Table 3.1. Relationship of Modulus of Subgrade Reaction (k_1) to Undrained Shearing Strength of Stiff Overconsolidated Clay (After Elson)

Consistency	Stiff	V. Stiff	Hard
Undrained shear strength (C_u) kN/m^2	50-100	100-200	>200
Range of k_1 MN/m^3	15-30	30-60	>60
Soil modulus (K) MN/m^2	3-6	6-12	>12

For most normally consolidated clays and for granular soils the soil modulus is assumed to increase linearly with depth, for which

Stiffness factor $T = \sqrt[5]{(EI/n_h)}$ (in units of length) Eqn. 3.3

Where: $K = n_h \times x/B$ Eqn. 3.4

Table 3.2. Values of n_h for cohesionless soils (Terzaghi, 1955)

Soil Compactness Condition	n_h (Above Groundwater) KN/m^3	n_h (Below Groundwater) KN/m^3
Loose	2200	1300
Compact	6600	4400
Dense	18000	11000

Having calculated the stiffness factors R or T, the criteria for behavior as a short rigid pile or as a long elastic pile are related to the embedded length L as follows in *Table 3.3.*

Table 3.3. Criteria for Short Pile vs. Long Elastic Pile

Pile Type	Soil modulus	
	Linearly increasing	Constant
Rigid (free head)	$L \leq 2T$	$L \leq 2R$
Elastic (free head)	$L \geq 4T$	$L \geq 3.5R$

Helical Pier Systems utilizes and recommends Broms' method to determine the ultimate lateral resistance for an Helical Pier Systems screw type piling. These piles are most often classified as "Unrestrained or Free-Head short rigid piles". (See Broms (1964a) and Broms (1964b) in the References).

LATERAL ULTIMATE RESISTANCE OF PILES

For uniform cohesionless soils, Broms (1964b) has established the graphical relationships for $H/K_pB^3\gamma$ and $Mu/ B^4\gamma K_p$ shown in *Figure 3.4 (For short piles) and Figure 3.5 (For long piles)*, from which the ultimate lateral resistance H_u can be determined.

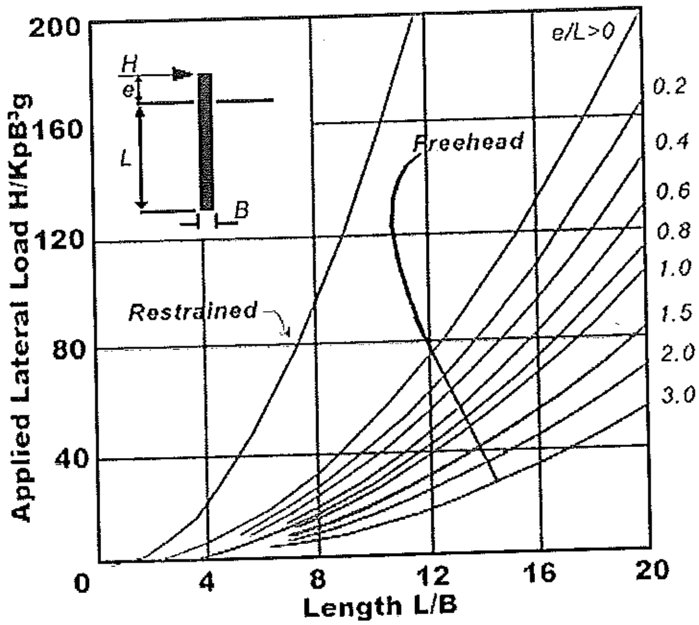


Figure 3.4: Ultimate Lateral Resistance of Short Pile in cohesionless soil related to embedded length (after Broms)

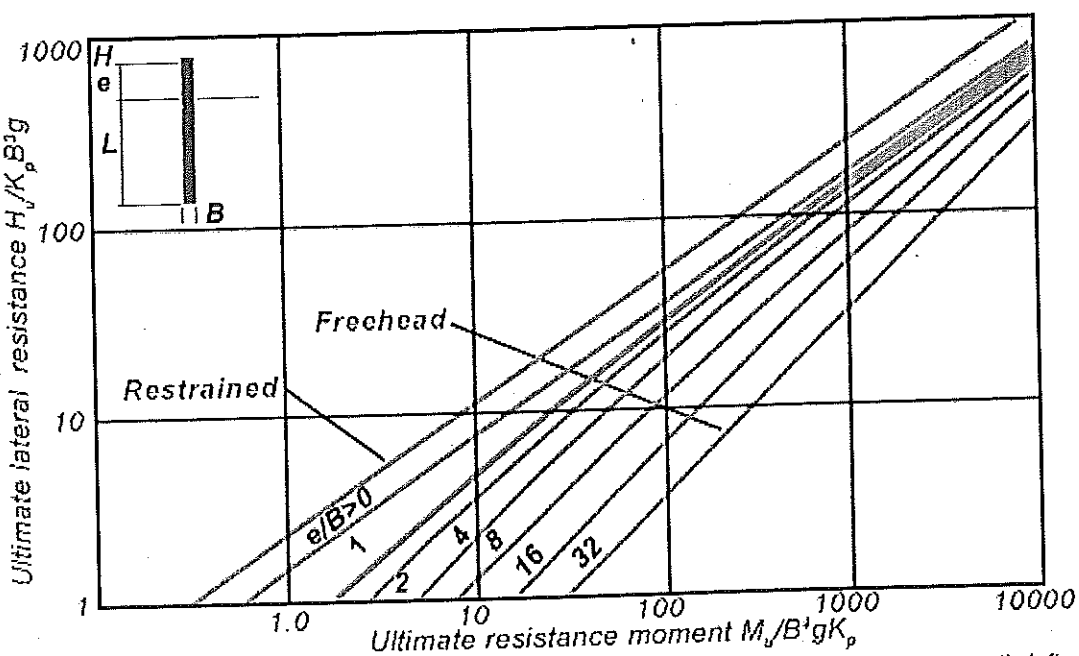


Figure 3.5: Ultimate Lateral Resistance of Long Pile in cohesionless soil related to embedded length (after Broms)

For uniform cohesive soils, Broms (1964) has established the graphical relationships for H/C_uB^2 and Mu/C_uB^3 *Figure 3.6(For short piles) and Figure 3.7 (For long piles),* from which the ultimate lateral resistance H_u can be determined.

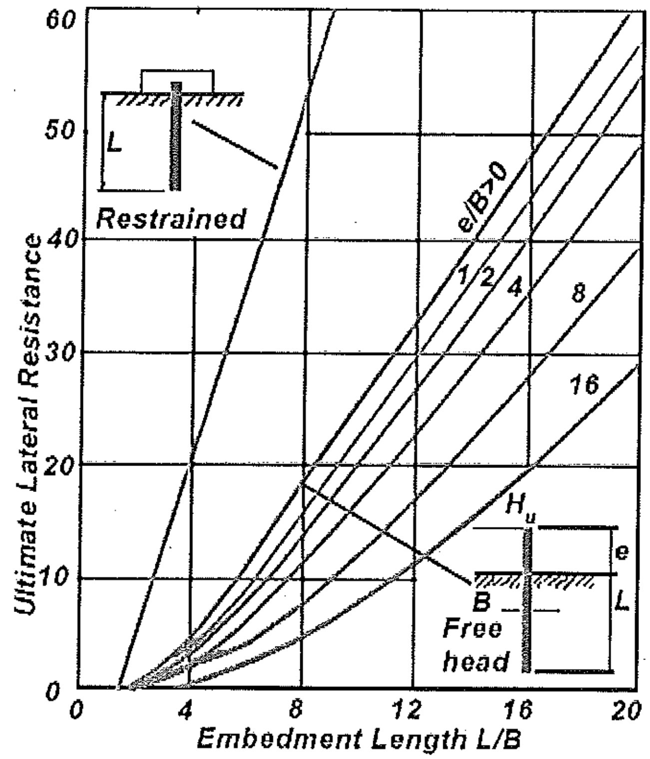


Figure 3.6: Ultimate Lateral Resistance of Short Pile in cohesive soil related to embedded length (after Broms)

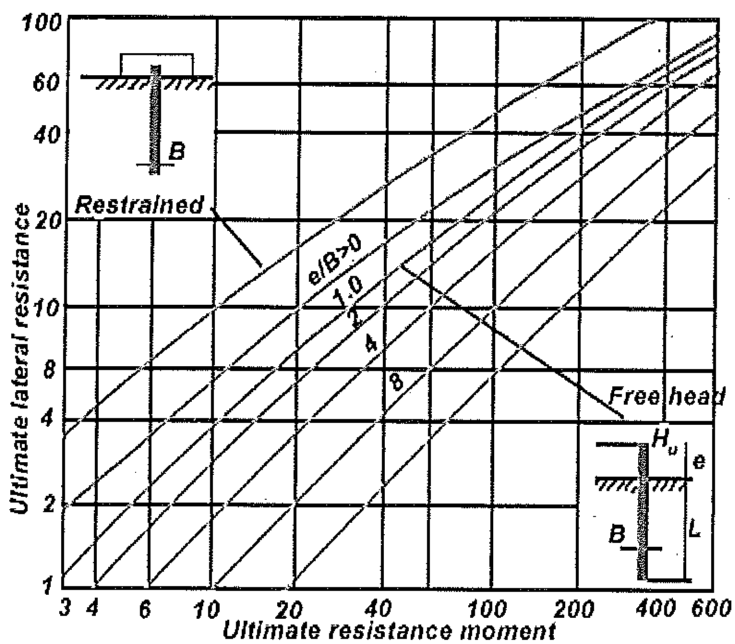


Figure 3.7: Ultimate Lateral Resistance of Long Pile in cohesive soil related to embedded length (after Broms)

DEFLECTION OF VERTICAL PILES CARRYING LATERAL LOADS

In cohesive soils the deflection behavior depends on the dimensionless length βL where

$$\beta = \sqrt[4]{(KB / 4EI)} \qquad \text{Eqn 3.5}$$

Where Y_0 is the pile head deflection for lateral load (H) in the dimensionless lateral deflection in Figure 3.8.

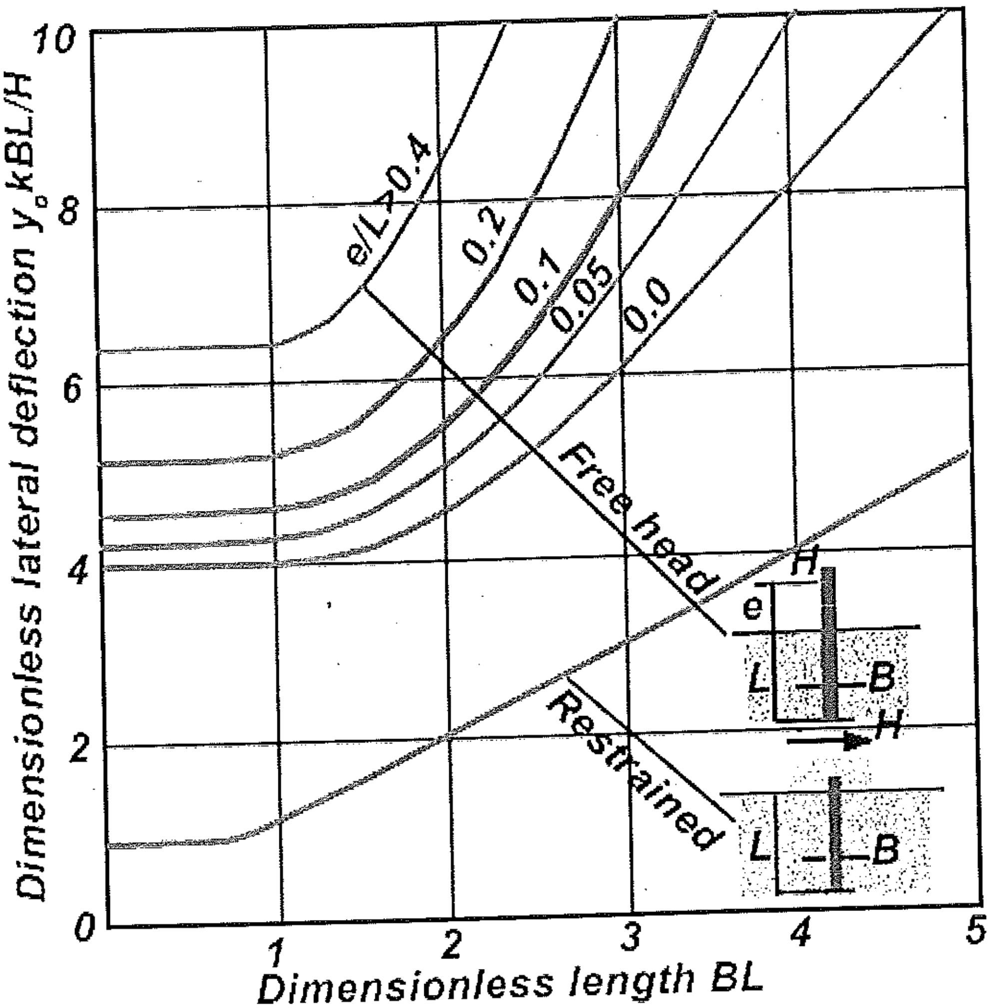


Figure 3.8: Lateral Deflection of Pile Head in cohesive soil (after Broms)

In cohesionless soils the deflection behavior depends on the dimensionless length ηL where:

$$\eta = \sqrt[5]{(\eta_h / EI)} \qquad \text{Eqn 3.6}$$

Where Y_0 is the pile head deflection for lateral load (H) in the dimensionless lateral deflection in Figure 3.9.

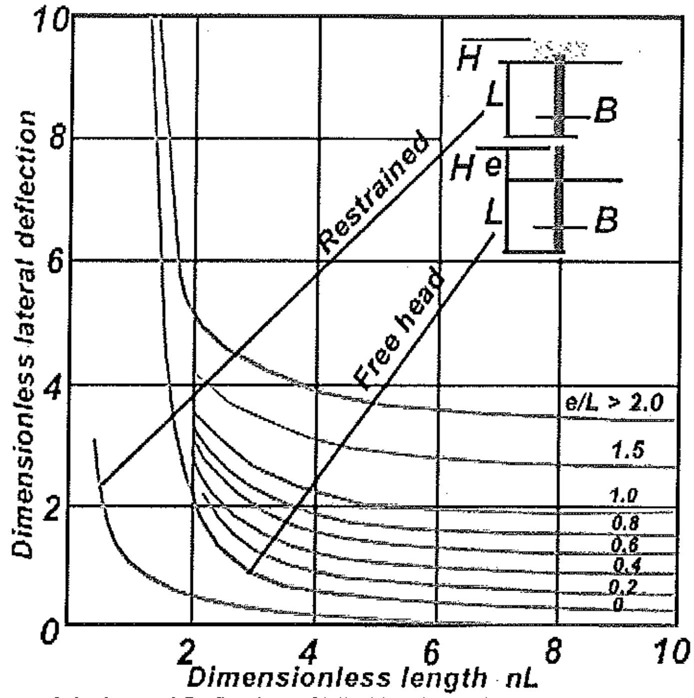


Figure 3.9: Lateral Deflection of Pile Head in cohesionless soil (after Broms)

Part 4. MOMENTS AND DEFLECTIONS (CFEM 1992)

Lateral Pile Deflections
For the subgrade reaction models, it is assumed that the soil around a pile can be simulated by a series of horizontal springs, each spring representing the behaviour of a layer of soil of unit height. When the pile is forced against the soil under the action of the horizontal loads, the soil deforms and generates an elastic reaction assumed to be identical to the force that would be generated by simulating spring subjected to the same deformation. With the further assumption that the soil is homogenous, i.e., all springs are identical, the soil's behaviour can be estimated if the equivalent spring constant is known. This spring constant is called the coefficient of subgrade reactions k_s (dimension: force/volume).

Coefficient of Subgrade Reaction
The coefficient of horizontal subgrade reaction may be estimated by the following method.

a) In *cohesionless soil*

$$k_s = n_h\,(z/d) \qquad \text{Eqn. 4.1}$$

where

k_s = coefficient of horizontal subgrade reaction (force per volume)
z = depth
d = pile diameter
n_h = coefficient related to soil density as given in *Table 4.1.*

Table 4.1. Values of n_h for Cohesionless Soils

SOIL COMPACTNESS CONDITION	n_h (kN/m^3)	
	Above Groundwater	Below Groundwater
Loose	2200	1300
Compact	6600	4400
Dense	18000	11000

b) In *cohesive soil*

$$k_s = 67\,C_u / d \qquad \text{Eqn. 4.2}$$

where

k_s = coefficient of horizontal subgrade reaction (force per volume)
C_u = undrained shear strength of the soil
d = pile diameter

Determination of Moments and Deflections
This section considers only the most common case of piles with a rigid cap at ground surface. (CFEM 1992)

The distribution and magnitude of moments and deflections in a pile subjected to horizontal forces are essentially a function of the relative stiffnesses, T, of the pile-soil system. For sand, *T* is given by the following relation:

$$T = (4\,EI / n_h)^{1/5}$$ Eqn. 4.3

and for overconsolidated clay

$$T = (EI / k_s\, d)^{1/4}$$ Eqn. 4.4

where
- E = elastic modulus of pile material
- I = moment of interia of pile cross section
- n_h = a constant as given in Table 4.1., above
- k_s = coefficient of horizontal subgrade reaction

From the value of T, the moments, M_p, in the pile and the deflections, δ_p, of the pile cap may be computed at any depth using the following formulae:

$$M_p = F_m\,(P\,T)$$ Eqn. 4.5

$$\delta_p = F_\delta(P\,T^3 / EI)$$ Eqn. 4.6

where
- M_p = moment at depth z
- δ_p = deflection at depth z
- F_m = moment coefficient at depth z, as given in *Figure 4.2*.
- F_δ = deflection coefficient at depth z, as given in *Figure 4.1*.
- P = applied horizontal load
- T = relative stiffness

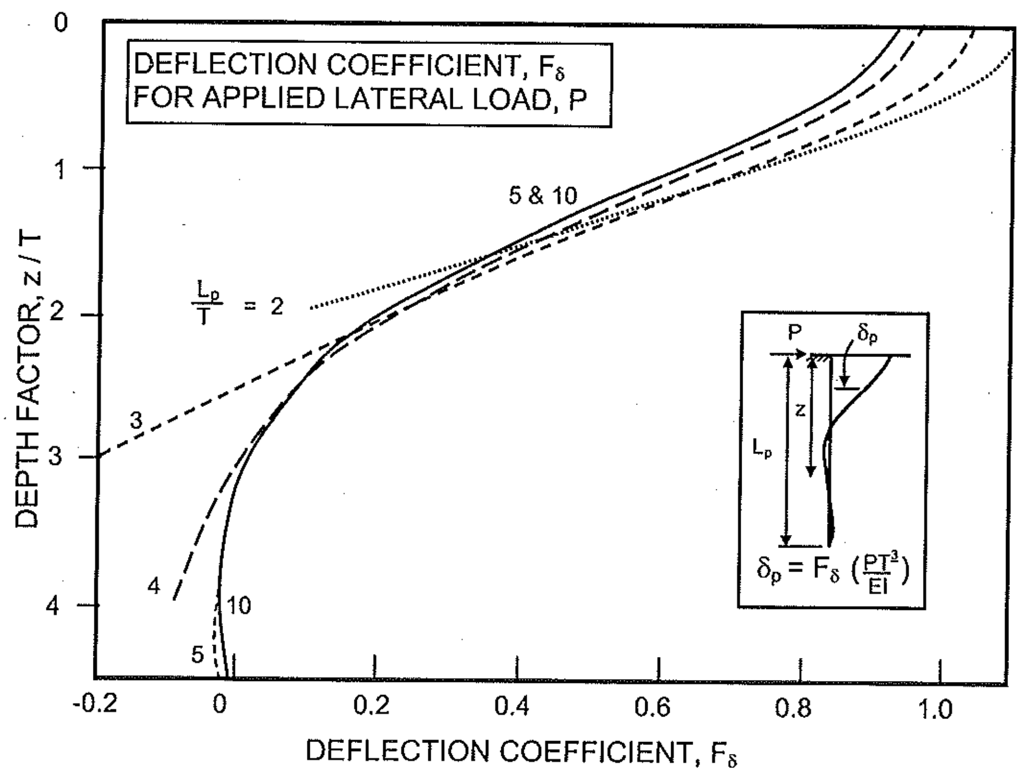


Figure 4.1 Deflection Coefficients for Laterally Loaded Piles. (CFEM 1992)

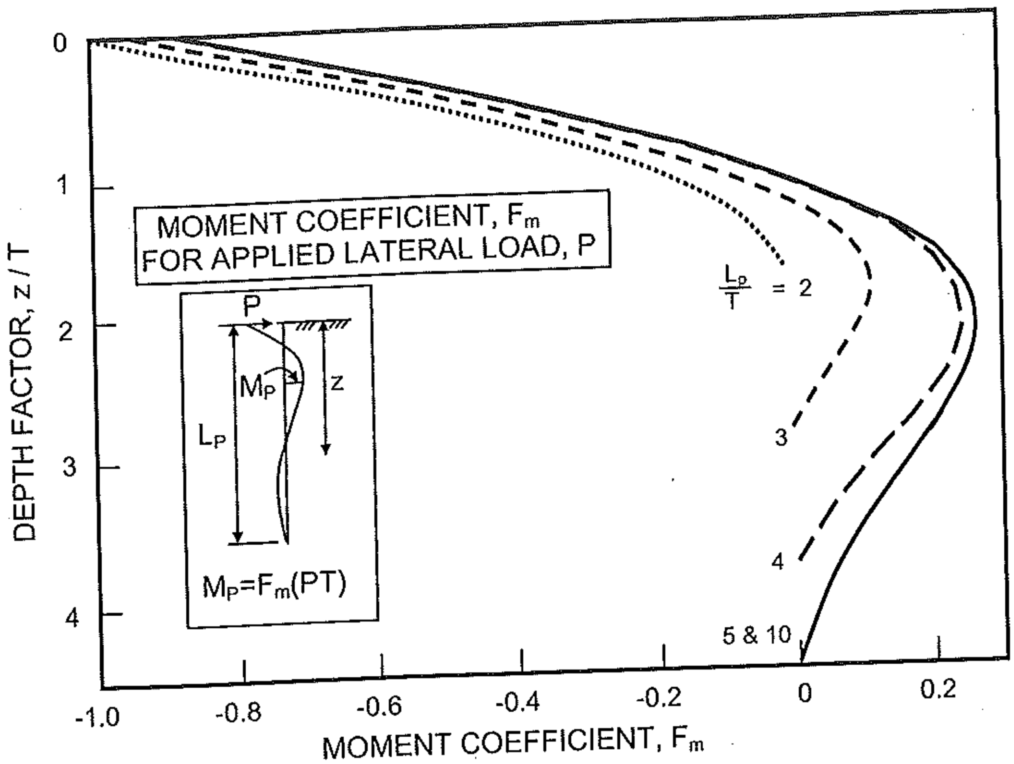


Figure 4.2 Moment Coefficients for Laterally Loaded Piles. (CFEM 1992)

Part 5. BUCKLING OF PILES

Screw piles by design are long and slender, and although extremely rare, are susceptible to buckling when placed under extreme compressive loading conditions. The buckling of piles can be caused by one of two situations. Extreme compressive forces may cause the shaft to fold and buckle. This would occur in the upper portion of the pile where the soil is weak. The more common buckling situation is when a pile is exposed to lateral loading. A pile exposed to lateral loading behaves similar to any supporting member under lateral loading. The lower part of the pile will remain stationary while the upper part will start to bend. A screw pile will behave similar to that of slender deep pile with the helix supplying little lateral or bending moment resistance, unless it is designed to supply resistance (i.e. shallow condition or shallow helix embedment).

Included in this section are varying methods for determining the structural capacity of the pipe shaft portion of the screw pile. There are different ways to determine the ultimate piling shaft capacity subjected to axial loading; and we have selected Poulos and Davis (1980) method to estimate the ultimate vertical capacity (Pr) the pile can take before starts buckling.

Poulos and Davis (1980) suggested the following:

During loading, a partly embedded vertical pile subjected to a vertical load. The stiffness factors R and T as calculated from *equations 3.1 and 3.3* and have been used to obtain the equivalent length of a freestanding pile with a fixed base, from which the factor of safety against failure due to buckling can be calculated using conventional structural design methods.

For a partly embedded pile carrying a vertical load P, the equivalent height L_e, of the fixed-base pile is shown in *Figure 5.1b.*

For soil with a constant modulus:

Depth to a point of fixity $z_f = 1.4\,R$ Eqn. 5.1

For soils having a linearly-increasing modulus:

$z_f = 1.8\,T$ Eqn. 5.2

The relationships of *equations 5.1 and 5.2* are only approximate, but they are valid for structural design purposes provided that l_{max}, which is equal to L/R, is greater than 4 for soils having a constant modulus and provided that z_{max}, which is equal to L/T, is greater than 4 for soils having a linearly-increasing modulus. From *equations 5.1 and 5.2* the equivalent length L_e of the fixed-base pile (or column) is equal to $e + z_f$ and the critical load for buckling is:

$$P_{cr} = \frac{\pi^2 E I}{4R^2 (S_R + Z_R)^2}$$ For free-headed conditions Eqn. 5.3

$$P_{cr} = \frac{\pi^2 E I}{R^2 (S_R + Z_R)^2}$$ For fixed- (and translating-) headed conditions Eqn. 5.4

Where:

$$S_R = L_S / R$$ Eqn. 5.5

$$J_R = L_U / R$$ Eqn. 5.6

L_S = *Equivalent free length of embedded portion of pile (Figure 5.1)*
L_U = *Unsupported pile length*

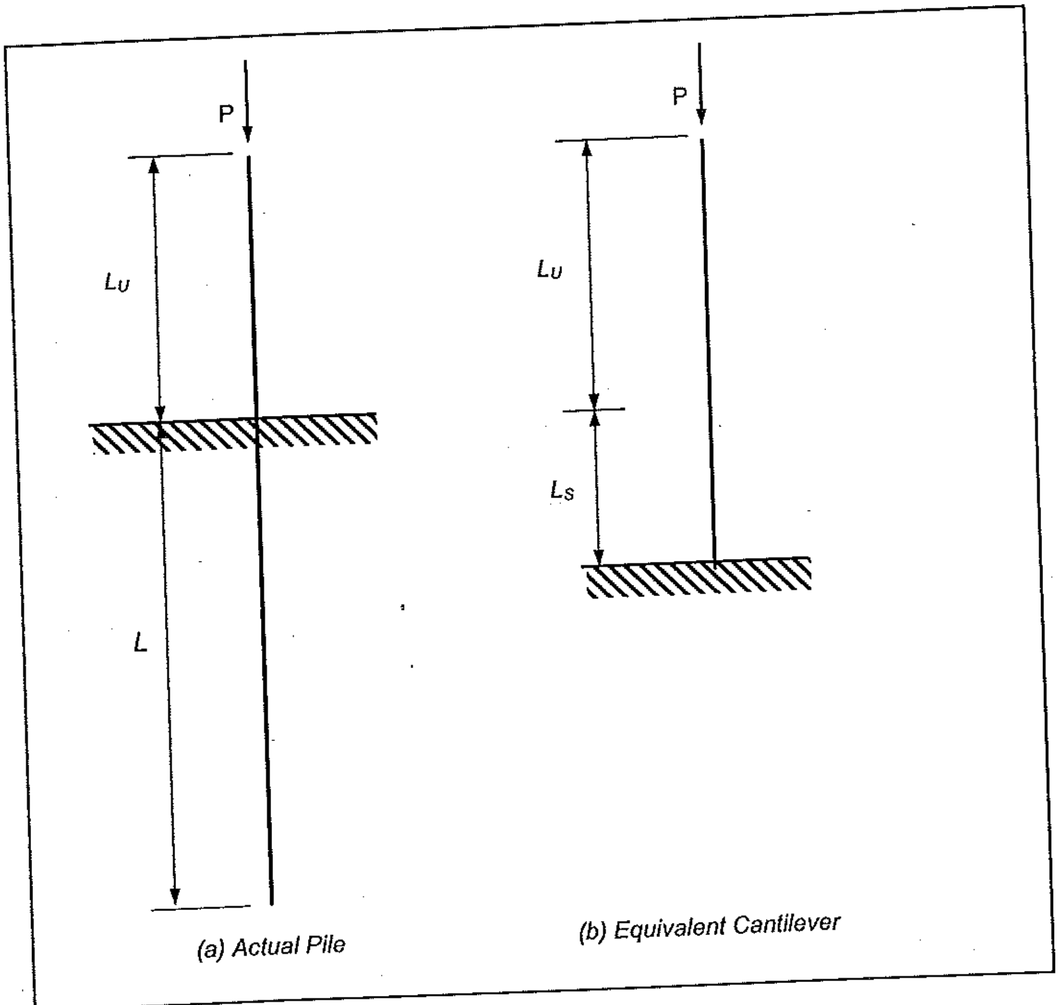


Figure 5.1 Partially Embedded Piles (after Poulos and Davis 1980)

Part 6. *USE OF SCREW PILES AS TIEBACK ANCHORS*

HELCAL PIER SYSTEMS LTD has been manufacturing multi-helix screw anchors since 1991. These anchors have established a consistent record of performance through extensive use in tieback applications for the electric utility and oil and gas industry. Construction application for screw anchors in retaining wall tie backs continue to grow.
Compared to a grouted anchor a screw anchor's advantage is how it removes the performance uncertainties and costs associated with a grouted anchor when used in loose sandy soils or in low shear strength clay soils. When placed in the soil, the screw anchor acts as a bearing device. This is a fundamental difference compared to a grouted anchor formed in the soil and reliant on friction between soil and grout. Collapse of a prepared hole can change a grouted anchor's dimensions. There is little opportunity to assess the problem's magnitude and exact location because it is in the hole, out of sight. Protecting grout from such an occurrence adds the extra costs of installing casing. A screw anchor averts these drawbacks by requiring neither an open hole nor a casing.
Screw anchors can be designed to hold large capacities.

Advantages of using HELICAL PIER SYSTEMS screw anchors as tiebacks include:

- Competitive installing costs
- Immediate proof testing and loading- no waiting time for grout to cure
- Installs in any weather
- Speeds excavation and construction
- Removable and Reusable
- No spoil to remove.

Estimating the lateral loads (Figure 6.1.) acting against retaining walls as exerted by the soil requires knowledge of:

- Soil type and conditions
- Structural dimensions of the retaining structure
- Ground water table.

Every wall tieback situation is unique, but there are some aspects that merit attention. The placement of the anchor is influenced by the height of the soil backfill against the wall. Figure 6.2 shows this condition and a guideline for setting the location of the tieback anchor. Experience indicated that the tieback should be located close to the point of maximum wall bulge and/or close to the most severe transverse crack. In many cases 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 anchor. Figure 6.2 also indicates that the minimum height of the cover is 6 times the diameter of the largest helical plate. Finally, the helical anchor must be installed a sufficient distance away from the wall in order that the helical plate(s) can develop an anchoring capacity by passive pressure. This requires the length of installation to be related to the height of soil backfill.
From all the above information we can figure out the soil active pressure and the water pressure against the wall. Upon preliminary design of anchor rows depth, the load on each row/ Meter width of the wall can be calculated. With HELICAL PIER SYSTEMS previous experience with Screw anchors, we can decide the horizontal spacing between anchors and accordingly the load on each screw anchor can be determined.

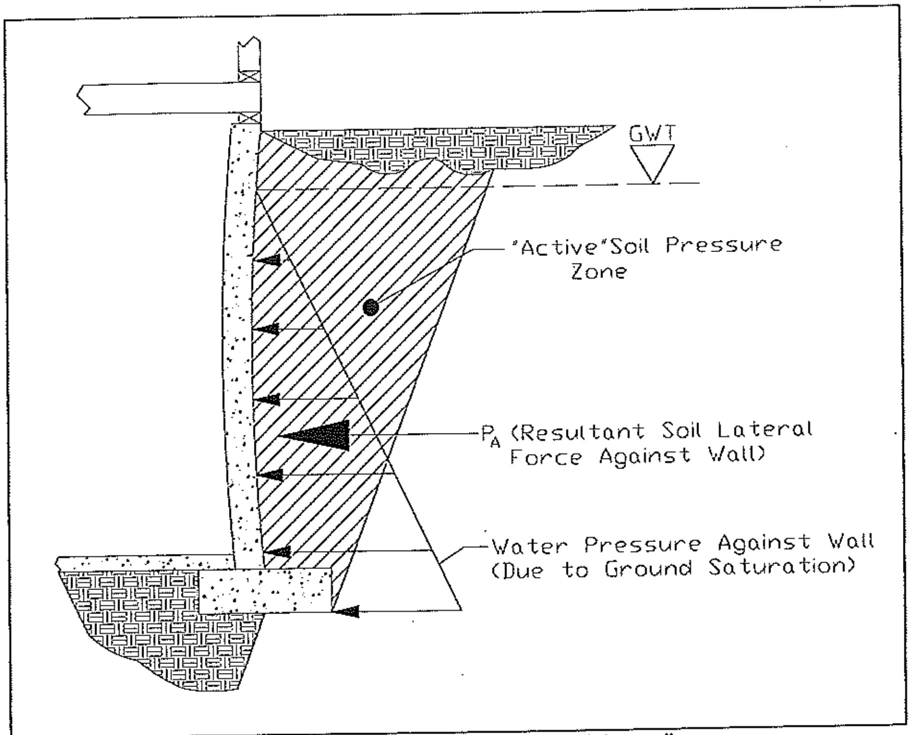


Figure 6.1: Earth and water pressure distribution behind retaining wall

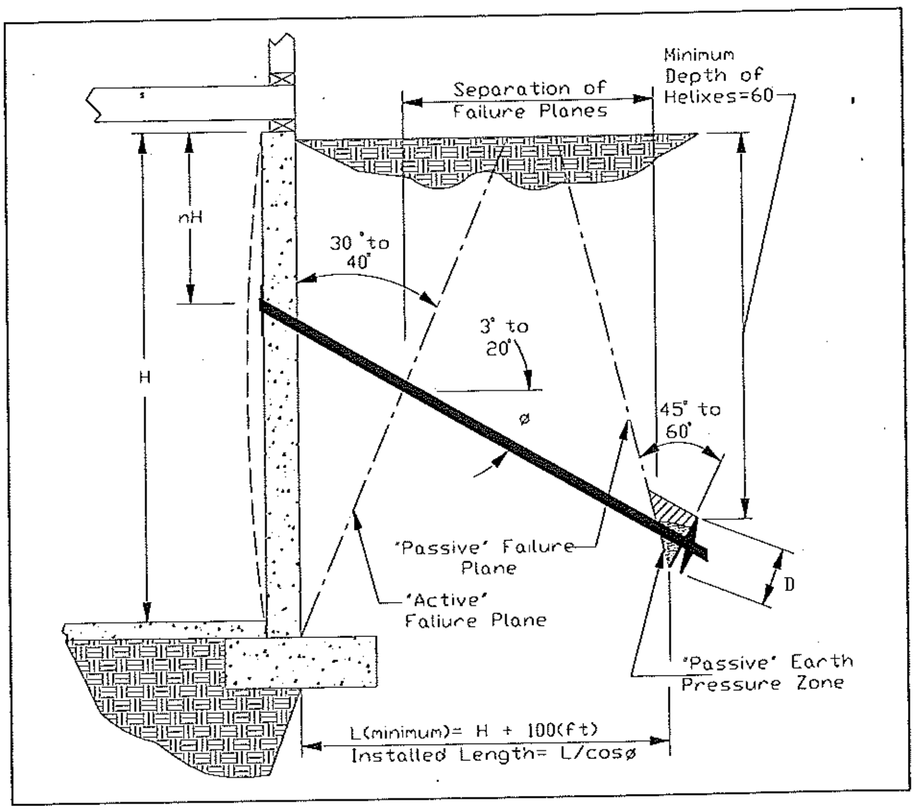


Figure 6.2: Typical installation depth and length for Helical tiebacks

Depending on the spacing between Helices (S) / Helix diameter (D) ratio, the design method of the Screw anchors will be either:

1. Individual Plate Method

   Adam and Klym (1971) stated that at $S/D \geq 2$, each helix plate can be assumed to behave independently of the other. ALMITA extensive tests (1999) showed that this method can be used if $S/D \geq 3$

   The individual bearing method assumes that bearing failure occurs above each individual helix. The total uplift resistance is the sum of the individual capacities.

   $$Q_t = Q_{shaft} + \sum Q\ I\ (\text{bearing}) \qquad \text{Eqn 6.1}$$

   Where:

   Q_t = Ultimate uplift capacity

   Q_{shaft} = Adhesion developed along the steel shaft (Chapter 3)

   $\sum Q\ I$ (bearing) = Sum of the bearing capacity of each individual helix (Chapter 3)

2. Cylindrical Shear Method

   Please refer back to Chapter 3 for the design.

Part 7. SELECTION OF SCREW PILE

Helical Pier Systems screw pile shaft sizes range from 2-7/8" to 36" in diameter with varying wall thicknesses. *Table 7.1* lists the most common and readily available pipe shaft sizes. The small diameter shafts are mainly used for compression and tension loads where lateral loads are minimal. Larger diameter shafts are used when the screw piling is subjected to large compressive loads and/or lateral loads and/or moments of overturn. Large diameter shafts are not always required to go to full depth embedment, the piling shaft can be reduced in size with a neck-down or smaller shaft located on the bottom end portion of the piling (see *Figure 7.1*). There are many determining factors that lead to the selection of a pipe shaft used for a screw piling. The criteria that directly lead to the selection of the appropriate shaft size are: axial load, tension load, lateral load, moment of overturn, torque considerations, installation equipment, helix size, soil conditions and possibly others. (See Parts 3, 4 and 5 for shaft designing).

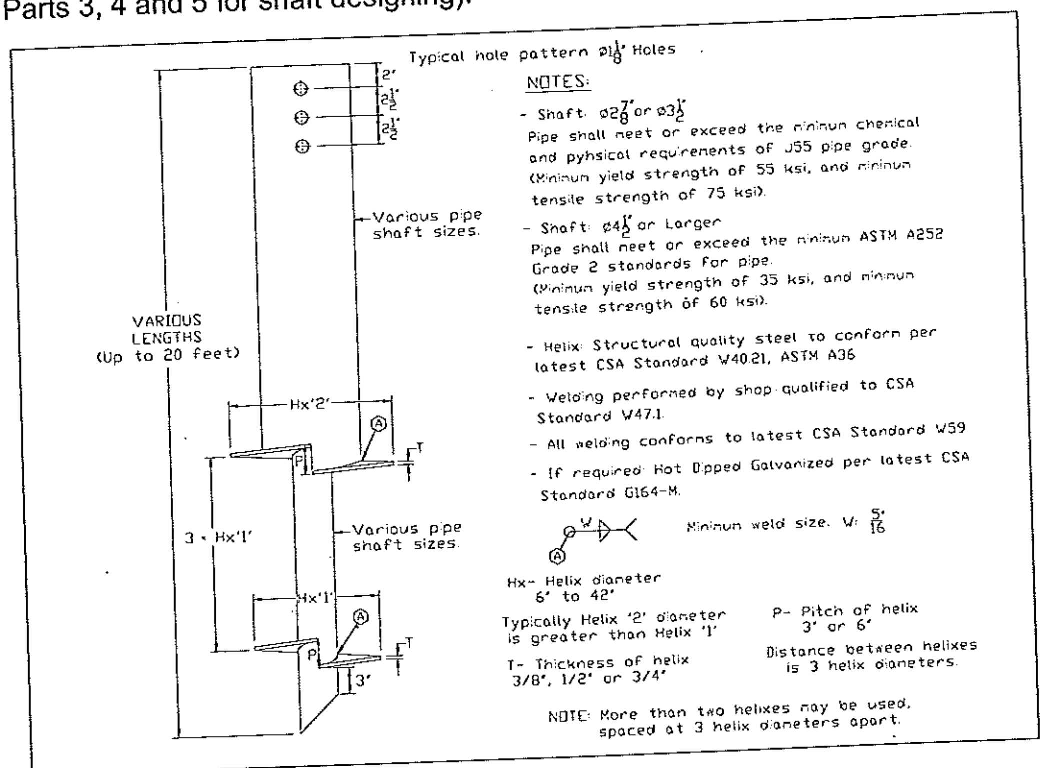


Figure 7.1 Neck-Down Pile

Table 7.1 Common Pipe Sizes

Pipe Shaft Outside Diameter	Common Wall Thickness *(Almita common)*	Maximum Torque (Ft.Lbs.)
2-7/8"	*0.217*	*8,000*
3-1/2"	*0.254*	*11,000*
4-1/2"	*0.250,* 0.237	*21,500* 20,400
5-1/2"	*0.275*	*43,600*
6-5/8"	*0.280,* 0.250	*53,900*
8-5/8"	*0.264,* 0.322	*67,000* 81,200
10-3/4"	*0.365,* 0.250	*+90,000*
12-3/4"	*0.375,* 0.250	*+90,000*

The critical factors that dictate the helix size are axial load, tension load, torque consideration, installation equipment, soil conditions and pipe shaft size (see Table 7.2). Table 7.2 shows the helix configurations that will fit on various pipe shaft sizes. The minimum sizes are the minimum physical sizes that will fit on pipe and the maximum are the maximum practical sizes available. (See Part 2 for Helix designing).

Table 7.2 Helix Diameter vs. Pipe Shaft

PIPE SHAFT O.D. (inches)	HELIX DIAMETER (inches)																					
	6	8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48
2-7/8	X	X	X	X	X	X																
3-1/2		X	X	X	X	X	X	X														
4-1/2			X	X	X	X	X	X	X	X												
5-1/2				X	X	X	X	X	X	X	X	X	X									
6-5/8					X	X	X	X	X	X	X	X	X	X	X	X						
8-5/8					X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X		
10-3/4						X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
12-3/4							X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
14								X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
16										X	X	X	X	X	X	X	X	X	X	X	X	X
20												X	X	X	X	X	X	X	X	X	X	X
24													X	X	X	X	X	X	X	X	X	X
30																X	X	X	X	X	X	X
36																			X	X	X	X

Helical Pier Systems Ltd helix sizes come in a wide variety of shapes and sizes, if necessary, custom designing to your specifications.
Helix diameters currently range from 6 to 48 inches, pitches are set at 3, 4, 6, 12 and 24, thickness of plate range from 1/4, 3/8, 1/2, 3/4, and 1 inches.

STEPS IN PILE SELECTION:

1. Determine applied loads on Pile: Dead Load, Live Load & Safety Factors.
2. Determine site specific soils information: Soil Type, Soil Description, Soil Classification, Water Table Levels and Depth of Frost Penetration.
3. Compare soils information with pile load and location information. Pile spacing – is there a group effect among piles?
4. Design Pile – Pile Geometry (See parts 1 thru 5 of manual)
   Select: Pile Shaft, Helix Diameter and Thickness, Number of Helixes, Embedment Depth, Extension Required? Bolt-On or Welded?
5. Estimate installation Torque.
6. Evaluate Design – Practical? Can designed pile be installed? Do soil conditions allow for installing? Equipment/Power?

   Possibly repeat Step 4.
7. Calculate ultimate pile capacity and apply Safety Factors (Minimum S.F. = 2.0).

The steps are to be used as a guide in the pile design process, other factors may come into play when designing a screw pile (ie. seismic considerations, soil chemistry, etc.)

Part 8. CORROSION RESISTANCE OF GALVANIZED SCREW PILES

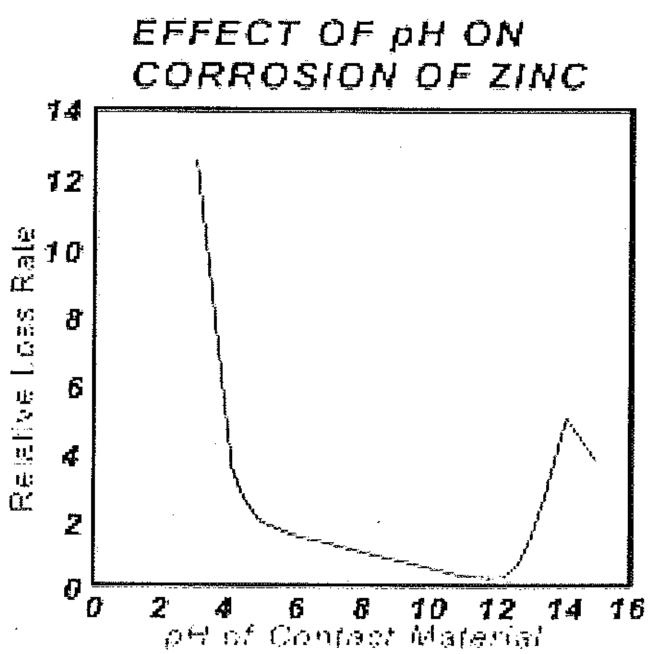
More than 200 different types of soils have been identified and are categorized according to texture, color and natural drainage. Coarse textured soils like gravel and sand permit free circulation of air and the corrosion process may closely resemble atmospheric corrosion. Clay and silt soils have a fine texture and hold water resulting in poor aeration and drainage. The corrosion process in these soils may resemble the corrosion process in water.

The corrosion of galvanized (zinc) coatings in clay and silts is a very complex issue with many factors affecting the expected life. The most important factors include; chloride content of water/soil and the hardness, soil/water pH, and soil resistivity. Here is a brief discussion on each factor:

As indicated by Porter in " Corrosion Resistance of Zinc and Zinc Alloys" p275; "Of all the anions, chloride is most corrosive to zinc in water (and soil), especially if it is present in amounts exceeding 50 mg/L. The softer the water, and the lower it is in carbonate the more pronounced is the effect of chloride. Thus, a chloride content of 80mg/L in soft water causes quite sever corrosion, while in hard water no corrosion occurs even with 700 mg/L." The reason for this is hard water forms a scale of insoluble salts on the galvanized coating. This scale combines with the zinc to from a protective barrier of calcium carbonate and zinc carbonate. This protective layer significantly increases the corrosion free life of the galvanized pile.

The soil resistivity is determined by the nature and concentration of the ions dissolved in the soil moisture and ground water. Therefore, in most cases, the impact that resistivity will have on the corrosion of the galvanized coating is limited to the chloride content, and hardness.

Figure 8.1



The pH range of the soil/water is another important factor. Galvanized coatings proved excellent corrosion resistance when the pH is above 4.0 and below 12.5. See Figure 8.1.

The National Bureau of Standards has conducted an extensive research program on the corrosion of metals in soils. Some of their research on galvanized steel pipe dates back to 1924. The expected life is based on a zinc coating thickness of 200μm.

The results of these tests also showed that the galvanized coating will prevent pitting of steel in soil, just as it does under atmospheric exposure, and that even in instances where the zinc coating was completely consumed, the corrosion of the underlying steel was much less than that of bare steel specimens exposed under identical conditions.

The expected life for a galvanized screw pile is calculated using a conservative coating thickness of 200 μm. The actual measured coating thickness of Helical Pier Systems' screw piles is usually in the 300 – 400 μm range. If this value is used then the life expectancy would be double.

The galvanized coating will provide 50 -100 years of corrosion free service. The study also showed that even after all of the galvanized coating is consumed the residual zinc in the soil would reduce the corrosion on the remaining steel pile.

Prepared by Darcy Pretula, P.Eng., Daam Galvanizing Inc. 21-November-2001

Part A STANDARDS, SPECIFICATIONS AND INFORMATION

- National Building Code of Canada
- Alberta Building Code
- ASTM A252 Welded and Seamless Steel Pipe Piles
- CSA G40.21-M Structural Quality Steel
- CSA W47.1 Certification of Companies for Fusion Welding of Steel Structures
- CSA W59 Welded Steel Construction Steel construction (Metal Arc Welding)
- CSGB 1-GP-184 Coal Tar Epoxy (black) Coating
- SSPC-SP6 Commercial Blast Cleaning
- ASTM A 153 Specification for Zinc Coatings (Hot-Dip) on Iron and Steel Hardware
- CSA G164 Hot Dip Galvanizing of Irregularly Shaped Articles

Helical Plate:
ASTM A36 or CSA G40.21 44W Hot Rolled Structural Steel Plate.

Screw Pile Pipe:
3-1/2" Diameters piers and under (includes 2-7/8" piers):
Pipe meets or exceeds the minimum requirements of API 5CT Grade J55 (min. yield strength of 55 ksi and min. tensile strength of 75 ksi).
4-1/2" Diameter piles and larger:
Minimum ASTM A252, Grade 2 or 3 Steel pipe piles, seamless or straight welded, Pipe wall thickness vary from Schedule 20 – Schedule 40 – Schedule 80, (min. yield strength of 35 ksi and min. tensile strength of 60 ksi).

Welding:
- Almita Manufacturing Ltd. is certified by the Canadian Welding Bureau (CWB) in Division 2.1. The welding design and welding fabrication of structural steel will be in accordance with the CSA Standard W47.1.
- All welding performed in accordance with the requirements of CSA Standard W59.1, Latest Edition.

Fasteners:
All bolts will be supplied as per customers' requirements.
Minimum requirements are ASTM A 325 bolts.
Bolts are bare metal (black), plated or hot-dipped galvanized.

Testing Standards:
When conducting Pile Load Tests they are preformed in accordance with ASTM D1143, Standard Method of Testing Piles Under Axial Compressive Load, ASTM D3689, Standard Method of Testing Individual Piles Under Static Axial Tensile Load, and ASTM D3966, Standard Method of Testing Piles Under Lateral Loads.

Specification	**A252 Piling Pipe**
Scope	Covers nominal (average) wall steel pipe piles of cylindrical shape and applies to pipe piles in which the steel cylinder acts as a permanent load-carrying member or as a shell to form cast-in-place concrete piles.
Kinds of Steel Permitted For Pipe Matedal	Open-hearth Basic-oxygen Electric-furnace
Permissible Variations in Wall Thickness	Not more than 12.5% under the nominal wall thickness specified.
Chemical Requirements	**Seamless and Welded Pipe:** Open-hearth, Electric-furnace or Basic-oxygen — **Phosphorus Max. %** 0.050
Hydrostatic Testing	None specified.
Permissible Variations in Weights per Foot	The weight of any length of pile shall not vary more than 15% over or 5% under the weight specified. Each length shall be weighed separately.
Permissible Variations in Outside Diameter	Shall not vary more than plus or minus 1% from the diameter specified.
Mechanical Tests Specified	**Tensile Test** -- Either longitudinal or transverse at option of manufacturer. Minimum yield determined by the drop of the beam, by the halt in the gage of the testing machine, or by the use of dividers.
Number of Tests Required	One tensile property test per 200 lengths.
Lengths	May be ordered in single or double random lengths or in uniform lengths: **Single Random** -16'-25' md. **Double Random** - Over 25' (mm. avg. of 35'). **Uniform** - Plus or minus 1 on length specified.
Required Markings on Each Length (On Tags attached to each Bundle in case of Bundled Pipe)	**Rolled, Die Stamped or Paint Stenciled (Mfgrs. option)** Manufacturer's name, brand or trademark, heat number, method of pipe manufacture, size, weight, length, wall thickness and ASTM A252 and the Grade.
General Information	Surface imperfections exceeding 25% of the nominal wall in depth are considered defects. Defects not exceeding 33.5% of the nominal wall in depth may be repaired by welding. Before welding, the defect shall be completely removed.

NOTE: This is summarized information from ASTM Standards and API Specification 5L. Please refer to the specific Standard or Specification for more details.

MECHANICAL PROPERTIES

MECHANICAL PROPERTIES		Grade 1	Grade 2	Grade 3
Tensile Strength, min	Psi	50,000	60,000	66,000
	Mpa	345	414	455
	kg/mm^2	35.2	42.2	46.4
Yield Strength, min	Psi	30,000	35,000	45,000
	Mpa	205	240	310
	kg/mm^2	21.1	24.6	31.6
Elongation, min	%	30	25	24
Gauge Length	in	2 / (48t + 15)	2 / (40t + 12.50)	2 / (32t + 1.00)

Grade J55 pipe

(1) Pier Shaft: API 5CT Grade J55 pipe (API- American Petroleum Institute) Seamless Tubing

Chemical Specifications (%):

Grade	C	Mn	Ni max	Cu max	P max	S max
J55	0.35 – 0.45	0.90 – 1.30	0.25	0.20	0.020	0.015
	Si	Cr	Mo	Al	V	Sn max
	0.15 – 0.35	max 0.25	-	min 0.015	-	0.03

Mechanical Specifications:

Yield Strength		Tensile Strength
min Psi (MPa)	max Psi (MPa)	Psi (MPa)
55000 (379)	80000 (552)	75000 (517)

Tolerances:

	Outside Diameter	Wall Thickness
API 5CT	+/- 1%	+20 / -0%

(2) Wall Thickness: 2-7/8" diam. (.217"w.t.) or 3-1/2" diam. (.254"w.t.)

(3) Torques: The maximum torque for the 2 7/8" pipe is 8,000 FT.LBS.
The maximum torque for the 31/2" pipe is 16,000 FT/LBS.

Grade A36 Carbon structural steel

(1) Pier Helicals: ASTM Grade A36 / A36M

Product	Plates		Shapes
Thickness, in.(mm)	To 3/4 (20), incl.	Over 3/4 to 1-1/2 (20 to 40), incl.	All
Carbon, max, %	0.25	0.25	0.26
Manganese, %	. . .	080 – 1.20	. . .
Phosphorus, max, %	0.04	0.04	0.04
Sulfur, max, %	0.05	0.05	0.05
Silicon, %	0.40 max	0.40 max	0.40 max
Copper, min, % when copper steel is specified	0.20	0.20	0.20

(2) Mechanical Specifications: ASTM Grade A36 / A36M

Plates, Shapes, and Bars:	
Tensile strength, ksi (MPa)	58 – 80 (400 – 550)
Yield point, min, ksi (MPa)	36 (250)
Plates and Bars:	
Elongation in 8 in. (200 mm), min, %	20
Elongation in 2 in. (50 mm), min, %	23

CSA W47.1 deals with the Certification of Companies for fusion welding of steel structures. Certification requires that the company has the organization, personnel, welding procedures, welding standards, and equipment as required of the **Division** to which it is **Certified**, to produce satisfactory welds and weldments.

To be certified in Division 2.1, the company shall retain part-time a registered professional engineer(s) responsible to the company for:

(a) welding design; and

(b) welding procedures and practice.

CANADIAN WELDING BUREAU

A Division of the CSB Group

[illegible]

<u>LETTER OF VALIDATION</u>

The Canadian Welding Bureau acknowledges that

Almita Manufacturing Ltd.

6696 42 Ave, Ponoka, AB

is certified to CSA Standard W47.1

"Certification of Companies for Fusion Welding of Steel"

in **DIVISION 2**

for the period **September 6, 2006** to **September 6, 2007**

[illegible]

COMPANY CODE: ALMITI

Scope: Structural steel fabrication –

fabrication of structural and miscellaneous steel for buildings, oil and gas

petro-chemical and mining equipment.

Addition extension, capping and [illegible] of screw piles for above

structures.

Registrar

CERTIFICATION SUBJECT TO ANNUAL RENEWAL

"CERTIFICATION MAKES THE DIFFERENCE"

VANCOUVER ▪ EDMONTON ▪ WINNIPEG ▪ TORONTO ▪ MONTREAL ▪ HALIFAX

***Figure A-1.** CWB Certification*

ISO 9000 is a set or family of generic standards adopted in over 90 countries worldwide. The goal was to develop for international trade purposes development of manufacturing, trade and communications standards. These standards provide quality assurance requirements and quality management guidance. Fundamentally, as it relates to quality, ISO 9000 requires a company to document what it does and do what it documents. This specifically relates to all activities pertinent to the quality of a product.

ISO 9000 is not a product or technical standard, it is a quality system standard. It lets your customer know you are in control of your manufacturing or service process. The standard enables your customer to be assured that you have identified and defined the critical elements to be taken into account in producing a quality product.

ANSI-RAB QMS ACCREDITED

D&T

Deloitte & Touche

Certificate of Registration

This certificate verifies that Deloitte & Touche Quality Registrar Inc. has assessed and registered the Management System of

ALMITA Manufacturing Ltd.

6606 – 42nd Avenue, Ponoka, Alberta T4J 1J8

To the Quality System Standard

ISO 9001:2000

The scope of the Quality System is applicable to

Design, manufacture, install and distribute screw piles, screw anchors, and related products that cater to the oil and gas, construction and other industries

Further clarifications regarding the scope of this certificate, and the applicability of ISO 9001:2000 requirements, may be obtained by consulting the organization

This certificate of registration is subject to the terms and conditions as described in the agreement between Deloitte & Touche Quality Registrar Inc. and the holder identified above.

Ben Panariti
Manager of Registration Operations
Deloitte & Touche Quality Registrar Inc.

Certificate Number: #00318
Date Issued: February 1, 2004
Expiration Date: February 1, 2007
Issued: Windsor, ON Canada N8X 1L9

Remi Tosti
President
Deloitte & Touche Quality Registrar Inc.

A-2: ISO 9001 Certificate of Registration

Part B REFERENCE

ASTM D 1143-81 (1981). "Standard Test Method for Piles Under Static Axial Compressive Load"; (Reapproved 1984). Annual Book of ASTM Standards, 1997, Vol. 04.08, pp. 95-105.

ASTM D 3689-90 (1990). "Standard Test Method for Individual Piles Under Static Axial Tensile Load"; (Reapproved 1995). Annual Book of ASTM Standard, 1997, Vol. 04.08, pp.366-375.

ASTM D 3966-90 (1990). "Standard Test Method for Individual Piles Under Lateral Loads"; (Reapproved 1995). Annual Book of ASTM Standard, 1997, Vol. 04.08, pp.389-399.

Canadian Institute of Steel Construction (1992). Handbook of Steel Construction, 5th Edition. Universal Offset Limited, Markham, ON. pp.1-40 and 4-68 to 4-74.

CFEM (1992). Canadian Foundation Engineering Manual. 3rd Edition. Canadian Geotechnical Society, Technical Committee on Foundations, BiTech Publishers Ltd., Richmond, B.C.

Broms, B.B. (1964a). "Lateral Resistance of Piles in Cohesive Soils"; Journal for Soil Mech. and Found. Engrg., ASCE, Vol. 90, SM2, pp. 27-64.

Broms, B.B. (1964b). "Lateral Resistance of Piles in Cohesionless Soils"; Journal for Soil Mech. and Found. Engrg., ASCE, Vol. 90, SM3, pp. 123-156.

Das, B.M. (1990) Earth Anchors. Elsevier, Amsterdam, 241p.

Meyerhof, G.G. (1976). "Bearing Capacity and Settlement of Pile Foundations"; Journal of th Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 102, No. GT3, March 1976, pp. 197-224.

Meyerhof, G.G., and Adams, J.I. (1968). "Ultimate Uplift Capacity of Foundations"; Canadian Geotechnical Journal, Vol. V, no.4, pp.225-244.

Mitsch, M.P., and Clemence, S.P. (1985). "The Uplift Capacity of Helix Anchors in Sand. Uplift Behavior of Anchor Foundations in Soil"; Proceedings of ASCE, New York, N.Y. pp. 26-47.

Poulos, H.G. and Davis, E.H. (1980) Pile Foundation Analysis and Design. University of Sydney, John Wiley and Sons, New York, N.Y. pp 324-327

Tomlinson, M.J. (1987) Pile design and Construction Practice, 3rd Edition. E & FN Spon, London, pp 205-215

Trofimenkov, J.G., and Mariupolskii, L.G. (1965). "Screw Piles Used for Mast and Tower Foundations"; Proceedings of Sixth International Conference on Soil Mechanics and Foundation Engineering, Montreal, Quebec, Vol. 11, pp.328-332.

U.S. Army Corps of Engineers, (1985) "Pile Construction", Field Manual No. 5-134, www.adtdl.army.mil/atdls.htm, Headquarters, Department of the Army, Washington, DC,

Zhang,Diane J.Y. (1999). "Predicting Capacity of Helical Screw Piles in Alberta Soils"; MSc Thesis, Geotechnical Engineering, Department of Civil and Environmental Engineering, University of Alberta.

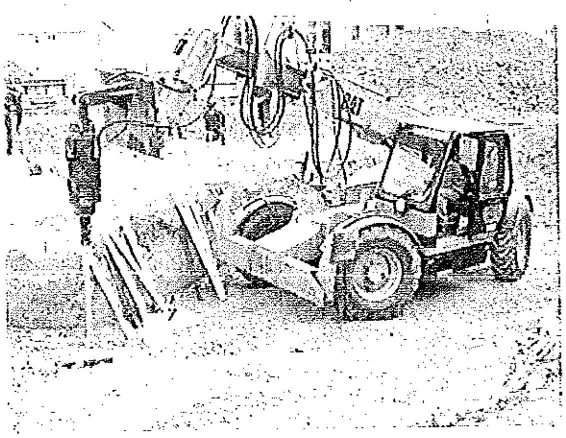
Helical Pier Systems Ltd. recommends field testing to verify the theoretically predicted anchor capacity and to determine allowable design loads and minimum acceptable Safety Factors for the specific project.

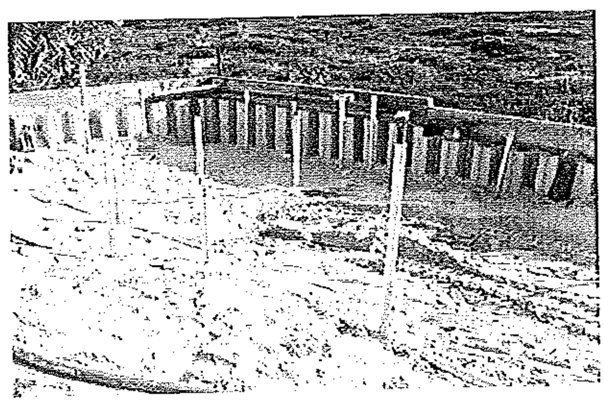
HELICAL PIER SYSTEMS LTD

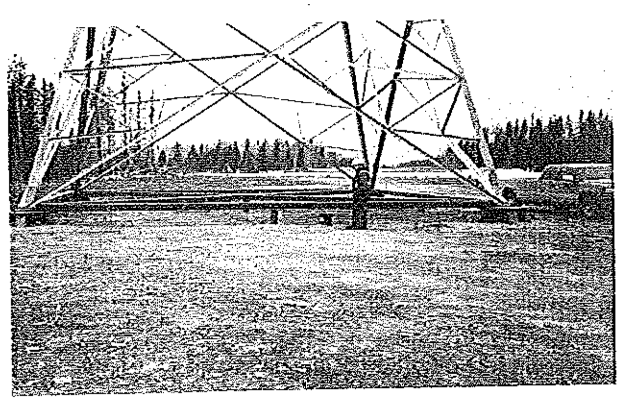
WHEN QUALITY COUNTS YOU CAN DEPEND ON H.P.S. CONTRACTORS

Residential / Commercial / Industrial
Round shaft helical piers

www.helicalpiersystems.com







Printed in Canada