piles are used for each leg of the tower and may be installed at a batter to better resist the horizontal shear loads. Steel grillages and reinforced concrete caps have been used to facilitate load transfer from the structure to the helical piles. This type design will not be covered in this design example since the intent is to focus on the guyed mast tower structure.

Figure 8-18 shows the tower that will be used for these sample calculations. It will be noted that the four upper guywires come to the ground at a single guywire point and that the three lower guywires come to ground at a different guywire point. There must be at least a single helical anchor installed at each of these points to provide restraint for the guywires which in turn stabilize the tower by resisting lateral loads on the structure.

For this tower, the vertical and horizontal components of the guywire loads are given and must be resolved into the tension load the helical guywire anchor is to resist.

UPPER GUYWIRE LOADS

- Vertical load component = 16.6 kip
- Horizontal load component = 17.9 kip
- Tension in the upper guywire anchor =
 $T_{ug} = (16.6^2 + 17.9^2)^{0.5} = 24.4 \text{ kip}$
- Helical guywire anchor installation angle =
 $IA_{ug} = \tan^{-1}(16.6/17.9) = 43^\circ$

LOWER GUYWIRE LOADS

- Vertical load component: 7.9 kip
- Horizontal load component: 9.7 kip
- Tension in the lower guywire anchor =
 $T_{lg} = (7.9^2 + 9.7^2)^{0.5} = 12.5 \text{ kip}$
- Helical guywire anchor installation angle =
 $IA_{lg} = \tan^{-1}(7.9/9.7) = 39^\circ$

MAST FOUNDATION LOADS

- Compression (C) = 68.0 kip
- Horizontal shear (V) = 0.3 kip

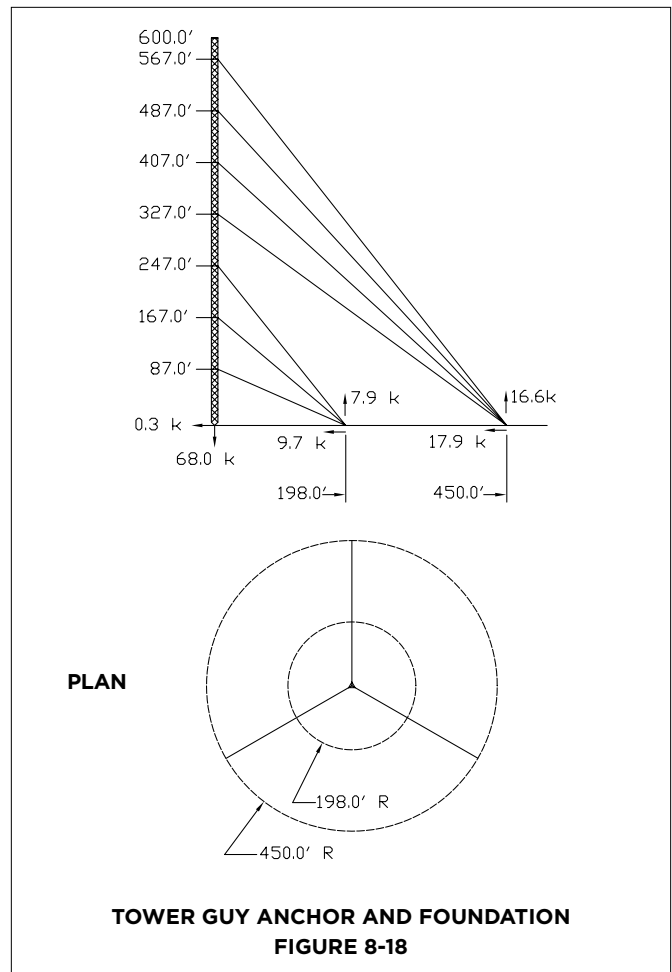
SELECTING HELICAL GUYWIRE ANCHORS

Hubbell Power Systems, Inc. HeliCAP® engineering software will be utilized to determine the appropriate helical anchor/pile sizes for this tower. Soil conditions are shown in the Sample Boring Log in Figure 8-19. The soil data and guywire anchor data was input into the HeliCAP engineering software to get an appropriate output. The minimum acceptable Factor of Safety (FS) = 2.

UPPER GUYWIRE HELICAL ANCHOR

The HeliCAP summary report for the upper guywire helical anchor is shown in Figure 8-20. This report provides the following information:

- Helical Anchor: SS5 (1.5" square shaft, 5,700 ft-lb torque rating, 70 kip ultimate tension rating)
- Lead Section: 4 helix (8"-10"-12"-14")
- Installation Angle: 43°
- Datum Depth (depth below grade where installation starts): 0 ft



- Length: 45 ft (along the shaft at the 43° installation angle)
- Recommended Ultimate Capacity (R_{uc}): 50.2 kip (tension)

The Factor of Safety for this tension anchor is $R_{uc} / T_{ug} = 50.2 / 24.4 = 2.05 > 2$ (OK). Use this helical anchor at each of three upper guywire anchor locations per tower.

The required average minimum installation torque (T) is:

EQUATION 8-57

$$\begin{aligned} T &= (T_{ug} \times FS) / K_t \\ &= (24,400 \times 2.0) / 10 \\ &= 4,900 \text{ ft-lb} \end{aligned}$$

where

$$\begin{aligned} K_t &= \text{Empirical torque factor} = 10 \text{ (default value for Type SS5 series)} \\ T &= 4,900 \text{ ft-lb is less than the rated torque (5,700 ft-lb) of the Type SS5 series. (OK).} \end{aligned}$$



LOWER GUYWIRE HELICAL ANCHOR

The HeliCAP® summary report for the lower guywire helical anchor is shown in Figure 8-21. This report provides the following information:

- Helical Anchor: SS5 (1.5" square shaft, 5,700 ft·lb torque rating, 70 kip ultimate tension rating)
- Lead Section: 4 helix (8"-10"-12"-14")
- Installation Angle: 39°
- Datum Depth (depth below grade where installation starts): 0 ft
- Length: 25 ft (along the shaft at the 39° installation angle)
- Recommended Ultimate Capacity (R_{uc}): 26.6 kip (tension)

The Factor of Safety for this tension anchor is $R_{uc} / T_{lg} = 26.6 / 12.5 = 2.12 > 2$ (OK) Use this helical anchor at each of three lower guywire anchor locations per tower.

EQUATION 8-58

$$\begin{aligned}
 T &= (T_{lg} \times FS) / K_t \\
 &= (12,500 \times 2.0) / 10 \\
 &= 2,500 \text{ ft·lb}
 \end{aligned}$$

where

- K_t = Empirical torque factor = 10 (default value for Type SS5 series)
- $T = 2,500 \text{ ft·lb}$ is less than the rated torque (5,700 ft·lb) of the Type SS5 series. (OK).

HELICAL PILE

Given:

- Compression Load = 68.0 kip
- Shear Load = 0.3 kip

Assume three helical piles installed at 120° intervals in plan view with each pile battered away from vertical at a 10° angle:

$$68/3 \text{ piles} = 22.67 \text{ kip ultimate/pile element.}$$

Assume entire shear (0.3 kip) is taken by one battered pile. Therefore, the resultant axial load (DL) to a battered pile is:

$$DL = (22.67^2 + 0.3^2)^{0.5} = 22.7 \text{ kip}$$

The HeliCAP summary report for the helical piles is shown in Figure 8-22. This report provides the following information:

- Helical Pile: SS175 (1.75" square shaft, 10,500 ft·lb torque rating, 100 kip ultimate tension rating)
- Lead Section: 4 helix (8"-10"-12"-14")
- Installation Angle: 80° below horizontal (10° away from vertical)
- Datum Depth: (depth below grade where installation starts): 0 ft
- Length: 34 ft (along the shaft at the 80° installation angle)
- Recommended Ultimate Capacity (R_{uc}): 50.7 kip (compression)

The Factor of Safety for this compression pile is $R_{uc} / DL = 50.7 / 22.7 = 2.23 > 2$ (OK) Use three SS175 helical piles per tower base. The three helical piles must be captured in a "pile cap." This may be a reinforced concrete cap, the design of which is beyond the scope of this design example. The design of this concrete pile cap is left to the structural engineer.

EQUATION 8-59

$$\begin{aligned}
 T &= (DL \times FS) / K_t \\
 &= (22,700 \times 2.0) / 10 \\
 &= 4,500 \text{ ft·lb}
 \end{aligned}$$

where

- K_t = Empirical torque factor = 10 (default value for Type SS175 series)
- $T = 4,500 \text{ ft·lb}$ is less than the rated torque (10,500 ft·lb) of the Type SS175 series. (OK).



DESIGN EXAMPLES

Project No.:		Boring Log		Rig: CME 56							
Project:				Location:							
Client:				Driller:							
Boring No.: 1											
SUBSURFACE PROFILE				SAMPLE							
Depth (ft.)	Symbol	Description	Q _{tip} , L.s.f.	Dry Density, P.C.F.	Depth/Elev.	Number	Type	Blowft.	Q _{tip} , T.S.F.	Standard Penetration Test blows/ft.	Water Content % Wp — WI
0		Ground Surface			0.0					10 20 30 40	10 20 30 40
		Topsoil (2")			-0.8						
		Light Gray Mottled Yellow Brown Silty Clay, (CL) Stiff, (CL)			0.8	1	SS	11		11	
5		Mottled Reddish Brown, Medium, (CL)	0.75			2	SS	6		6	
		Medium, (CL)	1.00			3	SS	6		6	
10		Mottled Yellow Brown, Grading to Clay (CH) @ 10.5 ft.	1.75			4	SS	7		7	
		Light Gray Mottled Yellow Brown Clay, Little Sand, Stiff, (CH)	2.50		-10.5	5	SS	12		12	
		Yellow Brown Mottled Light Gray, Stiff, (CH)	3.00		10.5	6	SS	11		11	
		Light Gray Mottled Yellow Brown w/Sand, Stiff, (CH)	3.00			7	SS	11		11	
20		Light Gray Mottled Yellow Brown Sandy Clay, Trace of Gravel, Wet, Stiff, (CL)	1.50		-19.0	8	SS	8		8	
		Yellow Brown Mottled Light Gray w/Gravel, Stiff, (CL)	2.50		19.0	9	SS	11		11	
25		Yellow Brown Mottled Light Gray Sandy Silty Clay w/Gravel, Very Stiff, (CL)	3.50		-24.0	10	SS	21		21	
		Very Stiff, (CL)	3.50		24.0	11	SS	17		17	
30		Very Stiff, (CL)	3.75			12	SS	14		14	
		Light Gray Mottled Yellow Brown Clay w/ Sand,			-31.5						
					31.5						
					32.5						

Drill Method: **3 1/4" HSA & SPT**
 Boring Started:
 Boring Completed:
 Tested By:
 Logging By:

Groundwater Elev. During Drilling: \approx
 Groundwater Elev. @ Comp.: \approx
 Groundwater Elev. @ 5.5 Hrs.: \approx **-22.0**
 Boring Location:

Sheet 1 of 2

SAMPLE BORING LOG
FIGURE 8-19



HelICAP SUMMARY REPORT

b Name: Tower Guy Calculations

C:\Documents and Settings\j\goen\Desktop\Tow

b Number: Upper Guy

6/1/2006 8:43:36 AM

Water Table Depth: 22 ft

ring No: 1

Anchor Use: Tension

Capacity Summary

Anchor Number	Anchor Family	Helix Depth (ft)	Helix Capacity (kips)	Total Anchor Capacity (kips)	Recommended Ultimate Capacity (kips)	Torque (ft-lbs)
Anchor 1	Angle: 43 Datum Depth: 0 Length: 45					
14" helix	SS 5	25.2	16.9t 24.8c			
12" helix	SS 5	27.2	17t 14.7c			
10" helix	SS 5	28.9	10.1t 9.5c	50.2t	50.2t	5502
8" helix	SS 5	30.3	6.1t 5.3c	54.4c	54.4c	

Soil Profile

Top of Layer Depth (ft)	Soil Type	Cohesion (lb/ft ²)	N	Angle of Internal Friction (Degrees)	Unit Weight (lb/ft ³)
0	Clay	1375	11	0	102
5	Clay	750	6	0	92
7	Clay	750	6	0	92
10	Clay	875	7	0	94
12	Clay	1500	12	0	104
15	Clay	1375	11	0	102
17	Clay	1375	11	0	102
20	Clay	1000	8	0	96
22	Clay	1375	11	0	102
25	Clay	2625	21	0	120
27	Clay	2125	17	0	114
30	Clay	1750	14	0	108
32	Clay	1750	14	0	108
35	Clay	1500	12	0	104
37	Clay	1625	13	0	106
40	Clay	1500	12	0	104
42	Clay	1375	11	0	102
45	Clay	2125	17	0	114
47	Clay	2500	20	0	120
50	Clay	6125	49	0	138

HELICAP® SOFTWARE SUMMARY REPORT FOR UPPER GUYWIRES
FIGURE 8-20



HeliCAP SUMMARY REPORT

Job Name: Tower Guy Calculations

C:\Documents and Settings\jlgoben\Desktop\Tower
6/1/2006 8:31:40 AM

Job Number: Lower Guy

Water Table Depth: 22 ft

Boring No: 1

Anchor Use: Tension

Capacity Summary

Anchor Number	Anchor Family	Helix Depth (ft)	Helix Capacity (kips)	Total Anchor Capacity (kips)	Recommended Ultimate Capacity (kips)	Torque (ft-lbs)
Anchor 1	Angle: 39 Datum Depth: 0 Length: 25					
14" helix	SS 5	10.6	7.4t 10.2c			
12" helix	SS 5	12.5	7.5t 10.3c			
10" helix	SS 5	14.1	7.1t 6.9c	26.6t	26.6t	3002
8" helix	SS 5	15.4	4.4t 4.2c	31.7c	31.7c	

Soil Profile

Top of Layer Depth (ft)	Soil Type	Cohesion (lb/ft2)	N	Angle of Internal Friction (Degrees)	Unit Weight (lb/ft3)
0	Clay	1375	11	0	102
5	Clay	750	6	0	92
7	Clay	750	6	0	92
10	Clay	875	7	0	94
12	Clay	1500	12	0	104
15	Clay	1375	11	0	102
17	Clay	1375	11	0	102
20	Clay	1000	8	0	96
22	Clay	1375	11	0	102
25	Clay	2625	21	0	120
27	Clay	2125	17	0	114
30	Clay	1750	14	0	108
32	Clay	1750	14	0	108
35	Clay	1500	12	0	104
37	Clay	1625	13	0	106
40	Clay	1500	12	0	104
42	Clay	1375	11	0	102
45	Clay	2125	17	0	114
47	Clay	2500	20	0	120
50	Clay	6125	49	0	138

HELICAP® SOFTWARE SUMMARY REPORT FOR LOWER GUYWIRES
FIGURE 8-21



HelicAP SUMMARY REPORT

Job Name: Tower Foundation Calculations

C:\Documents and Settings\jgoen\Desktop\Tow

6/1/2006 9:27:53 AM

Job Number: Three Foundations per Tower Base

Water Table Depth: 22 ft

Boring No: 1

Anchor Use: Compression

Capacity Summary

Anchor Number	Anchor Family	Helix Depth (ft)	Helix Capacity (kips)	Total Anchor Capacity (kips)	Recommended Ultimate Capacity (kips)	Torque (ft-lbs)
Anchor 1	Angle: 80 Datum Depth: 0 Length: 34					
14" helix	SS 175	25.6	16.9t 23.2c			
12" helix	SS 175	28.5	15.8t 13.8c			
10" helix	SS 175	31	8.9t 8.3c	47.1t	47.1t	5323
8" helix	SS 175	32.9	5.3t 5.3c	50.7c	50.7c	

Soil Profile

Top of Layer Depth (ft)	Soil Type	Cohesion (lb/ft2)	N	Angle of Internal Friction (Degrees)	Unit Weight (lb/ft3)
0	Clay	1375	11	0	102
5	Clay	750	6	0	92
7	Clay	750	6	0	92
10	Clay	875	7	0	94
12	Clay	1500	12	0	104
15	Clay	1375	11	0	102
17	Clay	1375	11	0	102
20	Clay	1000	8	0	96
22	Clay	1375	11	0	102
25	Clay	2625	21	0	120
27	Clay	2125	17	0	114
30	Clay	1750	14	0	108
32	Clay	1750	14	0	108
35	Clay	1500	12	0	104
37	Clay	1625	13	0	106
40	Clay	1500	12	0	104
42	Clay	1375	11	0	102
45	Clay	2125	17	0	114
47	Clay	2500	20	0	120
50	Clay	6125	49	0	138

HELICAP® SOFTWARE SUMMARY REPORT FOR FOUNDATIONS

FIGURE 8-22

DESIGN EXAMPLE 12: HELICAL ANCHORS FOR PIPELINE BUOYANCY CONTROL

PURPOSE

This Design Example provides an aid in the selection of appropriate helical anchors for pipeline buoyancy control.

ASSUMPTIONS

- Pipe contents: Natural gas
- Pipe Outside Diameter (OD): 42"
- Pipe Wall Thickness (T_w): 0.938"
- Grade of Pipe: API 5L, Grade X65
- Minimum Yield Strength Of Pipe (F_y): 65,000 psi
- Pipe design pressure (P_d): 1,440 psi
- Maximum Operating Pressure (P_m): 1,440 psi
- Maximum Operating Temperature (T_m): 85° F
- Construction type design factor (F): 0.50
- Longitudinal joint factor (E): 1.0
- Temperature Factor (T): 1.0 ($T_m < 250^\circ\text{F}$)
- Coating: Fusion Bonded Epoxy
- Density of coating (D_c): 70.0 pcf
- Coating thickness (T_c): 16 mils
- Pipeline placement: Land Based in Trench with 4'-0" of Cover above Top of Pipe
- Backfill material: Loose, Poorly Graded Silty Sand
- Specific Gravity of Backfill Material: 1.44
- Density of backfill material (D_b) = $1.44 \times 62.4 \text{ pcf} = 89.9 \text{ pcf}$ (use 90.0 pcf)
- Span between anchor sets: Simple Span with Pin-Pin Ends
- Maximum vertical displacement at Mid-Span between Anchor Sets = $L_d/360$
- Minimum Factor of Safety (FS) for Mechanical Strength Of Hardware/Anchors = 2.0
- Minimum Factor of Safety (FS) for Anchor Soil Capacity = 2.0
- Soil data: As shown in Figure 8-23

SOLUTION

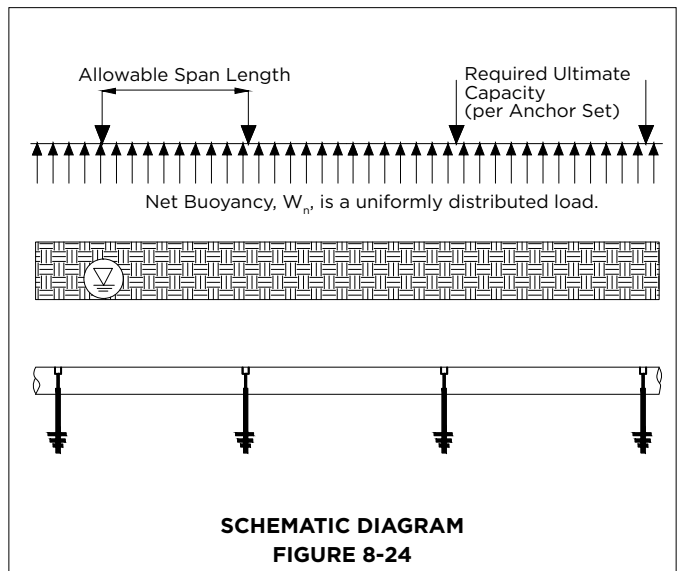
NET BUOYANCY (W_n)

Properties of pipe:

- Weight per linear foot (W_p):

Sample Problem - Natural Gas Pipeline			
Borehole BH-1			
HeliCAP® Software Input Values			
Depth (ft)	Clay Cohesion (psf)	Sand N-Value (SPT)	Soil
0		7	Sand
3		7	Sand
5		28	Sand
7		21	Sand
10		30	Sand
12		21	Sand
13	60		Clay
15	60		Clay
20	380		Clay
25	500		Clay
30	250		Clay
35	460		Clay
40	1250		Clay
45	2000		Clay
50	1560		Clay
55	1250		Clay
60	2250		Clay
65	1320		Clay
70	750		Clay
75	750		Clay

BOREHOLE BH-1 SAMPLE DATA
FIGURE 8-23



**EQUATION 8-60**

$$\begin{aligned} W_p &= [D_s \times \pi \times (42.0^2 - 40.124^2)] / (4 \times 144) \\ &= [490.0 \times \pi \times (1764.0 - 1609.935)] / (576) \\ &= 411.74 \text{ plf} \end{aligned}$$

- Moment of inertia (I) = 25515.8 in⁴
- Section modulus (S) = 0.7032 ft³

Properties of coating:

- Weight per linear foot (W_c):

EQUATION 8-61

$$\begin{aligned} W_c &= [D_c \times \pi \times (42.032^2 - 42.0^2)] / (4 \times 144) \\ &= [70.0 \times \pi \times (42.032^2 - 42.0^2)] / (4 \times 144) \\ &= 1.03 \text{ plf} \end{aligned}$$

Buoyancy:

- Gross buoyancy (W_g):

EQUATION 8-62

$$\begin{aligned} W_g &= [D_b \times \pi \times (42.032^2/12^2)] / 4 \\ &= [90.0 \times \pi \times (42.032^2/12^2) / 4 \\ &= 865.8 \text{ plf} \end{aligned}$$

- Net buoyancy (W_n):

EQUATION 8-63

$$\begin{aligned} W_n &= W_g - W_p - W_c \\ &= 865.8 - 411.74 - 1.03 \\ &= 453.03 \text{ plf (use 453.0 plf)} \end{aligned}$$

Allowable Span Length (L_b) Based on Bending Stress

- Maximum design pressure (P):

EQUATION 8-64

$$\begin{aligned} P &= [(2 \times F_y \times T_w)/OD] \times F \times E \times T \\ &= [(2 \times 65,000 \times 0.938)/42.0] \times 0.5 \times 1.0 \times 1.0 \\ &= 1451.7 \text{ psi (use given } P_d \text{ of 1440.0 psi)} \end{aligned}$$

- Hoop stress (F_h):

EQUATION 8-65

$$\begin{aligned} F_h &= (P_d \times OD)/(2 \times T_w) \\ &= (1440.0 \times 42.0)/(2 \times 0.938) \\ &= 32,238.8 \text{ psi} \end{aligned}$$

- Longitudinal stress (F_l):

EQUATION 8-66

$$\begin{aligned} F_l &= (0.25 \times P_d \times OD)/T_w \\ &= (0.25 \times 1440.0 \times 42.0)/0.938 \\ &= 16,119.4 \text{ psi} \end{aligned}$$

Allowable longitudinal bending stress (F_b):**EQUATION 8-67**

$$\begin{aligned} F_b + F_l &= 0.75 \times (F \times E \times T) \times F_y \\ F_b &= [0.75 \times (0.5 \times 1.0 \times 1.0) \times 65,000] - 16,119.4 \\ &= 8,255.6 \text{ psi} \end{aligned}$$

EQUATION 8-68

$$F_b = M_{\max}/S$$

where

M_{max} = Maximum moment at mid-span between pipeline anchor sets

$$= (W_n \times L_b^2)/8$$

$$L_b = [(8 \times S \times F_b)/W_n]^{1/2}$$

$$= [(8 \times 0.7032 \times 8255.6 \times 144)/453.0]^{1/2}$$

$$= 121.5 \text{ ft}$$

Allowable Span Length (L_d) Based on Mid-Span Deflection

- Mid-span vertical displacement (Y) at center of span:

EQUATION 8-69

$$Y = L_d/360$$

$$L_d/360 = (5 \times W_n \times L_d^4) / (384 \times E \times I)$$

$$L_d = [(384 \times E \times I) / (360 \times 5 \times W_n)]^{1/3}$$

$$L_d = [(384 \times 29,000,000 \times 25525.8/144) / (360 \times 5 \times 453.0)]^{1/3}$$

$$L_d = 134.2 \text{ ft}$$

$$Y = (134.2/360) \times 12 = 4.5 \text{ in}$$

ALLOWABLE SPAN LENGTH (L_p) BASED ON THE MECHANICAL STRENGTH OF PIPELINE BRACKET

- Rated ultimate mechanical strength (UC_p) of pipeline bracket = 80,000 lb
- Rated mechanical working capacity (WC_p) of pipeline bracket (using FS_m of 2.0):

EQUATION 8-70

$$\begin{aligned} WC_p &= UC_p/FS_m \\ &= 80,000/2 \\ &= 40,000 \text{ lb} \end{aligned}$$

EQUATION 8-71

$$\begin{aligned} WC_p &= (W_n \times L_p/2) \times 2 \\ L_p &= WC_p/W_n \\ &= 40,000/453.0 \\ &= 88.3 \text{ ft} \end{aligned}$$

ALLOWABLE SPAN LENGTH (L_a) BASED ON THE UPLIFT CAPACITY OF ANCHORS IN SOIL (BORING B-1)

- Ultimate uplift capacity (UC_a) ranges from 45,900 to 41,700 lb with overall anchor depths below ground line of 51'-0 to 60'-0. See Figure 8-25. Use UC_a = 40,000 lb.
- Working uplift capacity (WC_a) (using FS_s of 2.0):

EQUATION 8-72

$$\begin{aligned} WC_a &= UC_a/FS_s \\ &= 40,000/2 \\ &= 20,000 \text{ lb} \end{aligned}$$

- There are two anchors located at each anchor support location along the pipeline, therefore, the total working uplift capacity (WC_s) per anchor set = WC_a × 2 anchors = 20,000 × 2 = 40,000 lb.

**EQUATION 8-73**

$$\begin{aligned}
 L_a &= WC_s/W_n \\
 &= 40,000/453.0 \\
 &= 88.3 \text{ ft}
 \end{aligned}$$

SUMMARY

The uplift capacity plot data was obtained from the soil strength parameters shown in Figure 8-23 and capacities generated by HeliCAP® engineering software. The maximum span length between anchor sets is limited to 88 ft based on the ultimate mechanical strength of the pipeline brackets and the ultimate uplift capacity of the anchors in the soil boring shown in Figure 8-25.

Only one soil boring was provided along this proposed section of pipeline. If the soil conditions vary at the anchor set locations and the required average installation torque of 4,000 ft·lb for a span length of 88 ft cannot be achieved at reasonable anchor depths, the span lengths should be reduced as shown in Table 8-8.

Hubbell manufactures two band types for use with pipeline buoyancy control systems. See Figure 8-26. Each system has advantages depending on the application and local acceptance. Both systems will provide adequate buoyancy control with industry accepted Factors of Safety.

SPAN REDUCTION SCHEDULE, TABLE 8-8

SPAN LENGTH (ft)	REQUIRED UC _s PER ANCHOR SET (lb)	REQUIRED UC _a PER ANCHOR (lb)	MINIMUM INSTALLATION TORQUE (ft·lb)
88	80,000	40,000	4,000
77	70,000	35,000	3,500
66	60,000	30,000	3,000
55	50,000	25,000	2,500
44	40,000	20,000	2,000

SUMMARY OF DESIGN CRITERIA, TABLE 8-7

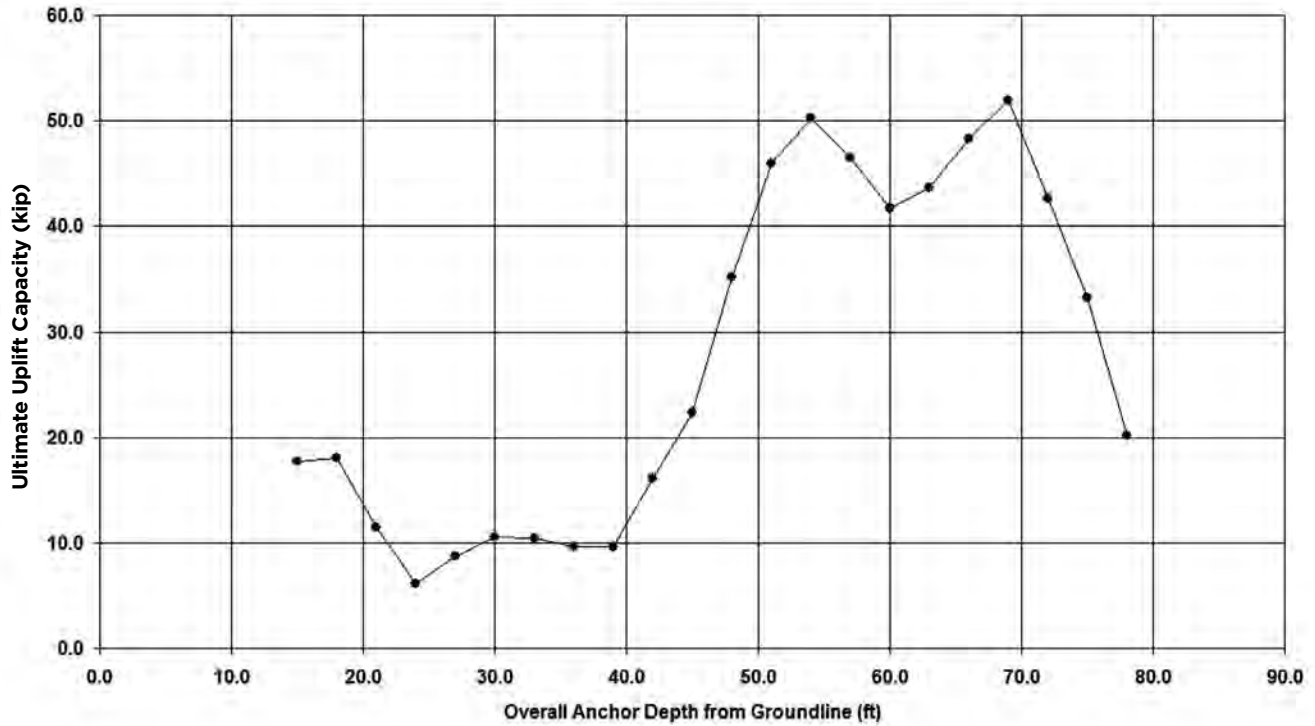
	MAXIMUM ALLOWABLE SPAN LENGTH (ft)	REQUIRED UC _s PER ANCHOR SET (lb) ²	REQUIRED UC _a PER ANCHOR SET (lb) ²	MINIMUM INSTALLATION TORQUE (ft·lb) ^{1,2}
Longitudinal Bending	121.5	110,080	55,040	5,500
Mid-Span Deflection	134.2	121,585	60,793	6,100
Mechanical Strength of Bracket	88.3	80,000	40,000	4,000
Anchor Capacity	88.3	80,000	40,000	4,000

Notes:

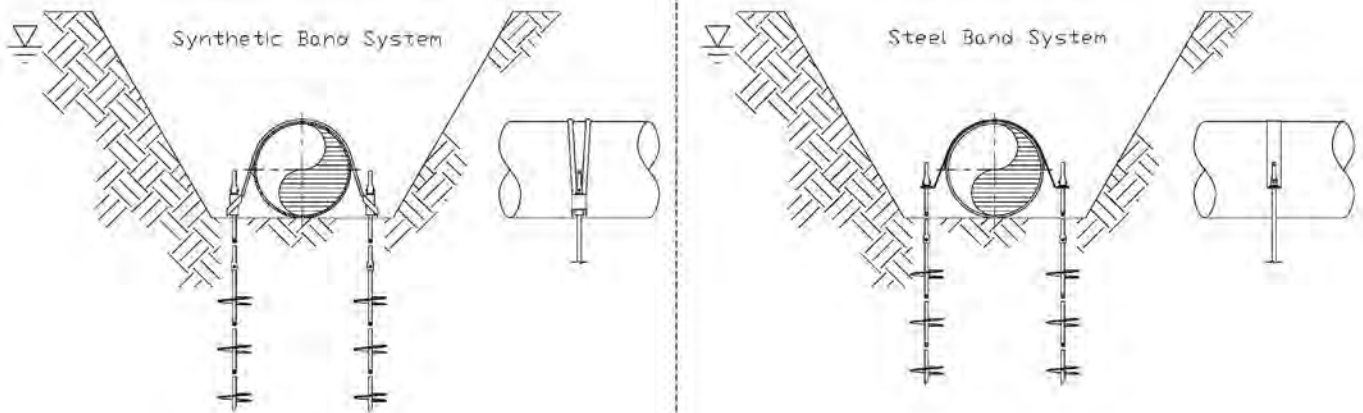
1. The required average minimum installation torque is based on using the published installation torque to ultimate capacity ratio (K_t) of 10:1 for the Type SS series anchor material. Torque = UC_a/K_t.
2. These values include a minimum acceptable industry standard Factor of Safety of 2 for helical anchors/piles when used in permanent applications. These pipeline anchors are considered by Hubbell Power Systems, Inc. to be a permanent application. If the client or their representative opts to use a lower Factor of Safety these values will have to be reduced accordingly. For example, at a span length of 88.3 ft, the working capacity per anchor set is 453.0 plf x 88.3 ft = 40,000 lb. Applying an FS of only 1.5, the required UC_s is 1.5 x 40,000 = 60,000 lb. The required UC_a is 60,000 lb/2 anchors = 30,000 lb. The required minimum installation torque is 30,000/10 = 3,000 ft·lb.

Sample Problem - Natural Gas Pipeline Project

Ultimate Uplift Capacity using a 14",14",14" Helical Lead Section - Boring B-1



ULTIMATE UPLIFT CAPACITY
FIGURE 8-25



BAND SYSTEMS
FIGURE 8-26

DESIGN EXAMPLE 13: TYPE RS HELICAL PILES FOR LATERAL SUPPORT

PROBLEM

A Chance® Type SS175 1-3/4" square shaft helical anchor/pile is proposed for a pedestrian bridge abutment. The top section of the shaft is to be encased in a 6" nominal steel pipe and grout to provide lateral resistance. The top ten feet of the soil profile is medium-stiff clay with a cohesion factor (c) of 1000 psf. Determine what length of 6" diameter steel case is required to resist 4400 lb of lateral load using the Broms' Method.

ASSUMPTIONS

- The 1-3/4" square shaft below the 6" cased section provides no lateral resistance.
- The solution method used is shown in Figure 8-27.
- Eccentricity is assumed to be 1 ft

SOLUTION

- P = Applied horizontal shear load: Use 4400 lb. Include a Factor of Safety of 2 in the calculations, thus doubling the horizontal shear load; $P = 2 \times 4400 = 8800$ lb
- Cu = Cohesion of clay: Use Cu = 1000 psf
- d = Diameter of foundation: Use d = 6.625" (6" nominal pipe size)
- e = Eccentricity; distance above grade to resolved load: Use e = 1 ft
- L = Minimum length of foundation based on above criteria.

EQUATION 8-74

$$\begin{aligned} f &= P/9 (Cu) d \\ &= 8800 \text{ lb}/9 (1000 \text{ psf}) (6.625 \text{ in}/12) \\ &= 1.771 \text{ ft} \end{aligned}$$

EQUATION 8-75

$$\begin{aligned} M_{\text{MAX}}^{\text{POS}} &= P [e + 1.5(d) + 0.5(f)] \\ &= 8800 \text{ lb} [1 \text{ ft} + 1.5 (6.625 \text{ in}/12) + 0.5 (1.771 \text{ ft})] \\ &= 23,880 \text{ ft}\cdot\text{lb} \end{aligned}$$

EQUATION 8-76

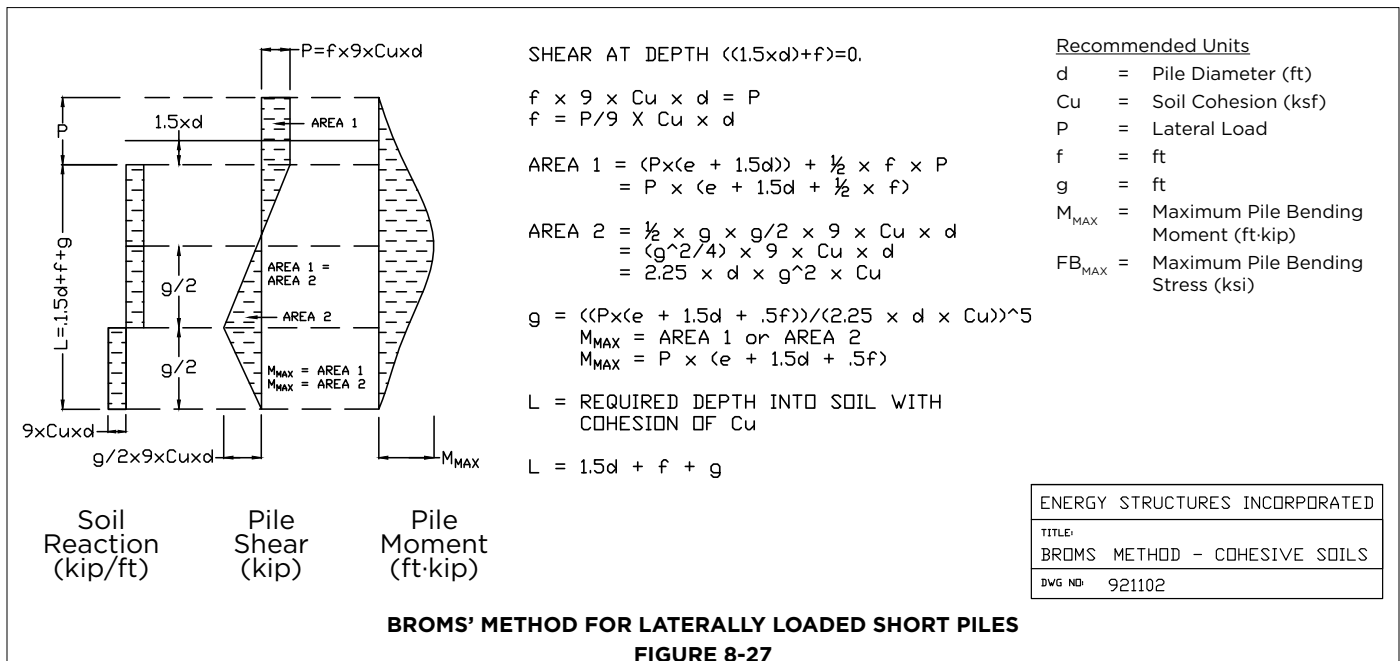
$$\begin{aligned} M_{\text{MAX}}^{\text{POS}} &= 2.25 (d) g^2 (Cu) \\ 23,880 \text{ ft}\cdot\text{lb} &= 2.25 (6.625 \text{ in}/12) g^2 (1000 \text{ psf}) \\ g^2 &= 19.22 \text{ ft}^2 \\ g &= \sqrt{19.22} \\ &= 4.38 \text{ ft} \end{aligned}$$

EQUATION 8-77

$$\begin{aligned} L &= 1.5d + f + g \\ &= 1.5 (6.625 \text{ in}/12) + 1.771 \text{ ft} + 4.38 \text{ ft} \\ &= 6.98 \text{ ft} \end{aligned}$$

SUMMARY

The 6" nominal steel case should be at least 7'-0" long to resist the 4400 lb lateral load.





DESIGN EXAMPLE 14: INSTANT FOUNDATIONS FOR STREET LIGHT SUPPORTS

PURPOSE

This Design Example provides example solutions to aid in the selection of appropriate Chance® Instant Foundation® products, also known as street light foundations (SLF), for different job parameters.

SLF LOADS

The resulting pole loads to be resisted by a street light foundation are dead or vertical down loads (DL), horizontal, lateral or shear loads (V) due to wind on the pole and luminaire (light fixture), and overturning moment loads (M) resulting from the tendency to bend at or near the ground line as the wind causes the pole to displace and the foundation restrains the pole base at one location (see Figure 8-28).

The DL for an SLF application is so small that a foundation sized to resist V and M will typically be much more than adequate to resist DL. Therefore, DL will not control the SLF design and will not be considered here. If DL is large enough to be of concern for an application where an SLF will be used, it may be evaluated based on bearing capacity equations applied to the soil around the helical bearing plate and friction along the shaft. These evaluations are beyond the scope of this design example, which will only deal with SLF applications.

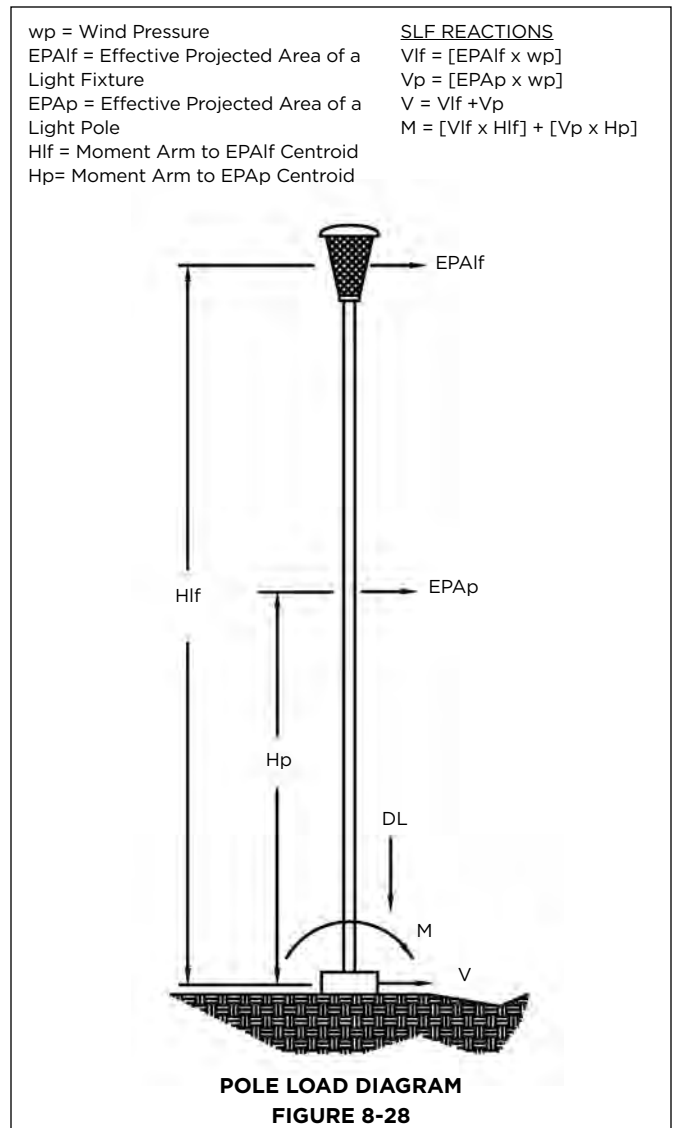
Since SLF products are used as lighting foundations along public highways, it is appropriate to mention the American Association of State Highway and Transportation Officials (AASHTO) publication Standard Specifications for Structural Support for Highway Signs, Luminaires and Traffic Signals. This document is often taken as the controlling specification for jobs using SLF's and will be referenced throughout this discussion.

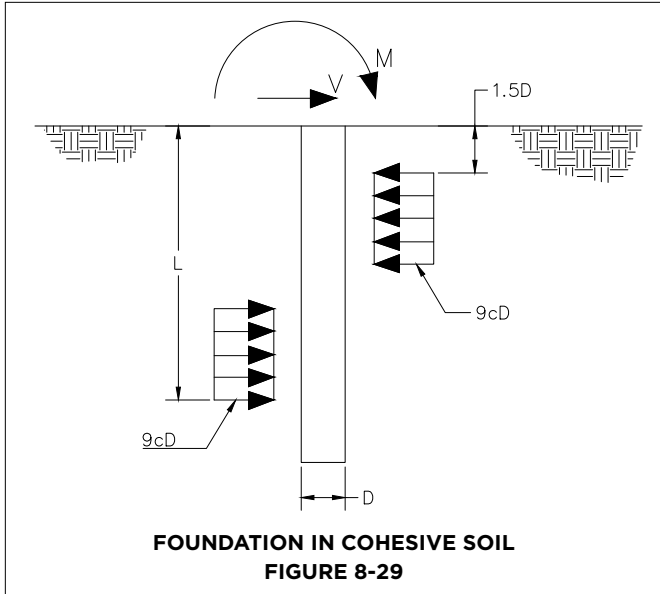
SLF SELECTION

The SLF selection process is a trial and error procedure that may require more than one iteration. First, select an SLF diameter based on the applied bending moment (M) that must be resisted. That is, ensure that the applied moment is less than the allowable moment on the shaft. Determining the allowable moment requires a structural analysis of the pipe shaft section capacities (often based on a reduced cross section through cable ways, bolt slots, base plate size, welds, etc). This effort should be familiar to engineers engaged in design work, so a sample of this process will not be given here.

The foundation shaft diameter will often be as large as or larger than the base diameter of the pole to be supported. Allowable moment capacities for Chance Instant Foundation products are provided in Table 10-2 in Section 10 of this manual. These capacities, when compared to the ground line reactions of the pole, can be used to choose a starting diameter to resist the applied loads. In this regard, shear is usually not the controlling factor for SLF shaft size but rather the moment load. (Note: The starting size may change as the given soil conditions for a job may dictate the final SLF size required.)

The design or selection of a foundation size to resist light pole loads in a given soil may be determined by various methods. Numerical methods using finite-element and finite-difference techniques may be used but have proven to be somewhat sophisticated for the rather simple SLF application. The Fourth Edition of the AASHTO specification lists a number of preliminary design methods that can be employed in the design process. Among those listed and discussed are the methods developed by Bengt B. Broms for embedment lengths in cohesive and cohesionless soils and a graphical method dealing with the embedment of lightly loaded poles and posts. The Broms' Method will be used for this design example as experience has shown these methods to both usable and appropriate. Calculations are provided for both cohesive soil (clay) and cohesionless soil (sand).





COHESIVE SOIL (SEE FIGURE 8-29)

Assumed values:

- Applied shear load at the groundline (V) = 460 lb
- Applied moment at the groundline (M) = 8600 ft·lb
- Foundation diameter is 6" nominal Schedule 40. Use 6.625" as the actual pipe size in calculations. Cableway openings are 2.5" wide by 12" high. The allowable moment capacity of this foundation shaft size and cableway opening is 10,860 ft·lb
- The required length (L) will be determined using the Broms' method.
- Cohesion (c) = 1000 psf
- Factor of Safety = 2

EQUATION 8-78

$$V_F = V (FS)$$

$$= 460 (2)$$

$$= 920 \text{ lb}$$

EQUATION 8-79

$$V_M = M (FS)$$

$$= 8600 (2)$$

$$= 17,200 \text{ ft·lb}$$

EQUATION 8-80

$$L = 1.5D + q [1 + \{ 2 + (4H + 6D)/q \}^{0.5}]$$

$$= 1.5 (6.625/12) + 0.185157 \times [1 + \{ 2 + (4 \times 18.69565 + 6 \times (6.625/12)) / (0.185157) \}^{0.5}]$$

$$= 4.82 \text{ ft}$$

where

- D = Diameter of foundation = 6.625 inches
- q = $V_F / 9cD = 920 / (9 \times 1000 \times 6.625/12) = 0.185157$ ft
- c = Shear strength of cohesive soil = 1000 psf

$$H = \text{Moment} / \text{Shear} = M/V = V_M / V_F = 17200 \text{ ft·lb} / 920 \text{ lb} = 18.69565$$

L = Calculated Foundation Length to Provide a SF of 2 Against Soil Failure.

The length required to provide a Factor of Safety of 2 against soil failure is 4.82 ft. Since SLF lengths are provided in even foot lengths, use L = 5 ft. For the required embedment length, the maximum moment in the shaft is:

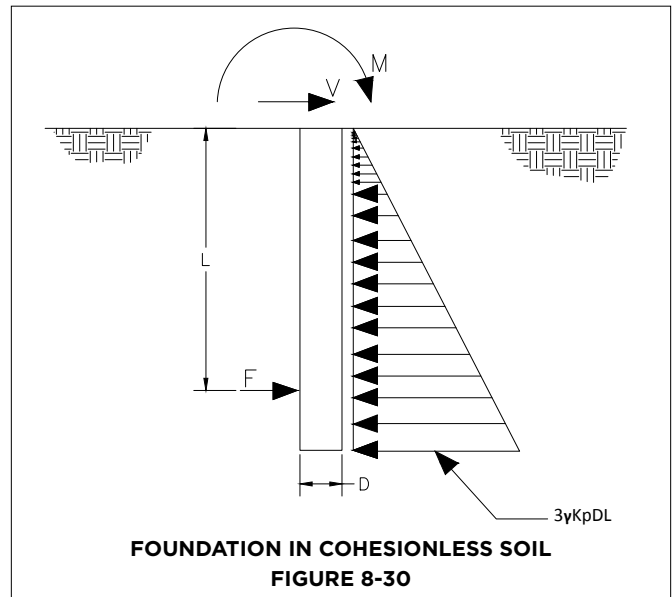
EQUATION 8-81

$$M_{MAX} = V (H + 1.5D + 0.5q)$$

$$= 460 (18.69565 + (1.5 \times 6.625/12) + (0.5 \times 0.185157))$$

$$= 9023.5 \text{ ft·lb}$$

Maximum moment can be compared with the allowable moment capacity of the foundation shaft to determine adequacy. For this example the allowable moment in the 6" pipe shaft is given as 10,860 ft·lb, which is greater than the applied moment. Therefore, the 6" diameter by 5' long SLF is adequate for the applied loads in the clay soil.



COHESIONLESS SOIL (SEE FIGURE 8-30)

Assumed values:

- Applied shear load at the groundline (V) = 460 lb
- Applied moment at the groundline (M) = 8600 ft·lb
- Foundation diameter is 6" nominal Schedule 40. Use 6.625" as the actual pipe size in calculations. Cableway openings are 2.5" wide by 12" high. The allowable moment capacity of this foundation shaft size and cableway opening is 10,860 ft·lb.
- The required length (L) will be determined using the Broms' method.
- $\phi = 30^\circ$
- $\gamma = 100 \text{ lb/ft}^3$
- Factor of Safety = 2.

**EQUATION 8-78**

$$\begin{aligned}
 V_F &= V (FS) \\
 &= 460 (2) \\
 &= 920 \text{ lb}
 \end{aligned}$$

EQUATION 8-79

$$\begin{aligned}
 V_M &= M (FS) \\
 &= 8600 (2) \\
 &= 17,200 \text{ ft}\cdot\text{lb}
 \end{aligned}$$

Broms' equation for cohesionless soil requires a trial and error solution. For the trial and error solution, start by assuming the foundation diameter (D) is 6.625" and the length (L) is 6 feet:

EQUATION 8-82

$$\begin{aligned}
 0 &\leq L^3 - (2V_F L / K_p \gamma D) - (2V_M / K_p \gamma D) \\
 &= 6^3 - [2 \times 920 \times 6 / (3 \times 100 \{6.625/12\})] - [(2 \times 17200) / (3 \times 100 \times \{6.625/12\})] \\
 &= -58.35
 \end{aligned}$$

where

$$\begin{aligned}
 0 &> -58.35 \\
 K_p &= \tan^2(45 + \phi/2) = 3.0 \\
 \gamma &= \text{Effective unit weight of soil} = 100 \text{ lb/ft}^3
 \end{aligned}$$

The 6 foot length is too short so we will try a 7 foot length and repeat the calculation:

$$\begin{aligned}
 0 &= 7^3 - [2 \times 920 \times 7 / (3 \times 100 \{6.625/12\})] - [(2 \times 17200) / (3 \times 100 \times \{6.625/12\})] \\
 &= 57.53 \\
 0 &< 57.53
 \end{aligned}$$

A 7 foot long SLF will be adequate. The maximum moment in the foundation shaft can be determined with the following equation:

EQUATION 8-83

$$\begin{aligned}
 M_{MAX} &= V (H + 0.54 \times (V / \gamma D K_p)^{0.5}) \\
 &= 460 (18.69565 + 0.54 \times (460/100 \times (6.625/12) \times 3)^{0.5}) \\
 &= 9013.968 \text{ ft}\cdot\text{lb}
 \end{aligned}$$

This is less than the allowable moment capacity of 10,860 ft·lb, therefore a 6" diameter by 7' long SLF is adequate for the applied load in the sandy soil.

DESIGN EXAMPLE 15: FOUNDATION EARTH PRESSURE RESISTANCE

PROJECT

A Chance® Type SS5 1-1/2" square shaft helical anchor is proposed as part of a pier and beam foundation for a residential structure (see Figure 8-31). The top of the helical anchor is fixed in a concrete grade beam that extends 4'-0" below grade. The surface soils are loose sands. Determine the lateral capacity of the grade beam using the Rankine earth pressure method.

ASSUMPTIONS

- The lateral capacity of the 1-1/2" square shaft helical anchor is limited based on shaft size. It is generally not assigned any contribution to the lateral capacity of a foundation
- The effective length of the grade beam for lateral resistance is 25'-0"
- Assume a unit weight of 95 pcf
- The water table is well below the the bottom of the grade beam
- There are no surcharge loads
- From Table 8-9, $K_a = 0.2$, $K_p = 3$

SOLUTION

EQUATION 8-84

$$P_a = 0.5K_a\gamma H^2$$

$$= 0.5 \times 0.2 \times 95 \times 4^2$$

$$= 152 \text{ lb/ft}$$

$$P_p = 0.5K_p\gamma H^2$$

$$= 0.5 \times 3 \times 95 \times 4^2$$

$$= 2280 \text{ lb/ft}$$

$$P_p - P_a = 2280 - 152$$

$$= 2128 \text{ lb/ft}$$

$$\text{Total lateral resistance} = 2128 \times 25'-0" = 53,200 \text{ lb}$$

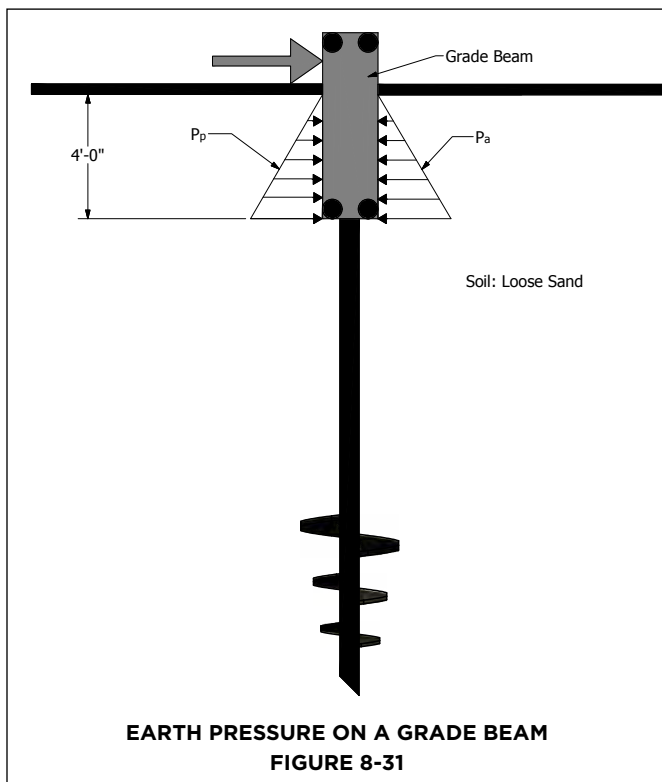
NOTE: In this example, more than 1" of movement will probably be required to fully mobilize the total lateral resistance. Partial mobilization requires less deflection.

COEFFICIENTS OF EARTH PRESSURE (DAS, 1987), TABLE 8-9

SOIL	K_o' DRAINED	K_o' TOTAL	K_a' TOTAL	K_p' TOTAL
Clay, soft ¹	0.6	1	1	1
Clay, hard ¹	0.5	0.8	1	1
Sand, loose	0.6	0.53	0.2	3
Sand, dense	0.4	0.35	0.3	4.6

Note:

¹ Assume saturated clays.





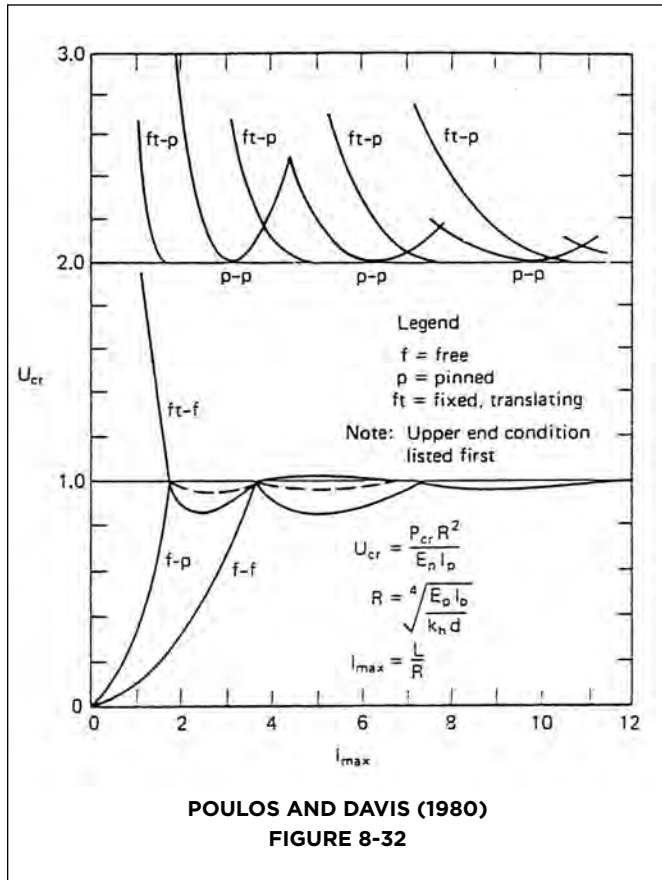
DESIGN EXAMPLE 16: BUCKLING EXAMPLE USING THE DAVISSON METHOD

PROJECT

A three-helix Chance® Type SS150 1-1/2" square shaft helical pile is to be installed into the soil profile as shown in Figure 8-33. The top three feet is uncontrolled fill and is assumed to be soft clay. The majority of the shaft length (12 feet) is confined by soft clay with a $k_h = 15$ pci. The helix plates will be located in stiff clay below 15 feet. The buckling model assumes a pinned-pinned end condition for the helical pile head and tip. Determine the critical buckling load using the Davisson method.

ASSUMPTIONS

- k_h is constant, i.e., it does not vary with depth. This is a conservative assumption because k_h usually varies with depth, and in most cases increases with depth.
- Pinned-pinned end conditions are assumed. In reality, end conditions are more nearly fixed than pinned, thus the results are generally conservative.
- From Figure 8-32, $U_{cr} \approx 2$



EQUATION 8-85

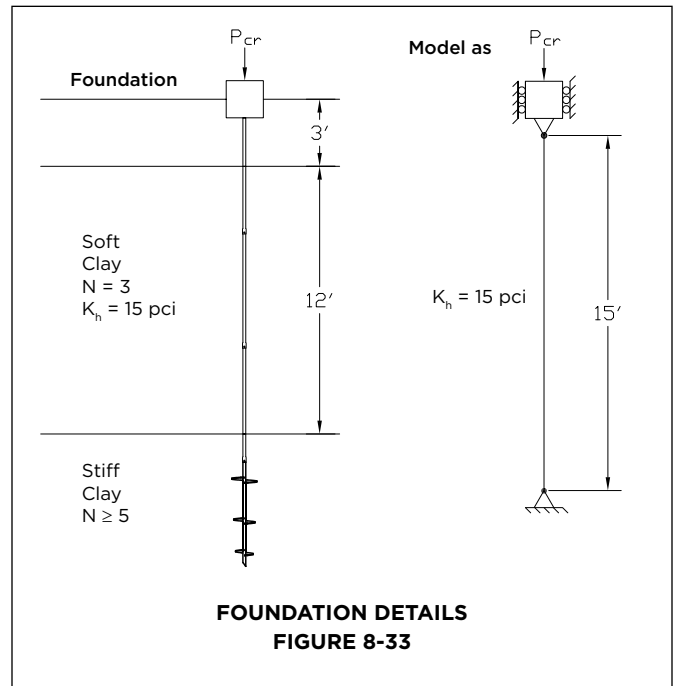
$$R = 4\sqrt{(30 \times 10^6 \times 0.396) / (15 \times 1.5)} = 26.96$$

$$I_{max} = (15 \times 12) / 26.96 = 6.7$$

$$P_{cr} = (2 \times 30 \times 106 \times 0.396) / 26.96^2 = 32.69 \text{ kip}$$

CHANCE TYPE SS150 SQUARE SHAFT FOUNDATIONS PHYSICAL PROPERTIES, TABLE 8-10

MODULUS of ELASTICITY (E_p)	MOMENT of INERTIA (I_p)	SHAFT DIAMETER (D)
30×10^6 psi	0.396 in^4	1.5 in



DESIGN EXAMPLE 17: BUCKLING EXAMPLE USING THE FINITE-DIFFERENCE METHOD

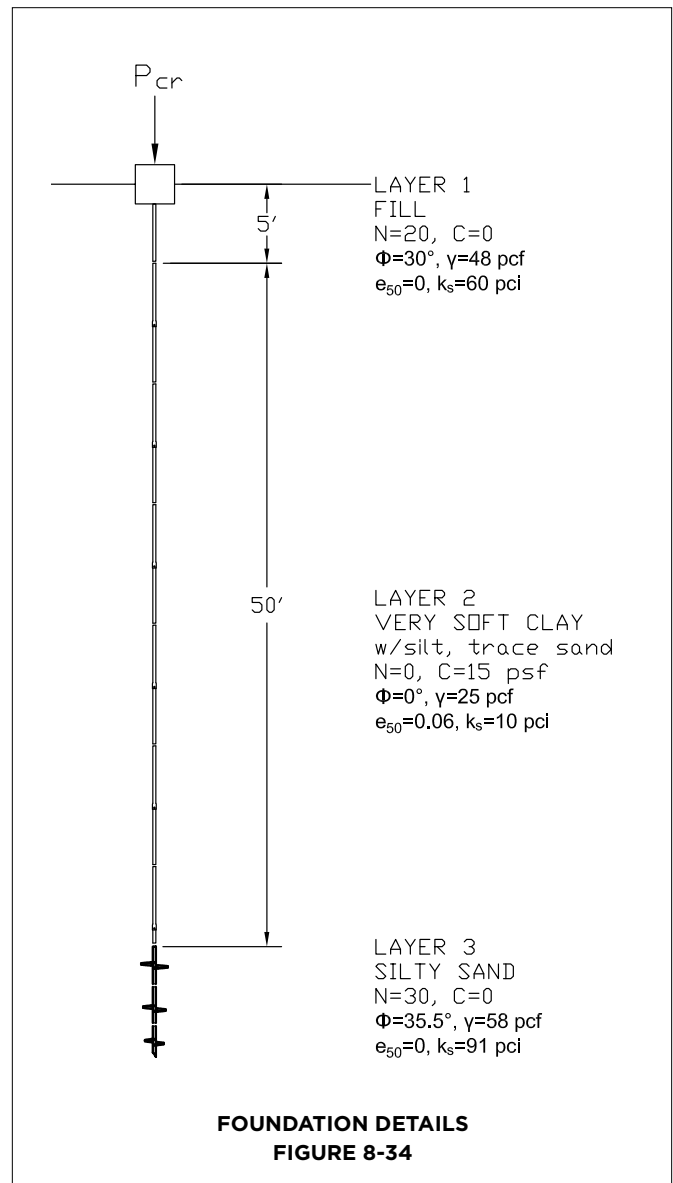
A four-helix Chance® helical pile is to be installed into the soil profile as shown in Figure 8-34. The top five feet is compacted granular fill and is considered adequate to support lightly loaded slabs and shallow foundations. The majority of the shaft length (50 feet) is confined by very soft clay described by the borings as “weight of hammer” (WOH) or “weight of rod” (WOR) material. WOH or WOR material means the weight of the 130 lb drop hammer or the weight of the drill rod used to extend the sampler down the borehole during the standard penetration test is enough to push the sampler down 18+ inches. As a result, a low cohesion value (15 psf) is assumed. The helix plates will be located in dense sand below 55 feet. Determine the critical buckling load of a Type SS175 1-3/4” square shaft and Type RS3500.300 round shaft piles using LPILE Plus 3.0 for Windows (ENSOFT, Austin, TX).

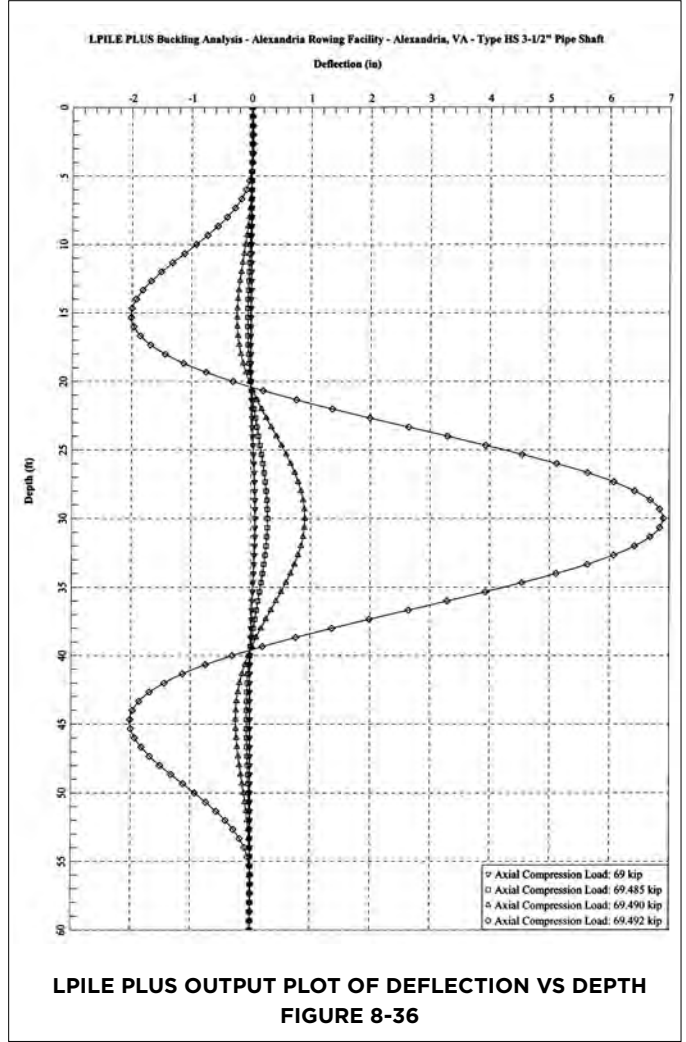
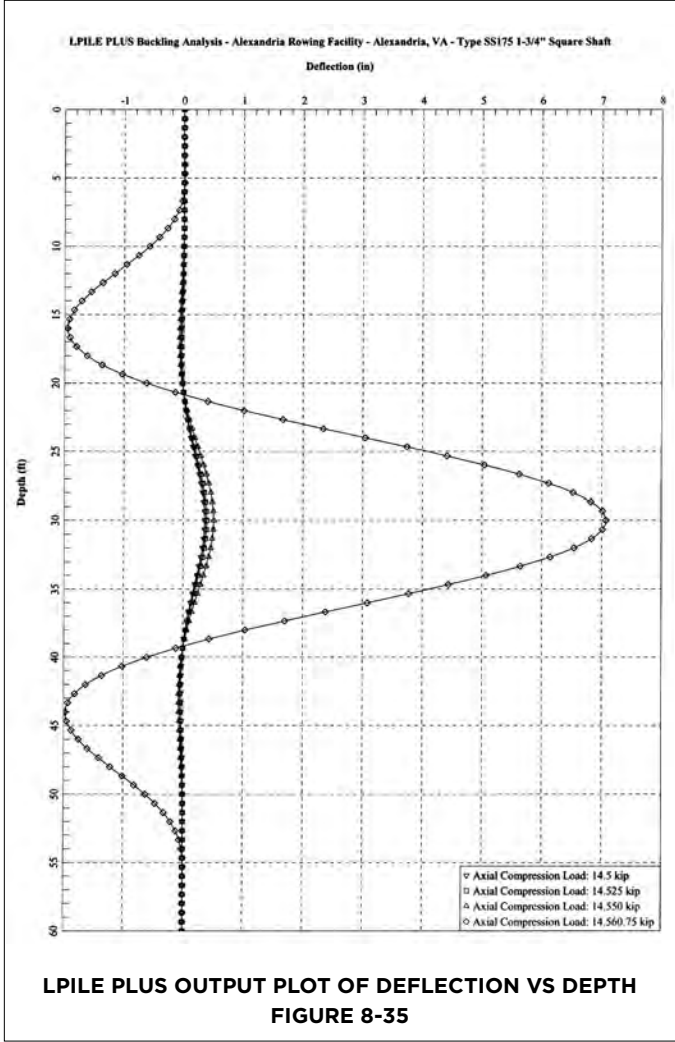
When the computer model is completed, the solution becomes an iterative process of applying successively increasing loads until a physically logical solution converges. At or near the critical buckling load, very small increasing increments of axial load will result in significant changes in lateral deflection - which is a good indication of elastic buckling. Figure 8-35 is an LPILE Plus output plot of lateral shaft deflection vs depth. As can be seen by the plot, an axial load of 14,561 lb is the critical buckling load for a Type SS175 1-3/4” square shaft because of the dramatic increase in lateral deflection at that load compared to previous lesser loads. Figure 8-36 indicates a critical buckling load of 69,492 lb for Type RS3500.300 round shaft.

Note that over the same 50-foot length of very soft clay, the well-known Euler equation predicts a critical buckling load for Type SS175 of 614 lb with pinned-pinned end conditions and 2,454 lb with fixed-fixed end conditions. The Euler critical buckling load for Type RS3500.300 is 3,200 lb for pinned-pinned and 12,800 lb for fixed-fixed. This is a good indication that shaft confinement provided by the soil will significantly increase the buckling load of helical piles. This also indicates that even the softest materials will provide significant resistance to buckling.

All extendable helical piles have couplings or joints used to connect succeeding sections together in order to install the helix plates in bearing soil. One inherent disadvantage of using the finite-difference method is its inability to model the effects of bolted couplings or joints that have zero joint stiffness until the coupling rotates enough to bring the shaft sides into contact with the coupling walls. This is analogous to saying the coupling or joint acts as a pin connection until it has rotated a specific amount, after which it acts as a rigid element with some flexural stiffness. All bolted couplings or joints, including square shaft and round shaft piles, have a certain amount of rotational tolerance. This means the joint initially has no stiffness until it has rotated enough to act as a rigid element. In these cases, it is probably better to conduct buckling analysis using other means, such as finite-element analysis, or other methods based on empirical experience as mentioned earlier.

If couplings are completely rigid, i.e., exhibit some flexural stiffness even at zero joint rotation, axial load is transferred without the effects of a pin connection, and the finite-difference method can be used. An easy way to accomplish rigid couplings with round shaft piles is to pour concrete or grout down the ID of the pipe after installation. Another method is to install a grout column around the square or round shaft of the foundation using the Chance Helical Pulldown® micropile (HPM) method. The HPM is an installation method initially developed to install helical anchor foundations in very weak soils where buckling may be anticipated.

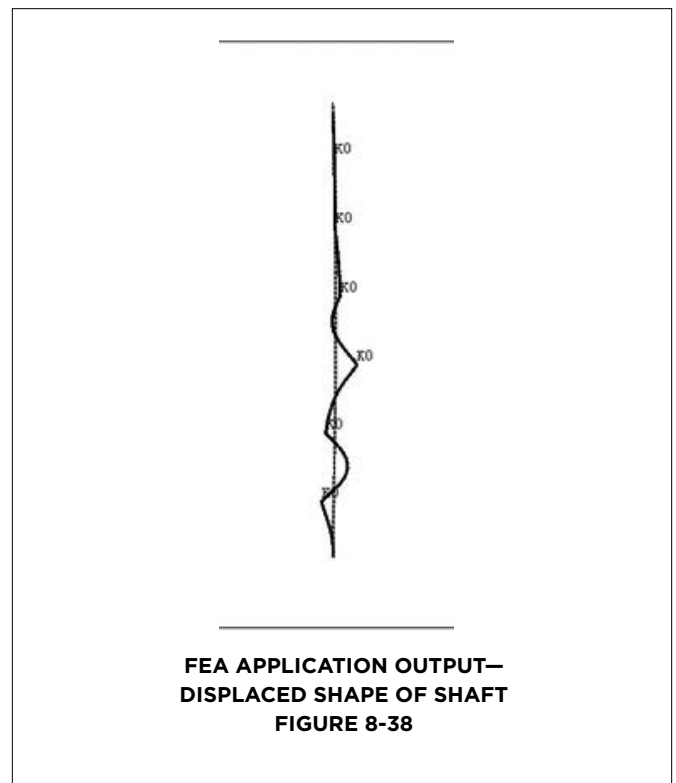
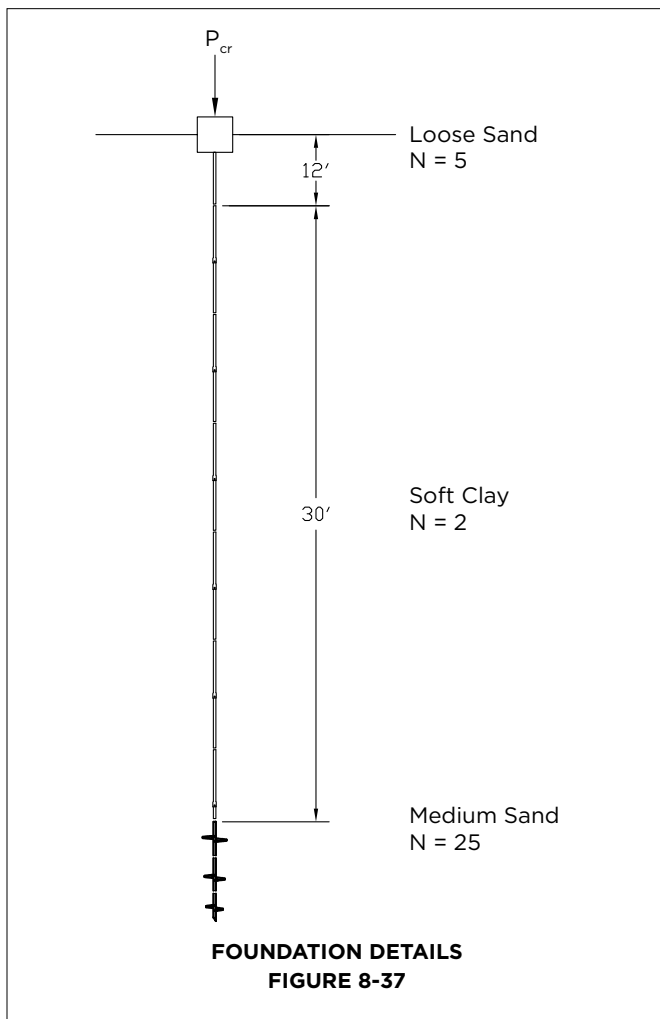




DESIGN EXAMPLE 18: BUCKLING EXAMPLE USING THE FINITE-DIFFERENCE METHOD

A three-helix Chance® Type SS5 1-1/2" square shaft helical pile is to be used to underpin an existing townhouse structure that has experienced settlement (see Figure 8-37 for soil profile details). The top 12 feet is loose sand fill, which probably contributed to the settlement problem. The majority of the shaft length (30 feet) is confined by very soft clay with an SPT blow count "N" of 2. As a result, a cohesion value (250 psf) is assumed. The helix plates will be located in medium-dense sand below 42 feet. Determine the critical buckling load using the finite-element application with integrated FEA software from ANSYS, Inc.

Output indicates the Type SS5 1-1/2" square shaft buckled at around 28 kip. Figure 8-38 shows the displaced shape of the shaft (exaggerated for clarity). The "K0" in Figure 8-38 are the locations of the shaft couplings. Note that the deflection response is controlled by the couplings, as would be expected. Also note that the shaft deflection occurs in the very soft clay above the medium-dense bearing stratum. Since the 28 kip buckling load is considerably less than the bearing capacity (55+ kip) it is recommended to install a grout column around the 1-1/2" square shaft using the Chance Helical Pulldown® micropile (HPM) method.





DESIGN EXAMPLE 19: MONOPOLE FOUNDATION WITH STEEL GRILLAGE AND RS5500 HELICAL PILES

DESIGN EXAMPLES

Provided Design Loads (on entire grillage)

$F_x = 34.06$ kip (axial)

$F_y = 49.41$ kip (shear)

$M_z = -35574$ kip-in (moment)

Reveal Height: 5.33 feet

These loads require a moment foundation that incorporates multiple helical piles into a steel grillage. Grillages that are mostly moment foundations will be limited in size by what can be transported to the job site. Moment foundations will also have helical piles installed at 5 degrees from vertical. This angle allows the piles to be spaced far enough apart at depth

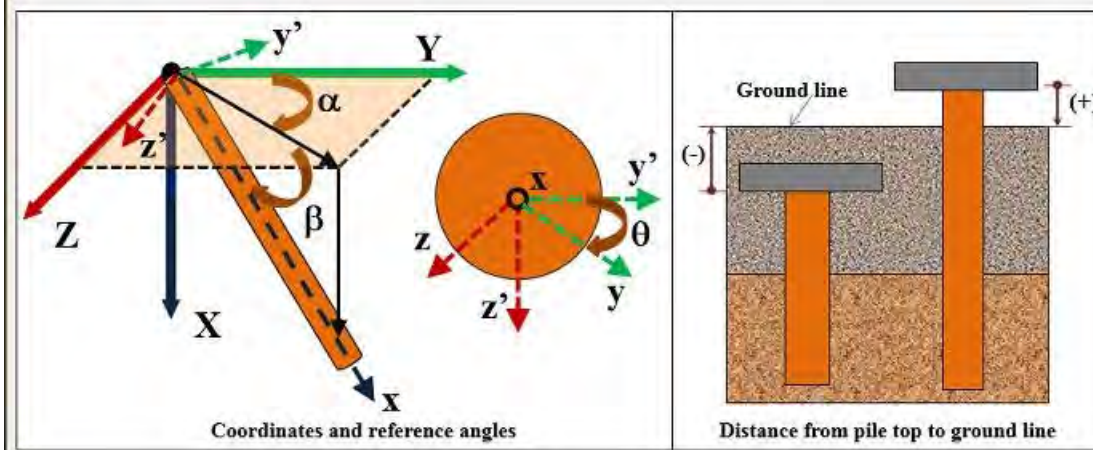
that the helices behave independently. The 5-degree angle also provides a better response to shear load. The first stage is an iterative solution using the design loads from the monopole with the quantity and coordinates of helical piles with Ensoft's GROUP software. Experience provides best practices to determine the number and coordinates of the helical piles. With these inputs, GROUP software will determine the actual loads on each pile. A model of the grillage can then be developed and analyzed with finite-element analysis (FEA) software to help size structural members within the grillage. With the loads applied to each pile obtained from the GROUP software, Chance® HeliCAP® software is used to design the helical piles. The following information is by no means a complete tutorial on the use of GROUP or HeliCAP software but is provided as an example of this type of solution.

GROUP SOFTWARE INPUTS

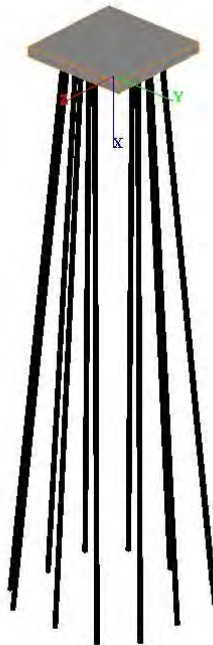
Concentrated	F_x	F_y	M_z	F_z	M_y	M_x	X_c	Y_c	Z_c
Load	(kips)	(kips)	(kips-in)	(kips)	(kips-in)	(kips-in)	(ft)	(ft)	(ft)
Edit Load 1	34.06	49.41	-35574	0	0	0	0	0	0

File #	Pile-Top Vertical	Pile-Top Horizontal	Pile-Top Horizontal	Angle Alpha* (use 0	Angle Beta** (use 90	Angle Theta	Vert. Dist. from Pile Top
	X-Coordinates, (ft)	Y-Coordinates, (ft)	Z-Coordinates, (ft)	for vertical pile) (DEG)	for vertical pile) (DEG)	(section rotation) (DEG)	to Ground Line***, (ft)
1	0	-1.5	-4	-135	85	0	5.33
2	0	1.5	-4	-45	85	0	5.33
3	0	-4	-1.5	-135	85	0	5.33
4	0	4	-1.5	-45	85	0	5.33
5	0	-4	1.5	135	85	0	5.33
6	0	4	1.5	45	85	0	5.33
7	0	-1.5	4	135	85	0	5.33
8	0	1.5	4	45	85	0	5.33
9	0	0	-4	-90	86.5	0	5.33
10	0	-4	0	180	86.5	0	5.33

* Angle Alpha, angle from Y[+] Axis to the projection of pile on the Y-Z plane (use 0 for vertical pile)
 ** Angle Beta, angle from the projection of pile on the Y-Z plane to the pile axis (use 90 for vertical pile)
 *** Distance from pile top to ground line
 (+) positive if soil ground line is below the pile top
 (-) negative if soil ground line is above the pile top

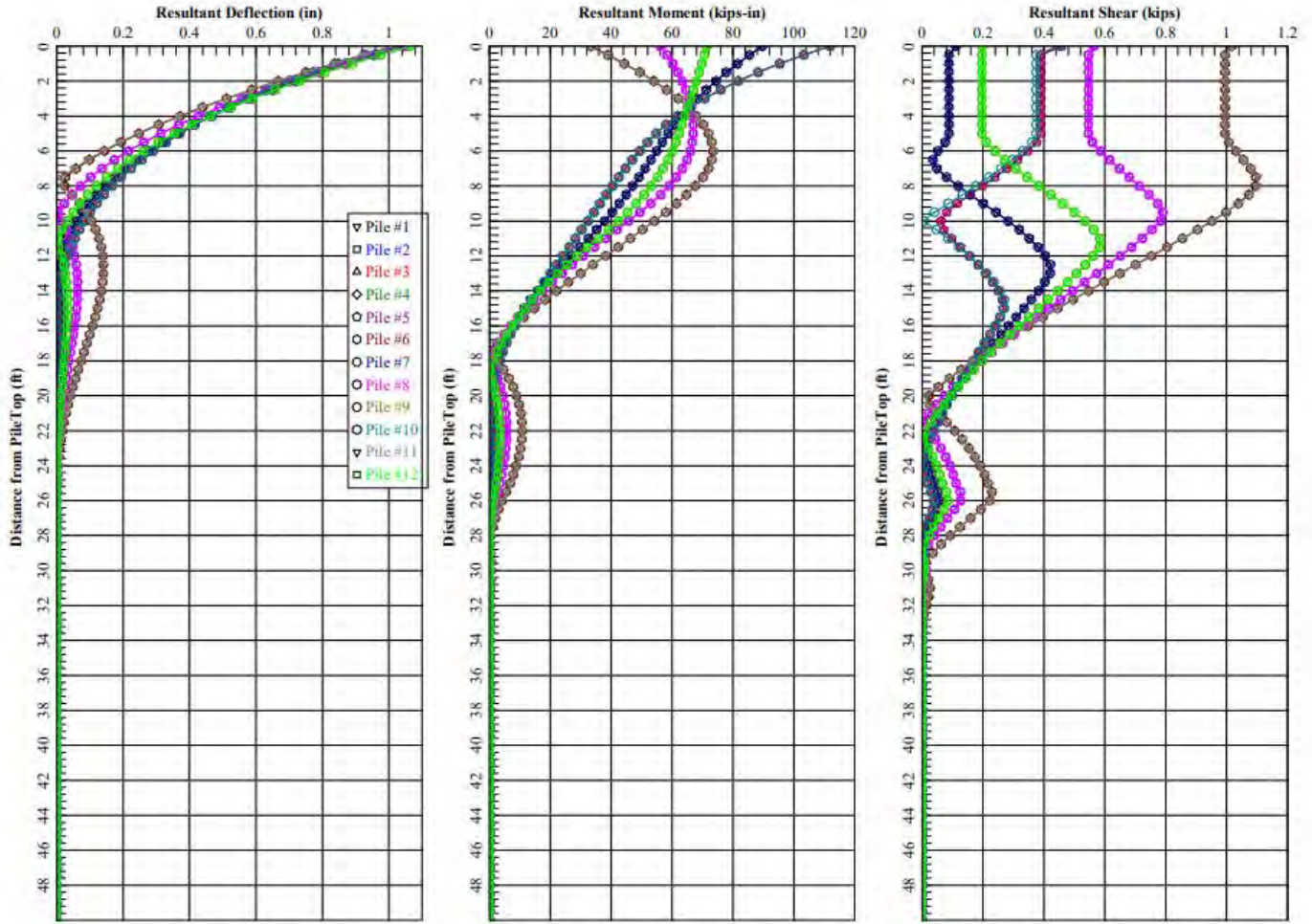


GROUP SOFTWARE OUTPUTS



*** PILE TOP REACTIONS ***

PILE GROUP	FOR. X, KIP	FOR. Y, KIP	FOR. Z, KIP	MOM X, KIP-IN	MOM Y, KIP-IN	MOM Z, KIP-IN
1	-52.110	3.3027	3.1845	-4.6490	2.8698	-89.504
2	59.252	3.1196	-3.6285	-2.5516	-3.3227	-56.055
3	-121.87	7.9135	7.4298	-5.7884	6.4998	-111.55
4	127.95	6.9203	-7.8594	-0.8874	-6.8043	-32.635
5	-121.87	7.9135	-7.4298	5.7884	-6.4998	-111.55
6	127.95	6.9203	7.8594	0.8874	6.8043	-32.635
7	-52.110	3.3027	-3.1845	4.6490	-2.8698	-89.504
8	59.252	3.1196	3.6285	2.5516	3.3227	-56.055
9	3.5658	-0.1966	-0.2157	-3.6645	-0.1791	-71.400
10	-122.21	7.8499	-2.4706E-10	2.7692E-09	1.4813E-08	-112.00
11	128.32	6.8509	1.8000E-10	3.1555E-09	-2.1137E-08	-32.724
12	3.5658	-0.1966	0.2157	3.6645	0.1791	-71.400
MINIMUM	-122.21	-0.1966	-7.8594	-5.7884	-6.8043	-112.00
Pile N.	10	9	4	3	4	10
MAXIMUM	128.32	7.9135	7.8594	5.7884	6.8043	-32.635
Pile N.	11	3	6	5	6	4



OUTPUT FROM GROUP SOFTWARE, MAXIMUM DESIGN LOADS:

Compression: 128.32 kip Tension: 122.21 kip Moment: 112.00 kip-in

Therefore, with Factor of Safety (FS) = 2

Required ultimate capacities:

$Q_{ult} = 256.64$ kip (compression), 244.41 kip (tension)



Summary Report

Boring 23: COMPRESSION

Helical Pile Number: 1	Product: RS5500.361HCP	
Length: 58.0 ft	Angle: 85.0 degree	Datum Depth: 0.0 ft
Friction Type: Steel	Analysis Method: US Navy	
Steel Diameter (in)	Steel Length (ft)	
5.5	45.0	
Helix Diameter (in)	Helix Depth (ft)	Ultimate Helix Geotechnical Capacity (qc) (kip)
18	45.3	34.3t 69.9c
18	49.8	77.4t 90.0c
16	53.8	72.6t 82.3c
14	57.3	63.2t 82.1c
Total Ultimate Helix Geotechnical Capacity (Qg) (kip)		247.4t 324.2c
Total Ultimate Helix Recommended Capacity (QR) (kip)		
Total Ultimate Friction Capacity (QF) (kip)		8.2tc
Total Ult. Combined Geotechnical Capacity (Qgc) (kip)		255.6t 332.4c
Total Ult. Combined Recommended Capacity (Qrc) (kip)		

DESIGN EXAMPLES

HeliCAP[®] Summary Report

Helical Capacity Design Software



The typical net deflection of end bearing helical piles at working load (safety factor of 2) averages 1/4 inch. See Chance Technical Design Manual Edition 4 Section 5.9 for more information.

Water Table Depth: 0

Hammer Efficiency: Safety Hammer (60%)

Critical Depth:

No Load Zone Depth Below Groundline: 10

Soil Profile

Depth (ft)	Soil Type	N	N ₆₀	Cohesion (psf)	Angle of Internal Friction (degrees)	In Situ Unit Weight (pcf)	Clay Bearing Capacity Factor (N _c)	Sand Bearing Capacity Factor (N _s)	Bond Value (psf)
0	Sand	3	3	0	28.2	75	0	10.46	
5	Sand	3	3	0	28.2	75	0	10.46	
10	Sand	2	2	0	27.9	70	0	10.07	
15	Sand	4	4	0	28.5	80	0	10.87	
20	Sand	4	4	0	28.5	80	0	10.87	
25	Sand	2	2	0	27.9	70	0	10.07	
30	Sand	3	3	0	28.2	75	0	10.46	
35	Sand	10	10	0	30.2	100	0	13.49	
40	Sand	16	16	0	31.8	106	0	16.56	
45	Sand	43	43	0	39.4	133	0	44.71	
50	Sand	42	42	0	39.1	132	0	42.97	

SOLUTION

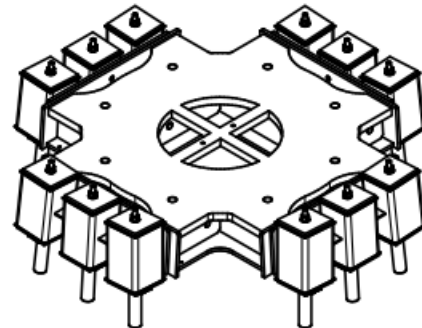
(12) helical piles, installed at 5 degrees from vertical with steel grillage

RS5500.361 w/14, 16, 18 & 18-inch diameter helices, installed 58 feet deep

$$\text{Minimum Torque: } T = Q_{ult}/K_t$$

Where $Q_{ult} = 256 \text{ kip}$ and $K_t = 5 \text{ (RS5500)}$

$$= 256/5 = 51 \text{ ft}\cdot\text{kip}$$





SECTION 9: SOIL SCREW® RETENTION WALL SYSTEM

CONTENTS

Introduction.....	9-2
Soil Screw® Retention Wall System.....	9-2
Preliminary Design Considerations	9-4
Geotechnical And Structural Engineering.....	9-5
Limiting Load Capacities	9-7
General Construction Considerations of Underpinning/Shoring Systems.....	9-7
Concepts And Applications Of Underpinning/Shoring Systems	9-8
Case Study 1 - High Foundation Line Load With Shallow Cut.....	9-9
Case Study 2 - Low Foundation Line With Deep Cut.....	9-12

RETENTION WALLS

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 should be conducted and state and local building codes and authorities should be consulted 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 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

Hubbell Power Systems, Inc. provides the Soil Screw® Retention Wall System as an efficient and economical system to retain soil during excavation and construction of structures below grade. The following are some of the advantages of this system over other soil retention methods:

- Fast installation without specialized equipment;
- Immediate support without curing time;
- Reduced installation time - post-tensioning not required;
- No need for H-piles, walers and heavy reinforced walls;
- Immediate on-site capacity verification; and
- Excavations adjacent to existing structures are possible when used with Atlas Resistance® Piers or Chance® Helical Piles;

The Chance® Underpinning/Shoring system provides for underpinning existing shallow footings, permitting excavation adjacent to the existing structure to a depth that would otherwise undermine the existing footing. The system allows excavation to proceed directly adjacent to an existing building without fear of vibration or structural damage to the building.

Commercial property owners often want to construct buildings with maximum possible footprints and a basement to maximize the potential of the site. If there is an existing building with a shallow footing adjacent to the proposed construction site, that building will need to be protected against damage from settlement due to removal of the soil that is laterally supporting the existing footing. Similar protection is required when a sloping excavation is cut next to an existing shallow footing in order to construct a building, parking lot, or roadway adjacent and down-slope of this footing.

The Soil Screw® Retention Wall System is designed to provide protection to the existing structure by using a combination of foundation support products. Atlas Resistance® Piers or Chance® Helical Piles are used to underpin the foundation of the existing structure. The structural load from the shallow footing is transferred down to a suitable bearing stratum below the depth of the intended excavation. The Soil Screw® Retention Wall System, combined with a reinforced shotcrete retaining wall is then used to maintain stability of the cut slope and the underpinning system as the excavation proceeds. For some conditions Chance® Helical Tieback Anchors can be used at the underpinning bracket to further ensure against lateral footing movement of existing buildings.

Other methods require the use of impact driven “soldier” piles. The major disadvantages to this system are the equipment size, noise and vibrations caused by the installation of the piles. This can be bothersome, annoying and stressful to the occupants of surrounding buildings, could damage sensitive electronics and/or could cause settlement of the building being protected. Because the Chance® Foundation Stabilization System and support uses hydraulic power for driving the underpinning, helical tieback anchors, and Helical Soil Screw® Anchors, it is extremely quiet and practically vibration free, thus allowing full use of neighboring buildings during the construction process.

SOIL SCREW® RETENTION WALL SYSTEM

PRODUCT BENEFITS

Chance Soil Screw® Retention Wall Systems offer the following benefits:

- Low installed cost
- No vibration
- Shorter installation lengths
- Ease of installation in limited access areas
- Minimum disturbance to site
- Immediate loading
- On-site load test capability
- Reusable in temporary stabilization applications

SYSTEM DESCRIPTION

The Chance Soil Screw® Retention Wall System creates an internally reinforced soil mass when closely spaced in a regular geometric pattern and protected by a reinforced facing of shotcrete. It differs from helical tieback anchors even though the appearance of the products is similar.

A tieback restrained wall is generally constructed by installing a structural wall facing system that is anchored to the earth by means of high strength helical anchors that are installed to a stratum of soil of sufficient strength to resist the forces placed upon the wall by the retained earth. The helical tieback anchor experiences a tension load equal to the retained earth forces. The structural retaining wall must be designed with sufficient strength to be able to support the soil load between tiebacks without excessive deformation.

Chance® Helical Soil Screw® Anchors are designed and installed differently than helical tieback anchors. They are generally seated at a shallower depth than helical tieback anchors when installed to retain similar soil masses. Most importantly, the Helical Soil Screw® Anchors are not tensioned after installation; they are passive elements. When the Soil Screw® Retention Wall System is installed it holds the soil as a single mass of sufficient internal stability to provide a suitable Factor of Safety (FS) against failure. The load on the Helical Soil Screw® Anchors is created across the movement plane as the soil mass moves slightly downward due to gravity.

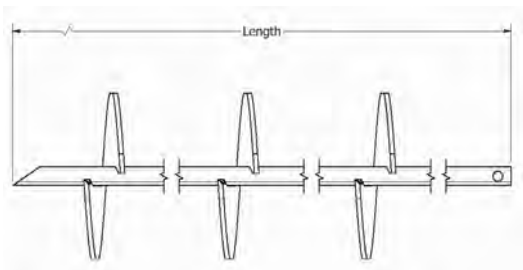
Many projects require that excavations be extremely close to existing structures. By combining Atlas Resistance® Modified Piers, or Chance® Helical Piles, Chance® Helical Tieback Anchors, and the Chance Soil Screw® Retention Wall System together, the designer can safely support an existing structure and the underlying soil mass during adjacent excavations. Atlas Resistance® Piers or Chance® Helical Piles support the structural load of the perimeter of the building, thus dramatically reducing the surcharge on the soil mass that must be retained. Chance® Helical Tieback Anchors are used for lateral support of the building's footing in projects where deep, adjacent excavations are required and/or

for buildings with perimeter weights exceeding 4,000 pounds per linear foot. With the surcharge loads properly transferred away from the soil mass under the building, the design for soil retention using Chance® Helical Soil Screw® Anchors is greatly simplified and requires fewer Helical Soil Screw® Anchors. In many instances, this method is the only economical way to accomplish this task. This method of structure/soil mass support prevents structure distress that may manifest itself during potential settlement as the soil mass loads the Chance Soil Screw® Retention Wall System.

SELECTION GUIDELINES

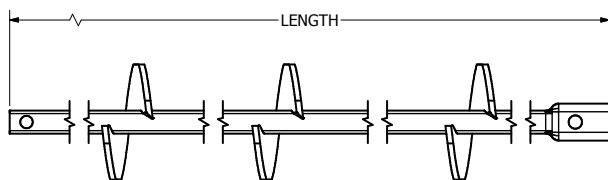
The Chance Soil Screw® Retention Wall System is available in two shaft sizes and two helix diameters. A variety of shaft lengths are offered to provide a designer an adequate selection for any application and load requirements. Design and installation requires input and supervision by a professional engineer and adequate site specific soil information.

CHANCE SOIL SCREW® RETENTION WALL SYSTEM (TYPE SS5 AND SS175 SERIES) LEAD SECTIONS



Product Designation	Product Series	Length	No. Plates	Plate Size	Weight lb.
C1100692	SS5	4'-11	2	8" Dia.	49
C1100691	SS5	7'-0	3	8" Dia.	69
C11002350301	SS175	5'-2	2	8" Dia.	62
T11006740302	SS175	6'-9	3	6" Dia.	75

CHANCE SOIL SCREW® RETENTION WALL SYSTEM (TYPE SS5 AND SS175 SERIES) EXTENSION SECTIONS



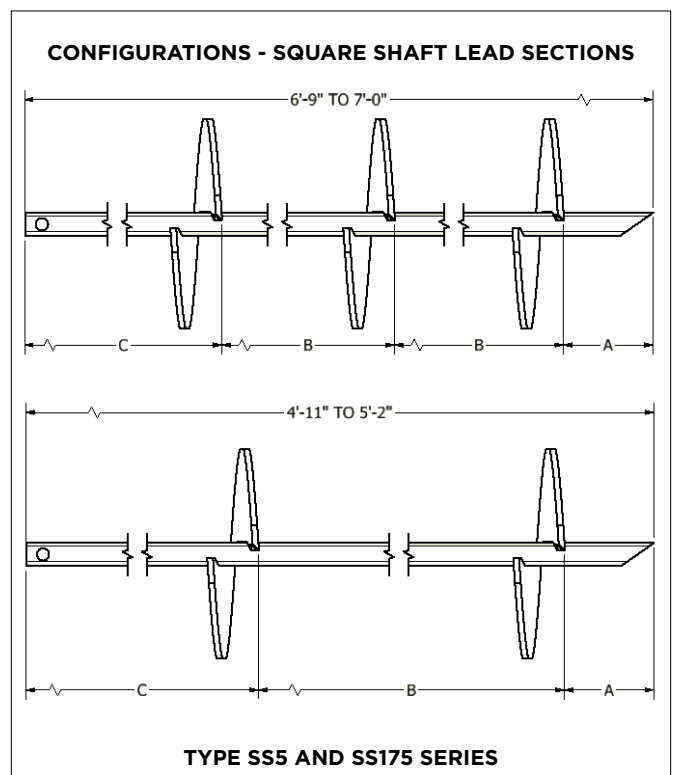
Product Designation	Product Series	Length	No. Plates	Plate Size	Weight lb.
C1100690	SS5	4'-9	2	8" Dia.	42
C1100689	SS5	6'-9	3	8" Dia.	50
C11004500301	SS175	6'-11	2	6" Dia.	70
C11004500302	SS175	6'-10	3	8" Dia.	75

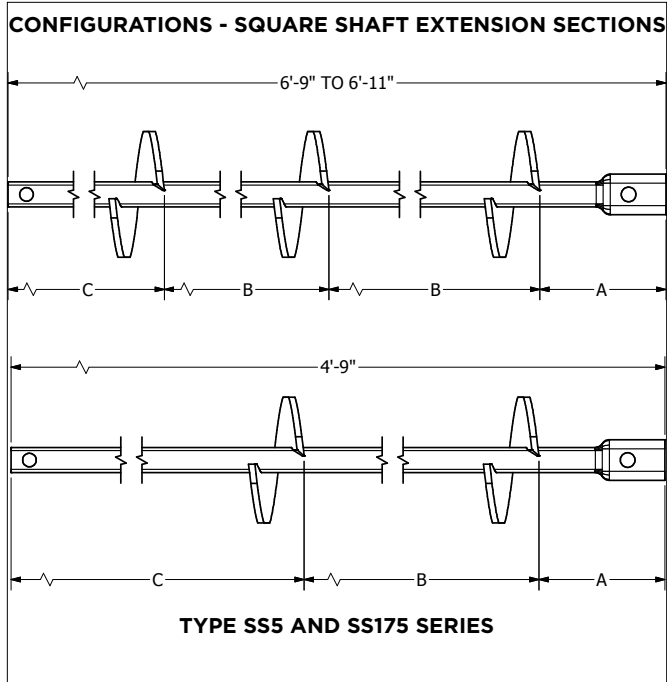
CHANCE SOIL SCREW® RETENTION WALL SYSTEM (TYPE SS5 AND SS175 SERIES) LEAD SECTIONS

CONFIGURATION TABLE (Leads and Extensions)						
Bar Size	Plate Size	Length	Dim A	Dim B	Dim C	No. Plates
1-1/2" Square Soil Screw® Lead Section	8" Dia.	4'-11	6"	29"	24"	2
		7'-0	6"	29"	20"	3
1-1/2" Square Soil Screw® Extension	8" Dia.	4'-9	5"	29"	23"	2
		6'-9	6"	29"	17"	3
1-3/4" Square Soil Screw® Lead Section	6" Dia.	5'-2	8"	30"	24"	2
	8" Dia.	6'-9	6"	30"	15"	3
1-3/4" Square Soil Screw® Extension	6" Dia.	6'-11	6"	30"	17"	3
	8" Dia.	6'-10	9"	29"	15"	3

NOTES - SOIL SCREW® ANCHOR PRODUCTS (TYPE SS5 AND SS175 SERIES):

- Refer to the schematic drawings at the bottom of page 9-4 and below for Dimensions A, B and C.
- All extensions include integrally forged couplings, machine bolts and hex nuts
- All helical plates are welded to the shaft in conformance to the American Welding Society (AWS) Structural Welding Code AWS D1.1" and applicable revisions.
- Available Finish: Hot Dip Galvanized (HDG)





PRELIMINARY DESIGN CONSIDERATIONS

The following requirements must be considered:

1. An evaluation of: (a) the foundation soil strata (below the reinforced soil mass), (b) the soil stratum into which the helix plates will be located, and (c) the soil behind the reinforced soil mass to be retained by the Soil Screw® Retention Wall System.
2. A selection of the appropriate Helical Soil Screw® Anchor including shaft size, helix plate diameter and length of embedment.
3. A determination of the ultimate tension capacity of the Helical Soil Screw® Anchors with a suitable Factor of Safety.

The following preliminary design guide for Helical Soil Screw® Anchors is intended to provide a basic understanding of Soil Screw® Retaining Wall theory.

Soil Screw® Anchor wall design requires professional geotechnical and engineering input. Specific information involving the structures, soil characteristics and foundation conditions must be used for the final design.

PRELIMINARY DESIGN RECOMMENDATIONS

- The top of the Helical Soil Screw® Anchor wall typically moves in the range of 0.1% to 0.3% of the wall height. Vertical and lateral movements are expected to be approximately 1/4" for a ten-foot cut and 1/2" for a 20-foot cut. This lateral movement is of concern when there is

a structure located at the top of the proposed cut. It is therefore required that either Atlas Resistance® Piers or Chance® Helical Piles underpin the existing structure. It is recommended to use Chance® Helical Tieback Anchors at each underpinning placement location whenever the cut exceeds 12 feet and/or the existing structural line load is greater than 4,000 lb/ft.

- Surcharge loads due to slabs, column footings, overburden soils, vehicular traffic, or other structures behind the wall must be considered when calculating the soil loads to be retained by the Helical Soil Screw® Anchors.
- The Chance® Soil Screw® Retention Wall System is best suited to cemented or medium-dense to dense sand and to low plasticity clay soils with Standard Penetration Test (SPT) N values ≥ 8. Use caution in highly plastic clays and silts.
- The Chance® Soil Screw® Retention Wall System is poorly suited for jointed weathered rock material that dips into the excavation, loose sand with SPT N values ≤ 7 and in those cohesive soils with SPT N values of ≤ 6 (clays with cohesion < 850 psf or an allowable bearing stress < 2,000 psf) anywhere in the depth profile of soil that is to be excavated.
- Clean to relatively clean cohesionless soils with poor stand-up time typically require a 1" (±) flash shotcrete coating to be placed simultaneously with the excavation. The maximum recommended incremental face cut height is four feet or less. Use Chance® Helical Tieback Anchors when underpinning/shoring next to an existing structure.
- Use of the underpinning/shoring system is permissible for excavations of up to 20 feet and under extremely favorable conditions shall not exceed 25 feet.
- The underpinning/shoring system is a temporary support system. Creep is generally not a problem, however, the system is not recommended when the Liquidity Index (LI) is >0.2.
- Soil Screw® Anchors must have helix plates of the same diameter continuously along the installed length.
- Soil Screw® Anchors must be installed at a minimum downward angle of 5° from horizontal and typically do not exceed 15° downward angle.
- Engineering design shall include verification of several levels of design analysis:

Internal stability: The soil mass acts as a coherent mass

External stability: The ability to resist lateral sliding

Global stability: The ability to resist massive rotational failure outside the "internally stabilized soil" mass

IMPORTANT NOTICE

A Registered Professional Engineer shall design the Chance Soil Screw® Retention Wall System. The installation shall be performed by trained and certified installing contractors/dealers.

GEOTECHNICAL AND STRUCTURAL ENGINEERING

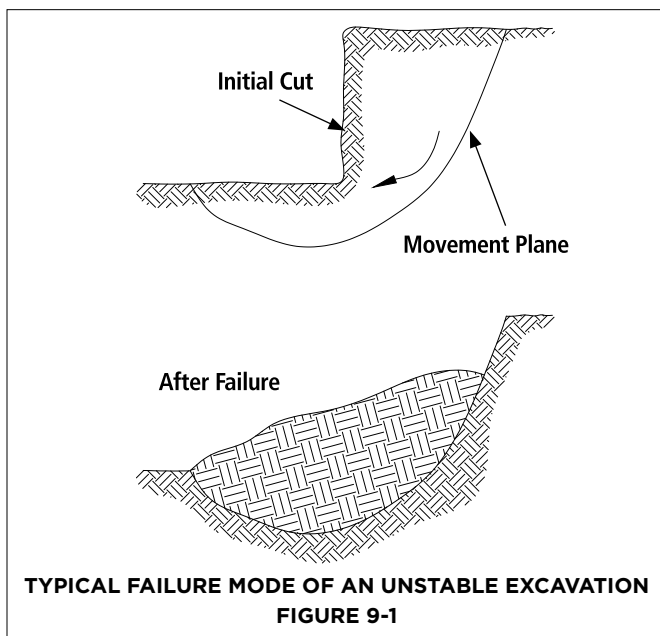
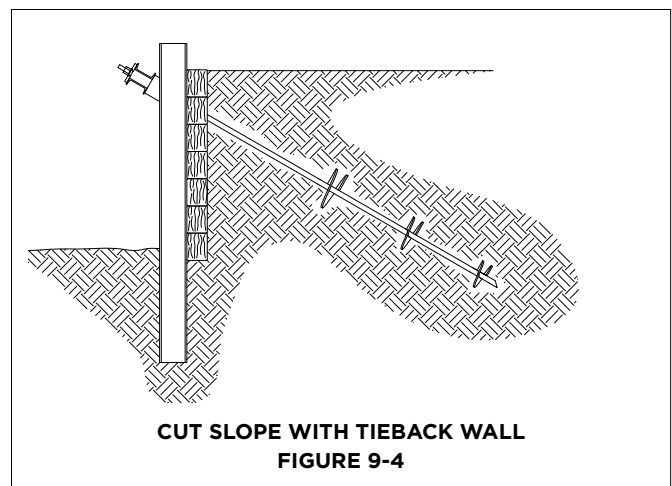
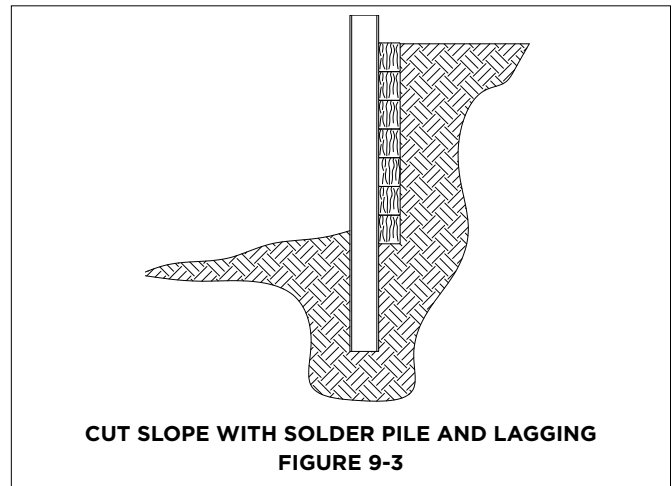
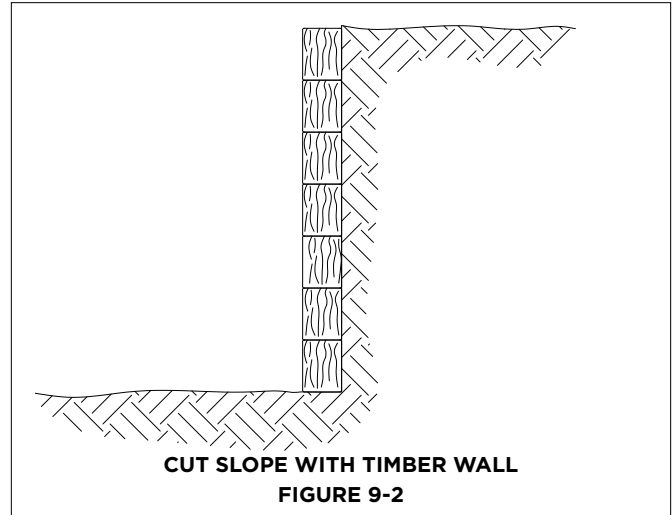
For an introduction and guidance on how to design retention walls using the Chance Soil Screw® Retention Wall System, refer to the Soil Screw® Retention Wall System Design Manual. For a copy of this manual, please contact your area Chance® Distributor or visit the Hubbell Power Systems, Inc. website at www.chancefoundationsolutions.com.

Design Example 10 in Section 8 provides a detailed wall design using the Chance Soil Screw® Retention Wall System.

Chance Helical Soil Screw® Anchors look similar to helical tie-back anchors, but they are different and they act differently to stabilize a slope. To understand how Helical Soil Screw® Anchors act and the differences between the two products, we must examine a cut slope that is unable to stand for an extended time on its own (see Figure 9-1).

A simple method to improve stability of the slope would be to stack railroad ties against the cut face so that the soil would have to push the ties over in the process of failing (see Figure 9-2). If this proves insufficient, driving “soldier” piles in front of the railroad ties (now termed “lagging”) enhances the stability. Now the soil must push the lagging and the soldier piles over before failure can occur (see Figure 9-3).

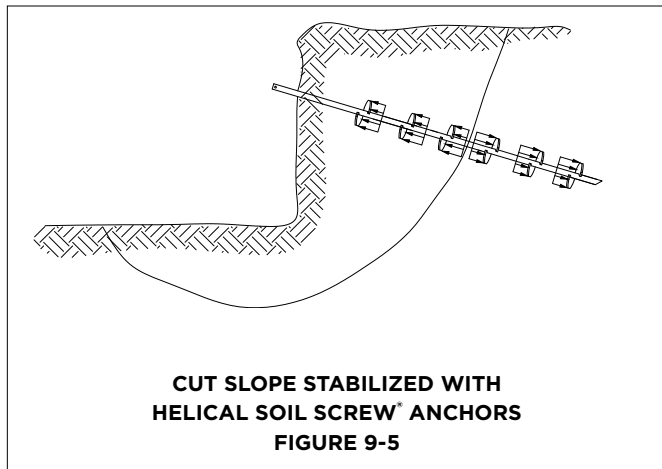
If this is still insufficient to stabilize the soil, a beam can be installed along the wall connecting the soldier piles. This beam is called a “waler” and it is anchored by helical tieback anchors to a stable portion of the soil mass behind the failure plane (see Figure 9-4). Now as the slope attempts to fail, the sliding soil pushes against the lagging, the lagging pushes against the soldier piles, the soldier piles push against the waler, and the waler pulls on the tiebacks. If the helical tieback anchors provide





enough resistance, the whole system is stable. The design of the wall system (the lagging, soldier piles and the waler) brings the distributed soil force against the lagging toward, and concentrates the load at, the helical tieback anchors. After the tiebacks are installed, they are usually post-tensioned. When helical tiebacks are used for this type of application, they are typically concentrated in a few tiers, and are designed so that all tension resistance is attained within the stable soil mass behind the potential movement plane.

Helical Soil Screw® Anchors differ from helical tieback anchors because they are designed to attain pullout resistance within the sliding soil mass as well as the stable mass behind the movement plane. For Helical Soil Screw® Anchors to be effective, they must have helices along the whole length of the shaft. When the unstable soil mass begins to slide, it moves against the helices buried within this unstable mass (see Figure 9-5). The resistance generated on the helices within the unstable mass secures the soil directly and reduces the resulting soil pressure against the wall. The net effect is that Helical Soil Screw® Anchors reduce the structural requirements for the wall system. In most cases the Helical Soil Screw® Anchors are connected directly to the wall without the use of soldier piles or walers. The retaining wall is therefore thinner than a wall required when using tieback anchors.

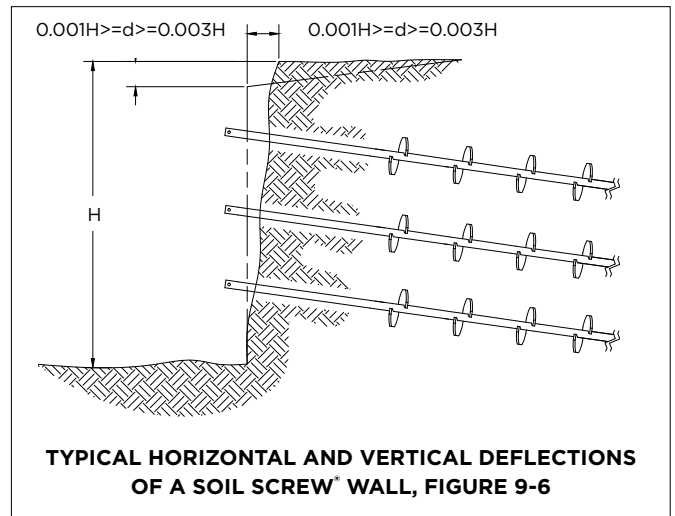


Helical Soil Screw® Anchors are more evenly distributed on the wall and therefore carry lighter loads than helical tieback anchors. Helical Soil Screw® Anchors should not be post-tensioned as post-tensioning puts bearing stresses on the wrong side of the helices that are embedded in the unstable soil mass. Some engineers require that a small load (1000 pounds or less) be applied to newly installed Helical Soil Screw® Anchors to remove any slack in the connections.

Because Helical Soil Screw® Anchors are not post-tensioned, the unstable soil mass has to slump slightly before the Soil Screw® System can develop resistance. Soil Screw® Retaining Walls deflect both vertically downward and laterally outward during this slumping process. The magnitudes of both deflections typically vary from 0.1% to 0.3% of the wall height (see Figure 9-6). For example, the top of a 12-foot high wall will typically deflect from 1/8" to 3/8" downward and outward. Because 3/8" settlement approaches the level that can cause

damage in some structures, the Hubbell Power Systems, Inc. Underpinning/Shoring System includes helical tieback anchors at the underpinning bracket whenever excavation depths exceed 12 feet or structural footing loads exceed 4,000 lb/ft. Post-tensioning these tieback anchors prior to excavation allows the deflections at the footing to be controlled to an acceptable level.

Because of the potential severity of a structural failure involving one of these systems, Hubbell Power Systems, Inc. recommends that a staff application engineer, or an engineer from an authorized Chance® Distributor perform a preliminary design and make a final wall design review. The preliminary design will give recommendations for the Helical Soil Screw® Anchors and, if the project requires, specific underpinning piers/piles and/or helical tieback anchors to be used on the specific project. Details for the placement of the products, the required embedment depths and minimum installation resistances and torques will be recommended. These preliminary recommendations, estimates of installation depths and wall thickness will aid in preparing cost estimates. Both the installing contractor/dealer and the Engineer of Record shall review these recommendations. The Chance® Distributor or Hubbell Power Systems, Inc. Engineer will work with the Engineer of Record as required to resolve any issues regarding the preliminary design. The Engineer of Record must accept and approve the final design before construction can begin.



SHOTCRETE

Shotcrete is portland cement concrete or mortar propelled at high velocity (typically by air pressure) onto a surface. With wet process shotcrete, the dry materials are mixed with water and pumped to a nozzle, where air is added to project the material onto the surface. Dry process shotcrete, also known as "gunite", delivers the dry material to the nozzle by air pressure where water is added at the point of discharge. The water and dry materials mix during deposition. Each process has its own advantages and disadvantages, but either, or both, may be used to construct the wall facing for the Chance Soil Screw® Retention Wall System.



The wet process allows for high deposition rates up to three times the rate attainable with gunite with less rebound (5% vs. 15% for gunite). In addition, the nozzleman need not be as highly skilled for this process. The major disadvantages to the shotcrete wet process are the extensive cleanup required and the difficulty scheduling ready-mix deliveries. The gunite (dry) process has the advantage of easy clean up and the ability to mix materials on site. Gunite has more disadvantages than shotcrete. Gunite has a relatively low deposition rate (slower application), has more rebound and requires highly skilled operators.

The functions of shotcrete in the Chance Soil Screw® Retention Wall System are:

- To prevent sloughing and spalling of the excavated soil face.
- To prevent buckling of the underpinning pier/pile, if required on the project.
- To transfer the earth pressures to the Helical Soil Screw® Anchors instead of the inner wall face.

In some instances, the system is exposed only temporarily. The excavation is usually filled in after the basement wall is constructed or permanent facing is built in front of the system's wall. In some cases, however, the system wall will be permanently exposed and must also perform cosmetic functions.

Flexural strength, shear strength and ductility are the important characteristics of the wall in this application. The wall must resist the movement of the retained soil and restrain the underpinning pier/pile (if used on the project) from buckling, both of which require flexural strength. The wall must also transfer load to the Soil Screw® Anchor head, which requires both shear and flexural strength. Because deformation is necessary to generate the resistance that makes the system stable, the wall must tolerate some deformation without losing its strength. The properties of the shotcrete that contribute to these wall characteristics are compressive strength and bond strength.

A structural engineer employed by the owner will typically prepare the final shotcrete wall design. Hubbell Power Systems, Inc. suggests that the wall design be reviewed by one of their staff application engineers or authorized Distributors.

LIMITING LOAD CAPACITIES

ULTIMATE TENSION STRENGTH

The ultimate tension strengths indicated in Table 9-1 represent the net tension strengths of the Helical Soil Screw® Anchor shaft/coupling systems. The designer must use an adequate Factor of Safety in the design to preclude Helical Soil Screw® Anchor failure in tension. A Factor of Safety of 2:1 is often used.

TORQUE STRENGTH RATING

The torque ratings indicated in Table 9-1 represent the maximum torque that should be applied to the Helical Soil Screw® Anchor during installation in homogeneous soils. The risk of torsional fracture increases significantly as the applied torque

increases beyond these limits. In obstruction-laden soils, the maximum torques that should be applied during installation are 80% of the table limits due to the increased risk of torsional fracture posed by impact loading. The designer must consider these torque ratings in evaluating whether the Helical Soil Screw® Anchors can be installed to the required depths. In addition, these torque ratings pose practical limits to the ultimate tension capacities that can be developed by limiting the strengths of soils into which the Helical Soil Screw® Anchors can be installed. The practical limit to the ultimate tension capacities that can be achieved (in lbs) is about ten times the installation torques (in ft-lbs) that may be applied during installation using a torque factor (K_t) of 10. See Section 6 for a detailed discussion of the correlation of installation torque of a helical anchor to its ultimate tension capacity.

ULTIMATE TENSION STRENGTHS AND TORQUE RATINGS FOR CHANCE® HELICAL SOIL SCREW® ANCHORS, TABLE 9-1

Chance® Soil Screw® Product	Ultimate Tension Strength	Torque Rating
SS5 Series 1-1/2" (38 mm) Round Corner Sq.	70,000 lbs	5,700 ft-lbs*
SS175 Series 1-3/4" (45 mm) Round Corner Sq.	100,000 lbs	10,500 ft-lbs*
* Refer to Ultimate Tension Strength and Torque Rating in the text. Practical load limits in the field may be limited due to the factors discussed above.		

GENERAL CONSTRUCTION CONSIDERATIONS OF UNDERPINNING/SHORING SYSTEMS

The Chance Soil Screw® Retention Wall System for underpinning/shoring next to an existing structure is a specialized construction process and must be installed by Certified Chance® Installer. Listed below are some general items regarding the construction procedures:

WARNING! DURING THE COURSE OF CONSTRUCTION, THE FOOTING AND FACE OF THE SHORING SHOULD BE CONTINUOUSLY MONITORED FOR ANY MOVEMENTS. IF MOVEMENTS ARE NOTED, THE CONSTRUCTION PROCESS SHOULD BE STOPPED, TEMPORARY BRACING INSTALLED AND THE ENGINEER AND/OR GEOTECHNICAL ENGINEER SHOULD BE IMMEDIATELY NOTIFIED FOR FURTHER DIRECTION.

1. As is the case in conventional underpinning of buildings using Atlas Resistance® Modified Piers or Chance® Helical Piles, the footing must be properly prepared so that the pier/pile bracket can be positioned under the footing with a minimum of eccentricity with the wall load. This process may involve chipping the concrete to provide a proper bearing surface and creating a notch in the spread footing to reduce pier/pile eccentricity.



2. For those projects requiring underpinning and Chance® Helical Tieback Anchors at the pier/pile bracket, the tieback must be installed to the required length and torque prior to installing the underpinning system.
3. If Atlas Resistance® Modified Piers are used as the underpinning system, the process requires the use of pier sleeving to prevent buckling at the joints of the pier pipe. Every sleeve joint must be at least 18" away from a pier pipe joint. In some cases grouting of the pier pipe along with the insertion of a steel reinforcement bar may be specified.
4. The pier sleeving must be installed to a minimum of 2 feet below the deepest excavation (cut).
5. If using Atlas Resistance® Modified Piers, the piers shall be driven to the required depth and load tested to 150% of the design load. Then each pier shall be preloaded to at least 95% of the design load and locked off. If using Chance® Helical Piles as the underpinning system, the helical piles shall be installed to the required minimum depths and minimum average installation torques.
6. When the Atlas Resistance® Pier or Chance® Helical Pile underpinning system installation is complete, the helical tieback anchor shall be attached to the pier/pile bracket and preloaded. Normally the tieback is preloaded to the design load.
7. Upon completion of all of the underpinning and tieback operations, the wall face excavation can commence. If the soils are generally cohesionless (sands, etc.) or there is any danger of the soil face sloughing off, a 1" thick flash coat of shotcrete shall be immediately placed against the face of the cut as the excavation proceeds. If the cut soil is capable of standing by itself, then the first layer of shotcrete can be applied after the initial cut is complete. The same procedure shall be followed for subsequent incremental excavations. Under no circumstances should a cut of any height be left open at the face for more than two hours.
8. The depth of cut on the first excavation, as well as on subsequent incremental excavations shall be at least one foot deeper than the depth of the row of Helical Soil Screw® Anchors. See Figure 9-13, which shows a 6-foot cut and 5-foot deep row of Helical Soil Screw® Anchors.
9. When the first excavation is complete (with or without shotcrete flash coating), the first row of Chance® Helical Soil Screw® Anchors is installed to the requirements indicated in the design specifications (length of installation, minimum torque, installation angle, etc.). A Helical Soil Screw® Anchor shall be positioned immediately adjacent to each underpinning pier/pile. Shotcrete is placed onto the cut face to 1/2 of the total specified shotcrete thickness.
10. The welded wire mesh reinforcement is set against the face of the wet shotcrete along the cut face of the wall with excess reinforcement turned outward at the bottom of the cut to allow for overlap of reinforcement on successive stages.
11. Welded rebar assemblies with bearing plates are positioned over each Helical Soil Screw® Anchor and secured against the welded wire mesh reinforcement and (still) wet shotcrete face.
12. The remaining shotcrete is installed to provide the total thickness specified.
13. Steps 7 through 12 above are repeated after each incremental excavation. Stabilization continues until all of the Helical Soil Screw® Anchors are installed and the reinforced shotcrete wall is completed to the design depth.

CONCEPTS AND APPLICATIONS OF UNDERPINNING/SHORING SYSTEMS

BACKGROUND

The construction of additions to office and commercial buildings or new construction adjacent to existing buildings requires earth excavation much deeper than the footing elevation of the immediately adjacent building(s). The use of sheet pile and/or H-piles with wood lagging to prevent adjacent footing subsidence requires the use of dynamic pile driving equipment with the attendant vibrations and noise levels. There are decided disadvantages to these traditional approaches since the vibrations may cause movement of the existing building foundation and subsequent structural damage. Additionally, the vibration levels can often lead to a shutdown of business operations if conducted during normal working hours.

Hubbell Power Systems, Inc. offers an underpinning/shoring system that not only avoids the vibrations and noise level issues, but also permits the shoring and excavation to proceed at a more rapid pace. In many cases this results in an overall cost savings to the prime contractor and owner. The examples covered below are intended to illustrate some of the design concepts and applications of this system.

In conducting preliminary designs for projects using the underpinning/shoring system and in the development of the case studies that follow, Hubbell Power Systems, Inc. uses certain guidelines. These guidelines are briefly summarized below:

1. Hubbell Power Systems, Inc. does not currently recommend using the underpinning/shoring system for excavations exceeding 25 feet.
2. Although Atlas Resistance® Piers or Chance® Helical Foundation Piles can be used for the underpinning stage; it is preferred to use the Atlas Resistance® Pier if "hard stratum" is within a reasonable depth at the proposed construction site.
3. The Atlas Resistance® Piers used for underpinning the existing building foundation must be sleeved with the joints of the sleeves offset from the joints of the underpinning pier pipe.

4. It is recommended in cases where the line load equals or exceeds 4,000 pounds per lineal foot and/or the depth of cut exceeds 12 feet to use a Chance® Helical Tieback integrated at the pier bracket level. This requirement uses the pier and tieback combination as illustrated in Figure 9-11. This helical product is used as a tieback anchor and not a Soil Screw® Anchor.
5. Helical Soil Screw® Anchors must be installed at a minimum downward angle of 5° and generally not to exceed 15°.
6. All Helical Soil Screw® Anchors have the same size helix plates continuously along the installed length of the shaft.
7. The bottom cantilever of shotcrete wall should be limited to 2/3 of the typical spacing for the Helical Soil Screw® Anchor row, but should not exceed 3 feet.
8. If the foundation soils to be excavated contain cohesionless soils (sands, silts and gravels and gravel and silty sands) a “flash coat” of shotcrete should be applied immediately as the cut is made.
9. Chance® Installers must receive formal training in the “concept” and “field installation technique” prior to using the underpinning/shoring system on an actual project.

NOTE: The designs and data shown in the following examples are not intended for use in actual design situations. Each project and application is different as to soils, structure and related factors.

CASE STUDY 1 - HIGH FOUNDATION LINE LOAD WITH SHALLOW CUT

Northern Excellence University is planning to construct an addition to the existing Book Science Building. The existing building has a continuous perimeter footing as shown in Figure 9-7. The building is a 3-story structure and has a foundation line load of 13,000 pounds per lineal foot. This reinforced concrete footing is seated about 4 feet below the existing ground line as noted in Figure 9-7. There are no column footings at the exterior wall of the existing building immediately adjacent to the proposed addition.

The proposed building addition will be placed immediately adjacent to a 100-foot section of one wall of the existing building as shown in Figure 9-10. The foundation for the new building will also be a reinforced concrete continuous footing, but it will be set eight feet below the bottom of the existing building footing as shown in Figure 9-7. The estimated footing load for the new addition is 10,000 pounds per lineal foot. As noted in Figure 9-7, a surcharge load will exist arising from the Live Load on the floor slab (100 lb/ft²), the weight of the concrete slab and the overburden pressure from approximately 3-1/2 feet of soil cover over the top of the existing footing.

A geotechnical investigation was conducted at the site and the results showed that below the first foot of topsoil, a stratum of silty to sandy clay existed to a depth of 18 feet. The

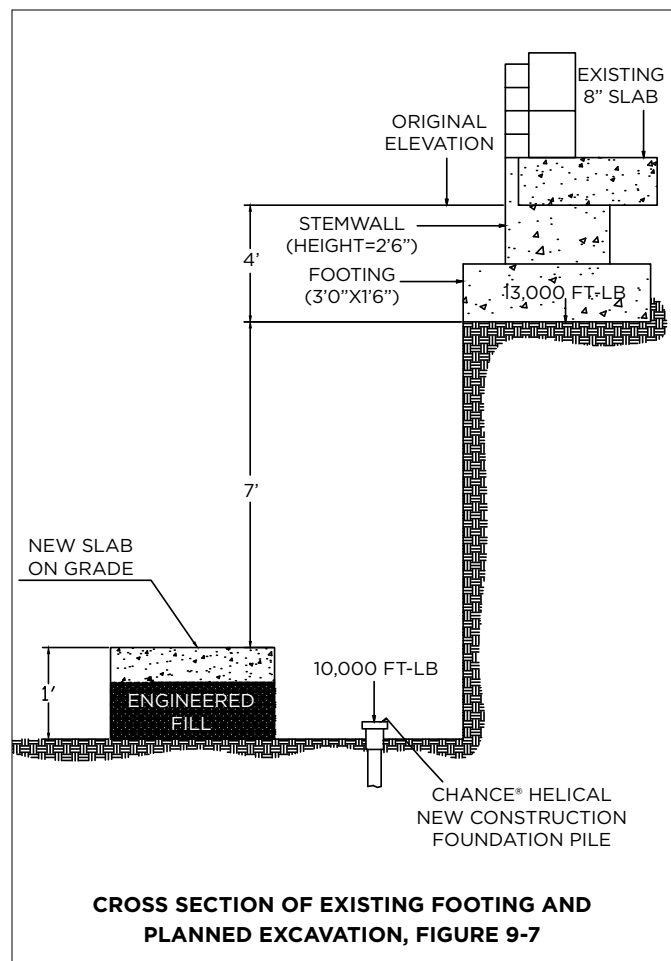
Standard Penetration Test (SPT) blow count, “N” for this soil was consistently in the 9 to 10 range through the 18 feet. Both by correlation with the “N” values and from the results of hand held penetrometer tests on the soil, this silty to sandy clay was determined to have a cohesion, “c” of 1,000 pounds per square foot and a friction angle, “φ” of 10°. Below the 18 feet of silty to sandy clay a stratum of weathered sandstone was encountered to the bottom of the borings at 20 feet at which the driller experienced auger refusal. No ground water was encountered during the soil borings.

UNDERPINNING SYSTEM - ATLAS RESISTANCE® MODIFIED PIERS

As noted above, a stratum of sandstone exists at the site beginning at a depth of 18 feet. Auger refusal was experienced at a depth of 20 feet. Allowing for four feet from the ground elevation of the boring log to the bottom of the footing to be underpinned indicates that the length of the underpinning pier pipe will be 16 feet. The existing footing line load is:

EQUATION 9-1

$p = 13,000 \text{ lb/ft}$



RETENTION WALLS



If we assume a pier spacing of 4 ft, center to center, the load per pier becomes:

EQUATION 9-2

$$\begin{aligned} pdes &= 13,000 \text{ lb (4 ft)} \\ &= 52,000 \text{ lbs} \end{aligned}$$

Based on a requirement of installing an Atlas Resistance® Pier to a tested load resistance of at least 50% higher than the design load leads to:

EQUATION 9-3

$$\begin{aligned} DS &= 52,000 (1.5) \\ &= 78,000 \text{ lbs} \end{aligned}$$

An Atlas Resistance® 2-Piece Modified Pier part number AP-2-4000.219[M] is selected. This pier is designed with a 4" diameter pier pipe and has an ultimate capacity of 98,000 lbs. The "M" indicates the use of 4-1/2" diameter sleeving over the pier pipe. The sleeved portion of the pier shall extend down to a depth of 10'-6" (three lengths of sleeve pipe). Since this is temporary construction, corrosion protection is unnecessary. Details of the underpinning and tieback anchorage are shown in Figure 9-11.

INTEGRATED TIEBACK SYSTEM - CHANCE® HELICAL TIEBACK ANCHORS

Following the recommendation of using an integrated tieback whenever the line load exceeds 4,000 lbs/ft, a Chance® Helical Tieback Anchor must be selected for used with each Atlas Resistance® 2-Piece Modified Pier placement. For this situation, the C1500006 Tieback Anchor Lead Section and C1500048 Tieback Extension with coupling and hardware is recommended.

The installed length is estimated to be 15 feet. The installed angle is 15° down from horizontal. The lead section consists of one 8-inch and one 10-inch diameter plate welded to a 1-1/2" square solid steel shaft. Installed torque is estimated to be 2,000 ft-lbs, minimum. No corrosion protection is required because the construction is temporary.

SOIL SCREW® RETENTION WALL SYSTEM

The body mass of soil that would slide along the movement plane if failure were to occur as excavation takes place is illustrated in Figure 9-8. If one uses the soil properties previously listed with an assumed failure plane angle (ϕ) of 51°, the driving force and resisting force may be calculated. In order to provide a Factor of Safety against failure of the body mass, a single line of Chance® Helical Soil Screw® Anchors will be used. A minimum Factor of Safety of 2.0 is required against such a failure. (Note that the typical design Factor of Safety for Helical Soil Screw® Anchors ranges from 1.3 to 2.0.) A Factor of Safety of 2.0 was selected because of the very high foundation line load of the existing footing above the excavation. In conducting the Soil Screw® Anchor analysis, it assumed that the Chance® Helical tieback anchors did not contribute to the holding capacity of the body mass of soil even though the tieback prevents cantilever at the top of the wall.

Also shown in Figure 9-8 is the resistance to movements that occur along the movement plane arising from the shear strength of the soil. This shear strength is made up of both the cohesion and friction acting along that plane.

In Figure 9-9 the same body mass of soil is shown, but now the single Helical Soil Screw® Anchor shown provides additional resistance to sliding that develops along the movement plane. If the installation angle of the Helical Soil Screw® Anchor is 10°, the new driving force and new resisting force may be calculated.

Generally, the Factor of Safety is illustrated by the following equation:

EQUATION 9-4

$$FS = RF / (DF - SSCF)$$

where

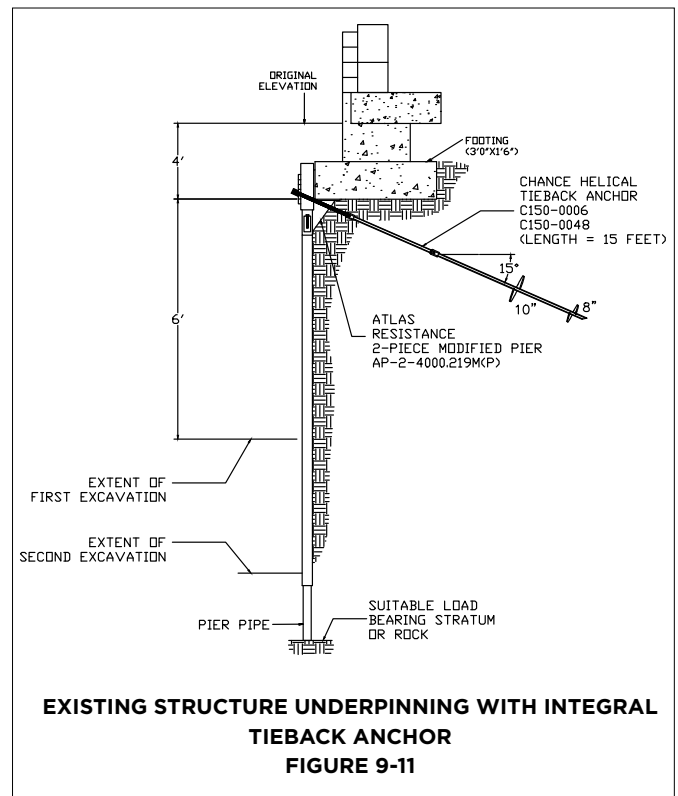
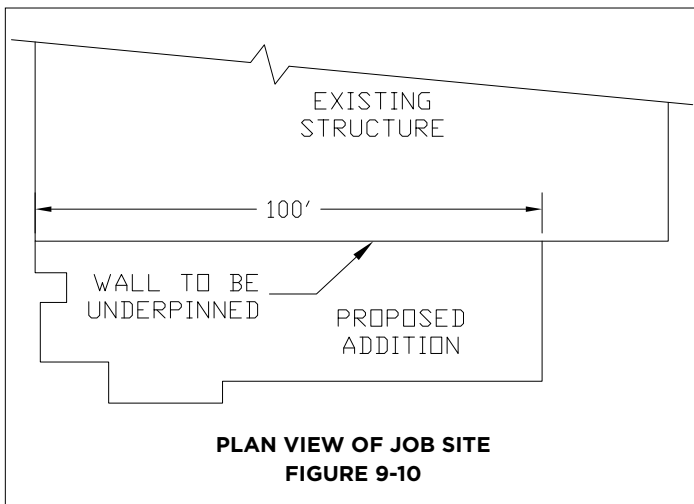
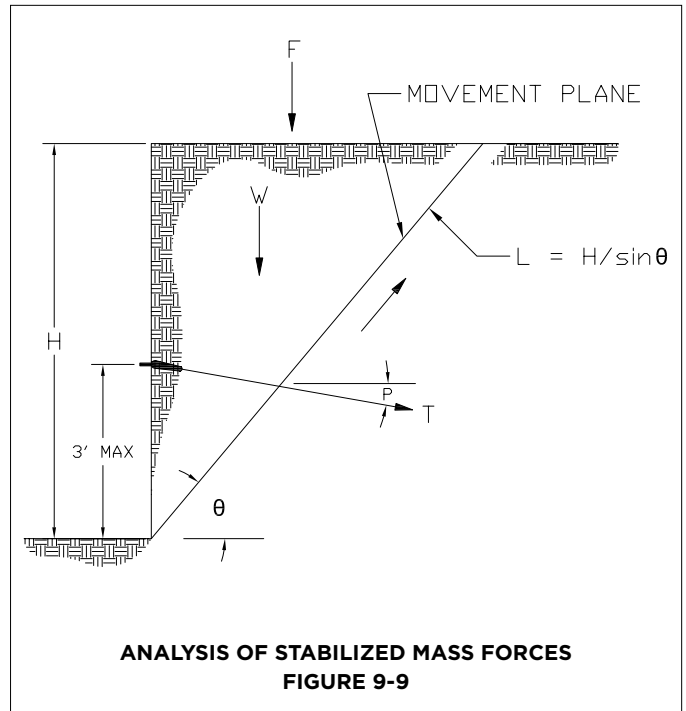
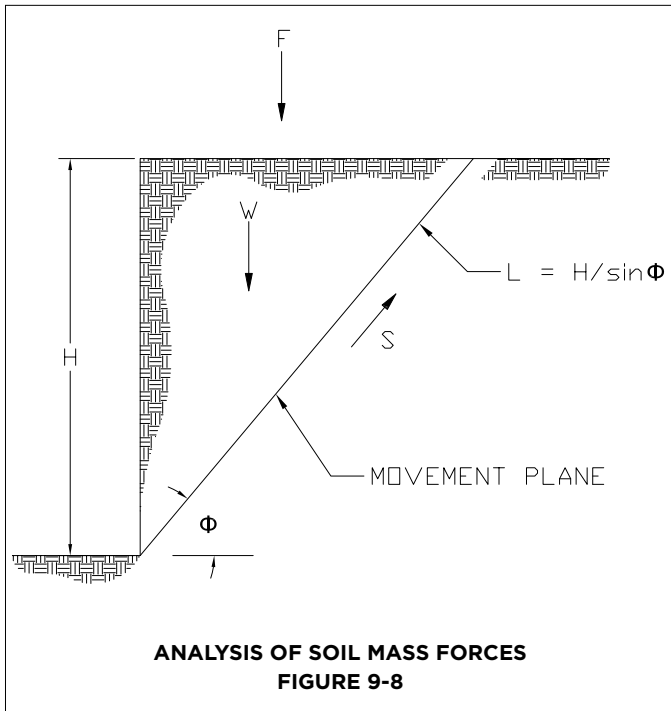
- FS = Factor of Safety
- RF = Resisting force
- DF = Driving force
- SSCF = Soil Screw® Anchor component force

Resisting Force (RF) arises from the shear strength of the soil (c and ϕ) along the movement plane and the Helical Soil Screw® Anchor component parallel to the movement plane. Driving Force (DF) is the component of the soil body mass (weight) in the direction of the movement plane. Helical Soil Screw® Anchor Component Force (SSCF) is the component of the total Helical Soil Screw® Anchor holding capacity (ultimate capacity) in the direction of the movement plane. Internal stability analysis as described herein is typically done with commercially available software such as SNAILZ (Caltrans) or Gold Nail (Golder Associates); see the Chance® Soil Screw® Retention Wall System Design Manual for an example. Helical Soil Screw® Anchor tension capacity is calculated with HeliCAP® Helical Capacity Design Software and input into the stability analysis software.

For the specific conditions defined above, the Chance® Helical Soil Screw® Anchor Lead Section C1100692 and C1100690 Extension is selected. The Helical Soil Screw® Anchor lead section consists of 8" diameter plates welded along the entire length of a 1-1/2" square shaft. Minimum installed length is 10 feet. Installed angle is 10° down from horizontal. Installed torque is estimated to be 1,500 ft-lb minimum. The single row of Helical Soil Screw® Anchors is set immediately adjacent to each underpinning pier pipe at a depth of 5 feet below the integrated tieback anchor (this will maintain the 3 foot maximum allowable bottom cantilever). No corrosion protection is required.

SHOTCRETE WALL

The shotcrete wall is a temporary facing for the excavation. Since there is a Chance® Helical Tieback Anchor at the top, the wall will be laterally anchored at the pier brackets to allow longer spacing for the single row of Helical Soil Screw® Anchors. The bottom cantilever should be 3 feet.



RETENTION WALLS



The vertical bearing bars are extended from the welded rebar head assembly to the dowels and waler at the top of the wall in order to augment the welded wire fabric reinforcing (see Figures 9-13 and 9-14).

The top wall segment is checked for flexure and shear using the distributed Soil Screw® Anchor head forces and one-way beam action. Two #4 reinforcing bar walers shall be placed continuously along the Soil Screw® Anchor row. The selected wall thickness is 4". Reinforcing is a welded wire fabric (WWF 6x6 W.14 or equivalent) spaced midway in the shotcrete wall at a 2" nominal depth.

SOIL SCREW® ANCHOR HEAD DESIGN

The shotcrete wall design is critical to the punching shear of the Soil Screw® Anchor heads and flexural strength of the all face between the Soil Screw® Anchor heads. The Soil Screw® Anchor head forces are expected to be approximately 1/2 of the total Soil Screw® Anchor tension load. The shotcrete facing is checked for flexure and punching shear using two-way slab action. This information is used in the internal stability analysis. A welded rebar head assembly can be used at each placement to provide local reinforcement. It is spliced to the horizontal walers and the vertical bearing bars previously described. To accomplish the proper positioning of the welded rebar head assembly and rebar, the welded wire fabric must be pushed into the initial 2" face coat of shotcrete approximately 1/2" at each Soil Screw® Anchor head. The 4" wall thickness and reinforcement selected above are adequate.

The first 6 feet of soil is excavated and the soil body mass is stabilized. Figure 9-13 shows the installation of a Chance® Helical Soil Screw® Anchor, welded wire reinforcement, welded rebar head assembly and shotcrete. Note that the shotcrete stops short of the bottom of the excavation to allow for splicing the welded wire mesh reinforcement and a suitable shotcrete joint. Figure 9-14 show excavation to the final elevation along with continued stabilization of the soil mass. Construction of the new foundation begins with the installation of Chance® New Construction Helical Piles.

CASE STUDY 2 - LOW FOUNDATION LINE WITH DEEP CUT

The City of High Hope is planning to build a new multi-purpose arena that will seat 8,000 people. The arena will be located within the downtown district. A 20-foot deep cut will be required for the new construction to provide sufficient elevation for the arena seating yet maintain a low ground level building profile. A portion of the arena wall will be immediately adjacent to the existing historic city market building (see Figure 9-15). The city market building is a single story warehouse that measures 60 by 120 feet. The back wall of the market building will abut the new arena wall. The market building was constructed in the early 1900s and has an unreinforced concrete grade beam foundation that measures three feet wide by two feet deep. The grade beam, seated three feet below the existing

grade, has a line load of 3,000 lbs per lineal foot. The general configuration of the footing along with installed underpinning and tieback is shown in Figure 9-16.

A geotechnical investigation conducted at the site found a 30-foot thick stratum of silty sand below approximately two feet of topsoil and fill material that consisted of silt, sand and cinders. The Standard Penetration Test (SPT) blow count "N" in this silty sand increased with depth from N=13 to N=18. Sufficient silt is present in the sand to hold a shallow vertical cut for a short period of time. Below the silty sand stratum at a depth of 32 feet the borings encountered a hard glacial till of clayey sand and gravel. The SPT value recorded were N=50+. By correlating the N values, the friction angle of the silty sand (ϕ) was estimated to be 30°. The ground water table (GWT) was located at 15 feet which means dewatering will be required prior to excavation.

Based on discussion with the designer and contractor, a decision was made to use the Chance® Helical underpinning/shoring technique in the immediate vicinity of the city market building. The Helical Soil Screw® Anchors will continue for an additional 50 feet on each side of the market building as the slope is cut in a benched pattern. Beyond this zone, adequate clear distance exists to back-slope the cut side without providing any wall retaining system.

UNDERPINNING SYSTEM - ATLAS RESISTANCE® MODIFIED PIERS

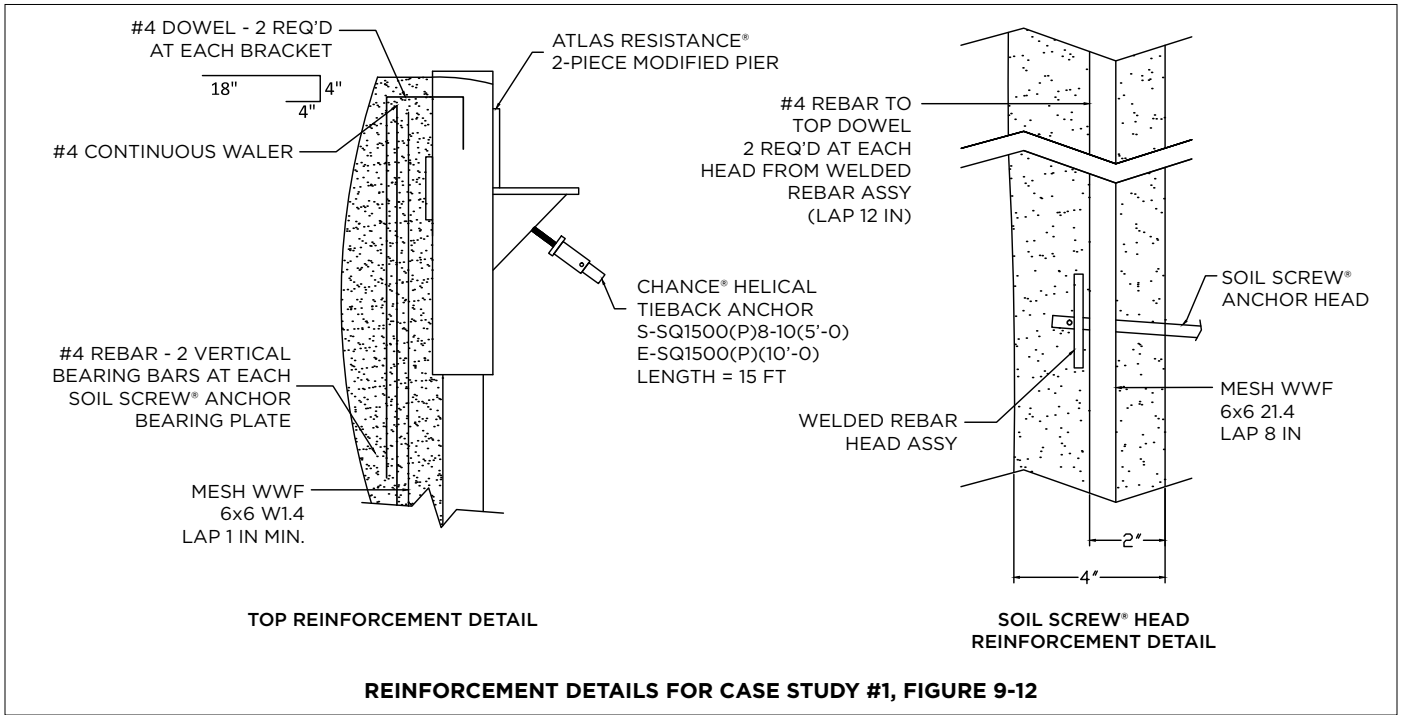
As noted above, a hard glacial till exists at a depth of 29 feet below the bottom of the market building footing. The estimated length of the underpinning pier pipe is 32 feet. The existing line load is 3,000 lb/ft. Although the footing line load is relatively light, the fact that the 24" thick footing is not reinforced will limit the spacing of the piers to five feet on center. Based on this spacing, the design load per pier becomes:

EQUATION 9-5	
P_{des}	= 3,000 lb (5 ft)
	= 15,000 lbs

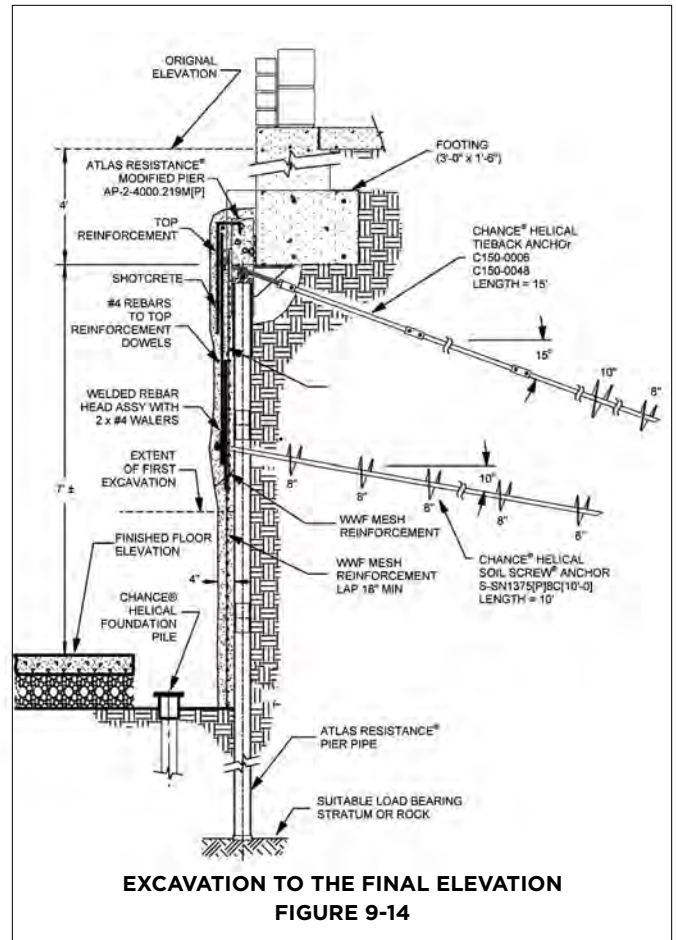
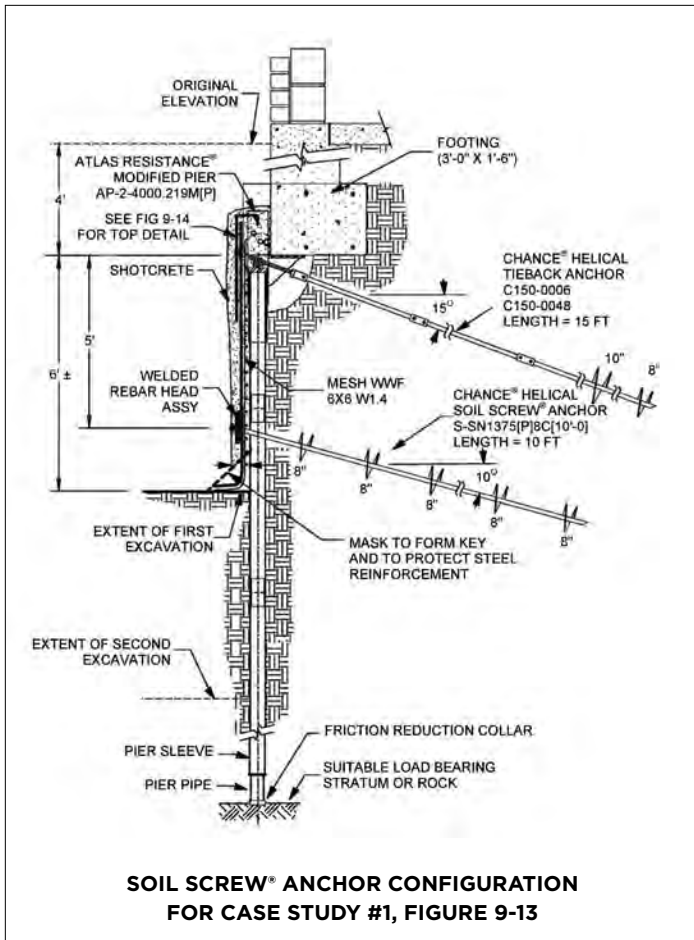
Based on the requirement of installing Atlas Resistance® Modified Piers to a tested load resistance of at least 50% higher than the design load leads to:

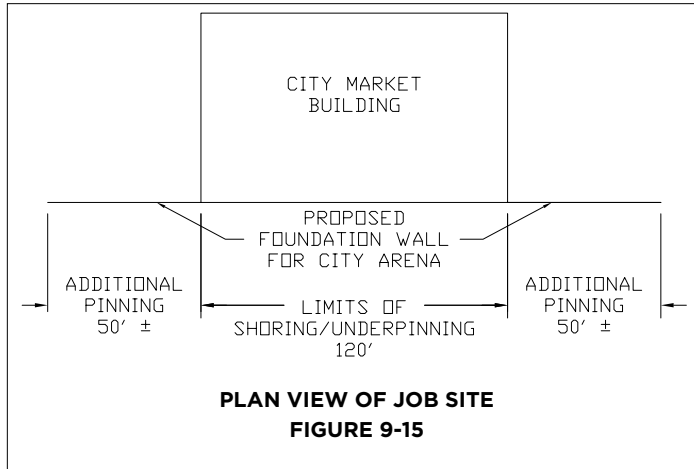
EQUATION 9-6	
DS	= 15,000 (1.5)
	= 22,500 lbs

For this requirement, the Atlas Resistance® AP-2-3500.165[PA] M 2-Piece Modified Pier is selected. The modified pier has a 3-1/2" diameter pier pipe and has an ultimate capacity of 91,000 lbs. "M" indicates the use of 4" diameter sleeving over the pier pipe. The sleeved portion of the pier shall extend down to a depth of 21 feet (six lengths of sleeve pipe). "PA" indicates the product is manufactured of mill finish steel (plain) with flow coated corrosion protection of the pier pipe. Since this is temporary construction, the corrosion protection is unnecessary; however this product is supplied with corrosion protected pipe as standard. Details of the underpinning and tieback anchorage are shown in Figure 9-16.



RETENTION WALLS





INTEGRATED TIEBACK SYSTEM - CHANCE® HELICAL TIEBACK ANCHORS

Although the footing line load is less than the 4,000 lb/ft criteria, the depth of the cut to be shored is 20 feet. This exceeds the recommended 12 foot limitation and as such a Chance® Helical Tieback Anchor must be selected for use with each modified pier placement. For this situation Type SS5 1-1/2" square shaft Lead Section and Extension are the recommended components.

The lead section consists of one 8" and one 10" diameter plate welded to a 1-1/2" square shaft. Minimum installed length is estimated to be 15 feet. Installed angle is 12° down from horizontal. Installed torque is estimated to be 1,800 ft-lb minimum. No corrosion protection is required since the construction is temporary.

SOIL SCREW® SHORING SYSTEM - CHANCE® HELICAL SOIL SCREW® ANCHORS

Because the depth of cut is 20 feet from grade (17 feet below the bottom of the footing of the market building), three Helical Soil Screw® Anchors are required. In this case a Factor of Safety of 1.5 was used because the existing market building is relatively light. In conducting the soil analysis, it was assumed that the Chance® Helical Tieback Anchor does not contribute to the holding capacity of the body mass of soil. As in Case Study 1, internal stability analysis is typically done with commercially available software such as SNAILZ (Caltrans) or GoldNail (Golder Associates), and Soil Screw® Anchor tension capacity is calculated with HeliCAP® Helical Capacity Design Software and input into the stability analysis software. In this project, the shear strength is from the frictional nature of the cohesionless soil (silty sand) and its magnitude is related to the friction angle ($\phi = 30^\circ$ in this case).

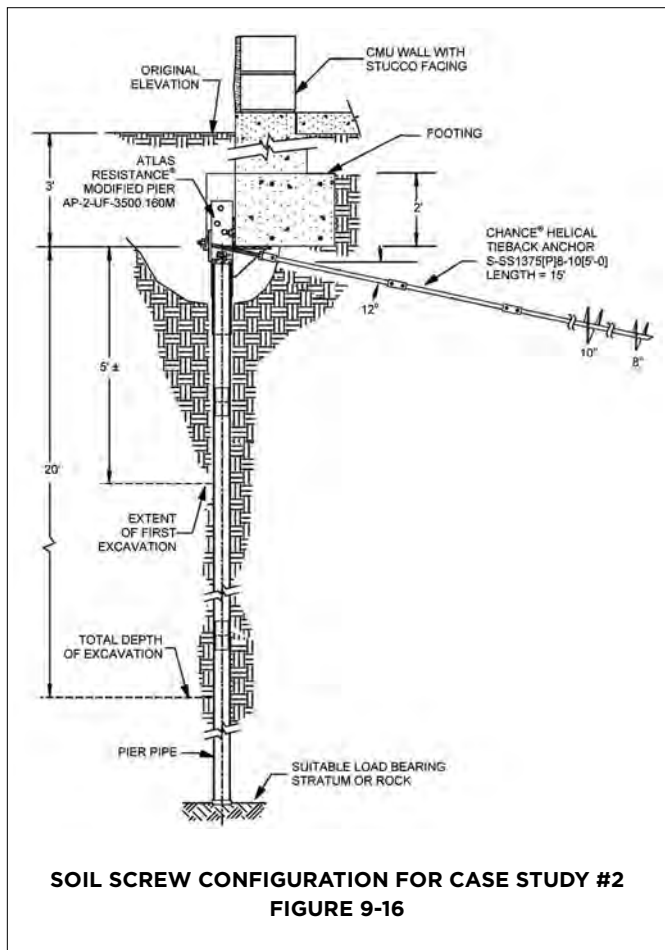
As described in the Chance® Soil Screw® Retention Wall System Design Manual, Soil Screw® Anchors add to the resisting force along the movement plane. In this case, however, the indicated force (T) is the resultant of all three rows of Helical Soil Screw® Anchors. Placement of the three rows of Helical Soil Screw® Anchors is shown in Figure 9-18. The value for the ultimate holding capacity required (including the Factor of Safety) is:

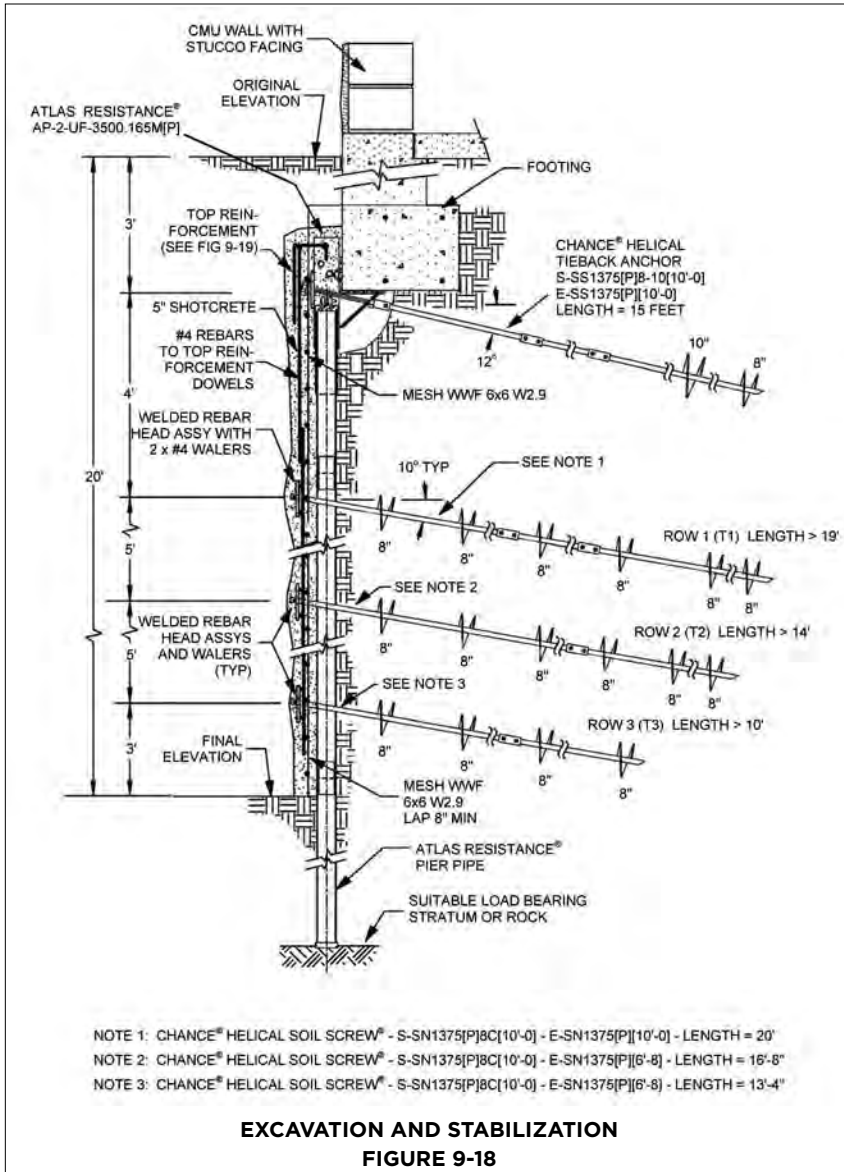
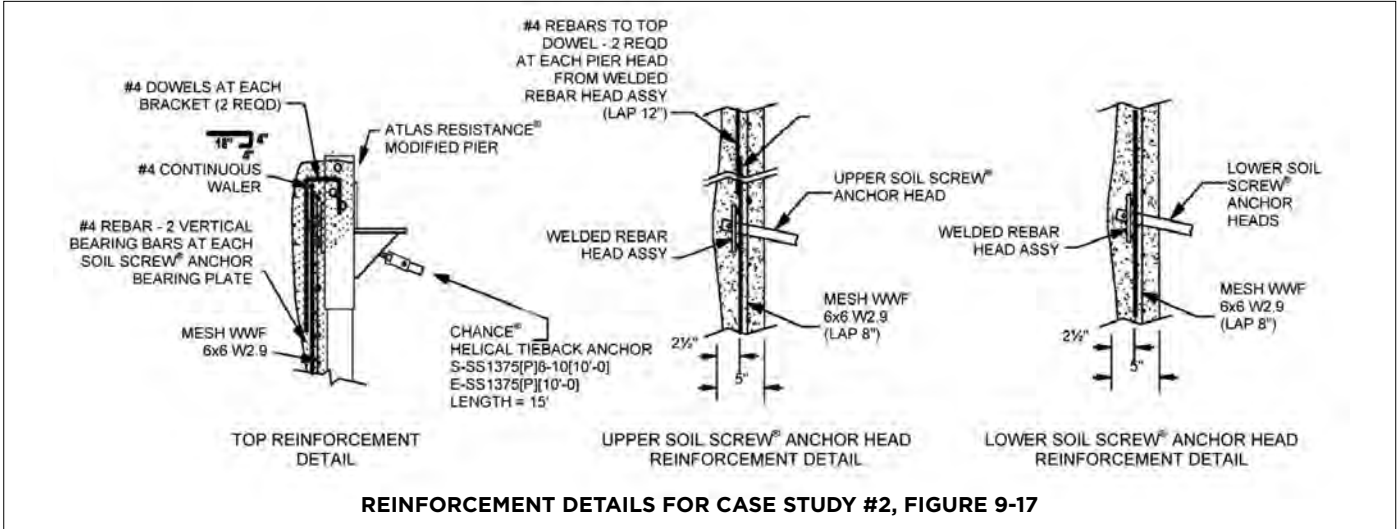
EQUATION 9-7

$$T = T_1 + T_2 + T_3$$

The results of extensive testing of soil nail walls indicate that the top row of soil nails or screws is most heavily loaded with the successively lower rows having lesser holding capacity requirements. The following are the recommended Chance® Helical Soil Screw® Systems for this project:

- Soil Screw® Anchor Row #1 (T1): C2200691 Lead and two C1100689 Extensions. The Soil Screw® Anchor has continuously spaced 8" diameter plates along the entire length of a 1-1/2" solid square steel shaft. The Soil Screw® Anchor will be installed to a minimum length of 19 feet, 10° down from horizontal and to an estimated torque of 2,500 ft-lbs.
- Soil Screw® Anchor Row #2 (T2): C2200691 Lead and one C1100689 Extension. The Soil Screw® Anchor has





continuously spaced 8" diameter plates along the entire length of a 1-1/2" solid square steel shaft. The Soil Screw® Anchor will be installed to a minimum length of 14 feet, 10° down from horizontal and to an estimated torque of 1,800 ft-lbs.

- Soil Screw® Anchor Row #3 (T3): C1100692 Lead and C1100690 Extension. The Soil Screw® Anchor has continuously spaced 8" diameter plates along the entire length of a 1-1/2" solid square steel shaft. The Soil Screw® Anchor will be installed to a minimum length of 10 feet, 10° down from horizontal and to an estimated torque of 1,000 ft-lbs.

SHOTCRETE WALL

The shotcrete wall is a temporary facing for the excavation. Since the soil analysis assumed that the Chance® Helical Tieback Anchors do not contribute to the holding capacity of the body mass of soil (see Figure 9-8), the Chance® Helical Soil Screw® Anchors were designed to hold the total body mass. The bottom cantilever should be limited to 2/3 of the typical spacing for the Soil Screw® Anchor row, but should not exceed 3 feet. In this case the cantilever is 3 feet.

Vertical bearing bars are extended from the welded rebar head assemblies at the upper row of Soil Screw® Anchors to the dowels and waler at the top of the wall in order to augment the selected shotcrete wall thickness (5"). Welded wire fabric reinforcing (WWF 6x6 W2.9 or equivalent) is spaced midway within the shotcrete wall at a 2-1/2" nominal depth. The top wall segment is checked for flexure and shear using the distributed Soil Screw® Anchor head forces

RETENTION WALLS



and one-way beam action. Two #4 reinforcing bar walers are placed continuously along each Soil Screw® Anchor row (see Figures 9-17 and 9-18).

SOIL SCREW® ANCHOR HEAD DESIGN

The Soil Screw® Anchor head forces are expected to be approximately 1/2 of the Soil Screw® Anchor tension load. The shotcrete facing is checked for flexure and punching shear using two-way slab action. This information is used in the internal stability analysis. A wall plate could have been placed at the wall face to maximize punching shear resistance, but in this example a welded rebar head assembly that includes a wall plate is placed on each Helical Soil Screw® Anchor at the middle of the shotcrete wall as shown in Figure 9-18 (refer to Soil Screw® Anchor Wall Accessories for details of the welded rebar head assembly). The welded rebar head assembly shall be spliced to the horizontal walers at each row of Helical Soil Screw® Anchors and to the vertical bearing bars between the upper row of Helical Soil Screw® Anchors and the dowels at the pier brackets. To properly position and embed the welded rebar head assembly and rebar, the welded wire fabric must be pushed into the initial 2-1/2" face coat of shotcrete approximately 1/2" at each Soil Screw® Anchor head. The 5" wall thickness and reinforcement described above are adequate.

REFERENCES:

1. AASHTO Highway Subcommittee on Bridges and Structures, Manual on Foundation Investigations, American Association of State Highway and Transportation Officials, 1978.
2. Federal Highway Administration Publication No. FHWA-SA93-026, Recommendations Clouterre, English Translation, 1993.
3. Federal Highway Administration Publication No. FHWA-SA-96-069, Manual for Design and Construction Monitoring of Soil Nail Walls, 1996.
4. Federal Highway Administration Publication No. FHWA-SA-96-071, Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines, 1996.
5. Federal Highway Administration Publication No. FHWA-SA-96-072, Corrosion/Degradation of Soil Reinforcement for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, 1996.



SECTION 10: POLE BASE FOUNDATIONS

CONTENTS

Introduction.....	10-2
Design.....	10-2
Laterally Loaded Foundations	10-6
Instant Foundations® System Specifications	10-7
Product Specifications.....	10-7
Pole Load Determination Data Sheet.....	10-9

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 should be conducted and state and local building codes and authorities should be consulted 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 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

Hubbell Power Systems, Inc. manufactures pole base foundations to provide resistance to lateral loads and moment loads due to wind and other load conditions. The versatility and ease of construction of these pole base foundations, known as Chance® Instant Foundations®, permits great flexibility in a number of applications. Typical uses for these products are foundations for equipment pads, foundation supports for signs, supports for light standards and decorative poles, EV charging station foundations, 5G pole foundations, and other eccentric load applications.

PRODUCT BENEFITS

Instant Foundations offer the following benefits:

- Fast installation.
- No vibration.
- Ease of installation in limited access areas.
- Minimum disturbance to site.
- No excavation required.
- All steel foundation.
- Immediate structure installation.
- Ready for immediate wiring.
- All weather installation.
- On-site load test capability.

This section describes the Chance Instant Foundations system products for overturning moment loads and lateral support that are typically maintained in stock to provide quick delivery to the project site. Table 10-1 and Figure 10-1 illustrate just a few of the Instant Foundations products that are available in each of the product series. Our manufacturing facility is capable of rapidly fabricating products to suit the application. For pole bases that require bumper/impact protection, a pre-cast collar option is available (see Figure 10-2)

INSTANT FOUNDATIONS SYSTEM PRODUCT SELECTION, TABLE 10-1

DETAIL	CATALOG NO	PILE DIA	LENGTH
A	T1120143	3-1/2"	5' - 0"
B	T1120338	4"	4' - 8"
C	C11232JG4VL	6-5/8"	5' - 0"
D	C11242NG4VP	8-5/8"	5' - 0"
E	T1120592	10-3/4"	5' - 0"

Notes:

1. Manufacturer to have in effect industry recognized written quality control for all materials and manufacturing processes.
2. All material to be new, unused and mill traceable meeting specifications found on product drawing.
3. Additional lengths and configurations are available as standard catalog numbers.

DESIGN

RECOMMENDED FACTORS OF SAFETY FOR DESIGN

The variability of soil conditions that may exist at a project site, plus the varied nature of loading on structures and how these loads are transferred through foundation elements, requires the consulting engineer and/or dealer/ installing contractor to use an appropriate factor of safety (FS) in design for use with the Chance® Instant Foundations® system. Generally this factor of safety is a minimum of 2:1 on all permanent loading conditions and a minimum of 1.5:1 For any temporary load situation. National and local building code regulations may require more stringent factors of safety on certain projects.

DESIGN GUIDELINES

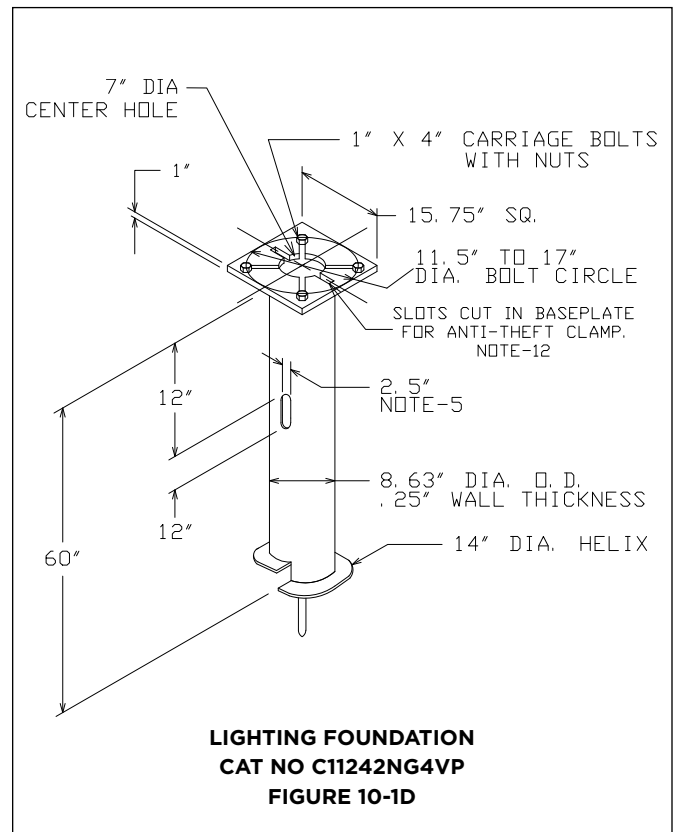
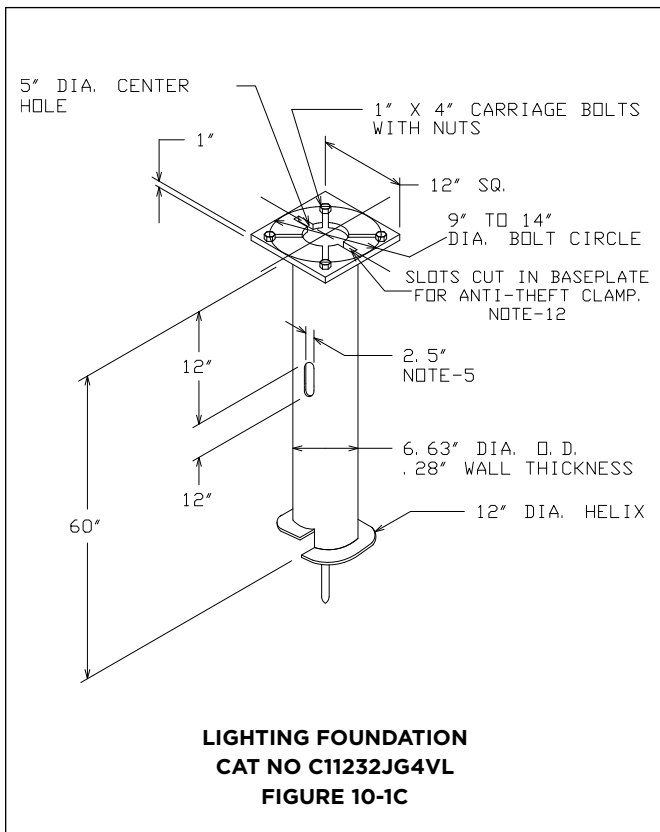
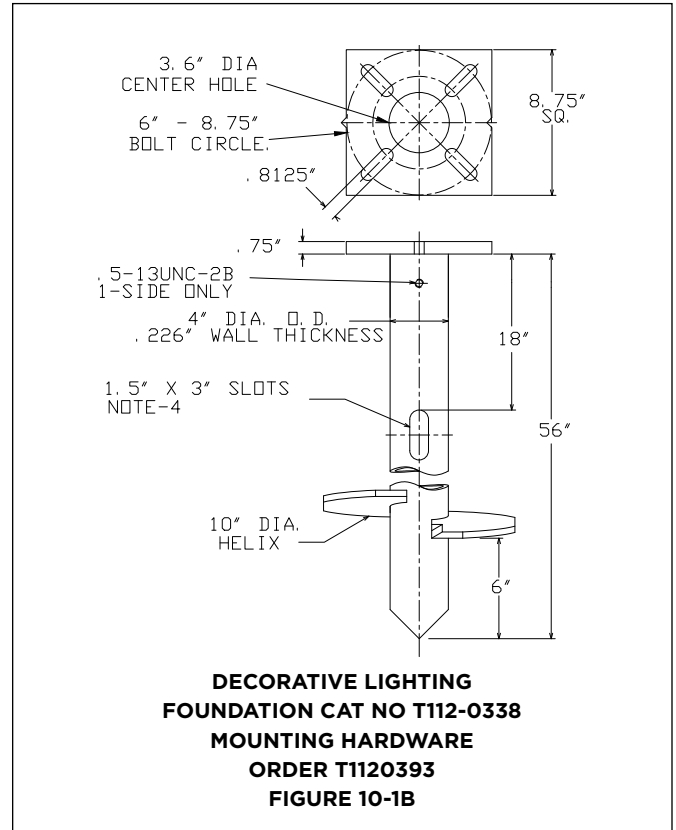
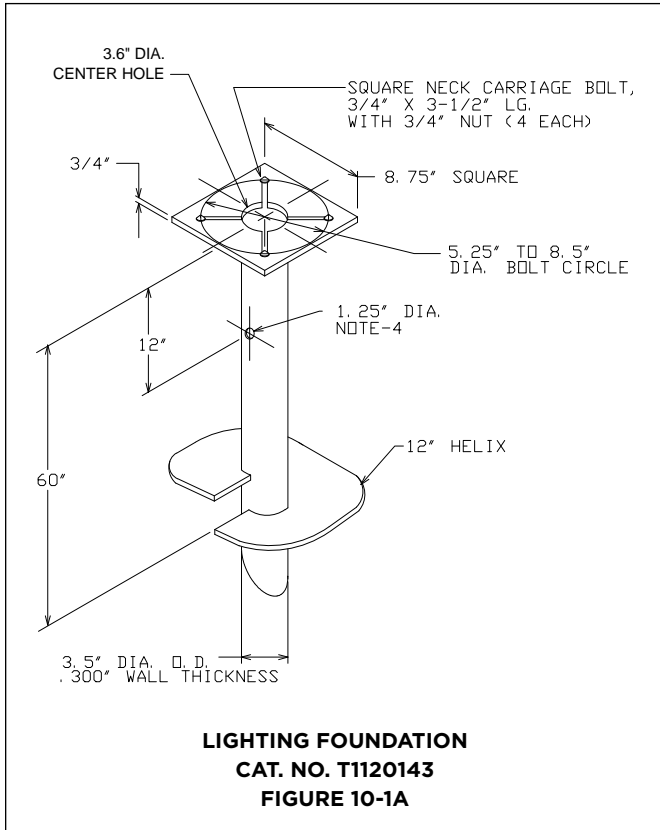
Instant Foundations provide manufactured single helix fixed length products for use as foundations for varied applications such as light poles, signs and equipment supports. There are many applications for these tubular helical specialty products. Each application will require:

1. An evaluation of the soil strata and soil characteristics of that stratum in which the product will be installed.
2. A selection of the appropriate Instant Foundation product shaft diameter, shaft length, base plate size, bolt diameter and bolt circle diameter.
3. A determination of the ultimate bearing capacity and suitable Factor of Safety.

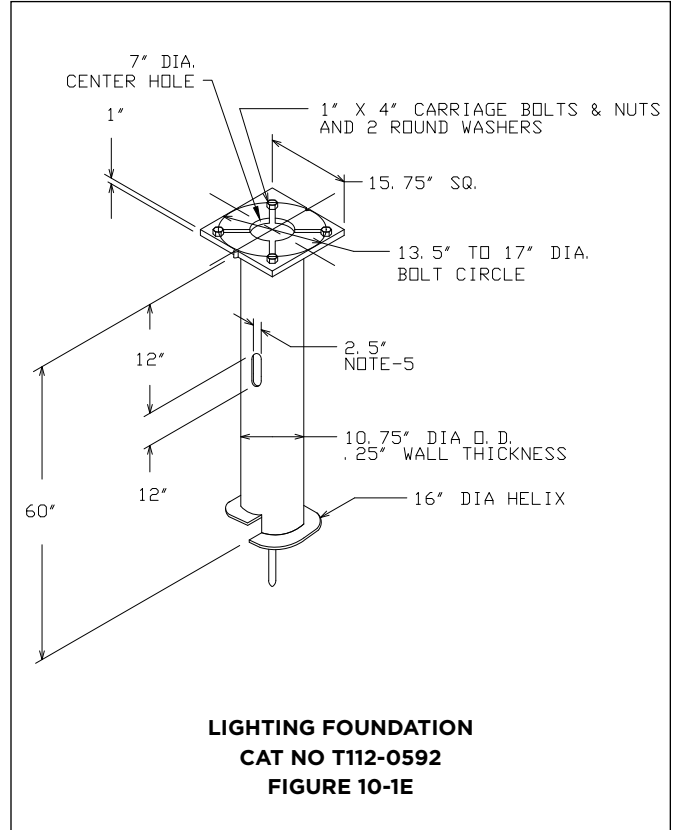
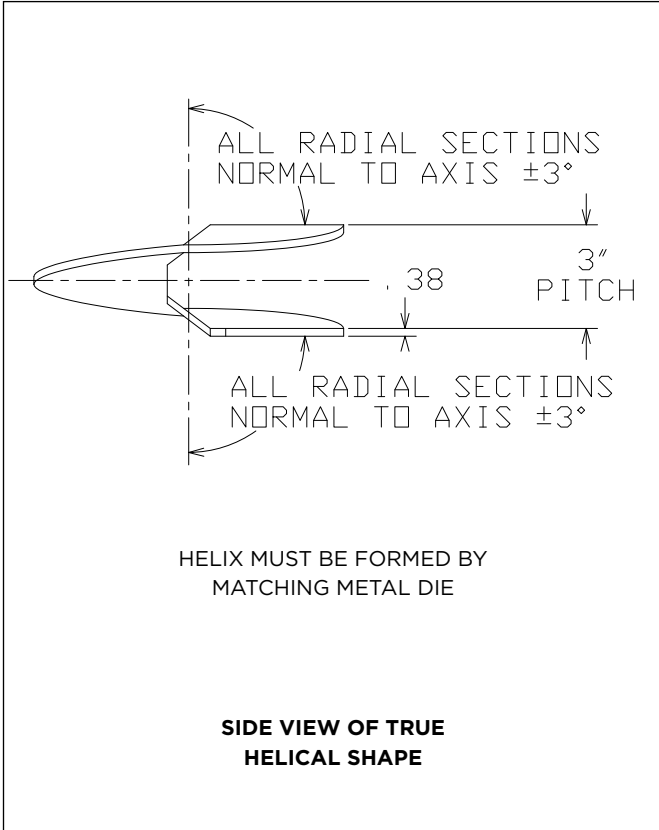
NOTE: The design should involve professional geotechnical and engineering input. Specific information involving the structures, soil characteristics and foundation conditions must be used for the final design.

The following preliminary design guide information is intended to assist dealers, installing contractors, and consulting engineers to select the appropriate Chance pole base foundation to resist overturning moment and lateral load.

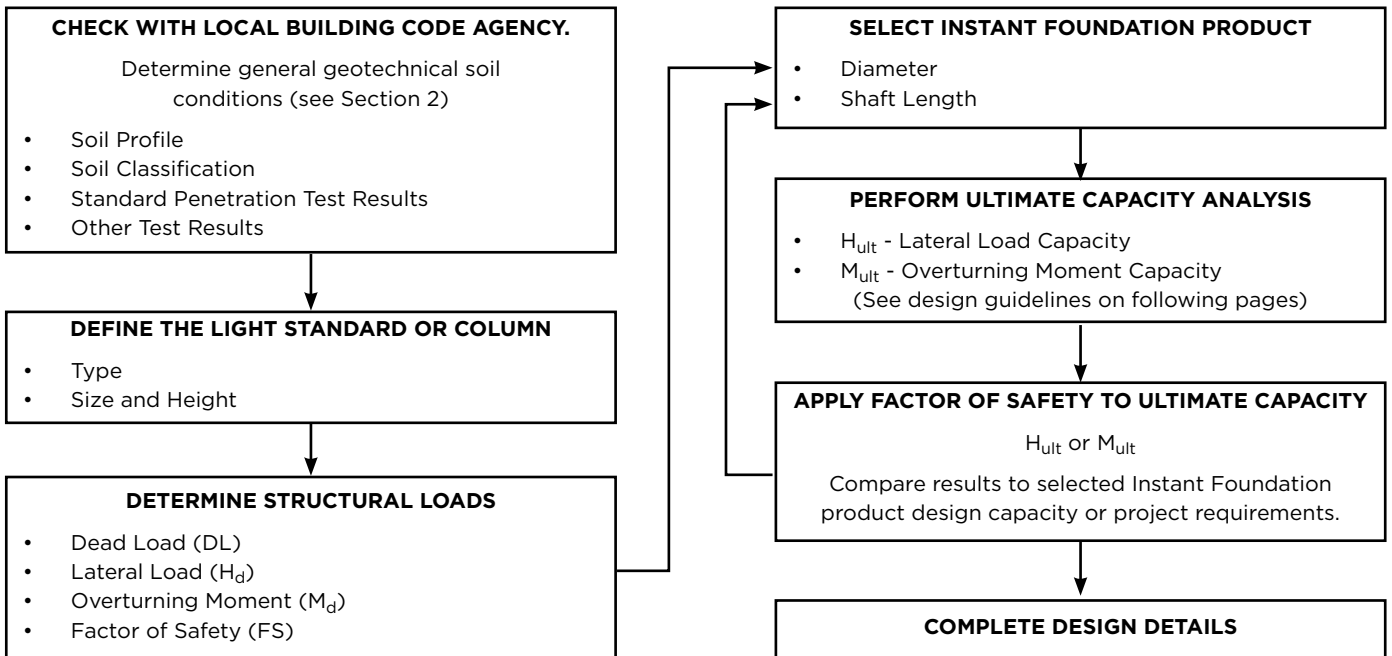
The Hubbell Power Systems, Inc. Pole Load Determination Data Sheet is provided in this section. This can be used to gather and record the information required to determine the loads to be applied to a light pole foundation. The loads and given soil conditions are then used to determine the appropriate Instant Foundation product size required for the job. The Select-a-Base™ Lighting Base Program is an on-line program used for preliminary foundation selection. The program incorporates a database of Chance pole bases. 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.hpsapps.com.



POLE BASE FOUNDATIONS



INSTANT FOUNDATION SYSTEM DESIGN STEPS



LIGHT POLE STANDARDS PRODUCTS

Chance® Instant Foundation® products for light pole standards are designed to resist both the lateral forces and overturning moments from wind loads. Controlling design standards for wind loads can be determined either by consulting local or national building codes or conformance to standards set by the American Association of State Highway and Transportation Officials (AASHTO). These standards will provide the required design wind load based on geographic region and the factors associated with the shape and type of structure in order to determine the resulting wind pressure. This wind pressure is then applied to the effective projected area (EPA) of the light pole, arm and fixture. These lateral forces can be used to determine the resultant lateral force and overturning moment applied to the foundation as shown in Figure 10-4. The luminaire or fixture supplier may be consulted to determine the actual effective projected area for the specific light assembly.

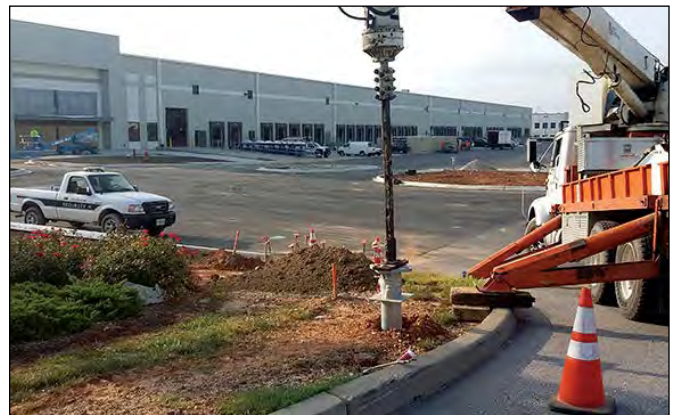
Table 10-2 provides the suggested shaft diameter and installation requirements for various lateral load-overturning moment ranges. Table 10-3 provides the minimum recommended design life based on the structure type. This has been reproduced from AASHTO Specification, 4th edition, 2001. The designer can make a site-specific analysis, or an analysis can be obtained by completing the Pole Load Determination Data Sheet on page 10-9 and submitting it to Hubbell Power Systems, Inc. to determine the most appropriate Instant Foundation® Product.

CHANCE® INSTANT FOUNDATION® PRODUCTS, TABLE 10-2

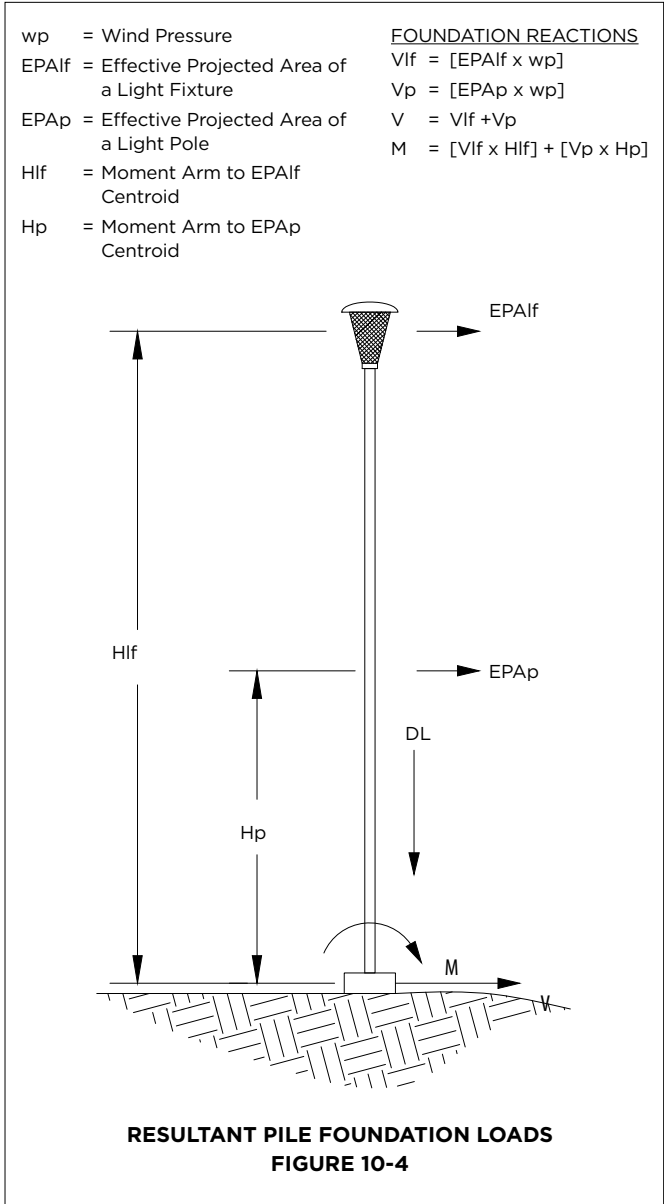
Design Lateral Load ²	Design Overturning Moment ²	Recommended Helical Foundation ²	Product Part Number
150 - 500 lb.	≤ 2,800 ft-lb.	3.5" Dia x 5' Long	T1120143
150 - 500 lb.	≤ 3,500 ft-lb.	4" Dia x 4'-8" Long	T1120338
500 - 1,000 lb.	≤ 10,500 ft-lb.	6-5/8" Dia x 5' Long	C11232JG4VL
1,000 - 1,200 lb.	≤ 21,000 ft-lb.	8-5/8" Dia x 5' Long	C11242NG4VP
1,200 - 1,500 lb.	≤ 37,000 ft-lb.	10-3/4" Dia x 5' Long	T1120592
Notes:			
1. The above lateral loads and overturning moments are mechanical ratings of the indicated foundation. Project soil conditions must be evaluated during preliminary design.			
2. These design loads are based on allowable bending in the pipe shaft with cableway widths of 1.25" in 3.5" dia, 1.5" in 4" dia and 2.5" in all other foundations.			



**INSTALLED LIGHT STANDARD
FIGURE 10-2**



**INSTANT FOUNDATIONS ARE INSTALLED USING
COMMON CONSTRUCTION EQUIPMENT
FIGURE 10-3**

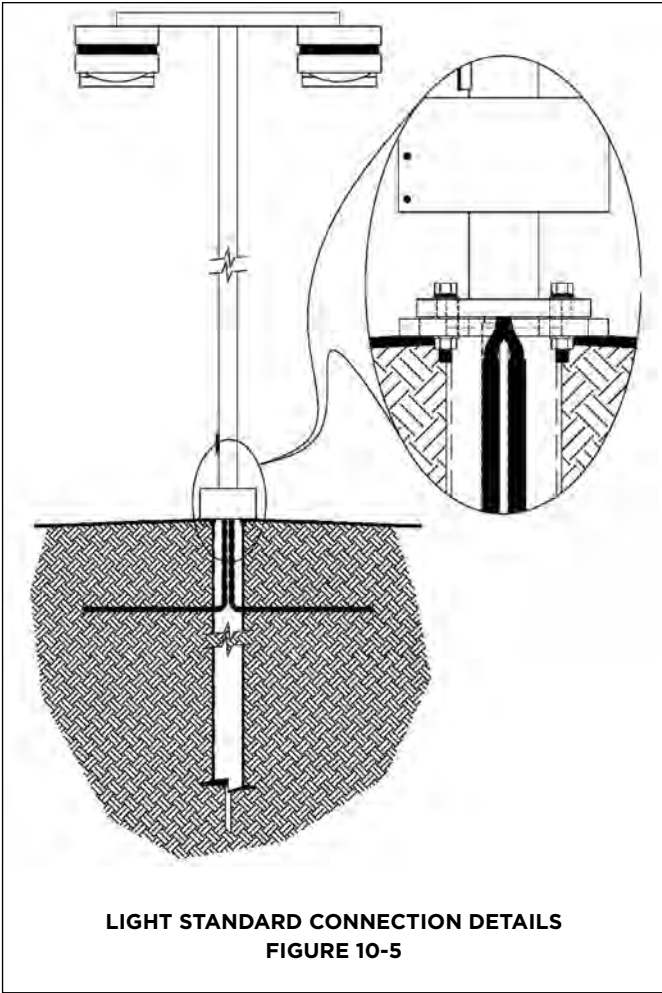


3. 5" dia, 1.5" in 4" dia and 2.5" in all other foundations.

RECOMMENDED MINIMUM DESIGN LIFE, TABLE 10-3

Design Life	Structure Type
50 Years	<ul style="list-style-type: none"> Luminaire support structures exceeding 15m (49.2 ft) in height. Overhead sign structures.
25 Years	<ul style="list-style-type: none"> Luminaire support structures less than 15m (49.2 ft) in height. Traffic signal structures.
10 Years	<ul style="list-style-type: none"> Roadside sign structures.

(Reproduced from AASHTO Specification, 4th edition, 2001)



LATERALLY LOADED FOUNDATIONS

Certain projects require a rapidly installed foundation that must resist lateral loads. Examples of these projects include:

- Equipment platforms for communication towers or mechanical systems.
- Seaside structures subjected to wave action.
- Temporary classroom/mobile building foundations.
- Solar Panels

Each project must be evaluated and designed and should include geotechnical and professional engineering input. Hubbell Power Systems, Inc. offers a "Preliminary Design Service" for evaluating the feasibility of using Instant Foundation® products on such specific projects.



INSTANT FOUNDATIONS® SYSTEM SPECIFICATIONS

The Specification at the end of this section provides a typical specification for a Chance® Instant Foundation.

1. American Association of State Highway and Transportation Officials (AASHTO) Specification, 6th Edition, 2013.
2. Uniform Building Code, Volume 2 - Division 3, 1997.

PRODUCT SPECIFICATIONS

CHANCE® INSTANT FOUNDATION® PRODUCTS

- 3-1/2" Dia x 0.300" Wall
- 4" Dia x 0.226" Wall
- 6-5/8" Dia x 0.280 Wall
- 8-5/8" Dia x 0.250" Wall
- 10-3/4" Dia x 0.250" Wall

The usual application for this foundation is where loads are moderate and the project requires greater column stiffness than is possible with the typical square shaft helical pile. Examples of applications are: Light Standards, Curbside Business Sign Support, electrical/Mechanical equipment Pad Support, Cantilevered Loads, etc.

PART 1 - GENERAL

1.1 SCOPE OF WORK

This work consists of furnishing labor, tools, equipment and materials associated with the preparation and installation of the Chance® Instant Foundations® for structural foundation support according to the specifications contained herein. The work includes, but is not limited to, the following:

1. Diligent investigation of the possible existence and location of underground utilities situated at or near the area of work;
2. Excavation and preparation of foundation soil to grade for foundation installation;
3. Mounting of the hydraulic gear motor on a backhoe unit or similar auxiliary powered equipment, and the installation of the Instant Foundations® product to the required torque resistance at the required depth (if torque resistance measurement is required).
4. Removal of the hydraulic gear motor.
5. Conducting an optional Field Load Test on one or more Instant Foundations® products.
6. Clean Up.

1.2 REFERENCES

1. Building Officials and Code administrators International, Inc. (BOCA) Basic National Building Code.
2. American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals.

1.3 DELIVERY, STORAGE AND HANDLING

All foundation products shall be handled and transported carefully to prevent any deformation or damage. Care should be taken to prevent the accumulation of dirt, mud or other foreign matter on the steel materials. Such accumulation shall be completely removed prior to installation.

PART 2 - MATERIAL

2.1 HYDRAULIC GEAR MOTOR

The torque rating of the hydraulic gear motor used to install the Instant Foundations® product shall be adequate to install the required foundation. It is suggested that the torque rating be 25 percent higher than the planned installation torque. Depending upon the soil conditions and pile configuration, different hydraulic gear motors may be required.

2.2 3-1/2" AND 4" DIAMETER HELICAL INSTANT FOUNDATIONS® SERIES

2.2.1 Foundation Shaft Section

The shaft section consists of a tubular hot rolled steel pile section 3-1/2" in diameter with a 0.300" wall thickness, or 4" diameter with a wall thickness of 0.226" conforming to ASTM A-53, A-252 and A-500. The length of the foundation shall be as specified: 4', 4'-8", 5', etc. The lead end of the 3.5" and 4" foundations shall have a single or double bevel cut to aid in starting the foundation installation. Welded to the shaft shall be one ASTM A-635 steel helical plate with a thickness of 3/8" and a 3" pitch.

2.2.2 Foundation System Base Mounting Plates

Foundation base plates may be round or square, of various sizes in plan view and may vary in thickness from 1/2" to 1-1/2" depending on job requirements.

2.3 6-5/8", 8-5/8" AND 10-3/4" DIAMETER HELICAL INSTANT FOUNDATIONS® SERIES

2.3.1 Foundation Shaft Section

The shaft section consists of 6" diameter (6-5/8" outside diameter with 0.280" wall), 8" diameter

(8-5/8" outside diameter with 0.250" wall) or 10" (10-3/4" outside diameter with 0.250" wall) steel pipe conforming to ASTM A-53, A-252 or A-500. The length of the foundation may be 4', 5', 7', 8' or 10' long as required by the application. The pile section shall have two wire access slots located 180° from each other. The integral foundation cap plate shall have an alignment notch located

Directly above one of the wire access slots. Welded to the lead end of the foundation shaft shall be a steel helical plate with a 3" pitch. To aid in starting the pile, a 1-1/4" diameter steel rod shall extend beyond the center of the helix to provide a pilot.

2.3.2 FOUNDATION SYSTEM BASE MOUNTING PLATES

Foundation base plates may be round or square, of various sizes in plan view and may vary in thickness from 3/4" to 1-1/2" depending on job requirements.



2.4 WELDMENTS

All welded connections shall conform to the requirements of the American Welding Society Structural Welding Code, AWS D1.1 and applicable revisions.

PART 3 - EXECUTION

The following is intended to provide the controlling specification for the major steps undertaken in the installation of the Chance® Instant Foundation® systems. Variations in the installation procedure may occur depending on the application and the structural support required.

WARNING! THOROUGHLY INVESTIGATE THE POSSIBLE EXISTENCE AND LOCATION OF ALL UNDERGROUND UTILITIES SITUATED AT OR NEAR THE AREA OF WORK BEFORE PROCEEDING. SERIOUS INJURY MAY RESULT FROM FAILURE TO LOCATE ALL UNDERGROUND UTILITIES.

3.1 PREPARATION

The soil shall be excavated to the proper grade for placement of the Chance® Instant Foundations® product. Stakes should be set at each foundation location prior to commencement of work. The foundation layout and staking should be under the supervision of the responsible structural engineer and be accomplished using fully qualified and trained technicians familiar with foundation layout.

3.2 INSTALLATION OF THE INSTANT FOUNDATIONS® PRODUCT

The hydraulic gear motor shall be installed on a backhoe or other suitable pile installation unit. Mount the Instant Foundations® Product to the hydraulic gear motor via the appropriate Kelly bar adapter and installing tool using two structural grade bolts and nuts. The foundation is positioned vertically over a marked pile location and driven into the soil by means of the hydraulic gear motor. Rotary installation continues until the required design torque is achieved at or below the predetermined depth. The baseplate is typically installed to grade or slightly above to allow clearance for bolt mounting of the pole base. It is important that the installation torque remain at or above the predetermined value during this process. Details of the installation shall be provided to the supervising engineer for review.

3.3 DOCUMENTATION

When required, the dealer/installing contractor shall monitor the torque applied to the foundation during installation. It is suggested, but not mandatory that the installation torque be recorded at one-foot intervals throughout the installation. The installation torque may be measured with a calibrated torque indicator. At the conclusion of the installation, a copy of the foundation installation record shall be provided to the engineer for review.

3.4 LOAD TEST (OPTIONAL)

A detailed description on the requirements and procedures for conducting a Load Test may be found in Appendix B (Load Tests). The results of the Field Load Test provide guidance for determining the ultimate and allowable foundation loads.

Load testing should be conducted under the supervision of the responsible engineer.

Depending on the project specifications, a Working Load Test may be required. Normally, the first installed foundation is selected for this test; however, some specifications require ultimate loading of the foundation. If an Ultimate Load Test is required, a test foundation must be installed in an alternate location on the site in addition to the pile locations marked. After the Ultimate Load Test is completed, the test foundation may be removed from the soil and used on the project, provided it is not damaged.

3.5 CLEAN UP

Upon completion of the installation of the Chance® Instant Foundations® product, all equipment shall be removed from the site. Any disturbed soils in the area of the foundation shall be restored to the dimensions and condition specified by the engineer.

END OF SPECIFICATION



Need by: _____

Pole Load Determination Data Sheet

Contact Name: _____ Phone: _____ Date: _____

Job Name: _____ Job Location: _____

***Select Appropriate Unit of Measure**

1. Luminaire Mounting Height: _____ m ft
2. Height of Pole: _____ m ft
3. Outside Diameter of Pole Top: _____ cm in
4. Outside Diameter of Pole Bottom: _____ cm in
5. Arm Length: _____ m ft
6. Arm Tip Outside Diameter: _____ cm in
7. Arm Bottom outside Diameter: _____ cm in
8. Luminaire Weight: _____ kg lb
9. Luminaire EPA (Projected Area x C_d): _____ m² ft²
10. Basic Wind Speed: _____ kph mph
11. Minimum Design Life (Select Choice): 10 25 50
Design life default is 25 years. See Table 3-3, below
12. Number of Arms: _____
13. Number of luminaires: _____
14. Pole Shape (Select choice from list below)

<input type="checkbox"/> Cylinder	<input type="checkbox"/> Hecdecagonal (16 Sides)	<input type="checkbox"/> Octagonal (8 Sides)
<input type="checkbox"/> Flat	<input type="checkbox"/> Dodecagonal (12 Sides)	<input type="checkbox"/> Square (4 Side) <input type="checkbox"/> Diamond
15. Arm Shape (Select choice from list below)

<input type="checkbox"/> Cylinder	<input type="checkbox"/> Hecdecagonal (16 Sides)	<input type="checkbox"/> Octagonal (8 Sides)
<input type="checkbox"/> Flat	<input type="checkbox"/> Dodecagonal (12 Sides)	<input type="checkbox"/> Square (4 Side) <input type="checkbox"/> Diamond
16. Anchor Bolt Diameter: _____ cm in
17. Number of Bolts (in base plate): _____
18. Bolt Circle Diameter: _____ cm in
19. Special Cableway Requirements: _____
20. Site Soil Conditions (if available): _____
21. CHANCE Precast Concrete Collar: No Yes - height: 12" H 24" H Other: _____
22. Solar Panel/Banner: Qty _____ Dimension ____"H x ____"W Distance from ground? _____
Location on Pole (one side or both) _____

Table 3-3. Recommended Minimum Design Life	
<i>Reproduced from AASHTO Specification, 4th Edition, 2001</i>	
Design Life	Structure Type
50 Years	-Luminaire support structures exceeding 15m (49.2 ft.) -Overhead sign structures
25 Years	-Luminaire support structures less than 15m (49.2 ft.) in height -Traffic signal structures
10 Years	-Roadside sign structures



CORROSION - AN OVERVIEW APPENDIX A

CONTENTS

Introduction.....	A-2
Corrosion Theory	A-2
Soil Environments	A-3
Predicting Corrosion Loss	A-5
Corrosion Loss Rates.....	A-6
Field Measurement of Soil Resistivity	A-7
Corrosion Control Techniques.....	A-8
Design Examples.....	A-13

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 should be conducted and state and local building codes and authorities should be consulted 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 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

Corrosion is defined as the degradation of a material or its properties due to a reaction with the environment. Corrosion exists in virtually all materials but is most often associated with metals. Metallic corrosion is a naturally occurring process in which the surface of a metallic structure is oxidized or reduced to a corrosion product such as rust by chemical or electrochemical reaction with the environment. The surface of metallic structures is attacked through the migration of ions away from the surface, resulting in material loss over time. Given enough time, the material loss can result in significant reduction of area, which in turn leads to a reduction in the structural capacity of a given metallic element. When corrosion eventually destroys a sufficient amount of the structure's strength, a failure will occur.

The corrosion mechanisms involved with buried metallic structures are generally understood, but accurate prediction of metal loss rates in soil is not always easily determined. This appendix provides an introduction to the concepts of underground corrosion and the factors that influence this corrosion in disturbed and undisturbed soils. A few design examples are provided to give the reader a better understanding as to whether corrosion is a critical factor in a Chance® Helical Pile/Anchor or Atlas Resistance® Pier applications. This section is not intended to be a rigorous design guide, but rather a "first check" to see if corrosion is a practical concern given the specific project site conditions. A qualified corrosion engineer should be consulted for a site specific recommendation if steel foundation products are to be used in a known corrosive soil.

Experience over the past 60 years has shown the vast majority of square shaft and round shaft helical anchors/ piles have a calculated service life well in excess of the design life of the structure (typically 50 to 75 years in the North America). In highly corrosive soils and areas of stray currents (e.g., underground transmission pipelines, DC railroads) additional measures must be taken to protect steel foundation products. In these cases, active protective measures such as sacrificial anodes are employed.

CORROSION THEORY

To understand why metallic corrosion occurs, it is necessary to understand how a metal, such as carbon steel, is formed. During the steel making process, natural low energy iron ore is refined into metal. This process adds a great deal of energy to the metal. When the steel is placed into a corrosive environment, it will by natural processes, return to its low energy state over time. To make the return trip, the steel must give up the energy gained at the mill. This is the essence of the reduction process that we call corrosion.

Mechanical strength, physical size and shape, and chemical composition of the steel are all properties that must be considered when designing Chance® Helical Pile/Anchor or Atlas Resistance® Piers. Mechanical and physical properties are well defined and controlled during the manufacturing process. This is also true of the chemical composition, primarily due to the superior process controls used by the steel mills. Of the three properties, chemical composition is the primary factor with respect to corrosion.

Corrosion of steel is an electrochemical process. Romanoff (1957) stated:

"For electrochemical corrosion to occur there must be a potential difference between two points that are electrically connected and immersed in an electrolyte. Whenever these conditions are fulfilled, a small current flows from the anode area through the electrolyte to the cathode area and then through the metal to complete the circuit, and the anode area is the one that has the most negative potential, and is the area that becomes corroded through loss of metal ions to the electrolyte. The cathode area, to which the current flows through the electrolyte, is protected from corrosion because of the deposition of hydrogen or other ions that carry the current.

"The electrochemical theory of corrosion is simple, i.e., corrosion occurs through the loss of metal ions at anode points or areas. However, correlation of this theory with actual or potential corrosion of metals underground is complicated and difficult because of the many factors that singly or in combination affect the course of the electrochemical reaction. These factors not only determine the amount or rate at which corrosion occurs but also the kind of corrosion."

Depending on the many factors that affect the electrochemical reaction, corrosion can affect a metal in several different ways. Some of these types are listed below:

CORROSION TYPES, TABLE A-1

TYPE	CHARACTERISTICS
Uniform or Near Uniform	Corrosion takes place at all area of the metal at the same or a similar rate.
Localized	Some areas of the metal corrode at different rates than other areas due to heterogeneities in the metal or environment. This type of attack can approach pitting.
Pitting	Very highly localized attack at specific areas resulting in small pits that may penetrate to perforation.

Considerations need to be applied as to the types and rates of corrosion anticipated. Current theory does not permit accurate prediction of the extent of expected corrosion unless complete information is available regarding all factors. Therefore, uniform corrosion will be the corrosion type discussed herein.

Romanoff states there are several conditions that must be met before the corrosion mechanism takes place. These are:

ELECTRICAL FACTORS

Two points (anode and cathode) on a metallic structure must differ in electrical potential. The anode is defined as the electrode of an electrochemical cell at which oxidation occurs, i.e., the negative terminal of a galvanic cell. The cathode is defined as the electrode of an electrochemical cell at which reduction occurs, i.e., the positive terminal of a galvanic cell. An electrical potential can be caused by differences in grain orientation within the steel structure, i.e., different orientations of the steel grain structure can cause some grains to act as anodes while others act as cathodes, while the rest of the steel material exhibits excellent

electrical conductivity. In addition, chemical anisotropy, non-metallic inclusions, strained and unstrained areas, and other imperfections on the surface of a metal can create potential differences that drive the corrosion process.

METALLIC PATH

The anode and the cathode must be electrically bonded or connected to complete the circuit.

ELECTROLYTE

The principle function of soil moisture is to furnish the electrolyte for carrying current. The ions in the electrolyte may be hydrogen and hydroxyl ions from the water itself and a variety of cations and anions, which depend upon the number and amount of soluble salts dissolved in the water. The presence of these ions determines the electrical conductivity, expressed as resistivity (measured in ohm-m or ohm-cm), of the electrolyte, as well as chemical properties such as acidity or alkalinity, and the development of chemical reactions between the primary products of corrosion and the electrolyte. For example, ferrous material is corroded by electrolytes that contain sulfates or chlorides from the soil because the corrosion products formed at the anode and the cathode are both soluble.

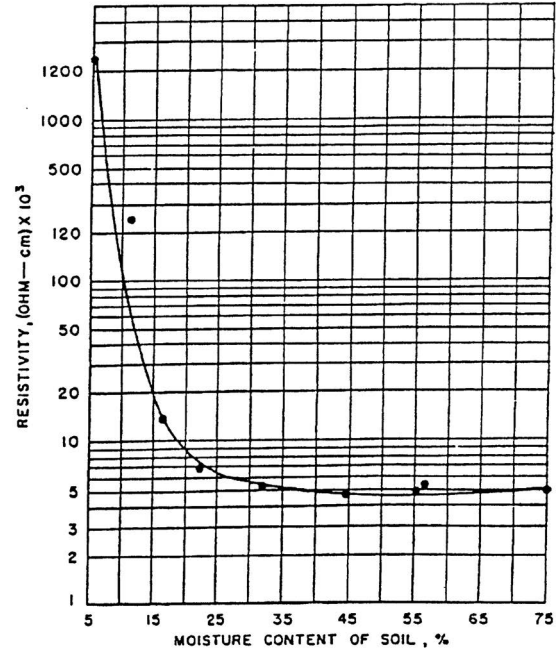
AERATION

Aeration affects the access of oxygen and moisture to the metal. Oxygen, either from atmospheric sources or from oxidizing salts or compounds, stimulates corrosion by combining with metal ions to form oxides, hydroxides, or metal salts. If corrosion products are soluble or are otherwise removed from the anodic areas, corrosion proceeds; but if the products accumulate, they may reduce corrosion by providing a barrier that is more noble (cathodic) than the bare metal. The aeration characteristics of a soil are dependent upon physical characteristics such as the particle size, particle size distribution, and unit weight. In volume change soils such as clay, a reduction in moisture content results in cracks that provide effective channels for the oxygen of the air to reach buried metal. Disturbed soils such as fill result in oxygen being more readily available. In some instances, atmospheric oxygen can become trapped in isolated pockets or cells creating the potential for localized anodic regions.

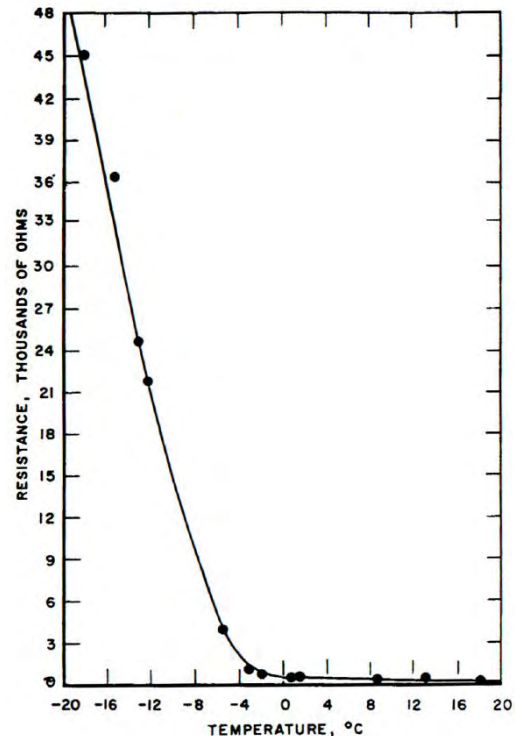
SOIL ENVIRONMENTS

SOIL TYPE

Soils constitute the most complex environment known to metallic corrosion. Corrosion of metals in soil can vary from relatively rapid material loss to negligible effects. Obviously, some soil types are more corrosive than others. The origin of soils, along with climate, geologic location, plant and animal life, and the effects of man all influence the corrosive potential of a given soil. Chemical analysis of soils is usually limited to determinations of the constituents that are soluble in water under standardized conditions. The elements that are usually determined are the base-forming elements, such as sodium,



EFFECT OF MOISTURE ON SOIL RESISTIVITY (ROMANOFF, 1957) FIGURE A-1



EFFECT OF TEMPERATURE ON EARTH RESISTANCE (ROMANOFF, 1957) FIGURE A-2

potassium, calcium, and magnesium; and the acid-forming elements, such as carbonate, bicarbonate, chloride, nitrate, and sulfate. The nature and amount of soluble salts, together with the moisture content of the soil, largely determine the ability of the soil to conduct an electric current. Therefore, fine-grained soils such as clays and some silts are considered to have a greater corrosion potential because they typically have lower hydraulic conductivity resulting in the accumulation of acid and base forming materials, which cannot be leached out very quickly. However, granular soils such as sands and gravels are considered to have a reduced corrosion potential because they typically have increased hydraulic conductivity, resulting in the leaching of accumulated salts.

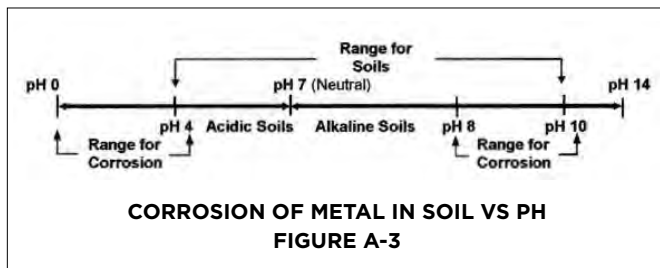
GROUND WATER

Moisture content in soil will probably have the most profound effect when considering corrosion potential than any other variable. No corrosion will occur in environments that are completely dry. The effect of moisture content on the resistivity of a clay soil is shown in Figure A-1. When the soil is nearly dry, its resistivity is very high (i.e., no corrosion potential). However, the resistivity decreases rapidly with increases in moisture content until the saturation point is reached, after which further additions of moisture have little or no effect on the resistivity. Figure A-2 shows the effect of temperature on the resistivity of a soil. As the temperature decreases down to the freezing point (32°F or 0°C), the resistivity increases gradually. At temperatures below the freezing point, the soil resistivity increases very rapidly.

SOIL pH

Soil pH can be used as an indicator of corrosion loss potential for metals in soil. The term “pH” is defined as the acidity or alkalinity of a solution that is assigned a number on a scale from 0 to 14. A value of 7 represents neutrality; lower numbers indicate increasing acidity and higher numbers increasing alkalinity. Each unit of change represents a ten-fold change in acidity or alkalinity which is the negative logarithm of the effective hydrogen-ion concentration or hydrogen-ion activity in gram equivalents per liter of solution. The development of acidity in soils is a result of the natural processes of weathering under humid conditions. Acidic soils are those that have had soluble salts and other materials removed, usually by moderate to high rainfall. In general, the soils of the Midwest and Eastern United States are acid to a considerable depth, whereas the soils whose development has been retarded by poor drainage or other conditions are alkaline. Most soils fall within a pH range that is strongly acid to mildly alkaline.

Extremely acid soils (below pH 4.5) and very strongly alkaline



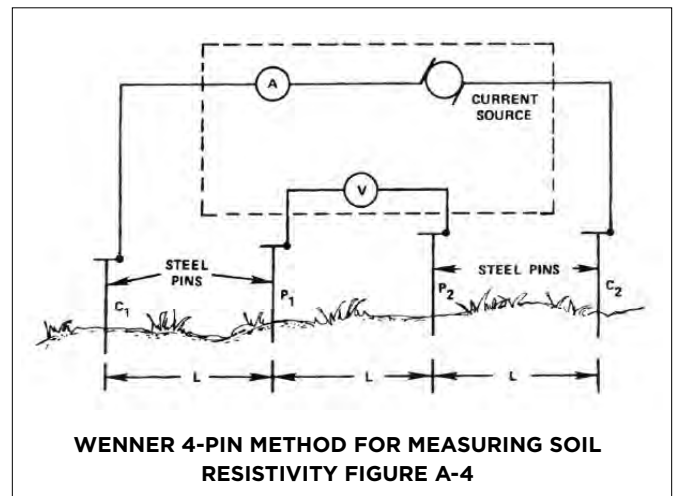
soils (above pH 9.1) have significantly high corrosion loss rates when compared to other soils (see Figure A-3). Soil pH is best measured in the field using a pH meter and following the methods defined in ASTM G 51 - 77.

SOIL RESISTIVITY

Soil resistivity (the reciprocal of conductivity) is the one variable that has the greatest influence on corrosion rate. However, other factors such as hydrogen-ion concentration, soluble salts and total acidity are interrelated, and it is difficult to control conditions so that there is only one variable. In general, the lower the resistivity the higher the corrosion rate. Metals buried in low resistivity soils will generally be anodic, whereas metals buried in adjacent high resistivity soils will generally be cathodic.

As shown in Figure A-1, moisture content has a profound effect on resistivity. Soil that is completely free of water has extremely high resistivity. For example, sandy soils that easily drain water away are typically non-corrosive; clayey soils that hold water have low resistivity and are typically corrosive. Backfill material will generally be more corrosive than native earth because the backfill soil has a higher moisture and oxygen content. In addition, backfill material typically never reconsolidates back to the same degree as native soil, allowing more penetration and retention of water.

Soil resistivity is typically measured using one or both of two methods: (1) testing onsite with the Wenner four-pin method, and/or (2) taking a soil sample to a laboratory for a soil box resistivity test. The recommended practice is the on-site Wenner four-pin method per AsTM G57-78. The four-pin method is recommended because it measures the average resistivity of a large volume of earth with relative ease. As Figure A-4 shows, this method places four pins at equal distances from each other. A current is then sent through the two outer pins. By measuring the voltage across the two inner pins, the soil resistance can be calculated using Ohm's law ($V=IR$). Soil resistivity can be determined using Equation A-1.



EQUATION A-1

$$\text{Resistivity} = 191.5 (R) (L) \text{ (ohm-cm)}$$

where

- R = Resistance measured with a soil resistivity meter
- L = Pin spacing (ft)

The soil box resistivity test is not recommended because it requires taking large number of samples for an accurate map of soil resistivities in a given area. The soil box test is also much more time-consuming than the four-pin method. Table A-2 is offered as a guide in predicting the corrosion potential of a soil with respect to resistivity alone.

SOIL RESISTIVITY AND POTENTIAL CORROSION RATE, TABLE A-2

RESISTANCE CLASSIFICATION	SOIL RESISTIVITY (ohm-cm)	CORROSION POTENTIAL
Low	0 - 2000	Severe
Medium	2000 - 10,000	Moderate
High	10,000 - 30,000	Mild
Very High	Above 30,000	Unlikely

PREDICTING CORROSION LOSS

BARE STEEL

The National Bureau of standards (NBS) performed extensive studies on underground corrosion between 1910 and 1955. More than 36,500 metal samples were exposed at 128 test locations throughout the United States. In 1957, Romanoff presented the results of these investigations in *Underground Corrosion* (1957). The studies showed that most underground corrosion was a complex electrochemical process dependent on the various properties discussed previously. The NBS studies were primarily concerned with buried pipeline corrosion. Since pipes are installed in backfilled trenches, the NBS work was performed on specimens placed in trenches ranging from 18 in (0.46 m) to 6 ft (1.8 m) deep. The following conclusions can be drawn from these studies:

- The metal loss rates reported were from samples placed in backfilled, i.e., disturbed soils.
- Atmospheric oxygen or oxidizing salts stimulate corrosion by combining with metal ions to form oxides, hydroxides, or metallic salts. This is particularly true in disturbed soils at or near the soil surface.
- The least corrosive soils had resistivities above 3,000 ohm-cm and low soluble salt concentrations.
- Metal loss rates in disturbed soils can be determined by assuming they will be similar to the loss rates found at test sites with similar pH and resistivity levels as provided in NBS Circular 579, Tables 6, 8 and 13

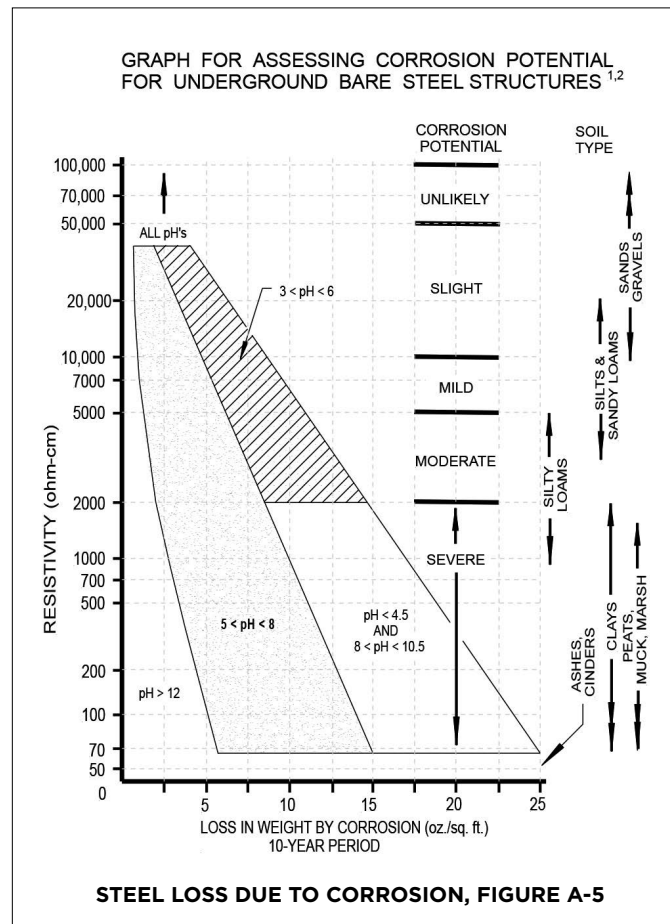
Hubbell Power Systems, Inc. bulletin 01-9204, *Anchor Corrosion Reference and Examples*, contains extensive metal loss rate data derived from Romanoff's work. It is recommended that

this information be used to determine the service life of non-galvanized steel in disturbed soil. The service life for most structures in the United States is 50 to 75 years. Assuming a corrosion allowance for steel piles/piers, Romanoff's metal loss rate data for specific soil types and locations can be used to determine if the required service life can be achieved.

Romanoff's data can also be arranged in easy-to-use graphs or tables. Figure A-5 provides a preliminary estimate for metal corrosion loss of bare steel if specific information is available on the soil (soil type, pH and resistivity). Figure A-5 provides a technique for quickly assessing those situations for which concern and design consideration for corrosion must be accounted for when metallic structures are placed below ground. For example, a clay soil with resistivity of 2000 ohm-cm and a pH of 6 will have an average metal loss rate of approximately 5 oz/ft²/10yrs, or 0.5 oz/ft²/yr. This figure was developed from the results of the NBS studies in addition to similar field experimentation results as presented in the *Proceedings, Eighth International Ash Utilization Symposium, Volume 2, American Coal Ash Association, Washington, DC, 1987.*

The Federal Highway Administration (FHWA) has proposed uniform corrosion loss rates based on a simple assessment of the electrochemical index properties. Per FHWA-RD-89-198, the ground is considered aggressive if any one of the critical indicators in Table A-3 shows critical values.

CORROSION





ELECTROMECHANICAL PROPERTIES OF MILDLY CORROSIVE SOILS, TABLE A-3

PROPERTY	TEST DESIGNATION	CRITERIA
Resistivity	AASHTO T-288-91	> 3000 ohm-cm
pH	AASHTO T-289-91	>5 < 10
Sulfates	AASHTO T-290-91	200 ppm
Chlorides	AASHTO T-291-91	100 ppm
Organic Content	AASHTO T-267-86	1% maximum

The design corrosion rates, per FHWA-SA-96-072, suitable for use in mildly corrosive soils having the electrochemical properties listed in Table A-3 are:

For zinc: 15 μm/year (0.385oz/ft²/yr) for the first two years; 4 μm/year (0.103 oz/ft²/yr) thereafter

For carbon steel: 12 μm/year (0.308 oz/ft²/yr)

Examples (Using Figure A-6):

- For pH of 6.5 and resistivity of 200 ohm-cm weight loss is approximately 1.3 oz/ft²/yr and expected life (for 1/8" shaft loss) is approximately 65 years.
- For pH of 7.5 and resistivity of 200 ohm-cm weight loss is approximately 2.3 oz/ft²/yr and expected life (for 1/8" shaft loss) is approximately 38 years.

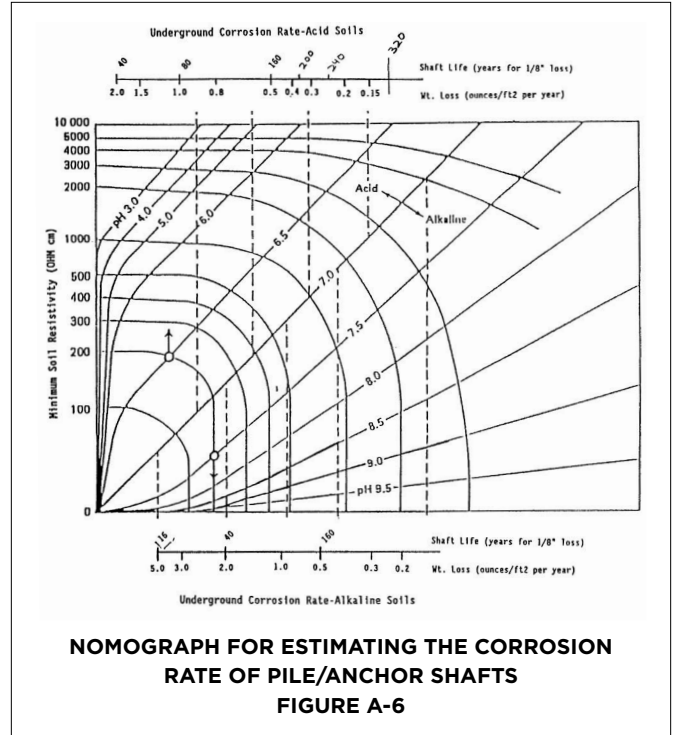
Other methods are available to predict corrosion loss rates. Figure A-6 is a nomograph for estimating the corrosion rate of helical anchor/pile/pier shafts. It is a corrosion nomograph adapted from the British Corrosion Journal (King, 1977). Its appeal is its ease of use. If the resistivity and soil pH are known, an estimate of the service life (defined as 1/8" material loss, for example) of a Chance® Helical Pile/Anchor or Atlas Resistance® Pier shaft can be obtained for either an acidic or alkaline soil.

CORROSION LOSS RATES

WATER/MARINE ENVIRONMENT

Factors other than resistivity and pH can have a strong influence on corrosion loss rates. It is well known that marine environments can be severely corrosive to unprotected steel, particularly in tidal and splash zones. Corrosion loss rates in these environments can be quite high, averaging 6.9 oz/ft.² (Uhlig, Corrosion Handbook, 2000). Salt spray, sea breezes, topography, and proximity all affect corrosion rate. Studies have shown that the corrosion rate for zinc exposed 80 ft (24.4 m) from shore was three times that for zinc exposed 800 ft (244 m) from shore.

Seawater immersion is less corrosive than tidal or splash zones. This is because seawater deposits protective scales on zinc and is less corrosive than soft water. Hard water is usually less corrosive than soft water toward zinc because it also deposits protective scales on the metallic surface. Table A-4 provides corrosion loss rates of zinc in various waters. In most situations, zinc coatings would not be used alone when applied to steel immersed in seawater, but would form the first layer of a more elaborate protective system, such as active protection using sacrificial anodes.



CORROSION OF ZINC IN VARIOUS WATERS (CORROSION HANDBOOK, VOLUME 13 CORROSION, ASM INTERNATIONAL), TABLE A-4

WATER TYPE	μ m/yr	mils/yr	oz/ft ² /yr
Seawater			
Global oceans, average	15 - 25	0.6 - 1.0	0.385 - 0.642
North Sea	12	0.5	0.308
Baltic Sea and Gulf of Bothnia	10	0.4	0.257
Freshwater			
Hard	2.5 - 5	0.1 - 0.2	
Soft river water	20	0.8	0.513
Soft tap water	5 - 10	0.2 - 0.4	0.128 - 0.257
Distilled water	50 - 200	2.0 - 8.0	1.284 - 5.130

CORROSION IN UNDISTURBED SOIL

In NBS Monograph 127, (Underground Corrosion of Steel Pileings) (Romanoff, 1972), it was reported that driven steel piles did not experience appreciable corrosion when driven into undisturbed soils. These findings were obtained during NBS studies of steel pile corrosion. Romanoff also stated that the NBS corrosion data for steel exposed in disturbed soils was not applicable to steel piles driven in undisturbed soil. He concluded:

“. . . soil environments which are severely corrosive to iron and steel buried under disturbed conditions in excavated trenches were not corrosive to steel piling driven in the undisturbed soil. The difference in corrosion is attributed to the differences in oxygen con-

centration. The data indicates that undisturbed soils are so deficient in oxygen at levels a few feet below the ground line or below the water table zone that steel pilings are not appreciably affected by corrosion, regardless of the soil types or the soil properties. Properties of soils such as type, drainage, resistivity, pH, or chemical composition are of no practical value in determining the corrosiveness of soils toward steel pilings driven underground."

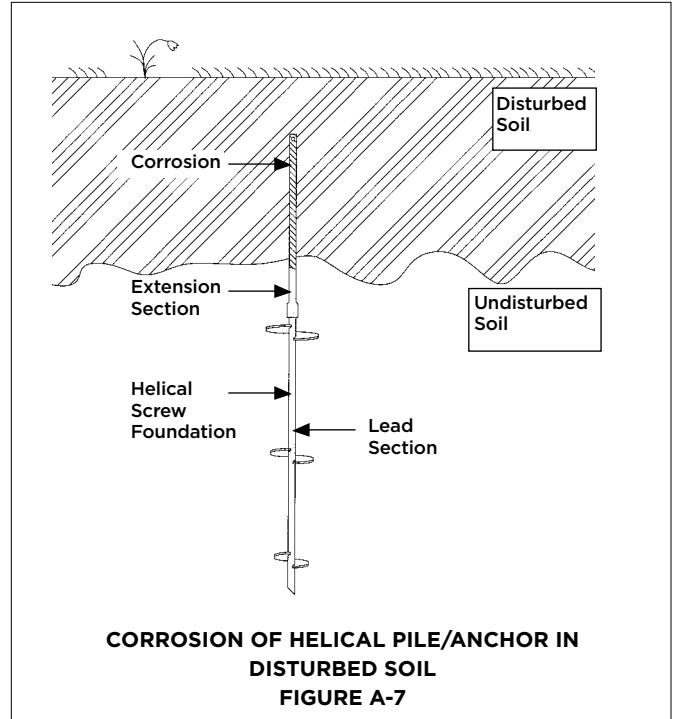
The following conclusions can be drawn from these studies:

- Oxygen is required at cathodic sites to support underground corrosion of a steel foundation product.
- Disturbed soils (fill) contain an adequate supply of oxygen to support underground corrosion, at least at shallow depths. Thus, the top-most extension(s) of the Chance® Helical Pile/Anchor or Atlas Resistance® Pier central steel shaft merits corrosion protection, either using passive protection like zinc, epoxy paint or Teflon® coatings or active protection like sacrificial anodes.
- The aggressiveness of disturbed soils can be measured, and they can be classified as aggressive and non-aggressive (see Table A-2).
- Undisturbed soils were deficient in oxygen a few feet below the ground surface, or below the water table. It is recommended to install the helical bearing plates of a helical pile/anchor into de-aerated soil.

The role of oxygen in an undisturbed soil overrides the effects of soil resistivity, pH, etc. In those situations where a steel foundation product is installed into a soil profile where a disturbed soil layer overlies undisturbed soil, the section of the central shaft in the disturbed soil is cathodic to the rest of the foundation in the undisturbed region as illustrated in Figure A-7. As a result, the most severe corrosion occurs on the section of the central shaft just below the disturbed layer.

Similarly, a steel foundation product located in undisturbed soil with a high water table can suffer some corrosion attack at the waterline as illustrated in Figure A-8. This combination does not result in serious attack, but it is believed that the situation is aggravated by a continuously changing ground water table, which would draw in oxygen as the waterline dropped. The section of the central shaft above the waterline acts as a weak cathode to the anode below the waterline.

Helical piles are commonly terminated in concrete cap or grade beams. The area of steel in the concrete forms a passive oxide film generated by the action of the highly alkaline environment, and this area is cathodic to the rest of the helical pile in the soil. However, the high resistivity of the concrete limits the effectiveness of the cathode, thereby limiting the small amount of corrosion attack to the region of the helical pile immediately outside the concrete as illustrated in Figure A-9.



FIELD MEASUREMENT OF SOIL RESISTIVITY

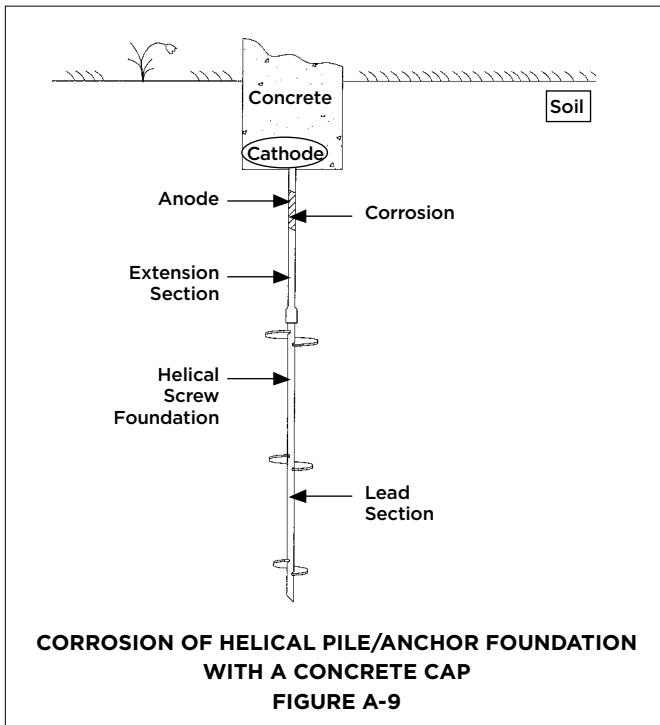
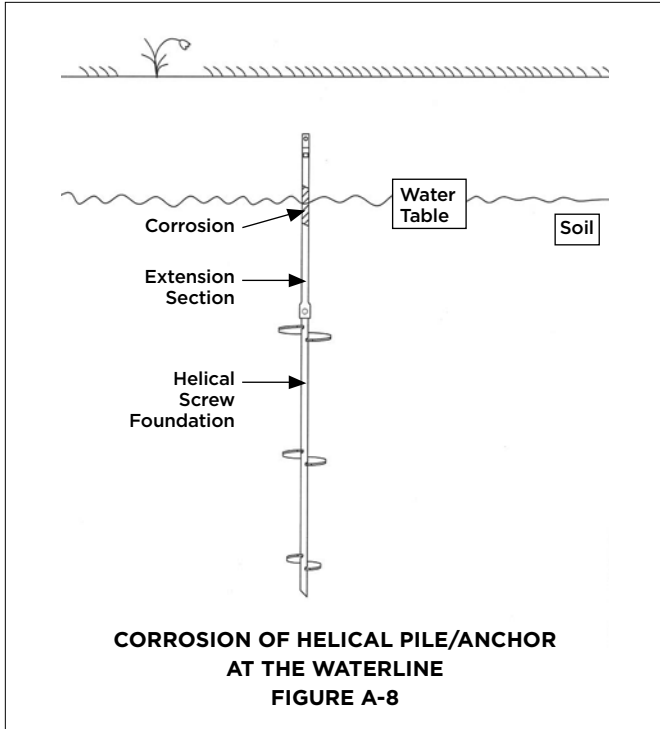
Field measurement of soil resistivity is not a difficult or time consuming process and results in the most accurate assessment of corrosion potential for the site. Hubbell Power Systems, Inc. recommends the use of the Nillson Model 400 Soil Resistance Meter System. The depth of the soil resistivity measurement is directly related to the pin spacing on the surface. The most accurate assessment is obtained by performing the test using a pin spacing of 5-20 foot intervals. In addition, the test should be repeated at a right angle to the original test to ensure that stray currents are not influencing the readings.

A. Equipment Set-Up

1. Insert the four sensor pins into the soil in a straight line leading away from the Resistivity Meter at a center-to-center distance of five feet (see Figure A-10).
2. Connect one wire to each pin and to the appropriate terminal on the Nillson meter.

B. Resistivity Measurement

1. Adjust the OHMS resistivity dial and the MULTIPLIER dial to the maximum setting (turned fully to the right) (see Figure A-11).
2. Place the SENSITIVITY switch in the LOW position and rotate the MULTIPLIER dial to the left until the meter needle goes past the NEUTRAL point, then rotate the MULTIPLIER one position to the right. Note the MULTIPLIER (M) amount on the field notes.
3. Move the OHMS dial to the left until the meter needle is at NEUTRAL.



4. Adjust the SENSITIVITY switch to HIGH position and adjust the OHMS dial to refine the reading.
5. Record the reading (R_{meter})
6. Return the OHMS and MULTIPLIER to the maximum settings and repeat the test.
7. Repeat the test with the pins spaced at 10-feet on center, then at 15-feet and 20-feet on center. Record the readings

C. Calculation of Soil Resistivity

EQUATION A-2

$$R = R_{\text{meter}} (M) (\text{WSF})$$

where

- R_{meter} = Meter resistance reading (ohms)
- M = Meter MULTIPLIER reading
- WSF = Wenner spacing factor = 191.5L (ft) = 628L (m)
- L = Pin spacing
- R = Soil resistivity (ohm-cm)

D. Additional Resistivity Measurements

1. The soil resistivity (R) is the average value over the depth of soil equal to the spacing of the pins. Therefore, to get a profile of the soil resistivity one must repeat the procedures in paragraph B above with the pins spaced at 10, 15 and 20 feet on center.

2. Repeat the entire test at right angles to the original alignment.

E. Documentation

Record the field data and the calculations onto the Soil Resistivity Log. A sample log is presented below (See Figure A-12).

F. Evaluate Results

When the Soil Resistivity (R) has been determined, refer to Figure A-5 to determine an estimate of the loss of weight by corrosion over a 10-year period for underground bare steel structures.

CORROSION CONTROL TECHNIQUES

The amount and type of corrosion control is a function of structure type, service life, and the overall aggressiveness of the project soils. The following requirements are typical. The specifier should review and edit as appropriate for the project.

- Structure Type: Temporary structures generally do not require corrosion protection. A temporary structure is defined within a specified time frame (i.e., months rather than years). In general, permanent structures have a service life greater than 24 months.
- Service Life: A typical service life of 50 to 75 years should be used unless otherwise specified. If the service life of a temporary CHANCE® Helical Pile/Anchor or Atlas Resistance® Pier is likely to be extended due to construction delays, it should be considered permanent. For a service life of less than 20 years in non-aggressive soil, corrosion protection is not recommended.
- Soil: Soil can be classified as aggressive or non-aggressive. See Guide to Model Specification-Helical Piles for Structural Support and Model Specification - Helical Tieback Anchors for Earth Retention found under the Resources tab on chancefoundationsolutions.

com for examples of aggressiveness classifications. It is recommended that steel foundation elements installed into soils classified as aggressive be provided with some type of corrosion protection.

Several alternatives are available to protect steel foundation products against corrosion and can be roughly categorized in terms of cost. Because of the added cost, the need for corrosion protection must be carefully determined and specified as necessary. Depending upon the classification as to the corrosion potential for a soil environment, several alternatives are available to deter the corrosion cycle and extend the performance life of the underground steel element. These control measures can be split into categories:

- **Passive Control:** For use in soils classified as mild to moderate corrosion potential. It typically consists of a metal loss allowance (i.e., 1/8") and/or coatings - such as galvanization or epoxy paint. Passive control is relatively inexpensive.
- **Active Control:** For use in soils classified as moderate to severe corrosion potential. It typically consists of cathodic protection via the use of sacrificial anodes. Active control is relatively expensive and is used in permanent applications.

PASSIVE CONTROL

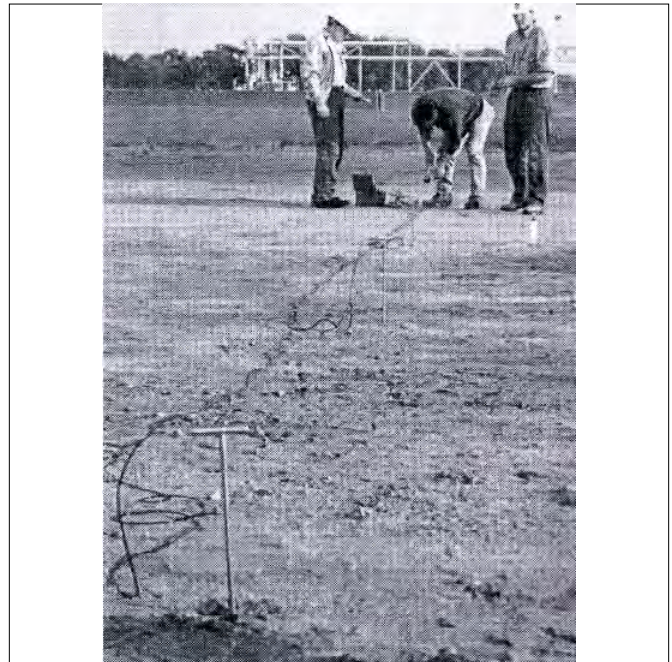
ALLOWABLE METAL LOSS RATE

As mentioned previously, Hubbell Power Systems, Inc. bulletin 01-9204, Anchor Corrosion Reference and Examples, contains extensive metal loss rate data derived from Romanoff's work. Other metal loss rate data is presented on pages A-5 through A-7. The design examples at the end of this section demonstrate passive control calculations that estimate the service life of helical pile shafts in soil using these metal loss rates. Design Example 1 uses the metal loss rates from Romanoff (Bulletin 01-9204). The service life is defined as the estimated length of time required for 1/8" of material loss to occur on the helical pile/anchor shaft. Design Example 2 uses the metal loss rates from Figure A-5 in conjunction with Equation A-3. The service life in this example is defined as the estimated length of time required for a 10% material loss to occur on the helical pile shaft. Design Example 3 uses the design corrosion rates per FHWA-SA-96-072 (as quoted here on page A-6) and an assumed service life of 85 years.

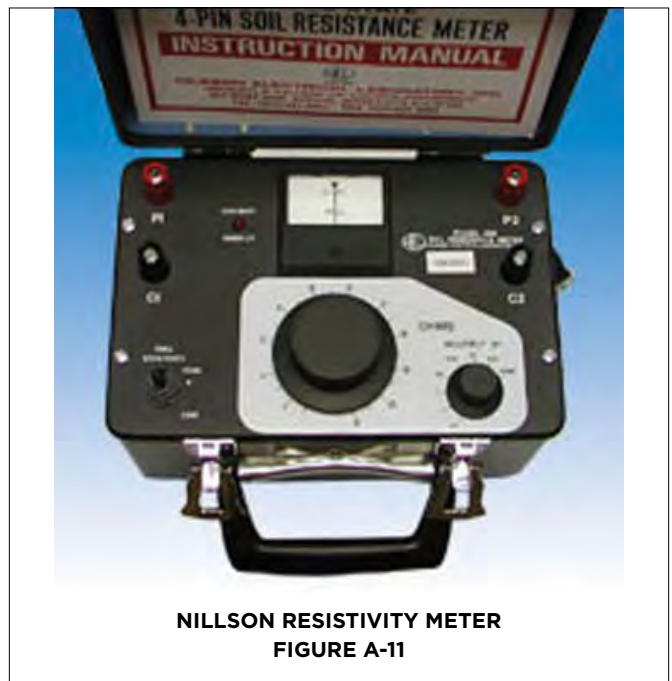
The amount of loss in these design examples is strictly arbitrary, but the assumed material loss of 1/8" in Design Example 1 is common for pile evaluation.

GALVANIZATION (PASSIVE CONTROL)

Aggressive soils, and the conditions illustrated in Figures A-7, A-8, and A-9 demonstrate the need to coat the section of the steel foundation product above the waterline in the disturbed soil and, in particular, the area of the central shaft in the concrete cap or grade beam. Thus, by removing the cathode, the anode/cathode system is disrupted resulting in reduced corrosion. If it were possible to apply a coating capable



**SENSOR PIN INSTALLATION
FIGURE A-10**



**NILLSON RESISTIVITY METER
FIGURE A-11**

of guaranteed isolation of the steel surface from the electrolyte (soil), all corrosion concerns would be solved. However, a coating capable of 100% guaranteed isolation has yet to be developed. Epoxy paint coatings provide excellent electrical isolation, but can chip and abrade easily during handling and installation. The same holds true for porcelain, teflon, and polyurethane coatings. A small chip or crack in the protective coating can cause corrosion activity to be highly localized, possibly



COMBINED WENNER 4-PIN SOIL RESISTIVITY LOG

Location:		Job No.
Date:	Weather Conditions:	Orientation of Pins:

WENNER METHOD OF SOIL RESISTIVITY

PIN SPACING (Depth in Feet)	METER RESISTANCE (R _{Meter}) (ohms)	METER MULTIPLIER (M)	WENNER SPACING FACTOR (WSF) (191.5* x Pin Spacing)	SOIL RESISTIVITY R = (R _{Meter}) x M x WSF

*** IF PIN SPACING IS MEASURED IN METERS, USE WENNER SPACING FACTOR (WSF) OF 628 INSTEAD OF 191.5
SAMPLE RESISTIVITY LOG, FIGURE A-12**

leading to severe damage. The single best coating for steel foundation products is hot dip galvanizing.

The first step in the galvanizing process is pickling the steel in dilute acid. This removes any rust, scale, oil or other surface contaminants. The clean steel is then dipped in a vat of molten zinc for time periods ranging up to several minutes depending on the size and thickness of the steel foundations. After the hold period, the zinc-coated steel is withdrawn from the vat at a controlled rate, which allows the coating to quickly cool and harden. The result is a tough, combined zinc and zinc-iron coating metallurgically bonded to the steel. Other galvanization processes, such as mechanical galvanizing and electroplating, do not form a coating that is metallurgically bonded to the steel.

Hubbell Power Systems, Inc. galvanizes to the latest ASTM standards – either ASTM A153 class B or ASTM A123. ASTM A153 Class B requires an average weight of zinc coating to be 2.0 oz./ft² (3.4 mils) and any individual specimen to be no less than 1.8 oz./ft² (3.1 mils). ASTM A123 can be used to specify thicker zinc coatings – up to 2.3 oz./ft² (3.9 mils) depending on the coating thickness grade used. For example, Grade 75 is 1.9 oz./ft² (3.0 mils). Regardless of which ASTM galvanizing specification is used, typical zinc coating thickness for hot-dip galvanized CHANCE® Helical Pile/Anchor or Atlas Resistance® Piers ranges between 4 and 6 mils.

Figure A-13 illustrates how zinc and steel react to form zinc-iron alloy layers. The bottom of the picture shows the base steel, then a series of alloy layers and, on the outside, the relatively pure outer zinc layer. The underlying zinc-iron alloy layers are actually harder than the base steel. Therefore, below the relatively soft pure zinc layer, the zinc-alloy layers provide protection in abrasive conditions such as dense sands and gravels.

Hot dip galvanized coatings protect the carbon steel shaft in two ways. First, the zinc coating provides a protective layer between the foundation's central shaft and the environment. Second, if the zinc coating is scratched and the steel surface exposed, the zinc, not the steel, will corrode. This is because zinc is a dissimilar metal in electrical contact with the steel, thus the difference in potential between the two metals and

their relative chemical performance (anode or cathode) can be judged by examining a galvanic series as shown in Table A-5. The materials at the top of the list are most active (anodic) compared to the noble (cathodic) materials at the bottom of the list. Steel is more noble than zinc, thus the more active zinc coating will act as an anode and corrode while the more noble steel will be the cathode and be protected.

SERVICE LIFE INCREASE THROUGH GALVANIZATION

Hubbell Power Systems, Inc. bulletin 01-9204, Anchor Corrosion Reference and Examples, contains extensive metal loss rate data on galvanized steel derived from Romanoff's work. It is recommended that this information be used to determine the service life of the hot dipped galvanized coating in disturbed soil. When hot-dip galvanized steel is used, the total service life should be increased by the time it takes the zinc coating to be lost due to corrosion. Another method for estimating service life increase is presented in the following paragraphs.

The results of the studies conducted by the National Bureau of Standards and by Porter indicated that a galvanized coating (zinc) was effective in delaying the onset of corrosion in the buried steel structures. Typical conclusions drawn from this study for 5 mil (3 oz./ft²) galvanized coatings include:

- It is adequate for more than 10 years corrosion protection for inorganic oxidizing soils.
- It is adequate for more than 10 years corrosion protection for inorganic reducing soils.
- It is insufficient for corrosion protection in highly reducing organic soils (pH<4), inorganic reducing alkaline soils and cinders, typically offering 3 to 5 years of protection in such cases.

It was also noted, however, that the use of a galvanized coating significantly reduces the rate of corrosion of the underlying steel structure once the zinc coating was destroyed.

The observed rates of corrosion for the galvanized coating were different (less) than that for bare steel in the NBS study. For galvanized coatings (zinc) of 5 mils, Equation A-3 can be used to estimate the corrosion (weight

loss) rate.

EQUATION A-3

$$CL_1 = 0.25 - 0.12 \log_{10} (R/150)$$

where

CL₁ = Weight loss (oz/ft²/yr)

R = Soil resistivity (ohm-cm)

NOTE: For thinner galvanized coatings, the rate of galvanized coating loss is two to three times the rate determined from Equation A-3.

MANUFACTURED METALLIC COATING (PASSIVE CONTROL)

Hubbell Power Systems, Inc. provides triple coat corrosion protection as a standard feature on the 3-1/2" diameter by 0.165" wall (3500.165 series) Atlas Resistance® Pier pipe and as an optional feature on the 2-7/8" diameter 0.165 wall (2875.165 series) Atlas Resistance® Pier pipe. The triple coating consists of:

- Hot-dipped uniform zinc galvanizing
- Chromate conversion coating
- Clear organic polymer coating

The triple coating can significantly reduce the corrosion process by mechanically preventing access of oxygen to the steel surface of the pipe. Data from the manufacturer indicates that this corrosion protection is equivalent to 3 mil (1.8 oz/ft²) of hot dip galvanizing. Because of the thinness of this film and possible scratching of the coating, this corrosion protection technique should not be used in soils classified as severe.

BITUMINOUS AND OTHER COATINGS (PASSIVE CONTROL)

Bituminous coatings as well as other materials have been used as coatings on buried steel elements for years as a corrosion protection technique. The primary requirements of a bituminous coating are good adherence (permanence), continuous coating and resistance to water absorption. The bituminous coating can either be heat baked onto the shaft or field applied just prior to installation. As is the case for the manufactured coatings, this coating technique prevents oxygen and water from contacting the metal surface, thus preventing or retarding the corrosion process.

Bituminous or asphaltic coatings or paints only provide physical protection from the environment. They will wear off quickly due to the abrasive action during installation of Chance® Helical Piles/Anchors and Atlas Resistance® Piers. Extension sections are typically hot-dip galvanized, but other coatings can be specified. Practical application of asphaltic coatings is generally limited to the extension sections located at or near the surface where the coating will provide the greatest benefit. Bituminous and other coatings are best applied in severely corrosive conditions where part of the helical anchor/pile is exposed above grade. Examples are steel foundations used in tidal marshes, coastal regions, and contaminated soils.

A limited amount of available data indicates that bituminous coatings can extend the performance life of underground steel piles and piers by 5 to 15 years, depending on the soil

environment and the thickness of the coating. For the vast majority of Chance® Helical Piles/Anchors and Atlas Resistance® Pier applications, the use of coating techniques (galvanized and/or bituminous) will provide a sufficiently long-term solution for corrosion protection.

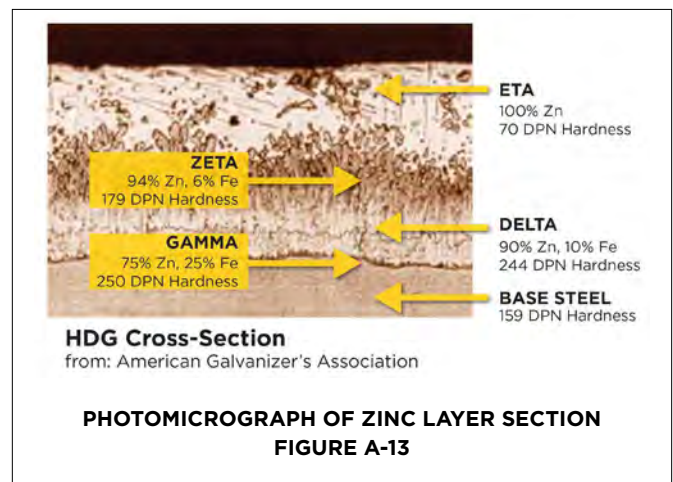
ACTIVE CONTROL

CATHODIC PROTECTION (ACTIVE CONTROL)

As indicated previously, corrosion is an electrochemical process that involves a flow of direct electrical current from the corroding (anodic) areas of the underground metallic structure into the electrolyte and back onto the metallic structure at the non-corroding (cathodic) areas. In situations where metallic structures such as Hubbell Power Systems, Inc. foundation products are to be placed in a severe corrosive soil environment, an active corrosion control technique should be used. This active control technique is termed cathodic protection. Cathodic protection is a method of eliminating corrosion damage to buried steel structures by the application of DC current. The effect of the DC current is to force the metallic surface to become cathodic (i.e., collecting current). If the current is of sufficient magnitude, all metallic surfaces will become cathodic to the external anode.

Both sacrificial anode and impressed current (rectifier and ground bed) cathodic protection systems are used to provide the required current. If the current source is derived from a sacrificial metal (magnesium and zinc are the two most common galvanic anodes used in soils), the effectiveness will depend on the soil properties in which it is placed. More available current is generated from a sacrificial anode in low resistance soils than high resistance soils. It is also best to place impressed current anode beds in lower resistant soils. However, since the available driving potential is greater (rectifier control), the soil resistivity is less significant.

Current requirements needed to protect a steel structure from corrosion will vary due to physical and environmental factors. These requirements could range from 0.01ma/ft² of metal surface for a well-applied, high-dielectric-strength plastic coating to 150 ma/ft² for bare steel immersed in a turbulent, high





velocity, salt-water environment. In soil, 1 to 3 ma/ft² is typically used as the required current to protect carbon steel.

The basic principle in cathodic protection is to apply a direct current of higher electromotive potential than that generated by the corroding metallic structure, thus effectively eliminating the corrosion process.

SACRIFICIAL ANODES (ACTIVE CONTROL)

In the case of Chance® Helical Piles/Anchors and Atlas Resistance® Piers, sacrificial anodes are the most common method of cathodic protection used. This is done by electrically connecting the steel to a properly selected anode of a less noble metal such as zinc or magnesium. The dissimilar metals buried in a common electrolyte (soil) form a galvanic cell. The cell works much like the battery in the family car; the less noble anode corrodes or sacrifices itself while the more noble cathode is protected. For steel to be cathodically protected, it is generally recognized that at least one of the following conditions must be met:

- The potential of the steel must be at -0.85 volts or more negative with respect to a saturated copper-copper sulfate half-cell in contact with the electrolyte, or
- A potential shift of -0.3 volts or more negative upon connection of the cathodic protection.

Magnesium, zinc and aluminum are the most commonly used galvanic sacrificial anodes. The sacrificial anode (galvanic) is attached to each underground metallic structure by a metallic conductor (cable) and placed within the common electrolyte (soil medium). The sacrificial anode works best when a small amount of current is needed and/or when the soil resistivities are low. Anodes are installed normally 3 feet below the surface

GALVANIC SERIES IN SEAWATER, TABLE A-5

<p>ACTIVE</p> <p>PASSIVE</p>	Magnesium
	Zinc
	Beryllium
	Aluminum Alloys
	Cadmium
	Mild Steel, Cast Iron
	300 Series Stainless Steel (Active)
	Aluminum Bronze
	Naval Brass
	Tin
	Copper
	Lead-Tin Solder (50/50)
	90-10 Copper Nickel
	Lead
	Silver
	300 Series Stainless Steel (Passive)
	Titanium
	Platinum
Graphite	

and 3 to 7 feet from the Chance® Helical Piles/Anchors and Atlas Resistance® Pier.

In designing and using sacrificial anode systems, the soil profile conditions as to the type of soil, resistivities, soil pH and location of the ground water table (GWT), if present, must be determined. Among the design considerations for the system:

- Use of wire type or canister type anode
- Selection of the appropriate anode material (magnesium, zinc, etc.)
- Designing the ground bed (location, dimensions, horizontal vs. vertical, depth of placement, type of backfill, etc.)
- Determining the number of piles/piers per anode
- Type, size and connections between pile(s) and the sacrificial anode.

The application of cathodic protection using galvanic sacrificial anode bags to underground metallic structures offers the following advantages:

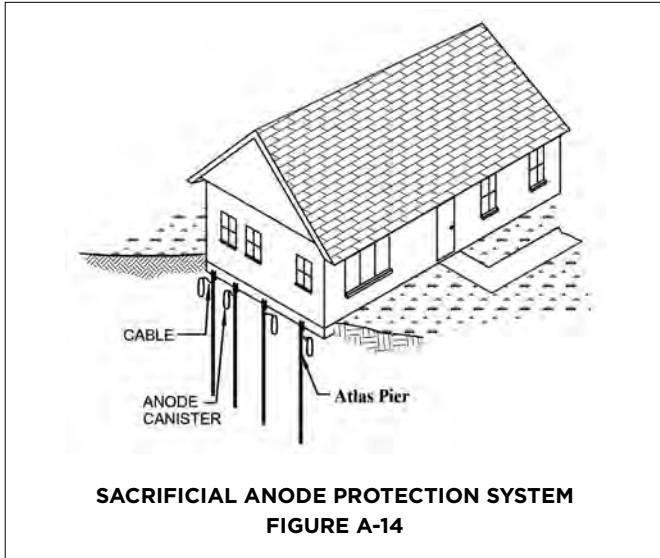
- No external power supply required
- Low system cost (bags and installation)
- Minimum maintenance costs

CATHODIC PROTECTION PRODUCTS

Hubbell Power Systems, Inc. recommends a selection of magnesium anodes (9, 17, 32, and 48-pound bag sizes) for cathodic protection of foundation support systems. Cathodic protection is generally used to extend the life of a steel product in corrosive soil beyond the added life available by hot dip galvanizing the components. While it is possible to protect mill finish steel, the engineer usually calls for the cathodic protection in addition to zinc galvanizing.

FACTORS INFLUENCING ANODE OUTPUT:

- Soil Resistivity: Current output from the magnesium anode increases as the soil resistivity decreases. Therefore, magnesium anodes are usually specified in applications where the soil resistivity is 5,000 ohm-cm or less. The effectiveness of this type of cathodic protection decreases as the resistivity increases above 5,000 ohm-cm. Above 10,000 ohm-cm resistivity, magnesium anodes are not effective.
- Anode Surface Area: The amount of current output generated by an anode is directly proportional to the surface area of the anode. Different manufacturers of cathodic protection produce anodes with different surface areas. Just because magnesium anodes from different manufacturers weigh the same is not to be assumed that the current output will be the same. The data presented here is representative for the products identified here.
- Alloy Potential: H-1 magnesium alloy has an open circuit potential of -1.53 to -1.55 volts, which works well with vertically installed foundation support systems.



High potential anodes are available from other sources. These high cost, high potential anodes are generally used along horizontal pipelines where the higher potential produced by the anode translates to fewer anodes being required. Table A-6 provides estimates of current output from a single, standard potential H-1 magnesium alloy anode as related to soil resistivity.

MAGNESIUM ANODES, TABLE A-6

	MAGNESIUM ANODES TYPE H-1 STANDARD POTENTIAL MAGNESIUM			
	Item No	Magnesium Weight	Package Size	Unit Weight
	PSA4438	9 lb.	6" Dia. x 17" Tall	27
	PSA4439	17 lb.	6-1/2" Dia. x 24" Tall	45
	PSA5106	32 lb.	8" Dia. x 28" Tall	72
	PSA4440	48 lb.	8" Dia. x 38" Tall	100

MAGNESIUM ANODE CURRENT OUTPUT - mA					
Resistivity - ohm-cm	1,000	2,000	3,000	4,000	5,000
9# Anode	106.5	53.3	35.5	26.6	21.3
17# Anode	150	75	50	37.5	30
32# Anode	159	79.5	53	39.8	31.8
48# Anode	163.5	81.8	54.5	40.9	32.7

Design Example 4 at the end of this section provides a method for estimating the service life of a sacrificial magnesium anode. For additional information on anode selection, refer to Hubbell Power Systems, Inc. bulletin 2-8307, Cathodic Protection of Anchors - A Basic Guide to Anode Selection and Hubbell Power Systems, Inc. bulletin 01-9204, Anchor Corrosion Reference

and Examples.

IMPRESSED CURRENT (ACTIVE CONTROL)

In areas of the most severe corrosion potential, where a larger current is required and/or in high resistance electrolytes, an impressed current system is generally recommended which requires a power source, rectifier and a ground bed of impressed current anodes. These systems require a continuous external power source.

The majority of applications where Hubbell Power Systems, Inc. foundation products may be specified will not require an active corrosion protection system. In those cases where the combination of soil and electrolyte conditions requires an active system, the sacrificial anode protection system will likely be the most economical approach.

Active cathodic protection systems must be individually designed to the specific application. The major variables are soil moisture content, resistivity of soil and pH. Each of these items influences the final selection of the cathodic protection system. Typical design life for the cathodic protection is 10 to 20 years, depending upon the size and length of the anode canister.

DESIGN EXAMPLES

DESIGN EXAMPLE 1:

PROJECT: Santa Rosa, CA Residence

The purpose of the calculations is to estimate the service life of Type SS Helical Pile Shafts on the subject project. Service life is defined as the estimated length of time required for 1/8" of material loss to occur on the helical pile shaft. This amount of loss is strictly arbitrary, but is common for pile evaluation.

GIVEN:

- Helical piles galvanized to ASTM A153 (Minimum Zinc Coating = 1.8 oz/ft²)
- Soil resistivity is 760 ohm-cm minimum
- Soil pH - 7.70
- Water soluble chloride - 11 ppm
- Water soluble sulfate - 417 ppm

ASSUMPTIONS:

It is assumed that the material loss rates will be similar to the loss rates found at test sites with similar pH and resistivity levels as given in Romanoff's Underground Corrosion, NBS Circular #579 (1957), Tables 6, 8 and 13.

In Circular #579, Site #5 is indicated as having a resistivity of 1,315 ohm-cm and a pH of 7.0. This soil is Dublin Clay Adobe and is located around Oakland, California. In addition, Site #2 is indicated as having a resistivity of 684 ohm-cm and a pH of 7.3. This soil is Bell Clay and is located around Dallas, Texas. The



corrosion rates for these two sites will be used to estimate the life of the Type SS helical pile shaft material.

ALLOWABLE STEEL LOSS:

Based on the loss of 1/8" thickness of the helical pile shaft, calculate the allowable steel loss (ASL) in terms of weight per unit area:

$$\begin{aligned}
 \text{ASL} &= (0.125 \text{ in}) (0.283 \text{ lb/in}^3) (16 \text{ oz/lb}) \\
 &= (0.566 \text{ oz/in}^2) (144 \text{ in}^2/\text{ft}^2) \\
 &= 81.5 \text{ oz/ft}^2
 \end{aligned}$$

AVERAGE METAL LOSS PER YEAR:

From Site #5: (Dublin Clay Adobe)

EXPOSURE DURATION (years)	WEIGHT LOSS (oz/ft ²)	LOSS PER YEAR (oz/ft ²)
1.9	1.4	0.737
4.1	2.2	0.585
6.2	4.8	0.774
8.1	5.2	0.642
12.1	5.4	0.446
17.5	8.3	0.474

The average metal loss per year is 0.61 oz/ft². Note that as the duration of exposure increases, the material loss per year generally decreases.

PILE SHAFT LIFE:

To determine the pile shaft service life (SL), the allowable steel loss is divided by the average loss per year:

$$\begin{aligned}
 \text{SL} &= (81.5 \text{ oz/ft}^2) / (0.61 \text{ oz/ft}^2) \\
 &= 133.6 \text{ years}
 \end{aligned}$$

TOTAL ZINC COATING LOSS:

Chance® Helical Piles/Anchors are typically provided already hot dip galvanized per ASTM A153. The coating thickness for ASTM A153 class B = 1.8 oz/ft². From Romanoff, NBS Circular #579, Page 110, Table 65 gives the following average loss rates for Site #5 soils:

EXPOSURE DURATION (years)	WEIGHT LOSS (oz/ft ²)	LOSS PER YEAR (oz/ft ²)
10.17	2.66	0.262

Estimated Life of Zinc: 1.8 oz/ft² / 0.262 oz/ft² = 6.9 years

Total Estimated Service Life of Helical Pile Shaft: 133.6 + 6.9 = 140.5 years

From Romanoff Site #2 (Bell Clay):

EXPOSURE DURATION (years)	WEIGHT LOSS (oz/ft ²)	LOSS PER YEAR (oz/ft ²)
2.1	2.4	1.143
4.0	3.0	0.750
5.9	3.4	0.576
7.9	3.6	0.456
12.0	5.9	0.492
17.6	8.1	0.460

The average loss per year is 0.65 oz/ft². Note that as the duration of exposure increases, the material loss per year generally decreases.

HELICAL PILE SHAFT LIFE:

To determine the helical pile shaft's service life (SL), the allowable steel loss is divided by the average loss per year.

$$\begin{aligned}
 \text{SL} &= (81.5 \text{ oz/ft}^2) / (0.65 \text{ oz/ft}^2) \\
 &= 125.4 \text{ years}
 \end{aligned}$$

TOTAL ZINC COATING LOSS:

Chance® Civil Construction helical anchors/piles are already provided hot dip galvanized per ASTM A153. The coating thickness for ASTM A153 class B = 1.8 oz/ft². From Romanoff, NBS Circular #579, Page 110, Table 65 gives the following average loss rates for site #2 soils.

EXPOSURE DURATION (years)	WEIGHT LOSS (oz/ft ²)	LOSS PER YEAR (oz/ft ²)
9.92	0.44	0.044

ESTIMATED LIFE OF ZINC: 1.8 oz/ft² / 0.044 oz/ft² = 40.9 years

Total Estimated Service Life of Helical Pile Shaft: 125.4 + 40.9 = 166.3 years

SUMMARY:

Total estimated service life of helical pile shaft in Site #5 soils = 140.5 years

Total estimated service life of helical pile shaft in Site #2 soils = 166.3 years

These calculations are an estimate of the service life only (1/8" material loss from shaft) and are based upon loss rates obtained from Romanoff's disturbed soil sites. It is generally accepted that the majority of any corrosion will occur at or near the surface. Therefore, it is very likely that helical pile shaft metal loss will control the design. In the event the estimated service life does not meet the design requirements, one option is to use a larger sized helical pile shaft.

DESIGN EXAMPLE 2:



PROJECT: An access bridge designed to cross a wetland area.

The purpose of the calculations is to estimate the service life of Type RS3500.300 Helical Piles on this project. The service life is defined as the estimated length of time required for a 10% metal loss to occur to the helical pile shaft.

GIVEN:

1. Helical Piles will receive a hot dipped galvanized coating (G) of 5-mil thick (3-oz/ft²)
2. Soil Resistivity (R) - 1,000 ohm-cm
3. Soil pH - 6.0
4. Soil type - organic silt in top 10' with SPT blow counts of 2 to 4 blows per foot.

ASSUMPTIONS:

1. The metal loss rates will be based on the values given in Figure A-5 with a pH of 6.0 and a resistivity of 1,000 ohm-cm. These values place the organic silt in the severe corrosion environment region.
2. The galvanized coating loss rates will be based on Equation A-3 as shown on page A-12.

ESTIMATED LIFE OF GALVANIZED COATING:

To estimate average life for galvanized coating in a location with a soil resistivity of 1000 ohm-cm, Equation A-3 is used:

$$\begin{aligned}
 CL_1 &= 0.25 - 0.12 \log_{10} (R/150) \\
 &= 0.25 - 0.12 \log_{10} (1000/150) \\
 &= 0.25 - 0.12 (0.824) \\
 &= 0.15 \text{ oz/ft}^2/\text{yr}
 \end{aligned}$$

where: CL_1 = Weight loss per year

The estimated life of the galvanized coat is:

EQUATION A-4

$$\begin{aligned}
 L_1 &= G/CL_1 \\
 &= (3 \text{ oz/ft}^2) / (0.15 \text{ oz/ft}^2) \\
 &= 20 \text{ years}
 \end{aligned}$$

where

- G = Amount of galvanized coating = 3.0 oz/ft² for typical hot dipped galvanized coating (5 mil)
- L₁ = Life expectancy (yrs)

ESTIMATED LIFE OF OF STEEL:

The formula for estimating average life for loss in steel wall

thickness is given in Equation A-5:

EQUATION A-5

$$L_2 = W_s/K_2$$

where

- L₂ = Life expectancy (yrs)
- W_s = Weight of steel pile (oz/ft²)
- K₂ = Loss in weight by corrosion (oz/ft²/yr) as determined from Figure A-5

Reference to Figure A-5 indicates a corrosion weight loss range for bare steel of approximately 3 to 10 oz/ft² for a 10-year period. In this case (also checking the NBS data) an estimate was used of 8 oz/ft² for 10 years. Therefore K₂ = 8.0 oz/ft² per 10 years or 0.8 oz/ft²/year.

A 10% weight loss of the wall thickness of the steel for the RS3500.300 pile results in:

$$\begin{aligned}
 W_s &= 0.1 (0.300 \text{ in}/12 \text{ in/ft}) (489.6 \text{ lb/ft}^3) (16 \text{ oz/lb}) \\
 &= 20 \text{ oz/ft}^2
 \end{aligned}$$

The estimated additional life becomes:

$$\begin{aligned}
 L_2 &= W_s / K_2 \\
 &= (20 \text{ oz/ft}^2) / (0.8 \text{ oz/ft}^2/\text{yr}) \\
 &= 25 \text{ yrs}
 \end{aligned}$$

LIFE ESTIMATE SUMMARY (GALVANIZED STEEL ROUND SHAFT):

Based upon the assumptions, the results of this analysis indicate that the Chance® Type RS3500.300 helical pile as specified for the bridge foundation will experience an average 40 to 45 year estimated life.

DESIGN EXAMPLE 3:

Extendable helical piles/anchors consist of segmented elements that are coupled together with structural bolts. It is possible for coupling bolts to be located near the surface in disturbed soils. Therefore, it is recommended that the coupling bolt service life be calculated based on corrosion loss rates. This can be accomplished using methods similar to those shown in Design Example 1.

Determine the diameter reduction of Type SS5/150 coupling bolts using corrosion loss rates per FHWA-SA-96-072. for mildly corrosive soils. Type SS5/150 Helical Piles/Anchors use 3/4" diameter bolts per ASTM A325. Assume a service life of 85 years.

TOTAL ZINC COAT LOSS:

Hubbell Power Systems, Inc. provided fasteners are hot dip galvanized per ASTM A153. The coating thickness for ASTM A153 class B = 1.8 oz/ft².

$$\text{Zinc loss the first two years} = 0.385 \text{ oz/ft}^2/\text{year} \times 2 \text{ years} = 0.77 \text{ oz/ft}^2$$



Estimated life of zinc coating = $[1.8 \text{ oz/ft}^2 - 0.77 \text{ oz/ft}^2 = 1.03 \text{ oz/ft}^2 / 0.103 \text{ oz/ft}^2 = 10 \text{ years}] + 2 \text{ years} = 12 \text{ years}$

TOTAL STEEL LOSS:

Coupling bolt steel loss will occur after the zinc coating is lost. The exposure time to corrosion for the bolt steel is: 85 years - 12 years = 73 years.

Bolt steel loss over 73 years: = $0.308 \text{ oz/ft}^2/\text{year} \times 73 \text{ years} = 22.5 \text{ oz/ft}^2$

$22.5 \text{ oz/ft}^2 / 144 \text{ in}^2/\text{ft}^2 \times 16 \text{ oz/lb} \times 0.283 \text{ lb/in}^3 = 0.035''$ (0.9 mm)

Diameter reduction after 85 years is $0.75'' - 2 \times 0.035'' = 0.68''$ (17.3 mm)

Determine the tensile load capacity reduction of Type SS5/150 Coupling Bolts: The minimum ultimate tensile strength for Chance® Type SS5/150 Helical Piles/Anchors is 70 kip. The failure mechanism is double shear of the coupling bolt. Assuming a linear relationship between diameter and shear capacity, the bolt diameter reduction from an 85-year exposure per FHWA-SA-96-072 corrosion loss rates suitable for use in mildly corrosive soils will result in a reduced tension load capacity, i.e., $0.68 \times 70 / 0.75 = 63.5 \text{ kips}$.

DESIGN EXAMPLE 4:

1. Estimated Average Life of Sacrificial Magnesium Type

REFERENCES:

1. A.B. Chance Company, Anchor Corrosion Reference and Examples, Bulletin 01-9204, A.B. Chance Company, Centralia, MO, 1992.
2. A.B. Chance Company, Chance Anchor Corrosion Report, Bulletin 31-9403, reprinted with permission from the Texas Department of Transportation, A.B. Chance Company, Centralia, MO, 1994.
3. Corrosion and Its Control: An Introduction to the Subject, Second Edition, NACE International, 1995.
4. Corrosion Tests and Standards - Application and Interpretation, ASTM Manual Series, MNL 20, 1995.
5. Escalante, Edward, Concepts of Underground Corrosion, part of ASTM STP 1013, Effects of Soil Characteristics on Corrosion, Edited by V. Chaker and J.D. Palmer, American Society for Testing and Materials, Philadelphia, PA, 1989.
6. Federal Highway Administration, Publication No. FHWA-SA-96-072, Corrosion/Degradation of Soil Reinforcement for Mechanically Stabilized Earth Walls and Reinforced Slopes.
7. King, R.A., Corrosion Nomograph, TRRC Supplementary Report, British Corrosion Journal, 1977.
8. Metals Handbook, Volume 13, Ninth Edition, Corrosion, ASM International, Metals Park, Ohio, 1987.
9. Porter, Frank, Corrosion Resistance of Zinc and Zinc Alloys, Marcel Dekker, Inc., New York, NY.
10. Proceedings, Eighth International Ash Utilization Symposium, Volume 2, American Coal Ash Association, Washington, DC, 1987.
11. Rabeler, R.C., Soil Corrosion Evaluation of Screw Anchors, ASTM STP 1013, Effects of Soil Characteristics on Corrosion, Edited by V. Chaker and J.D. Palmer, American Society for Testing and Materials, Philadelphia, PA, 1989.
12. Romanoff, Melvin, Corrosion of Steel Pilings in Soil, part of National Bureau of Standards Monograph 127, NBS Papers on Underground Corrosion of Steel Piling 1962-71, published March, 1972.
13. Romanoff, Melvin, Underground Corrosion, National Association of Corrosion Engineers, Houston, TX, 1989. (Republished from National Bureau of Standards Circular 579, 1957)
14. Suzuki, Ichero, Corrosion-Resistant Coatings Technology, Marcel Dekker, Inc., New York, NY, 1989.
15. Tefankjian, D.A., Application of Cathodic Protection, Proceedings of the 19th Annual Underground Corrosion Short Course.
16. Uhlig's Corrosion Handbook, Second Edition, Edited by R. Winston Revie, Electrochemical Society Series, 2000.
17. West, Edward L., Cathodic Protection of Anchors - A Basic Guide to Anode Selection, A.B. Chance Bulletin 2-8307, A.B. Chance Company, Centralia, MO, 1983.
18. Winterkorn, Hans F., and Hsai-Yang Fang, Foundation Engineering Handbook, Van Nostrand Reinhold Company, New York, NY, 1962.
19. Various Cathodic Protection System Vendors.

Anode:

The formula for estimating average life for sacrificial magnesium anode life is given in Equation A-6:

EQUATION A-6

$L_3 = [57.08 (K_3) (W_a)] / I$

where

- L_3 = Life expectancy of magnesium or zinc anode (yrs)
- K_3 = Efficiency of anode bag (60%-70%)
- W_a = Weight of anode (lbs)
- I = Current output of anode (mA). Available from Table A-5 for Chance® Civil Construction supplied anodes or from the vendor when using other anodes.

NOTE: Equation A-6 is not unit consistent.

Assume that in the previous Design Example 2, the pile performance life is to be further extended (beyond 40 to 45 years) by use of a 48-pound magnesium sacrificial anode for each pile. For this size bar and soil resistivity condition ($R = 1000 \text{ ohm-cm}$), the vendor indicates $I = 163.5 \text{ mA}$ and $K = 65\%$. Therefore, Equation A-6 becomes:

$$L_3 = [57.08 (0.65) (48)] / 163.5$$

$$= 11 \text{ yrs}$$



APPENDIX B: LOAD TESTS

CONTENTS

Static Load Tests (Tiebacks)..... B-2

Static Axial Load Tests (Compression/Tension) B-4

Static Load Tests (Lateral) B-6

Acceptance Criteria B-8

LOAD TESTS

DISCLAIMER

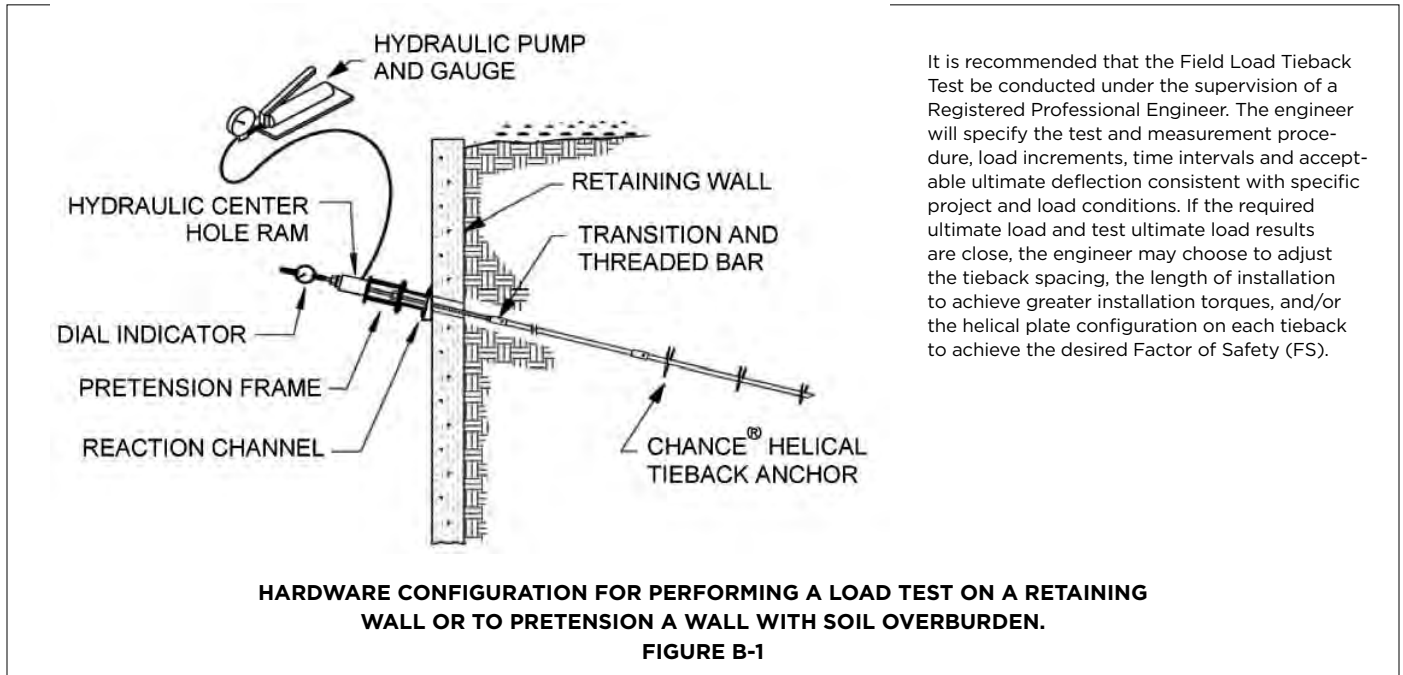
The information in this manual is provided as a guide to assist you with your design and in writing your own specifications. Installation conditions, including soil and structure conditions, vary widely from location to location and from point to point on a site.

Independent engineering analysis should be conducted and state and local building codes and authorities should be consulted 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 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.

STATIC LOAD TESTS (TIEBACKS)



It is recommended that the Field Load Tieback Test be conducted under the supervision of a Registered Professional Engineer. The engineer will specify the test and measurement procedure, load increments, time intervals and acceptable ultimate deflection consistent with specific project and load conditions. If the required ultimate load and test ultimate load results are close, the engineer may choose to adjust the tieback spacing, the length of installation to achieve greater installation torques, and/or the helical plate configuration on each tieback to achieve the desired Factor of Safety (FS).

TEST PROCEDURE

WARNING! DO NOT ALLOW ANYONE TO STAND BEHIND OR IN LINE WITH THE THREADED BAR AND JACK DURING THIS TEST. SERIOUS INJURY MAY OCCUR IF A COMPONENT FAILS DURING TESTING.

1. Determine the required length of the helical tieback anchor to locate the helix plates into the target soil stratum as determined from the project boring logs. Use this data to select the tieback design and ultimate tension capacity and the estimated installation torque. Install the helical tieback anchor to the determined length and torque requirements.
2. If the soil overburden has not been excavated from behind the wall, connect the thread bar adapter/transition to the helical tieback by reaching through the hole in the wall. Install the continuously threaded bar, reaction channel, hydraulic ram (loading device), pretension frame (if required), dial indicator (or other measuring device such as Total Station Unit), hydraulic pump and gauge (see Figure B-1). The magnitude of the test pressure is determined as follows:

$$P_T = \frac{DL \times FS}{A}$$

where

- P_T = Test pressure (psi)
- DL = Design Load (lb)
- FS = Factor of Safety = 1.25 to 2.5
- A = Effective cylinder area (in²)

NOTE: The effective cylinder areas (A) are available from the manufacturers of center hole rams (i.e., Enerpac, Power Team, Simplex, etc).

The load application system, i.e., center hole ram and pump, shall be calibrated by an independent testing agency prior to the load testing of any tiebacks. For additional details, refer to the Model Specification - Helical Tieback Anchors for Earth Retention at www.chancefoundationsolutions.com.

An Alignment Load (AL), usually 5% to 10% of the Design (Working) Load (DL), should be applied to the helical tieback anchor prior to the start of field load tests. The initial alignment load helps to remove any looseness in the tieback shaft couplings and thread bar transition system.

3. Pre-Production Tests (Optional):

Load tests shall be performed to verify the suitability and capacity of the proposed helical tieback anchor, and the proposed installation procedures prior to the installation of production tiebacks. The owner shall determine the number of pre-production tests, their location and acceptable load, and movement criteria. Such tests shall be based, as a minimum, on the principles of the performance test as described below. If pre-production tiebacks are to be tested to their ultimate capacity, then an additional purpose of the pre-production tests is to empirically verify the ultimate capacity to average installing torque relationship of the helical tiebacks for the project site. Testing above the performance test maximum applied load of 125% x DL should follow the loading procedures and increments as given in the Static Axial Load Tests (Compression/Tension) section to follow.

4. Performance Tests:

The number of tiebacks that require performance testing shall be defined in the project specifications. The minimum

number of tiebacks for performance testing shall be two (2). Helical tieback anchors shall be performance tested by incrementally loading and unloading the tieback in accordance with the Performance Test Schedule (see Table B-1). The applied load shall be increased from one increment to the next immediately after recording the anchor movement. The load shall be held long enough to obtain and record the movement reading at all load increments other than the maximum test load. The maximum test load (1.25 x DL) shall be held for a minimum of 10 minutes. Anchor movements shall be recorded at 0.5, 1, 2, 3, 4, 5, 6, and 10 minutes. Refer to Acceptance Criteria on later in section B for additional hold periods, if required, and acceptable movement criteria.

5. Proof Testing:

All anchors which are not performance tested shall be proof tested. The proof test shall be performed by incrementally loading the helical anchor in accordance with the Proof Test Schedule (see Table B-2). The load shall be raised from one increment to another after an observation period. At load increments other than the maximum test load, the load shall be held for a period not to exceed two (2) minutes. The two minute observation period shall begin when the pump begins to load the anchor to the next load increment. Movement readings shall be taken at the end of the two minute observation period.

The dealer/installing contractor or engineer shall plot the helical anchor displacement vs. load for each load increment in the proof test. The 1.25DL test load shall be maintained for five (5) minutes. This five minute observation period shall commence as soon as 1.25DL is applied to the anchor. Displacement readings shall be recorded at 0.5, 1, 2, 3, 4, and 5 minutes. Refer to Acceptance Criteria later in section B for additional hold periods, if required, and acceptable displacement criteria.



**ANCHOR TENSION LOAD TEST IN MINNEAPOLIS, MN
FIGURE B-2**

PERFORMANCE TEST SCHEDULE, TABLE B-1

PERFORMANCE TEST SCHEDULE				
CYCLICAL LOAD INCREMENTS (%DL/100)				
AL	AL	AL	AL	AL
0.25DL*	0.25DL	0.25DL	0.25DL	0.25DL
	0.25DL	0.50DL	0.50DL	0.50DL
		0.75DL*	0.75DL	0.75DL
			1.00DL*	1.00DL
				1.25DL*
				Reduce to lock-off load#

AL = Alignment Load, usually 10 to 15% of DL.
 DL = Design (Working) Load
 * The dealer/installing contractor shall plot the helical anchor movement for each load increment marked with an asterisk (*) in the performance schedule and plot the residual displacement at each alignment load versus the highest previously applied load.
 # Helical tieback anchors which are performance tested may be completely unloaded prior to the lock-off load procedure. Final adjusting to the lock-off load does not require further movement readings. Lock-off load magnitude is typically 70% - 80% of the Design Load (DL).
 See the Performance Testing Procedures in the Model Specification - Helical Tieback Anchors for Earth Retention at www.chancefoundationsolutions.com for further information regarding load test equipment, load test set-up, dial gauges for monitoring anchor displacement, etc.

PROOF TEST SCHEDULE, TABLE B-2

PROOF TEST SCHEDULE	
LOAD TEST SCHEDULE (%DL/100)	OBSERVATION PERIOD (MIN.)
AL	AL
0.25DL	2.0
0.50DL	2.0
0.75DL	2.0
1.00DL	2.0
1.25DL	5.0
Reduce to lock-off load#	

AL = Alignment Load, usually 10 to 15% of DL.
 DL = Design (Working) Load
 # Helical tieback anchors which are proof tested may be completely unloaded prior to the lock-off load procedure. Final adjusting to the lock-off load does not require further displacement readings. Lock-off load magnitude is typically 70% - 80% of the Design Load (DL).
 See the Proof Testing Procedures in the Model Specification - Helical Tieback Anchors for Earth Retention at www.chancefoundationsolutions.com for further information regarding load test equipment, load test set-up, dial gauges for monitoring anchor displacement, etc.



STATIC AXIAL LOAD TESTS (COMPRESSION/TENSION)

PRE-PRODUCTION LOAD TESTS

Load tests shall be performed to verify the suitability and capacity of the proposed helical anchor/pile, and the proposed installation procedures prior to installation of production helical anchors/piles. These load tests shall be performed prior to the installation of the production helical anchors/piles. The Owner shall determine the number of pre-production load tests, their location, acceptable load and displacement criteria, and the type(s) of load direction (i.e., tension, compression, or both). An additional purpose of pre-production tests is to empirically verify the ultimate capacity to the average installing torque relationship of the helical pile/anchor for the project site with the torque measurement equipment used for the project. Pre-production helical pile/anchor installation methods, procedures, equipment, and overall length shall be identical to the production helical anchors/piles to the extent practical except where approved otherwise by the Owner.

It is recommended that any field load test for compression or tension be conducted under the supervision of a Registered Professional Engineer. The engineer will specify the test and measurement procedure, load increments, time intervals, and acceptable ultimate displacement consistent with specific project and load conditions. Test procedures shall conform to ASTM D-1143-07, Standard Test Method for Pile under Static Axial Compressive Load and/or ASTM D3689-07, Standard Test Method for Pile under Static Axial Tension Load unless otherwise specified by the engineer. These ASTM specifications do not specify a particular method to be used, but rather provide several slow-testing and quick-testing optional methods.

Citing the Canadian Foundation Engineering Manual, 2007:

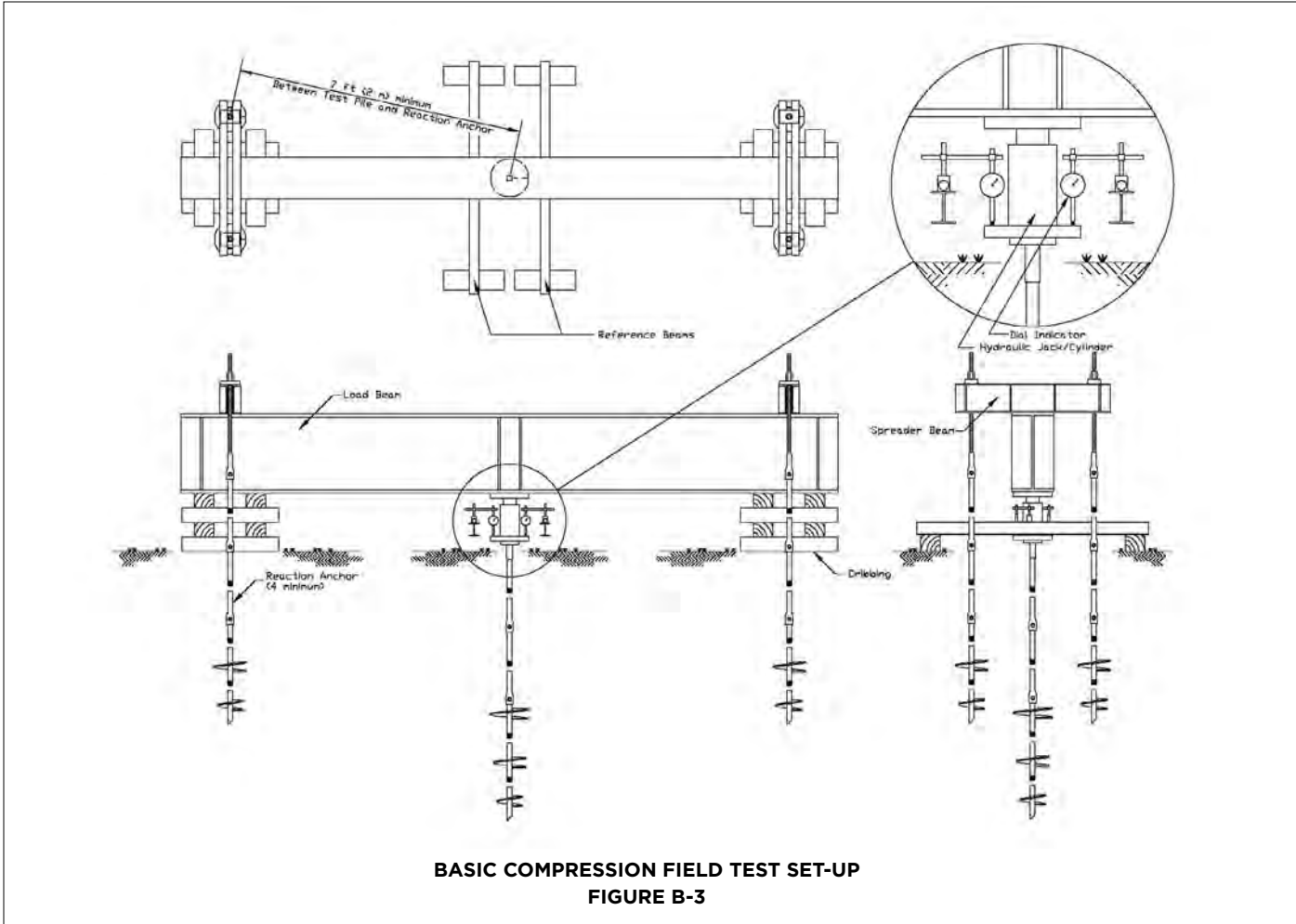
“The slow-testing methods . . . (outlined by the ASTM D1143-07. . . are very time-consuming. When the objective of the test is to determine the bearing capacity of the pile, these methods can actually make the data difficult to evaluate and disguise the pile true load movement behavior, thereby counteracting the objective of the test. The benefit of the (slow) test methods lies in the additional soil-pile behavior information, occasionally obtained, which the interpreting engineer can use, when required, in an overall evaluation of the piles.

“. . . For routine testing and proof testing purposes, the quick methods . . . are sufficient. Where the objective is to determine the bearing capacity of the pile . . . the quick test is technically preferable to the slow methods.”

Therefore, the following test procedure is based on the “Quick Load Test Method for Individual Piles”. This test procedure shall be considered to meet the minimum requirements for load testing. It is not intended to preclude local building codes, which may require the use of other testing methods as described in the ASTM specifications.

PRE-PRODUCTION LOAD TESTS

1. Determine the depth to the target stratum of soil from the geotechnical site investigation report that includes boring logs. Use these data to select an pile/anchor design capacity, ultimate capacity and estimate the installation torque at the target stratum and depth.
2. Set the spacing and install the four reaction anchors at the test site (see Figure B-3). The recommended spacing between the test pile and the reaction anchors is at least 5D, where D = diameter of the largest helical plate. For tension only tests, the reaction anchors are not required.
3. Install the test helical pile at the centroid of the reaction anchors to the target depth and torque resistance. For tension tests, install the test anchor at the desired location to the target depth and torque resistance.
4. Mount the two anchor beams on the four reaction anchors/piles and the reaction beam between the anchor beams (see Figure B-3). For tension tests, center the reaction beam over the anchor and support each end of the beam on cribbing or dunnage. The helical reaction piles are not required if the surface soils have sufficient bearing strength to support the cribbing/dunnage under the applied loading without excessive deflections.
5. Install a load cell, hydraulic load jack, actuator and pressure gauge. The center hole load jack will be mounted below the reaction beam for a bearing (compression) test (see Figure B-3) and above the reaction beam for an anchor (tension) test. A solid core hydraulic jack can be used for compression tests.
6. Set the displacement measuring devices. Deflection measuring devices can include analog dial or electronic digital gauges (must be accurate to .001”) mounted on an independent reference beam, a transit level surveying system, or other types of devices as may be specified by the engineer.
7. Apply and record a small alignment or seating load, usually 5% to 10% of the design load. Unless otherwise defined, the maximum test load shall be assumed equal to 200% of the design load. Hold the seating load constant for 10 minutes or until no further displacement is measured.
8. Set the displacement measuring device(s) to zero.
9. Axial compression or tension load tests shall be conducted by loading the helical anchor/pile in step-wise fashion as shown in Table B-3 to the extent practical. Pile/anchor head displacement shall be recorded at the beginning of each step and after the end of the hold time. The beginning of the hold time shall be defined as the moment when the load equipment achieves the required load step. There is a generalized form for recording the applied load, hold periods, and pile/anchor head deflections provided at the end of this Section.



10. Test loads shall be applied until continuous jacking is required to maintain the load step or until the test load increment equals 200% of the design load (i.e., 2.0 x DL), whichever occurs first. The observation period for this last load increment shall be 10 minutes or as otherwise specified. Displacement readings shall be recorded at 1, 2, 3, 4, 5 and 10 minutes (load increment maxima only).
11. The applied test load shall be removed in four approximately equal decrements per the schedule in Table B-3. The hold time for these load decrements shall be 1 minute, except for the last decrement, which shall be held for 5 minutes. Refer to Acceptance Criteria on later in section B for acceptable movement criteria.

NOTE: Refer to Helical Pile Load Tests in the Model Specification - Helical Piles for Structural Support at www.chancefoundationsolutions.com for further information regarding load test equipment, load test setup, dial gauges for monitoring anchor displacement, etc..

PRE-PRODUCTION TEST SCHEDULE, TABLE B-3

PRE-PRODUCTION TEST SCHEDULE			
CYCLICAL LOAD INCREMENTS (%DL/100)			
Load Increment	Hold Period (Min.)	Load Increment	Hold Period (Min.)
AL	1.0	AL	1.0
0.20DL	4.0	0.50DL	4.0
0.40DL	4.0	1.00DL	4.0
0.60DL	4.0	1.20DL	4.0
0.80DL	4.0	1.40DL	4.0
1.00DL	4.0	1.60DL	4.0
0.75DL	4.0	1.80DL	4.0
0.50DL	4.0	2.00DL	10.0
0.25DL	4.0	1.50DL	4.0
		1.00DL	4.0
		0.50DL	4.0
		AL	5.0

AL = Alignment Load, usually 10% of DL; DL = Design (Working) Load



PRODUCTION LOAD TEST PROCEDURES (OPTIONAL - AS SPECIFIED)

1. Follow the test setup procedures listed under Pre-Production Load Test Procedures (Items 1 through 7), except the maximum test load to be applied to the pile/anchor is the Design Load (DL). (This may be the only type of load test conducted depending on the conditions.)
2. The Contractor shall perform axial load tests on the number and location of helical piles as specified by the Owner. At the Contractor's suggestion, but with the Owner's permission, tension tests may be performed in lieu of compression tests up to 1.00 DL for helical piles with sufficient structural tension capacity. The requirements of Table B-4 may be regarded as a minimum, however, it is not recommended to test production helical piles to values of up to 2.0 DL unless the helical pile's failure load is significantly higher than 2.0 DL. The maximum production helical pile test load shall be determined by the Owner. For example, ASTM D1143 stipulates testing to 2.0 DL.
3. Axial compression or tension load tests shall be conducted by loading the helical pile/anchor in the load sequence as shown in Table B-4. Anchor/pile head displacement shall be recorded at the beginning of each step and after the end of the hold time. The beginning of the hold time shall be defined as the moment when the load equipment achieves the required load step. The observation period for this last load increment shall be 5 minutes or as otherwise specified. Displacement readings shall be recorded at 0.5, 1, 2, 3, 4, and 5 minutes (load increment maxima only).
4. The applied test load shall be removed in four approximately equal decrements per the schedule in Table B-4. The hold time for these load decrements shall be 1 minute, except for the last decrement, which shall be held for 5 minutes. Refer to Acceptance Criteria on later in section B for acceptable displacement criteria.

PRODUCTION TEST SCHEDULE (OPTIONAL - AS SPECIFIED), TABLE B-4

PRODUCTION TEST SCHEDULE	
LOAD INCREMENT	HOLD PERIOD (MIN.)
AL	0
0.20 DL	4.0
0.40 DL	4.0
0.60 DL	4.0
0.80 DL	4.0
1.00 DL	5.0
0.60 DL	1.0
0.40 DL	1.0
0.20 DL	1.0
AL	5.0

AL = Alignment Load, usually 10 of DL.
DL = Design (Working) Load

STATIC LOAD TESTS (LATERAL)

Helical pile/anchor offer maximum benefits structurally when loaded axially (concentrically) either in tension or compression. In certain design situations, the anchors/piles may be subjected to lateral loads and it is important to establish their lateral load capacity. Such applications may include support for communication equipment platforms, foundations for light poles, and sign standards or use as foundation systems for modular homes. It is recommended that the Field Lateral Load Test on pile/anchor be conducted under the supervision of a Registered Professional Engineer. The engineer will specify the test and measurement procedure, load increments, time intervals, and acceptable ultimate deflection consistent with specific project and load conditions. If the desired ultimate lateral load capacity and test lateral load capacity results are close, the engineer may choose to increase the diameter of the anchor/pile shaft and/or use a concrete collar on the anchor/pile head in order to achieve the desired Factor of Safety. Lateral load tests shall be conducted in accordance with ASTM D-3966-07, Standard Test Method for Piles under Lateral Load.

TEST PROCEDURE

1. In order to conduct a lateral load test on an installed pile/anchor, it is necessary to install a reaction anchor system. The reaction anchor system consists of helical pile/anchor installed at a battered angle, and using a test apparatus setup such as shown in Figure B-5. Once the reaction anchor system is installed, the test pile/anchor is installed to the specified estimated depth and design torque.
2. Threaded steel bar or cable shall be used to connect the test pile to the reaction anchor frame. A hydraulic ram and pressure gauge is installed to apply the test load(s) and to measure the applied force.
3. Set the displacement measuring devices. Displacement measuring devices can include analog dial or electronic digital gauges (must be accurate to 0.001") mounted on a reference beam, a transit surveying system, or other type of device as specified by the engineer.
4. For the Load Capacity Tests, follow steps 7 through 11 in the Static Axial Load Tests on the preceding pages.
5. A failure criterion is often established by the project engineer and will reflect project specific conditions. The load versus lateral deflection is plotted. Interpretation of these results to determine the ultimate and working lateral load capacities often requires engineering judgment. Refer to Acceptance Criteria later in this section for acceptable displacement criteria.

CAPACITY VERIFICATION FOR ATLAS® RESISTANCE PIERS

On occasion, a building owner or engineer may want confirmation that existing Atlas Resistance® pier underpinning is



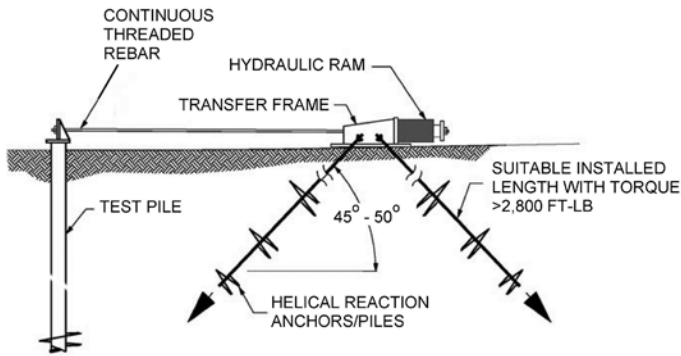
**INDOOR COMPRESSION TEST
FIGURE B-4**

supporting the load as initially designed. Many times this request comes as a result of a client seeing tension cracks in the drywall or masonry. Many such requests are generated as a result of the owner failing to improve a poor drainage situation, from a failure to maintain the soil moisture around the perimeter of the structure or from leaks in the plumbing system. It is possible that the stratum upon which the pier is founded is receding. Changes can also occur that increase subsurface water near the structure such as a drainage system becoming clogged or an inoperative sump pump. In partial underpinning situations, additional loads may be imposed from adjacent areas experiencing further settlement resulting in a much greater load from the time of the previous installation. In these conditions, additional piers will be required along with adjustment of affected earlier installed piers.

The following gives the dealer/installing contractor and engineer guidance for answering these concerns and the engineer assistance with specifications for pier bearing verification.

TEST AND ADJUSTMENT PROCEDURE

1. Excavate and expose the top half of the pier bracket at each location to be tested and adjusted.
2. Check the pier pins to see if they are tight by tapping the heads of the pier pins with a hammer and then attempting to remove the pins using pliers.
 - a. If the pier pins are loose:
 - The pier may be bearing on a stratum that is receding or that has deteriorated. Load test the pier.
 - The pier pipe or pier bracket component may have failed. If inspection of the components reveals a failure, replace the failed component and return it to Hubbell Power Systems, Inc. for evaluation. Load test the pier.
 - The footing may have heaved from expansion of the underlying soil if the floor slopes toward the interior. If evaluation of the structural elements, elevation measurements, drainage, and soil moisture content reveals heaving, then correcting the drainage or plumbing may allow the pier to return to the desired elevation. Schedule another inspection after the remedial work is complete and the soil has stabilized.
 - b. If the pier pins are tight but the floor slopes toward the perimeter:
 - The interior floor may be heaving. If an evaluation of the structural elements and elevations reveal interior heaving, a plumbing test, an evaluation of the surface drainage, and subsurface soil conditions should be performed and the deficiencies must be corrected before any attempt to adjust the perimeter is performed.
 - The bearing stratum may be receding or compressing under the pier load as the structure continues to settle. Load test the pier.
3. Load testing procedure for Atlas Resistance® piers:
 - a. Install a lift head onto the pier bracket and place a 25 ton hydraulic ram with hose, gauge, and hand pump on the top pier platform.
 - b. Slowly advance the ram while monitoring the top pier platform for creep.
 - c. If little or no movement is observed, then the end of the pier is probably still founded upon competent material. Continue to increase the force on the ram until the structure begins to lift. (If the pier advances into the soil more than the stroke of the ram, skip to step f below.)
 - d. Record the load test force that was required to begin to lift the structure. The formula for this force is: Gauge Pressure x 5.15 = Verification or Test Force (verify effective area of ram).
 - e. Compare this force to the force indicated on the original pier log. (Variation of ±15% is acceptable.) (Skip to step i below.)
 - f. Remove lift head assembly and top pier platform and install the pier driving equipment, drive stand, hydraulic drive cylinder, gauge, and gasoline or electric pump. Drive the pier pipe as if this was a new installation until suitable bearing is obtained. Record the driving force. The formula for this force is: Gauge Pressure x 8.29 = Driving Force (verify effective area of drive cylinder).
 - g. Cut the added pier pipe to proper length and record the added length required at this pier.
 - h. Install the top pier platform and lift head.
 - i. Repeat steps a through e for each pier that requires load bearing verification.
4. Procedure for Adjusting Piers:
 - a. Prepare a system of hydraulic rams and manifold(s) that are connected to all of the piers that need to be adjusted. Follow the normal elevation recovery procedure as described in the Typical Specification for the Atlas Resistance® pier system being tested. Typical Specifications are available on the Hubbell Power Systems, Inc. website, www.chancefoundationsolutions.com.



**LATERAL LOAD TEST EQUIPMENT CONFIGURATION
FIGURE B-5**



**LATERAL LOAD TEST APPARATUS
FIGURE B-6**

- The amount of downward movement required before reaching this force,
 - The lifting force, and
 - The amount of lift that was required to restore the foundation to the target elevation.
- b. Report to the engineer or owner any surface or subsurface drainage conditions observed and any suspected plumbing problems (such as water seeping into all or only several excavations). It is important that the Owner understand that any plumbing leaks or drainage deficiencies that are observed at the time of the adjustment be corrected immediately, otherwise stability issues may continue.

ACCEPTANCE CRITERIA

STATIC LOAD TESTS (TIEBACKS)

PRE-PRODUCTION AND PERFORMANCE TESTS

The net displacement shall not exceed 0.05" during the first log cycle of time, i.e., 1 min to 10 min. If the anchor movement between the one (1) minute and ten (10) minute readings exceeds 0.05", then the 1.25 DL test load shall be maintained for an additional 20 minutes. Displacements shall be recorded at 15, 20, 25, and 30 minutes. Net displacement is the difference between the movement recorded at the initial time increment and the final time increment of the log cycle of time. The initial time increment is 1 min and the final time increment is 10 min for the first log cycle of time for Pre-Production and Performance Tests.

The net displacement shall not exceed 0.10" during the final log cycle of time (examples, 3 min to 30 min, 6 min to 60 min, etc). If the acceptance criteria is not satisfied, then the anchor test shall be continued for an additional 30 minutes. Displacements shall be recorded at 45 and 60 minutes. If the acceptance criteria is not satisfied after this extended observation period, then the contractor shall exercise one of the options as provided in Section 6.5, Acceptance Criteria, in the Model Specification - Helical Tieback Anchors for Earth Retention found on www.chancefoundationsolutions.com.

PROOF TESTS

The net movement shall not exceed 0.05" during the first log cycle of time, i.e., 0.5 min to 5 min. If the anchor movement between the one-half (1/2) minute and five (5) minute readings exceeds 0.05", then the 1.25 DL test load shall be maintained for an additional 5 minutes. Displacements shall be recorded at 6 and 10 minutes.

The net displacement shall not exceed 0.10" during the final log cycle of time (examples, 1 min to 10 min, 3 min to 30 min, etc). If the acceptance criteria is not satisfied, then the anchor test shall be continued for an additional 20 minutes. Displacements

- b. Carefully apply pressure using the hand pump to restore the lost elevation. Valve off each ram as the foundation elevation reaches the target. Record the lifting force and the amount of lift at each placement. The formula is: Gauge Pressure x 5.15 = Lifting Force.
 - c. Once the structure has reached the target elevation, install pier shims and pier pins as described in the Typical Specification for the Atlas Resistance[®] pier system being tested. The Typical Specifications are available on the Hubbell Power Systems, Inc. website, www.chancefoundationsolutions.com.
 - d. Carefully reduce the hydraulic pressure at each ram, remove the rams and lift heads
 - e. Replace and compact the excavated soil and leave the area clean and neat.
5. Report the results:
- a. A Pier Installation Report shall be prepared that includes:
 - A pier layout of the area of work with each pier location indicated,
 - The verification or test force,

shall be recorded at 15, 20, 25, and 30 minutes. If the acceptance criteria is not satisfied after this extended observation period, then the contractor shall exercise one of the options as provided in Section 6.5, Acceptance Criteria, in the Model Specification - Helical Tieback Anchors for Earth Retention found on www.chancefoundationsolutions.com.

STATIC AXIAL LOAD TESTS (COMPRESSION/TENSION)

PRE-PRODUCTION LOAD TESTS

Acceptance of the load test results is generally governed by the building code for that jurisdiction and is subject to review by the structural designer. The structural designer determines the maximum displacement the structure can withstand without undue loss of function or distress. The acceptance criteria must be defined prior to conducting the load test.

The load displacement data may be plotted for a quick overview of the results. Figure B-7 shows a sample test plot. Various building codes have their own acceptance criteria, which is generally a limit on deflection at the factored load. A fast way to determine the ultimate geotechnical capacity is by use of a technique called the “intersection of tangents.” This is accomplished by graphically constructing two tangent lines. One line is drawn tangent to the second “straight line” portion of the load curve, which is beyond the curved or non-linear portion of the load deflection curve. The other line is drawn tangent to the initial “straight line” portion of the load deflection curve. The point where the two tangents intersect identifies an estimate of the ultimate capacity.

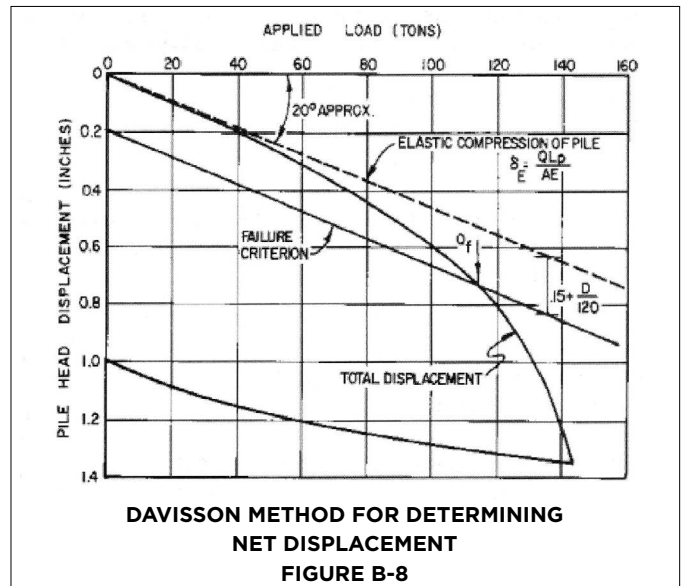
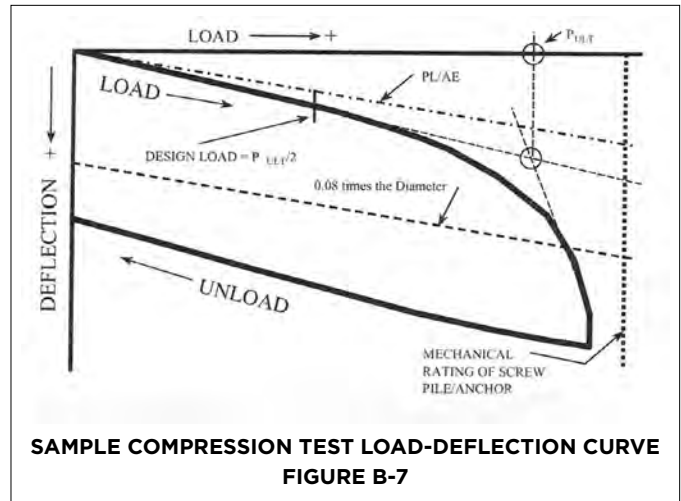
An example of a Code-based acceptance criteria for the allowable capacity is the Chicago and New York City Code, which calls for the design load to be the lesser of:

1. 50% of the applied load causing a net displacement (total displacement less rebound) of the pile of 0.01” per ton of applied load, or
2. 50% of the applied load causing a net displacement of the pile of 1/2”. Net displacement is defined as the gross displacement at the test load less the elastic compression.

Other allowable capacity acceptance criteria include:

- Maximum total displacement under a specified load.
- Maximum net displacement after the test load.
- Maximum displacement under the design load, or various techniques such as that defined by the Davisson Method (1973) and shown in Figure B-8.

The recommended acceptance criteria for the allowable geotechnical capacity for helical piles/anchors is 1/2 of the applied test load causing a net displacement (gross displacement less the elastic compression/tension) not to exceed 0.10 times the average diameter of the helix plate(s). This is the acceptance criteria used in ICC-ES Acceptance Criteria AC358 for Helical Systems and Devices, per Section 4.4.1.2. It is often referred to as the “modified Davisson” method.



Some jurisdictions such as the New York City Building Code, limit the allowable capacity of helical piles to 30 tons. The Chicago Building Code limits the allowable capacity of helical piles to 10 tons without a load test.

Figure B-9 is a plot of results from a compression “quick test” per ASTM D1143-07 of a 12 ft long, 1-1/2” square shaft helical pile having 10” and 12” helix plates. It was installed in the residual fine grained soils of Roanoke, Virginia and tested immediately after installation. The load-displacement curve is completely below the elastic compression line, indicating no skin friction was acting on the shaft during the test. The load-displacement curve does not cross the $PL/AE + 0.10D_{ave}$, which indicates the maximum test load is less than the ultimate geotechnical capacity of the helical pile.

Figure B-10 is a plot of results from a tension “quick test” per ASTM D3689-07 of a 16 foot long, 1-1/2” square shaft helical anchor having 8”, 10” and 12” helix plates. It was installed in the residual fine grained soils of Centralia, MO and tested immediately after installation. The load-displacement curve is



completely above the elastic tension line (red line), indicating no skin friction was acting on the shaft during the test. The load-displacement curve crosses the $PL/AE + 0.10D_{ave}$ line at approximately 41 kip. The average installation torque over the last three readings was 3,450 ft-lb. The torque correlation method (K_t) of capacity prediction says the ultimate geotechnical capacity is $3,450 \times 10 = 34,500$ lb (34.5 kip), using a K_t of 10 ft^{-1} as outlined in Section 6. The tested ultimate geotechnical capacity based on 10% average helix diameter net displacement is 41 kip. Therefore, the K_t based on the load test is $41,000/3450 = 11.9$ approximately 12.

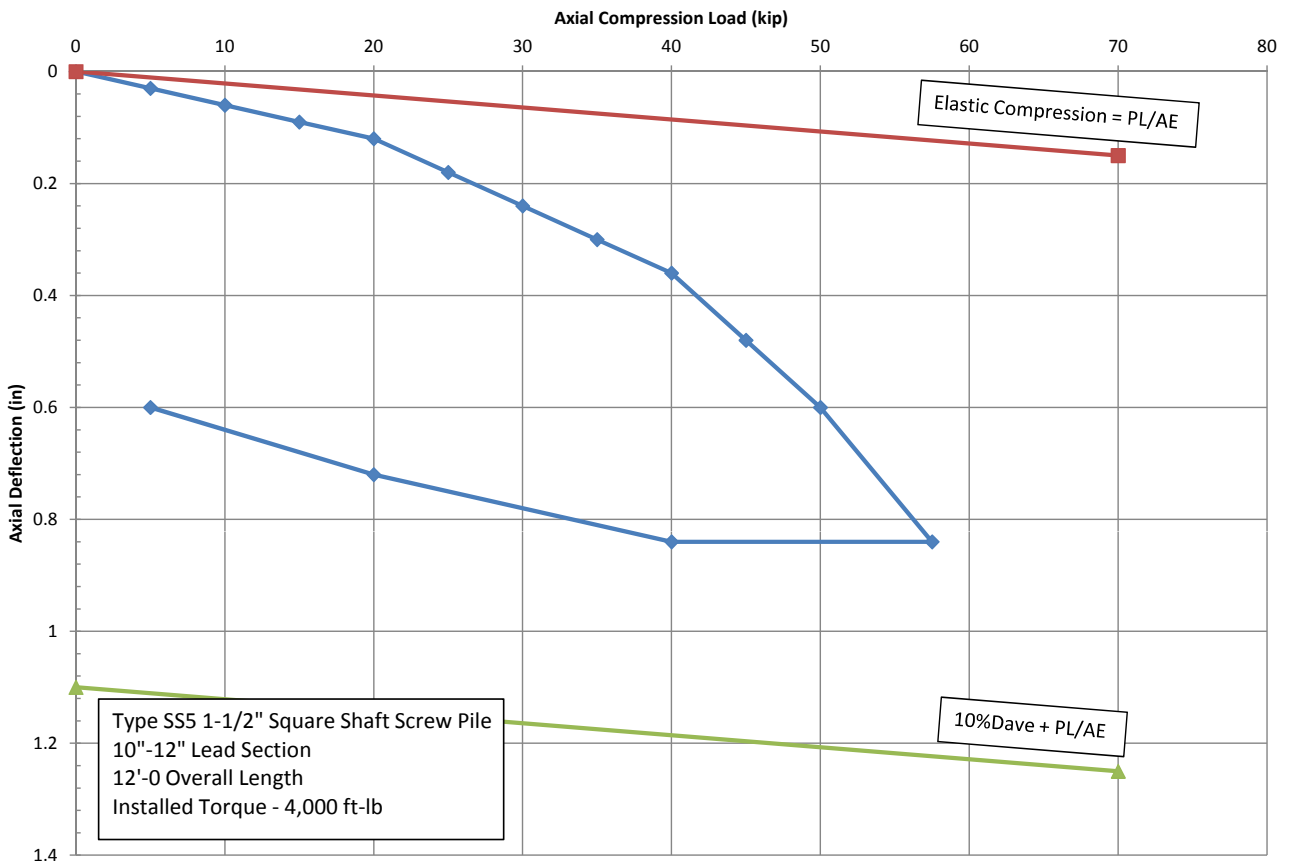
PRODUCTION LOAD TESTS (OPTIONAL)

Some projects are large enough in size to justify the expense of several production tests. Production tests are useful to verify helical anchor/pile capacity at multiple locations across the project site, especially with varying soil conditions. The net displacement of helical anchor/piles at the allowable load (1/2 the geotechnical capacity) typically ranges between 0.25 inches (25 mm) and 0.5 inches (51 mm) total vertical movement as measured relative to the top of the helical anchor/pile prior to

the start of testing. The Owner or structural engineer usually determines what the allowable displacement is, and it must be defined prior to conducting the Production Load Test. Limiting axial net deflections of 1" to 1-1/2" at the ultimate geotechnical capacity are typical.

STATIC LOAD TESTS (LATERAL)

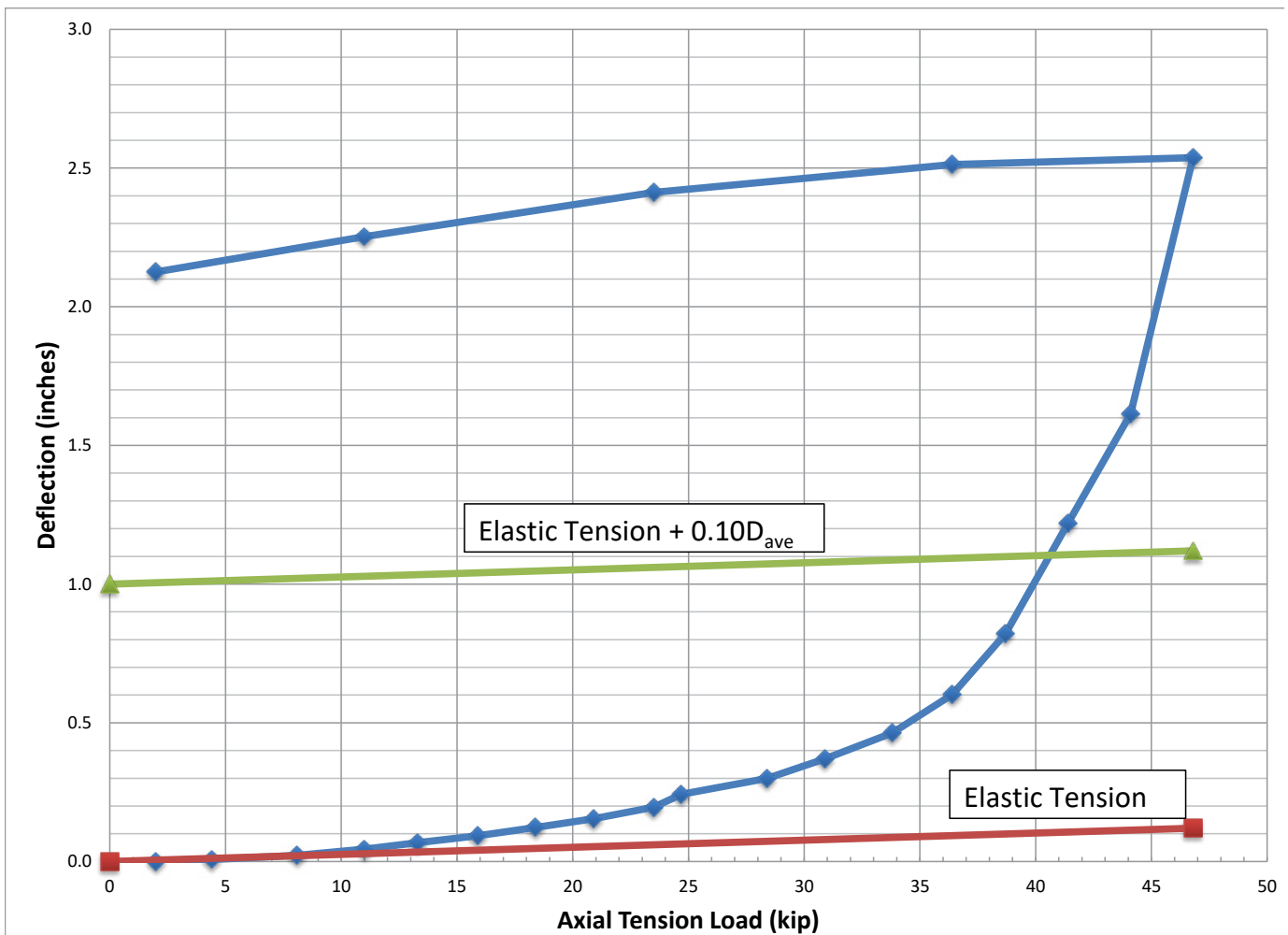
Acceptance Criteria for Helical Systems and Devices AC358 states the allowable load capacity shall be equal to half the load required to cause 1 inch (25 mm) of lateral deflection as measured from the ground surface. The acceptance criteria must be defined prior to conducting the Lateral Load Test. The acceptance criteria must be realistic in its magnitude so as not to potentially damage the structure. Limiting lateral deflections of 1"+ at the ultimate load capacity have been used on some projects. It is suggested that large lateral loads be resisted through some other means (such as helical anchors, battered helical piles, or enlarged concrete pile caps/grade beams).



ASTM D1143 "QUICK TEST" COMPRESSION PLOT, FIGURE B-9

REFERENCES

1. AC308 Acceptance Criteria for Helical Systems and Devices, ICC-Evaluation Services, June 2013 Revision.
2. ASTM D1143-07, Static Load Test Method for Piles under Static Axial Compressive Load, American Society for Testing and Materials, Philadelphia, PA.
3. ASTM D3689-07, Standard Test Method for Pile under Static Axial Tension Load, American Society for Testing and Materials, Philadelphia, PA.
4. ASTM D-3966-07, Standard Test Method for Piles under Lateral Load, American Society for Testing and Materials, Philadelphia, PA.
5. Canadian Foundation Engineering Manual, Canadian Geotechnical Society, 1985.
6. Crowther, Carroll L., Load Testing of Deep Foundations, John Wiley and Sons, 1988.
7. Davisson, M.T., High Capacity Piles, Department of Civil Engineering, Illinois Institute of Technology, Chicago, IL, 1973.



ASTM D3689 "QUICK TEST" TENSION PLOT, FIGURE B-10



Chance® Helical Pile/Anchor Axial Test

Project:	Date:	Sheet	of
Pile/Anchor Number:	Product Series: <input type="checkbox"/> SS <input type="checkbox"/> RS		
Helix Configuration:	Total Depth:		
Time: Start	Finish	Recorded by:	

LOAD TESTS

PRESS (psi)	LOAD (kip)	TIME (min)	DISPLACEMENT		
			GAUGE A (in)	GAUGE B (in)	GAUGE C (in)



APPENDIX C: HOW TO USE THIS MANUAL - DESIGN OF HELICAL PILES & ANCHORS A BASIC GUIDELINE FOR DESIGNERS

Contributors:

Cary Hannon, PE – Vice President of Engineering Foundation Technologies, Inc.
Gary L. Seider, PE – Engineering Manager, Hubbell Power Systems, Inc.

CONTENTS

- I. Introduction C-2
- II. Helical Pile Capacity C-2
- III. Design Process C-2
 - A. Data Gathering C-3
 - B. Feasibility C-3
 - C. P1, P2, P3 & P4 C-3
- IV. P4 - Geotechnical Capacity C-3
- V. P1, P2 And P3 - Structural Strength C-6
- VI. Summary C-11
- VII. Reliability C-11
- VIII. Other Topics Related To Design C-11
- IX. How To Specify Helical Piles C-13
- X. Construction Documents C-13

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 should be conducted and state and local building codes and authorities should be consulted 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 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.



I. INTRODUCTION

This Technical Design Manual (TDM) is a comprehensive collection of information for the express purpose to educate the practicing engineer in the art of helical pile design. The amount of information is extensive, and we recognize the need to provide a short length “primer” for the busy professional who does not have the time to read and learn all the comprehensive methods used to design helical piles. The goal of this “How To” is to bring the design and selection of helical piles and anchors into a short easy-to-follow Guideline. This Guideline will provide the design method used every day by the Application Engineering Staffs at Hubbell Power Systems, Inc. and its authorized Civil Construction Distributors. Citations throughout will direct the designer where to find the required information in the Technical Design Manual. The result is a simple step-by-step process culminating in a helical pile design that can then be correctly written into a project specification.

II. HELICAL PILE CAPACITY

The design method for helical pile capacity is simple. It consists of two limit states criteria; namely the Ultimate Resistance and the Serviceability Limit. Serviceability is the behavior of a helical pile at a particular load that is less than the ultimate resistance. For helical pile design, the Serviceability Limit primarily deals with limiting the deflection or displacement of the pile at a specified service load. Ultimate Resistance is the limit state based on the structural strength or the geotechnical capacity of the helical pile, defined as the point at which no additional load can be applied without failure. For helical pile design, ultimate resistance typically consists of two elements – the geotechnical capacity and the structural capacity, or strength. It is more descriptive to refer to structural “strength” of the helical pile components, which is the approach taken in the TDM.

A. According to the International Building Code (IBC) Section 1810.3.3.1.9, there are four ways to determine the ultimate resistance of helical piles.

- Method 1: Base resistance plus shaft resistance of the helical pile, where the base resistance is equal to the sum of the areas of the helical bearing plates times the ultimate bearing resistance of the soil or rock comprising the bearing stratum, and shaft resistance is equal to the frictional resistance of the soil times the shaft area above the helix bearing plates. This is commonly referred to as the theoretical geotechnical limit state method. It is described in great detail in Section 5 of the TDM.
- Method 2: Ultimate capacity determined from well documented correlations with installation torque. This is commonly referred to as the empirical geotechnical limit state method. The key words are “well documented” which will be discussed later. Torque correlation is described in Section 6 of the TDM.
- Method 3: Ultimate capacity determined from load tests. This is the most direct method to determine the geotechnical capacity of any pile, not just helical piles. Load testing of helical anchors and pile is described in Appendix B of the TDM.

- Method 4: Resistance of the pile’s structural elements (shaft, helix, couplings, connection to structure). Structural strength is described in Sections 5 & 7 of the TDM.

Of the four methods above, the only one that is unique to helical piles is Method 2, commonly referred to as torque correlation.

B. According to IBC Section 1810.3.3.1.9, the geotechnical capacity (Methods 1, 2, or 3 above) shall not exceed the strength of the pile’s structural elements (Method 4); including the pile connection to structure, pile shaft, pile shaft couplings, and the helix bearing plates. The structural strength of Chance brand helical piles is described in Section 7 of the TDM.

C. Therefore, both the geotechnical capacity and the structural strength of the helical pile must be determined; and whichever limit state is the lesser, will control the capacity. This is the ultimate resistance of the helical pile. In most cases, the geotechnical capacity will be the limit state, but the structural strength can sometimes control.

D. Allowable Strength Design (ASD) or Limits States Design (LRFD). ASD has been used for many years for the geotechnical capacity of deep foundations. It is sometimes referred to as deterministic design since the factor of safety is determined based on standard practice. LRFD is sometimes referred to as probabilistic design. It uses load factors and resistance factors based on statistically based probabilities of uncertainty. In the United States, most geotechnical design is deterministic based (global factor of safety); whereas in Canada most geotechnical design is probabilistic (limit states – ULS, SLS). The TDM includes both LRFD design and ASD allowable strength values, so the design can use either design method.

E. The Serviceability Limit may also control. Serviceability is the load/deflection response of a helical pile at a particular load of interest, i.e. a factored load well below the ultimate resistance limit state. There may be strict deflection limits required based on the application; the structure may be sensitive to overall settlement or differential settlement, which may require the helical pile ultimate resistance to be increased. For example, a deflection limit may be specified at the working/design load. Cherry and Perko (2012) reviewed hundreds of tension and compression load tests. They suggested that for end-bearing helical anchors/piles, the net displacement of the helix plates at the working loads averaged about 0.25 in (6.4mm). The working load is based on the geotechnical capacity divided by a factor of safety of 2 (deterministic design). Chance application engineers have either conducted or reviewed the results of several hundred load tests, which support the findings of Cherry and Perko. Serviceability limits should also take into account the elastic response of the helical pile material, which can be significant for deep piles with slender shafts.

III. DESIGN PROCESS

The designer has a specific task to perform, or problem to solve to which helical piles can offer a solution. At the beginning of the design process, it is best to keep all options on the table until circumstances dictate one foundation option(s) as being the better choice for the client. The designer should always



keep in mind the client is best served with a good solution at a reasonable price, both of which are not always intuitively obvious. As with any deep foundation, helical pile design has several steps. The steps can be summarized as:

A. Data Gathering:

The loads applied to the foundation. Section 4 of the TDM is a brief review of structural loads and provides several tables that can be used to estimate dead and live loads for various residential & commercial structures. If applicable, lateral loads must be included.

- The description and strength characteristics of the project soils. See Section 2 of the TDM for a brief review of soil mechanics and the procedures used for site investigations, which are typically summarized in the geotechnical report. Information needed in the geotechnical report includes: soil profile, N_{spt} values per ASTM D1586, depth to ground water, the presence of fill, debris, or cobbles, and bedrock.
- The designer must determine load resistance requirements and serviceability based on the application. This includes choosing either ASD with a deterministic factor of safety, or LRFD with probabilistic load and resistance factors. Section 5 for the TDM provides guidelines to evaluate soil properties for foundation design, and also gives estimates of helical pile displacement at working loads. Section 5 also provides the design methodology used with HeliCAP®, which is the design software most often used to determine the axial capacity of helical piles.
- The applicability of local, regional, or national building codes. The designer must comply with code requirements depending on the jurisdiction. For example, some codes require helical piles to be tested for every project. Others only require load tests if the pile capacity is above a certain limit. Codes often dictate acceptance criteria in terms of allowable displacement for deep foundations, such as the City of Chicago and New York building codes.
- Location tolerances. The helical pile designer must understand the location tolerances for the piles. For example, most Chance helical piles can be installed to a location tolerance of 1 inch or less, and an elevation tolerance of 1/8 inch. Angular tolerances are typically less than 2°.

B. Feasibility:

- Helical piles are designed to transfer load to soil or bedrock with a reasonable displacement. However, they are not designed to drill into solid rock. Table 7-4 is a quick reference guide for feasibility. It lists helical pile type based on the upper limit N_{spt} range of soils that pile type can be installed into, along with the typical upper limit of ultimate resistance. It's a good place to start for helical pile feasibility. For example, Type RS2875.276 2-7/8" OD pipe shaft helical piles can be installed into soils with N_{spt} blow counts up to 35 bpf.
- The size (diameter) of the helical pile shaft should be closely tied to its application. Chance offers small displacement (up to 4 in.), medium displacement (4 in., to 8 in.), and large displacement (> 8 in) helical piles. The pile

shaft should be large enough to transfer the axial and lateral loads to the soil. However, it is detrimental to oversize the helical pile shaft. This is because of torque correlation – the relationship between the amount of torque energy required to install a helical pile and its load capacity. Smaller diameter helical piles more easily advance like a screw, which minimizes soil disturbance and increases capacity efficiency. More information about shaft type and size will be presented later.

- Project site factors such as equipment access, overhead clearance, right-of-way restrictions, spoils disposal, noise restrictions, etc. must be considered. This is often where helical piles turn out to be the most cost effective deep foundation. Small equipment results in low mobilization cost and easy access.
- Manageable schedule must be considered as well. Helical piles and anchors can be loaded immediately after installation, which can save time compared to waiting for concrete or grout to cure.

C. It is convenient to break down the geotechnical capacity and the structural strength into subcategories or groups. For helical piles and anchors the groups are:

- P1 – bracket or connection to structure
- P2 – shaft, including couplings
- P3 – Helix(s)
- P4 – Soil (geotechnical) capacity, including resistance to both axial and lateral loads

We recommend the design sequence be inverted – start with P4 – soil (geotechnical) capacity because it usually will control the ultimate resistance.

IV. P4 – GEOTECHNICAL CAPACITY

The axial and lateral capacity is determined per the methods detailed in Section 2 and Section 5 of the TDM. Installation torque requirements can be estimated at this point. If a geotechnical report is available, use HeliCAP® Helical Capacity Design Software to determine the axial capacity (tension, compression, or both) via bearing capacity on the helix plates and side resistance on the shaft [Method 1]. HeliCAP® will help determine the shaft type (square shaft, pipe shaft, Combo Pile, or grouted Pulldown Pile), shaft size (diameter), pile depth, helix configuration (number and size of helix plates), and estimate the torque required to install the pile.

If a geotechnical report is not available, then axial capacity must be determined by other methods. Helical piles have the advantage of being installed (screwed) into the ground and then removed (unscrewed) quickly. A “probe” helical pile can be installed to assess the relative shear strength of the soil profile using torque correlation relationships per TDM Section 6. Well documented correlations with torque are used to estimate helical pile capacity based on the torque measured with the probe pile [Method 2]. The shaft type, shaft size (diameter), pile depth, helix configuration can be determined based on the probe pile.



The axial capacity can also be determined from full-scale load tests per Appendix B of the TDM [Method 3]. Full-scale tests are often used to verify Method 1 capacity and Method 2 torque correlation.

If a geotechnical report is available, the lateral capacity of a vertical shaft can be determined with various methods including the Finite Difference method (LPILE & GROUP by Ensoft®) and the Broms' Method (1964a) and (1964b) as detailed in Section 5 of the TDM [Method 1]. Each of these methods may be applied to Round Shaft helical piles or Pulldown® Micropiles. 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 piles can be used to resist lateral loads by components of the axial capacity on the battered pile. The induced shear and moment in battered piles often dictates the shaft size and batter angle.

If a geotechnical report is not available, the lateral capacity of a vertical shaft must be determined from load tests per Appendix B of the TDM [Method 3].

P4 SHAFT TYPE AND SIZE:

The shaft type/size is critical to both the axial and lateral capacity - especially for compression in soft/loose overburden soils where lateral stability of the shaft must be considered. The following is a brief summary of the 4 different shaft types for helical piles.

- Type 1 - Square Shaft: Square shaft piles are foundation elements that range in size from 1-1/2" solid round-cornered-square (RCS) to 2-1/4" solid RCS. They are compact sections, meaning they have relatively low section properties, but relatively large cross-sectional area since they are solid bars. They are more efficient than pipe shaft helical piles in regards to axial capacity derived from installation energy. A square shaft helical pile will have more axial capacity than a pipe shaft helical pile installed with the same amount of torsional energy into the same soil profile. Therefore, square shaft helical piles are better at penetrating dense material than pipe shaft helical piles.

Square shaft piles have slender cross sections. Therefore, they do not have a large cross section to resist much lateral load via passive earth pressure along the side of the shaft. In addition, they do not have much section modulus/stiffness to resist buckling under compressive loads without support from the surrounding soil. As long as there is sufficient soil confinement around the pile to prevent buckling, square shaft piles are suitable for compressive loads. As a general rule, if the soil profile has ASTM D1586 SPT N₆₀ value of 5 or greater, there is sufficient lateral support to prevent the square shafts from buckling at the compressive loads that they are rated for. If SPT N₆₀ values are 4 or less, then square shaft buckling may be a practical concern. A rigorous analysis can be done if enough reliable soil data is available, but the problem is best solved by selecting

either a pipe shaft or Helical Pulldown Micropile as described in the following sections.

The designer is encouraged to use square shaft helical piles as much as possible due to their advantages with torque correlation efficiency and better penetration in dense soil.

- Type 2 - Pipe Shaft: Pipe shaft piles are foundation elements that range in size from 2-7/8" OD pipe shaft to 10-3/4" OD pipe shaft with various wall thicknesses and material strengths. Pipe shaft piles have larger section properties compared to square shaft, so they are used to resist lateral load, or to provide stability when columnar buckling or potential unsupported length is a concern. The designer may ask why not use pipe shaft helical piles exclusively? The answer is square shaft helical piles offer greater axial capacity for a given amount of installation energy due to their greater efficiency (see the torque correlation in Table C-1). In addition, pipe shaft helical piles do not penetrate dense material as effectively as square shaft. Therefore, the designer must size the helical pile shaft large enough to transfer/resist all loads, but no larger than necessary. Helical piles evaluated per ICC-ES AC358 comply with the requirement of International Building Code (IBC) Section 1810.3.3.1.9 for the use of "well documented" correlations with installation torque.

Helical piles, whether they are square shaft or pipe shaft, are generally considered to be slender members. The lateral capacity is dependent on the effective projected area of the pile shaft, the flexural stiffness of the pile, and the resistance of the soil as the pile deflects laterally under load. Due to their slender size, helical pile shafts have relatively small effective projected area for the soil to bear against. Therefore, helical piles with shaft diameter ≤ 4" have about 4 kip lateral resistance; shaft diameters ≤ 8" have about 10 kip lateral resistance; and shaft diameters ≤ 10" have about 20 kip lateral resistance at typical allowable lateral displacements of 1" or less. As mentioned previously, square shaft helical piles don't have any significant lateral capacity.

TABLE C-1

Product Series	Description	Evaluated Per Ac358	K _t
SS125	1.25" Round Cornered Square Bar		10
SS5	1.50" Round Cornered Square Bar	Yes	10
SS150	1.50" Round Cornered Square Bar		10
SS175	1.75" Round Cornered Square Bar	Yes	10
SS200	2.00" Round Cornered Square Bar		10
SS225	2.25" Round Cornered Square Bar		10
RS2875.203	2.875" OD, 0.203 Wall Pipe	Yes	9
RS2875.276	2.875" OD, 0.276 Wall Pipe	Yes	9
RS3500.300	3.500" OD, 0.300 Wall Pipe	Yes	7
RS4500.237	4.500" OD, 0.237 Wall Pipe		6
RS4500.337	4.500" OD, 0.337 Wall Pipe	Yes	5.6
RS6625	6.625" OD, Varying Wall Pipe		5
RS8625	8.625" OD, Varying Wall Pipe		4
RS1075	10.750" OD, Varying Wall Pipe		2-3



- Type 3 - Combo Pile: A combo pile (Combination Pile) is a compression helical pile that has the advantages of both square shaft and pipe shaft. A combo pile has a square shaft lead section that is better at penetrating dense material and generating bearing capacity; and is then transitioned to a pipe shaft for the plain extensions where over-burden soils are softer/less dense and a larger section modulus is desired for lateral stability and/or buckling resistance, or when lateral load resistance is required. Another advantage provided by combo piles is the torque correlation factor (K_t) is increased compared to the straight pipe shaft pile per Table C-2 and Table C-3. Note as the overall shaft length increases, the K_t factor decreases.

TABLE C-2 - COMBO PILE LENGTH LESS THAN 30'-0

COMBO PILE TYPE	K_t , SAND	K_t , CLAY	K_t , COMBINED
SS5/150/RS2875	10	9.5	10
SS175/RS3500	9.5	9	9
SS200/RS3500	9.5	9	9
SS200/225/RS4500	7.5	7	7
SS175/RS2875	9.5	9.5	9.5

TABLE C-3 - COMBO PILE LENGTH GREATER THAN 30'-0

COMBO PILE TYPE	K_t , SAND	K_t , CLAY	K_t , COMBINED
SS5/150/RS2875	9.5	9.5	9.5
SS175/RS3500	9	8.5	8.5
SS200/RS3500	8.5	8	8
SS200/225/RS4500	7	7	7
SS175/RS2875	9.5	9.5	9.5

- Type 4 - A Helical Pulldown Micropile is a helical pile that has the shaft section encased in a small diameter grout column, typically 5" - 7" in diameter. Both square shaft and pipe shaft helical piles can be encased in a grout column, but square shaft is much more common. It has the advantage of the square shaft lead section to penetrate dense material for end-bearing. The added grout column provides greater section properties for shaft stability and lateral resistance in soft soils. Lateral load resistance with grouted shafts requires a steel case - typically extending 5'-0 to 10'-0 from the pile head. The grout in contact with the soil will develop side resistance via a bond zone in suitable soil stratum. This can greatly increase the total axial capacity of the pile (end-bearing and side resistance) as well as stiffen the axial load response of the pile. The grout column also provides additional corrosion protection to the steel shaft.

Grouted shaft Helical Pulldown Micropiles are recommended for square shaft piles in soft soils, when additional capacity via side resistance is needed, or when working loads exceed about 60 kip. To-date, Helical Pulldown Micropiles have achieved 450 kip ultimate resistance.

P4 OTHER CONSIDERATIONS:

There are several design considerations that should be taken into account when choosing the required shaft type. This is of-

ten the most important aspect of specifying a helical pile and too often receives the least amount of attention prior to installation.

1. Is the shaft section sufficient to carry the intended axial load? This will have a great deal to do with the selection of the shaft type. Refer to Table 7-4 of the TDM as a good place to start. It lists torque correlated capacities for shaft diameters up to 4.5" OD [Method 2]. Large diameter pipe shaft ($\geq 6"$) and Pulldown Piles can achieve higher capacities than those listed in Table 7-4. Allowable load upper limit for Chance helical piles up to 10" nominal diameter is 100 ton. Tension capacity is controlled by the structural strength of the couplings as detailed in P2 below.
2. The helix plates must generate the downward thrust required to advance the shaft through the soil. Helical piles (i.e. screw piles) are displacement piles that have the advantage of no spoils. The soil that is displaced by the shaft during installation is displaced to the side. The smaller the shaft size relative to the diameter of the helical plates (higher aspect ratio), the more efficient the pile will be in regards to capacity derived from the same installation energy. A helical pile that has a smaller shaft size relative to the size of the helical plates will be better at penetrating dense soil than one with a larger shaft size relative to the size of the helical plates (lower aspect ratio). Displacing more soil will require more installation energy, i.e. additional installation torque and down pressure. The greater the installation energy, the larger the required equipment to install the pile. For example, a 25 ton allowable load square shaft helical pile can be installed with a mini-excavator or skid-steer. However, an 8" diameter pipe shaft helical pile requires a 20 to 25 ton track-hoe excavator.
3. If a soil stratum is too dense, or the shaft too large relative to the size of the helix plates, the pile could "spin-out". "Spin-out" means that the pile is still being rotated but is not advancing, and installation torque drops dramatically. This is similar to "stripping" a screw. The capacity-to-torque correlation is no longer valid for spun-out piles. (Note: see Section 6 - Installation Methodology of the TDM for a complete explanation of torque correlation for helical anchors and piles). A spun-out pile is just an end bearing pile that was advanced to depth via a screw mechanism. This does not mean that the pile has no capacity, but rather that the capacity cannot be estimated by torque correlation as is normally done for a normally installed helical pile. The pile's capacity will depend on the type of material the helical plate(s) are in, how much the soil was disturbed, and whether or not the shaft tip, or pilot point, contributes to the capacity in end bearing. High capacities can be possible if the shaft tip is sitting on rock.
4. Lateral resistance requires either pipe shaft or Helical Pulldown Micropiles. A Helical Pulldown Micropile with a steel casing at the top of the pile will offer the stiffest pile section and the most resistance to lateral loads. Lateral capacity ranges from 2 to 4 kip for 3" to 4" diameter piles, 10 kip for 6" to 8" diameter helical piles, and up to 20 kip for 10" diameter piles at allowable lateral displacements of 1"



or less. The use of battered (inclined) piles can be utilized to resist lateral loads if needed and are discussed in Section 5 of the TDM.

5. For tension only foundation elements, square shaft is always the logical choice. As noted above, square shaft helical anchors are more efficient in regards to load capacity versus installation energy (torque correlation), are better at penetrating dense soils, and have less surface area for corrosion potential. The size and strength of the square shaft section is governed by the required installation torque, not the tension capacity. There is more steel section available than is required to carry the rated axial tension load. The reason for this is because the steel in the shaft is subjected to more stress during installation than it will ever see while in service. Once the helical anchor is installed, the tension strength is governed by the shear strength of the coupling bolt – see Section 7 of the TDM.
6. For piles required to resist compression and tension loads, the designer must recognize that helical piles are a pre-manufactured product with bolted connections. There is manufacturing tolerance in each connection. For example, most helical piles have up to 1/8" axial tolerance in each connection. The tolerance is required to ensure the connections fit together in the field. If the load reverses, the top of the pile will displace (up or down) a distance equaling the sum of the bolt tolerance in all of the bolted con-

nections before it can resist the reversed load. This may or may not be of concern to the designer and is dependent on the type of structure that is being supported with the piles. The grout column of Helical Pulldown Micropiles fills the connections, thereby removing the bolt tolerance as well as stiffening the axial load response. That is why grouted shafts are often utilized for piles with reversing load conditions. Grouting the ID of pipe shaft helical piles will also stiffen the coupling for reversing load conditions. Pipe shaft piles with couplings above grade should be grout filled to stiffen the connection.

V. P1, P2 AND P3 – STRUCTURAL STRENGTH

The axial and lateral strength of the helical pile components (shaft, helix and connection to structure) is determined per the methods detailed in AISC 360-10 Steel Construction Manual and Chapter 18 of the International Building Code (IBC). The structural strength of Chance helical piles is detailed in Section 7 of the TDM [Method 4]. The factors required for structural design are soil strength (firm, soft, fluid), the strength of the concrete, end condition (pinned, fixed, free), Application (new construction, remedial repair, tiebacks), coupling strength, and load direction (tension, compression, or both).

Soil strength is an important factor because it affects buckling & bracing of helical piles. It is important to categorize the proj-

TABLE C-C2.2 - APPROXIMATE VALUES OF EFFECTIVE LENGTH FACTOR K, FROM AISC 360-05

Buckled shape of column is shown by dashed line.	(a)	(b)	(c)	(d)	(e)	(f)
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code	<p> <i>Rotation fixed and translation fixed</i> <i>Rotation free and translation fixed</i> <i>Rotation fixed and translation free</i> <i>Rotation free and translation free</i> </p>					



ect soils as either “fluid” ($N_{60}=0$), “soft” ($0 < N_{60} < 5$) or “firm” ($N_{60} \geq 5$) as detailed in Section 5 of the TDM. If the soil is “fluid”, then buckling is possible and the shaft size is determined based on the critical buckling load. Examples are provided in Section 8 of the TDM (Examples 16, 17 & 18). If the soil is “soft” or “firm” buckling is not the concern, but depth to fixity and lateral support is. The term “fully braced” is used by some in the industry to describe a pile shaft with complete soil confinement all the way from the pile head to the tip. However, Hubbell Power Systems Inc. application engineers believe the term “fully braced” is unachievable from a practical standpoint. A “fully braced” condition is not listed as an option in Section 7 of the TDM since it is considered unrealistic and ensures the capacity of the helical pile will better match long term performance. Therefore, Section 7 details the nominal, LRFD design and ASD allowable compression strength of helical piles in terms of “firm soil” [5’-0 depth to fixity] and “soft soil” [10’-0 depth to fixity].

Once the soil strength is determined, the designer must now consider the end condition (K) at the pile head and how it affects the effective length of the pile shaft. The connection to the structure (and the effective length) greatly affects the structural capacity of the pile. A pinned condition means the pile head is restricted against lateral translation (side to side movement) but is free to rotate as shown in Table C-C2.2 (b) from AISC 360-05. A pinned condition uses a K of 0.7. A fixed condition mean the pile head is restricted against both lateral translation and rotation as shown in Table C-C2.2 (a). A fixed condition uses a K of 0.5. To achieve a fixed end condition, the pile head has to be embedded at least 7.5” from the bottom of a concrete pile cap/footing/grade beam. Anything less than that is typically considered pinned. A pile with a fixed end condition has a shorter effective length, thereby having a greater stability and higher axial compressive strength. The compressive strength of a “free” headed helical pile (Table C-C2.2 (e)) is not provided in the TDM. It can be provided as needed using a K factor of 2.0.

The strength of the concrete will also factor into the axial compressive strength of helical piles. Higher strength concrete results in higher bearing pressure with both embedded new construction pile caps (P1) and foundation repair brackets (P1).

Helical piles can be one-piece foundation elements, but are more commonly produced in sections that are coupled together during installation. Therefore, the strength of the coupling must be considered in the design as part of the shaft (P2). Chance helical pile couplings are designed to meet or exceed the torque correlated geotechnical capacity [Method 2]. They are also designed to meet or exceed the bending strength of the shaft itself. Structurally, the couplings limit both the tension and compression strength. For Chance Type SS helical piles, the coupling bolt is the limiting factor for tension strength.

Load direction is an important consideration and strongly affects the shaft type and size required. This was discussed previously under P4. The Application (new construction, foundation repair, earth retention, etc.) also affects the shaft type and size required. For example, it is not practical to use large diameter shaft helical piles for underpinning existing building structures.

Section 7 is broken down by specific helical pile product families. Each family sub-section lists the tension and compression strengths in various tables, in addition to specifications and available configurations. For example, the P2 (shaft) strength and P4 (geotechnical) tension capacity for Type SS175 helical piles are shown in Table C-4.

The pre-qualified and verified torque correlation factor (K_t) is 10 for Type SS175. The torque rating for SS175 is 10,500 ft-lb. Therefore, per P4 [Method 2], the torque correlated capacity limit for SS175 is 105 kip (see Section 6 TDM). The nominal strength of Type SS175 shaft (P2) is limited to 100 kip by the shear strength of the coupling bolt. Comparing the two, 105

TABLE C-4 - SS175 - P2 TENSION STRENGTH AND P4 TORQUE CORRELATED CAPACITY

TORQUE PROPERTIES				
Torque Correlation Factor	10 ft ⁻¹		33 m ⁻¹	
Torque Rating	10,500 ft-lb		14,240 N-m	
STRUCTURAL CAPACITY				
	Nominal		LRFD Design	
Tension Strength	100 kip	445 kN	75 kip	334 kN
Allowable Tension Strength	50 kip		222 kN	
TORQUE CORRELATED CAPACITY				
	Ultimate		Allowable	
Capacity Limit Based on Torque Correlation, Tension/Compression	105 kip	467 kN	52.5 kip	234 kN

kip > 100 kip, therefore P2 tension strength controls at max torque. If the installation torque is less than 10,000 ft-lb, then P4 [Method 2] will control. The allowable geotechnical capacity of 52.5 kip is based on a deterministic factor of safety of 2.

It is convenient to tabulate axial compression strength in terms of either P2 (shaft) & P3 (helix), or P1 (bracket) & P2 (shaft). Table C-5, from Section 7 of this manual, lists the P2 (shaft) and P3 (helix) ASD allowable strengths for Type SS175 square shaft helical piles. It is used to easily determine P2 and P3, which can then be compared to P4 to see which will control the design. The table is broken down by soil type, end condition, and number/diameter of the helix plates.

For example, a Type SS175 helical pile in firm soil & fixed end condition with multi-helix plates (3 or more plates) has ASD allowable compression strength of 98.3 kip. However, that exceeds the P4 geotechnical allowable capacity of 52.5 kip. But if the soil is soft with the same fixed end condition, the ASD allowable compression strength is 30.2 kip; which is less than the P4 geotechnical allowable capacity of 52.5 kip. The difference is the depth to fixity, which is 5’-0 in firm soils and 10’-0 in soft soils.

Table C-6, reprinted from Section 7 of the TDM lists the P1 (new construction bracket) and P2 (shaft) ASD allowable strengths for Type SS175 helical piles. It is used to easily determine P1 and



P2, which can then be compared to P4 to see which will control the design. The table is broken down by concrete strength, soil type and end condition.

For example, a Type SS175 helical pile in firm soil & fixed end condition with a new construction cap embedded in 2500 psi concrete has an ASD allowable compression strength of 52.7 kip based on the strength of the cap (P1). The P4 geotechnical allowable capacity of 52.5 kip is basically the same. But if the soil is soft with the same fixed end condition, the ASD allowable compression strength is 30.2 kip based on the shaft

strength; which is less than the P4 geotechnical allowable capacity of 52.5 kip. Again, the difference is the depth to fixity, which is 5'-0 in firm soils and 10'-0 in soft soils.

Table C-7, repeated from Section 7 of the TDM, lists the P1 (remedial repair bracket) and P2 (shaft) ASD allowable strengths for Type SS175 helical piles. It is used to easily determine P1 and P2, which can then be compared to P4 to see which will control the design. The table is broken down by concrete strength and soil type. Chance Remedial Repair Brackets provide fixed end condition at the bracket-shaft connection.

TABLE C-5 - SS175 - P2 SHAFT COMPRESSION STRENGTH AND P3 HELIX STRENGTH IN FIRM OR SOFT SOIL

Section Type & Helix Count	ASD Allowable Axial Compression Strength, kip (kN)			
	Firm Soil		Soft Soil	
	Fixed	Pinned	Fixed	Pinned
Lead, Single Helix	See Helix Strength Table Above	See Helix Strength Table Above	30.2 (134.3)	15.4 (68.5)
Lead, Single 12" Helix			28.7 (127.7)	
Lead, Single 14" Helix			25.9 (115.2)	
Lead, 2-Helix 8"-10"	94.7 (421.2)	61.7 (274.5)	30.2 (134.3)	15.4 (68.5)
Lead, 2-Helix 10"-12"	61.8 (274.9)	61.7 (274.5)		
Lead, 2-Helix 12"-14"	54.6 (242.9)	54.6 (242.9)		
Lead, 2-Helix 14"-14"	51.8 (230.4)	51.8 (230.4)		
Lead, Multi-Helix	98.4 (437.7)	61.7 (274.5)	30.2 (134.3)	15.4 (68.5)
Extension	98.4 (437.7)	61.7 (274.5)	30.2 (134.3)	15.4 (68.5)

TABLE C-6 - SS175 - P1 PILE CAP COMPRESSION STRENGTH AND P2 SHAFT COMPRESSION STRENGTH IN FIRM AND SOFT SOILS

ASD ALLOWABLE STRENGTHS OF NEW CONSTRUCTION PILE CAPS LOADED IN COMPRESSION^{1,2,3,4,5,6,7,8}

Catalog Number	Pile Model	ASD Allowable Compression Strength kip (kN)											
		2500 psi Concrete ⁶				3000 psi Concrete ⁶				4000 psi Concrete ⁶			
		Firm Soil		Soft Soil		Firm Soil		Soft Soil		Firm Soil		Soft Soil	
		Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned	Fixed
C1500458(G)	SS5	32.6 (145)	33.7 (150)	8.1 (36)	16.0 (71)	32.6 (145)	34.6 (154)	8.1 (36)	16.0 (71)	32.6 (145)	36.4 (162)	8.1 (36)	16.0 (71)
C1500459(G)	SS175	52.7 (234)	52.7 (234)	15.4 (69)	30.2 (134)	60.0 (267)	60.0 (267)	15.4 (69)	30.2 (134)	60.0 (267)	60.0 (267)	15.4 (69)	30.2 (134)
C1500465(G)	SS5	32.6 (145)	33.7 (150)	8.1 (36)	16.0 (71)	32.6 (145)	34.6 (154)	8.1 (36)	16.0 (71)	32.6 (145)	36.4 (162)	8.1 (36)	16.0 (71)
C1500467(G)	SS175	52.7 (234)	52.7 (234)	15.4 (69)	30.2 (134)	60.0 (267)	60.0 (267)	15.4 (69)	30.2 (134)	60.0 (267)	60.0 (267)	15.4 (69)	30.2 (134)

TABLE C-7 - SS175 - P1 REPAIR BRACKET COMPRESSION STRENGTH AND P2 SHAFT COMPRESSION STRENGTH IN FIRM AND SOFT SOILS

ASD ALLOWABLE STRENGTHS OF C1500299 REMEDIAL REPAIR BRACKETS & HELICAL PILES^{1,2,3,4,5}

Bracket Catalog Number	T-Pipe Catalog Number	Pile Model	ASD Allowable Strength in Axial Compression kip (kN)					
			2500 psi Concrete ⁵		3000 psi Concrete ⁵		4000 psi Concrete ⁵	
			Firm Soil	Soft Soil	Firm Soil	Soft Soil	Firm Soil	Soft Soil
C1500299	C1500488	SS175	34.7 (154)	27.7 (123)	39.3 (175)	27.7 (123)	47.9 (213)	30.2 (134)



For example, a Type SS175 helical pile in firm soil with a remedial repair bracket connected to an existing 2500 psi concrete footing has an ASD allowable compression strength of 36.8 kip based on the strength of the repair bracket (P1). The P4 geotechnical allowable capacity of 52.5 kip is greater, which means the bracket strength controls the design. This is also true if the soil is soft; the ASD allowable compression strength is 27.7 kip based on the bracket strength. The allowable load for remedial repair brackets is less because of the eccentric compressive load. Note from Tables C-6 and C-7 the allowable strength can increase with stronger concrete.

Note from Table C-5 that the allowable shaft (P2) compressive strength for SS175 in soft soils is significantly less than the torque correlated (P4) capacity. That is one reason why pipe shaft or grouted shaft helical piles are used.

Table C-8, from Section 7 of the TDM, lists the P2 (shaft) and P3 (helix) ASD allowable strengths for Type RS3500 3-1/2" OD pipe shaft helical piles. It is used to easily determine P2 and P3, which can then be compared to P4 to see which will control the design. The table is broken down by soil type, end con-

dition, and number/diameter of the helix plates. For example, a Type RS3500 helical pile in firm soil & fixed end condition with multi-helix plates (3 or more plates) has ASD allowable compression strength of 76.6 kip. But the ASD allowable compression strength in soft soil is either 65.9 kip with a fixed end condition, or 54.3 with a pinned end condition. These ASD allowable strengths are much higher than for SS175 in soft soil, which are 30.2 kip and 15.4 kip with fixed and pinned end conditions respectively. The P4 torque based geotechnical allowable capacity for RS3500 is 45.5 kip, which is less than the structural strength of RS3500 for any combination of soil type and end condition and thus controls the design. This is why SS/RS Combo piles are a good choice in soft overburden soil conditions.

Another way to increase structural strength is with grouted shaft Helical Pulldown Micropiles per Table C-9. The grout column increases the section modulus, which in turn increases the axial compression strength. Another benefit of the grout column is increased axial capacity (P4) base and shaft resistance, due to the soil-grout bond [Method 1].

TABLE C-8 - RS3500 - P2 SHAFT COMPRESSION STRENGTH AND P3 HELIX STRENGTH IN FIRM OR SOFT SOIL

ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE RS3500.300 HELICAL PILE LEAD & EXTENSION SECTIONS^{1,2}

Section Type & Helix Count	ASD ALLOWABLE AXIAL COMPRESSION STRENGTH KIPS (KN)			
	FIRM SOIL		SOFT SOIL	
	FIXED	PINNED	FIXED	PINNED
Lead, Single Helix	For Single 8" - 76.6 (340.7)	For Single 8" - 73.0 (324.7)	65.9 (293.1)	54.3 (241.5)
	See Helix Strength Table Above for 10", 12" & 14"	See Helix Strength Table Above for 10", 12" & 14"	For Single 12" - 49.2 (218.9)	For Single 12" - 49.2 (218.9)
Lead, 2-Helix 8"-10"	76.6 (340.7)	73.0 (324.7)	65.9 (293.1)	54.3 (241.5)
Lead, 2-Helix 10"-12"				
Lead, 2-Helix 12"-14"				
Lead, 2-Helix 14"-14"				
Lead, Multi-Helix	76.6 (340.7)	73.0 (324.7)	65.9 (293.1)	54.3 (241.5)
Extension	76.6 (340.7)	73.0 (324.7)	65.9 (293.1)	54.3 (241.5)

TABLE C-9 - SS175 GROUTED SHAFT IN SOFT SOILS P2 SHAFT COMPRESSION STRENGTH

NOMINAL, LRFD DESIGN, AND ASD ALLOWABLE COMPRESSION STRENGTHS OF CHANCE® TYPE SS175 GROUTED SHAFT PILES IN SOFT SOIL^{1,2,3}

Grout Column Diameter	Nominal, LRFD Design, and ASD Allowable Compression Strengths kip (kN)					
	Soft Soil					
	Pinned			Fixed		
	Nominal	Design	Allowable	Nominal	Design	Allowable
No Grout	25.8 (115)	23.2 (103)	15.4 (69)	50.5 (225)	45.4 (202)	30.2 (134)
5" OD	66.6 (296)	49.9 (222)	33.3 (148)	127.2 (566)	95.4 (424)	63.6 (283)
6" OD	111.5 (496)	83.6 (372)	55.7 (248)	185.6 (826)	139.2 (619)	92.8 (413)
7" OD	158.3 (704)	118.7 (528)	79.1 (352)	236.2 (1051)	177.2 (788)	118.1 (525)
8" OD	209.2 (931)	156.9 (698)	104.6 (465)	290.4 (1292)	217.8 (969)	145.2 (646)



For example, an SS175 helical pile with a 5" diameter grout column more than doubles the ASD allowable compression strength of the P2 shaft. Larger grout columns increase the structural strength even higher. This is an example where torque correlation [Method 2] does not limit the (P4) geotechnical capacity. Base and side resistance [Method 1] calculated with HeliCAP® Helical Capacity Design Software is often greater than strictly torque relationships.

The helix strength (P3) is best determined directly by testing.

The photos in Figures C-1 and C-2 (courtesy of CTL | Thompson) show how the helix strength can be determined. The load is applied through the shaft and resisted by the helix shaped fixture. The line of bearing is located at the average helix radius. The load is applied until the helix plate closes or the welds fail due to bending and shear. The test is stopped when the applied load begins to drop off. The maximum test load is considered the ultimate strength of the helix.



FIG. C-1 - P3 HELIX STRENGTH SET-UP



FIG. C-2 - RS2875 14" DIA. HELIX - TEST RESULTS

TABLE C-10 - SS175 P3 HELIX STRENGTH

NOMINAL, LRFD DESIGN & ASD ALLOWABLE STRENGTHS OF SS175 HELIX PLATES FOR SHAFT AXIAL TENSION & COMPRESSION¹

Helix Diameter in (mm)	Thickness in (mm)	Nominal Strength, kip (kN)	LRFD Design Strength, kip (kN)	ASD Allowable Strength, kip (kN)
6 (150)	0.5 (13)	123.3 (548.5)	92.5 (411.4)	61.6 (274)
8 (200)	0.375 (9.5)	84.5 (375.9)	63.4 (282)	42.3 (188.2)
10 (250)	0.375 (9.5)	66.1 (294)	49.6 (220.5)	33.1 (147.2)
12 (300)	0.375 (9.5)	57.5 (255.8)	43.1 (191.9)	28.7 (127.7)
14 (350)	0.375 (9.5)	51.8 (230.4)	38.9 (172.8)	25.9 (115.2)



The allowable helix strength (P3) must equal or exceed the end-bearing capacity (P4) of the of the helix plates. It is possible for the bearing capacity of a helix plate to exceed the structural strength of the helix plate. For example, an SS175 10" diameter helix plate has an allowable strength of 33.1 kip per Table C-10. If the maximum allowable torque based capacity of an SS175 helical pile (52.5 kip) is needed, then more than one 10" helix is required to meet structural strength requirements since 33.1 kip is less than 52.5 kip. A twin-helix or triple-helix configuration will work. This is an example where the designer may want to specify a minimum number of helix plates in the project plans.

As helix plate diameter increases, the helix strength (P3) generally decreases. This is because the line of bearing (average radius) increases with increasing diameter, which in turn increases the moment arm distance. The increased distance increases the bending forces at the helix/shaft welded connection.

Load tests [Method 3] are used to verify the feasibility and capacity of helical piles/anchors and are described in detail in Appendix B of the TDM. They can be part of a pre-production test program where at least one helical pile is installed and tested to determine the ultimate resistance and the load/deflection response. Project requirements may also require production tests on a specified number of helical piles/anchors to ensure capacity and performance requirements are being met. It is VERY IMPORTANT that the performance requirements be clearly specified BEFORE the start of work. It should be part of the data gathering process and feasibility assessment for helical piles. Helical piles are primarily end-bearing foundation elements, meaning they derive most of their resistance with the helix plates transferring load to the soil at the pile tip. Therefore, the load/deflection response of a helical pile at a particular load (serviceability) must take into account the section modulus and length of the shaft. The designer must understand that long end-bearing piles will displace more than short end-bearing piles because of the pile length.

The recommended acceptance criteria for the allowable capacity of helical piles/anchors is 50% of the applied test load causing a net displacement equal to 10% of the average helix diameter (D_{ave}). This means that total displacement of the pile/anchor may exceed 1 inch in order to fully mobilize the bearing capacity of the helix plates. This is the acceptance criteria used in ICC-ES Acceptance Criteria AC358 for Helical Systems and Devices, per Section 4.4.1.2. It can be expressed mathematically as $PL/AE + 0.10D_{ave}$, where "PL/AE" is the elastic shortening or lengthening of the pile shaft under load. As mentioned previously, the net displacement of the helix plates at allowable loads will average 0.25 in (6.4mm) \pm 0.12 in when using a geotechnical factor of safety of two.

VI. SUMMARY

In summary, helical pile design determines the geotechnical resistance (P4) and structural capacity (P1, P2, & P3), typically in that order. Probe helical piles and load tests are often done before start of work when a geotechnical report is not available or when verification of capacity is required. The geotechnical and structural resistance are separate limit states and which-

ever one is the lesser will control the design. In most cases, the geotechnical resistance (P4) will be the controlling factor. The designer is encouraged to design helical piles so that the geotechnical resistance (P4) controls to make the most efficient use of the soil's ability to bear load. This often means choosing the right shaft type/size, end condition, and helix configuration to maximize capacity.

VII. RELIABILITY

Reliability is an important aspect of helical pile design. Reliability is defined as the probability of long-term satisfactory performance. The better the capacity prediction method(s) used, the greater the reliability. Hubbell Power Systems, Inc. recommends using base plus shaft resistance [Method 1] and torque correlation [Method 2] to determine capacity whenever possible. Perko 2009 did a statistical analysis of helical pile capacity in order to check the reliability of this approach. He used a database of several hundred load tests in the analysis and used a factor of safety of 2 to determine a safe allowable load (deterministic approach). Using bearing capacity theory, the load test data suggests that 1 out of 10 helical piles will exhibit unsatisfactory performance. That is a 90% success rate, but still means 10% will have unacceptable performance. Using torque correlation, load test data suggests that 0.3 out of 10 will exhibit unsatisfactory performance. That's a 97% success rate which is much better, but still means that 3% will have unacceptable performance. Methods 1 and 2 are independent methods used to determine helical pile capacity. When two independent methods are statistically combined, the result of poor helical pile performance drops to only 3 piles out of 1000, or 0.3%. That is a 99.7% success rate, which most engineers agree is acceptable reliability. Loads tests [Method 3] is another independent method of capacity prediction which can be used when soil data is lacking or uncertain, or when soil conditions change.

VIII. OTHER TOPICS RELATED TO DESIGN

CORROSION POTENTIAL: Underground corrosion is discussed in detail in Appendix A of the TDM. In most ground conditions, corrosion is not a practical concern for deep foundations, including helical piles. There is typically little to no oxygen in undisturbed soils, especially below the ground water table. Driven steel piles have been installed with pile hammers for more than a century and are still commonly used today. The vast majority of interstate highway bridges in the Piedmont regions of the southeast United States are bearing on driven steel H-piles. If the geotechnical report declares the corrosion potential is moderate to severe for a given project, then a square shaft helical pile is a good choice because of its solid cross section and low perimeter surface area compared to a pipe shaft; which is hollow and has more perimeter surface area relative to the cross-sectional area of steel. Hot-dip galvanization adds a thick coating of zinc to the steel pile. It provides a durable coating that increases service life. Service life calculations based on metal loss rates can be done when corrosion potential data is



available. Appendix A of the TDM contains 4 design examples for corrosion design.

A Helical Pulldown® Micropile with its solid square shaft encased in a very dense grout mixture provides the most resistance to corrosion since the grout acts as an additional layer of protection. Cathodic protection, or adding a corrosion allowance (additional thickness of sacrificial steel) are also options in aggressive environments.

HELIX STRENGTH: The structural strength of an individual helix is dependent on the plate thickness, grade of steel, diameter, and strength of the weld that connects it to the pile/anchor shaft. There must be enough helix plates so that the sum of their individual strengths can share the load that is required of the pile/anchor. The product family sub-sections in Section 7 of the TDM provide the P3 helix strengths. A performance-based specification requires a minimum number of helix plates required to share the load. The size of each helix plate is left up to the installation contractor as long as the minimum number plates is provided, and that other requirements are met, such as minimum depth and installation torque. For example, if 60 kip capacity is required, and the individual helix strength is 40 kip, then a minimum of two helix plates are required to share the 60 kip load. A prescriptive-based specification would be explicit on the exact number and size of the helix plates.

HELIX SIZE AND CONFIGURATION: The size (diameter) of helix plates have a significant influence on the installation and performance of a helical pile/anchor. The helical configuration (number and size of helix plates) can change from pile to pile. The designer can choose between a performance based design and a prescriptive based design. A performance based design means the helical pile contractor is responsible for some design and construction procedures. A prescriptive based design means the owner or designer has the sole responsibility for all aspects of helical pile design and installation. Hubbell recommends using a performance based design in most situations.

An example of a performance based design for helical piles is minimum number of helix plates, minimum installation torque, and minimum depth. The contractor can then decide the actual number and size of helix plates, depths and torque required to achieve the required resistance; so long as the specified minimums are met. A prescriptive based design is the actual number and size of helix plates, actual installation torque, and actual depth. A prescriptive design may be required for comparative bid reasons and is fine as long as a payment mechanism for adjustment is provided. Typically, the denser the soil, the helix plates must be smaller. Alternately, the softer or less dense the bearing soil strata, the helix plates must be larger to generate the required torque/capacity.

It is important that the smallest helix plate be the bottom-most helix. A multi-helix pile will then have subsequent helices increasing in size. Generally, the same size helix is not repeated until the largest size available is reached. For example, a typical three-helix configuration would be an 8"/10"/12" or 10"/12"/14". The larger the shaft size, the larger the smallest helix diameter. For example, the smallest helix plate on pipe shaft is typically 10 in or larger.

Helical piles with multiple helix plates will drive straighter, and are more likely to advance properly than single helix configurations, and perform better. If too few helical plates are used, the most likely installation problem is "spinning out". This can be solved by adding more helix plates, larger helix plates, and/or more crowd pressure (downward force from installing equipment). Increasing crowd pressure may require a larger piece of equipment (excavator, backhoe etc.). Generally, adding more helical plates is more economical compared to upsizing to larger equipment. If too many helical plates are used, the likely installation problem is that the torque capacity of the shaft is reached prior to reaching the required depth. Helical extensions can be removed by unscrewing the pile/anchor, taking them off and reinstalling the pile/anchor. If helix plates on the lead section need to be removed, it will require the installation contractor to supply a different configuration lead section or remove helical plates in the field with a torch or saw. Removal of helix plates in the field is done quite often, but for cost/time reasons the installing contractor would prefer not having to remove helical plates regardless of the method.

MINIMUM LENGTH (DEPTH): The minimum length (depth) for helical piles to behave as a deep foundation is controlled by the depth to the top-most helix plate. The plate closest to the ground surface should be a minimum vertical depth of 5 diameters (5D) where D is the diameter of the largest helix. If the helix plate is not installed to this depth, the failure mode will be similar to a shallow foundation, i.e. a rupture of soil at the surface if there is not enough confining pressure. For example, if a site has loose overburden sand that trends to medium-dense sand with increasing depth, the minimum length requirement may be "the uppermost helix must be 5D below sub-grade". Most specifications simplify this to 5 feet below subgrade.

Helical piles are required to be a minimum length to ensure that the pile is deep enough to provide reliable, long term capacity. Minimum depth ensures the helix plate(s) are located in a soil stratum that will bear load over the long term with reasonable settlement. Geotechnical reasons can override the 5D requirement. Geotechnical reasons that affect minimum length are frost depth, seasonal change in moisture content, depth of fill, organic soils, volume change (shrink-swell) soils, expansive soils, liquefiable soils, and ground water fluctuations. For example, if it is known that a compressible peat layer exists between 15' and 20' depth, then it is important for the pile to bear in soil stratum below the peat layer. Therefore, a minimum depth should be required that locates the helix plates in a bearing soil below the peat layer, thereby ensuring the pile will not settle over time as the peat consolidates.

TENSION PILES/ANCHORS: The 5D requirement over the uppermost helix for tension elements is very important. If this requirement is not met, there is not enough confining pressure and a wedge or plug of soil can erupt to the surface as the anchor fails. 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.



For helical tieback anchors, the 5D requirement is 5D beyond the active failure plane, which is dependent on the friction angle of the soil and the wall height. It is important that the helical plates are not stressing soil in the active failure wedge. If this happens, the wall could experience a global type failure. Again, most specifications simplify this dimension to 5 feet beyond the active failure plane. Therefore, the minimum length requirement for helical tiebacks should be “the uppermost helix must be 5 feet beyond the active failure plane”. There should be a schedule, table, or formula for determining this in the field to ensure that the minimum length is achieved.

COST: The total installed length has a direct impact on the cost of the helical pile/anchor in both material cost and installation time. The designer must always keep this in mind. The length defined (or undefined) by the bidding documents has enormous ramifications on the cost. Well written bidding documents should define the piles well enough to obtain the pile/anchor performance that the owner requires, as well as obtain competitive pricing from the installing contractor. If the helical piles are not well defined, the installation contractor that leaves the most out of his bid will likely get the job. This is not good for the owner as it increases the likelihood that the owner is not going to get the performance from the piles that is needed; or be presented with an expensive change order after construction has begun. Bidding should be based upon a minimum estimated bid length with some method for adjustment for differing lengths. This approach better utilizes the flexibility of helical piles, which is one of their advantages. A thorough discussion of bidding and construction documents and strategies is discussed in Section X of this Guide, titled “Construction Documents”

IX. HOW TO SPECIFY HELICAL PILES

A. MINIMUM CAPACITY OR INSTALLATION TORQUE: Whether using a performance or prescriptive specification, the helical pile/anchor capacity (ultimate resistance) should be specified in order to ensure that the required pile/anchor resistance is achieved. This can be done by specifying the minimum capacity directly or indirectly by specifying the required installation torque. The designer can choose either way.

A.1: MINIMUM CAPACITY: Regardless of the design method used, the ultimate resistance is the same. Ultimate resistance is the limit state based on the structural strength or the geotechnical capacity of the helical pile, defined as the point at which no additional load can be applied without failure.

A factor of safety (or a resistance factor) is applied to the ultimate resistance to provide a reserve capacity greater than expected loads. This “normal use” load is commonly referred to as service, design, working, SLS or un-factored load. The safety or resistance factor may be prescribed by building code, but is often left up to the designer. A proper factor of safety/resistance is a combination of economics and statistics. It is not typically economically feasible to design for zero probability of failure. Generally the more uncertainty, the higher the factor of safety/resistance applied. Conversely, the less uncertainty, the

lower the factor of safety/resistance applied. For ASD design, the industry standard for helical piles is a factor of safety of 2 for permanent applications. For LRFD design, the resistance factor (ϕ) recommended for helical piles used in compression range from 0.65 to 0.75. The resistance factor (ϕ) recommended for helical piles used in tension range from 0.55 to 0.65.

For tieback anchors that are going to be individually post-tensioned and tested, a factor of safety of 1.5 is used. A lower factor of safety is justified since there is less uncertainty (the tieback is tested).

One problem with construction documents regarding helical piles/anchors is clearly identifying the capacity required. The best method is to clearly define the ultimate resistance required. If the designer chooses to specify the un-factored load, then the loads should be clearly identified as (service/ design/ working/SLS/un-factored loads) and clearly state what the required factor of safety/resistance is.

A.2: INSTALLATION TORQUE: Installation torque can also be specified as the minimum requirement as it relates to the pile/anchor capacity required. This should only be done for piles/anchors that will not receive a proof test. Installation torque should not be used to specify minimum capacity for helical tieback anchors when each anchor will be post tensioned and proof tested. In that case, passing the proof test is the only criteria that matters and obtaining a minimum torque is really a convenience for the contractor to ensure the anchorage does not fail the proof test.

If the installation torque approach is utilized, the designer should be aware that torque capacity correlations only apply to helical piles with advancement rate that equals or exceeds 85% of the helix pitch per revolution at the time of final torque measurement. Refer to Section 6 of the TDM for a full discussion of torque correlation (K_t) relationships. On-site testing can be used to obtain a site specific K_t , otherwise use the default values listed in Table C-1.

Also, tension and multi-helix compression capacity should be determined based on the average torque measured over the last three helix diameters of installed length. Most specifications simplify this to 3 feet. The reason this is done is to better predict the bearing capacity of the helix plates as they distribute load to the soil in a passive pressure bulb either below (compression) or above (tension) the helix plate(s). Depending on how fast the torque increases over the last 3 feet of penetration will have a significant impact on the capacity of the helical pile/anchor. Note that it is virtually impossible to average a helical anchor/pile’s maximum torque rating over the last three average helix diameters, which means a shaft with higher torque strength may be needed in very dense soils.

X. CONSTRUCTION DOCUMENTS

A. CONSTRUCTION PLANS: The previous sections presented the various design elements that should be considered when using helical piles/anchors. Each one of the following design elements should be defined in the construction plans on a well-engineered project.



- Shaft Type
- Shaft Size
- Helix Configuration
- Pile/Anchor Length
- Minimum Capacity or Install Torque

By defining the parameters that will be acceptable for each of these design elements, more favorable results will be obtained from both a pricing and performance perspective. It is the authors' experience that summarizing the pile/anchor parameters in a format similar as listed above works well.

For example, consider using the following format or similar plans:

TABLE C-11 - HELICAL PILE DATA SUMMARY

Pile Type	Square Shaft Helical Pile
Shaft Material:	Chance Type SS175 1-3/4" Solid Square Shaft
Helix Configuration:	8"/10"/12" Helix Plates
Bid Length:	28'-0
Ultimate Resistance, or	80 kip Minimum
Installation Torque:	8,000 ft-lb Min Average

Other design parameters can also be added such as grout column diameter for grouted Helical Pulldown® Micropiles, minimum length (if different from bid length), termination type, angle of installation, or required casing diameter & length. Soil conditions may also require the pile head end condition (fixed or pinned) be specified if shaft capacity controls the design.

The above summary provides enough information for bidders to aggressively bid on the same items as other bidders. It reduces the risk of being undercut by a contractor bidding with either lesser material, or a lesser estimated length. This also gives the owner and the engineer a comparative basis for their bid analysis. A method for payment should also be established for deviations from the bid length and should be considered in the bid analysis.

B. BIDDING DOCUMENTS: Well-crafted construction documents will allow installation contractors to accurately bid and properly install helical piles to serve their intended purpose. It is in the owner's and engineer's best interest for contractors to have the proper information to be able to accurately bid and properly install the piles/anchors. Poorly-crafted construction documents with lack of definition will result either in high pricing because the contractor has to assume an inordinate amount of risk, less than desired performance from the piles/anchors, installation problems, or change orders from the contractor. None of these things make the designer, or helical piles, attractive to the owner for future projects.

Bid processes can be handled in several different ways, and are dependent on the particular aspects and needs of each project. No two projects are exactly the same. Therefore, different aspects of the project may be the driving force behind the bid process or bid structure. These could be price, speed, or function. Helical piles/anchors are used in design/build projects, lump sum bids and projects with a unit pricing structure. It is

the writers' experience that unless there is a wealth of geotechnical information that is available to the bidder's, lump sum pricing is generally not in the owner's best interest. A pricing structure that shares some of the risk with the owner and the contractor tends to result in better overall pricing. One exception to this would be if the bidders are allowed access to the site to install probe or exploratory helical piles prior to bidding. Helical piles/anchors are well suited to exploratory installations because of torque-to-capacity relationships, the pile/anchor material can be recovered, and there is minimal disruption to the site. The less risk the contractor assumes, the better the pricing will be.

Generally, a pricing structure that allows for per/pile price to a specified bid depth with unit pricing for additional/deductible length works best. For example, if the geotechnical information available indicates the average pile/anchor depth to be between 25'-0 and 30'-0, then a bid length of 28'-0 might be established with unit pricing by the foot for piles that exceed or are short of that length. Unit pricing would likely be even better if it is based on increments of helical pile section lengths (5'-0 & 7'-0) rather than 1' increments, since 7'-0 is the most common section length. This is because the same amount of material is likely to be used once the contractor has to add an additional section. In other words, if the pile depth exceeds 28'- 0, there is an additional unit cost per unit additional 7'-0 extension. Some situations may lend themselves to providing a unit price for helical extensions. Many helical tieback projects have benefited by utilizing this approach.

Another unit pricing strategy is to have the bidders provide a unit price per foot for the entire length of piling or anchorage on the project and not have a price per pile/anchor. In other words, the construction plans might show 100 piles at an average 50' depth and the bid quantity would be set up for unit pricing by the foot, (or 7' increments) for 5000 lineal feet (LF) of piling. Payment would be made by the unit price for the quantity of piling installed, whether it is 4500 LF or 5500 LF.

C. TECHNICAL SPECIFICATIONS: Technical Specifications are an important part of well-crafted construction documents and should further define the details regarding helical piles or anchors. Technical Specifications should define anything that affects the pricing or performance of the piles or anchors. At a minimum, the following should be defined:

- Pile materials
- Installation tools and equipment
- Quality control methods
- Installation records required
- Installation tolerances and techniques
- Load testing requirements, procedures, and acceptance criteria (if any)

Model specifications for helical piles, anchors, and tiebacks that can be used as templates and edited for your specific project needs are included on www.chancefoundationsolutions.com.



APPENDIX D: FORMS

CONTENTS

Preliminary Design Request Form.....	D-2
Chance® Helical Pile/Anchor Axial Test Form	D-3
Atlas Resistance® Piers Installation Log	D-4
Chance® Helical Pile/Anchor Installation Log.....	D-5
Chance Helical Pulldown® Micropile Installation Log	D-6
Atlas Resistance® Piers - Project Summary Log.....	D-7
Pole Load Determination Data Sheet.....	D-8
Site Inspection Form.....	D-9

FORMS

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 should be conducted and state and local building codes and authorities should be consulted 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 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.



PRELIMINARY DESIGN REQUEST FORM

Contact at Chance Civil Construction: _____

Installing Contractor

Firm:	Contact:		
Phone:	Fax:	Cell:	

Project

Name:	Type:	<input type="checkbox"/> Foundation	<input type="checkbox"/> Underpinning/Shoring
Address:		<input type="checkbox"/> New Construction	<input type="checkbox"/> Rock
		<input type="checkbox"/> Tieback Retaining	<input type="checkbox"/> Other:
		<input type="checkbox"/> 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.



Chance® Helical Pile/Anchor Axial Test	
Project:	Date: Sheet _____ of _____
Pile/Anchor Number:	Product Series: <input type="checkbox"/> SS <input type="checkbox"/> RS
Helix Configuration:	Total Depth: _____
Time: Start _____ Finish _____	Recorded by: _____

PRESS (psi)	LOAD (kip)	TIME (min)	DISPLACEMENT		
			GAUGE A (in)	GAUGE B (in)	GAUGE C (in)

FORMS



ATLAS RESISTANCE® PIERS INSTALLATION LOG

Project:	Sheet: of:
Pier Number:	
Pier Designator:	Installation Date:
Maximum Work Capacity:	Installation Technician:
Installation Cylinder Effective Area:	

FORMS

DEPTH (ft)	PIER SECTION	PRESSURE (psi)	LOAD (lbs)	NOTES
3'-6	1			
7'-0	2			
10'-6	3			
14'-0	4			
17'-6	5			
21'-0	6			
24'-6	7			
28'-0	8			
31'-6	9			
35'-0	10			
38'-6	11			
42'-0	12			
45'-6	13			
49'-0	14			
52'-6	15			
56'-0	16			
59'-6	17			
63'-0	18			
66'-6	19			
70'-0	20			
73'-6	21			
77'-0	22			
80'-6	23			
84'-0	24			
87'-6	25			

Total Full Section Length:	Length of Cut-Off Section:
Depth to Pier:	Total Depth from Grade:

Lifting Log

Lift Ram Effective Area:	Date of Lift:					
Final Lift	Comments:					
<table border="1"> <tr> <td>Lift Amount (in)</td> <td>Pressure (psi)</td> <td>Load (lbs)</td> </tr> <tr> <td> </td> <td> </td> <td> </td> </tr> </table>		Lift Amount (in)	Pressure (psi)	Load (lbs)		
Lift Amount (in)	Pressure (psi)	Load (lbs)				



CHANCE [®] HELICAL PILE/ANCHOR INSTALLATION LOG			
Project:		Date:	Sheet: of:
Pile/Anchor Number:		Product Series: <input type="checkbox"/> SS <input type="checkbox"/> RS	
Helix Configuration:		Installation Angle:	
Time: Start	Finish	Recorded by:	
Chance Helical Pulldown [®] Micropile Only:			
Grout Column Diameter:		Sleeve Depth: From to	

DEPTH (ft)	PRESSURE (psi)	TORQUE (ft-lb)	COMMENTS or MICROPILE GROUT FLOW (Volume/Shaft/Length)

FORMS



Atlas Resistance® Piers - Project Summary Log

Project: _____ Project Completion Date: _____

Sheet _____ of _____

Pier Number	Date	Total Depth	Install Pressure	Install Load	Stage	Final Lift Pressure	Final Lift Load	Final Lift Amount	FS Drive vs Lift
1									
2									
3									
4									
5					←				
6									
7									
8									
9									
10					DRIVE				
11									
12					←				
13									
14									
15									
16									
17					→				
18									
19									
20					LIFT				
21									
22									
23									
24									
25									
26					→				
27									
28									
29									
30									

Report Prepared By: _____ Date: _____

FORMS



Chance Helical Pulldown[®] Micropile Installation Log

Project:	Date:	Sheet	of
Pile/Anchor Number:	Product Series: <input type="checkbox"/> SS <input type="checkbox"/> RS		
Helix Configuration:	Installation Angle:		
Grout Column Diameter:	Sleeve Depth: From to		
Time: Start	Finish	Recorded by:	

DEPTH (ft)	PRESSURE (psi)	TORQUE (ft-lb)	GROUT FLOW (Volume/Shaft/Length)

FORMS



Need by: _____

Pole Load Determination Data Sheet

Contact Name: _____ Phone: _____ Date: _____

Job Name: _____ Job Location: _____

***Select Appropriate Units of Measure**

1. Luminaire Mounting Height: _____ m ft
2. Height of Pole: _____ m ft
3. Outside Diameter of Pole Top: _____ cm in
4. Outside Diameter of Pole Bottom: _____ cm in
5. Arm Length: _____ m ft
6. Arm Tip Outside Diameter: _____ cm in
7. Arm Bottom outside Diameter: _____ cm in
8. Luminaire Weight: _____ kg lb
9. Luminaire EPA (Projected Area x C_d): _____ m² ft²
10. Basic Wind Speed: _____ kph mph
11. Minimum Design Life (Select Choice): 10 25 50
Design life default is 25 years. See Table 3-3, below
12. Number of Arms: _____
13. Number of Luminaires: _____
14. Pole Shape (Select choice from list below)

<input type="checkbox"/> Cylinder	<input type="checkbox"/> Hecdecagonal (16 Sides)	<input type="checkbox"/> Octagonal (8 Sides)
<input type="checkbox"/> Flat	<input type="checkbox"/> Dodecagonal (12 Sides)	<input type="checkbox"/> Square (4 Side) <input type="checkbox"/> Diamond
15. Arm Shape (Select choice from list below)

<input type="checkbox"/> Cylinder	<input type="checkbox"/> Hecdecagonal (16 Sides)	<input type="checkbox"/> Octagonal (8 Sides)
<input type="checkbox"/> Flat	<input type="checkbox"/> Dodecagonal (12 Sides)	<input type="checkbox"/> Square (4 Side) <input type="checkbox"/> Diamond
16. Anchor Bolt Diameter: _____ cm in
17. Number of Bolts (in base plate): _____
18. Bolt Circle Diameter: _____ cm in
19. Special Cableway Requirements: _____
20. Site Soil Conditions (if available): _____
21. CHANCE Precast Concrete Collar: Yes No 12" H or 24" H

Table 3-3. Recommended Minimum Design Life

Reproduced from AASHTO Specification, 4th Edition, 2001

Design Life	Structure Type
50 Years	-Luminaire support structures exceeding 15m (49.2 ft.) -Overhead sign structures
25 Years	-Luminaire support structures less than 15m (49.2 ft.) in height -Traffic signal structures
10 Years	-Roadside sign structures

TD_04_196E

FORMS



SITE INSPECTION FORM

Customer Name:	Date:	Sheet: of:
Address:	City/State/Zip:	
Phone:	Email:	
Time: Start Finish	Inspected by:	

POTENTIAL MAJOR GEOLOGICAL HAZARDS

Location of faults _____
 Stability of hillside _____
 Flood plain location _____
 Potential for excessive water runoff due to future development

ADDITIONAL NOTES AND SKETCHES

SITE CONDITIONS

Slope of the land _____
 Configuration of fill wedge _____
 Depth of rock _____
 Depth of water table _____
 Examination of nearby creeks _____
 Examination of nearby house _____
 Condition of roadway adjacent to house (potential slab creep & slope failure problems) _____
 Examination of road curbs _____
 General Topography _____
 Surface Drainage _____

ACCESSIBILITY

Equipment Access on site? _____
 Head-room (If interior install) _____
 What length of extensions are needed? _____
 Does homeowner want to: Lift structure Stabilize
 Areas to avoid (gardens, patios, etc.) _____

FOUNDATION CONDITION

Foundation Type: Basement Crawl space Slab-on-grade
 Foundation construction: Block Stone Mono-block
 General appearance of foundation _____
 Map cracks in foundation _____
 Proximity of trees to foundation _____
 Gutter and downspouts _____
 Water in crawl space _____

STRUCTURAL CONDITION

Examination of roof-line to detect general movement _____
 Openings square _____
 Cracks in plaster around opening _____
 Cracks around windows and doors (put on sketch) _____
 Floor level, may require levels conducted _____
 Construction of home: _____ Brick: _____
 Frame: _____ Age: _____
 Floor system construction (include description of framing, joist size, and spacing) _____
 Structure has been previously pierced: Yes No



Chance® Civil Construction | Hubbell Power Systems, Inc.
Centralia, Missouri 65240 | U.S.A.

© Copyright 2023 Hubbell Incorporated. Chance® is a registered trademark of Hubbell, Inc. All product names and brands are property of their respective owners.



Printed in the U.S.A. on recycled paper.

Because Hubbell has a policy of continuous product improvement, we reserve the right to change design and specifications without notice.

CENT-CV-TECHNICAL-DESIGN-MANUAL-TECHP-EN-V5-00350 Bulletin
01-0605, Revised 04/28/23



A proud member of the Hubbell family.