II/11

Design Examples—Steel Bearing Piles

Steel bearing pile design is discussed, and procedures and formulas are given in the United States Steel Corporation's "Highway Structures Design Manual," Volume I, Chapter 10. The following examples illustrate the design of steel bearing pile foundations most used for highway structures. Five examples are given:

1. Stub Abutment for Short Span Steel Bridges
2. Pile Bent
3. Cantilever Abutment (design procedure also typical for retaining wall)
4. Bridge Pier on Land
5. Bridge Pier in Water (with Tremie Concrete Seal)

Various soil conditions are assumed to illustrate the design of friction piles in fine grained material, friction piles in coarse grained material, and piles bearing on rock.

A single row of piles is used in Examples 1 and 2; larger pile groups are used in Examples 3, 4 and 5.

The design forces for stub abutments are dependent upon the arrangement of spans, span lengths, type of bearing device for the roadway stringers, temperature variation and type of soil. Abutments with shallow backwalls generally have the capability to move as the roadway stringers expand or contract with temperature changes. Since these movements are generally small for short span bridges, the longitudinal force induced in the pile due to movement is small and within the allowable lateral pile load. Thus it is desirable to "fix" the roadway stringers to both abutments for relatively short spans and carry all longitudinal forces due to wind, traction and temperature changes to the abutments. As the span length of the superstructure increases, the temperature movement to be accommodated by the abutment and piles increase. Excessive lateral forces and excessive bending stresses may be induced in the piles. When the length between abutments exceeds about 250′ to 300′, it is desirable to provide expansion bearings on both abutments with traction and longitudinal wind forces carried by the intermediate pile bents. The only longitudinal force carried by the abutment is due to dead load friction in the expansion devices. Example 1 assumes that the thermal movement of the stringers exceeds the permissible movement of the abutment and piles and, that expansion type bearings are used on both abutments.

Since pile bents are relatively flexible, movement due to temperature changes will cause only small loads and moment in the piles. Where fixed bearings are used on both abutments, the only longitudinal force on pile bents is caused by temperature changes. Example 2 illustrates this condition.

Where expansion bearings are used on the abutments and the roadway stringers are fixed to each pile bent, the pile bents must be designed for traction, longitudinal wind and temperature changes.

If a series of continuous spans are supported by pile bents, longitudinal forces due to traction, wind and temperature changes must be carried by the pile bents.

The following general method of design is recommended for Example 1:

1. Determine the vertical and horizontal load per pile. If one pile is placed under each stringer, the dead load and live load to each pile is equal to the stringer reaction plus a portion of the weight of the pile cap.

2. Choose the pile size and determine the depth of penetration required to develop the axial load in the pile.

3. Determine the allowable lateral load per pile and compare to the actual lateral load per pile. The allowable lateral load per pile generally is controlled by the permissible lateral movement of the pile at the ground surface. If the actual lateral load exceeds the allowable; increase the pile size, increase the number of piles, or batter piles.

4. Check settlement of the piles if critical to design of the structure.

The following general method of design is recommended for Example 2:

1. Determine the vertical and horizontal load per pile. If one pile is placed under each stringer, the dead load and live load to each pile is equal to the stringer reaction plus a portion of the weight of the pile cap.

Determine the horizontal loads due to wind, traction and centrifugal force and resolve into components parallel to the row of piles (H_x) and normal to the row of piles (H_y) as shown in Figure A. Determine the vertical load per pile by taking moments about the top of piles and applying the following equation:

$$Q_m = \frac{F_v}{r} \pm \frac{M_y x}{\Sigma x^2} \qquad \text{(Eq. A)}$$

where Q_m = vertical load on any given pile m

F_v = vertical load due to dead load and live load

M_y = moment with respect to the Y axis through the centroid of the pile group

x = distance of pile from Y axis

r = number of piles

(For definition of above terms, see Figure A)

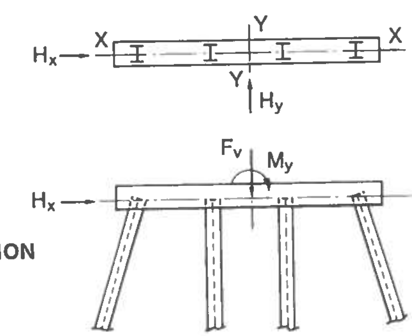


FIGURE A—DEFINITION OF TERMS USED FOR CALCULATION OF PILE LOADS FOR PILE BENTS

If piles are battered parallel to the row of piles, determine the amount of horizontal load carried by the batter and subtract this from the total horizontal load parallel to the row of piles. The remaining horizontal load is divided equally among the piles and is carried by lateral bearing of the piles against the soil.

2. Tentatively choose the pile size and determine the depth of penetration required to develop the axial load in the pile. If the pile extends above the ground surface, combined stresses due to axial load and bending may govern the pile size.

3. Determine the depth below ground surface at which the piles may be considered fixed.

4. Determine the allowable compressive stress in the pile based upon the appropriate column formula.

5. Determine the longitudinal force on the piles due to temperature change in the superstructure.

6. Determine the maximum stresses in the pile due to axial load plus bending.

7. Check settlement of the piles if critical to design of the structure.

In Examples 3, 4 and 5, groups of piles are embedded in rigid concrete footings. The following steps are recommended for the design of such a foundation:

1. Determine loads on the pile group at the elevation of the top of piles. Vertical load, moments about the two perpendicular axes passing through the centroid of the pile group, and horizontal shears parallel to these two axes are required. The vertical load, moments and shears often are calculated at the top of footing as part of the pier design. The designer then transfers these to the elevation of the top of piles by increasing the vertical load by the weight of footing plus earth cover, and changing each moment by the pertinent horizontal shear multiplied by the depth from the top of footing to the top of piles.
2. Tentatively choose the pile size and allowable load. If friction piles are used, determine the length of embedment required to develop the allowable load. If the soil quality is sufficient to develop high pile capacities, greatest economy is obtained with a smaller number of large piles.
3. Tentatively choose a footing size and pile pattern based upon past designs and the relationship of moments to vertical loads. For example, if moments on the pile group are small, the number of piles will be approximately equal to the vertical load divided by the allowable pile load; if moments on the pile group are large, the number of piles may be greater than two times the vertical load divided by the allowable pile load, and it may be necessary to use a large spacing between piles or to concentrate piles near the periphery of the footing so as to increase resistance to overturning moments.
4. Determine the maximum pile load for the trial pile group based upon the following assumptions:
   (a) The footing is perfectly rigid.
   (b) The tops of piles are hinged to the footing so that no bending moment is transferred from the footing to a pile.
   (c) Piles are short elastic columns so that deformations and the stress distribution are linear.

These assumptions permit the use of the following elastic equation for the calculation of pile loads: (See Figure B)

$$Q_m = \frac{F_v}{r} \pm \frac{M_y x}{\Sigma x^2} \pm \frac{M_x y}{\Sigma y^2} \qquad \text{(Eq. B)}$$

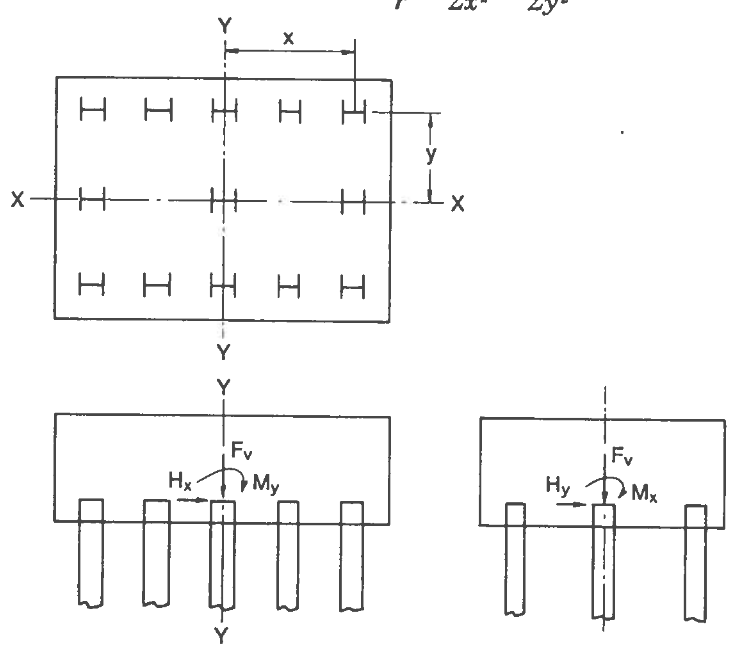


FIGURE B—DEFINITION OF TERMS USED FOR CALCULATION OF PILE LOADS FOR PILE GROUP SUBJECTED TO VERTICAL LOAD PLUS MOMENT where Q_m = vertical load on any given pile m

F_v = total vertical load acting at the centroid of the pile group

r = number of piles

M_x = moment with respect to the X axis through the centroid of the pile group

M_y = moment with respect to the Y axis through the centroid of the pile group

x = distance of pile from Y axis

y = distance of pile from X axis

(For definition of above terms, see Figure B)

5. If the maximum pile load determined in Step 4 differs from the allowable pile load, change the number of piles and/or the arrangement of the piles, and if necessary change the size of the footing. In some cases, it may be necessary to change the size of pile. Recalculate the maximum pile load and adjust the pile pattern until the maximum pile load is approximately equal to the allowable pile load.

6. Determine the lateral load capacity of the pile group and compare to the actual horizontal loads applied to the pile group.

7. If the actual horizontal loads exceed the lateral load capacity of the pile group, batter sufficient piles so that the lateral load capacity of the pile group plus the horizontal component of the battered piles equals or exceeds the actual horizontal loads.

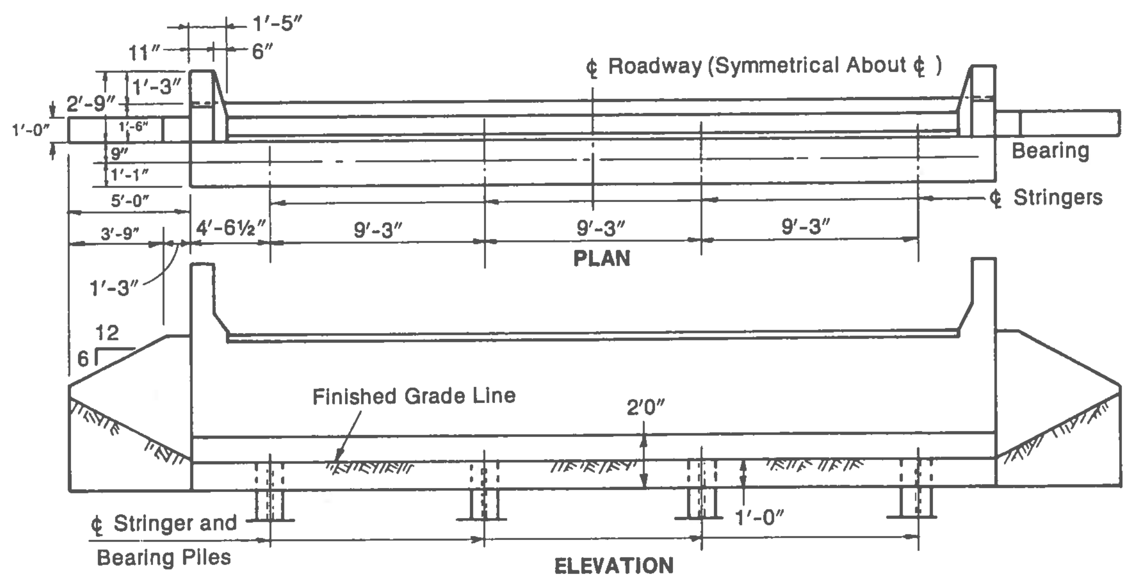
8. When designing friction piles in fine grained materials, determine if the allowable axial load per pile is reduced due to group action.

9. Check settlement of the pile group if critical to design of the structure.

Equations, figures and tables referred to in the examples may be found in the United States Steel Corporation's "Highway Structures Design Manual," Volume I, Chapter 10.

Example 1—Stub Abutment for Short Span Steel Bridges

This type of abutment is applicable for short span bridges with a shallow backwall. A single row of vertical piles is used with one pile beneath each stringer. The pile design generally is governed by the lateral displacement of the piles. If horizontal loads are large or the soil quality is poor, it may be necessary to batter some piles, to increase the pile size or to increase the number of piles. Piles for this abutment are designed as friction piles in coarse grained soil (Part A) and as friction piles in fine grained soil (Part B).



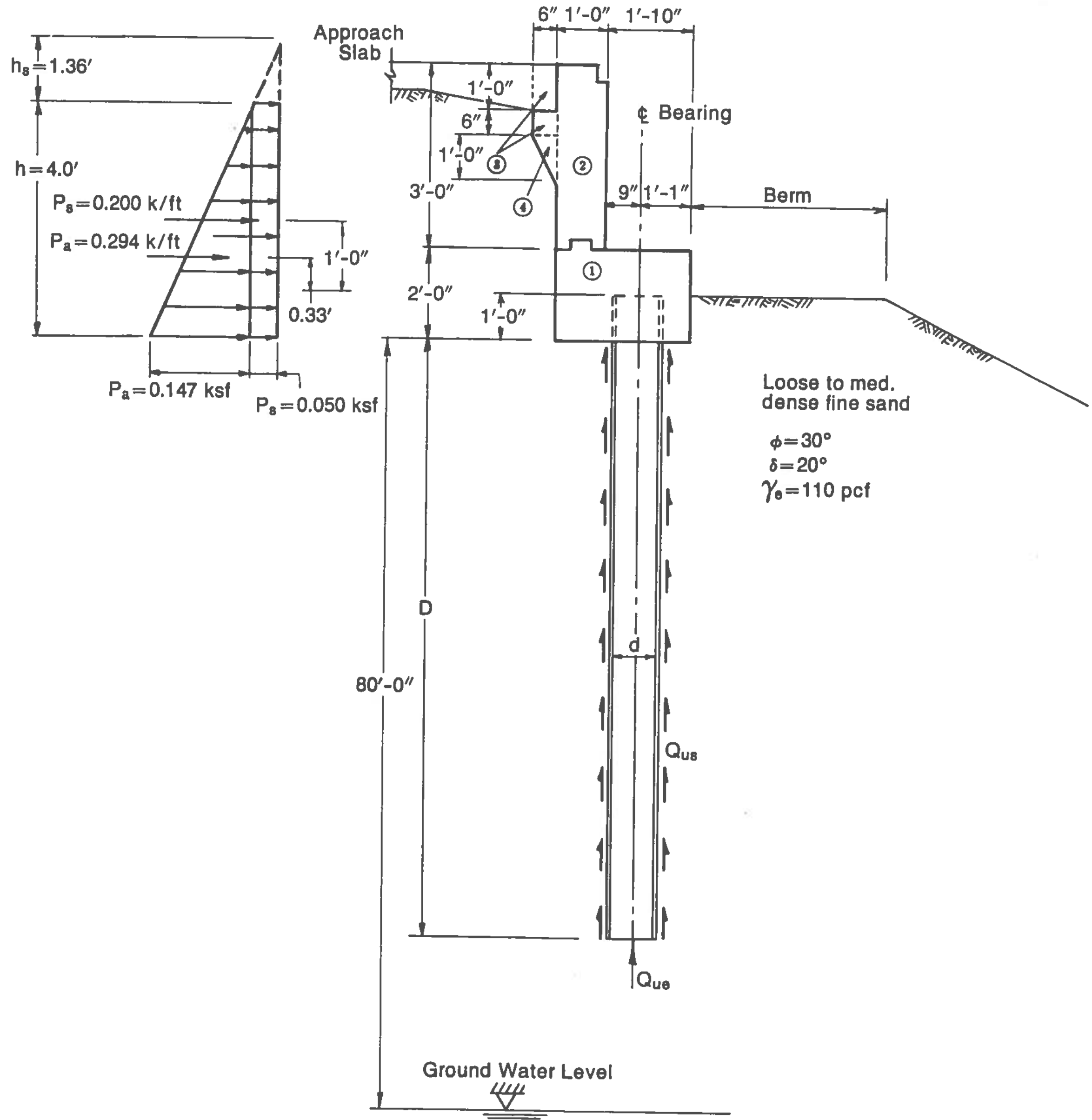
STRINGER REACTIONS

The maximum stringer reactions are assumed to be:

Dead load = 36 kips
Live load = 53 kips

In accordance with AASHO, Section 1.2.12, impact is not applied to piles below the ground surface.

Since the bearings are an expansion type, the maximum horizontal force that can be applied to the abutment by the stringers is equal to the horizontal force necessary to overcome friction in the bearings. This friction force varies with the type of expansion bearing. For this example, assume that the friction force is 10% of the dead load reaction.



SECTION

PART A—SINGLE ROW OF PILES IN COARSE GRAINED MATERIAL

Abutment Dead Load (Interior Pile)

Determine vertical dead load force and dead load moment for the interior piles spaced at 9′3″ center to center.

Loading	F_v (kips)	Arm about ℄ Brg. (ft)		Moment about ℄ Brg (kip-ft)	
		Left	Right	↶	↷
① 2.0×2.83×0.15×9.25	7.85	0.33	—	2.59	—
② 1.0×3.0 ×0.15×9.25	.4.16	1.25	—	5.20	—
③ 1.5×0.5 ×0.15×9.25	1.05	2.00	—	2.10	—
④ 1.0×0.5 ×½×0.15×9.25	0.35	1.92	—	0.68	—
Total	13.41			10.57	—

EARTH PRESSURE

Assume soil properties of the abutment backfill to be the same as the original soil properties.

Active earth pressure, $P_a = K_a \gamma_e h$

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30°}{1+\sin 30°} = \frac{1-0.500}{1+0.500}$$

$$K_a = 0.333$$

$$P_a = 110 \text{ pcf} \times 0.333 \times 4.0 = 147 \text{ psf}$$
$$= 0.147 \text{ ksf}$$

$$P_a = 0.147 \frac{(4.0)}{2} = 0.294 \text{ k/ft}$$

Surcharge Load

Section 1.2.19 of the AASHO specifications states that live load surcharge is not considered where an adequately designed reinforced concrete approach slab is provided. However, the weight of the slab is supported by the backfill and is considered as dead load surcharge. This load is equated to an equivalent height of soil.

$$h_s = \frac{\text{wt. of approach slab}}{\text{wt. of soil}} = \frac{1.0 \times 0.15}{0.11} = 1.36 \text{ ft}$$

$$\therefore P_s = 110 \times 0.333 \times 1.36 = 49.8 \text{ psf} \cong 0.050 \text{ ksf}$$

$$\text{and } P_s = 0.050 \times 4.0 = 0.200 \text{ k/ft}$$

Earth Pressure for Interior Piles at 9′-3″ Center to Center
Forces about Top of Piles

Force	H (kips)	Arm (ft)	M (kip-ft)
Earth Pressure—0.294×9.25	2.72	0.33	+0.91↷
Surcharge —0.200×9.25	1.85	1.0	+1.85↷
Total	4.57		+2.76↷

Passive soil resistance in front of the abutment should not be considered.

FORCES AT TOP OF INTERIOR PILES

Loads for AASHO Groups I and III are tabulated. The design of stub abutments is governed by Group I loading for axial pile capacity and Group III loading for lateral pile capacity.

Group I—Dead Load, Live Load and Earth Pressure

Loading	F_v (kips)	Transverse Direction	
		H (kips)	M (kip-ft)
D.L. Superstructure	36.0	—	—
D.L. Abutment	13.4	—	−10.57
L.L.	53.0	—	—
Earth Pressure	—	4.57	+ 2.76
Total	102.4	4.57	− 7.81

Group III—Dead Load, Live Load, Earth Pressure and Friction

Loading	F_v (kips)	Transverse Direction	
		H (kips)	M (kip-ft)
D.L. Superstructure	36.0	—	—
D.L. Abutment	13.4	—	−10.57
L.L.	53.0	—	—
Earth Pressure	—	4.57	+ 2.76
Friction	—	3.60	+ 3.60
Total	102.4	8.17	− 4.21

PILE DESIGN

1. Size

Piles of relatively low capacity generally are sufficient to support pile cap abutments. However, for a given load per pile, a pile with greater perimeter and greater end area will develop the load with less depth of penetration.

For example, compare an HP 10×42 pile with an HP 8×36 pile. The HP 10×42 pile has approximately 25% more perimeter and 56% more end area with an increase of only 17% in weight. The HP 10×42 pile also offers greater resistance to lateral loads.

Use HP 10×42 piles for this design example.

Pile Properties

$$A_s = 12.4 \text{ in.}^2 = 0.086 \text{ ft}^2$$
$$d = 9.72 \text{ in.} = 0.810 \text{ ft}$$
$$b_f = 10.078 \text{ in.} = 0.839 \text{ ft}$$
$$P = 2(d + b_f) = 3.30 \text{ ft}$$
$$A = db_f = 0.680 \text{ ft}^2$$
$$S_x = 43.4 \text{ in.}^3$$
$$I_x = 211 \text{ in.}^4$$

2. Depth of penetration

The ultimate bearing capacity of a pile (Q_u) is equal to the sum of the end bearing (Q_{ue}) plus the skin friction (Q_{us}).

$$Q_u = Q_{ue} + Q_{us}$$

a. End Bearing Component

Assume that the depth of pile penetration is at least 10 times the section depth b of the pile and the ground water level is at least a distance of 1.5 b below the pile tip.

$$\therefore Q_{ue} = (\tfrac{1}{2}\gamma_e d N_\gamma + K_b \gamma_e D N_q)A \qquad \text{(Eq. 2)}$$

Using Fig. 8 and $\phi = 30°$ determine values for N_γ, N_q and K_b.

$$N_\gamma = 65 \qquad N_q = 82 \text{ and } K_b = 0.40$$

$$\begin{aligned} Q_{ue} &= [(\tfrac{1}{2} \times 0.110 \times 0.810 \times 65) + (0.40 \times 0.110 \times 82D)]0.680 \\ &= [2.93 + 3.61D]0.68 = 1.99 + 2.46D \end{aligned}$$

b. Skin Friction Component

$$\begin{aligned} Q_{us} &= \tfrac{1}{2} p K_b \gamma_e D^2 (\tan \delta) \qquad \text{(Eq. 4)} \\ &= \tfrac{1}{2} \times 3.30 \times 0.40 \times 0.110 D^2 \times \tan 20° \\ &= 0.026 D^2 \end{aligned}$$

c. Required Penetration

Group I loading governs, $Q_m = 102.4$ k/pile

$$F_s \times Q_m = Q_u = Q_{us} + Q_{ue}$$

Use a safety factor $= 2.5$

$$\begin{aligned} 2.5 \times 102.4 &= 1.99 + 2.46D + 0.026D^2 \\ 256.0 - 1.99 &= 2.46D + 0.026D^2 \\ D^2 + 94.6D + (47.3)^2 &= 9770 + (47.3)^2 \\ (D + 47.3)^2 &= 12007 \\ D + 47.3 &= 109.6 \\ D &= 62.3\text{; use } 63.0 \text{ ft} \end{aligned}$$

The depth of pile penetration should be verified by either a load test or careful observance of the resistance to driving.

Check assumption that $D \geq 10b$

$$63.0 > 10 \times 0.810 = 8.10 \text{ ft}$$

If D had been less than $10b$, depth of penetration would be recalculated using Eq. 3. Also, since the ground water level is $> 1.5d$ below the pile tip, it will not influence the depth of pile penetration.

LATERAL LOAD CAPACITY

Estimates of lateral load capacities are calculated using a simplified method by Broms. Since the piles are embedded only 12 inches into the concrete cap and the cap is relatively free to rotate, consider the piles as being "free-headed." Also consider that the soil is disturbed down to the bottom of the pile cap and neglect passive soil pressure acting on the pile cap.

Determine if Piles are "Long," "Short" or "Intermediate"

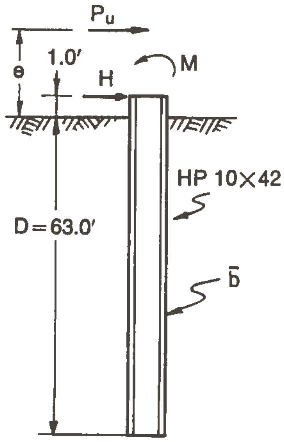
From Fig. 16, estimate the modulus of subgrade reaction, n_h, for a loose to medium dense sand to be 28 k/ft³.

$$\eta = \sqrt[5]{\frac{n_h}{EI}} \qquad \text{(Eq. 21)}$$

$$\eta = \sqrt[5]{\frac{28.0}{29 \times 10^3 \times \frac{211}{144}}} = \sqrt[5]{0.000659} = 0.230 \text{ ft}^{-1}$$

$$\eta D = 0.230 \times 63.0 = 14.49$$

Since ηD is greater than 4.0, consider the pile as being "long."



Calculate Effective Height of Lateral Load

Group III loading governs

$H = 8.17$ kips

$M = 4.21$ kip-ft or -4.21 kip-ft

$$e = 1.00 - \frac{4.21}{8.17} = 1.00 - 0.52$$

$e = 0.48$ ft

Find P_u from Fig. 18 by substituting values for the following quantities:

$$\frac{P_u}{K_p\bar{b}^3\gamma_e'};\quad \frac{M_{plastic}}{K_p\bar{b}^4\gamma_e'};\quad \frac{e}{\bar{b}} \qquad \text{(Eq. 23)}$$

Since the pile is loaded both axially and laterally, the reduced plastic moment capacity is dependent upon the ratio of axial load to the plastic axial capacity of the pile section.

Determine if Q_m/Q_y is equal to or less than 0.15

$$\frac{Q_m}{Q_y} = \frac{102.4}{12.4 \times 36.0} = 0.229 > 0.15$$

$$\begin{aligned} \therefore M_o &= 1.18M_{plastic}(1 - Q_m/Q_y) \qquad \text{(Eq. 35)} \\ &= 1.18M_{plastic}(1 - 0.229) \\ &= 0.910M_{plastic} \end{aligned}$$

$$M_{plastic} = n_s f_y s \qquad \text{(Eq. 24)}$$

Since the pile is loaded in the direction of maximum moment resistance of the pile, n_s is equal to 1.1.

$$M_o = 0.910 \times 1.1 \times 36.0 \times 43.4 = 1564 \text{ kip-in.} = 130.3 \text{ kip-ft}$$

$$K_p = \frac{1 + \sin\phi}{1 - \sin\phi} = \frac{1 + 0.500}{0.500} = 3.00$$

$$\frac{M_o}{K_p\bar{b}^4\gamma_e} = \frac{130.3}{3.00(0.839)^4 \times 0.110} = 797$$

$$\frac{e}{\bar{b}} = \frac{0.48}{0.839} = 0.572$$

From Fig. 18:

$$\frac{P_u}{K_p\bar{b}^3\gamma_e} = 130$$

$$\begin{aligned} P_u &= 130 \times 3.00(0.839)^3\,0.110 \\ &= 25.3 \text{ kips} \end{aligned}$$

Using a safety factor of 2.5, the allowable lateral load is:

$$P_a = \frac{P_u}{2.5} = \frac{25.3}{2.5} = 10.1 \text{ kips}$$

For Group III loading, a stress increase of 25% is allowed.

$$H = 8.17 \text{ k} < 10.1 \times 1.25 = 12.6 \text{ kips}$$

Thus, the HP 10×42 pile section is adequate to carry the lateral design load.

LATERAL DISPLACEMENT

Estimates of lateral displacements are calculated using a simplified method by Broms. The lateral load capacity generally is limited by the permissible lateral displacement due to the working load. The lateral displacements, for permanent and temporary lateral forces, at the ground surface are estimated using Fig. 19 and substituting values for the following quantities:

$$\frac{\Delta_a (EI)^{3/5}(n_h)^{2/5}}{P_a D}; \quad \eta D; \quad \frac{e}{D} \qquad \text{(Eq. 25)}$$

$$\eta D = 14.49 \text{ and } \frac{e}{D} = \frac{0.48}{63.0} = 0.0076$$

For a "long" pile, the lateral displacement is independent of the pile embedment. Since $\eta D = 14.5$ corresponding to $D = 63$ ft is beyond the range of Fig. 19, calculate the lateral displacement for $\eta D' = 10$.

For $\eta D' = 10$,

$$D' = \frac{10}{0.230} = 43.5 \text{ ft and } \frac{e}{D'} = \frac{0.48}{43.5} = 0.0110$$

From Fig. 19:

$$\frac{\Delta_a (EI)^{3/5}(n_h)^{2/5}}{P_a D'} = 0.31$$

$$\Delta_a = \frac{0.31(43.5)(12)P_a}{\left(\frac{29 \times 10^3 \times 211}{144}\right)^{3/5}(28)^{2/5}} = \frac{161.8 P_a}{600 \times 3.80}$$

$$= 0.0710\ P_a$$

Group I Loading—Earth Pressure

$$P_a = H = 4.57 \text{ kips}$$

$$\Delta_a = 0.0710 \times 4.57$$

$$= 0.324'' = \tfrac{5}{16} \text{ in.}$$

Group III Loading—Earth Pressure and Friction

$$P_a = H = 8.17 \text{ kips}$$

$$\Delta_a = 0.0710 \times 8.17$$

$$= 0.580'' = \tfrac{9}{16} \text{ in.}$$

Thus a permanent lateral displacement of 5/16 in. will occur with an additional lateral movement of 1/4 in. occurring with the application of the friction force. Where accurate determinations of these values are required, field tests should be made.

SETTLEMENT AT TOP OF PILE

Settlement will result from settlement of the pile tip plus the elastic deformation of the pile. The settlement of the pile tip results from that portion of the load which reaches the tip while the elastic deformation is based on the average of the load at the top and tip of the pile.

In the analysis for the depth of penetration, the ultimate end bearing component is:

$$Q_{ue} = 1.99 + 2.46D = 1.99 + (2.46 \times 63.0) = 157.0 \text{ kips}$$

$$\% \text{ of ultimate load at the pile tip} = \frac{157.0}{2.5 \times 102.4}$$

$$= 0.613 = 61.3\%$$

The approximate working load at the pile tip:

$$Q_m = 102.4 \times 0.613 = 62.8 \text{ kips}$$

Settlement of Pile Tip

Estimate from Fig. 11 using the average settlement curve.

$$\text{load/ultimate load} = 62.8/2.5 \times 102.4 = 0.245$$

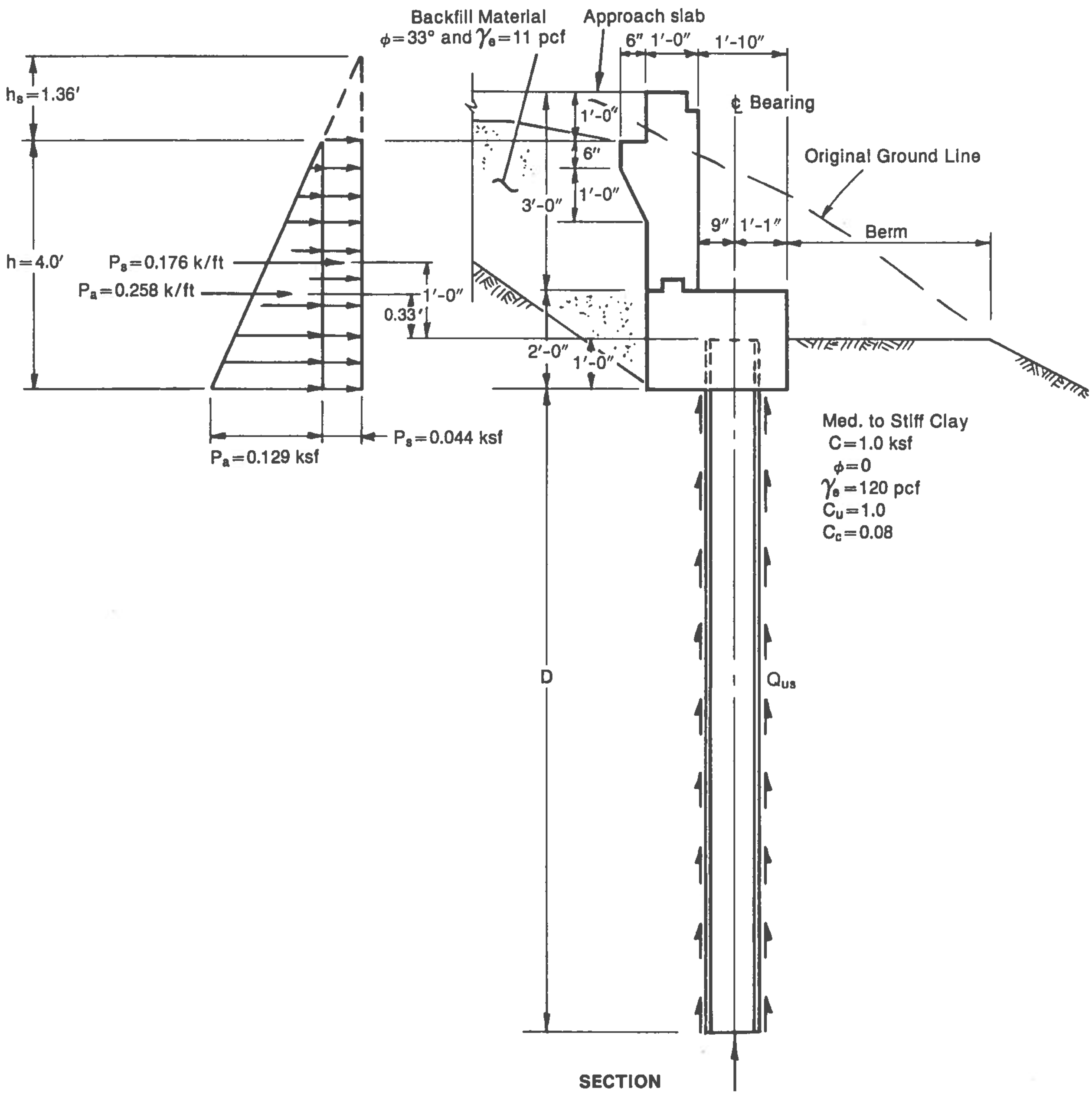
$$\therefore \Delta_T = 0.08 \text{ in.}$$

Elastic Deformation

Load @ Top of piles = 102.4 kips
Load @ Tip of piles = 62.8
Average pile load = 82.6

$$\Delta_1 = \frac{QD}{A_s E} = \frac{82.6 \times 63.0 \times 12}{12.4 \times 29 \times 10^3} = 0.174 \text{ in.}$$

$\therefore$ Total Settlement at Top of Pile $= 0.256'' \approx \frac{1}{4}$ in.



PART B—SINGLE ROW OF PILES IN FINE-GRAINED MATERIAL

EARTH PRESSURE

Active earth pressure, $P_a = K_a \gamma_s h$

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 33°}{1+\sin 33°} = \frac{1-0.545}{1+0.545}$$

$$K_a = 0.294$$

$$P_a = 0.294 \times 0.110 \times 4.0 = 0.129 \text{ ksf}$$

$$P_a = 0.129(4.0)/2 = 0.258 \text{ k/ft}$$

Surcharge Load

Section 1.2.19 of the AASHO Specifications states that live load surcharge is not considered where an adequately designed reinforced concrete approach slab is provided. However, the weight of the slab is supported by the backfill and considered as dead load surcharge. This load is equated to an equivalent height of soil.

$$h_s = \frac{\text{wt. of approach slab}}{\text{wt. of soil}} = \frac{1.0 \times 0.15}{0.110} = 1.36 \text{ ft}$$

$$P_s = 0.110 \times 0.294 \times 1.36 = 0.044 \text{ ksf}$$

$$P_s = 0.044 \times 4.0 = 0.176 \text{ k/ft}$$

Earth Pressure for Interior Piles (9 ft-3 in. Center to Center)
Forces about Top of Piles

Force	H (kips)	Arm (ft)	M (kip-ft)
Earth Pressure—0.258×9.25	2.40	0.33	0.79
Surcharge —0.176×9.25	1.63	1.00	1.63
Total	4.03		2.42

Passive soil resistance in front of the abutment should not be considered.

STRINGER REACTIONS—Refer to Part A.

ABUTMENT DEAD LOAD (Interior Pile)—Refer to Part A.

FORCES AT TOP OF INTERIOR PILES

Loads for AASHO Group I and III are tabulated. The design of stub abutments is governed by Group I loading for axial pile capacity and Group III loading for lateral pile capacity.

Group I—Dead Load, Live Load and Earth Pressure

Loading	F_v (kips)	Transverse Direction	
		H (kips)	M (kip-ft)
D.L. Superstructure	36.0	—	—
D.L. Abutment	13.4	—	−10.57
L.L.	53.0	—	—
Earth Pressure	—	4.03	+ 2.42
Total	102.4	4.03	− 8.15

Group III—Dead Load, Live Load, Earth Pressure and Friction

Loading	F_v (kips)	Transverse Direction	
		H (kips)	M (kip-ft)
D.L. Superstructure	36.0	—	—
D.L. Abutment	13.4	—	−10.57
L.L.	53.0	—	—
Earth Pressure	—	4.03	+ 2.42
Friction	—	3.60	+ 3.60
Total	102.4	7.63	− 4.55

PILE DESIGN

1. Size—For discussion refer to Part A. Use HP 10×42 Pile.

2. Depth of Penetration

For H-piles, the end bearing component (Q_{ue}) of the ultimate bearing capacity (Q_u) of a pile is negligible and the ultimate bearing capacity is equal to the skin friction component (Q_{us}). The lesser of the Q_{us} values found by equations 10, 11 and 12 is used to determine the depth of pile penetration.

a. Cohesion around the net perimeter

$$\begin{aligned} Q_{us} &= 2(d+b_f)\,cD \qquad \text{(Eq. 10)} \\ &= 2(0.810+0.839)1.0D \\ &= 3.30D \end{aligned}$$

b. Adhesion around the entire perimeter

$$Q_{us} = 2(d+2b_f)c_aD \qquad \text{(Eq. 11)}$$

From Fig. 9, for $c = 1.0$, $\frac{c_a}{c} = 0.73$

$$\therefore c_a = 0.73 \times 1.0 = 0.73$$

$$\begin{aligned} Q_{us} &= 2(0.810+2\times0.839)0.73D \\ &= 3.63D \end{aligned}$$

c. Adhesion to the flange and cohesion across the web opening

$$\begin{aligned} Q_{us} &= 2(dc+b_fc_a)D \qquad \text{(Eq. 12)} \\ &= 2[(0.810\times1.0)+(0.839\times0.73)]D \\ &= 2.84D \qquad \textit{governs} \end{aligned}$$

$$\therefore Q_u = Q_{us} = 2.84D$$

Use safety factor = 2.5

Group I loading governs, $Q_m = 102.4$ k/pile

$$Q_{us} = 2.5\ Q_m = 2.5 \times 102.4 = 2.84D$$

$$D = 90.1 \text{ ft} \qquad \text{Use } 90.0 \text{ ft}$$

The depth of pile penetration should be verified by either a load test, driving records, or pile capacities and performances of existing pile installations at nearby sites.

LATERAL LOAD CAPACITY

Estimates of lateral load capacities are calculated using a simplified method by Broms. Since the piles are embedded only 12 inches into the concrete cap and the cap is relatively free to rotate, consider the piles as being "free-headed." Also consider that the soil is disturbed down to the bottom of the pile cap and neglect passive soil pressure acting on the pile cap.

Determine if Piles are "Long" or "Short"

$$\beta = \sqrt[4]{\frac{k\bar{b}}{4EI}} \qquad \text{(Eq. 26)}$$

$$K = \frac{160\bar{m}c}{\bar{b}} \qquad \text{(Eq. 27)}$$

$$\therefore \beta = \sqrt[4]{\frac{160\bar{m}c}{4EI}}$$

for $c = 1.0$ use $\bar{m} = 0.34$

$$\beta = \sqrt[4]{\frac{160 \times 0.34 \times 1.0 \times 144}{4 \times 29 \times 10^3 \times 211}} = \sqrt[4]{0.000320}$$

$$= 0.134 \text{ ft}^{-1}$$

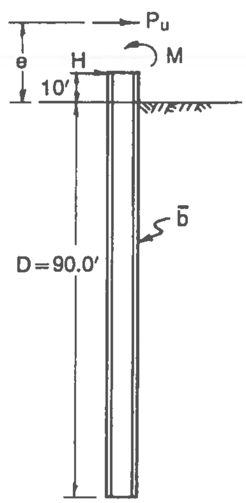
$$\beta D = 0.134 \times 90.0 = 12.06$$

Since βD is greater than 2.25, the pile is considered as being "long."

Calculate Effective Height of Lateral Load

Group III loading governs

$$H = 7.63 \text{ kips}$$



$$M = -4.55 \text{ kip-ft}$$

$$e = 1.00 - \frac{4.55}{7.63} = 1.00 - 0.60$$

$$e = 0.40 \text{ ft}$$

Determine P_u from Fig. 21 by substituting values for the following quantities:

$$\frac{P_u}{c\bar{b}^2}; \quad \frac{M_o}{c\bar{b}^3}; \quad \frac{e}{\bar{b}} \qquad \text{(Eq. 29)}$$

From Part A, $M_o = 137.1$ kip-ft

$$\frac{M_o}{c\bar{b}^3} = \frac{130.3}{1.0(0.839)^3} = 220$$

$$\frac{e}{\bar{b}} = \frac{0.40}{0.839} = 0.48$$

From Fig. 21: $\frac{P_u}{c\bar{b}^2} = 51$

$$P_u = 51 \times 1.0(0.839)^2 = 35.9 \text{ kips}$$

Using a safety factor $= 2.5$

$$P_a = \frac{35.9}{2.5} = 14.4 \text{ kips}$$

For Group III loading, a stress increase of 25% is allowed.

$$H = 7.63 < 14.4 \times 1.25 = 18.0 \text{ kips}$$

Thus, the HP 10×42 pile section is adequate to carry the lateral design load.

LATERAL DISPLACEMENT

Estimates of lateral displacements are calculated using a simplified method by Broms. The lateral displacement of long piles ($\beta D > 2.25$) is independent of the pile embedment. Lateral displacement at the ground surface, at working lateral loads less than one-half the ultimate values, may be approximated using Fig. 22 and substituting values for the following quantities:

$$\frac{\Delta_a k \bar{b} D}{P_a}; \quad \beta D; \quad \frac{e}{D} \qquad \text{(Eq. 30)}$$

$$\beta D = 12.06 \text{ and } \frac{e}{D} = \frac{0.40}{90.0} = 0.0044$$

For a long pile, the lateral displacement is independent of the pile embedment. Since $\beta D = 12.06$ corresponding to $D = 90$ ft is beyond the range of Fig. 22, calculate the lateral displacement for $\beta D' = 5$.

$$D' = \frac{5.0}{0.134} = 37.3' \text{ and } \frac{e}{D} = \frac{0.40}{37.3} = 0.0107$$

From Fig. 22:

$$\frac{\Delta_a k \bar{b} D}{P_a} = 10.4$$

$$k = \frac{160 \bar{m} c}{\bar{b}} = \frac{160 \times 0.34 \times 1.0}{0.839} = 64.8 \text{ kcf}$$

$$\Delta_a = \frac{10.4 P_a}{64.8 \times 0.839 \times 37.3} = 0.0051 P_a$$

Group I Loading—Earth Pressure

$$P_a = H = 4.03 \text{ kips}$$
$$\Delta_a = 0.0051 \times 4.03$$
$$= 0.0206' \times 12 = 0.247'' = 1/4 \text{ in.}$$

Group III Loading—Earth Pressure and Friction

$$P_a = H = 7.63 \text{ kips}$$
$$\Delta_a = 0.0051 \times 7.63$$
$$= 0.0389 \times 12 = 0.467'' \approx 7/16 \text{ in.}$$

Lateral displacement in fine-grained soils increase with time due to consolidation and creep of the soil. The approximate long term displacement may be found by reduction of the k value to 1/4 that for a clay with a cohesive strength between 0.5 and 1.5. For long term displacement use H resulting from earth pressure and surcharge.

$$k = \frac{160 \bar{m} c}{\bar{b}} \times 1/4 = 64.8 \times 1/4 = 16.2 \text{ kcf}$$

$$\beta = \sqrt[4]{\frac{16.2 \times 0.839 \times 144}{4 \times 29 \times 10^3 \times 211}} = \sqrt[4]{0.000080}$$
$$= 0.095 \text{ ft}^{-1}$$

$$\beta D = 0.095 \times 90.0 = 8.55$$

Use Fig. 22 and $\beta D = 5.0$

$$D' = \frac{5.0}{0.059} = 84.75 \qquad \frac{e}{D} = \frac{0.40}{84.75} = 0.0047$$

$$\frac{\Delta_a k \bar{b} D}{P_a} = 10.3$$

$$\Delta_a = \frac{10.3 P_a}{16.2 \times 0.839 \times 52.6} = 0.0144 P_a$$

Long term displacement due to earth pressure

$$P_a = H = 4.03 \text{ kips}$$

$$\Delta_a = 0.0144 \times 4.03$$
$$= 0.0580' \times 12 = 0.696 \approx 11/16 \text{ in.}$$

Thus a lateral displacement of ¼ in. will occur at the time of initial earth pressure loading. Due to consolidation and creep of the soil, the lateral displacement due to earth pressure will increase to 11/16 in. An additional ⅛ in. lateral movement will occur with application of the friction force. Where accurate determination of these values is required, field tests should be made.

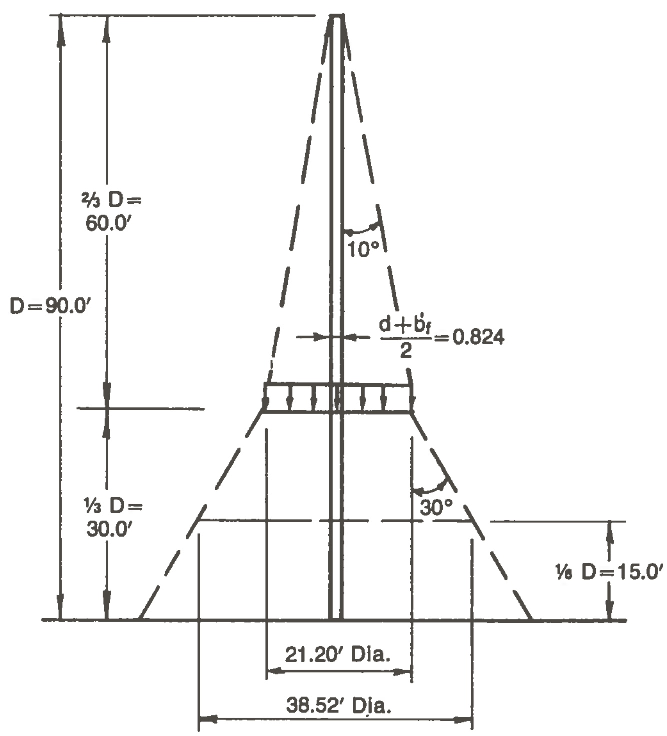
SETTLEMENT AT TOP OF PILE

Consolidation Settlement

Settlement is computed by a consolidation analysis for a soil layer starting at the tip of the pile and extending upward a distance of $D/3$. The assumed load distribution for computing settlement is given in Fig. 13. The amount of settlement is determined by the following equation:

$$\Delta_T = \frac{C_c D/3}{1+e_o}\left[\log\left(\frac{q_o+\Delta_q}{q_o}\right)\right]$$

where q_o = Existing Effective Overburden pressure at a height $D/6$ above the pile tip.
Δ_q = Pressure due to Q at a height $D/6$ above the pile tip.
e_o = Void Ratio
C_c = Compression Index



Determine the existing overburden pressure at a height of $D/6$ above the pile tip. The height of overburden can be approximated by the distance from the original ground surface to the point where the pressure is to be found.

$$H \approx (4.0+90.0-15.0) = 79.0 \text{ ft}$$

$$q_o = 78.5 \times 0.12 = 9.48 \text{ ksf}$$

Determine change in pressure due to pile load at a height of $D/6$ above the pile tip.

$$\Delta_q = \frac{Q}{A_H}$$

where A_H = area at section located at a height $D/6$ above the pile tip.

$$\Delta_q = \frac{102.4}{\frac{\pi}{4}(32.89)^2} = 0.12 \text{ ksf}$$

Since the piles are spaced 9′-3″ on centers, the pressure distribution for the piles will overlap almost 100%. Therefore, the total change in pressure will be approximately $2 \times \Delta_q = 2 \times 0.12 = 0.24$ ksf

The approximate settlement is:

$$\Delta_T = \frac{0.08(30.0)}{1+1.0}\left(\log \frac{9.42+0.24}{9.42}\right) = 1.20(\log 1.025)$$

$$= 1.00(0.01072) = 0.0107' = 0.128 \text{ in.}$$

$$\Delta_T = 1/8 \text{ in.}$$

Settlement due to Elastic Deformation

The average pile load, $Q' = \frac{102.4}{2} = 51.2$ kips

$$\Delta_e = \frac{Q'D}{A_s E} = \frac{51.2 \times 90.0 \times 12}{12.4 \times 29 \times 10^3} = 0.154 \text{ in.}$$

Total settlement $= 0.128 + 0.154 = 0.282'' \cong 1/4$ in.

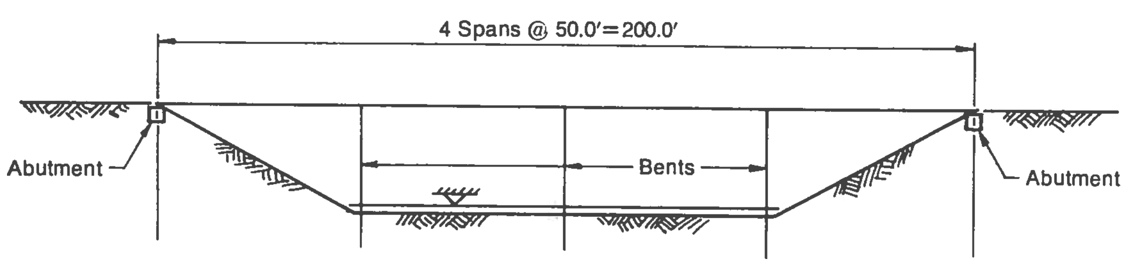
Example 2—Pile Bents

Pile bents provide an economical method of support for short span steel bridges. One pile is placed under each roadway stringer, and the tops of piles are tied together by steel members or a concrete cap. This gives a rigid frame to carry horizontal loads applied parallel to the pile bent.

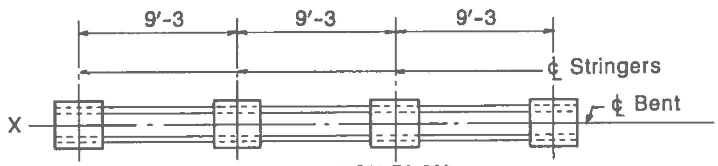
Roadway stringers are attached to the pile bents by bearing devices that permit rotation of the stringers, but prohibit longitudinal movement of the stringers with respect to the bents. When roadway stringers expand or contract due to temperature changes, the top of the pile bents move with the stringers. Since pile bents are relatively flexible, this movement causes only small loads and moments in the piles.

An interior pile bent of a four span continuous unit will be designed. The ends of the four span continuous unit are supported by abutments that longitudinally are much stiffer than the pile bents. Therefore, it will be assumed that all longitudinal force due to traction and wind are transferred to the abutments.

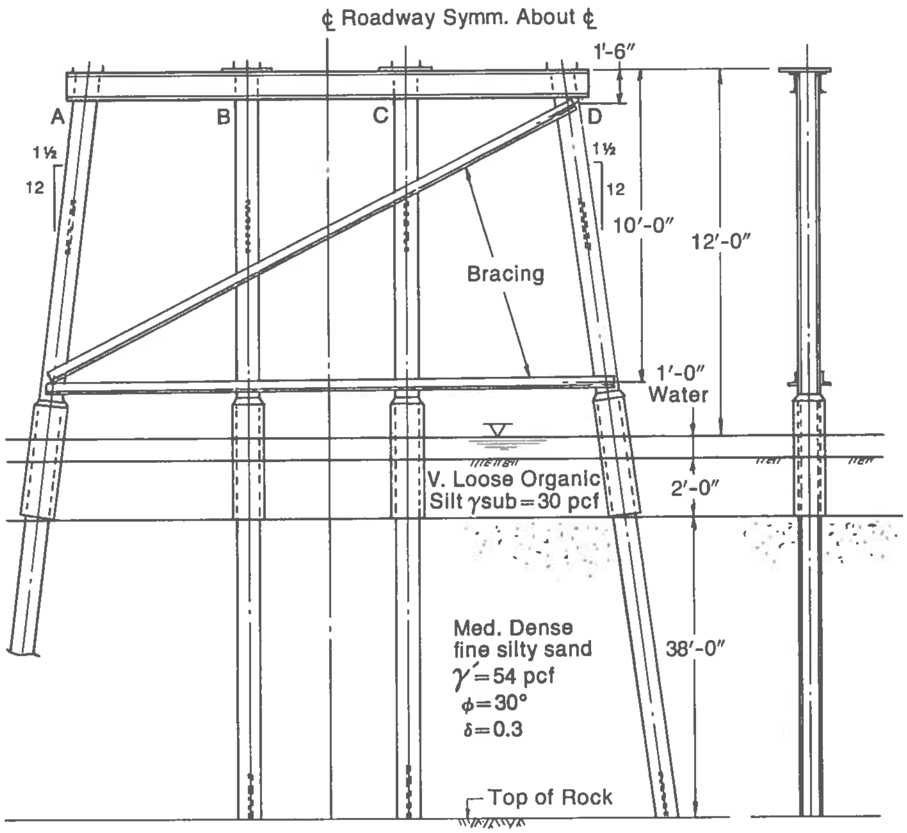
Pile bents may be built with or without bracing. Bracing reduces the bending stresses in the piles. This bent will be analyzed without bracing and with bracing as shown on the elevation view of the bent.



If the superstructure of this example had been longer (over 300 feet) or if a series of continuous spans had been used, the roadway stringers would still be fixed to each pile bent but expansion bearings would be used at the abutments. Each pile bent would then be designed for longitudinal traction and wind forces in addition to temperature forces. Larger piles would then be required for the bents with fixed bearings. An analysis of a pile bent subjected to longitudinal traction and wind forces would be similar to that shown in this example for temperature forces.



TOP PLAN



ELEVATION

END VIEW

LOADS

Determine total vertical and horizontal loads at the top of the pile bent for Groups I, II and III loading. Loading combinations are given in AASHO Specification, Section 1.2.22.

Expansion and contraction of the superstructure will place additional longitudinal loads on the piles. This additional horizontal force is dependent upon the pile section properties and cannot be determined until the pile section is chosen, and the point of fixity is determined. The longitudinal load due to temperature change does not affect the axial load on the pile, but does affect the bending stress in the pile.

Group I_a—Dead Load+Live Load

Loading	F_v (kips)	Transverse Direction	
		H_x (kips)	M_y (kip-ft)
Dead Load	308	—	—
Live Load	228	—	—
Total	536	—	—

Group I_b—Dead Load+Live Load+Impact

Loading	F_v (kips)	Transverse Direction	
		H_x (kips)	M_y (kip-ft)
Dead Load	308	—	—
Live Load	228	—	—
Impact	65	—	—
Total	601	—	—

Group II—Dead Load+Wind

Loading	F_v (kips)	Transverse Direction	
		H_x (kips)	M_y (kip-ft)
Dead Load	308	—	—
Wind on Structure	—	10.0	20.0
Total	308	10.0	20.0

Group III—Dead Load+Live Load+Impact+Traction+0.3 Wind+Wind on Live Load

Loading	F_v (kips)	Transverse Direction	
		H_x (kips)	M_y (kip-ft)
Dead Load	308	—	—
Live Load	328	—	—
Impact	65	—	—
Traction	—	—	—
Wind on Live Load	—	5.0	45.0
0.3 Wind on Structure	—	3.0	6.0
Total	601	8.0	51.0

PILE LOADS

Calculate the vertical pile loads at the top of the pile bent:

$$Q_m = \frac{F_v}{r} \pm \frac{M_y x}{\Sigma x^2} \qquad \text{(Eq. A)}$$

where $r = 4$

$$\Sigma x^2 = 2(1)[(4.625)^2 + (13.875)^2]$$
$$= 427.8 \text{ pile-ft}^2$$

Group I_a Loading

Vertical component of $Q = \frac{536}{4} = 134.0$ kips

Since the end piles are battered 1½ horizontal : 12 vertical, the maximum axial pile load is

$$Q = 134.0 \frac{\sqrt{(12.0)^2 + (1.5)^2}}{12} = 134.0 \times \frac{12.09}{12}$$
$$= 135.0 \text{ kips}$$

Group I_b Loading

Vertical component of $Q = \frac{601}{4} = 150.3$ kips

The maximum axial pile load is:

$$Q = 150.3 \times \frac{12.09}{12}$$
$$= 151.4 \text{ kips}$$

Group II Loading

$$\text{Maximum vertical component of } Q = \frac{308}{4} + \frac{20.0 \times 13.87}{427}$$
$$= 77.0 + 0.7$$
$$= 77.7 \text{ kips}$$

$$\text{Minimum vertical component of } Q = 77.0 - 0.7$$
$$= 76.3 \text{ kips}$$

The maximum axial pile load is:

$$Q = 77.7 \times \frac{12.09}{12}$$
$$= 78.3 \text{ kips}$$

Group III Loading

$$\text{Maximum vertical component of } Q = \frac{601}{4} + \frac{51.0 \times 13.87}{427}$$
$$= 150.3 + 1.7$$
$$= 152.0 \text{ kips}$$

$$\text{Minimum vertical component of } Q = 150.3 - 1.7$$
$$= 148.6 \text{ kips}$$

The maximum axial pile load is:

$$Q = 152.0 \times \frac{12.09}{12} = 153.1 \text{ kips}$$

Determine the approximate pile size for an allowable bearing stress of 9.0 ksi.

Group I_a Loading

Pile area $= \frac{135.0}{9.0} = 15.0 \text{ in.}^2$

Try an HP 12×53 pile

Pile Properties

$A_s = 15.6 \text{ in.}^2$

$I_x = 394 \text{ in.}^4$	$I_y = 127 \text{ in.}^4$
$r_x = 5.03 \text{ in.}$	$r_y = 2.86 \text{ in.}$
$S_x = 66.9 \text{ in.}^3$	$S_y = 21.1 \text{ in.}^3$

DETERMINE THE DEPTH OF FIXITY

Assume that the very loose organic silt strata offers no lateral resistance to pile movement. Therefore, the depth of fixity will begin at the top of the medium dense silty sand strata. The depth $\bar{D}$ is the point where the pile is considered restrained against rotation and is given by the following equation:

$$\bar{D} = 1.8 \sqrt[5]{\frac{EI}{n_h}} \qquad \text{(Eq. 36)}$$

Longitudinal Direction

$$\bar{D} = 1.8 \sqrt[5]{\frac{29 \times 10^3 \times 394}{28 \times 144}} = 1.8 \sqrt[5]{2834} = 1.8(4.90)$$

$$= 8.82 \text{ ft}; \qquad \text{Use } 10.00 \text{ ft}$$

Embedment length of the pile $= 38 \text{ ft} > 3D = 30.0 \text{ ft}$; therefore, "fixity" can be assumed.

Transverse Direction

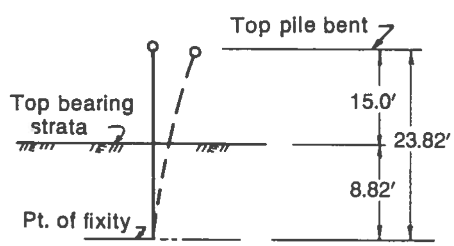
$$\bar{D} = 1.8 \sqrt[5]{\frac{29 \times 10^3 \times 127}{28 \times 144}} = 1.8 \sqrt[5]{913} = 1.8(3.91)$$

$$= 7.04 \text{ ft}$$

DETERMINE EFFECTIVE L/r OF PILES AND ALLOWABLE COMPRESSIVE STRESS

Longitudinal Direction

In the longitudinal direction, the top of the pile bent is free to rotate and translate. For this condition, Table C1.8.1 (AISC Specification) recommends an effective length of $2.1l$.



$$\text{Effective length, } l = 2.1(23.82)$$
$$l = 50.02 \text{ ft}$$

$$\text{Effective } l/r_x = \frac{50.02 \times 12}{5.03} = 119.3$$

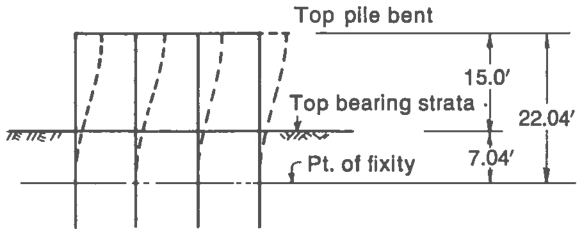
The allowable compressive stress is:

$$F_a = 16{,}000 - 0.38\left(\frac{l}{r}\right)^2$$

$$F_a = 16{,}000 - 0.38(119.3)^2 = 10{,}592 \text{ psi} = 10.59 \text{ ksi}$$

Transverse Direction—Unbraced

In the transverse direction, the top of the pile bent is fixed against rotation but free to translate. For this condition, Table C1.8.1 (AISC Specification) recommends an effective length of $1.2l$.



$$\text{Effective length, } l = 1.2(22.04) = 26.45 \text{ ft}$$

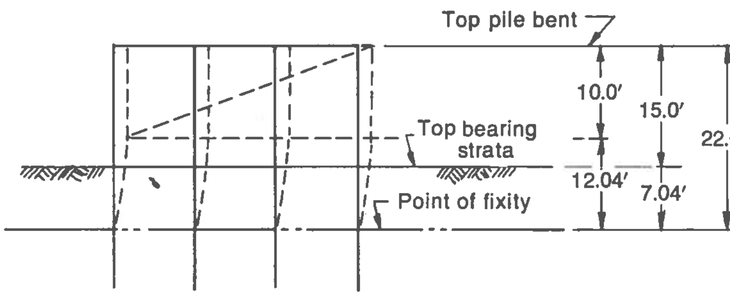
$$\text{Effective } l/r_y = \frac{26.45 \times 12}{2.86} = 111.0$$

The allowable compressive stress is:

$$F_a = 16{,}000 - 0.38\left(\frac{l}{r}\right)^2$$

$$F_a = 16{,}000 - 0.38(111.0)^2 = 11{,}319 \text{ psi} \cong 11.32 \text{ ksi}$$

Transverse Direction—Braced



$$\text{Effective length, } l = 1.2(12.04) = 14.45 \text{ ft}$$

$$\text{Effective } l/r_y = \frac{14.45 \times 12}{2.86} = 60.62$$

$$\text{Allowable } F_a = 16{,}000 - 0.38(60.62)^2 = 14{,}604 \text{ psi} = 14.60 \text{ ksi}$$

The compressive stress due to axial load is:

Group I_b Loading

$$f_a = \frac{151.4}{15.6} = 9.71 \text{ ksi} < 10.59 \text{ ksi}$$

Additional stresses due to bending must also be considered.

DETERMINE THE LONGITUDINAL FORCE DUE TO TEMPERATURE CHANGE

Stringers under the roadway will expand or contract as the temperature changes. Each pile bent will move longitudinally an amount equal to the temperature movement of the stringers. The resulting longitudinal force H_y in each pile is calculated by:

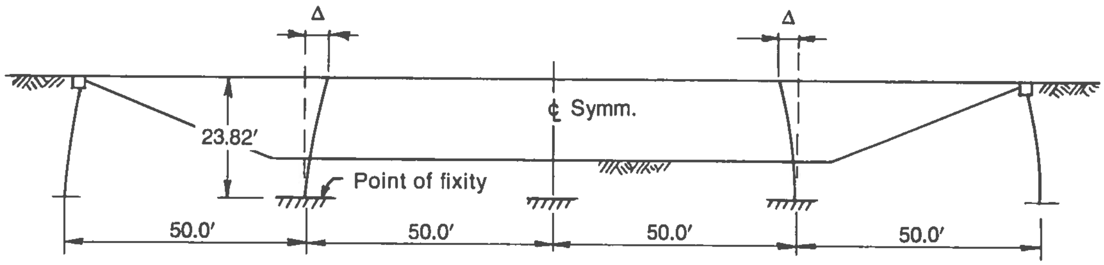
$$H_y = \frac{3EI\Delta}{L^3}$$

where E = modulus of elasticity of steel

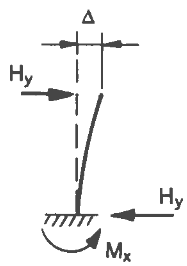
I = moment of inertia of the pile

Δ = movement of roadway stringer due to temperature change

L = length from top of pile to point of fixity



Longitudinal Direction



Design for a temperature drop of 78°

Δ = coefficient of thermal expansion × length × temp. change

$= 0.0000065 \times 50 \times 12 \times 78 = 0.304$ in.

$$H_y = \frac{3EI\Delta}{L^3} = \frac{3 \times 29 \times 10^3 \times 394 \times 0.304}{(23.82)^3 \times 1728} = 0.45 \text{ kips}$$

CHECK STRESSES IN PILES DUE TO AXIAL LOAD PLUS BENDING—UNBRACED PILE BENT

In the transverse direction, the piles are considered fixed at the point of fixity and fixed at the top of bent with a point of inflection at mid-height.

Therefore, maximum $M_y = \frac{22.04}{2} H_x = 11.02\, H_x$ at top of pile and at point of fixity.

Check stresses at the top of pile. Buckling must be considered at the top of pile while the pile is laterally supported at the point of fixity.

Check stresses by the AISC Interaction Formula

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}} \leq 1$$

(Eq. 1.6-1a, p. 5-22 1970 AISC Specs.)

where F_a = axial stress that would be permitted if axial force alone existed

F_b = compressive bending stress that would be permitted if bending moments alone existed

$$F'_e = \frac{12\pi^2 E}{23\left(K\frac{l_b}{r_b}\right)^2} = \frac{149{,}000{,}000}{\left(K\frac{l_b}{r_b}\right)^2}$$

l_b = actual unbraced length in the plane of bending

r_b = corresponding radius of gyration

K = effective length factor in the plane of bending

f_a = computed axial stress

f_b = computed compressive bending stress at the point under consideration

C_m = 0.85 for compression members in frames subject to joint translation

Group II

$$F_v = 308 \text{ kips} \quad H_x = 10.0 \text{ kips}$$

$$Q_{max} = 78.3 \text{ k/pile}$$

$$H_x \text{ carried by batter} = (77.7 - 76.3)\,\frac{1.5}{12} = 0.18 \text{ kips}$$

$$\text{Remaining } H_x = \frac{10.00 - 0.18}{4} = 2.46 \text{ k/pile}$$

$$M_y = 11.02 \times 2.46 = 27.1 \text{ kips}$$

$$f_a = \frac{78.3}{15.6} = 5.02 \text{ ksi} \qquad f_{by} = \frac{27.1 \times 12}{21.1} = 15.4 \text{ ksi}$$

$$F'_{ey} = \frac{149{,}000{,}000}{\left(K\,\frac{l_{by}}{r_{by}}\right)^2} = \frac{149{,}000{,}000}{111^2} = 12{,}090 \text{ psi} = 12.1 \text{ ksi}$$

$$\text{Check } \frac{f_a}{F_a} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_b} < 1.0$$

$$\frac{5.02}{1.25 \times 10.59} + \frac{0.85 \times 15.4}{\left(1 - \frac{5.02}{1.25 \times 12.1}\right) 20.0 \times 1.25} = 0.379 + 0.782 = 1.161$$

No Good

Obviously, HP12×53 piles without bracing are not sufficient. However, other loading cases will be investigated to illustrate the procedure for design without bracing.

Group III

$$F_v = 601 \text{ kips} \quad H_x = 8.00 \text{ kips}$$

$$Q_{max} = 153.1 \text{ kips}$$

$$H_x \text{ carried by batter} = (152.0 - 148.6)\,\frac{1.5}{12} = 0.43 \text{ kips}$$

$$\text{Remaining } H_x = \frac{8.00 - 0.43}{4} = 1.89 \text{ k/pile}$$

$$M_y = 11.02 \times 1.89 = 20.9 \text{ kip-ft}$$

$$f_a = \frac{153.1}{15.6} = 9.81 \text{ ksi} \qquad f_{by} = \frac{20.9 \times 12}{21.1} = 11.89 \text{ ksi}$$

$$\text{Check } \frac{f_a}{F_a} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_b} < 1$$

$$\frac{9.81}{10.59 \times 1.25} + \frac{0.85 \times 11.89}{\left(1 - \frac{9.81}{12.1 \times 1.25}\right) 20.0 \times 1.25} = 0.740 + 1.151 = 1.891$$

No Good

Group VI = Group III plus a longitudinal force due to temperature of 0.45 k/pile.

The maximum moment due to a longitudinal force occurs at the point of fixity. However, stresses at the point of fixity are not critical because the pile is laterally supported and buckling need not be considered.

Assuming full lateral support at midway between the top of the bearing stratum and the point of fixity, check stresses and consider buckling.

$$Q_m = 153.1 \text{ kips}$$

$$M_y = \left(11.02 - \frac{7.04}{2}\right)1.89 = 14.18 \text{ kip-ft}$$

$$M_x = \left(15 + \frac{7.04}{2}\right)0.45 = 8.33 \text{ kip-ft}$$

$$f_a = \frac{153.1}{15.6} = 9.81$$

$$f_{by} = \frac{14.18 \times 12}{21.1} = 8.06 \qquad f_{bx} = \frac{8.33 \times 12}{66.9} = 1.49$$

$$F'_{ex} = \frac{149{,}000{,}000}{\left(K \frac{l_{bx}}{r_{bx}}\right)^2} = \frac{149{,}000{,}000}{(119.3)^2} = 10{,}470 \text{ psi} = 10.47 \text{ ksi}$$

$$\text{Check } \frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right)F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right)F_{bx}} < 1.0$$

$$\frac{9.81}{10.59 \times 1.40} + \frac{0.85 \times 1.49}{\left(1 - \frac{9.81}{10.47 \times 1.40}\right)20.0 \times 1.40} + \frac{0.85 \times 8.06}{\left(1 - \frac{9.81}{12.1 \times 1.04}\right)20.0 \times 1.40}$$

$$0.661 + 0.136 + 0.581 = 1.378 \qquad \text{No Good}$$

Since the piles are considerably overstressed without bracing, provide bracing and recheck stresses.

CHECK STRESSES IN PILES DUE TO AXIAL LOAD PLUS BENDING—BRACED PILE BENT

In the transverse direction, assume a point of inflection midway between the point of fixity and the elevation of the bottom bracing strut.

Therefore, maximum $M_y = \frac{12.04}{2} H_x = 6.02\ H_x$ at the point of fixity and at the elevation of the bottom bracing strut. Check stresses in the pile at the bottom bracing strut. Buckling must be considered at the elevation of the bottom bracing strut while the pile is laterally supported at the point of fixity.

Group II

$$F_v = 30.8 \text{ kips} \quad H_x = 10.0 \text{ kips}$$

$$Q_{max} = 78.3 \text{ k/pile}$$

$$H_x \text{ carried by batter} = (77.7 - 76.3)\frac{1.5}{12} = 0.18 \text{ kips}$$

$$\text{Remaining } H_x = \frac{10.00 - 0.18}{4} = 2.46 \text{ k/pile}$$

$$M_y = 6.02 \times 2.46 = 14.8 \text{ kip-ft}$$

$$f_a = \frac{78.3}{15.6} = 5.02 \text{ ksi} \qquad f_{by} = \frac{14.8 \times 12}{21.1} = 8.40 \text{ ksi}$$

$$F'_{ey} = \frac{149{,}000{,}000}{\left(K \frac{l_{by}}{r_{by}}\right)} = \frac{149{,}000{,}000}{60.62^2} = 40{,}547 \text{ psi} = 40.55 \text{ ksi}$$

$$\text{Check } \frac{f_a}{F_a} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_b} \quad < 1$$

$$\frac{5.02}{10.59 \times 1.25} + \frac{0.85 \times 8.40}{\left(1 - \frac{5.02}{40.55 \times 1.25}\right) 20 \times 1.25} = 0.379 + 0.317 = 0.696 \quad < 1$$

Group III

$$F_v = 6.01 \text{ kips} \quad H_x = 8.00 \text{ kips}$$

$$Q_{max} = 153.1 \text{ kips}$$

$$H_x \text{ carried by batter} = \frac{8.00 - 0.43}{4} = 1.89 \text{ k/pile}$$

$$M_y = 6.02 \times 1.89 = 11.4 \text{ kip-ft}$$

$$f_a = \frac{153.1}{15.6} = 9.81 \text{ ksi} \qquad f_{by} = \frac{11.4 \times 12}{21.1} = 6.48 \text{ ksi}$$

$$\text{Check } \frac{f_a}{F_a} + \frac{C_m f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_b} \quad < 1$$

$$\frac{9.81}{10.59 \times 1.25} + \frac{0.85 \times 6.48}{\left(1 - \frac{9.81}{40.55 \times 1.25}\right) 20 \times 1.25} = 0.741 + 0.273 = 1.014$$

Slightly greater than 1 OK

Group VI = Group III plus a longitudinal force due to temperature of 0.45 k/pile.

Since the moments due to longitudinal force are relatively small, combined stresses in the pile at the elevation of the bottom bracing strut govern.

$$Q_{max} = 153.1$$

$$M_y = 6.02 \times 1.89 = 11.4 \text{ kip-ft}$$

$$M_x = 10.0 \times 0.45 = 4.50$$

$$f_a = \frac{153.1}{15.6} = 9.81 \text{ ksi} \qquad f_{by} = \frac{11.4 \times 12}{21.1} = 6.48 \text{ ksi}$$

$$f_{bx} = \frac{4.50 \times 12}{66.9} = 0.81 \text{ ksi}$$

$$F'_{ex} = \frac{149{,}000{,}000}{\left(K \frac{l_{bx}}{r_{bx}}\right)^2} = \frac{149{,}000{,}000}{119.3^2} = 10{,}470 \text{ psi} = 10.47 \text{ ksi}$$

$$\text{Check } \frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{mx} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}} \quad < 1$$

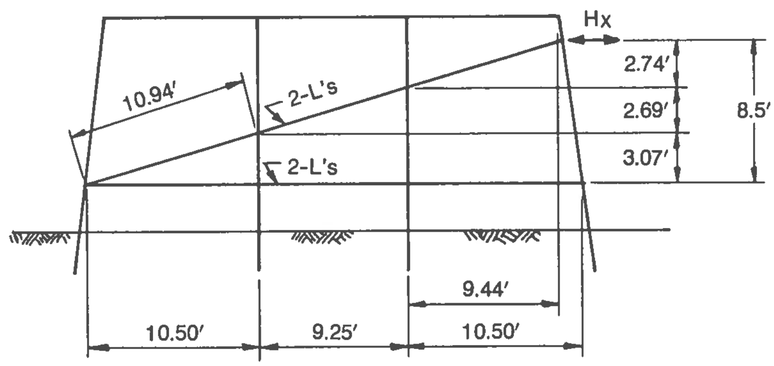
$$\frac{9.81}{10.59 \times 1.40} + \frac{0.85 \times 6.48}{\left(1 - \frac{9.81}{40.55 \times 1.40}\right) 20 \times 1.40} + \frac{0.85 \times 0.70}{\left(1 - \frac{9.81}{10.47 \times 1.40}\right) 20 \times 1.40}$$

$$0.662 > 0.238 > 0.074 = 0.974 \quad < 1 \text{ OK}$$

Therefore, HP 12×53 piles with bracing are sufficient.

Bracing Design

Weld single angles to each flange of piles.

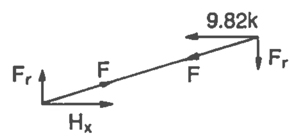


Group II

H_x carried by batter $=0.18$ kips

H_x carried by bracing $=10.00-0.18=9.82$ kips

Bracing in compression



Axial compressive force, $F=9.82\times\dfrac{10.94'}{10.50'}=10.23$ kips

Section 1.7.12, AASHO, limits the l/r for bracing members to 140.

$$\text{Min. } r_z=\frac{10.94\times 12}{140}=0.94$$

Try 2-L $5\times 5\times \frac{5}{16}$ in.
Area $=6.06$ sq in.
$r_z=0.994$

$$f_a=\frac{10.23\text{ kips}}{6.06}=1.69\text{ ksi}$$

$$l/r=\frac{10.94\times 12}{0.994}=132$$

$$F_a=16{,}000-0.30(132)^2=10{,}770\text{ psi}=10.77\text{ ksi}$$

$\therefore$ Angles are adequate in compression

Bracing in tension

Axial tension force, $F=10.23$ kips

Section 1.7.15, AASHO, limits the effective area to the net area of the connected leg plus one-half of the area of the outstanding leg.

$$\text{Effective Area}=[3.03-\tfrac{1}{2}(4.69\times 0.313)]2$$
$$=4.60\text{ sq in.}$$

$$f_t=\frac{10.23}{4.60}=2.22\text{ ksi}<20.0\text{ ksi}$$

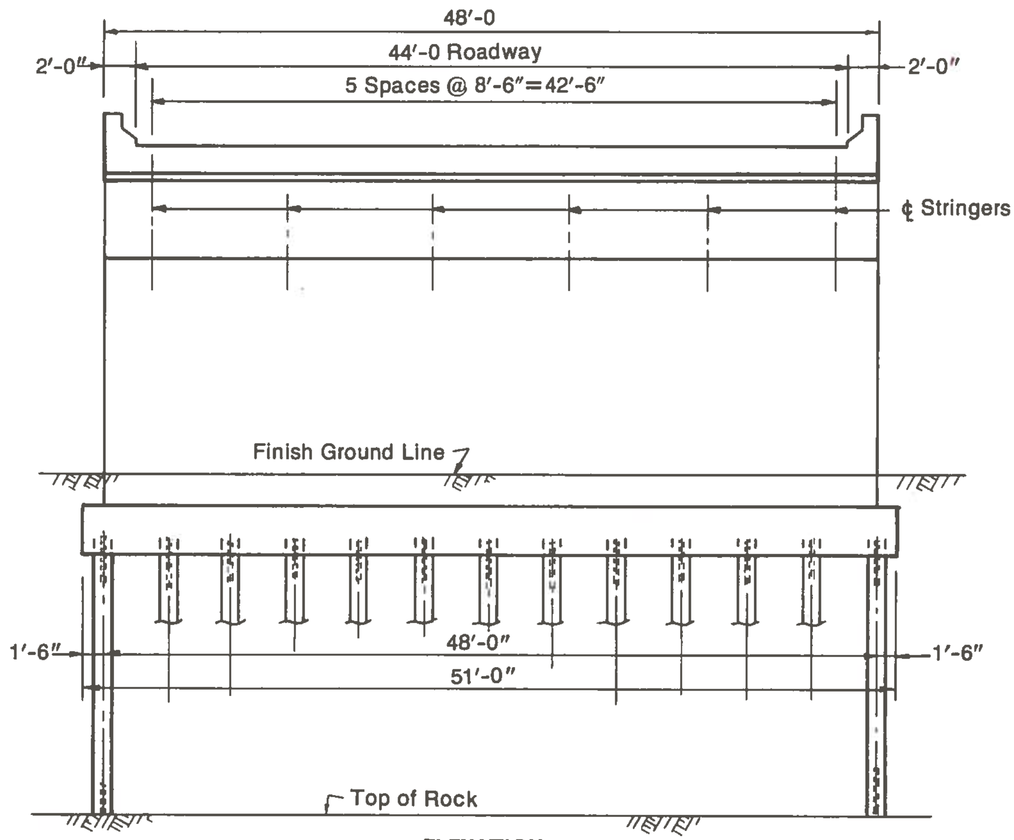
Use 2-L's $5\times 5\times \frac{5}{16}$ in. for all bracing

Example 3—Cantilever Abutment

This design is typical for abutments that must extend several feet above the finished ground line. Two or more rows of piles at relatively close spacing are required. The size of the footing and the pile spacing is adjusted to give approximately equal loads to all piles. Piles are battered to carry a large portion of the horizontal load caused by earth pressure.

Piles in this example are designed for rock bearing. An abutment on friction piles is designed exactly the same except the additional steps of calculating pile penetration and pile settlement are included.

The design of pile foundations under retaining walls is similar to this example except that the only loads considered are the weight of the wall, weight of the earth above the footing, and earth pressure.



ELEVATION

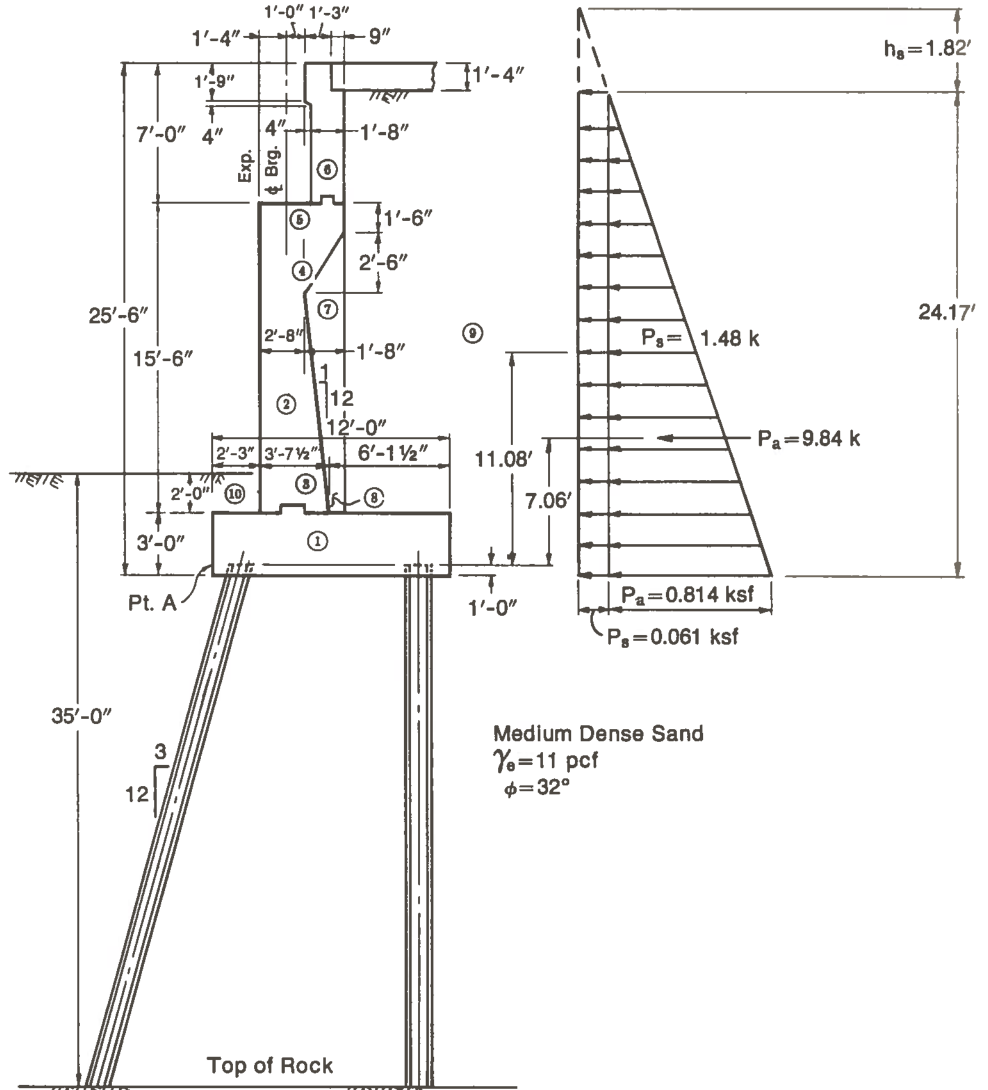
STRINGER REACTIONS

The maximum stringer reaction are:

$$DL = 126 \text{ k/stringer}$$
$$LL = 56 \text{ k/stringer}$$

In accordance with AASHO, Section 1.2.12, impact is not applied to piles below the ground surface. Since the bearings are an expansion type, the maximum horizontal force that can be applied to the abutment by the stringers is equal to the horizontal force necessary to overcome friction in the bearings. This friction force varies with the type of expansion bearings used. For this example, assume that the friction force is 10% of the dead load reaction.

$$F = 126.0 \text{ k} \times 0.10 = 12.6 \text{ k/stringer}$$



SECTION

Determine reaction for 1.0 ft. of abutment width by dividing the stringer reaction by the stringer spacing.

Dead Load = 126/8.50 = 14.8 k/ft
Live Load = 56/8.50 = 6.6 k/ft
Friction = 12.6/8.50 = 1.48 k/ft

EARTH PRESSURE

Assume the soil properties of the abutment backfill to be the same as the original soil properties.

Active earth pressure, $P_a = K_a \gamma_s H$

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 32°}{1+\sin 32°} = \frac{1-0.531}{1+0.531}$$
$$= 0.306$$

$$P_a = 0.110 \text{ kcf} \times 0.306 \times 24.17 = 0.814 \text{ ksf}$$

$$P_a = 0.814 \times \frac{24.17}{2} = 9.84 \text{ kips}$$

Surcharge

Section 1.2.19 of the AASHO Specifications states that live load surcharge is not considered where an adequately designed reinforced concrete approach slab is provided. However, the weight of the slab is supported by the backfill and is considered as dead load surcharge. This load is equated to an equivalent height of soil.

$$h_s = \frac{\text{wt. of approach slab}}{\text{wt. of soil}} = \frac{0.150 \times 1.33}{0.110} = 1.82 \text{ ft}$$

$$P_s = 1.82 \times 0.110 \times 0.306 = 0.061 \text{ ksf}$$

$$P_s = 0.061 \times 24.17 = 1.48 \text{ kips}$$

Passive soil resistance in front of the abutment should not be considered.

DEAD LOAD PER FOOT OF ABUTMENT ABOUT PT. "A"

	F_v (kips)	Arm about Pt. "A" (ft)	Moment about Pt. "A" (kip-ft)
① 3.0×12.0×0.15	5.40	6.00	32.4
② 2.67×14.0×0.15	5.61	3.58	20.1
③ 0.96×½×11.5×0.15	0.83	5.24	4.3
④ 2.5×1.67×½×0.15	0.31	5.50	1.7
⑤ 1.5×4.33×0.15	0.97	4.42	4.3
⑥ 1.67×7.0×0.15	1.75	5.75	10.1
⑦ 1.67×14.0×½×0.110	1.29	6.00	7.7
⑧ 0.71×½×11.5×0.110	0.45	6.12	2.8
⑨ 23.2×5.42×0.110	13.83	9.29	128.5
⑩ 2.0×2.25×0.110	0.50	1.13	0.6
	30.94		212.5

Determine the total vertical and horizontal loads about Pt. "A" (top of piles) for the following load cases.

Case I—Dead Load+Earth Pressure

Loading	F_v (kips)	H (kips)	Arm about Pt. "A" (ft)	Moment about Pt. "A" (kip-ft)
Dead Load of Abutment	30.9	—	6.88	+212.5
Dead Load of Superstructure	14.8	—	3.58	+ 53.0
Earth Pressure	—	9.84	7.06	− 69.5
Surcharge	—	1.48	11.08	− 16.4
Total—Case I	45.7	11.32		+179.6

$$\text{Location of resultant, } e = \frac{179.6}{45.7} = 3.93 \text{ ft}$$

Case II—Dead Load+Live Load+Earth Pressure

Loading	F_v (kips)	H (kips)	Arm about Pt. "A" (ft)	Moment about Pt. "A" (kip-ft)
Case I	45.7	11.32	3.93	+179.6
Live Load	6.6	—	3.58	+ 23.6
Total—Case II	52.3	11.32		+203.2

$$\text{Location of resultant, } e=\frac{203.2}{52.3}=3.89 \text{ ft}$$

Case III—Dead Load+Earth Pressure+Friction

Loading	F_v (kips)	F_h (kips)	Arm about Pt. "A" (ft)	Moment about Pt. "A" (kip-ft)
Case I	45.7	11.32	3.93	+179.6
Friction	—	1.48	17.5	− 25.9
Total—Case III	45.7	12.80		+153.7

$$\text{Location of resultant, } e=\frac{153.7}{45.7}=3.36 \text{ ft}$$

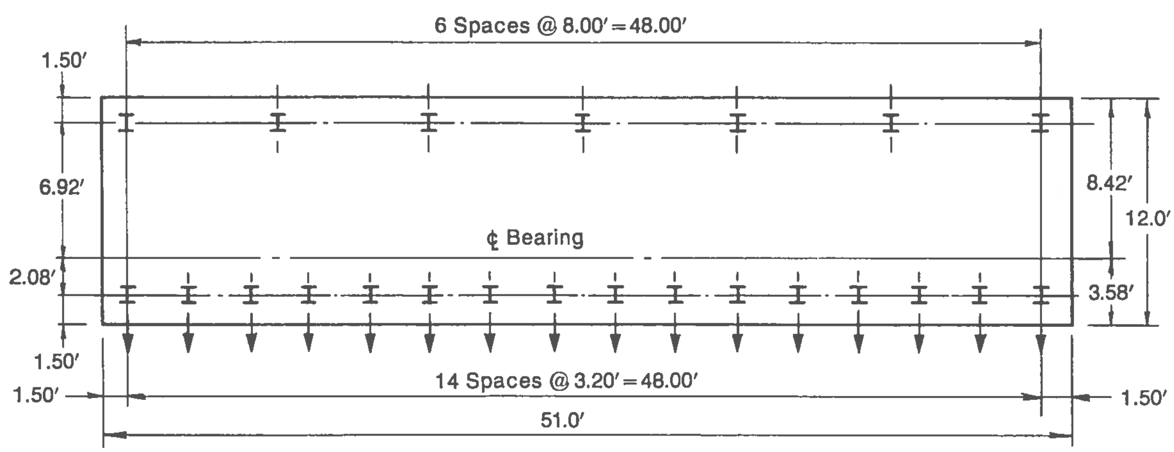
Normally, Case II is critical when determining maximum pile load. Either Case I or Case II is critical when determining the required pile batter.

PILE LOAD

Assume a footing size, pile spacing and pile size. A footing width equal to about one half of the abutment height is reasonable for a first trial. Piles in the front row usually are spaced at 3 ft-0 in. to 4 ft-0 in., in the rear row at 6 ft-0 in. to 10 ft-0 in. When needed, a third row may be placed 3 ft-0 in. to 4 ft-0 in. behind the front row and spaced at a multiple of the front row spacing.

10 in. or 12 in. piles are sufficient under most abutment and retaining walls. This example will be based upon HP 12×53 piles loaded to 9 kips per square inch, or 140 kips per pile.

Assume the following footing and pile pattern.



FOOTING PLAN

Determine the centroid of the pile group measured from the edge of footing (Pt. "A").

$$\text{Row 1: } \frac{1 \text{ pile}}{3.20 \text{ ft}} = 0.313 \text{ piles/ft} \times \; 1.5 \text{ ft} = 0.469$$

$$\text{Row 2: } \frac{1 \text{ pile}}{8.00 \text{ ft}} = \frac{0.125 \text{ piles/ft}}{0.438 \text{ piles/ft}} \times 10.5 \text{ ft} = \frac{1.310}{1.779}$$

$$\text{Centroid of pile group} = \frac{1.779}{0.438} = 4.06 \text{ ft from Pt. "A"}$$

Moment of Inertia of pile group

$$\Sigma x^2 = 0.313(2.56)^2 + 0.125(6.44)^2$$
$$= 7.23 \text{ pile-ft}^2$$

Determine the vertical load carried by each pile:

Case I

$$\text{Moment about centroid of pile group} = 45.7(4.06-3.93)$$
$$= 5.94 \text{ kip-ft}$$

$$\text{Vertical component of pile load, } Q_m = \frac{F_v}{r} \pm \frac{M_y x}{\Sigma x^2}$$

$$\text{Row 1: } Q_m = \frac{45.7}{0.438} + \frac{5.94 \times 2.56}{7.23} = 104.3 + 2.1 = 106.4 \text{ kips}$$

$$\text{Row 2: } Q_m = 104.3 - \frac{5.94 \times 6.44}{7.23} = 104.3 - 5.3 = \; 99.0 \text{ kips}$$

Case II

$$\text{Moment about centroid of pile group} = 52.3(4.06-3.89)$$
$$= 8.89 \text{ kip-ft}$$

$$\text{Row 1: } Q_m = \frac{52.3}{0.438} + \frac{8.89 \times 2.56}{7.23} = 119.4 + 3.1 = 122.5 \text{ kips}$$

$$\text{Row 2: } Q_m = 119.4 - \frac{8.89 \times 6.44}{7.23} = 119.4 - 7.9 = 111.5 \text{ kips}$$

Case III

$$\text{Moment about centroid of pile group} = 45.7(4.06-3.36)$$
$$= 32.0 \text{ kip-ft}$$

$$\text{Row 1: } Q_m = \frac{45.7}{0.438} + \frac{32.0 \times 2.56}{7.23} = 104.3 + 11.3 = 115.6 \text{ kips}$$

$$\text{Row 2: } Q_m = 104.3 - \frac{32.0 \times 6.44}{7.23} = 104.3 - 28.5 = 75.8 \text{ kips}$$

LATERAL CAPACITY

Determine the horizontal load per pile.

Case I

$$P_a = \frac{H}{\text{Piles per foot}} = \frac{11.32}{0.438} = 25.8 \text{ k/pile}$$

From Fig. 25, for an HP 12×53 pile embedded at least 18 feet in medium dense coarse-grained material above the water table, the ultimate lateral load capacity divided by a safety factor of 2.5 is equal to 15.0. Also 7.0 k will cause a lateral movement of ¼ in. Since these capacities are much less than the computed lateral load of 25.8 k/pile batter the front two rows of pile. Try a 3 horizontal : 12 vertical batter.

$$\text{Horizontal load per ft carried by batter} = \frac{\text{Vertical pile load}}{\text{Pile spacing}} \times \text{Pile batter}$$

$$H = 11.32 \text{ k/ft}$$

$$\text{Horizontal load taken by Row 1 battered piles} = \frac{106.4}{3.2} \times \frac{3}{12} = 8.31 \text{ kips}$$

$$\text{Lateral load to each pile} = \frac{\text{Applied horiz.} - \text{horiz. load carried by batter}}{\text{piles per foot}}$$

$$= \frac{11.32 - 8.31}{0.438} = 6.9 \text{ k/pile} < 7.0 \text{ k/pile}$$

Case III

$$H = 12.80 \text{ k/ft}$$

$$\text{Horizontal load taken by Row 1 battered piles} = \frac{115.6}{3.2} \times \frac{3}{12} = 9.03 \text{ kips}$$

$$\text{Lateral load to each pile} = \frac{12.80 - 9.03}{0.438}$$

$$= 8.65 \text{ k/pile} < 7.0 \times 1.25 = 8.75$$

Therefore, by battering the front row of piles, lateral movement of the abutment will be about ¼ inch.

Check HP 12×53 for maximum axial load including the effect of batter.

Group II (*Governs*)

Vertical component of pile load = 122.5 kips

Horiz. component of pile load $= 122.5 \times \frac{3}{12} = 30.6$

$$\therefore \text{Axial pile load} = \sqrt{(122.5)^2 + (30.6)^2}$$

$$= 126.3 \text{ kips}$$

Allowable pile load = 140 kips

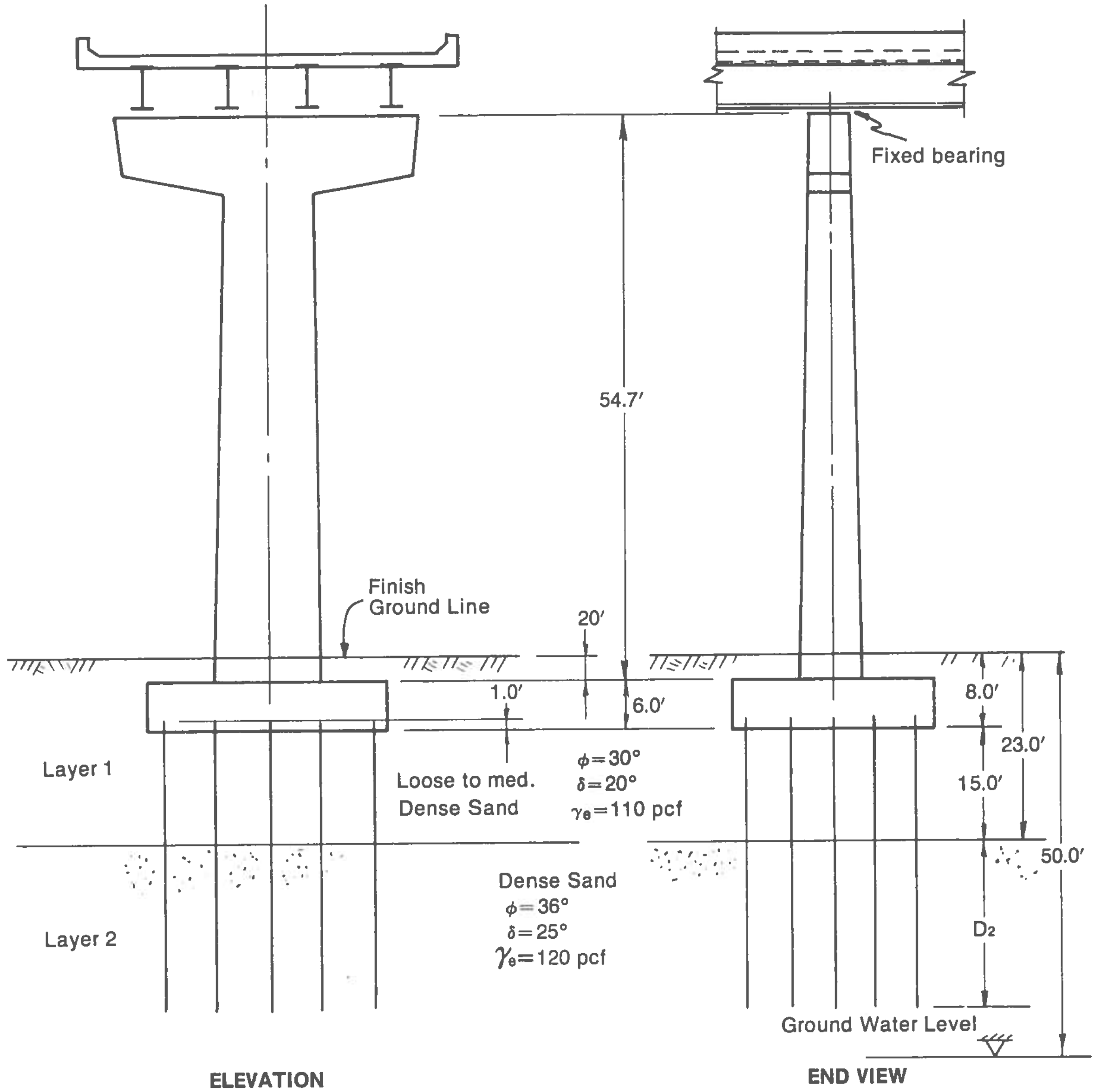
HP 12×53 piles spaced as shown are adequate.

Example 4—Bridge Pier on Land

The design of piles for this type of foundation is governed primarily by the axial capacity of the piles. For this bridge pier the piles are designed as friction piles in coarse-grained soil (Part A) and as friction piles in fine-grained soil (Part B).

FORCES AT TOP OF FOOTING

Determine forces at the top of the footing. These forces are used to design the column section and then to estimate the footing size and pile pattern. Loads for AASHO Groups I, II and III are tabulated. The design of many piers is governed by Group III loading.



PART A—PILE GROUP IN COARSE-GRAINED MATERIAL

Group I—Dead Load+Live Load+Impact

Loading	F_v (kips)	Transverse Direction		Longitudinal Direction	
		H_x (kips)	M_y (kip-ft)	H_y (kips)	M_x (kip-ft)
D.L. of Superstructure	966	—	—	—	—
D.L. of Pier	510	—	—	—	—
L.L.	206	—	680	—	—
Impact	45	—	150	—	—
Total Group I	1727	—	830	—	—

Group II—Dead Load+Wind

Loading	F_v (kips)	Transverse Direction		Longitudinal Direction	
		H_x (kips)	M_y (kip-ft)	H_y (kips)	M_x (kip-ft)
D.L. of Superstructure	966	—	—	—	—
D.L. of Pier	510	—	—	—	—
Wind on Superstructure	—	63.0	3930	15.0	951
Wind on Pier	—	8.5	231	28.3	959
Total Group II	1476	71.5	4161	43.3	1910

Group III—Dead Load+Live Load+Impact+Traction+0.3 Wind+Wind on Live Load+Centrifugal Force+Friction

Loading	F_v (kips)	Transverse Direction		Longitudinal Direction	
		H_y (kips)	M_y (kip-ft)	H_y (kips)	M_x (kip-ft)
D.L. of Superstructure	966	—	—	—	—
D.L. of Pier	510	—	—	—	—
L.L.	206	—	680	—	—
Impact	45	—	150	—	—
0.3 Wind on Superstructure	—	18.9	1180	4.5	286
0.3 Wind on Pier	—	2.6	69	8.5	288
Wind on Live Load	—	10.0	716	4.0	287
Centrifugal Force	—	42.0	3000	—	—
Friction	—	—	—	48.3	2652
Traction	—	—	—	27.4	1502
Total Group III	1727	73.5	5795	92.7	5015

PILE SIZE

High capacities piles concentrated near the periphery of a footing provide the most economical foundation when large loads and large moments must be supported.

However, high capacity piles should be used only if the soil is capable of supporting the high loads delivered by each pile.

Try HP 14×73 piles with an allowable steel stress of 9.0 ksi.

Allowable load per pile, $Q=21.5\times 9.0=193.5$ kips

Pile Properties

$$A_s=21.5 \text{ psf} \quad =0.149 \text{ ft}^2$$
$$d=13.64 \text{ in.} \quad =1.13 \text{ ft}$$
$$b_f=14.586 \text{ in.} =1.21 \text{ ft}$$
$$p=2(d+b_f) \quad =4.68 \text{ ft}$$
$$A=db_f \quad =1.37 \text{ ft}^2$$

DEPTH OF PENETRATION

Determine the depth of penetration required to develop an allowable load of 193.5 kips on an HP 14×73 pile.

Piles driven in a group in coarse-grained material can be designed based upon the capacity of a single pile.

The ultimate bearing capacity of a pile penetrating different strata of soil is equal to the sum of the skin friction in each strata plus the end bearing at the tip of the pile.

$$Q_u = \Sigma Q_{us} + Q_{ue}$$

Skin Friction Components

Layer 1: Using Fig. 8, for $\phi = 30° - N_\gamma = 65$, $N_q = 82$ and $K_b = 0.40$

$$Q_{us} = \tfrac{1}{2} p K_{b1} \gamma_{e1} D_1^2 (\tan \delta) \qquad \text{(Eq. 4)}$$
$$= \tfrac{1}{2} \times 4.68 \times 0.40 \times 0.110 (15.0)^2 \tan 20°$$
$$= 8.4 \text{ kips}$$

Layer 2: Using Fig. 8, for $\phi = 36° - N\gamma = 230$, $N_q = 205$ and $K_b = 0.52$

$$Q_{us} = [\gamma_{e1} D_1 + \tfrac{1}{2} \gamma_{e2} D] D_2 p K_b (\tan \delta) \qquad \text{(Eq. 4a)}$$
$$= [(0.110 \times 15.0) + (\tfrac{1}{2} \times 0.120 D_2] D_2 (4.68 \times 0.52) \tan 25°$$
$$= [1.650 + 0.060 D_2] 1.135 D_2$$
$$= 1.873 D_2 + 0.0681 D_2^2$$

End Bearing Component

$$Q_{ue} = [\tfrac{1}{2} \gamma_{e2} d N\gamma_2 + K_b N_{q2} (\gamma_{e1} D_1 + \gamma_{e2} D_2)] A \qquad \text{(Eq. 2a)}$$
$$= [(\tfrac{1}{2} \times 0.120 \times 1.13 \times 230) + 0.52 \times 205 (0.110 \times 15.0 + 0.120 \times D_2)] 1.37$$
$$= [15.6 + 106.6 (1.65 + 0.120 D_2)] 1.37$$
$$= [15.6 + 175.9 + 12.79 D] 1.37$$
$$= 262.4 + 17.52 D$$

Required Penetration

$$Q_u = F_s \times Q_m = \Sigma Q_{us} + Q_{ue}$$

Using safety factor = 2.5

$$Q_u = 2.5 \times 193.5 = 3.4 + 1.873 D_2 + 0.0681 D_2^2 + 262.4 + 17.52 D_2$$

$$483.8 = 270.8 + 19.39 D_2 + 0.0681 D_2^2$$

$$D_2^2 + 284.7 D_2 + (142.4)^2 = 3127 + (142.4)^2$$

$$(D_2 + 142.4)^2 = 234.05$$

$$D_2 + 142.4 = 153.0$$

$$D_2 = 10.6; \text{ use } 11.0 \text{ ft}$$

The depth of pile penetration should be verified by either a load test or careful observance of the resistance to driving.

Check assumption that total pile penetration $> 10d$

$$15.0 + 11.0 - 26.0 \geq 10 \times 1.13 = 11.3 \text{ ft}$$

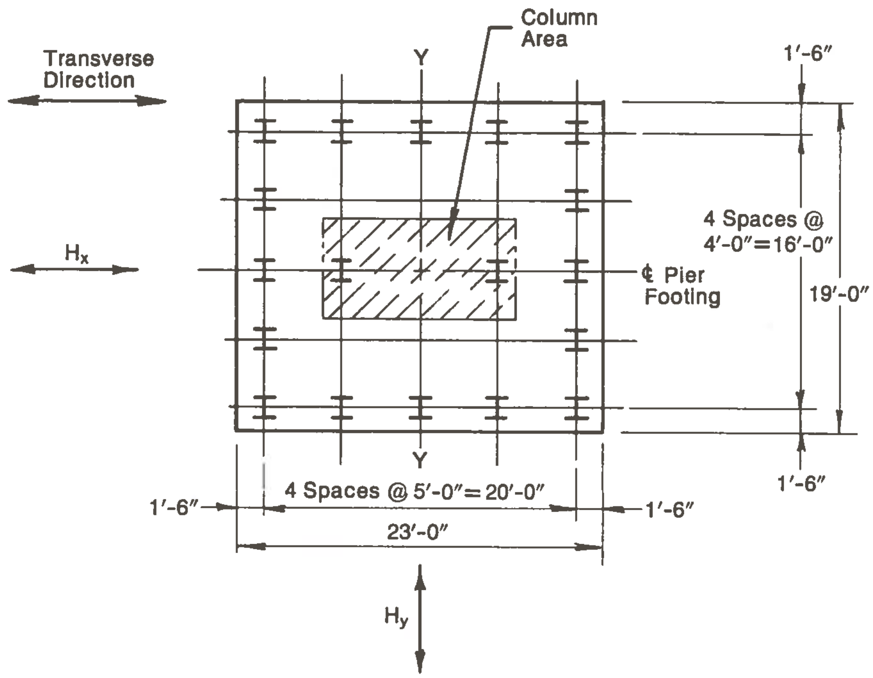
Since the depth of ground water is greater than $1.5d = 1.7$ ft below the pile tip, it will not influence the depth of pile penetration.

FOOTING SIZE AND PILE PATTERN

Estimate the required number of piles by assuming that the vertical load will cause one-half of the total pile load.

$$\text{No. of piles} = \frac{1727}{193.5} \times 2 \approx 18 \text{ piles}$$

Assume the footing dimensions and pile pattern shown. The piles are concentrated near the periphery of the footing to resist overturning moments.



Moment of Inertia of Pile Group

$$\Sigma x^2 = 5 \times 2(10.0)^2 + 2 \times 3(5.0)^2 = 1150 \text{ pile-ft}^2$$

$$\Sigma y^2 = 5 \times 2(8.0)^2 + 2 \times 2(4.0)^2 = 704 \text{ pile-ft}^2$$

FORCES AT TOP OF PILES

Transfer the forces and moments acting at the top of the footing down to the top of the piles. To these forces add the weight of the footing and the soil on top of the footing. Since the footing and piles are below the ground surface, impact is deducted from the loading.

Group I—Dead Load+Live Load

Loading	F_v (kips)	Transverse Direction		Longitudinal Direction	
		H_x (kips)	M_y (kip-ft)	H_y (kips)	M_x (kip-ft)
Forces at top of footing	1727	—	830	—	—
Impact	−45	—	−150	—	—
Wt. of footing	393	—	—	—	—
Wt. of soil	81	—	—	—	—
Total Group I	2156	—	680	—	—

Group II—Dead Load+Wind

Loading	F_v (kips)	Transverse Direction		Longitudinal Direction	
		H_x (kips)	M_y (kip-ft)	H_y (kips)	M_x (kip-ft)
Forces at top of footing	1476	71.5	4161	43.3	1910
Wt. of footing	393	—	—	—	—
Wt. of soil	81	—	—	—	—
$H_x(5.0)=71.5\times5.0$	—	—	358	—	—
$H_y(5.0)=43.3\times5.0$	—	—	—	—	217
Total Group II	1950	71.5	4519	43.3	2127

Group III—Dead Load+Live Load+Impact+Traction+0.3 Wind+Wind on Live Load+Centrifugal Force+Friction

Loading	F_v (kips)	Transverse Direction		Longitudinal Direction	
		H_v (kips)	M_y (kip-ft)	H_y (kips)	M_x (kip-ft)
Forces at top of footing	1727	73.5	5795	92.7	5015
Impact	−45	—	−150	—	—
Wt. of footing	393	—	—	—	—
Wt. of soil	81	—	—	—	—
$H_x(5.0)=73.5\times5.0$	—	—	368	—	—
$H_y(5.0)=92.7\times5.0$	—	—	—	—	464
Total Group III	2156	73.5	6013	92.7	5479

By inspection Group III loading with an allowable stress increase of 25% will govern.

PILE LOADING

$$Q_m=\frac{F_v}{m}\pm\frac{M_yx}{\Sigma x^2}\pm\frac{M_xy}{\Sigma y^2} \qquad \text{(Eq. B)}$$

Max. pile load, $Q_m=\frac{2156}{18}+\frac{6013\times10.0}{1150}+\frac{5479\times8.0}{704}$

$=119.8+52.3+62.3$

$=234.4$ kips

Min. pile load, $Q_m=119.8-52.3-62.3$

$=+5.2$ kips $\quad\therefore$ No uplift occurs

The pile pattern and footing size selected are satisfactory.

DETERMINE IF BATTERED PILES ARE REQUIRED

The limited data available indicates that the lateral capacity of a pile group is approximately equal to the capacity of a single free-headed pile times the number of piles in the group. Also in a multistrata condition the surface layer governs the lateral load. Assume a lateral movement of ¼ inch at the ground surface is permissible. From Fig. 25, the allowable lateral load for a single free-headed HP 14×73 pile in sand with ϕ equal to 30° is 9.0 kips per pile perpendicular to the pile flange and $\frac{2}{3} \times 9.0$ or 6.0 kips per pile parallel to the pile flange.

The allowable lateral load parallel to the x-axis is:

$$18 \times 6.0 \times 1.25 = 135 \text{ k} \gg 73.5 \text{ kips}$$

The allowable lateral load parallel to the y-axis is:

$$18 \times 9.0 \times 1.25 = 202 \text{ k} \gg 92.7 \text{ kips}$$

Therefore, battered piles are not required.

SETTLEMENT AT TOP OF PILES

The settlement of a pile group is greater than the settlement of an individual pile for the same unit pile loading. Fig. 12 relates the ratio between the settlement of a pile group and the settlement of a single pile to the width of the foundation.

$$\text{Avg. width of foundation} = \frac{23.0 + 19.0}{2} = 21.0 \text{ ft}$$

From Fig. 12:

$$\frac{\text{Foundation Settlement}}{\text{Single Pile Settlement}} = 7.6$$

Calculate the settlement due to dead load plus live load. Secondary loads such as wind, traction, friction and centrifugal force act for only short periods of time and cause unequal pile loads, resulting in a slight rotation of the footing.

$$\therefore \text{Pile reaction for } DL,\ Q_m = \frac{2156 - 206}{18} = 108.3 \text{ k/pile}$$

$$\text{and Pile reaction for } LL,\ Q_m = \frac{206 \text{ k}}{18} = 11.4 \text{ k/pile}$$

Using the average curve in Fig. 12, determine the single pile settlement for dead and live loads.

$$\frac{\text{Pile dead load}}{\text{Ultimate load}} = \frac{108.3}{483.8} = 0.223 \text{ and } \Delta = 0.06 \text{ in.}$$

$$\frac{\text{Pile live load}}{\text{Ultimate load}} = \frac{11.4}{483.8} = 0.023 \text{ and } \Delta = 0$$

Settlement of Pile Group at Pile Tip

Dead Load: $\Delta_T = 0.06 \times 7.6 = 0.456$ in.

Live Load: $\Delta_T = 0$ $\qquad 0$

Total $= 0.456$ in. $\approx \frac{7}{16}$ in.

Elastic Deformation

In the analysis for the depth of penetration, the ultimate end bearing component is:

$$Q_{ue} = 262.4 + 17.52 D_2 = 262.4 + 17.52(10.6)$$
$$= 448.0 \text{ kips}$$

$$\% \text{ of ultimate load to pile tip} = \frac{448.0}{483.8} = 92.6\%$$

The approximate dead and live loads reaching the pile tip are:

D.L.: $Q = 108.3 \times 0.926 = 100.3$ kips

L.L.: $Q = 11.4 \times 0.926 = 10.6$ kips

The loads carried by skin friction are:

$$\text{D.L.: } Q = 108.3 - 100.3 = 8.0 \text{ kips}$$
$$\text{L.L.: } Q = 11.4 \; - 10.6 \; = 0.8 \text{ kips}$$

The average pile loads causing elastic deformation are:

$$\text{D.L.: } Q = 100.3 - \frac{8.0}{2} = 96.3 \text{ kips}$$
$$\text{L.L.: } Q = \; 10.6 - \frac{0.8}{2} = 10.2 \text{ kips}$$

The deformation due to dead load is:

$$\Delta = \frac{96.3 \times 26.0 \times 12.0}{21.5 \times 29 \times 10^3} = 0.048 \text{ in.}$$

The deformation due to live load is:

$$\Delta = \frac{10.2 \times 26.0 \times 12.0}{21.5 \times 29 \times 10^3} = \underline{0.005} \text{ in.}$$
$$\text{Total } \Delta = 0.053 \text{ in.} \cong 1/16 \text{ in.}$$

The total settlement at top of piles ≈ ½ in.

FORCES AT TOP OF FOOTING

Refer to Part A.

PILE SIZE

High capacity piles concentrated near the periphery of a footing provide an economical foundation when large loads and large moments must be supported.

High capacity piles should be used only if the soil is capable of supporting the high loads delivered by each pile. High pile loads are more difficult to develop in fine-grained material than in coarse-grained material.

A "medium capacity" pile—an HP 12×53 stressed to 9 ksi will be used in this design example.

Allowable load for a single pile, $Q = 15.6 \times 9.0 = 140.4$ kips.

Pile Properties

$$A_s = 15.6 \text{ in.}^2 = 0.108 \text{ ft}^2$$
$$d = 11.78 \text{ in.} = 0.982 \text{ ft}$$
$$b_f = 12.046 \text{ in.} = 1.004 \text{ ft}$$
$$p = 2(d + b_f) = 3.97 \text{ ft}$$
$$A = db_f = 0.986 \text{ ft}^2$$

DEPTH OF PENETRATION

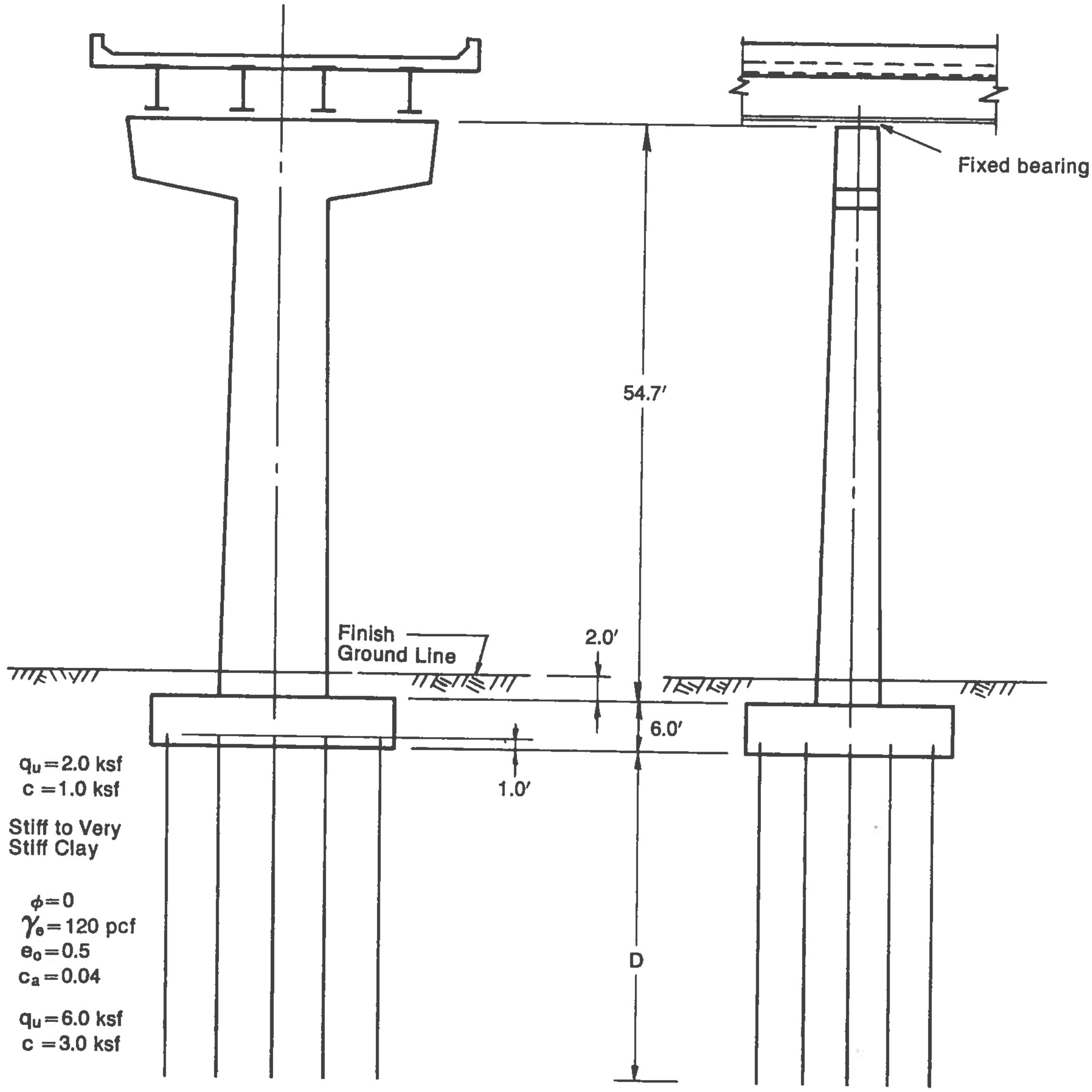
Determine the depth of penetration required to develop an allowable load of 140 kips on an HP 12×53 pile. Assume that single pile action controls the required depth of penetration. After the pile pattern has been established, determine if group action governs.

Determine Penetration for a Single Pile

For H-piles in fine-grained material, the end bearing capacity is negligible. Therefore, the ultimate bearing capacity is equal to the skin friction component (Q_{us}). The lesser of the Q_{us} values found by equations 10, 11 and 12 determines the depth of penetration.

Cohesion Value

$$\text{Use Avg. } c = \frac{1.0 + 3.0}{2} = 2.0 \text{ ksf}$$



PART B—PILE GROUPING IN FINE-GRAINED MATERIAL

Adhesion Value

Using Fig. 9, for $c=2.0$, $\frac{c_a}{c}=0.35$

$\therefore c_a=0.35\times 2.0=0.70$ ksf

a. Cohesion around the net perimeter

$$Q_{us}=2(d+b_f)cD \qquad \text{(Eq. 10)}$$
$$=2(0.982+1.004)2.0D$$
$$=7.94D$$

b. Adhesion around the entire perimeter

$$Q_{us}=2(d+2b_f)\,c_aD \qquad \text{(Eq. 11)}$$
$$=2[(0.982+(2\times 1.004)]0.70D$$
$$=4.19D \qquad \textit{Governs}$$

c. Adhesion to the flange and cohesion across the web opening

$$Q_{us}=2(dc+b_f\,c_a)D \qquad \text{(Eq. 12)}$$
$$=2[(0.982\times 2.0)+(1.004\times 0.70)]D$$
$$=5.33D$$

Adhesion around the entire perimeter governs. Using a safety factor = 2.5,

$$D = \frac{140\ \mathrm{k} \times 2.5}{4.19}$$

$$D = 83.5\ \mathrm{ft};\ \text{Use } 84.0\ \mathrm{ft}$$

This depth of pile penetration should be verified by a pile load test.

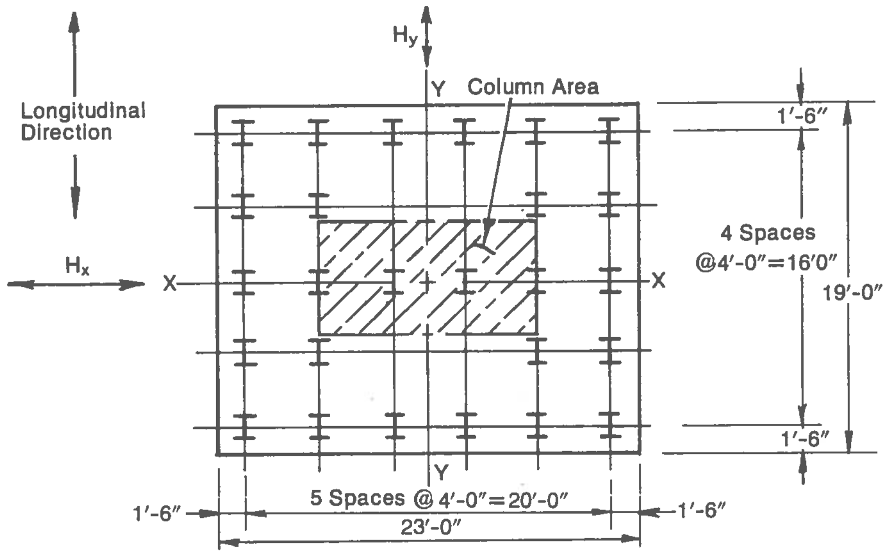
Check for group action after determining the footing size, number of piles and pile spacing.

FOOTING SIZE AND PILE PATTERN

Estimate the required number of piles by assuming that the vertical load will cause one-half of the total pile load.

$$\text{No. of piles} = \frac{1727}{140.4} \times 2 = 25\ \text{piles}$$

Assume the footing dimensions and pile pattern shown.



Moment of Inertia of Pile Group

$$\Sigma x^2 = 2 \times 5(10.0)^2 + 2 \times 5(6.0)^2 + 2 \times 3(2.0)^2 = 1384\ \text{pile-ft}^2$$

$$\Sigma y^2 = 2 \times 6(8.0)^2 + 2 \times 4(4.0)^2 = 896\ \text{pile-ft}^2$$

FORCES AT TOP OF PILES

Transfer the forces and moments acting at the top of the footing down to the top of the piles. To these forces, add the weight of the footing and the soil on top of the footing. Since the footing and piles are below the ground surface, impact is deducted from the loading.

Group I—Dead Load + Live Load

Loading	F_v (kips)	Transverse Direction		Longitudinal Direction	
		H_x (kips)	M_y (kip-ft)	H_y (kips)	M_x (kip-ft)
Forces at top of footing	1727	—	830	—	—
Impact	−45	—	−150	—	—
Wt. of footing	393	—	—	—	—
Wt. of soil	81	—	—	—	—
Total Group I	2156	—	680	—	—

Group II—Dead Load+Wind

Loading	F_v (kips)	Transverse Direction		Longitudinal Direction	
		H_x (kips)	M_y (kip-ft)	H_y (kips)	M_x (kip-ft)
Forces at top of footing	1476	71.5	4161	43.3	1910
Wt. of footing	393	—	—	—	—
Wt. of soil	81	—	—	—	—
$H_x(5.0)=71.5\times5.0$	—	—	358	—	—
$H_y(5.0)=43.3\times5.0$	—	—	—	—	216
Total Group II	1950	71.5	4519	43.3	2126

Group III—Dead Load+Live Load+Friction+Traction+0.3 Wind+Wind on Live Load+Centrifugal Force

Loading	F_v (kips)	Transverse Direction		Longitudinal Direction	
		H_x (kips)	M_y (kip-ft)	H_y (kips)	M_x (kip-ft)
Forces at top of footing	1727	73.5	5795	92.7	5015
Impact	−45	—	−150	—	—
Wt. of footing	393	—	—	—	—
Wt. of soil	81	—	—	—	—
$H_x(5.0)=73.5\times5.0$	—	—	368	—	—
$H_y(5.0)=92.7\times5.0$	—	—	—	—	464
Total Group III	2156	73.5	6018	92.7	5479

By inspection Group III loading with an allowable stress increase of 25% will govern.

PILE LOADING

$$Q_m=\frac{F_v}{r}\pm\frac{M_y x}{\Sigma x^2}\pm\frac{M_x y}{\Sigma y^2} \qquad \text{(Eq. B)}$$

$$\text{Max. pile load, } Q_m=\frac{2156}{26}+\frac{6018\times10}{1384}+\frac{5479\times8.0}{896}$$
$$=82.9+43.5+48.9$$
$$=175.3<140.4\times1.25=175.5 \text{ kips}$$

$$\text{Min. pile load, } Q_m=82.9-43.5-48.9$$
$$=-9.5 \text{ kips} \quad \text{uplift}$$

Determine if the piles can resist this amount of uplift.

ULTIMATE PULL OUT CAPACITY

The short term uplift capacity of a friction pile in fine-grained material is equal to the skin friction on the pile. The long term uplift capacity is sometimes taken as 0.7

of the skin friction. However, AASHO limits the allowable uplift to 0.4 of the allowable compressive load.

$$\therefore \text{Allowable uplift} = 0.4 \times 140.4 \times 1.25 = 70.2 \text{ kips} > 9.5 \text{ kips actual}$$

The footing dimensions and pile pattern selected are satisfactory.

DETERMINE IF BATTERED PILES ARE REQUIRED

The limited data available indicates that the lateral capacity of a pile group is approximately equal to the capacity of a single free-headed pile times the number of piles in the group. Assume a lateral movement of ¼ inch is permissible. From Fig. 27 the allowable lateral load for a single free-headed HP 12×53 pile in stiff clay is 5 kips per pile for short term loading and 2 kips per pile for long term loading. Since horizontal loads on this pier are secondary forces, use the short term allowable load of 5 kips perpendicular to the pile flange and ¾×5.0 or 3.75 kips per pile parallel to the pile flange.

The allowable lateral load parallel to the x-axis is:

$$(26 \times 3.75)1.25 = 122 \text{ kips} > 73.5 \text{ kips}$$

The allowable lateral load perpendicular to the y-axis is:

$$(26 \times 5.0)1.25 = 162 \text{ kips} > 92.7 \text{ kips}$$

Therefore, battered piles are not required.

CAPACITY OF THE PILE GROUP

The ultimate capacity of the pile group,

$$Q_u = Q_{ue} + Q_{us}$$

where $Q_{ue} = 9c\,(BL)$ (Eq. 18)

$Q_{us} = 2D(B+L)c$ (Eq. 17)

$$Q_u = 9c\,(BL) + 2D(B+L)\,c$$

where $B = 20 + 0.982 = 20.982$ ft, say 21.0 ft

$L = 16 + 1.004 = 17.004$ ft, say 17.0 ft

$$Q_u = 9 \times 3.0(21.0+17.0) + 2 \times 86.0(21.0+17.0)2.0$$

$$Q_u = 1026 + 6536 = 7556 \text{ kips}$$

Applying a factor of safety of 2.5, the allowable vertical load on the pile group

$$= 7562/2.5$$
$$= 3025 \text{ kips}$$

The maximum load on the pile group is 2156 kips. No reduction in pile capacity is required for group action.

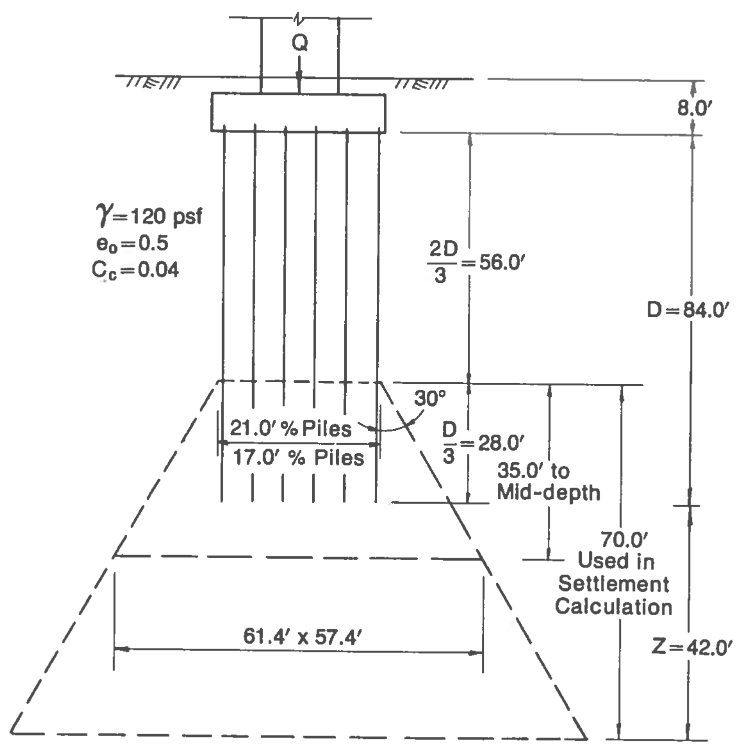
SETTLEMENT

Settlement of a pile group in a fine-grained material may be approximated by assuming that the imposed loading is applied as a uniform pressure within the perimeter of the group, at a level ⅔ down the pile penetration. Determine settlement for DL and LL.

Settlement is determined for a soil layer beginning a distance Z below the pile tip and extending upward from the pile tip a distance $D/3$. The distance Z is determined by assuming that settlement will occur to a depth of twice the largest group width below the pile tip.

$$\therefore Z = 2 \times 21.0 \text{ ft} = 42.0 \text{ ft}$$

$$\Delta_T = \frac{C_c(D/3+Z)}{1+e_o}\left[\log\left(\frac{q_o+\Delta_q}{q_o}\right)\right]$$



where q_o = existing effective overburden pressure at mid-depth of the soil layer in which settlement is considered.

Δ_q = Pressure due to Q at mid-depth of the soil layer in which settlement is considered.

C_c = compression index

e_o = void ratio

From laboratory tests the following values were found.

$$e = 0.5 \text{ and } C_c = 0.04$$

Determine Existing Overburden Pressure

$$q_o = \gamma_e\left[\frac{2}{3}D + \frac{\left(\frac{D}{3}+Z\right)}{2} + 8.0\right]$$
$$= 0.12(58.0+35.0+8.0) = 0.12[101.0]$$
$$= 12.12 \text{ ksf}$$

Determine Change In Pressure Due To Pile Loading

a. Dead load, $Q = 2156 \text{ kips} - 206 \text{ kips} = 1950 \text{ kips}$

$$\Delta_q = \frac{1950}{61.4 \times 57.4} = 0.553 \text{ ksf}$$

Consolidation Settlement

$$\Delta_T=\frac{0.04(70.0)}{1+0.5}\left(\log\frac{12.12+0.553}{12.12}\right)=1.866(\log 1.045)$$
$$=1.866(0.01912)=0.036\text{ ft}$$
$$=0.432\text{ in.}$$

Settlement Due To Elastic Deformation Of The Pile

Average pile load (over length of pile) $=\frac{1950\text{ kips}}{26}\times\frac{1}{2}=37.5$ kips

$$\Delta=\frac{37.5\times 84.0\times 12}{15.6\times 29\times 10^3}=0.083\text{ in.}$$

Total D.L. settlement $=0.432+0.083=0.515$ in. $=\frac{1}{2}$ in.

b. Live Load, $Q=206$ kips

$$\Delta_q=\frac{206}{61.4\times 57.4}=0.058\text{ ksf}$$

Consolidation Settlement

$$\Delta_T=\frac{0.04(70.0)}{1+0.5}\left(\log\frac{12.12+0.058}{12.12}\right)=1.866(1.004)$$
$$=1.866(0.00173)=0.0032\text{ ft}$$
$$=0.039\text{ in.}$$

Settlement Due To Elastic Deformation Of The Pile

$$\text{Average pile load}=\frac{206}{26\times 2}=3.96$$

$$\Delta=\frac{3.96\times 84.0\times 12}{15.6\times 29\times 10^3}=0.008\text{ in.}$$

Total LL settlement $=0.039+0.008=0.047$ in. $\approx\frac{1}{16}$ in.
Total $DL+LL$ settlement $=\frac{9}{16}$ in.

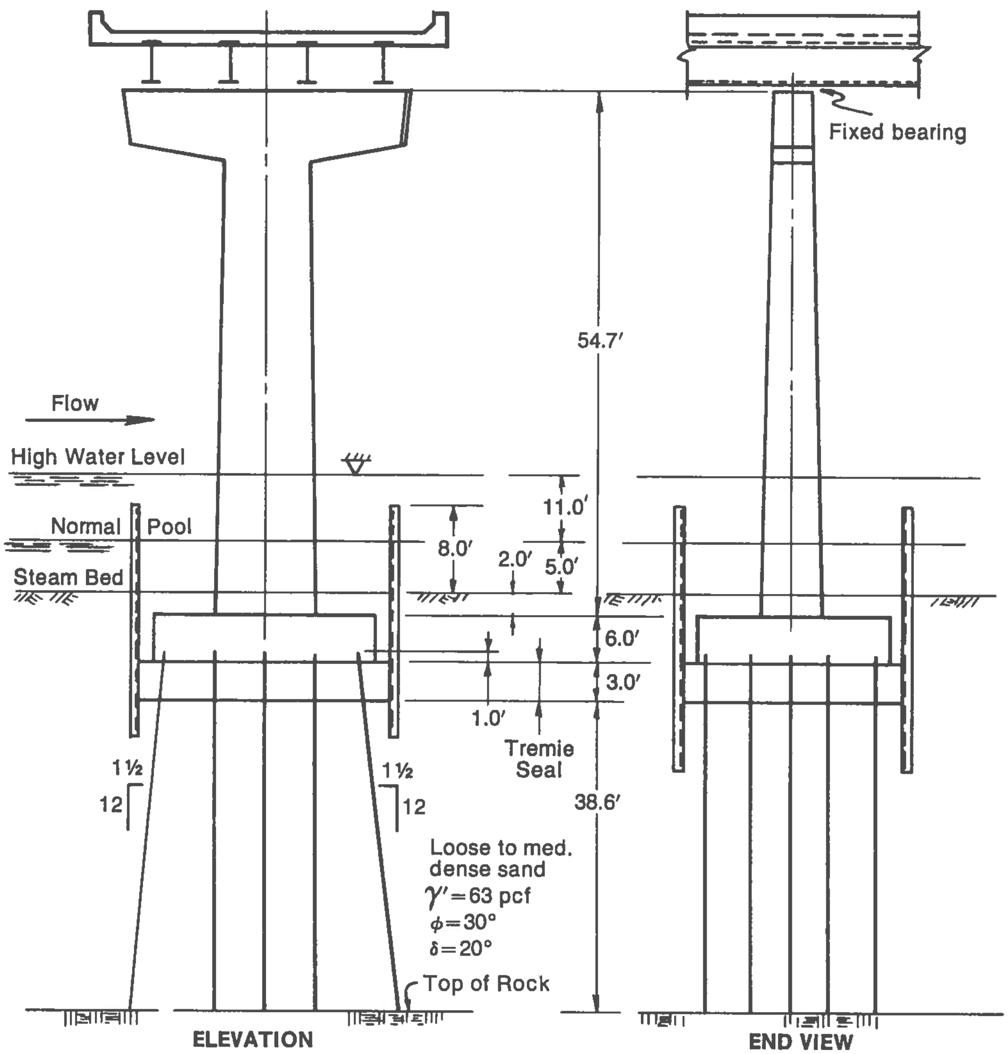
The above magnitude of calculated settlements should be regarded as only a rough approximation. However, it can be used to judge the adequacy of the foundation.

Example 5—Bridge Pier in Water (with Tremie Concrete Seal)

This design is typical for a piled foundation placed in a shallow stream. Steel sheet piling is driven to form a cofferdam, and the area inside the cofferdam is excavated underwater. Bearing piles are driven, and tremie concrete is placed underwater. After the tremie concrete has gained sufficient strength, the cofferdam is dewatered and the pier footing is constructed in the dry.

Tremie seals generally are at least 5 feet thick so as to permit the tremie concrete to flow and cover the full plan area of the cofferdam without excessive movement of the tremie pipe. The tremie seal is made slightly larger than the footing to simplify placing of footing form work and to catch and drain any water seeping into the cofferdam.

Piles are designed as rock bearing in this example. However, the design procedure for friction piles is similar with added steps to determine the depth of penetration.



FORCES AT TOP OF FOOTING

Determine forces at the top of the footing. These forces are used to design the column section and then to estimate the footing size and pile pattern. Loads for AASHO Groups I, II, III, VIII and IX are tabulated.

Group I—Dead Load+Live Load+Impact+Stream Flow

Loading		F_v (kips)	Transverse Direction		Longitudinal Direction	
			H_x (kips)	M_z (kip-ft)	H_y (kips)	M_x (kip-ft)
D.L. of Superstructure		966	—	—	—	—
D.L. of Pier		510	—	—	—	—
L.L.		206	—	680	—	—
Impact		45	—	150	—	—
Stream Flow	Normal Pool	—	2.8	12	—	—
	High Water	—	8.8	88	—	—
Total Group I	Normal Pool	1727	2.8	842	—	—
	High Water	1727	8.8	918	—	—

Group II—Dead Load+Stream Flow+Wind

Loading		F_v (kips)	Transverse Direction		Longitudinal Direction	
			H_x (kips)	M_y (kip-ft)	H_y (kips)	M_x (kip-ft)
D.L. of Superstructure		966	—	—	—	—
D.L. of Pier		510	—	—	—	—
Wind on Superstructure		—	63.0	3930	15.0	951
Stream Flow	Normal Pool	—	2.8	12	—	—
	High Water	—	8.8	88	—	—
Wind on Pier	Normal Pool	—	7.6	220	26.3	910
	High Water	—	5.9	203	21.9	873
Total Group II	Normal Pool	1476	73.4	4162	41.3	1861
	High Water	1476	77.7	4221	36.9	1824

Group III—Dead Load+Live Load+Impact+Stream Flow+0.3 Wind +Wind on Live Load+Traction+Centrifugal Force+Friction

Loading		F_v (kips)	Transverse Direction		Longitudinal Direction	
			H_x (kips)	M_z (kip-ft)	H_y (kips)	M_x (kip-ft)
D.L. of Superstructure		966	—	—	—	—
D.L. of Pier		510	—	—	—	—
L.L.		206	—	680	—	—
Impact		45	—	150	—	—
Stream Flow	Normal Pool	—	2.8	12	—	—
	High Water	—	8.8	88	—	—
0.3 Wind on Superstructure		—	18.9	1180	4.5	286
0.3 Wind on Pier	Normal Pool	—	2.3	66	7.9	273
	High Water	—	1.8	61	6.6	262
Wind on Live Load		—	10.0	716	4.0	287
Centrifugal Force		—	42.0	3000	—	—
Friction		—	—	—	48.3	2652
Traction		—	—	—	27.4	1502
Total Group III	Normal Pool	1727	76.0	5804	92.1	5000
	High Water	1727	81.5	5875	90.8	4989

Group VIII—Group I+Ice

Loading	F_v (kips)	Transverse Direction		Longitudinal Direction	
		H_x (kips)	M_y (kip-ft)	H_y (kips)	M_x (kip-ft)
Group I at Normal Pool	1727	2.8	842	—	—
Ice	—	115.2	806	—	—
Total Group VIII	1727	118.0	1648	—	—

Group IX—Group II+Ice

Loading	F_v (kips)	Transverse Direction		Longitudinal Direction	
		H_x (kips)	M_z (kip-ft)	H_y (kips)	M_x (kip-ft)
Group II at Normal Pool	1476	73.4	4162	41.3	1861
Ice	—	115.2	806	—	—
Total Group IX	1476	188.6	4968	41.3	1861

Group III loading with an allowable stress increase of 25% will govern for maximum pile load and for maximum uplift. Group IX loading with an allowable stress increase of 50% will govern for lateral load design and battering of the piles.

PILE SIZE

AASHO Specifications permit pile stresses higher than 9000 psi if substantiated by field load tests. For this design, assume that a load test is performed and the allowable load is based on a pile steel stress of 12,000 psi. Since it is more economical to drive fewer large piles than many small piles, select an HP 14×73 pile section.

Allowable load per pile $= 21.5 \text{ in.}^2 \times 12.0 \text{ ksi} = 258.0$ k/pile

Pile Properties

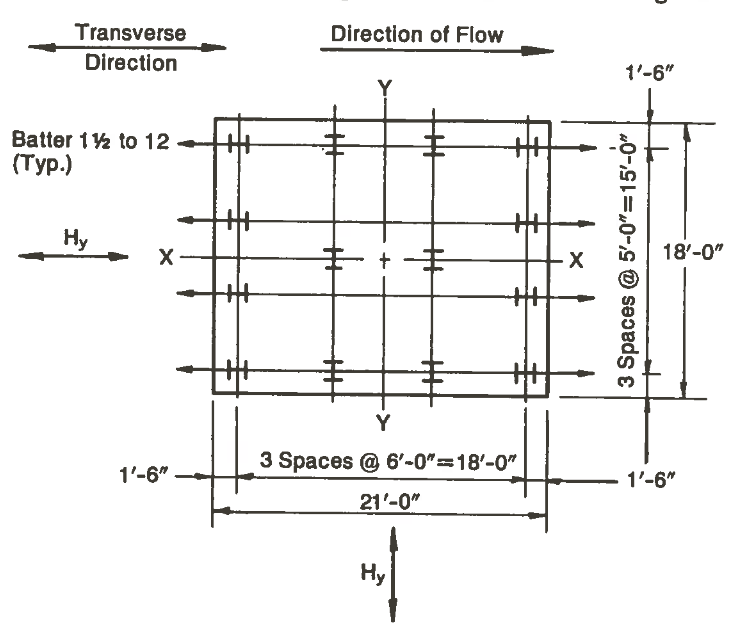
$$A_s = 21.5 \text{ in.}^2 = 0.149 \text{ ft}^2$$
$$d = 13.64 \text{ in.} = 1.13 \text{ ft}$$
$$b_f = 14.536 \text{ in.} = 1.21 \text{ ft}$$
$$p = 2(d + b_f) = 4.68 \text{ ft}$$

FOOTING SIZE AND PILE PATTERN

Estimate the required number of piles by assuming that the vertical load will cause one-half of the total pile load.

$$\text{No. of piles} = \frac{1727}{2580} \times 2 \cong 14 \text{ piles}$$

Assume the footing dimensions and pile pattern shown. The piles are concentrated near the periphery of the footing to resist the overturning moments.



Moment of Inertia of Pile Group

$$\Sigma x^2 = 2\times 4)9)^2 + 2\times 3(3)^2 = 648 + 54 = 702 \text{ pile-ft}^2$$

$$\Sigma y^2 = 2\times 4(7.5)^2 + 2\times 2(2.5)^2 = 450 + 25 = 475 \text{ pile-ft}^2$$

FORCES AT TOP OF PILES

Transfer the forces and moments acting at the top of the footing for Groups III and IX loading down to the top of the piles. To these forces add the weight of the footing, the weight of the tremie seal, and the saturated weight of the soil and water acting on the footing and tremie seal. Upward buoyant forces for the various river stages are considered to act on the bottom of the tremie seal.

Since the footing and piles are below the ground surface, impact is deducted from the loading.

To determine the above weights assume the tremie seal to be 23×20×5 ft thick.

Group III—Dead Load+Live Load+Friction+Stream Flow+0.3 Wind +Wind on Live Load+Traction+Centrifugal Force+Buoyancy at Normal Pool

Loading	F_v (kips)	Transverse Direction		Longitudinal Direction	
		H_x (kips)	M_y (kip-ft)	H_y (kips)	M_x (kip-ft)
Forces at top of footing— Water at Normal Pool	1727	76.0	5804	92.1	5000
Impact	−45	—	−150	—	—
Wt. of footing	340	—	—	—	—
Wt. of tremie seal	345	—	—	—	—
Wt. of sat. soil	154	—	—	—	—
Wt. of water	123	—	—	—	—
Buoyancy	−515	—	—	—	—
$H_x(5.0) = 76.0\times 5.0$	—	—	380	—	—
$H_y(5.0) = 93.1\times 5.0$	—	—	—	—	461
Total Group III at Normal Pool	2129	76.0	6034	92.1	5461

Group III—Dead Load+Live Load+Stream Flow+0.3 Wind+Wind on Live Load+Traction+Centrifugal Force+Friction+Buoyancy at High Water

Loading	F_v (kips)	Transverse Direction		Longitudinal Direction	
		H_x (kips)	M_y (kip-ft)	H_y (kips)	M_x (kip-ft)
Forces at top of footing— at High Water	1727	81.5	5875	90.8	4989
Impact	−45	—	−150	—	—
Wt. of footing	340	—	—	—	—
Wt. of tremie seal	345	—	—	—	—
Wt. of water	394	—	—	—	—

(Continued)

(Table Continued)

Group III—Dead Load+Live Load+Stream Flow+0.3 Wind+Wind on Live Load+Traction+Centrifugal Force+Friction+Buoyancy at High Water

Loading	F_v (kips)	Transverse Direction		Longitudinal Direction	
		H_x (kips)	M_y (kip-ft)	H_y (kips)	M_x (kip-ft)
Wt. of sat. soil	154	—	—	—	—
Buoyancy	−830	—	—	—	—
$H_x(5.0) = 31.5 \times 5.0$	—	—	418	—	—
$H_y(5.0) = 90.8 \times 5.0$	—	—	—	—	454
Total Group III at High Water	2085	81.5	6143	90.8	5443

Group IX—Dead Load+Stream Flow+Wind+Ice+Buoyancy at Normal Pool

Loading	F_v (kips)	Transverse Direction		Longitudinal Direction	
		H_x (kips)	M_y (kip-ft)	H_y (kips)	M_x (kip-ft)
Forces at top of footing—at Normal Pool	1476	188.6	4963	41.3	1861
Wt. of footing	340	—	—	—	—
Wt. of tremie seal	345	—	—	—	—
Wt. of sat. soil	154	—	—	—	—
Wt. of water	123	—	—	—	—
Buoyancy	−515	—	—	—	—
$H_x(5.0) = 188.6 \times 5.0$	—	—	943	—	—
$H_y(5.0) = 41.3 \times 5.0$	—	—	—	—	206
Total Group IX at Normal Pool	1923	188.6	5911	41.3	2067

PILE LOADING

$$Q_m = \frac{F_v}{r} \pm \frac{M_y x}{\Sigma x^2} \pm \frac{M_x y}{\Sigma y^2} \qquad \text{(Eq. B)}$$

Group III at Normal Pool

Max. pile load, $Q_m = \frac{2129}{14} + \frac{6034 \times 9.0}{702} + \frac{5461 \times 7.5}{475}$

$= 152.1 + 77.4 + 86.3$

$= 315.8$ kips $< 258 \times 1.25 = 322.5$ kips

Governs for max. pile load

Min. pile load, $Q_m = 152.1 - 77.4 - 86.3$

$= -11.6$ kips

Group III at High Water

$$\text{Max. pile load, } Q_m = \frac{2085}{14} + \frac{6143 \times 9.0}{702} + \frac{5443 \times 7.5}{475}$$
$$= 148.9 + 78.8 + 85.9$$
$$= 313.6 \text{ kips} < 258 \times 1.25 = 322.5 \text{ kips}$$

$$\text{Min. pile load, } Q_m = 148.9 - 78.8 - 85.9$$
$$= -15.8 \text{ kips}$$

Governs for max. uplift

Group IX

$$\text{Max. pile load, } Q_m = \frac{1924}{14} + \frac{5911 \times 9.0}{702} + \frac{2067 \times 7.5}{475}$$
$$= 137.4 + 75.8 + 32.6$$
$$= 245.8 \text{ kips} \ll 259 \times 1.5 = 387.0 \text{ kips}$$

$$\text{Min. pile load, } Q_m = 137.4 - 75.9 - 32.6$$
$$= +29.0 \text{ kips}$$

ULTIMATE PULL OUT CAPACITY

The ultimate pull out capacity of a pile is equal to the skin friction developed between the pile and soil plus the weight of the pile. The skin friction value is:

$$Q_{us} = \tfrac{1}{2} p K_b \gamma_s D^2 \tan \delta \qquad \text{(Eq. 4)}$$

Using Fig. 8 and $\phi = 30°$, $K_b = 0.40$

$$Q_{us} = \tfrac{1}{2} \times 4.68 \times 0.40 \times 0.063 (38)^2 \times \tan 20°$$
$$= 30.9 \text{ kips}$$

Pile weight $= 0.073$ lb/ft $\times 38$ ft $= 2.77$ kips

$$Q_{uo} = 30.9 + 2.77 = 33.67 \text{ kips}$$

Using a safety factor $= 2.5$

$$\text{Allowable uplift} = \frac{33.67}{2.5} = 13.5 \text{ kips}$$

For Group III Loading at High Water

Uplift force $= 15.8 < 13.5 \times 1.25 = 16.8$ kips

$\therefore$ The footing size and pile pattern are satisfactory.

DETERMINE IF BATTERED PILES ARE REQUIRED

The limited data available indicates that the lateral capacity of a pile group is approximately equal to the capacity of a single free-headed pile times the number of piles in the group. Assume a lateral movement of ¼ inch is permissible. From Fig. 26 the allowable lateral load for a single free-headed HP 14×73 pile in saturated sand with ϕ equal to 30° is 7.0 kips per pile perpendicular to the pile flange and ⅔ × 7.0 or 4.67 kips per pile parallel to the pile flange.

The total allowable lateral load parallel to the x-axis is:

$$(8 \times 7.0) + (6 \times 4.67) = 84.0 \text{ kips}$$

The total allowable lateral load parallel to the y-axis is:

$$(8 \times 4.67) + (6 \times 7.0) = 79.3 \text{ kips}$$

Group III Loading at High Water

$$H_x = 81.5 < 84.0 \times 1.25 = 105.0 \text{ kips}$$

$$H_y = 90.8 < 79.3 \times 1.25 = \ \ 99.1 \text{ kips}$$

Group IX Loading

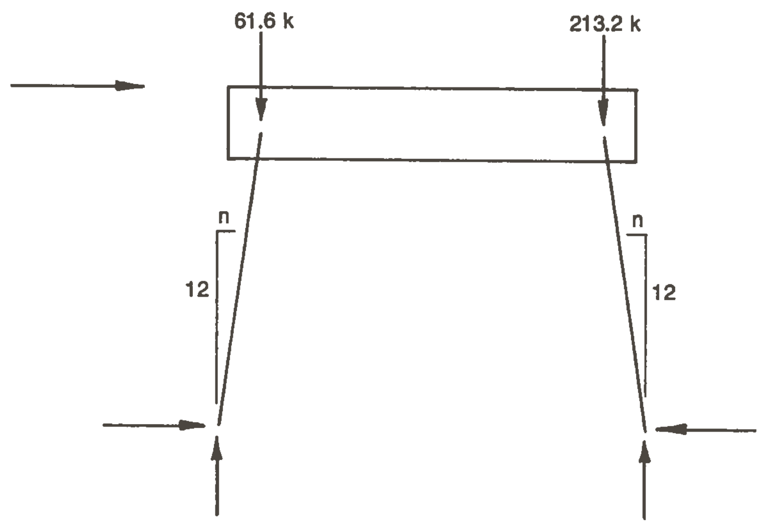
$$H_x = 188.6 > 84.0 \times 1.5 = 126.0 \text{ kips}$$

$$H_y = \ \ 41.3 < 79.3 \times 1.5 = 119.0 \text{ kips}$$

Therefore it is required to batter the outer row of piles parallel to the x-axis.

Large horizontal loads are applied to the pier infrequently. A symmetrical pattern of pile batter is provided so that the horizontal components of load in all battered piles sums up to approximately zero under the usual loading condition of vertical load only on the pier.

Batter four piles upstream and four piles downstream.



The load to be taken by the battered piles is:

$$\Delta H_x = 188.6 - 126.0 = 62.6 \text{ kips}$$

The average pile load on the downstream piles = 137.4 kips + 75.8 = 213.2 kips

The average pile load on the upstream piles = 137.4 − 75.8 = 61.6 kips

The pile batter can be specified as n horizontal : 12 vertical. The required pile batter can then be found by the following ratio:

$$\frac{n}{12} = \frac{62.6}{4(213.2 - 61.6)}$$

$$n = \frac{12 \times 62.6}{4 \times 151.6} = 1.238$$

Batter outer four piles, perpendicular to x-axis, 1½ horizontal: 12 vertical.
Thus the maximum axial pile load is:

$$\text{Max. } Q_m = 315.8 \times \frac{\sqrt{(1.5)^2 + (12)^2}}{12}$$

$$= 318.0 \text{ kips} < 258 \times 1.25 = 322.5 \text{ kips}$$

∴ Pile section is adequate

SETTLEMENT AT TOP OF PILES

Since the piles are driven to rock bearing, the only settlement will result from elastic deformation of the piles.

$$\text{Dead load per pile} = \frac{1476}{14} = 105.4 \text{ kips}$$

$$\Delta_{DL} = \frac{QL}{A_s E} = \frac{105.4 \times 38 \times 12}{21.5 \times 29 \times 10^3}$$
$$= 0.0771 \text{ in.} \approx 1/8 \text{ in.}$$

Generally the elastic deformation is small and need not be calculated.

TREMIE SEAL

The weight of the tremie seal plus the skin friction on the piles must resist the buoyant force on the bottom of the tremie seal. The critical condition for design of the tremie seal and resulting uplift on the piles occurs when the stream level is slightly below the top of the cofferdam.

$$\text{Buoyant Force} = 23 \times 20 \times 21 \times 0.0624 \text{ k/cf}$$
$$= 603 \text{ kips}$$

$$\text{Uplift to be resisted by piles} = 603 - 345$$
$$= 258 \text{ kips}$$

$$\text{Uplift/pile} = 258/14 = 18.4 \text{ kips}$$

$$\text{The ultimate pull out capacity} = 33.67 \text{ kips}$$

$$\therefore \text{Safety factor} = \frac{33.67}{18.4} = 1.83; \text{ adequate for temporary construction.}$$

Check Bond Between Pile and Tremie Seal

Section 1.5.1 (C)(3) AASHO Specification, specifies an allowable bond between pile and seal of 10 psi.

$$\text{Perimeter of pile} = (14.586 \times 4) + (13.64 \times 2)$$
$$= 85.62 \text{ in.}$$

$$\text{Allowable uplift on piles} = 85.62 \text{ in.} \times 5.0 \times 12 \times 0.01$$
$$= 51.4 \text{ kips} \gg 18.4 \text{ kips actual}$$

Thus the 5.0 ft seal thickness is adequate.