BATTERED CHANCE® HELICAL ANCHORS/PILES for LATERAL LOADING

Lateral loads are commonly resolved with battered helical anchors and piles. 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 anchors and piles to resist only vertical loads and battered helical anchors and piles to resist only lateral loads.

There are some engineers who feel that battered piles in seismic areas "attract seismic forces" during an earthquake, and may therefore rupture. This restriction requires seismic loads to be resisted by means other than battered piles. Other designers allow battered tension devices to elongate elastically and act as a damper, but do not consider the tension anchor capable of resisting compression. CHANCE® Civil Construction helical anchors and piles have been supplied to the seismic prone areas of the west coast of the United States and Canada for over 20 years for civil construction projects. In tension applications, they have been in service for over 40 years. They have been subjected to many earthquakes and aftershocks with good experience. Our helical pre-engineered products have been used far more extensively than any other manufacturer's helical product in these areas. To date, there have been no ill effects observed using battered helical anchors and piles 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 affects. Anecdotal evidence indicates 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*.

Additional Comments

The lateral capacity of round shaft (Type RS) helical anchors and piles is greater than the square shaft (Type SS) helical anchors and piles because of the larger section size. Typical pipe diameters of 2-7/8" (73mm), 3½" (89 mm) and 4-1/2" (114 mm) OD are used for CHANCE® Civil Construction 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 CHANCE® Civil Construction is 10¾" diameter.

There are several other methods used to analyze the lateral capacity of the shaft of the pile, including the early researchers Davis (1961) and Brinch Hansen (1961), with the most commonly used being Broms (1964). The Davis research applied the principles of plane strain to the problem. Other simplifying assumptions made to the Brinch Hansen method are: the shape of the pile has no influence of the pressure magnitude or distribution, the full lateral resistance is mobilized at the movement considered and the distribution of the passive earth pressure is three times the Rankine passive earth pressure.

BUCKLING/SLENDERNESS CONSIDERATIONS

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 summarized description of the procedures available to study the question of buckling of helical piles, and recommendations that aid the systematic performance of buckling analysis.

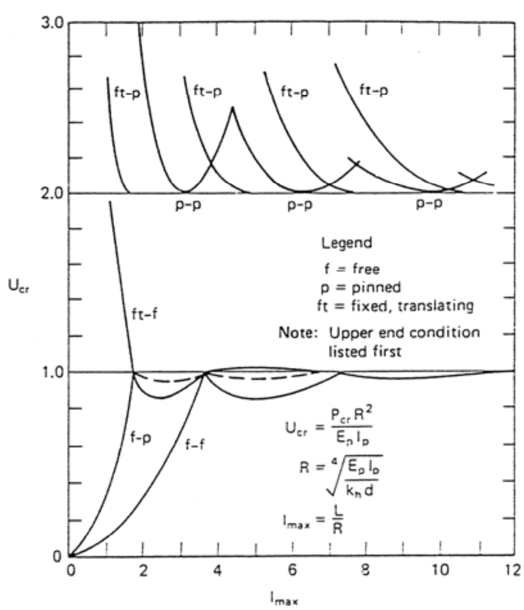
Background

Buckling of columns most often refers 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.

$$P_{crit} = \pi^2 EI/(KL_u)^2 \quad \text{(Equation 5-21)}$$

where:
- E = Modulus of elasticity
- I = Moment of inertia
- K = End condition parameter
- L_u = Unsupported length

It is obvious that helical piles have slender shafts which can lead to very high slenderness ratios (Kl/r), depending on the length of the foundation 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 above.



Poulos and Davis (1980)
Figure 5-10

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 5-21*. As a practical guideline, soil with SPT blow counts per ASTM D-1586 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 foundation. Only the very weak soils are of practical concern. For soils with 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^{PLUS} computer program (ENSOFT, Austin, TX). In addition, the engineers at CHANCE® Civil Construction have developed a macro-based computer solution using the finite-element technique with the ANSYS® analysis software. 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® Civil Construction Distributor in your area. Contact information for CHANCE® Civil Construction Distributors can be found at www.abchance.com. These professionals will help you to collect the data required to perform a buckling analysis. The distributor will either send this data to CHANCE® Civil Construction for a buckling analysis or provide this service themselves.

Buckling Analysis by Davisson 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 Method as described below. Solutions for various boundary conditions are presented by Davisson in *Figure 5-10*. The axial load is assumed to be constant in the pile – that is no load transfer due to skin friction occurs and the pile initially is perfectly straight. The solutions shown in *Figure 5-10* are in dimensionless form, as a plot of U_{cr} versus I_{max}.

$$U_{cr} = P_{cr}R^2/E_pI_p \quad or \quad P_{cr} = U_{cr}E_pI_p/R^2 \qquad \text{(Equation 5-22)}$$

$$R = \sqrt[4]{E_pI_p/k_hd} \qquad \text{(Equation 5-23)}$$

$$I_{max} = L/R \qquad \text{(Equation 5-24)}$$

where:
- P_{cr} = Critical buckling load
- E_p = Modulus of elasticity of foundation shaft
- I_p = Moment of inertia of foundation shaft
- k_h = Modulus of subgrade reaction
- d = Foundation shaft diameter
- L = Foundation shaft length over which k_h is taken as constant
- U_{cr} = Dimensionless ratio

By assuming a constant modulus of subgrade reaction (k_h) for a given soil profile to determine R, and using *Figure 5-10* to determine U_{cr}, *Equation 5-21* can be solved for the critical buckling load. Typical values for k_h are shown in *Table 5-2*.

Modulus of Subgrade Reaction - Typical Values
Table 5-2

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

Figure 5-10 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 method to determine the critical buckling load.

Buckling Analysis by Finite Differences

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 the gravity of the end cross-sections of the shaft, leading to the differential equation:

$$EI(d^4y/dx^4) + Q(d^2y/dx^2) + E_s y = 0 \qquad \text{(Equation 5-25)}$$

where:

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
EI	=	Flexural rigidity of the foundation 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

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 5-25* 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 foundation shaft properties that vary with depth, and can be used advantageously for design of 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. This instability may be seen as a convergence to a physically illogical configuration or 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.

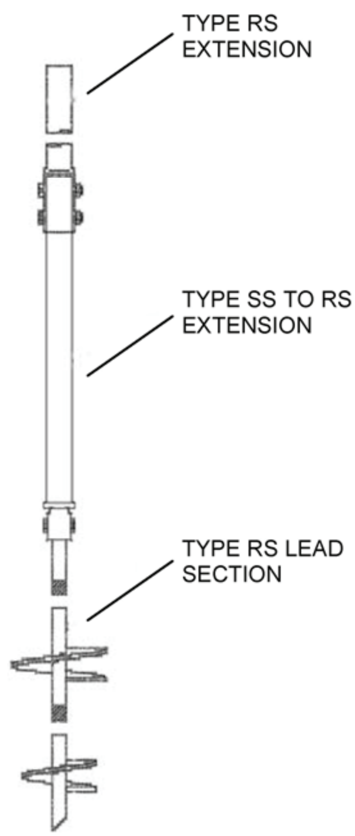
Buckling Analysis by Finite Elements

CHANCE® Civil Construction has developed a design tool, integrated with ANSYS® finite element software, to determine the load response and buckling of helical piles. The method uses a limited non-linear model of the soil to simulate soil resistance response without increasing 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 3D beam elements assumed to have elastic response. Couplings are modeled from actual test data, which includes an initial zero stiffness, an 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 ANSYS® 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 output 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.

Practical Considerations – Buckling



Type SS to RS Combination Pile
Figure 5-11

As stated previously, where soft and/or loose soils (SPT 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. Concern 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 available with pipe shafts (Type RS). Pipe shaft helical piles have better buckling resistance than plain square shaft (Type SS) because they have greater section modulus (flexural resistance), plus they have larger lateral dimensions, which means they have greater resistance to lateral deflection in soil. See *Tables 7-4 and 7-7* in *Section 7* for the section properties and dimensions of both Type SS and RS helical anchors/piles.

Type SS helical anchors/piles provide the most efficient capacity-to-torque relationship (see *Section 6, Installation Methodology*). Type RS helical anchors/piles 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 Type SS square shaft material for the lead section and Type RS pipe shaft material for the extension sections (*see Figure 5-11*). The combo pile provides the advantages of both Type SS and RS material, which enables the helical anchor/pile to penetrate dense/hard soils, while at the same time provide a larger shaft section in the soft/loose soils above the bearing strata. See *Section 7* for more information on combo piles.

The Helical Pulldown® Micropile is a method for constructing a grout column around the shaft of a Type SS 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 or adhesion along the shaft; plus the load/deflection response of the helical pile is stiffer. See *Section 7* for more information on Helical Pulldown® Micropiles.

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 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. More research is required, but a practical limit for grouted shafts is overburden soil with SPT blow counts equal to or less than 20 blows/ft. Increasingly dense soil makes installation more difficult for the displacement element, which has to force soil laterally outward away from the central steel shaft.

CHANCE® HELICAL ANCHOR/PILE FRICTIONAL CAPACITY

The general equation is:

$$Qf = \Sigma[\pi D f_S \Delta L_f] \quad \text{(Equation 5-26)}$$

where: D = Diameter of timber, steel or concrete pile column
f_S = Sum of friction and adhesion between soil and pile
ΔL_f = incremental pile length over which πD and f_S are taken as constant

There are several empirical methods to calculate friction, including the following:

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

  Type I: Sands and gravels with $35^o < \phi < 45^o$ and c = 0:

  $$f_s = \sigma_o \tan \phi \quad \text{(Equation 5-27)}$$

  where: σ_o = Mean normal stress for the grout column

  Type II: Mixed soils; fine loose silty sands with $20^o < \phi < 30^o$ and sandy clays with 205 psf < C < 1024 psf (9.8 kPa < C < 49 kPa)

  $$f_s = \sigma_o \sin \sigma_o \tan \phi + C \cos \phi \quad \text{(Equation 5-28)}$$

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

  $$f_s = C \quad \text{(Equation 5-29)}$$

  where: 1025 psf < c > 2048 pfs (49 kPa < c < 98 kPa)

  and:

  $$f_s = 2048 \text{ psf (98 kPa)} \quad \text{(Equation 5-30)}$$

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

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

- Department of the Navy Design Manual 7 Method:

  For cohesive soils (α Method):

  $$Q_f = \Sigma[\pi D C_a \Delta L_f] \quad \text{(Equation 5-31)}$$

  where: C_a = Adhesion factor

For cohesionless soils (α Method):

$$Qf = \Sigma[\pi D(qK\tan\phi)\Delta Lf] \quad \text{(Equation 5-32)}$$

where:

q = Effective vertical stress on element ΔLf

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

ϕ = Effective friction angle between soil and plate material

Alternate Method

$$Qf = \Sigma[\pi D(S)\Delta Lf] \quad \text{(Equation 5-33)}$$

where:

S = Average friction resistance on pile surface area = $P_o \tan\phi$

P_o = Average overburden pressure

Recommended Adhesion Values *
Table 5-3

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

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

Straight Concrete Piles
Table 5-4

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

Straight Steel or Timber Piles
Table 5-5

P_o (psf)	Angle of Internal Friction (degrees)				
	20	25	30	35	40
	S = Average Friction Resistance on Pile Surface (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
Note: Values shown are 75% of the values given for straight concrete piles in Table 5-5 due to lower coefficients of friction.					

NOTE

Tables 5-3, 5-4 and 5-5 are derived from graphs in the Department of the Navy Design Manual 7, *Soil Mechanics, Foundations and Earth Structures* (1974). Later editions of this manual limit the depth at which the average overburden pressure is assumed to increase. The following is an excerpt from the manual regarding this limiting depth:

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

Design Example 8-5 in *Section 8* illustrates the use of the Navy Design Manual 7 method to calculate the friction capacity of a Helical Pulldown® Micropile.

References:

1. Specification ASTM D 1586, *Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils,* American Society for Testing and Materials.
2. Bjerrum, L., *Norwegian Experiences with Steel Piles to Rock,* Geotechnique, Vol 7, 1957.
3. Bowles, J.E., *Foundation Analysis and Design,* First Edition, McGraw-Hill, 1968.
4. Bowles, J.E., *Foundation Analysis and Design,* Fourth Edition, McGraw-Hill, 1988.
5. Brinch Hansen, J., *The Ultimate Resistance of Rigid Piles Against Transversal Forces*, Geoteknish Institute Bulletin No. 12, Copenhagen, 1961.
6. 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.
7. 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.
8. Cadden, Allen and Jesus Gomez, *Buckling of Micropiles,* ADSC-IAF Micropile Committee, Dallas, TX, 2002.
9. Clemence, Samuel P. and others, *Uplift Behavior of Anchor Foundations in Soil,* American Society of Civil Engineers, 1985.
10. Das, Braja M., *Theoretical Foundation Engineering*, Elsevier Science Publishing Company Inc., New York, NY, 1987.
11. Davis, E.H., *The Application of the Theory of Plasticity to Foundation Problems-Limit Analysis*, Post Graduate Course, University of Sydney, Australia, 1961.
12. 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.
13. Davisson, M.T., *Laterally Loaded Capacity of Piles*, Highway Research Record, No. 333: 104-112, 1970.
14. Design Manual DM7, NAVFAC, *Foundations and Earth Structures*, Government Printing Office, 1986.
15. Design Manual DM7, NAVFAC, *Soil Mechanics,* Government Printing Office, 1986.
16. Gouvenot, D., *Essais en France et a l'Etranger sur le Frottement Lateral en Fondation: Amelioration par Injection,* Travaux, 464, Nov, Paris, France, 1973.
17. HeliCALC Micropile Design Assessment Program, *Theoretical and User's Manual,* Hubbell Power Systems/A.B. Chance Co., 2001.
18. Hetenya, M., *Beams on Elastic Foundations,* The University of Michigan Press, Ann Arbor, MI, 1946.
19. 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.
20. 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.
21. Poulos, H.G., *Analysis of Piles in Soils Undergoing Lateral Movements*, JSMFD, ASCE, Vol. 99, SM5, 1973.

22. 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.

23. Reese, L.C. and S.J. Wright, *Drilled Shaft Design and Construction Guidelines Manual,* US Department of Transportation, Federal Highway Administration, 1977.

24. Reese, L.C., W.M. Wang, J.A. Arrellaga, and J. Hendrix, *Computer Program LPILE^{PLUS} Technical Manual*, Version 3.0, Ensoft, Inc., Austin, TX, 1997.

25. Terzaghi, K. and R.B. Peck, *Soil Mechanics in Engineering Practice,* John Wiley and Sons, Inc., 1967.

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