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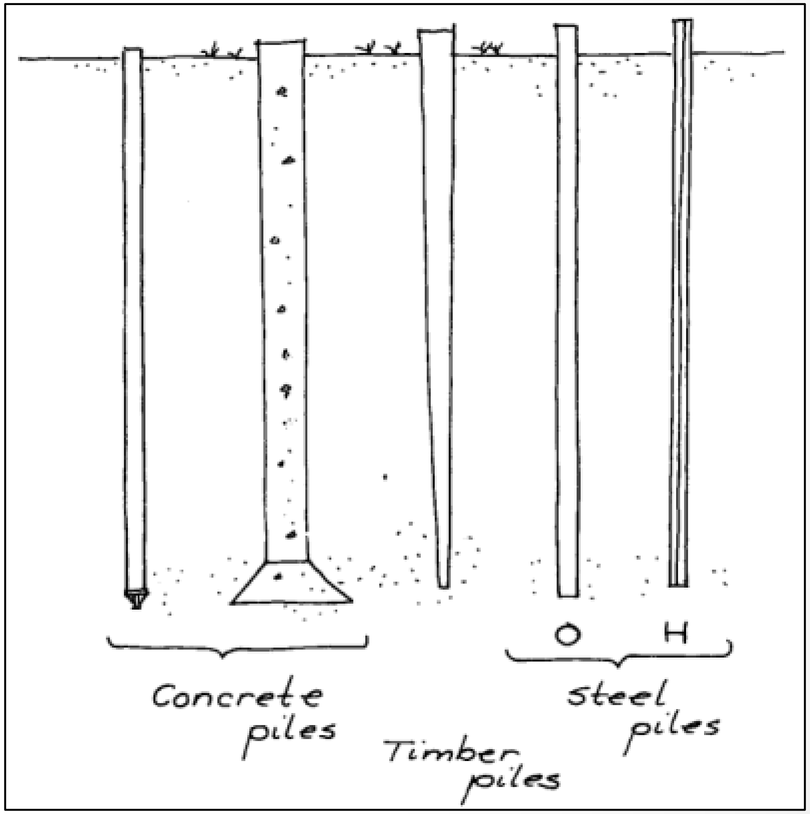
ENCE 4610
Foundation Analysis and Design

Lecture 24
Deep Foundations
Structural Design

Structural Design

- Structural Design of Deep Foundations is a very broad topic, we can only cover certain cases
  - Overview of LRFD philosophy as applied to deep foundations
  - Design of H-Beams
  - Design of Prestressed Concrete Piles
- Drilled shaft design is based on concrete columns, which in fact is what they are, so we will not cover them here

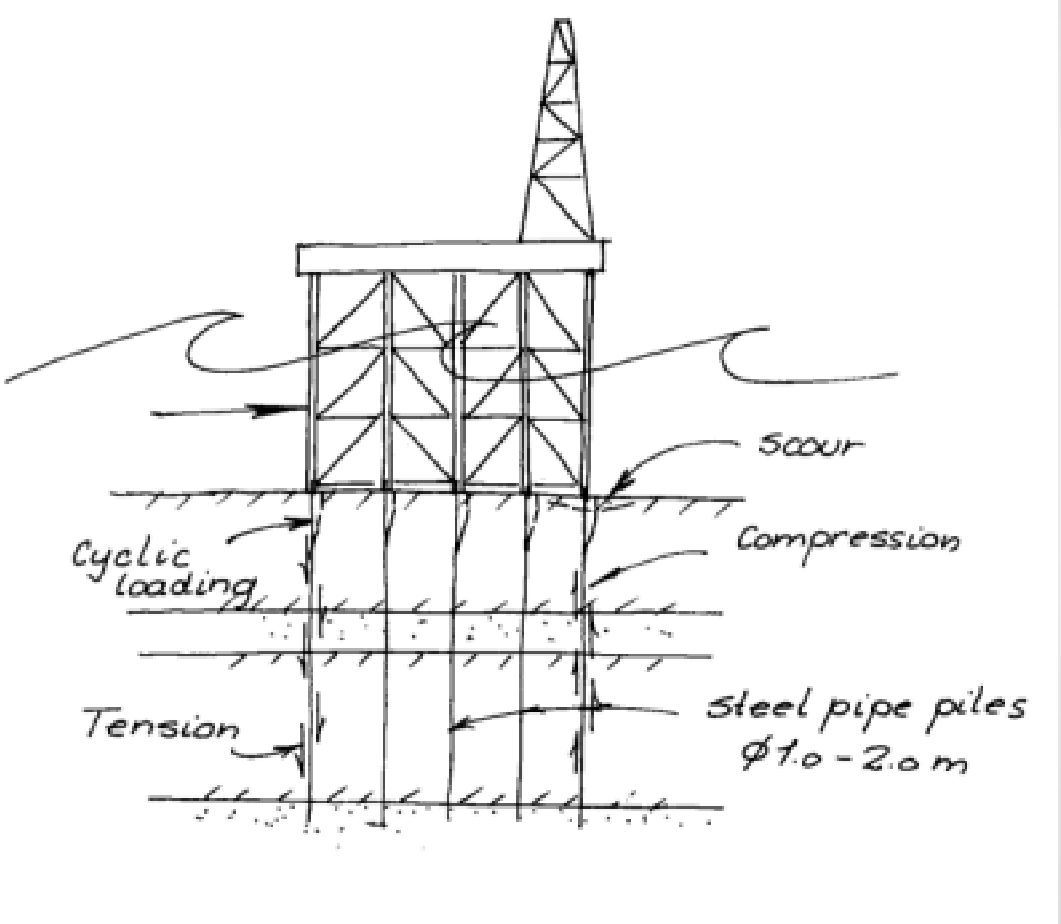


Design Philosophy

- Comparison with Superstructure Design
  - Conservative design is used with deep foundations because:
    - Construction tolerances are wider and quality control is more difficult
    - Piles can be damaged during driving, reducing their capacity
    - Drilled shafts of all kinds can be formed with voids, aggregate separation, soil contamination, and other problems
    - Residual stresses can be present in driven piles
  - Some specs for driven piles are conservative to limit the driving stresses more than to take into consideration foundation conditions in service
- Buckling
  - Traditionally, most driven piles have not been designed to resist buckling
  - However, newer design codes (AASHTO 2014) virtually mandate the inclusion of buckling calculations
  - This will be discussed in the slides below

Design Philosophy

- ASD vs. LRFD
  - Traditionally, deep foundations have been designed using ASD
  - LRFD becoming more common due to impetus by FHWA and AASHTO
  - Concrete elements are structurally designed almost exclusively by LRFD using ACI codes
    - Spread Footings (seen earlier)
    - Precast and Prestressed Concrete Piles
    - Drilled Shafts and CFA (auger cast) piles
    - Pile Caps and Grade Beams
  - With LRFD, basic design methods (structural and geotechnical) are the same, but application of factors are different



Structural Analysis of Driven Steel Piles

- Driven piles can be loaded either axially or laterally
- Buckling is checked
  - When buckling is not considered, the pile is treated as a compression (or tension) member for axial loading
- Lateral loads are treated like beam loads (which in fact they are)
- We will present an LRFD based design procedure for structural analysis of driven piles, specifically steel piles
- The load factors are the same as those for geotechnical loads
- Generally, geotechnical configuration is done first, then structural
- The reference for this is FHWA-NHI-16-009, Vol. 1, pp. 445-453

Resistance Factors for Driven Piles

8.3.3 Resistance Factors

A discussion and step by step determination of the nominal structural resistance for timber, steel, and concrete piles is provided in the following sections. The AASHTO (2014) specifications form the basis of these respective sections. Following the Load and Resistance Factor Design (LRFD) approach, a resistance factor is applied to the calculated nominal structural resistance.

In practical terms, the imposed factored load must be less than or equal to the factored resistance. Chapter 2 provides a discussion on load combinations in which load factors are applied to respective load effects. The critical load combination is applied as axial, shear and moment loads on the structural member and will be described herein as follows:

Factored axial load:

$$\sum(n_i\gamma_i)Q_i = Q \rightarrow \mathrm{P}_u \qquad \text{Eq. 8-10}$$

Factored moment load:

$$\sum(n_i\gamma_i)Q_i = Q \rightarrow \mathrm{M}_u \qquad \text{Eq. 8-11}$$

Factored shear load:

$$\sum(n_i\gamma_i)Q_i = Q \rightarrow \mathrm{V}_u \qquad \text{Eq. 8-12}$$

The nominal structural resistance is specific to pile properties such as material, size, and shape. A separate discussion based on pile material is provided in Section 8.4 through Section 8.7. On the resistance side, the factored structural resistance will be described as shown below.

Factored resistance in axial compression:

$$\sum\phi_i R_i = \varphi_c P_n = \mathrm{P}_r \qquad \text{Eq. 8-13}$$

Factored resistance in flexure:

$$\sum\phi_i R_i = \varphi_f M_n = \mathrm{M}_r \qquad \text{Eq. 8-14}$$

Factored resistance in shear:

$$\sum\phi_i R_i = \varphi_v V_n = \mathrm{V}_r \qquad \text{Eq. 8-15}$$

AASHTO (2014) recommended resistance factors for driving and structural resistance are provided in Table 8-5 and Table 8-6, respectively. As an example, for a steel H-pile with good driving conditions and with the use of a pile tip (e.g. pile point or shoe), ϕ_{da} = 1.0 would be used in Equation 8-33 (Section 8.5.1) along with the material yield stress to determine the nominal driving stress limits. For nominal structural resistance, ϕ_c = 0.60, would be entered into Equation 8-34 (Section 8.5.2). A similar procedure is therefore used for alternative pile types, driving conditions, and loading cases.

Resistance Factors for Driven Piles

Table 8-6 Structural Limit Resistance Factors for Piles in Compression

Pile Material	Design Resistance	Resistance Factor
Steel (AASHTO 6.5.4.2)	Axial - Good driving conditions where pile tip is not necessary	
	H-piles	ϕ_c = 0.60
	Pipe Piles	ϕ_c = 0.70
	Axial -Pile is subject to damage due to severe driving conditions where pile tip is necessary	
	H-piles	ϕ_c = 0.50
	Pipe Piles	ϕ_c = 0.60
	Combined Axial and Flexural for Undamaged Piles	
	Axial – H-piles	ϕ_c = 0.70
	Axial – Pipe Piles	ϕ_c = 0.80
	Flexure – Both Pile Types	ϕ_f = 1.00
	Shear	ϕ_v = 1.00
Concrete (AASHTO 5.5.4.2)	Tension Controlled	
	Reinforced Concrete	ϕ_c = 0.90
	Prestressed Concrete	ϕ_c = 1.00
	Compression Controlled	
	Prestressed Concrete	ϕ_c = 0.75
Timber (AASHTO 8.5.4.2)	Compression Parallel to Grain	ϕ_c = 0.90
	Flexure	ϕ_f = 0.85
	Shear	ϕ_v = 0.75

Table 8-5 Resistance Factors During Pile Driving

Condition	Resistance Determination Method	Resistance Factor
Pile Drivability, ϕ_{da}	Steel Piles (AASHTO 6.5.4.2)	ϕ_{da} = 1.0
	Concrete Piles (AASHTO 5.5.4.2)	ϕ_{da} = 1.0
	Timber Piles (AASHTO 8.5.4.2)	ϕ_{da} = 1.15

Steel Pile Design

8.5 STEEL PILES

8.5.1 Driving Stresses

Steel piles can handle higher stresses than concrete or timber during driving, and the limit is governed the steel yield strength. Several grades of steel are routinely produced and higher grades may be specified if warranted. Table 8-1 provides an overview of typical steel grades and their respective yield strength. Pile driving does not typically generate sufficiently high tensile stresses to yield steel piles, therefore a stress limit is only required in compression. For steel piles, AASHTO (2014) specifications limit nominal compression driving stresses as follows:

$$\sigma_{dr} = \varphi_{da}\left(0.9\,F_y\right) \qquad \text{Eq. 8-33}$$

Where:

σ_{dr} = driving stress limit (ksi).
φ_{da} = resistance factor during driving (1.0 for steel piles, Table 8-5).
F_y = yield stress of steel (ksi) (Table 8-1).

8.5.2 Structural Resistance

8.5.2.1 Axial Compression

For axial compression loads, the factored structural limit state is taken as:

$$P_r = \varphi_c P_n \qquad \text{Eq. 8-34}$$

Where:

P_r = factored resistance in axial compression (kips).
φ_c = resistance factor (Table 8-6).
P_n = nominal resistance in axial compression (kips).

To determine the nominal resistance in axial compression, pile strength and buckling failure should be considered. The following step by step procedure should be used to calculate the nominal resistance.

STEP BY STEP PROCEDURE FOR: "NOMINAL COMPRESSION RESISTANCE"

STEP 1 Determine the equivalent nominal yield resistance, P_o.

The equivalent nominal yield resistance, P_o, is a function of the material yield stress, cross sectional area and slenderness reduction factor, if applicable. For non-slender piles in compression, the slenderness reduction factor, Q, is taken as 1.0. However for slender piles, the full nominal yield strength under uniform axial compression is limited by local buckling. This reduction factor is governed by section buildup, pile dimensions and material properties, therefore, a further discussion of slender members and direction for calculating Q may be found in AASHTO (2014) Article 6.9.4.2.2.

$$P_o = QF_yA_g \qquad \text{Eq. 8-35}$$

Where:

A_g = gross cross-sectional area (in^2).
P_o = equivalent nominal axial yield resistance (kips).
F_y = yield stress of steel (Table 8-1) (ksi).
Q = slender element reduction factor (dimensionless).

To satisfy the slender element requirement for local buckling, Equation 8-36 is used for H-piles and Equation 8-38 is used for unfilled pipe piles.

$$\frac{b_f}{2t_f} \le 0.64\sqrt{\frac{k_c E_{st}}{F_y}} \qquad \text{Eq. 8-36}$$

And:

$$0.35 \le k_c \le 0.76$$

In which:

$$k_c = \frac{4}{\sqrt{\frac{d_w}{t_w}}} \qquad \text{Eq. 8-37}$$

Where:

b_f = flange width (inches).
t_f = flange thickness (inches).
F_y = yield stress of steel (Table 8-1) (ksi).
E_{st} = elastic modulus of steel (ksi).
d_w = web depth (inches).
t_w = web thickness (inches).

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$$\frac{D}{t} \leq 0.11\frac{E_{st}}{F_y} \qquad \text{Eq. 8-38}$$

Where:

D = diameter of pipe (inches).
t = wall thickness (inches).
F_y = yield stress of steel (Table 8-1) (ksi).
E_{st} = elastic modulus of steel (ksi).

STEP 2 Determine the elastic critical buckling resistance, P_e.

In determination of the nominal resistance in axial compression, buckling may occur with a lack of sufficient bracing. This topic is discussed further in Section 8.2.3. AASHTO (2014) requires both flexural and torsional modes of buckling be checked if applicable. For fully embedded piles, the flexural buckling mode will be used. However, when the pile extends through water or air, doubly symmetric open section members (e.g., H-piles) must be evaluated for torsional buckling as well. The critical failure mode is the lesser buckling resistance, and is employed to define the nominal resistance in axial compression.

Flexural buckling:

$$P_e = \frac{\pi^2 E_{st} A_g}{\left(\frac{Kl}{r_s}\right)^2} \qquad \text{Eq. 8-39}$$

Where:

P_e = elastic critical buckling resistance (kips).
E_{st} = elastic modulus of steel (ksi).
A_g = gross cross-sectional area (in^2).
K = effective length in the plane of buckling (Figure 8-4) (dimensionless).
l = unbraced length in the plane of buckling (Section 8.3) (inches).
r_s = radius of gyration about axis normal to plane of buckling (inches).

Torsional buckling:

$$P_e = \left[\frac{\pi^2 E_{st} C_w}{(K_z l_z)^2} + GJ\right]\frac{A_g}{I_x + I_y} \qquad \text{Eq. 8-40}$$

In which:

$$C_w = \frac{I_y h^2}{4} \qquad \text{Eq. 8-41}$$

$$G = 0.385 E_{st} \qquad \text{Eq. 8-42}$$

Where:

P_e = elastic critical buckling resistance (kips).
E_{st} = elastic modulus of steel (ksi).
C_w = warping torsional constant (doubly symmetric open sections) (in^6).
K_z = effective length for torsional buckling (Figure 8-4) (dimensionless).
l_z = unbraced length for torsional buckling (inches).
G = shear modulus (ksi).
J = St. Venant torsional constant (in^4).
A_g = gross cross-sectional area (in^2).
I_x, I_y = moments of inertia about the major and minor principal axes of cross section, respectively (in^4).
h = distance between flange and centroids (inches).

STEP 3 Determine the nominal resistance in axial compression, P_n.

With the above resistances defined, the nominal resistance for axial compression may be evaluated using the following equations, which are provided in AASHTO (2014) Article 6.9.4.1.

If $\frac{P_e}{P_o} \geq 0.44$:

$$P_n = P_o\, 0.658^{\frac{P_o}{P_e}} \qquad \text{Eq. 8-43}$$

If $\frac{P_e}{P_o} < 0.44$:

$$P_n = 0.877 P_e \qquad \text{Eq. 8-44}$$

Where:

P_n = nominal resistance in axial compression (kips).
P_o = equivalent nominal yield resistance (Equation 8-35) (kips).
P_e = elastic critical buckling resistance (Equation 8-39 or 8-40) (kips).

Steel Pile Design

8.5.2.2 Flexure

The factored resistance in flexure is computed as follows:

$$M_r = \varphi_f M_n \qquad \text{Eq. 8-45}$$

Where:

- M_r = factored resistance in flexure (kip-in).
- φ_f = resistance factor (Table 8-6).
- M_n = nominal resistance in flexure (kip-in).

The nominal flexural resistance is a function of pile shape as well as general pile properties. Steel piles are primarily H-piles or pipe piles. Therefore the step by step procedure that follows will consider only these two steel pile types. If alternative sections are used, the engineer is referred to Article 6.12.2.2 of the AASHTO (2014) specifications. Steel H-piles and I-sections are treated equally for flexural resistance. Hence, part A of this procedure applies to both steel H-piles and miscellaneous I sections.

STEP BY STEP PROCEDURE FOR: "NOMINAL FLEXURAL RESISTANCE"

A. Steel H-Sections.

STEP 1 Check flange slenderness ratio and limiting slenderness.

$$\lambda_f = \frac{b_f}{2t_f} \qquad \text{Eq. 8-46}$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_{yf}}} \qquad \text{Eq. 8-47}$$

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_{yf}}} \qquad \text{Eq. 8-48}$$

Where:

- λ_f = slenderness ratio for flange.
- λ_{pf} = limiting slenderness ratio for a compact flange.
- λ_{rf} = limiting slenderness ratio for a non-compact flange.

- b_f = flange width (inch).
- t_f = flange thickness (inch).
- E_{st} = elastic modulus of steel (ksi).
- F_{yf} = minimum yield strength of lower strength flange (ksi).

STEP 2 Determine the nominal flexural resistance.

To determine the nominal flexural resistance, the above slenderness definitions should first be resolved. These functions serve to define the limiting flexural resistance. In the case where the limiting slenderness ratio of a compact flange is greater than the slenderness ratio, the plastic moment about the weak axis will limit resistance. For H-piles, Eq. 8-49 can be used. Conversely, Eq. 8-51 should be used when the slenderness ratio is greater than the limiting slenderness ratio of a compact flange.

If $\lambda_f \le \lambda_{pf}$:

$$M_n = M_p \qquad \text{Eq. 8-49}$$

In which, for HP-sections about weak axis:

$$M_p = 1.5\, F_y\, S_y \qquad \text{Eq. 8-50}$$

If $\lambda_{pf} < \lambda_f \le \lambda_{rf}$ the nominal flexural resistance about the weak axis is:

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_{yf}}}}\right)\right] f_{yf} Z_y \qquad \text{Eq. 8-51}$$

Where:

- M_n = nominal resistance in flexure (kip-in).
- M_p = plastic moment about the weak axis (kip-in).
- S_y = elastic section modulus about weak axis (in^3).
- Z_y = plastic section modulus about weak axis (in^3).
- λ_f = slenderness ratio for flange (Equation 8-46, dimensionless).
- λ_{pf} = limiting slenderness ratio for a compact flange (Equation 8-47, dimensionless).

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E_{st} = elastic modulus of steel (ksi).
F_y = yield stress of steel (ksi).
F_{yf} = minimum yield strength of lower strength flange (ksi).

B. Steel Pipe Piles.

STEP 1 Check diameter to thickness ratio.

If the diameter to thickness ratio is sufficiently large, local buckling limits flexural resistance. To determine whether the plastic moment or local buckling will govern flexural resistance, Equation 8-52 should be applied. If Equation 8-52 is satisfied, the plastic moment will yield the steel pile and Step 2a should follow. Conversely, local buckling will limit flexural resistance if Equation 8-2 is not satisfied, and therefore Step 2b should follow.

$$\frac{D}{t} \leq 0.07 \frac{E_{st}}{F_y} \qquad \text{Eq. 8-52}$$

Where:
D = outside diameter of pipe (inch).
t = pipe thickness (inch).
E_{st} = elastic modulus of steel (ksi).
F_y = yield strength of steel (Table 8-1) (ksi).

STEP 2a Determine nominal flexural resistance by plastic moment.

The nominal flexural resistance can be taken as follows:

$$M_n = M_p = F_y Z_y \qquad \text{Eq. 8-53}$$

Where:
M_n = nominal resistance in flexure (kip-in).
M_p = plastic moment (kip-in).
Z_y = plastic section modulus about weak axis (in^3).
F_y = yield strength of steel (Table 8-1) (ksi).

STEP 2b Determine nominal flexural resistance by local buckling.

Where local buckling will limit the nominal resistance in flexure, the following checks apply.

If $\frac{D}{t} \leq 0.31 \frac{E_{st}}{F_y}$:

$$M_n = \left(\frac{0.021\, E_{st}}{\frac{D}{t}} + F_y\right) S_y \qquad \text{Eq. 8-54}$$

If $\frac{D}{t} > 0.31 \frac{E_{st}}{F_y}$:

$$M_n = f_{cr}\, S_y \qquad \text{Eq. 8-55}$$

In which:

$$f_{cr} = \frac{0.33\, E_{st}}{\frac{D}{t}} \qquad \text{Eq. 8-56}$$

Where:
D = outside diameter of pipe (inch).
t = pipe thickness (inch).
E_s = elastic modulus of steel (ksi).
F_y = yield strength of steel (Table 8-1) (ksi).
M_n = nominal flexural resistance (kip-in).
S_y = elastic section modulus about weak axis (in^3).
f_{cr} = elastic local buckling stress (ksi).

8.5.2.3 Combined Axial Compression and Flexure

Combined axial compression and flexure checks are only applied to pile groups with vertical piles. At this time, AASHTO (2014) does not have a recommendation to include battered piles. For combined compression and flexure of vertical piles, AASHTO (2014) requires the factored structural limit state to satisfy the following limit state checks.

If $\frac{P_u}{P_r} < 0.2$:

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$$\frac{P_u}{2.0\,P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \qquad \text{Eq. 8-57}$$

If $\frac{P_u}{P_r} \geq 0.2$:

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \qquad \text{Eq. 8-58}$$

Where:

P_u = factored load in axial compression (kips).
P_r = factored resistance in axial compression (kips) (Section 8.5.2.1).
M_{ux} = factored moment about x-axis (kip-ft).
M_{rx} = factored resistance in flexure about x-axis (kip-ft) (Section 8.5.2.2).
M_{uy} = factored moment about y-axis (kip-ft).
M_{ry} = factored resistance in flexure about y-axis (kip-ft) (Section 8.5.2.2).

8.5.2.4 Shear

Piles used for bridge foundations are generally not also used to resist high shear loads as significant lateral pile deflections may negatively impact bridge serviceability. For shear loads, the factored structural limit state is taken as:

$$V_r = \varphi_v V_n \qquad \text{Eq. 8-59}$$

Where:

V_r = factored resistance in shear (kips).
φ_v = resistance factor (Table 8-6).
V_n = nominal resistance in shear (kips).

A straightforward calculation of the nominal structural resistance in shear is shown in Equation 8-60 and Equation 8-61 for an H-pile section. Reference should be made to the AASHTO (2014) specifications for additional design requirements concerning piles subject to significant shear loads or for alternative non-composite pile sections.

$$V_n = C\,V_p \qquad \text{Eq. 8-60}$$

$$V_p = 0.58\,F_{yw}\,d_w\,t_w \qquad \text{Eq. 8-61}$$

Where:

V_n = nominal resistance in shear (kips).
C = ratio of shear-buckling resistance to shear yield strength, typically 1.0 for H-piles.
V_p = plastic shear force (kips).
F_{yw} = yield strength of web (Table 8-1) (ksi).
d_w = web depth of pile section (inch).
t_w = web thickness of pile section (inch).

8.5.3 Example Calculations for H Pile Structural Resistance.

The following example provides calculations for an HP 14x117 which will support an integral abutment. This H-pile section is produced with a yield stress, F_y, of 50 ksi, an elastic modulus, E_{st}, of 29,000 ksi while the radius of gyration in the weak direction, r_s, is 3.59 inches (Figure 8-2). Based upon project specific conditions, an unbraced length, l, of 120 inches is assumed for scour. In addition, rotation is fixed while translation is free at the pile head, whereas both rotation and translation are fixed for at the pile toe, thus an effective length factor of 1.2 results (Figure 8-4).

Under the given conditions, the pile section easily satisfies main member slenderness limits (kl/r<120). However a further inspection of local buckling under compression loads is required. First the buckling coefficient is determined.

$$k_c = \frac{4}{\sqrt{\frac{d_w}{t_w}}} = \frac{4}{\sqrt{\frac{14.2\ (inches)}{0.805\ (inches)}}} = 0.952 \qquad \text{[Eq. 8-37]}$$

But, k_c must satisfy:

$$0.35 \leq k_c \leq 0.76$$

Therefore:

$$k_c = 0.76$$

After calculating the buckling coefficient, the following condition is checked.

$$\frac{b_f}{2t_f} \leq 0.64\sqrt{\frac{k_c E_{st}}{F_y}} \qquad \text{[Eq. 8-36]}$$

$$\frac{14.9\,(inches)}{2*0.805\,(inches)} \leq 0.64\sqrt{\frac{0.76*29{,}000\,(ksi)}{50(ksi)}}$$

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$$9.25 \leq 13.44 \quad => \quad Q = 1.0$$

Based upon Equation 8-36, the pile section is a non-slender element, and therefore the slenderness reduction factor Q is set equal to 1.0. The equivalent nominal yield resistance is determined using Equation 8-35.

$$P_o = QF_yA_g \qquad \text{[Eq. 8-35]}$$

$$P_o = 1.0 * 50\ (ksi) * 34.4\ (in^2)$$

$$P_o = 1720\ kips$$

Next, elastic critical buckling resistance in the section is determined.

$$P_e = \frac{\pi^2 E_s A_g}{\left(\frac{Kl}{r_s}\right)^2} \qquad \text{[Eq. 8-39]}$$

$$P_e = \frac{\pi^2 * (29{,}000\ \text{ksi}) * (34.4in^2)}{\left(\frac{(1.2) * (120\ inches)}{(3.59\ inches)}\right)^2}$$

$$P_e = 6120\ kips$$

From Step 3 of Section 8.5.2.1, the nominal resistance in axial compression, P_n, is determined by applying either Equation 8-43 or 8-44. For this example, the ratio of elastic critical buckling resistance, P_e, to yield resistance, P_o, satisfies criteria for Equation 8-43 and is therefore shown in the following calculations.

If $\frac{P_e}{P_o} \geq 0.44$:

$$\frac{P_e}{P_o} = \frac{6120\ kips}{1{,}720\ kips} = 3.56 \geq 0.44$$

Therefore:

$$P_n = P_o\, 0.658^{\frac{P_o}{P_e}} \qquad \text{[Eq. 8-43]}$$

$$P_n = 1{,}720\ kips * 0.658^{\frac{1{,}720\ kips}{6120\ kips}}$$

$$P_n = 1{,}529\ \ kips$$

After calculation of the nominal resistance in axial compression, a resistance factor can be applied to determine the factored structural resistance in axial compression. Because good driving conditions were encountered and pile toe protection was not necessary, a resistance factor, ϕ_c = 0.60, is used (Table 8-6). This factor is applied when only axial compression is considered. When using the combined axial and flexure interaction equations as shown in Section 8.5.2.3, ϕ_c = 0.70 is used.

$$P_r = \varphi_c P_n \qquad \text{[Eq. 8-34]}$$

$$P_r = 0.60 * 1{,}529\ \ kips$$

$$P_r = 917\ kips$$

Continuing with the example, the factored resistance in flexure is determined. To begin, the flange slenderness is inspected for the HP 14x117 pile section. In addition, compact and non-compact flange slenderness ratios are calculated.

Flange slenderness ratio:

$$\lambda_f = \frac{b_f}{2t_f} \qquad \text{[Eq. 8-46]}$$

$$\lambda_f = \frac{14.9\ (inches)}{2*0.805\ (inches)}$$

$$\lambda_f = 9.25$$

Limiting slenderness ratio for a compact flange:

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_{yf}}} \qquad \text{[Eq. 8-47]}$$

$$\lambda_{pf} = 0.38\sqrt{\frac{29{,}000\ (ksi)}{50\ (ksi)}}$$

$$\lambda_{pf} = 9.15$$

Limiting slenderness ratio for a non-compact flange:

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_{yf}}} \qquad \text{[Eq. 8-48]}$$

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$$\lambda_{rf} = 0.83\sqrt{\frac{29{,}000\ (ksi)}{50\ (ksi)}}$$

$$\lambda_{rf} = 19.98$$

After determining the slenderness ratio functions, a comparison is then made. For this particular pile, Equation 8-48 will be used to define the nominal flexural resistance. Reference should be made to Figure 8-2 for H-pile section properties such as the elastic and plastic section moduli.

If $\lambda_{pf} < \lambda_f \leq \lambda_{rf}$:

$$9.15 < 9.25 \leq 19.98$$

Therefore:

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_{yf}}}}\right)\right] F_{yf} Z_y \qquad \text{[Eq. 8-55]}$$

$$M_n = \left[1 - \left(1 - \frac{59.5\ (\text{in}^3)}{91.4\ (\text{in}^3)}\right)\left(\frac{9.25 - 9.15}{0.45\sqrt{\frac{29{,}000\ (ksi)}{50\ (ksi)}}}\right)\right] 50\ (ksi) * 91.4(\text{in}^3)$$

$$M_n = 4555\ kip - in$$

Following the calculation of nominal resistance in flexure, a resistance factor can be applied to determine the factored structural resistance in flexure. The AASHTO recommended resistance factor for flexure is, ϕ_f = 1.0, is used (Table 8-6).

$$M_r = \varphi_f M_n \qquad \text{[Eq. 8-41]}$$

$$M_r = 1.0 * 4555\ kip - in$$

$$M_r = 4555\ kip - in$$

Finally, the nominal resistance in shear of the pile section is determined using Equations 8-60 and 8-61. For the HP 14x117 pile section, the web depth is 14.2 inches and the web thickness is 0.805 inches (Figure 8-2). The plastic shear force is calculated using Equation 8-61.

$$V_p = 0.58\ F_{yw}\ d_w\ t_w \qquad \text{[Eq. 8-61]}$$

$$V_p = 0.58 * (50\ ksi) * (14.2\ inches) * (0.805\ inches)$$

$$V_p = 332\ kips$$

The nominal structural resistance in shear is calculated using Equation 8-60.

$$V_n = C\ V_p \qquad \text{[Eq. 8-60]}$$

$$V_n = (1.0) * (332\ kips)$$

$$V_n = 332\ kips$$

By multiplying the nominal structural resistance in shear by the AASHTO (2014) recommended resistance factor, the factored structural resistance in shear is determined.

$$V_r = \varphi_v V_n \qquad \text{[Eq. 8-59]}$$

$$V_r = (1.0) * (332\ kips)$$

$$V_r = 332\ kips$$

Neglect concrete in tension or flexure

Allowable Axial Stresses in Driven Piles

Table 9-11. Maximum allowable stresses in pile for top driven piles (after AASHTO, 2002; FHWA, 2006a)

Pile Type	Maximum Allowable Stresses (f_y = yield stress of steel; f'_c = 28-day compressive strength of concrete; f_{pe} = pile prestress)
Steel H-Piles	**Design Stress** $0.25\ f_y$ $0.33\ f_y$ If damage is unlikely, and confirming static and/or dynamic load tests are performed and evaluated by engineer. **Driving Stress** $0.9\ f_y$
	32.4 ksi (223 MPa) for ASTM A-36 (f_y = 36 ksi; 248 MPa) 45.0 ksi (310 MPa) for ASTM A-572 or A-690, (f_y = 50 ksi; 345 MPa)
Unfilled Steel Pipe Piles	**Design Stress** $0.25\ f_y$ $0.33\ f_y$ If damage is unlikely, and confirming static and/or dynamic load tests are performed and evaluated by engineer. **Driving Stress** $0.9\ f_y$
	27.0 ksi (186 MPa) for ASTM A-252, Grade 1 (f_y = 30 ksi; 207 MPa) 31.5 ksi (217 MPa) for ASTM A-252, Grade 2 (f_y = 35 ksi; 241 MPa) 40.5 ksi (279 MPa) for ASTM A-252, Grade 3 (f_y = 45 ksi ; 310 MPa)
Concrete filled steel pipe piles	**Design Stress** $0.25\ f_y$ (on steel area) ***plus*** $0.40\ f'_c$ (on concrete area) **Driving Stress** $0.9\ f_y$
	27.0 ksi (186 MPa) for ASTM A-252, Grade 1 (f_y = 30 ksi; 207 MPa) 31.5 ksi (217 MPa) for ASTM A-252, Grade 2 (f_y = 35 ksi; 241 MPa) 40.5 ksi (279 MPa) for ASTM A-252, Grade 3 (f_y = 45 ksi ; 310 MPa)
Precast Prestressed Concrete Piles	**Design Stress** $0.33\ f'_c - 0.27\ f_{pe}$ (on gross concrete area) ; f'_c minimum of 5.0 ksi (34.5 MPa) f_{pe} generally > 0.7 ksi (5 MPa)
	Driving Stress Compression Limit $< 0.85\ f'_c - f_{pe}$ (on gross concrete area) Tension Limit (1) $< 3\ (f'_c)^{1/2} + f_{pe}$ (on gross concrete area) **US Units*** $< 0.25\ (f'_c)^{1/2} + f_{pe}$ (on gross concrete area) **SI Units** * Tension Limit (2) $< f_{pe}$ (on gross concrete area) (1) - Normal Environments ; (2) - Severe Corrosive Environments ***Note:** f'_c and f_{pe} must be in psi and MPa for US and SI equations, respectively.
Conventionally reinforced concrete piles	**Design Stress** $0.33\ f'_c$ (on gross concrete area) ; f'_c minimum of 5.0 ksi (34.5 MPa)
	Driving Stress Compression Limit $< 0.85\ f'_c$; Tension Limit $< 0.70\ f_y$ (of steel reinforcement)
Timber Pile	**Design Stress** 0.8 to 1.2 ksi (5.5 to 8.3 MPa) for pile toe area depending upon species
	Driving Stress Compression Limit $< 3\ \sigma_a$ Tension Limit $< 3\ \sigma_a$ σ_a - AASHTO allowable working stress

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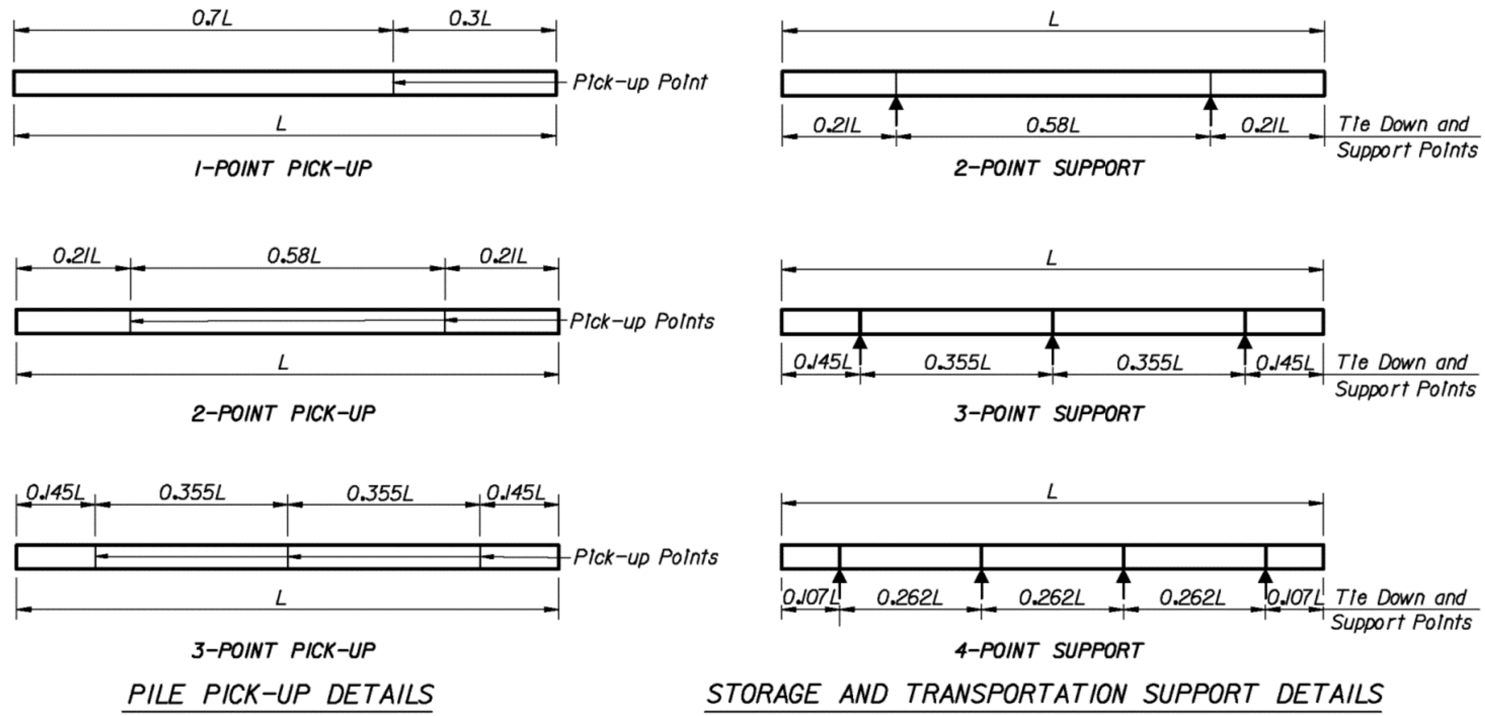
Handling Stresses Concrete Piles

Two-Point Pickup of Concrete Pile

One-Point Pile Pickup

Concrete Pile Pick-up Configurations (FL DOT)



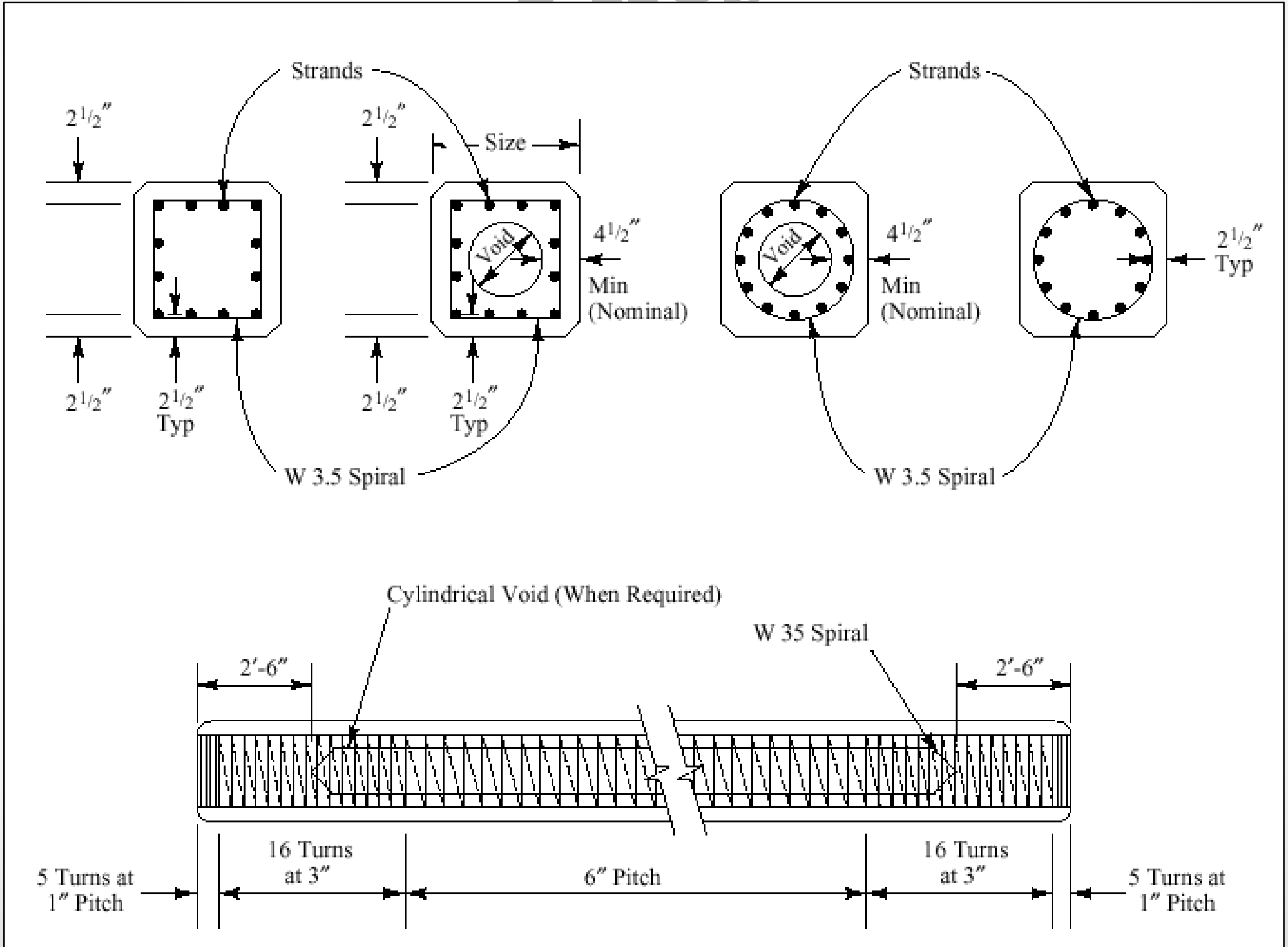
Maximum Allowable Handling Stress :

$$F_b = 6\sqrt{f_c'} + f_{pc} \text{ (US units, } f_c' \text{ in psi)}$$

$$F_b = \frac{1}{2}\sqrt{f_c'} + f_{pc} \left(\text{SI units, } f_c' \text{ in MPa}\right)$$

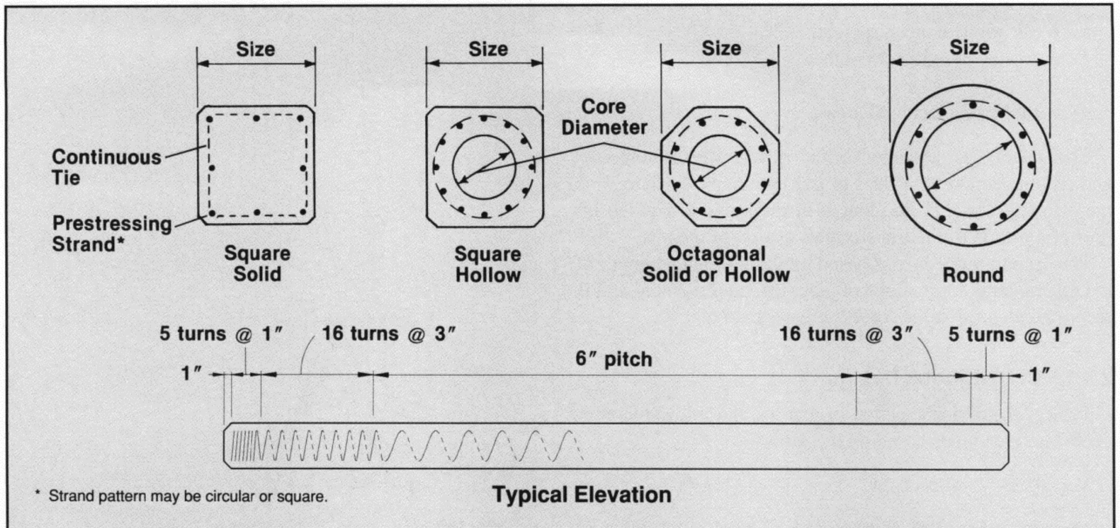
TABLE OF MAXIMUM PILE PICK-UP AND SUPPORT LENGTHS								
	D = Square Pile Size (Inches)						Required Storage and Transportation Detail	Pick-Up Detail
	12	14	18	20	24	30		
Maximum Pile Length (Feet)	48	52	59	62	68	87	2, 3, or 4 point	1 Point
	69	75	85	89	98	124	2, 3, or 4 point	2 Point
	99	107	121	128	140	178	3 or 4 point	3 Point

Prestressed Concrete Piles



Prestressed Concrete Piles

Table 2.4. Section properties and allowable service loads of prestressed concrete piles.



Size (in.)	Core Diameter (in.)	Section Properties[1]: Area (in.^{2})	Weight (plf)	Moment of Inertia (in.^{4})	Section Modulus (in.^{3})	Radius of Gyration (in.)	Perimeter (ft)	Allowable Concentric Service Load, Tons[2], f'_c: 5,000	6,000	7,000	8,000
Square Piles											
10	Solid	100	104	833	167	2.89	3.33	73	89	106	122
12	Solid	144	150	1,728	288	3.46	4.00	105	129	152	176
14	Solid	196	204	3,201	457	4.04	4.67	143	175	208	240
16	Solid	256	267	5,461	683	4.62	5.33	187	229	271	314
18	Solid	324	338	8,748	972	5.20	6.00	236	290	344	397
20	Solid	400	417	13,333	1,333	5.77	6.67	292	358	424	490
20	11 in.	305	318	12,615	1,262	6.43	6.67	222	273	323	373
24	Solid	576	600	27,648	2,304	6.93	8.00	420	515	610	705
24	12 in.	463	482	26,630	2,219	7.58	8.00	338	414	491	567
24	14 in.	422	439	25,762	2,147	7.81	8.00	308	377	447	517
24	15 in.	399	415	25,163	2,097	7.94	8.00	291	357	423	488
30	18 in.	646	672	62,347	4,157	9.82	10.00	471	578	685	791
36	18 in.	1,042	1,085	134,815	7,490	11.38	12.00	761	933	1,105	1,276
Octagonal Piles											
10	Solid	83	85	555	111	2.59	2.76	60	74	88	101
12	Solid	119	125	1,134	189	3.09	3.31	86	106	126	145
14	Solid	162	169	2,105	301	3.60	3.87	118	145	172	198
16	Solid	212	220	3,592	449	4.12	4.42	154	189	224	259
18	Solid	268	280	5,705	639	4.61	4.97	195	240	284	328
20	Solid	331	345	8,770	877	5.15	5.52	241	296	351	405
20	11 in.	236	245	8,050	805	5.84	5.52	172	211	250	289
22	Solid	401	420	12,837	1,167	5.66	6.08	292	359	425	491
22	13 in.	268	280	11,440	1,040	6.53	6.08	195	240	283	328
24	Solid	477	495	18,180	1,515	6.17	6.63	348	427	506	584
24	15 in.	300	315	15,696	1,308	7.23	6.63	219	268	318	368
Round Piles											
36	26 in.	487	507	60,007	3,334	11.10	9.43	355	436	516	596
42	32 in.	581	605	101,273	4,823	13.20	11.00	424	520	616	712
48	38 in.	675	703	158,222	6,592	15.31	12.57	493	604	715	827
54	44 in.	770	802	233,373	8,643	17.41	14.14	562	689	816	943
66	54 in.	1,131	1,178	514,027	15,577	21.32	17.28	826	1,013	1,199	1,386

(1) Form dimensions may vary with producers, with corresponding variations in section properties.
(2) Allowable loads based on $N = A_c\,(0.33\,f'_c - 0.27\,f_{pc})$; f_{pc} = 700 psi. Check local producer for available concrete strengths.

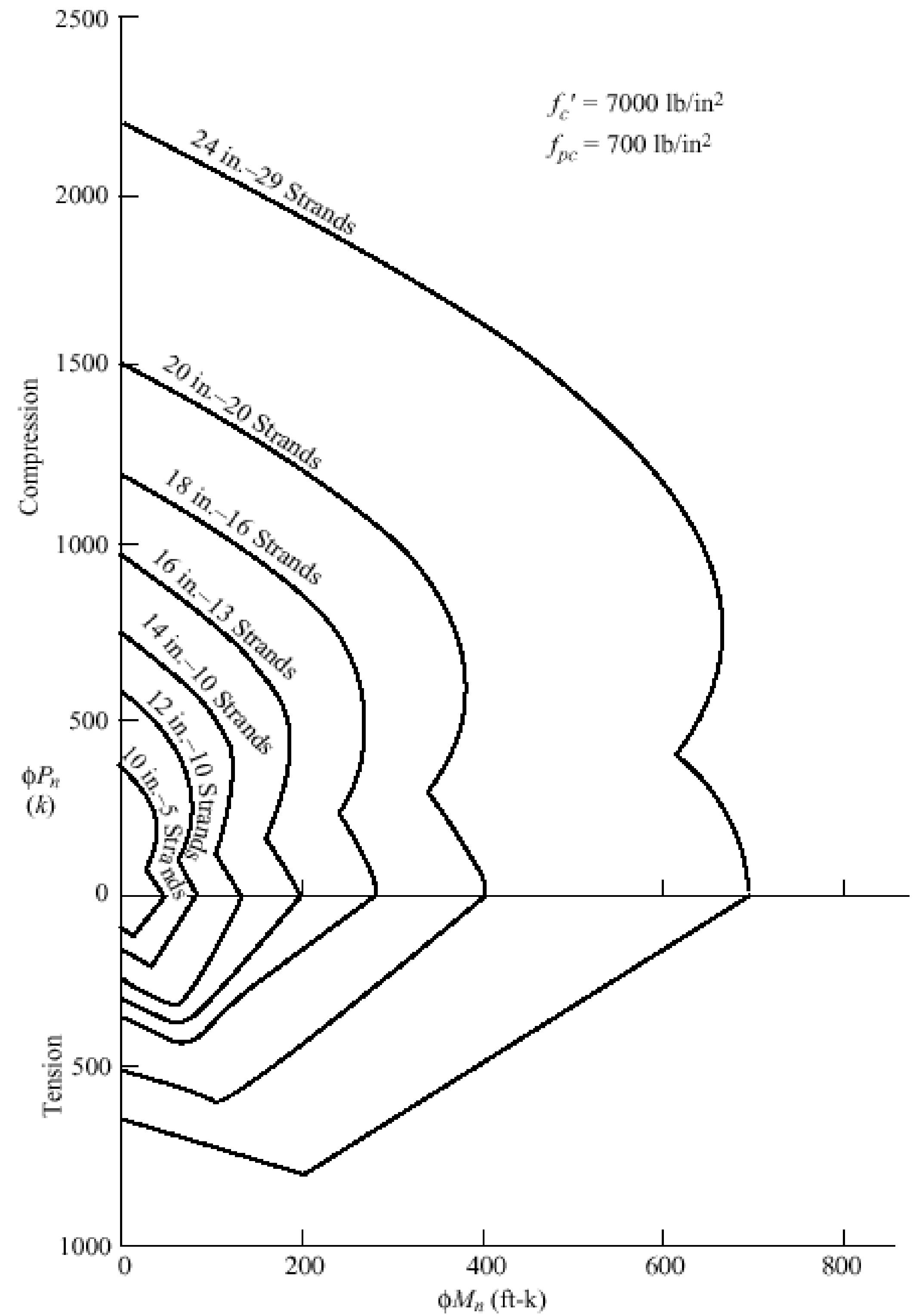
Prestressed Concrete Piles

- AASHTO allowable compressive stress on concrete piles

$$F_a = 0.33\ f'_c - 0.27\ f_{pc}$$

  - F_a = allowable compressive stress in concrete due to axial load
  - f'_c = 28-day compressive strength of concrete
  - f_{pc} = effective prestress on the gross section
- Prestressed concrete piles specifically excluded from ACI 318-99 code
- Code references ACI 543-74, PCI and "general building code"
  - Surprisingly vague from code point of view
- Design Procedure
  - Use code requirements for normal prestressed methods
  - Check against AASHTO equations
  - Use PCI influence diagrams

Influence Diagram for Square Prestressed Concrete Piles



Prestressed Concrete Pile Example

- Given
  - Square Prestressed Concrete Pile
    - Axial Compression
      - Dead Load = 100 kips
      - Live Load = 120 kips
    - Moment
      - Dead Load = 50 ft-kips
      - Live Load = 20 ft-kips
- Find
  - Pile size that will satisfy these loads
- Step 1: Compute unfactored compressive load acting on pile
  - As was the case with shallow foundations, we will restrict ourselves to foundations with dead and live loads only, thus U = D + L for ASD unfactored loads
  - U = 100 + 120 = 220 kips

Prestressed Concrete Pile Example

- Step 2: Using concrete pile tables, select minimum pile size for axial compression
  - $f'_c = 7000$ psi
  - $f_{pc} = 700$ psi

$$F_a = 0.33 \times 7000 - 0.27 \times 700$$

$$F_a = 2121 \quad psi$$

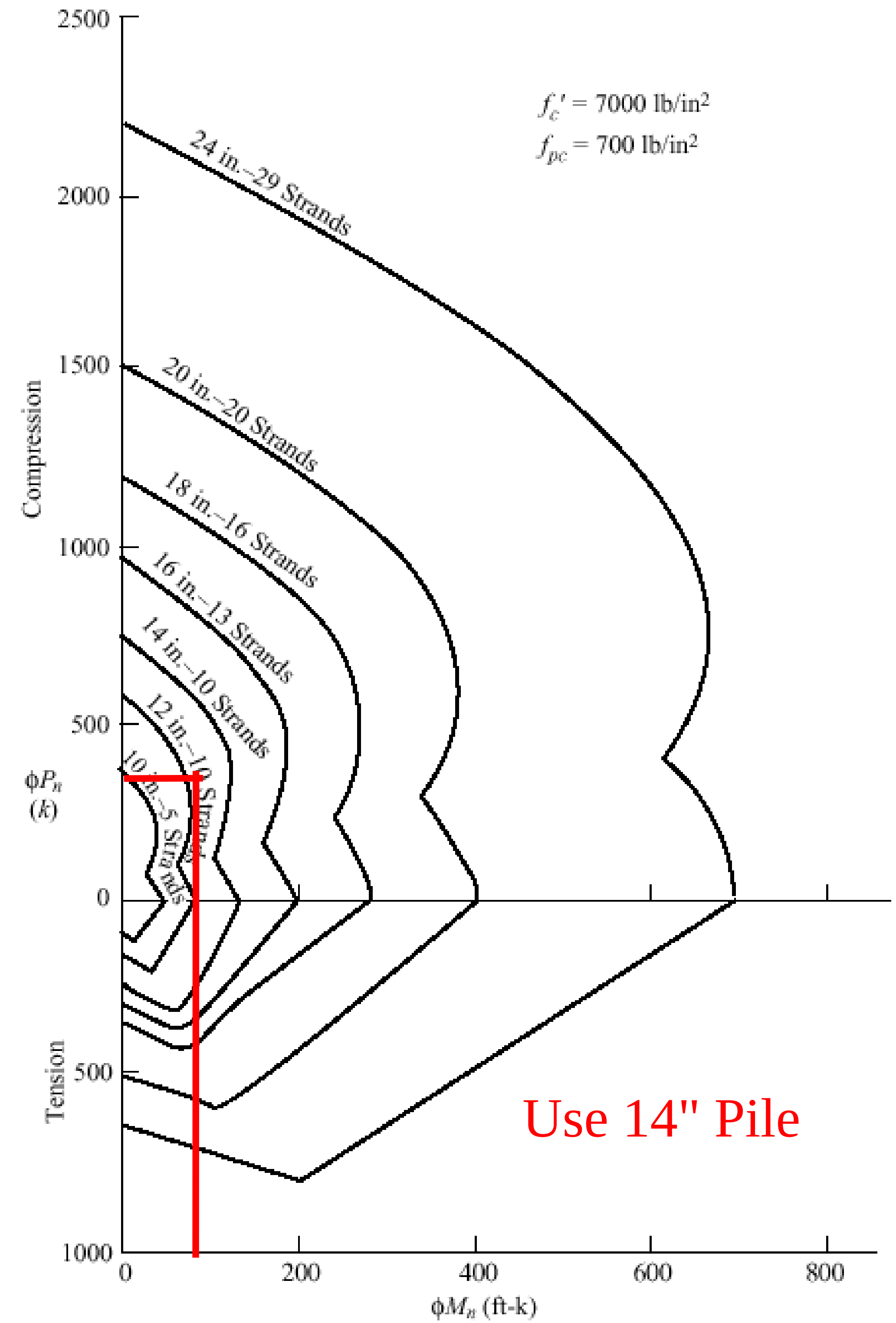
$$A = \frac{P}{F_a} = \frac{220000}{2121} = 103.7 \quad in^2$$

  - Since smallest pile with this area is 12" square (144 in^2), we select 12" square prestressed concrete pile

- Step 3: Compute ACI factored compressive, tensile and moment loads
  - U = 1.2D + 1.6L
  - P_u = (1.2)(100) + (1.6)(120) = 312 kips
  - M_u = (1.2)(50) + (1.6)(20) = 92 ft-kips

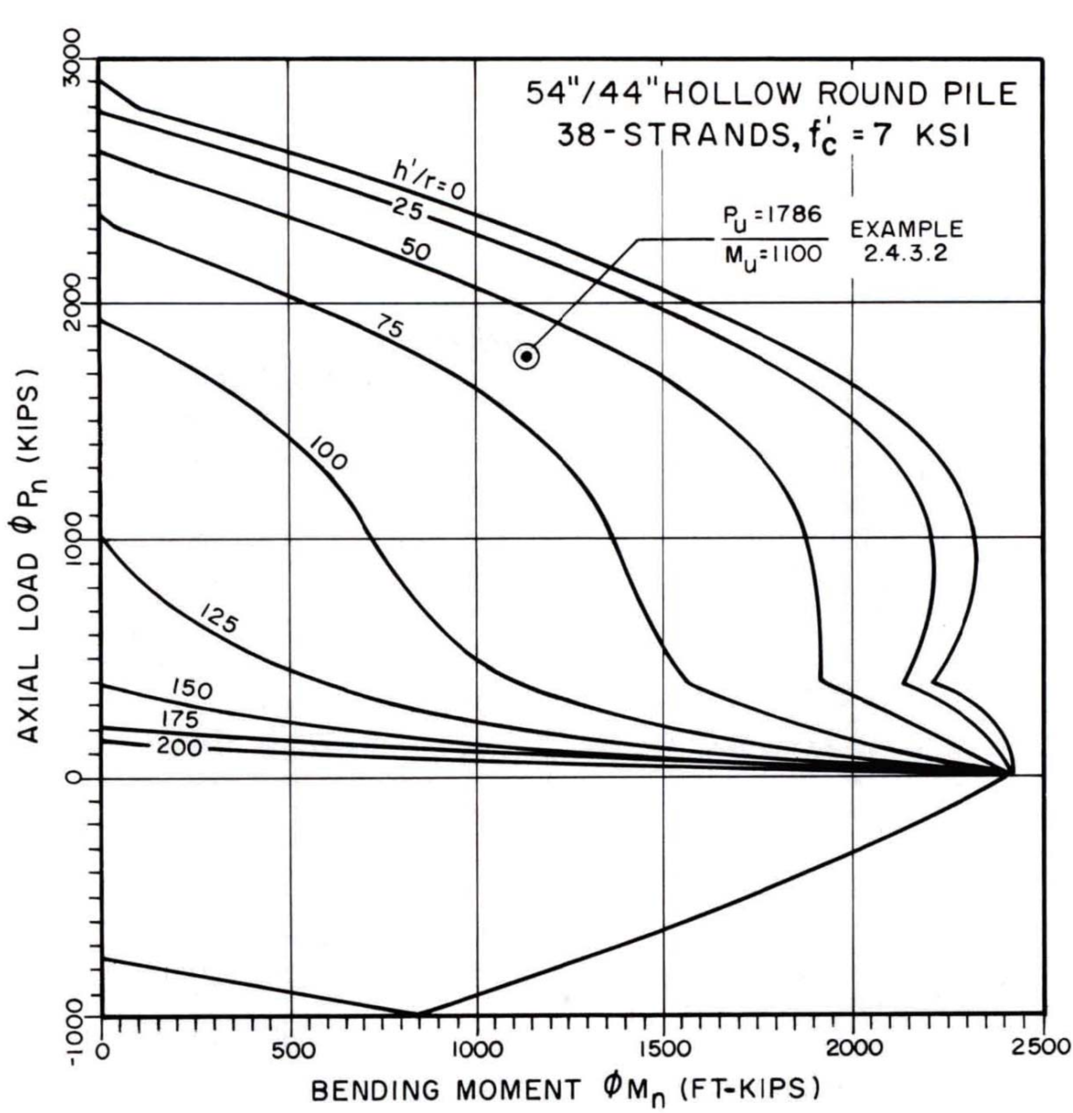
Prestressed Concrete Pile Example

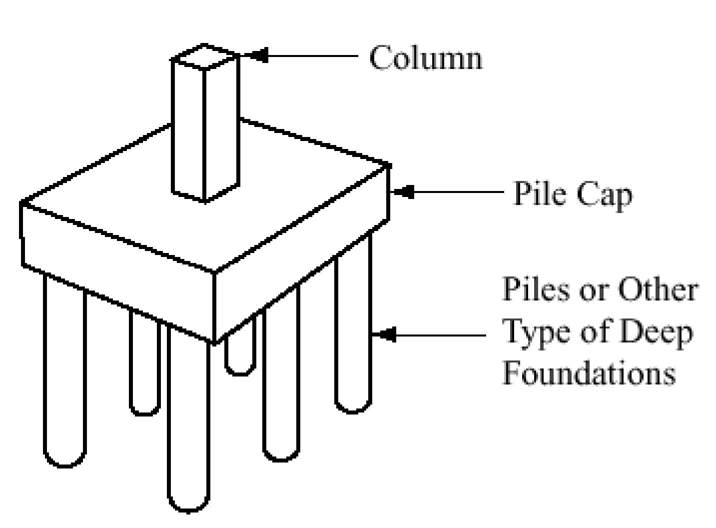
- Step 4: Use interaction diagram to determine pile size
  - $P_u = (1.2)(100) + (1.6)(120) = 312$ kips
  - $M_u = (1.2)(50) + (1.6)(20) = 92$ ft-kips
-



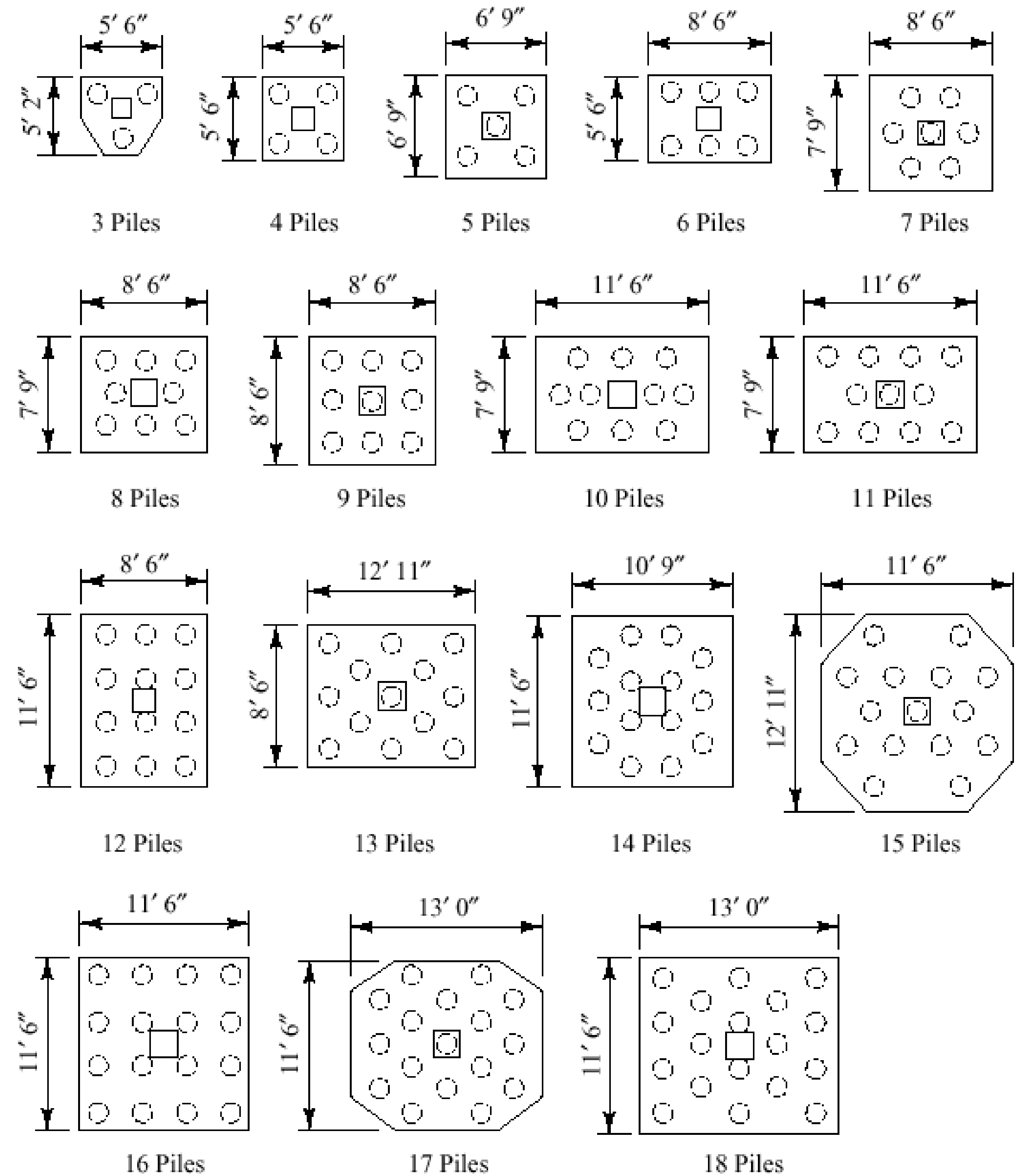
Influence Diagram for 54" Cylinder Piles

- Previous influence diagram only covers square concrete piles which are fully embedded into the soil
- Diagram at the right is for 54" OD 5" wall cylinder piles
  - Only difference in use is the h'/r ratio
    - h' = length of pile protruding above groundline
    - r' = radius of gyration of pile (given in pile table)




Column
Pile Cap
Piles or Other Type of Deep Foundations

Pile Caps


5′ 6″
5′ 2″
3 Piles
5′ 6″
5′ 6″
4 Piles
6′ 9″
6′ 9″
5 Piles
8′ 6″
5′ 6″
6 Piles
8′ 6″
7′ 9″
7 Piles
8′ 6″
7′ 9″
8 Piles
8′ 6″
8′ 6″
9 Piles
11′ 6″
7′ 9″
10 Piles
11′ 6″
7′ 9″
11 Piles
8′ 6″
11′ 6″
12 Piles
12′ 11″
8′ 6″
13 Piles
10′ 9″
11′ 6″
14 Piles
11′ 6″
12′ 11″
15 Piles
11′ 6″
11′ 6″
16 Piles
13′ 0″
11′ 6″
17 Piles
13′ 0″
11′ 6″
18 Piles

Pile Caps

- Requirements for Pile Caps
  - Same as spread footings with the following additions
    - Design must satisfy the punching shear in the vicinity of the individual piles or shafts
    - The effective depth d must be at least 300 mm (12"). This implies a minimum thickness T of 400 mm (18").
    - The bearing force between the individual piles or shafts and the caps must not exceed the capacity of either element
- Driven piles are almost inevitably placed in groups
  - Drilled shafts can be placed this way, but usually are not
- The reinforced concrete element that connects the column with its multiple deep foundations is called a cap or a pile cap
- Design process is similar for spread footings
  - Loads acting on caps are larger than spread footing loads
  - Loads on cap are distributed over a small portion of the bottom of the footing

Grade Beams

- Deep foundations are sometimes connected with grade beams
- Grade beams are required for all deep foundations subject to seismic loads
  - For seismic design, they must resist a horizontal load equal to 10% of the column vertical load
- Grade beams must be designed without the support of the underlying soil

Questions?