Chapter 2
Design Stresses

1. Constraints

The design of deep foundations is usually determined by limits specified for lateral or vertical displacement of the pile and placement tolerances. Limiting values of allowable stresses for different deep foundations are included in Table 1-4. Structural capacity rarely controls design, except when piles are founded on rock. Driven piles experience maximum stress during driving, while maximum vertical stresses in drilled shafts usually occur under static conditions.

a. Limiting deformations. Vertical and lateral live load displacements should be limited to 0.5 inch. However, operational requirements may necessitate additional restrictions. Long-term displacements may be larger than computed values due to creep. Cyclic loads and close spacings may increase displacements and should be considered in the design. Methods are presented later for the computation of displacements of deep foundations under vertical and lateral loadings.

b. Geometric constraints

(1) Driven piles. Piles are normally spaced three to four times the diameter from center to center. Typical tolerance of lateral deviation from the specified location at the butt is not more than 3 to 6 inches horizontally. The slope from vertical alignment is typically not more than 0.25 inch per foot of length for large pile groups. A deviation of ± 1 inch from the specified cutoff elevation is reasonable. Sloping land surfaces may require adjustment of the pile location if the surface varies from the reference plane used in the plans to depict pile locations. Other geometric constraints could be related to the following:

(a) Pile spacing. Bearing and lateral resistance of each pile will be reduced if piles are spaced too closely; close spacing might cause foundation heave or damage to other already driven piles. End bearing piles should usually be spaced not less than three pile diameters from center to center, while friction piles should be spaced a minimum of three to five pile diameters from center to center. Large groups of nine or more piles may be checked for pile interference using program CPGP (Wolff 1990). Methods presented later in Chapter 5 for computing the capacity of closely spaced piles may be used in a specific design to find the optimum spacing for piles of a given type.

(b) Pile batter. Batter piles are used to support structures subject to large lateral loads or for smaller lateral loads if the upper foundation stratum will not adequately resist lateral movement of vertical piles. Piles may be battered in opposite directions or used in combination with vertical piles. The axial load on a batter pile should not exceed the allowable design load for a vertical pile, and batter should not be greater than 1 horizontal to 2 vertical; the driving efficiency of the hammer decreases as the batter increases.

(c) Sweep. Specifications should include initial sweep (camber) limitations, because piles curved as a result of excessive sweep will be driven out of tolerance. Sweep for steel H-piles, for example, may be limited to 2 inches and for H-piles, up to 42 feet in length. Refer to the American Institute of Steel Construction "Manual of Steel Construction" (AISC 1989) for further information. The required number and locations of permissible pick-up points on the pile should also be clearly indicated in the specifications. Loading and unloading of long steel piles should be done by support at a minimum of two points about one-fourth of the pile length from each end of the pile. Precast concrete piles should be supported at several points.

(2) Drilled shafts. Drilled shafts are normally placed vertically and spaced at relatively large distances exceeding eight times the shaft diameter. Guidelines for placement tolerances are given in Table 2-1. Greater tolerances can be considered for drilled shafts in difficult subsoils. The additional axial and bending moments stresses caused by accidental eccentricity or batter can be calculated by methods in Chapter 4.

2. Factored Loads

The driven pile or drilled shaft in a group carrying the maximum factored load will be checked for structural failure.

a. Criterion. Calculation of the factored load from the dead and live loads on a pile or drilled shaft is given by equation 2-1:

$$F_e \, \phi_{pf} \, Q_{cap} \geq F_{DL} \, Q_{DL} + F_{LL} \, Q_{LL} \tag{2-1}$$

where

F_e = eccentricity factor, Table 2-2

ϕ_{pf} = performance factor, Table 2-2

Q_{cap} = nominal structural capacity, kips

F_{DL} = dead load factor equals 1.35 for drilled shafts

F_{LL} = live load factors equals 2.25 for drilled shafts

Q_{DL} = dead load, kips

Q_{LL} = live load, kips

Table 2-1
Tolerances in Drilled Shaft Construction

Location of axis	Within 3 inches of the plan location
Vertical plumb or batter alignment	Within 1.5 inches for first 10 feet and 0.5 inch for each additional 10 feet of the total length; maximum 2 percent of shaft length for battered shafts
Top elevation	Not more than 1 inch above or 3 inches below the plan elevation
Cross sections of shafts and underreams	Not less than design dimensions; not more than 10 percent greater than design cross section in shrink/swell soil; underream diameter not to exceed three times the shaft diameter
Reinforcement cage	Not more than 6 inches above or 3 inches below the plan elevation; at least 3 inches of concrete cover around the cage perimeter

b. Calculation of maximum load. The maximum load on a single pile in a group or on a drilled shaft can be determined from computer or hand solutions.

(1) Computer solutions. The pile or drilled shaft carrying the maximum axial load in a group can be determined from computer program CPGA (U.S. Army Engineer Waterways Experiment Station Technical Report ITL-89-3), which computes the distribution of axial loads in a pile group for a rigid pile cap. The maximum inclined and eccentric load in a group can be determined from computer program BENT1 (U.S. Army Engineer Waterways Experiment Station Miscellaneous Paper K-75-2). The total vertical and lateral loads are input into program BENT1.

(2) Hand solutions. If all piles in a group have similar geometry, the axial load on any pile Q_{xy} of an eccentrically loaded pile group may be calculated by hand (Scott 1969)

$$Q_{xy} = Q_g \left[\frac{1}{n} + \frac{e_x X}{\Sigma x^2} + \frac{e_y Y}{\Sigma y^2} \right] \tag{2-2}$$

where

Q_{xy} = load on a pile at a distance x and y from the centroid, kips

Q_g = total vertical load on a group of n piles at a distance e_x and e_y from the centroidal axes, Figure 2-1.

n = number of piles in the group

Σx^2 = sum of the square of the distance x of each pile from the centroid in the x direction, ft^2

Σy^2 = sum of the square of the distance y of each pile from the centroid in the y direction, ft^2

The x and y summations of the pile group in Figure 2-1 are: $\Sigma x^2 = 8 \times 1.5^2 = 18$ ft^2 and $\Sigma y^2 = 4 \times 1.5^2 + 4 \times 4.5^2 = 90$ ft^2. The pile with the maximum load is No. 4 in Figure 2-1, and it is calculated from equation 2-2

$$Q_4 = Q_g \left[\frac{1}{8} + \frac{0.8 \times 1.5}{18} + \frac{3.0 \times 4.5}{90} \right]$$
$$= Q_g \, [\, 0.125 + 0.067 + 0.150 \,] = 0.342 Q_g$$

Pile No.5 carries the minimum load, which is a tension load

$$Q_5 = Q_g \left[\frac{1}{8} + \frac{0.8 \times (-1.5)}{18} + \frac{3.0 \times (-4.5)}{90} \right]$$
$$= Q_g \, [\, 0.125 - 0.067 - 0.150 \,] = -0.092 Q_g$$

c. Buckling resistance. Driven piles or drilled shafts that extend above the ground surface through air or water, or when the soil is too weak to provide lateral support, may buckle under axial loads. Buckling failure may control axial load capacity of the pile.

(1) Buckling capacity. The critical buckling load Q_{cb} of partially embedded piles or drilled shafts may be estimated by (Davisson and Robinson 1965) as follows:

Table 2-2
Performance and Eccentricity Factors (Barker et al. 1991) (Copyright permission, National Cooperative Highway Research Program)

Type of Pile	Performance Factor, ϕ_{pf}	Eccentricity Factor, F_e
Prestressed concrete	Spiral columns: 0.75 Tied columns: 0.70	Spiral columns: 0.85 Tied columns: 0.80
Precast concrete	Spiral columns: 0.75 Tied columns: 0.70	Spiral columns: 0.85 Tied columns: 0.80
Steel H-piles	0.85	0.78
Steel pipe	0.85	0.87
Timber	1.20*	0.82
Drilled shafts	Spiral columns: 0.75 Tied columns: 0.70	Spiral columns: 0.85 Tied columns: 0.80

Note: ϕ_{pf} is greater than unity for timber piles because the average load factor for vertical loads is greater than the *FS*.

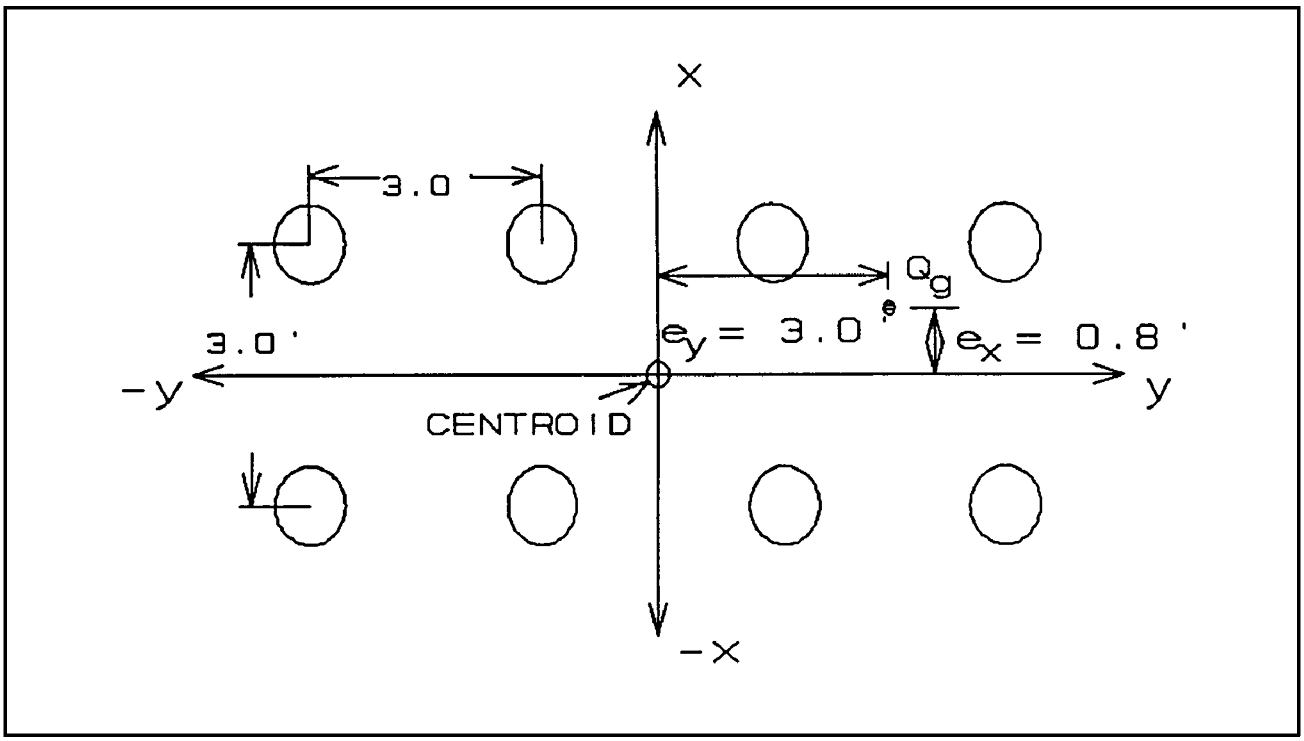


Figure 2-1. Eccentric load on a pile group

BRACED AT TOP

Rigid Cap: $$Q_{cb} = \frac{4\pi^2 E_p I_p}{L_{eq}^2} \qquad (2\text{-}3a)$$

Flexible Cap: $$Q_{cb} = \frac{2\pi E_p I_p}{L_{eq}^2} \qquad (2\text{-}3b)$$

UNBRACED AT TOP

Rigid Cap: $$Q_{cb} = \frac{\pi^2 E_p I_p}{L_{eq}^2} \qquad (2\text{-}3c)$$

Flexible Cap: $$Q_{cb} = \frac{\pi^2 E_p I_p}{4 L_{eq}^2} \qquad (2\text{-}3d)$$

where

Q_{cb} = critical buckling load, kips

E_p = pile (shaft) elastic modulus, ksf

I_p = pile (shaft) moment of inertia, ft^4

L_{eq} = equivalent pile length, ft

The safe design load Q_d will be calculated using normal design procedures for columns and beam-columns when end moments and eccentricity are considered or by equation 2-1. This load will be less than Q_{cb}.

(2) Equivalent length. The equivalent length L_{eq} for long piles (where buckling may occur) can be calculated using the modulus of horizontal subgrade reaction, E_{1s}.

Constant E_{1s}: $$L_{eq} = L_e + 1.4 K_r$$

where

$$K_r = \left(\frac{E_p I_p}{E_{1s}} \right)^{1/4} \qquad (2\text{-}4a)$$

E_{1s} Increasing

Linearly with depth: $$L_{eq} = L_e + 1.8 K_r$$

where

$$K_r = \left(\frac{E_p I_p}{K_s} \right)^{1/5} \qquad (2\text{-}4b)$$

where

L_e = unsupported length extending above ground, ft

K_r = relative stiffness factor, ft

E_{1s} = modulus of horizontal subgrade reaction, ksf

k_s = constant relating E_{1s} with depth, ksf/ft

Refer to paragraph 1-7c (4)(d) for methods of estimating E_{1s}.

(3) Group effects. Adjacent piles at spacings greater than eight times the pile width or diameter have no influence on the soil modulus or buckling capacity. The E_{1s} decreases to one-fourth of that applicable for single piles when the spacing decreases to three times the pile diameter. The E_{1s} can be estimated by interpolation for intermediate spacings between piles in a group.

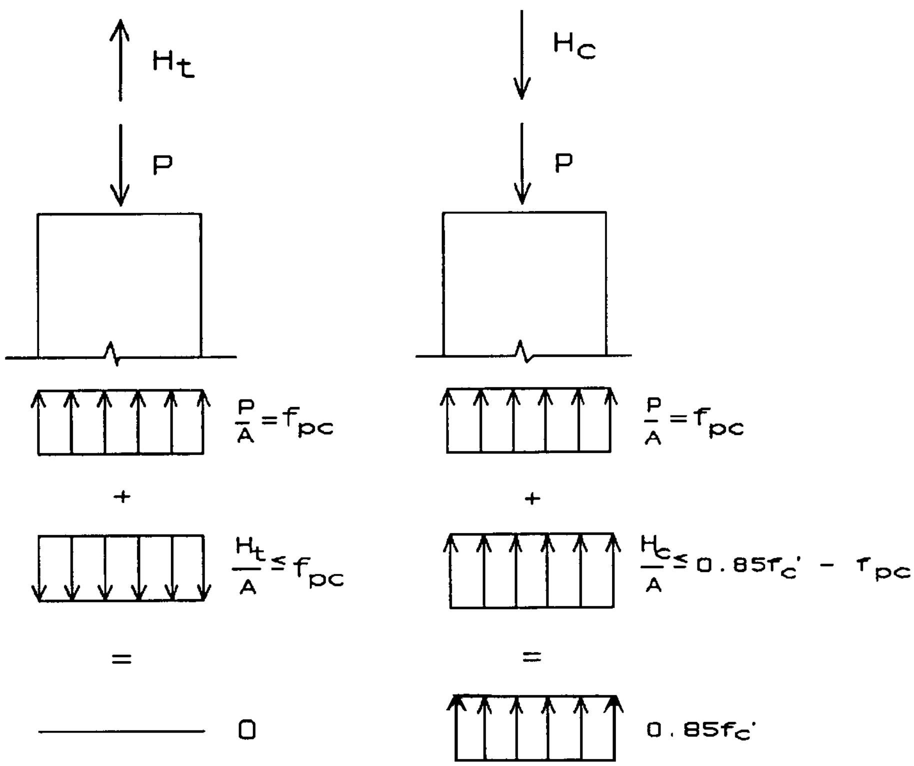
3. Structural Design of Driven Piles

Allowable stresses in piles should be limited to values described below for steel, concrete, and timber piles and will not exceed the factored structural capacity given by equation 2-1. Allowable stresses may be increased by 33 percent for unusual loads such as from maintenance and construction. Allowable stresses may be increased up to 75 percent for extreme loads such as accidental or natural disasters that have a very remote probability of occurrence and that require emergency maintenance following such disasters. Special provisions (such as field instrumentation, frequent or continuous field monitoring of performance, engineering studies and analyses, constraints on operational or rehabilitation activities) are required to ensure that the structure will not catastrophically fail during or after extreme loads. Figure 2-2 provides limiting axial driving stresses. Driving stresses may be calculated by wave equation analyses described in Chapter 6. Structural design in this publication is limited to steel, concrete, and timber piles.

a. Steel piles. Pile shoes should be used when driving in dense sand, gravel, cobble-boulder zones and when driving

Extended Driving and Seating Using Program GRLWEAP

Pile Material	Allowable Driving Stresses
Steel	$0.85f_y$
Reinforced Concrete	
Compression	$0.85f'_c$
Tension	500 psi
Timber	3000 psi



H_c – Compressive force induced by hammer, lb
H_t – Tensile force in pile during driving, lb
f_{pc} – Effective prestress after losses, psi
P – Prestressing force, lb
f'_c – 28 day concrete compressive strength, psi
f_y – Minimum steel yield strength, psi

Figure 2-2. Limits to pile driving stresses piles to refusal on a hard layer such as bedrock. Bending is usually minimal in the lower part of the pile.

(1) The upper portion of a pile may be subject to bending and buckling as well as axial load. A higher allowable stress may be used when the pile experiences damage because the pile enters the inelastic range. This will cause some strain hardening of the steel and increase the pile load capacity. Since damage in the upper region is usually apparent during driving, a higher allowable stress is permitted.

(2) The upper portion of a pile is designed as a beam-column with consideration of lateral support. Allowable stresses for fully supported piles of A-36 steel of minimum yield strength f_y = 36 kips per square inch (ksi) are given in Table 2-3.

(a) Limits to combined bending and axial compression in the upper pile are given by

Table 2-3
Allowable Stresses for Fully Supported Piles. (English Units)

Concentric axial tension or compression in lower pile	Allowable stress, q_a, ksi (f_y = 36 ksi)
10 ksi, (1/3 × f_y × 5/6)	10
Driving shoes, (1/3) · f_y	12
Driving shoes, at least one axial load test and use of a pile driving analyzer to verify pile capacity and integrity, (1/2.5) · f_y	14.5

$$\left| \frac{f_a}{F_a} \pm \frac{f_{bx}}{F_b} \pm \frac{f_{by}}{F_b} \right| \le 1.0 \quad and \quad \frac{f_a}{F_a} \le 0.15 \tag{2-5}$$

where

f_a = computed axial unit stress, ksi

F_a = allowable axial stress in absence of bending stress, $f_y/2$, ksi

f_y = minimum yield strength, ksi

f_{bx} = computed bending stress in x-direction, ksi

f_{by} = computed bending stress in y-direction, ksi

F_b = allowable bending stress in absence of axial stress, ksi

F_b = $0.6 \times f_y$ = 18 ksi for A-36 noncompact sections

F_b = $0.66 \times f_y$ = 20 ksi for A-36 compact sections

A noncompact section can develop yield stress in compression before local buckling occurs, but it will not resist inelastic local buckling at strain levels required for a fully plastic stress distribution. A compact section can develop a fully plastic stress distribution and possess rotation capacity of about 3 before local buckling occurs. Moment rotation capacity is $(\theta_u / \theta_p) - 1$ where θ_u is the rotation at the factored load state and θ_p is the idealized rotation corresponding to elastic theory when the moment equals the plastic moment. Refer to the ASIC Manual of Steel Construction (1989) for further information on noncompact and compact sections.

(b) Allowable stresses for laterally unsupported piles should be 5/6 of those for beam columns given by the AISC Manual of Steel Construction.

(c) A computer program for the analysis of beam columns under lateral loading, as provided in Chapter 4, may be used to compute combined stresses, taking into account all the relevant parameters.

(d) Cross sections of pipe piles may be determined from Appendix B.

(3) Allowable driving stresses are limited to $0.85 f_y$, Figure 2-2.

b. Concrete piles.

(1) Prestressed concrete piles. Allowable concrete stresses should follow the basic criteria of ACI 318-89, except the strength performance factor ϕ_{pf} will be 0.7 for all failure modes and the load factors for both dead and live loads F_{DL} = F_{LL} will be 1.9. The specified load and performance factors provide an FS = 2.7 for all combinations of dead and live loads.

(a) The computed axial strength of the pile shall be limited to 80 percent of pure axial strength or the pile shall be designed for a minimum eccentricity of 10 percent of the pile width.

(b) Driving stresses should be limited as follows:

Compressive stresses: $0.85f'_c$ - effective prestress

Tensile stresses: effective prestress

(c) Cracking control in prestressed piles is achieved by limiting concrete compressive and tensile stresses under service conditions to values indicated in Table 2-4.

(d) Permissible stresses in the prestressing steel tendons should be in accordance with ACI 318-89.

Table 2-4
Allowable Concrete Stresses, Prestressed Concrete Piles

Uniform Axial Tension	0
Bending (extreme fiber)	
Compression	$0.45 \times f'_c$
Tension	0
Combined axial load and bending, the concrete stresses should be proportioned so that:	$f_a + f_b + f_{pc} \le 0.4 \times f'_c$ $f_a - f_b + f_{pc} \ge 0$

where tension is negative for

f_a = computed axial stress, ksi
f_b = computed bending stress, ksi
f_{pc} = effective prestress, ksi
f'_c = concrete compressive strength, ksi

(e) Minimum effective prestress of 700 psi compression is required for piles greater than 50 feet in length. Excessively long or short piles may necessitate deviation from the minimum effective stress requirement.

(f) Strength interaction diagrams for prestressed concrete piles may be developed using computer program CPGC (WES Instruction Report ITL-90-2).

(2) Reinforced concrete piles. These piles will be designed for strength in accordance with ACI 318-89, except that the axial compression strength of the pile shall be limited to 80 percent of the ultimate axial strength or the pile shall be designed for a minimum eccentricity equal to 10 percent of the pile width. Strength interaction diagrams for reinforced concrete piles may be developed using the Corps computer program CASTR (U.S. Army Engineer Waterways Experiment Station Instruction Report ITL-87-2).

(3) Cast-in-place and Mandrel-driven piles. For a cast-in-place pile, the casing is top driven without the aid of a mandrel, typically using casing with wall thickness ranging from 9 gauge to 1/4 inch.

(a) Casing must be of sufficient thickness to withstand stresses due to the driving operation and to maintain the pile cross section. Casing thickness for mandrel-driven piles is normally 14 gauge.

(b) Cast-in-place and mandrel-driven piles should be designed for service conditions and stresses limited to those values listed in Table 2-5.

(c) Allowable compressive stresses are reduced from those recommended by ACI 543R-74 as explained for prestressed concrete piles.

(d) Cast-in-place and mandrel-driven piles will be used only when full embedment and full lateral support are assured and for loads that produce zero or small end moments so that compression always controls. Steel casing will be 14 gauge (U.S. Standard) or thicker, be seamless or have spirally welded seams, have a minimum yield strength of 30 ksi, be 16 inches or less in diameter, not be exposed to a detrimental corrosive environment, and not be designed to carry any working load. Items not specifically addressed will be in accordance with ACI 543R-74.

c. Timber piles. Representative allowable stresses for pressure-treated round timber piles for normal load duration in hydraulic structures are given in Table 2-6.

(1) Working stresses for compression parallel to the grain in Douglas Fir and Southern Pine may be increased by 0.2 percent for each foot of length from the tip of the pile to the critical section. An increase of 2.5 psi per foot of length is recommended for compression perpendicular to the grain.

(2) Values for Southern Pine are weighted for longleaf, ash, loblolly, and shortleaf.

(3) Working stresses in Table 2-6 have been adjusted to compensate for strength reductions due to conditioning and treatment. For piles, air-dried or kiln-dried before pressure treatment, working stresses should be increased by dividing the tabulated values as follows:

Pacific Coast Douglas Fir:	0.90
Southern Pine:	0.85

(4) The *FS* for allowable stresses for compression parallel to the grain and for bending are 1.25 and 1.3, respectively (International Conference of Building Officials 1991).

Table 2-5
Cast-in-Place and Mandrel-driven Piles, Allowable Concrete Stresses

Uniform axial compression	
Confined	$0.33 \times f'_c$
Unconfined	$0.27 \times f'_c$
Uniform axial tension	0
Bending (extreme fiber)	
Compression	$0.40 \times f'_c$
Tension	0

Combined axial loading and bending

$$\left| \frac{f_a}{F_a} + \frac{f_b}{F_b} \right| \le 1.0$$

where

f_a = computed axial stress, ksi
F_a = allowable axial stress, ksi
f_b = computed bending stress, ksi
F_b = allowable bending stress, ksi

Note: Participation of steel casing or shell is disallowed.

Table 2-6
Allowable Stresses for Pressure-treated Round Timber Piles for Normal Loads in Hydraulic Structures

	Allowable Stresses, psi	
	Pacific Coast Douglas Fir	Southern Pine
Compression parallel to grain, F_a	875	825
Bending, F_b	1,700	1,650
Horizontal shear	95	90
Compression perpendicular to grain	190	205
Modulus of elasticity	1,500,000	1,500,000

d. Design example. A 30-foot length L of A-36 unbraced H-pile, HP14 × 73, has a cross section area A_y = 21.4 in.^{2}, Table 1-3. This pile is made of A-36 steel with f_y = 36 ksi, E_p = 29,000 ksi, and I_p = 729 in.^{4} for the X-X axis and 261 in.^{4} for the Y-Y axis. Dead load Q_{DL} = 40 kips and live load Q_{LL} = 60 kips. Load factors F_{DL} = 1.3 and F_{LL} = 2.17. Free-standing length above the ground surface is 10 feet. The soil is a clay with a constant modulus of horizontal subgrade reaction E_{ls} = 1 ksi. Spacing between piles is three times the pile width.

(1) Design load. The applied design load per pile Q_d from equation 2-1 is

$$Q_d = F_{DL} Q_{DL} + F_{LL} Q_{LL}$$

$$= 1.3 \times 40 + 2.17 \times 60 = 182.2 \text{ kips}$$

(2) Structural capacity. The unfactored structural capacity Q_{cap} is $f_y A_y$ = 36 × 21.4 = 770.4 kips. From Table 2-1, ϕ_{pf} = 0.85 and F_e = 0.78. The factored structural capacity is $F_e \phi_{pf} Q_{cap}$ = 0.78 × 0.85 × 770.4 = 508.4 kips > Q_d = 182.2 kips. Table 2-3 indicates that the allowable stress q_a = 10 ksi. $q_a A_y$ = 10 × 21.4 = 214 kips > Q_d = 182.2 kips.

(3) Buckling capacity. A flexible cap unbraced at the top is to be constructed for the pile group. The E_{ls} = (1/4) × 1 ksi = 0.25 ksi because the spacing is three times the pile width. The equivalent length L_{eq} for the constant E_{ls} = 0.25 ksi from equation 2-4a for the minimum I_p of the X-X axis is

$$K_r = \left(\frac{E_p I_p}{E_{ls}} \right)^{1/4} = \left(\frac{29{,}000 \times 261}{0.25} \right)^{1/4} = 131.5 \text{ in.}$$

$$L_{eq} = L_e + 1.4 K_r = 10 \times 12 + 1.4 \times 131.5 = 304.1 \text{ in.}$$

The minimum critical buckling load Q_{cb} from equation 2-3d is

$$Q_{cb} = \frac{\pi^2 E_p I_p}{4 L_{eq}^2}$$

$$= \frac{9.87 \times 29{,}000 \times 261}{4 \times (304.1)^2} = 201 \text{ kips}$$

The Q_{cb} = 201 kips > Q_d = 182.2 kips. Buckling capacity is adequate.

Table 2-7
Minimum Requirements for Drilled Shaft Design

a. Nomenclature

Symbol	Description
A_c	Cross-sectional area of concrete within spiral or cage, inches^2
A_g	Gross area of the shaft section, inches^2
A_s	Area of reinforcement steel, inches^2
A_{sp}	Cross-sectional area of spirals or cage, inches^2
B_s	Shaft outside diameter, inches
Cover	Concrete cover over longitudinal steel, in.
d_b	Bar diameter, in.
d_t	Tie diameter, in.
d_{sp}	Spiral diameter, in.
D_m	Mean diameter of the equivalent steel ring, $B_s - 2(\text{Cover} + r_s)$, in.
e	Eccentricity, M_{max}/Q_u, in.
F_{DL}	Dead load factor to protect against material failure
F_{LL}	Live load factor to protect against material failure
FS	Factor of safety to limit vertical displacement in the foundation soil
f'_c	Ultimate concrete compressive strength, psi
f_y	Yield strength of reinforcement steel, psi
L_{sp}	Length of spiral in one path, in.
M_{max}	Maximum bending moment, lb-in.
N	Number of bars
pitch	Distance from any point of a spiral cage to the adjacent point on the cage measured parallel with the longitudinal axis of the cage
P_{max}	Maximum uplift thrust or pullout force, lb
Q_{cap}	Dependable axial load capacity of the shaft determined by $\phi_{pf}[0.85 \cdot f'_c(A_g - A_s) + A_s f_y]$, lb
Q_{DL}	Dead load, lb
Q_{LL}	Live load, lb
Q_{md}	Maximum downdrag force, lb
Q_u	Maximum factored axial load, lb
R_d	Diameter ratio of reinforcement steel, D_m/B_s
R_s	Ratio of reinforcement steel area to gross area, A_s/A_g
r_s	Radius of reinforcement steel, in.
S_t	Tie spacing, in.
T_{max}	Maximum applied lateral load, lb
v_{max}	Maximum applied shear stress, V_{max}/A_g
V_{max}	Maximum applied shear force, $T_{max}F_{LL}$
ρ_{sp}	Volumetric ratio of spiral reinforcement required
ϕ_{pf}	Capacity performance factor, 0.70 for a tied column of reinforcement steel 0.75 for a spiral column after ACI SP-17 (1985)

(Sheet 1 of 3)

Table 2-7 (Continued)

b. Procedure

Step	Description
1	Determine that Q_w is well within Q_{cap}, $$Q_{cap} > Q_w$$ $$Q_{cap} = \phi_{pf}[0.85 \cdot f'_c(A_g - A_s) + A_s f_y]$$ $$Q_w = F_{DL}Q_{DL} + F_{LL}Q_{LL}$$ For a shaft in compressible soil, evaluate downdrag force Q_{nd} (Chapter 3) and add to Q_w. For a shaft in expansive soil, evaluate uplift thrust P_{max} (negative, tensile force) and calculate the required reinforcement area from $$A_s = -\frac{P_{max}}{\phi_{pf} f_y}$$ Percent steel for f_y = 60,000 psi is approximately $-0.003P_{max}/B_s^2$
2	Determine $v_{max} \leq 10 \cdot (f'_c)^{1/2}$.
3	Compute eccentricity e from given M_{max} and Q_w of the specifications, or 2 in. or $0.1B_s$, whichever is greater for tied columns or 1 in. or $0.05B_s$ for spiral columns of reinforcement steel. M_{max} may also be evaluated from lateral load analysis described in Chapter 4. Calculate eccentricity ratio e/B_s.
4	Calculate D_n. Estimate R_d from the trial ring of reinforcement steel.
5	Determine by interpolation steel ratio R_s from design charts in ACI SP-17, Vol I. If R_s < 1 percent, increase R_s to 1 percent. If R_s > 8 percent, increase the shaft diameter and repeat the above steps.
6	Select the actual steel reinforcement; i.e., size and number of bars. 6 bars minimum, 6 inches spacing maximum. Check for sufficient spacing for flow of concrete, spacing = $(\pi D_n - Nd_b)/N$
7	Calculate R_d for the designed shaft cross section and check it against R_d assumed in step 4. Repeat steps 4 and 6 if the assumed R_d is significantly different from the calculated R_d.
8	Select appropriate ties or spirals to construct the reinforcement cage according to ACI 318-89 requirements or AASHTO specifications. Ties (ACI): Vertical spacing shall not exceed the lesser of (1) $S_t = 16d_b$ (2) $S_t = 48d_t$ (3) $S_t = B_s$ No. 3 ties may be used for longitudinal bars < No. 11; No. 4 or larger ties will be used for No. 11 bars or larger Spirals (ACI): $$\rho_{st} = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y}$$ $$L_{st} = \sqrt{\pi(d_{st}+d_t)^2 + pitch^2}$$ $$A_{sp} = \frac{\rho_{st} \cdot pitch \cdot A_c}{L_{st}}$$

(Sheet 2 of 3)

Table 2-7 (Concluded)

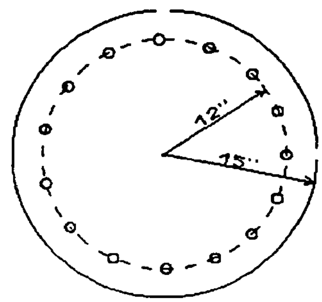
c. Example calculation of steel reinforcement for a tied reinforcement cage

Step | **Calculation**

1 Input parameters:

T_{max} = 10,000 lb	Cover = 3 in.	f'_c = 3,000 psi
Q_{DL} = 500,000 lb	B_c = 30 in.	f_y = 60,000 psi
Q_{LL} = 500,000 lb	F_{DL} = 1.35	M_{max} = $8 \cdot 10^6$ lb-in.
	F_{LL} = 2.25	ϕ_{sf} = 0.70

Assume initital R_s = 2 percent. Calculate $A_g = \pi B_c^2/4 = 706.86$ in.^{2}. Initial estimate of $A_s = 0.02 \cdot A_g = 14.14$ inches. Fifteen No. 8 bars will be required for the initial estimate of steel reinforcement with r_s = 0.5 inch as shown in the sketch.



$Q_u = 1.35 \cdot 500{,}000 + 2.25 \cdot 500{,}000 = 1{,}800{,}000$ lb
$Q_{cap} = 0.7[0.85 \cdot 3{,}000(706.86 - 14.14) + 14.14 \cdot 60{,}000\} = 1{,}830{,}385$ lb
$1{,}830{,}385 > Q_u = 1{,}800{,}000$ lb; Q_{cap} is adequate

2 $v_{max} = T_{max}F_{LL}/A_g = 10{,}000 \cdot 2.25/706.86 = 31.83$ psi $\leq 10 \cdot (f'_c)^{1/2}$

$10 \cdot (f'_c)^{1/2} = 10 \cdot (3{,}000)^{1/2} = 547.7$ psi
Therefore, $v_{max} \leq 10 \cdot (f'_c)^{1/2}$

3 $e/B_c = M_{max}/(Q_u B_c) = 8 \cdot 10^6/(787{,}500 \cdot 30) = 0.339$

4 $D_s = B_c - 2(\text{Cover} + r_s) = 30 - 2(3 + 0.5) = 23$ in.
$R_d = 23/30 = 0.767$

5 Determine R_s by interpolation from design charts, ACI SP-17 (85):
Therefore, $R_s = 0.023$ for $R_d = 0.767$

R_d	R_s
0.7	0.026
0.8	0.021

6,7 $A_s = 0.023 \cdot 706.86 = 16.26$ in.^{2}

Spacing between bars:
21 No. 8 bars ($A_s = 21 \cdot 0.79 = 16.59$ in^2, d_b = 1.000 inch):
spacing = $(\pi \cdot 23 - 21 \cdot 1.000)/21 = 2.44$ in.
17 No. 9 bars ($A_s = 17 \cdot 1.00 = 17.00$ in^2, d = 1.128 inch):
spacing = $(\pi \cdot 23 - 17 \cdot 1.128)/17 = 3.12$ in.
13 No. 10 bars ($A_s = 13 \cdot 1.27 = 16.51$ in.^{2}, d = 1.270 inch):
spacing = $(\pi \cdot 23 - 13 \cdot 1.127)/13 = 4.43$ in.
17 No. 9 bars will be adequate assuming 1 inch maximum aggregate and spacing of at least 3 times the maximum aggregate size

8 Ties: (1) $S_t = 16 \cdot 1.000 = 16$ in.
(2) $S_t = 48 \cdot 0.375 = 18$ inches for No. 3 tie
(3) $S_t = 30$ in.
Select No. 3 bars at 16 inches center to center spacing

Spirals: (standard spiral sizes are 3/8, 1/2 and 5/8 inch; select 1/2 inch spiral size and 3-inch pitch)

A_g = 706.9 inches^2 $\quad A_c = \pi \cdot 12^2 = 452.4$ inches^2
$\rho_s = 0.45 \cdot [(706.9/452.4) - 1] \cdot 3{,}000/60{,}000 = 0.0152$
$L_s = \{[\pi \cdot (24 + 0.5)]^2 + 3^2\}^{1/2} = 77$ in.
$A_{sp} = \rho_s \cdot \text{pitch} \cdot A_c/L_s = 0.0152 \cdot 3 \cdot 452.4/77 = 0.268$ inches^2
Select No. 5 spiral at 3-inch pitch

4. Structural Design of Drilled Shafts

Most drilled shaft foundations will be subject to lateral loads, bending moments, and shear stresses in addition to compressive stresses from vertical loads. Eccentrically vertical applied loads can generate additional bending moments.

a. Eccentricity. If bending moments and shears are not specified, the minimum eccentricity should be the larger of 2 inches or $0.1B_s$, where B_s is the shaft diameter, when tied cages of reinforcement steel are used and 1 inch or $0.05B_s$ when spiral cages are used. The minimum eccentricity should be the maximum permitted deviation of the shaft out of its plan alignment that does not require special computations to calculate the needed reinforcement if larger eccentricities are allowed.

b. Design example. Table 2-7 describes evaluation of the shaft cross section and percent reinforcement steel required for adequate shaft strength under design loads.

(1) The maximum bending moment, M_{max}, is required to determine the amount of reinforcement steel to resist bending. The maximum factored vertical working load, Q_w, and the estimate of the maximum applied lateral load, T_{max}, are used to calculate M_{max}. The full amount of reinforcing steel is not required near the bottom of the pile because bending moments are usually negligible near the pile bottom. Chapter 4 discusses procedures for calculating the distribution of bending moments to determine where steel will be placed in the pile.

(2) Load factors are applied to the design live and dead loads to ensure adequate safety against structural failure of the shaft. An example is worked out in Table 2-7c for F_{DL} = 1.35 and F_{LL} = 2.25 for a shaft supporting a bridge column.

(3) The minimum reinforcement steel, normally recommended, is 1 percent of the total cross-sectional area of drilled shaft expected to be exposed along their length by scour or excavation. Reinforcement steel should be full length for shafts constructed in expansive soil and for shafts requiring casing while the hole is excavated. Shaft diameter should be increased if the reinforcement steel required to resist bending such that adequate voids through the reinforcement cage will be provided to accommodate the maximum aggregate size of the concrete.

(4) The maximum applied axial load should also include maximum downdrag forces for a shaft in compressible soil and the maximum uplift thrust for a shaft in expansive soil. Uplift thrust may develop before the full structural load is applied to the shaft. Under such conditions, smaller amounts of reinforcement may be used if justified on the basis of relevant and appropriate computations.