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Foundations & Retaining Structures

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Registered Structural Engineer—California

CIVIL ENGINEERING FOUNDATIONS & RETAINING STRUCTURES

Review for the Breadth/Depth Exam in Civil Engineering

Alan Williams, Ph.D., S.E., C.Eng.
Registered Structural Engineer-California

Engineering Press
Austin, TX

List of Knowledge clusters with percentage of questions	AM Test	PM Test				
PM Test Option Number		**1**	**2**	**3**	**4**	**5**
TRANSPORTATION KNOWLEDGE	20%	65%	5%			5%
Traffic Analysis	X	X				
Facility Design Criteria (policies & guidelines)		X	X			
Construction Techniques, Equipment, and.Materials	X	X	X			X
Geometric Design (Analytical Geometry)	X	X	X			
STRUCTURAL KNOWLEDGE	20%		65%			20%
Loadings	X		X			X
Analysis	X		X			
Mechanics of Materials	X		X			
Materials	X		X			X
Member Design	X		X			X
Failure Analysis	X		X			
Design Criteria; i.e. codes			X			
WATER RESOURCES KNOWLEDGE	20%	20%	10%	65%	25%	
Hydraulics	X	X	X	X	X	
Hydrology	X	X	X	X	X	
Meteorological and Climatological Data Analysis	X	X		X	X	
Water Treatment	X			X	X	
ENVIRONMENTAL KNOWLEGDE	20%			25%	65%	
Biology (including micro- and aquatic)	X			X	X	
Bacteriology	X			X	X	
Solid/Hazardous Waste	X				X	
Ground Water and Well Fields	X			X	X	X
GEOTECHNICAL KNOWLEDGE	20%	15%	20%	10%	10%	65%
Field Exploration and Laboratory Testing	X	X	X	X	X	X
Soil Mechanics Analysis	X	X	X	X	X	X
Foundation analysis	X	X	X	X	X	X
Retaining Structures	X		X			X
Total Percentage	100%	100%	100%	100%	100%	100%

The Proposed Breadth/Depth Exam

The Breadth/Depth examination format may be used as early as October 2000. At this writing the final details of the exam have not been announced, but the topic content of these exams is shown in the NCEES Appendix C table on the following page. The table shows the topic content of the exam. The morning exam will be the breadth of the topics with each topic equally weighted. The exam will be objectively scored by computer using A,B,C,D multiple choice answers. The test will probably consist of 40-60 single weight questions. The afternoon examination booklet will contain 5 exams, the candidate will choose one of the five options. In each option, a major topic consists of 65% of the option with the balance of the questions being of related topics. The afternoon exam will also be objectively scored by computer using 40-60 questions with multiple-choice answers.

Taking The Exam

Exam Dates

The National Council of Examiners for Engineering and Surveying (NCEES) prepares Civil Engineering Professional Engineer exams for use on a Friday in April and October each year. Some state boards administer the exam twice a year in their state, while others offer the exam once a year. The scheduled exam dates are:

	April	October
1999	23	29
2000	14	27
2001	20	26
2002	19	25

People seeking to take a particular exam must apply to their state board several months in advance.

4

Foundations and Retaining Structures

Alan Williams

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Isolated Column Spread Footing • Combined Footing • Strap Footing • Eccentric Footing • Footing with Eccentric Load

Retaining Wall 4-23
Cantilever Retaining Wall • Gravity Retaining Wall • Cantilevered Sheetpile Wall • Anchored Sheetpile Retaining Wall

Pile Foundations 4-35
Pile Group with Vertical Piles • Pile Group with Inclined Piles • Pile Cap Design

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Table 1-1. State Boards of Registration for Professional Engineers

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FOOTING DESIGN

Isolated Column Spread Footing

An isolated column footing transfers the loads from a single column to the supporting soil. The size of the footing is determined by the allowable soil bearing pressure. The footing is designed for flexure, punching or two-way shear, and flexural or one-way shear. The depth of the footing is generally governed by punching shear.

The ACI Code[1], in Sections 15.4 and 11.12, specifies the critical sections in the footing for flexure and shear. For a reinforced concrete column the reaction plane is defined in ACI Section 15.4.2 as being located at the face of the column. The location of the critical sections for flexure and shear are specified with respect to this plane and are illustrated in Fig. 4-1 (a–d). The length of the critical perimeter for punching shear is given by

$$b_o = 2(c_1 + c_2) + 4d$$

where c_1 = short side of column

c_2 = long side of column

d = effective depth of footing reinforcement.

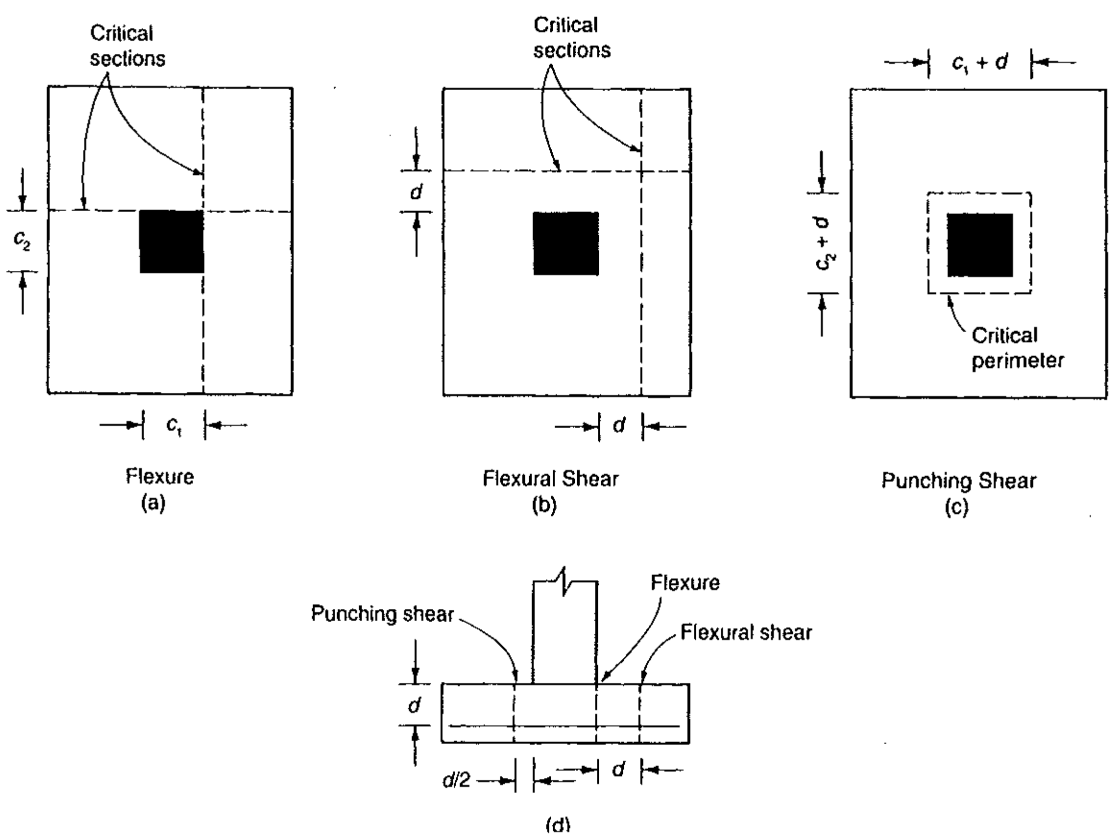


Fig. 4-1 (a–d). Critical sections: footing with reinforced concrete column

The reaction plane for a footing supporting a column with a steel base plate is specified as half-way between the face of the column and the edge of the base plate. The maximum moment is computed at this plane, with the flexural shear at a distance d from this plane, and the punching shear at a distance of $d/2$ from this plane, as shown in Fig. 4-2.

The reaction plane for a footing supporting a masonry column is specified as half way between the center and the face of the column. The maximum moment is computed at this plane, with the flexural shear at a distance d from this plane and the punching shear at a distance of $d/2$ from this plane as shown in Fig. 4-3.

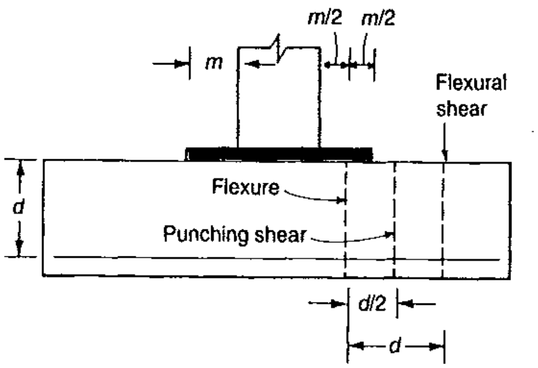


Fig. 4-2. Critical sections: footing with steel base plate

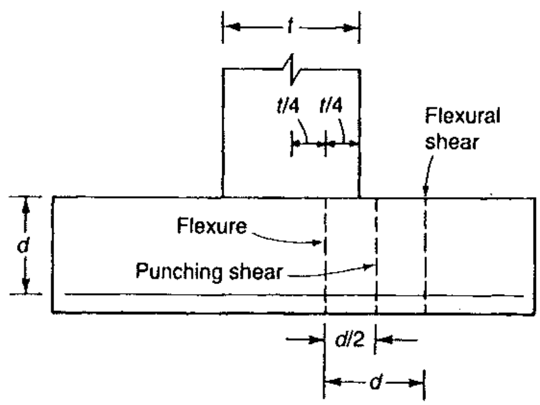


Fig. 4-3. Critical sections: footing with masonry column

Reinforcement is designed for the maximum moment at the reaction plane and is distributed uniformly across the base in the case of a square footing. For a rectangular footing, reinforcement parallel to the shorter side should be concentrated in a central band width equal to the length of the shorter side as illustrated in Fig. 4-4. The area of reinforcement required in the central band is given by ACI Equation (15-1) as

$$A_b = 2A_s/(\beta + 1)$$

where, A_s = total required reinforcement area

$\beta = l_2/l_1$ = ratio of the long side to the short side of the footing

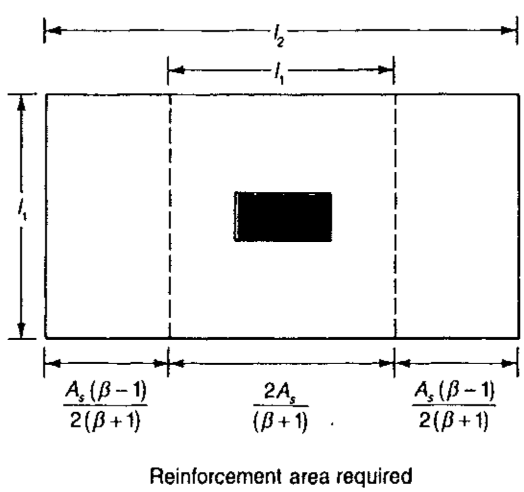


Fig. 4-4. Rectangular footing: reinforcement areas

The capacity of a footing for flexural shear is given by ACI Equation (11-3) as:

$$\phi V_c = 2\phi bd\sqrt{f_c'}$$

where, ϕ = strength reduction factor = 0.85 from ACI Section 9.3

b = width of footing

d = effective depth

f_c' = concrete strength

The capacity of a footing for punching shear is given by ACI Equation (11-35) as

$$\phi V_c = (2 + 4/\beta_c)\phi b_o d\sqrt{f_c'}$$
$$\leq 4\phi b_o d\sqrt{f_c'}$$

where $\beta_c = c_2/c_1$ = ratio of long side to short side of column

b_o = length of critical perimeter for punching shear = $2(c_1 + c_2) + 4d$.

Load transfer between a reinforced concrete column and the footing may be provided by the bearing capacity of the column and the footing. The bearing capacity of the column concrete at the interface is given by ACI, Section 10.17.1 as:

$$\phi P_n = 0.85\phi f_c' A_1$$

where, ϕ = strength reduction factor = 0.7 from ACI Section 9.3

A_1 = area of column

f_c' = strength of column concrete.

The bearing capacity of the footing concrete at the interface is given by ACI Section 10.17.1 as

$$\phi P_n = 0.85\phi\ f_c' A_1\sqrt{A_2/A_1}$$
$$\leq 0.85\phi f_c' A_1 \times 2$$

where, f_c' = strength of footing concrete

ϕ = strength reduction factor = 0.7 from ACI Section 9.3

A_2 = area of the base of the pyramid, with side slopes of 1:2, formed within the footing by the column base.

In accordance with ACI Section 15.8.1.2, when the bearing strength of the concrete is exceeded, reinforcement must be provided at the interface to transfer the excess load. The capacity of this reinforcement is:

$$\phi P_s = \phi A_s f_y$$

where, ϕ = strength reduction factor = 0.7 from ACI Section 9.3

f_y = reinforcement yield strength

A_s = reinforcement area.

In addition, in accordance with ACI Section 15.8.2.1, a minimum area of reinforcement must be provided across the interface given by

$A_{s(\min)} = 0.005A_1$.

Example 1

Fig. 4-5 indicates the loads acting on a steel column with a reinforced concrete footing. Check the adequacy of the footing and determine the required base plate thickness and reinforcement area, using grade 60 bars, for a concrete strength of 2000 pounds per square inch and a base plate of grade A36 steel.

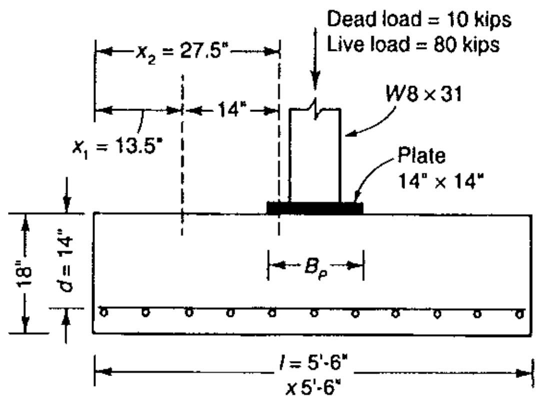


Fig. 4-5

Solution

The base plate thickness may be computed by the design method presented in the AISC Manual,[2] Section 3-106.

The bearing pressure on the base plate due to the applied service loads is:

$$f_p = (P_D + P_L)/A_p$$
$$= (10 + 80)/(14 \times 14)$$
$$= 0.46 \text{ kips per square inch}$$

The relevant design parameters are:

$$m = (B_p - 0.95d)/2$$
$$= (14 - 0.95 \times 8)/2$$
$$= 3.2$$

$$n = (B_p - 0.80b_f)/2$$
$$= (14 - 0.80 \times 8)/2$$
$$= 3.8 \ldots \text{governs.}$$

Hence, the required base plate thickness is given by

$$t_p = 2n\sqrt{f_p / F_y}$$
$$= 2 \times 3.8 \times \sqrt{0.46/36}$$
$$= 0.86 \text{ inches}$$
$$= 7/8 \text{ inch.}$$

The factored applied load is given by ACI Equation (9-1) as

$$P_u = 1.4P_D + 1.7P_L$$
$$= 1.4 \times 10 + 1.7 \times 80$$
$$= 150 \text{ kips}$$

The area of the base of the pyramid, with side slopes of 1:2, formed within the footing by the base plate area is:

$$A_2 = (B_p + 4d)^2$$
$$\leq l^2$$

where, d = effective depth = 18 – 3 – 1 = 14 inches,

l = size of footing = 66 inches.

Then, $A_2 = (14 + 4d)^2$

$= 4356 \ldots$ maximum, and

$$\sqrt{A_2 / A_p} = \sqrt{4356/14^2}$$
$$> 2$$

Hence, the bearing capacity of the footing concrete is given by ACI Section 10.15.1 as

$$\phi P_n = 0.85\phi f_c' A_p \times 2$$
$$= 0.85 \times 0.7 \times 2000 \times 196 \times 2/1000$$
$$= 466 \text{ kips}$$
$$> P_u \ldots \text{ satisfactory.}$$

The net factored pressure acting on the underside of the footing is

$$q_u = P_u/A_f$$
$$= 150/5.5^2$$
$$= 4.96 \text{ kips per square foot}$$

The critical section for flexural shear is located a distance from the edge of the footing, which is given by:

$$x_1 = l/2 - (B_p + b_f)/4 - d$$
$$= 5.5/2 - (14 + 8)/48 - 14/12$$
$$= 1.125 \text{ feet}$$

The factored applied shear at the critical section is:

$$V_u = q_u l x_1$$
$$= 4.96 \times 5.5 \times 1.125$$
$$= 30.69 \text{ kips.}$$

The flexural shear capacity of the footing is given by Equation (11-3) as:

$$\phi V_c = 2\phi ld \sqrt{f_c'}$$
$$= 2 \times 0.85 \times 5.5 \times 12 \times 14 \times \sqrt{2000}/1000$$
$$= 70.25 \text{ kips}$$
$$> V_u \ldots \text{satisfactory.}$$

The length of one side of the critical perimeter for punching shear is:

$$b_o/4 = (B_p + b_f)/2 + d$$
$$= (14 + 8)/24 + 14/12$$
$$= 2.08 \text{ feet.}$$

The factored, applied shear at the critical perimeter is:

$$V_u = P_u - q_u(b_o/4)^2$$
$$= 150 - 4.96 \times 2.08^2$$
$$= 128.47 \text{ kips.}$$

If the ratio of the base plate sides is:

$$\beta_c = 1$$
$$< 2,$$

then, the punching shear capacity of the footing is given by ACI Equation (11-35) as

$$\phi V_c = 4\phi b_o d \sqrt{f_c'}$$
$$= 4 \times 0.85 \times 4 \times 2.08 \times 12 \times 14 \times \sqrt{2000}/1000$$
$$= 212.53 \text{ kips}$$
$$> V_u \ldots \text{satisfactory.}$$

The critical section for flexure is located a distance from the edge of the footing, which is given by:

$$x_2 = l/2 - (B_p + b_f)/4$$
$$= 5.5/2 - (14 + 8)/48$$
$$= 2.29 \text{ feet.}$$

The factored, applied moment at this section is:

$$M_u = q_u l x_2^2/2$$
$$= 4.96 \times 5.5 \times 2.29^2/2$$
$$= 71.53 \text{ kip feet.}$$

The required flexural reinforcement ratio may be obtained by calculator program[3] and is given by:

$$\rho = 0.85 f_c' \left(1 - \sqrt{1 - 2K/0.765 f_c'}\right)/f_y$$

where, $K = 12M_u/bd^2 = 0.0664$

$\rho = 0.126$ percent..

The maximum reinforcement ratio is given by ACI Section 10.3 as

$$\rho_{max} = 0.75 \times 0.85 \times 87\ \beta_1 f_c'/f_y(87 + f_y)$$
$$= 1.07 \text{ percent}$$
$$> \rho \ldots \text{satisfactory.}$$

The minimum reinforcement ratio in a footing slab for grade 60 bars is given by ACI, Section 7.12 as

$$\rho_{min} = 0.18 \text{ percent of the gross area} \ldots \text{governs}$$
$$> \rho.$$

Hence, the required reinforcement area is:

$$A_s = lh\rho_{min}$$
$$= 66 \times 18 \times 0.0018$$
$$= 2.14 \text{ square inches.}$$

Hence, provide eleven Number 4 bars to give an area of

$A_s' = 2.20$ square inches.

Allowing for an end cover of three inches, the available anchorage length for the bars is:

$$l_{da} = x_2 - 3$$
$$= (2.29 \times 12) - 3$$
$$= 24.5 \text{ inches.}$$

For Number 4 bars in 2000 pound per square inch concrete, the development length of ACI Equation (12-1) governs with:

$$2.5 = (c + K_{tr})/d_b$$
$$1 = \alpha = \beta = \lambda$$
$$\gamma = 0.8$$

The required development length is given by:

$$l_d = 32.2\ d_b$$
$$= 32.2 \times 0.5$$
$$= 16.1 \text{ inches.}$$
$$< l_{da} \ldots \text{satisfactory}$$

The footing is adequate.

Combined Footing

The rectangular footing shown in Fig. 4-6 has one column adjacent to the property line of the building, and this limits the allowable length of the footing. The centroid of the footing is designed to coincide with the centroid of the service loads on the two columns thus providing a uniform soil bearing pressure for service loads. The factored bearing pressure under factored loads will not, however, be uniform unless the ratio of the factored load to service load for both columns is identical.

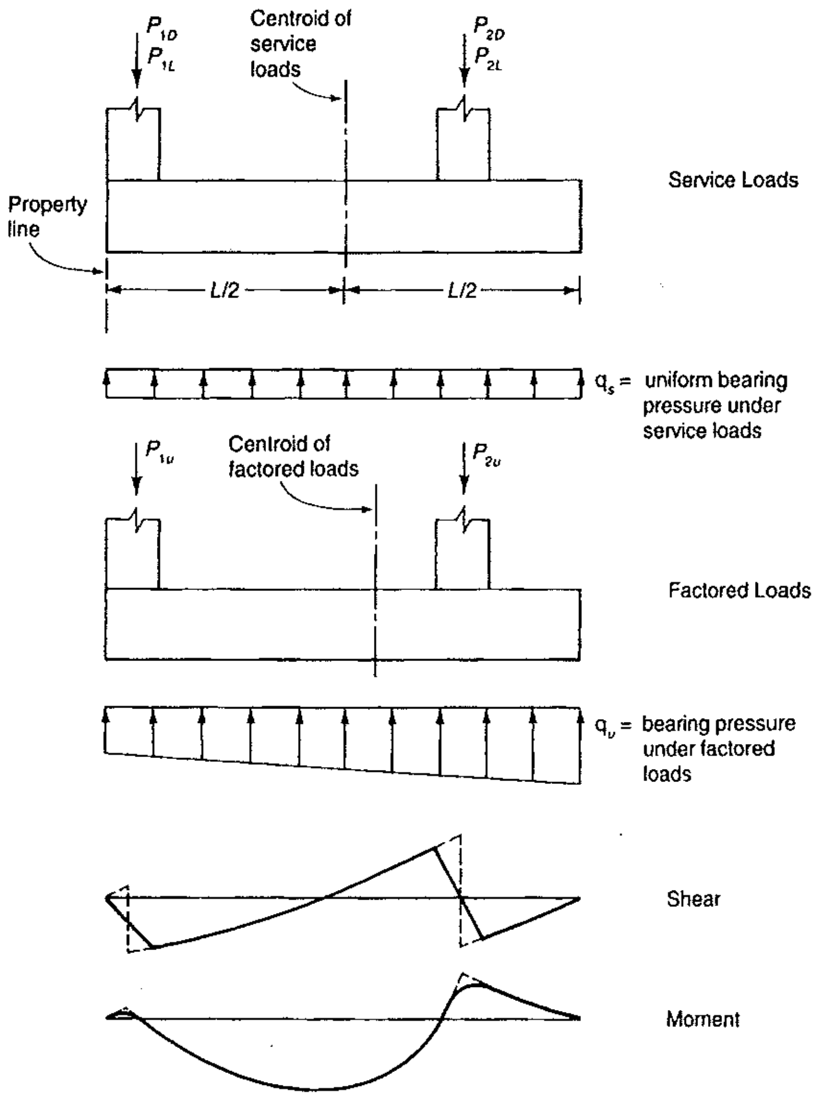


Fig. 4-6. Combined footing

Example 2

Fig. 4-7 indicates the service loads supported by two 12 inch × 12 inch columns and their location with respect to the property line. The depth of the footing is 24 inches and the concrete strength is 3000 pounds per square inch. Determine the dimensions of a combined footing which will provide an allowable soil bearing pressure of 2000 pounds per square foot. If the ratio of factored load to service load for both columns is 1.5, determine if the grade 60 reinforcement indicated and the depth of the footing are adequate.

Solution

Allowing for the weight of the footing, the equivalent soil pressure produced by the column service loads is

$q = q_s - 150\text{h}$

$= 2000 - (150 \times 2)$

$= 1700$ pounds per square foot.

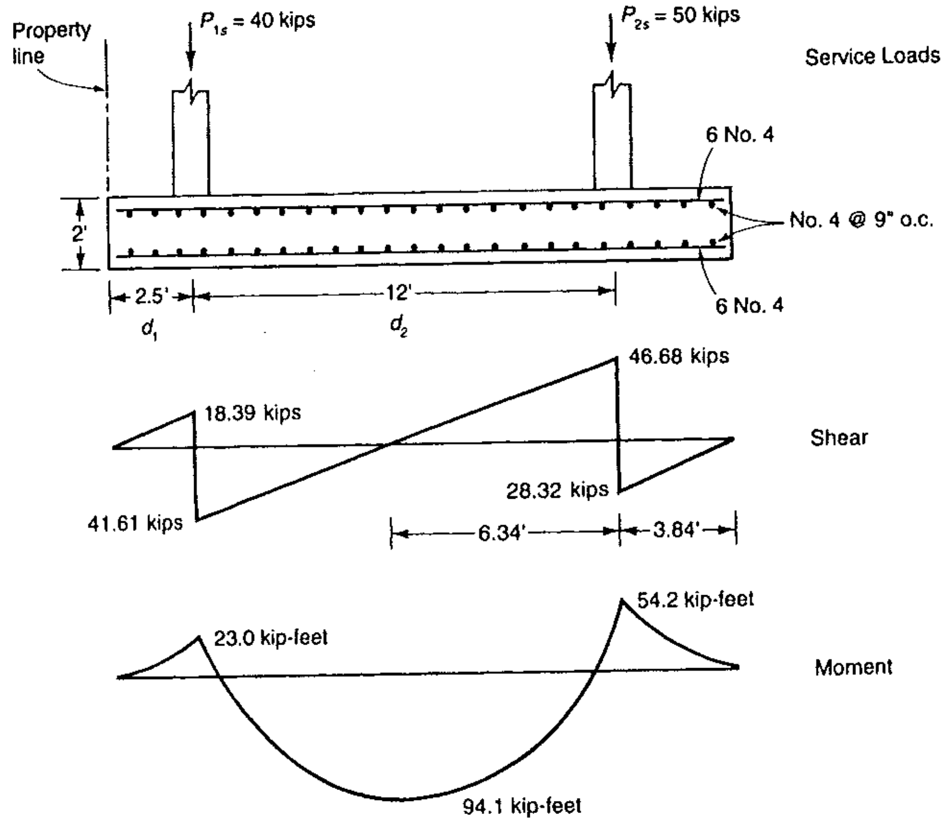


Fig. 4-7

The centroid of the service loads is located a distance from the property line given by:

$x_o = (2.5\ P_{1s} + 14.5P_{2s})/(P_{1s} + P_{2s})$

$= 2.5 \times 40 + 14.5 \times 50)/(40 + 50)$

$= 9.17$ feet.

Then, to produce a uniform pressure under the footing, the length of the footing is:

$L = 2x_o$

$= 2 \times 9.17$

$= 18.34$ feet.

The required width of footing to produce an allowable soil pressure of 2000 pounds per square foot is:

$B = (P_{1s} + P_{2s})/qL$

$= (40{,}000 + 50{,}000)/(1700 \times 18.34)$

$= 2.89$ feet.

Since the ratio of factored load to service load for both columns is 1.5, the net factored pressure acting on the underside of the footing for factored loads is:

$q_u = (P_{1u} + P_{2u})/L$

$= 1.5(40 + 50)/18.34$

$= 7.36$ kips per linear foot.

The shear force and bending moment diagrams for the footing (for factored loading) are shown in Fig. 4-7. Allowing for 3 inches of cover for the longitudinal reinforcement, the effective depth is given by:

$d = h - 3 - d_b/2$

$= 24 - 3 - 0.5/2$

$= 20.75$ inches.

The critical section for flexural shear is located a distance equal to the effective depth from Column 2. The factored applied shear at this critical section is given by:

$V_u = V_{2u} - q_u(d + c/2)/12$

$= 46.68 - 7.36(20.75 + 6)/12$

$= 30.27$ kips.

The flexural shear capacity of the footing is given by Equation (11-3) as:

$$\phi V_c = 2\phi Bd\sqrt{f_c'}$$

$$= 2 \times 0.85 \times 2.89 \times 12 \times 20.75\sqrt{3000}/1000$$

$$= 66.31 \text{ kips}$$

$$> V_u \ldots \text{satisfactory.}$$

The length of the critical perimeter for punching shear is given by:

$b_o = 4(c + d)$

$= 4(12 + 20.75)$

$= 131$ inches.

The column side ratio is

$\beta_c = 1$

$< 2.$

Then the punching shear capacity of the footing is:

$\phi V_c = 4\phi b_o d\sqrt{f_c'}$

$= 4 \times 0.85 \times 131 \times 20.75 \times \sqrt{3000}/1000$

$= 506$ kips

$> P_{2u} \ldots$ satisfactory.

The minimum reinforcement ratio in a footing slab for grade 60 bars is given by ACI Section 7.12 as:

$$\rho_{min} = 0.18 \text{ percent of the gross area}$$

The minimum area of reinforcement required is:

$A_{s(min)} = \rho_{min} hB$

$= 0.18 \times 24 \times 2.89 \times 12/100$

$= 1.5$ square inches.

The sum of the reinforcement areas in the top and bottom of the footing, which consists of a total of twelve Number 4 bars, is

$2A_s = 2.4$ square inches

> 1.5 . . . satisfactory.

The maximum reinforcement ratio is given by ACI Section 10.3 as

$$\rho_{max} = 0.75 \times 0.85 \times 87\beta_1 f_c'/f_y(87 + f_y)$$
$$= 1.6 \text{ percent}$$

The reinforcement ratio provided in the top of the footing is

$\rho = A_s/Bd$

$= 1.2 \times 100/(2.89 \times 12 \times 20.75)$

$= 0.17$ percent

$< \rho_{max}$ . . . satisfactory.

Neglecting compression reinforcement, the design moment strength of the reinforcement in the top of the footing is

$\phi M_n = 0.9A_s f_y d(1 - 0.59\rho f_y / f_c')/12$

$= 109.8$ kip feet

> 94.1 . . . satisfactory.

The reinforcement provided in the bottom of the footing is clearly adequate.

The critical section for flexure in the transverse direction is at the face of the columns and the factored moment at this section is

$M_u = q_u L(B - c)^2/8$

$= 7.36 \times 18.34\,(2.89 - 1)^2/8$

$= 60.3$ kip feet.

The required reinforcement ratio in the bottom of the footing is given by

$\rho = 0.85\ f_c'(1 - \sqrt{1 - 2K/0.765f_c'})/f_y$

where $K = 12M_u/bd^2$

$\rho = 0.0142$ percent.

Hence, the required reinforcement area is

$A_s = \rho bd$

$= 0.0142 \times 18.34 \times 12 \times 20.75/100$

$= 0.65$ square inches.

The reinforcement area provided at the bottom of the footing in the transverse direction exceeds this, and the combined reinforcement area at the top and bottom of the footing exceeds the requirement for minimum reinforcement of 0.18 percent of gross area, so the footing is satisfactory.

Strap Footing

The strap footing shown in Fig. 4-8 consists of two columns on individual pad footings which are connected by a strap beam. The soffit of the strap is not subject to soil pressure because it is poured on a layer of styrofoam. It is assumed that the strap and footings act as a rigid body with a uniform bearing pressure under the footings. The soil reactions, R_1 and R_2, act at the center of the footings, and the pressure under the footings is given by:

$$q_1 = q_2 = q.$$

The base area of each footing is:

$A_1 = B_1L_1$

$A_2 = B_2L_2$

The soil reactions are given by:

$R_1 = qA_1$

$R_2 = qA_2$

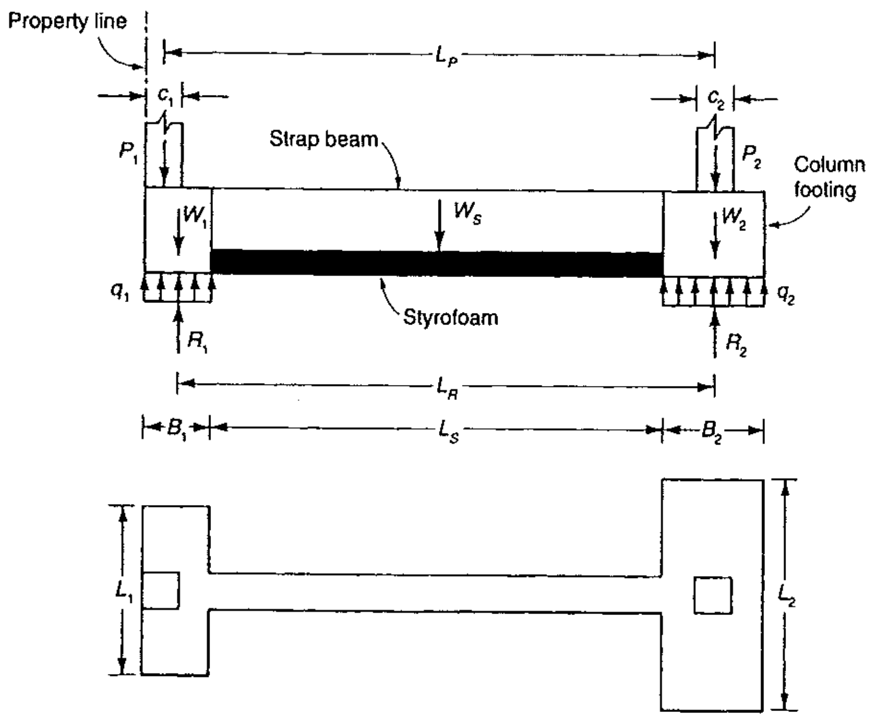


Fig. 4-8. Strap footing

By locating the footing under Column 2 symmetrical with respect to the center of the column, the lines of action of R_2 and P_2 coincide. Then, from the Figure, the distance between R_1 and R_2 is:

$L_R = L_P + c_1/2 - B_1/2$

The strap length is:

$L_S = L_R - (B_1 + B_2)/2$

Two equations of statics are available for solving the two unknowns, R_1 and R_2. However, the values of R_1 and R_2 also influence the required dimensions for the members and an iterative technique is required.

Foundations and Retaining Structures

Appropriate dimensions are initially selected for B_1 and B_2, the depth of the footings and strap, and the width of the strap. Hence, the weight of the strap, W_S, may be determined. An initial estimate is made of the soil reaction R_1 and the corresponding values of A_1 and W_1 are determined. An estimate is made of W_2. Resolving vertically gives:

$$R_2 = P_1 + P_2 - R_1 + W_1 + W_2 + W_S$$

Hence, A_2 may be determined and the initial estimate of W_2 revised. Taking moments about the center of Footing 2 gives

$$R_1 = [P_1L_P + W_1L_R + W_S(L_S + B_2)/2]/L_R$$

The initial estimate of R_1 may now be revised and the process repeated until convergence is reached.

Example 3

Design the strap beam of the strap footing shown in Fig. 4-9. The allowable soil bearing pressure is 3000 pounds per square foot, concrete strength is 3000 pounds per square inch and all reinforcement is Grade 60.

Solution

As an initial estimate, assume that the value of the soil reaction under Column 1 is given by:

$$\begin{aligned} R_1 &= 1.42\,P_1 \\ &= 1.42(60 + 40) \\ &= 142 \text{ kips.} \end{aligned}$$

The area of the footing required for Column 1 is:

$$\begin{aligned} A_1 &= R_1/q \\ &= 142/3 \\ &= 47.33 \text{ square feet.} \end{aligned}$$

The weight of this footing is

$$\begin{aligned} W_1 &= 0.15\,A_1h_1 \\ &= 0.15 \times 47.33 \times 3.5 \\ &= 24.81 \text{ kips.} \end{aligned}$$

The weight of the strap beam is

$$\begin{aligned} W_S &= 0.15L_SB_Sh_s \\ &= 0.15 \times 12.5 \times 2 \times 3 \\ &= 11.25 \text{ kips.} \end{aligned}$$

As an initial estimate, assume that the weight of the footing required for Column 2 is

$$W_2 = 30.6 \text{ kips.}$$

Then, resolving vertically, the value of the soil reaction under Column 2 is

$$\begin{aligned} R_2 &= P_1 + P_2 - R_1 + W_1 + W_2 + W_S \\ &= 100 + 150 - 142 + 24.81 + 30.6 + 11.25. \end{aligned}$$

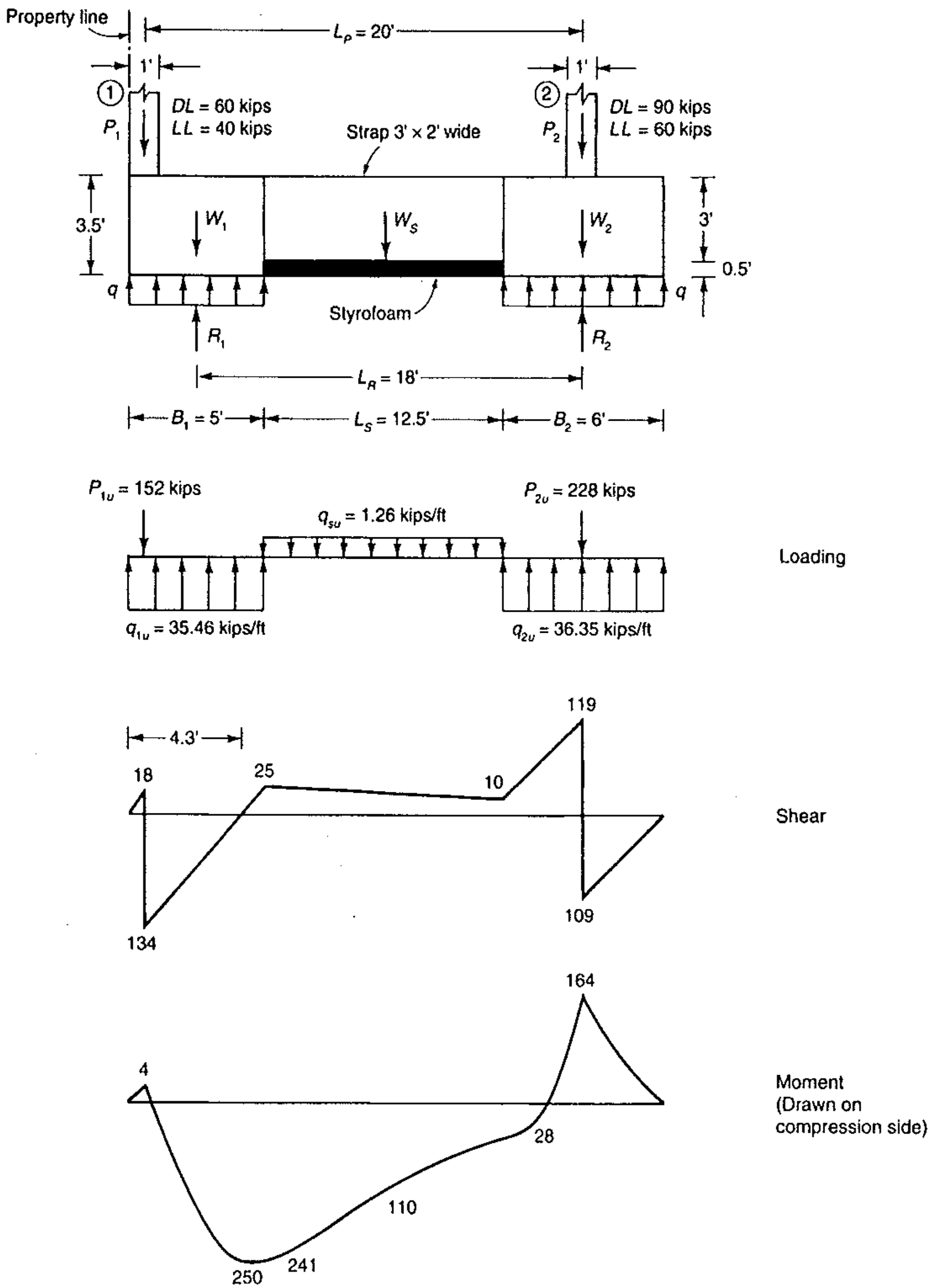


Fig. 4-9

The area of the footing required for Column 2 is:

$A_2 = R_2/q$

$= 174.7/3$

$= 58.23$

A revised estimate of the weight of this footing is:

$$W_2 = 0.15\,A_2 h_2$$
$$= 0.15 \times 58.23 \times 3.5$$
$$= 30.57 \text{ kips}$$
$$\approx 30.6$$

Hence, the initial estimate of W_2 is sufficiently accurate.

Taking moments about the center of Footing 2 gives:

$$R_1 = [P_1 L_P + W_1 L_R + W_S(L_S + B_2)/2]/L_R$$
$$= (100 \times 20 + 24.81 \times 18 + 11.25 \times 18.5/2)/2]/18$$
$$= 141.7 \text{ kips}$$
$$\approx 142$$

Hence, the initial estimate of R_1 is sufficiently accurate.

The factored loading on Column 1 is given by ACI Equation (9-1) as

$$P_{1u} = 1.4P_D + 1.7P_L$$
$$= 1.4 \times 60 + 1.7 \times 40$$
$$= 152 \text{ kips.}$$

The ratio of factored load to service load for Column 1 is:

$$P_{1u}/P_{1s} = 152/100$$
$$= 1.52$$

The same ratio is applicable for Column 2, which has a factored load of:

$$P_{2u} = 228 \text{ kips.}$$

So, neglecting the effect of the footings' factored dead loads, a uniform factored pressure acts on the underside of both footings under factored loading. The factored weights of the footings are

$$W_{1u} = 1.4W_1$$
$$= 34.73 \text{ kips}$$
$$W_{2u} = 1.4W_2$$
$$= 42.80 \text{ kips}$$
$$W_{Su} = 1.4W_S$$
$$= 15.75 \text{ kips.}$$

The total factored load is:

$$P_u = P_{1u} + P_{2u} + W_{1u} + W_{2u} + W_{Su}$$
$$= 473.28 \text{ kips.}$$

The uniform factored soil pressure at both footings is:

$$q_u = P_u/(A_1 + A_2)$$
$$= 473.28/(47.33 + 58.23)$$
$$= 4.48 \text{ kips per square foot.}$$

The net factored pressures acting on the underside of both footings are:

$q_{1u} = (q_u A_1 - W_{1u})/B_1$

$= (4.48 \times 47.33 - 34.73)/5$

$= 35.46$ kips per linear foot

$q_{2u} = (q_u A_2 - W_{2u})/B_2$

$= (4.48 \times 58.23 - 42.80)/6$

$= 36.35$ kips per linear foot.

The factored self weight of the strap beam is:

$q_{Su} = W_{Su}/L_S$

$= 15.75/12.5$

$= 1.26$ kips per linear foot.

The loading, shear and moment diagrams are shown in Fig. 4-9.

The maximum negative moment in the strap occurs at the interface with Footing 1 and has the value:

$M_u = 241$ kip feet.

The required flexural reinforcement ratio is given by:

$$\rho = 0.85f_c'(1 - \sqrt{1 - 2K/0.765f_c'})/f_y$$

where, $K = 12M_u/bd^2 = 0.1066$

$d = 33.625$ inches . . . using Number 6 bars with 2 inches cover

$\rho = 0.202$ percent.

The minimum reinforcement ratio in the strap beam for Grade 60 bars is given by ACI Sections 10.5.1 and 10.5.3 as:

$\rho_{\min} = 200/f_y$

$= 0.333$ percent

$\leq 1.333\rho$

$= 0.269$ percent . . . governs.

The reinforcement area required in the top of the strap beam is given by:

$A_s = bd\rho_{\min}$

$= 24 \times 33.625 \times 0.00269$

$= 2.17$ square inches

Providing five Number 6 bars gives a reinforcement area:

$A_s' = 2.20$ square inches

$> A_s$ . . . satisfactory.

The maximum shear in the strap occurs at the interface with Footing 1 and has the value:

$V_u = 25$ kips

The design shear strength of the concrete section is given by ACI Equation (11-3) as:

$$\begin{aligned}\phi V_c &= 2\phi\sqrt{f_c'}\, b_w d \\ &= 2 \times 0.85 \times \sqrt{3000} \times 24 \times 33.625/1000 \\ &= 75 \text{ kips} \\ &> 2V_u\end{aligned}$$

So, in accordance with ACI Section 11.5.5 no shear reinforcement is required.

Eccentric Footing

An eccentric footing, as shown in Fig. 4-10, may be utilized when the wall of a building must be located on the property line. It is assumed that the footing acts as a rigid body with a uniform soil pressure under the base and that the lateral soil pressures on either side of the footing are balanced. The total gravity load acting as the footing is

$\Sigma W = W_L + W_F + W_W + W_S + W_B$

The soil reaction under the base is

$R = \Sigma W$

The required length of the base is

$L = R/q$

where, q = allowable soil pressure.

The counterclockwise couple produced by R and ΣW is

$M_R = Re$

where, e is the eccentricity between R and ΣW.

The frictional force produced at the underside of the base is

$F = \mu R$

where, μ is the coefficient of friction.

The frictional force is balanced by an equal and opposite tensile force in the top slab and this produces the clockwise couple $M_F = FH$.

Equating the two couples gives

$M_F = M_R$

$FH = Re$

$H = Re/F = e/\mu.$

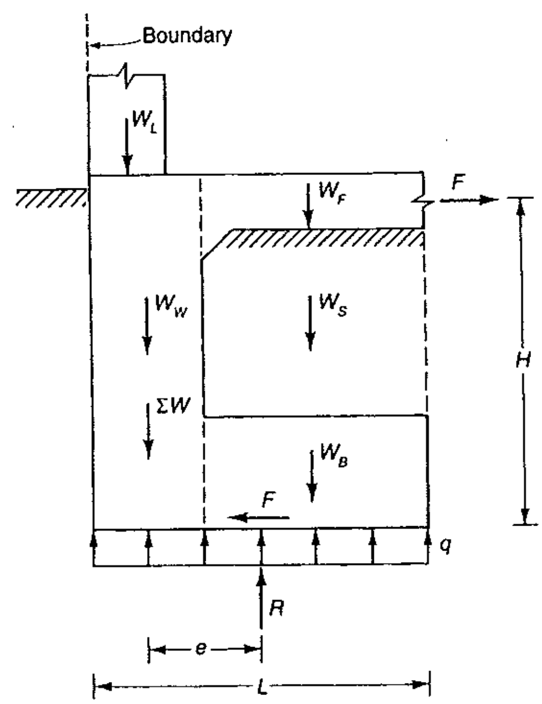


Fig. 4-10. Eccentric footing

By assuming an initial value for H, W_F, W_S and W_B, corresponding values of L and e may be determined and a revised value of H computed. The process is repeated until convergence is reached.

Example 4

Design the eccentric footing shown in Fig. 4-11(a) to support a superimposed dead plus live load of 6.2 kips per linear foot. The coefficient of friction is 0.3 and the unit soil weight is 120 pounds per cubic foot. The allowable soil bearing pressure is 2500 pounds per square foot. Concrete strength is 3000 pounds per square inch and all reinforcement is Grade 60.

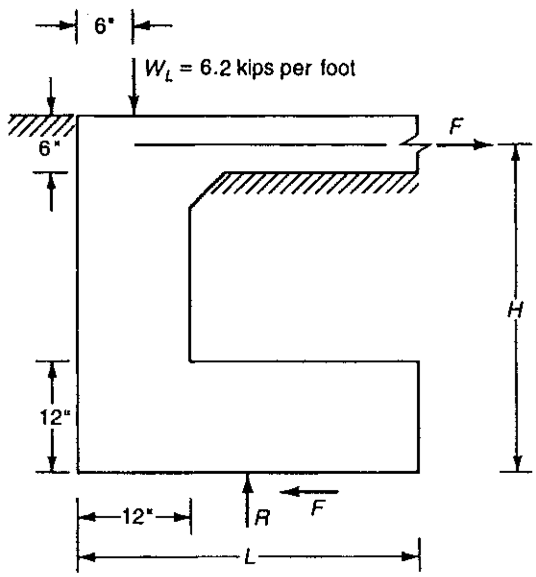


Fig. 4-11(a)

Solution

As initial estimates, assume that:

$H = 2.75$ feet

$W_F = 0.15$ kips

$W_B = 0.30$ kips

$W_S = 0.36$ kips.

Then,

$$W_W = 0.15 \times 3 \times 1$$
$$= 0.45 \text{ kips.}$$

$$\Sigma W = R = W_L + W_F + W_B + W_S + W_W$$
$$= 6.2 + 0.15 + 0.30 + 0.36 + 0.45$$
$$= 7.46 \text{ kips.}$$

The base length required is given by:

$$L = R/q$$
$$= 7.46/2.5$$
$$= 2.984 \text{ feet.}$$

The eccentricity between the lines of action of R and ΣW is:

$$
\begin{aligned}
e &= L[1 - (W_F + W_B + W_S)/\Sigma W]/2 - 0.5 \\
&= 2.984(1 - 0.81/7.46)/2 - 0.5 \\
&= 0.83.
\end{aligned}
$$

The required value for H is given by:

$$
\begin{aligned}
H &= e/\mu \\
&= 0.83/0.3 \\
&= 2.77 \text{ feet} \\
&\approx 2.75
\end{aligned}
$$

Hence, the initial estimate of H is sufficiently accurate.

Revised values for the footing and soil weights are:

$$
\begin{aligned}
W_F &= 0.15 \times 1.984 \times 0.5 \\
&= 0.149 \\
&\approx 0.150 \\
W_B &= 0.15 \times 1.984 \times 1.0 \\
&= 0.298 \\
&\approx 0.300 \\
W_S &= 0.12 \times 1.984 \times 1.52 \\
&= 0.362 \\
&\approx 0.360 \\
W_W &= 0.15 \times 3.02 \times 1 \\
&= 0.453 \\
&\approx 0.450.
\end{aligned}
$$

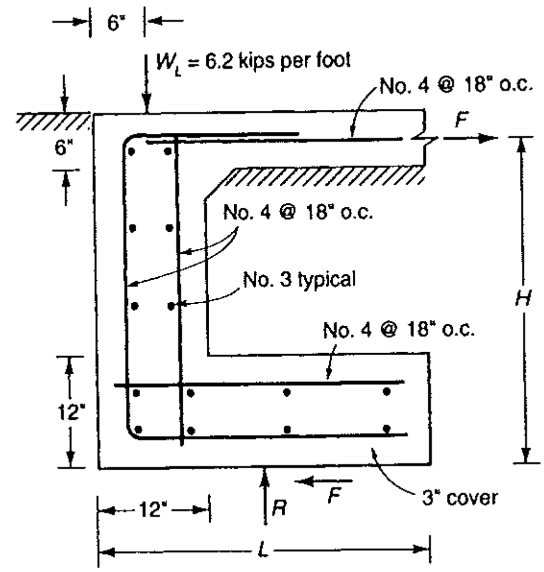


Fig. 4-11(b)

The net pressure acting on the base of the footing is:

$$
\begin{aligned}
q' &= q - (W_F + W_S + W_B)/(L - 1) \\
&= 2.5 - 0.408 \\
&= 2.09 \text{ kips per linear foot.}
\end{aligned}
$$

The bending moment at the face of the front wall is:

$$
\begin{aligned}
M &= q'(L - 1)^2/2 \\
&= 4.11 \text{ kip feet.}
\end{aligned}
$$

Minimum reinforcement is adequate and is given by ACI Section 7.12.2 as:

$$
\begin{aligned}
A_s &= 0.0018bh \\
&= 0.0018 \times 12 \times 12 \\
&= 0.26 \text{ square inches per foot} \\
&= \text{Number 4 bars at 18 inch spacing, top and bottom.}
\end{aligned}
$$

Using six Number 3 bars transversely provides a reinforcement ratio of:

$\rho = 0.66 \times 100/(12 \times 12)$

$= 0.20$ percent

> 0.18 . . . satisfactory.

Minimum reinforcement in the front wall is governed by ACI Section 14.3. Vertical reinforcement consisting of Number 4 bars at 18 inch spacing front and back provides a reinforcement ratio of:

$\rho = 0.4 \times 100/(12 \times 18)$

$= 0.19$ percent

> 0.12 . . . satisfactory.

Horizontal reinforcement consisting of eight Number 3 bars provides a reinforcement ratio of:

$\rho = 0.88 \times 100/(12 \times 36)$

$= 0.204$ percent

> 0.20 . . . satisfactory.

In accordance with ACI Section A.3.2, the area of tensile reinforcement required in the floor slab is given by:

$A_s = F/f_s$

$= 0.3 \times 7.46/24$

$= 0.093$ square inches per foot.

Using Number 4 bars at 18 inch spacing provides an area

$A_s' = 0.13$ square inches

> 0.093 . . . satisfactory.

The reinforcement layout is shown in Fig. 4-11(b).

Footing with Eccentric Load

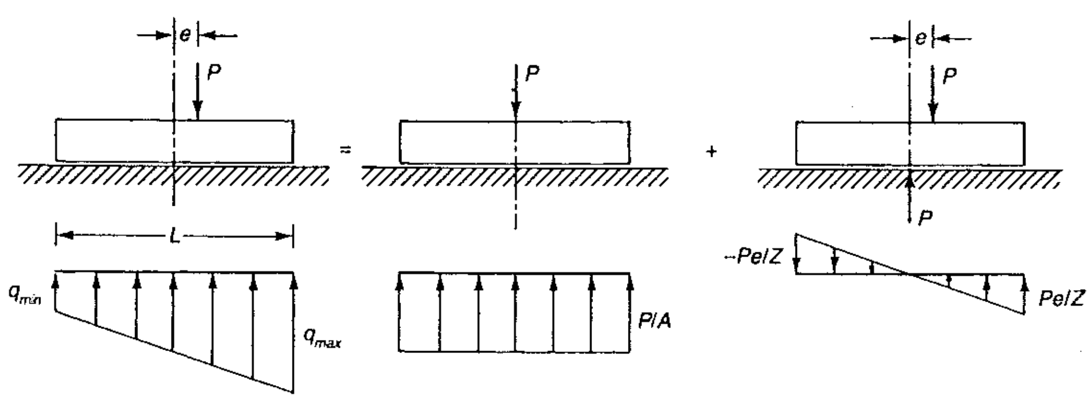


Fig. 4-12. Footing with eccentric load

An applied load with an eccentricity less than L/6 produces the pressure distribution shown in Fig. 4-12. The maximum and minimum bearing pressure under the footing is given by

$$q = P/A \pm Pe/S$$

$$= P(1 \pm 6e/L)/BL$$

where, e = eccentricity of the applied load P

B = width of footing

L = length of footing.

An axial load plus bending moment applied to a footing, as shown in Fig. 4-13, produces an equivalent eccentricity of $e = M/P$.

The bearing pressure under the footing is similarly obtained as:

$q = P(1 \pm 6e/L)/BL$

When the magnitude of the eccentricity, as shown in Fig. 4-14, is given by:

$e = L/6$

the bearing pressure under the footing is:

$q_{max} = 2P/BL$

$q_{min} = 0$

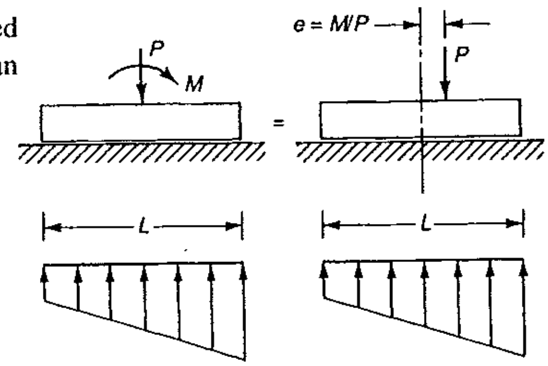


Fig. 4-13. Footing with applied moment

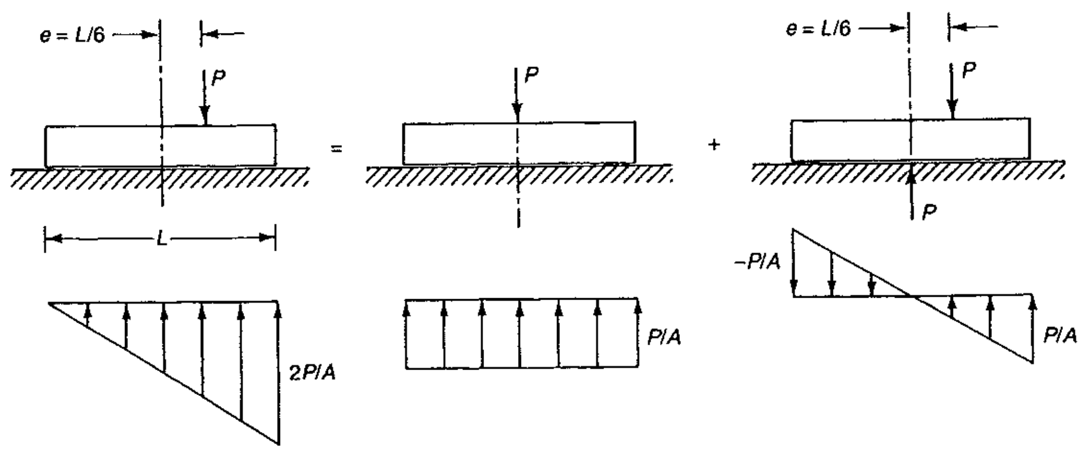


Fig 4-14. Eccentricity = $L/6$

When the eccentricity exceeds $L/6$, as shown in Fig. 4-15, no tension is possible between the soil and the footing and the bearing pressure under the footing is given by:

$q_{max} = 2P/3Be'$

where, $e' = L/2 - e$

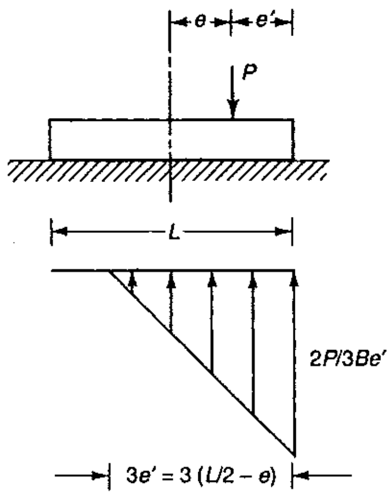


Fig. 4-15. Eccentricity > L/6

Foundations and Retaining Structures

RETAINING WALL

Cantilever Retaining Wall

The forces acting on a reinforced concrete cantilever retaining wall are shown in Fig. 4-16. The weight of the stem wall, base and shear key in pounds per foot length are:

$$W_W = 150L_W(H - h)$$

$$W_B = 150hL_B$$

$$W_K = 150L_W(H_K - h)$$

The live load surcharge behind the stem and the weight of the fill supported by the heel are:

$$W_L = wL_H$$

$$W_S = \gamma_S L_H(H - h)$$

The active earth pressure behind the wall, acting at a height of H/3 above the base, is:

$$H_A = p_A H^2/2$$

where, $p_A = K_A\gamma_S$ = lateral pressure equivalent to a fluid of density p_A pounds per cubic foot

γ_S = density of fill

K_A = Rankine coefficient of active earth pressure = $(1 - \sin\phi)/(1 + \sin\phi)$

ϕ = angle of internal friction

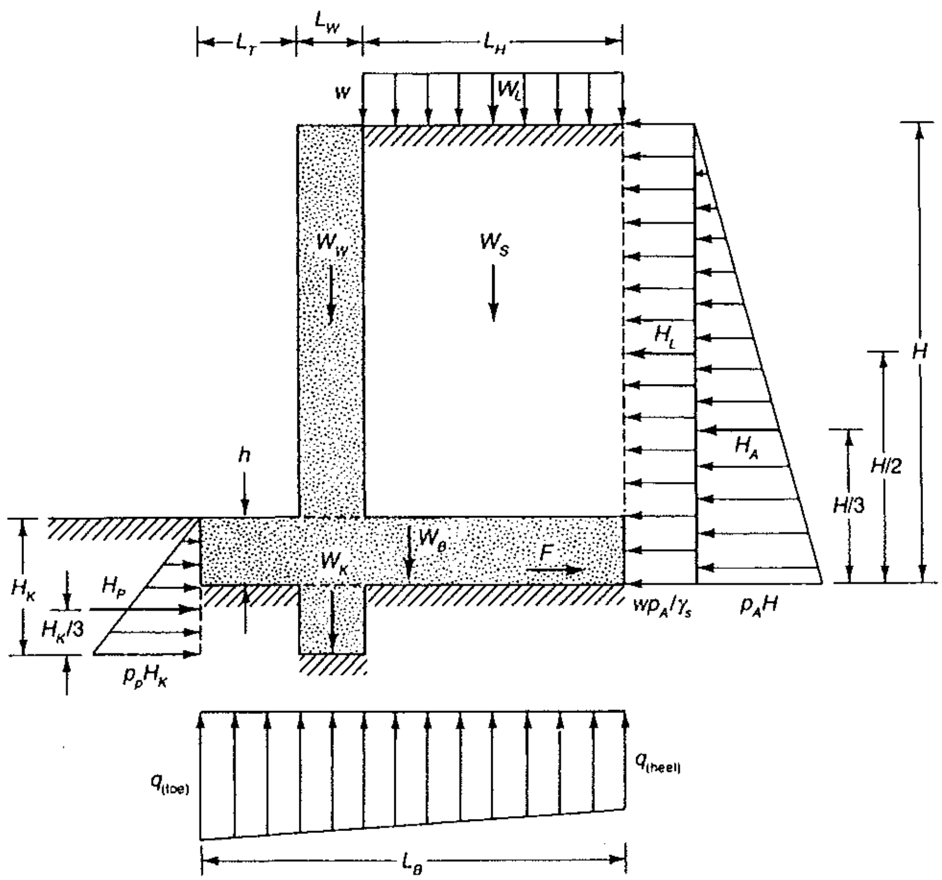


Fig. 4-16. Cantilever retaining wall

Hence, the equivalent fluid pressure for a fill material with a density of 110 pounds per cubic foot and an angle of internal friction of 35° is:

$p_A = 110\,(1 - \sin 35°)/(1 + \sin 35°)$

$= 30$ pounds per square foot per foot.

The live load surcharge of w pounds per square foot is equivalent to an additional height of fill which is given by:

$$\overline{H} = w/\gamma_S$$

This produces a uniform pressure over the height of the fill of $p_L = \overline{H}p_A$

The total surcharge pressure, acting at a height of $H/2$ above the base, is:

$$H_L = p_L H$$
$$= wp_A H/\gamma_S$$

The frictional force produced at the underside of the base is:

$$F = \mu\Sigma W$$

where, μ = coefficient of friction.

When the frictional force is insufficient to provide an adequate factor of safety against sliding, a shear key is provided to mobilize the passive pressure of the undisturbed soil in front of the wall. The passive earth pressure in front of the wall, acting at a height of $H_K/3$ above the bottom of the key, is:

$$H_P = p_P H_K^2/2$$

where, $p_P = K_P\gamma_S$ = lateral pressure equivalent to a fluid of density p_P pounds per cubic foot

K_P = Rankine coefficient of passive earth pressure = $(1 + \sin\phi)/(1 - \sin\phi)$.

So, the equivalent fluid pressure for a material with a density of 110 pounds per cubic foot and an angle of internal friction of 35° is:

$P_P = 110\,(1 + \sin 35°)/(1 - \sin 35°)$

$= 400$ pounds per square foot per foot.

To provide an adequate factor of safety against sliding a depth of shear key must be provided to ensure that

$F + H_P \geq 1.5(H_A + H_L)$.

Similarly, the required factor of safety against overturning about the toe is 1.5.

By determining the eccentricity of the resultant of all forces about the center of the base, the bearing pressure under the base may be obtained.

To design the stem wall and the base, the required strength is determined from ACI Equation (9-4) as:

$$U = 1.4D + 1.7L + 1.7H$$

where, D = dead load

L = live load

H = earth pressure.

The minimum reinforcement required in the stem wall is specified by ACI Section 14.3. For bars larger than Number 5, the reinforcement ratios, based on the gross concrete area, for vertical and horizontal reinforcement are given by:

$\rho_{vert} = 0.15$ percent

$\rho_{hor} = 0.25$ percent

For bars not larger than Number 5, the corresponding ratios are:

$\rho_{vert} = 0.12$ percent

$\rho_{hor} = 0.20$ percent

For walls exceeding ten inches thickness, two layers of reinforcement are required, as detailed in Fig. 4–17.

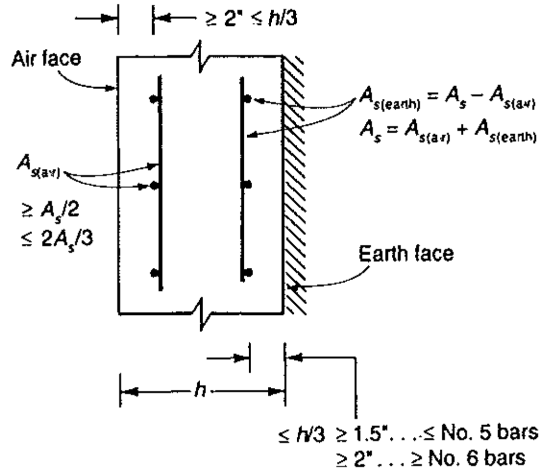
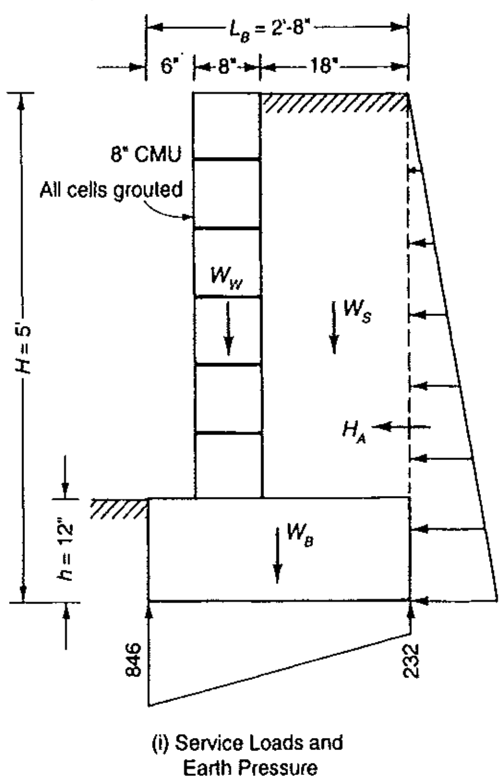


Fig. 4-17. Wall reinforcement

Example 5

Determine the bearing pressure and factors of safety for overturning and sliding of the retaining wall shown in Fig. 4-18(i). The wall's backfill weighs 120 pounds per cubic foot, equivalent fluid pressure in the backfill behind the wall is 30 pounds per square foot per foot, and the coefficient of friction at the underside of the base is 0.40. The nominal eight inch solid grouted concrete masonry stem wall, with a specified strength of 1500 pounds per square inch and without special inspection provided during construction, is located in seismic zone Number 1. The base concrete has a compressive strength of 3000 pounds per square inch. Using Grade 60 deformed bars, determine the reinforcement required in the stem and base.



(i) Service Loads and Earth Pressure

Fig. 4-18(i)

Solution

The service loads acting on the retaining wall are given by:

H_A = lateral pressure from backfill

$= p_A H^2/2$

$= 30 \times 5^2/2$

$= 375$ pounds.

W_W = weight of stem wall

$= 80\,(H - h)$

$= 80 \times 4$

$= 320$ pounds.

W_B = weight of base

$= 150\, hL_B$

$= 150 \times 1.0 \times 2.67$

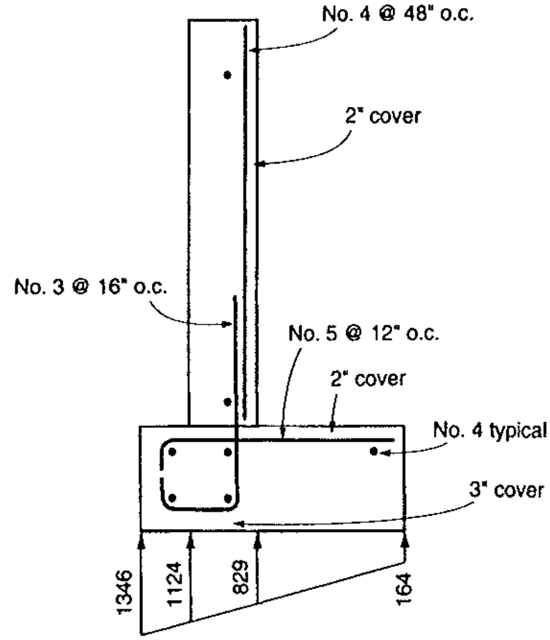
$= 400$ pounds.

W_S = weight of backfill

$= \gamma_S L_H (H - h)$

$= 120 \times 1.5 \times 4$

$= 720$ pounds.



(ii) Factored Earth Pressure and Reinforcement

Fig. 4-18(ii)

The restoring and overturning moments about the bottom corner of the toe are shown below:

Table 4-1

Part	W	x	Wx	H	y	Hy
Backfill	720	1.92	1380	375	1.67	625
Wall	320	0.83	267			
Base	400	1.33	533			
Total	1440		2180	375		625

The factor of safety against overturning is given by

$\Sigma Wx/\Sigma Hy = 2180/625 = 3.5.$

The frictional force produced at the underside of the base is given by

$F = \mu\Sigma W$

$= 0.40 \times 1440$

$= 576$ pounds

The factor of safety against sliding is given by:

$F/\Sigma H = 576/375 = 1.5$

The eccentricity of the applied loads about the toe is given by:

$e' = (\Sigma Wx - \Sigma Hy)/\Sigma W$

$= (2180 - 625)/1440$

$= 1.08$ feet.

The eccentricity about the base centroid is:

$e = L_B/2 - e'$

$= 2.67/2 - 1.08$

$= 0.25$ feet

$< L_B/6$ . . . no tension under the base.

The pressure under the toe is:

$q_{(\mathrm{toe})} = \Sigma W(1 + 6e/L_B)/BL_B$

$= 1440\ (1 + 6 \times 0.25/2.67)/(1 \times 2.67)$

$= 846$ pounds per square foot.

The pressure under the heel is

$q_{(\mathrm{heel})} = \Sigma W(1 - 6e/L_B)/BL_B$

$= 232$ pounds per square foot.

The critical section for moment and shear in the stem is at the base of the stem and, due to service loads, these values are:

$V = p_A\,(H - h)^2/2$

$= 30(5 - 1)^2/2$

$= 240$ pounds per foot

$M = V(H - h)/3$

$= 240(5 - 1)/3$

$= 320$ pounds feet per foot.

The allowable stresses, without special inspection provided, in accordance with UBC Sections 2107.1 and 2107.2, are:

$F_b = 0.5 \times 0.33\,f'_m = 250$ pounds per square inch

$F_v = 0.5\sqrt{f'_m} = 19.36$ pounds per square inch

$F_s = 24{,}000$ pounds per square inch

$E_m = 750f'_m = 1125$ kips per square inch

$E_s = 29{,}000$ kips per square inch.

The relevant section properties of the stem are:

$b = 12$ inches

$d = 7.63 - 2.0 - 0.5/2 = 5.38$ inches.

Providing Number 4 bars at 48 inch spacing, the masonry stresses may be obtained from UBC, Section 2107.2 with the aid of a calculator program,[3] and are:

$n = E_s/E_c = 25.8$

$p = A_s/bd = 0.049/(12 \times 5.38) = 0.076$ percent

$k = (n^2p^2 + 2np)^{0.5} - np = 0.179$

$j = 1 - k/3 = 0.940$

$f_b = 24M/jkbd^2 = 131$ pounds per square inch

$< F_b$ . . . satisfactory.

$f_s = 12M/jdA_s = 15{,}492$ pounds per square inch

$< F_s$ . . . satisfactory.

$f_v = V/bjd = 4$ pounds per square inch

$< F_v$ . . . satisfactory.

The reinforcement is indicated in Fig. 4-18(ii).

To determine the reinforcement in the base, factored loads must be used. In accordance with ACI Section 9.2.4, the factored total vertical load, restoring moment and overturning moment about the toe are:

$\Sigma(\gamma W) = 1.4 \times 1440 = 2016$ pounds

$\Sigma(\gamma W)x = 1.4 \times 2180 = 3052$ pounds feet

$\Sigma(\gamma H)y = 1.7 \times 625 = 1063$ pounds feet

The eccentricity of the factored loads about the toe is given by:

$e' = [\Sigma(\gamma W)x - \Sigma(\gamma H)y]/\Sigma(\gamma W)$

$= (3052 - 1063)/2016$

$= 0.99$ feet.

The eccentricity about the base centroid is:

$e = L_B/2 - e'$

$= 2.67/2 - 0.99$

$= 0.35$ feet

$< L_B/6$ . . . no tension under the base.

The pressure under the toe is:

$q_{(\text{toe})} = \Sigma(\gamma W)(1 + 6e/L_B)/BL_B$

$= 2016\ (1 + 6 \times 0.35/2.67)/(1 \times 2.67)$

$= 1346$ pounds per square foot

The pressure under the heel is:

$q_{(\text{heel})} = \Sigma(\gamma W)(1 - 6e/L_B)/BL_B$

$= 164$ pounds per square foot.

The critical section for shear and moment in the heel is at the rear face of the wall and, due to factored loads, these values are:

$V_u = \gamma W_S + 18\gamma W_B/32 - 1.5\ (829 + 164)/2$

$= 1.4 \times 720 + 18 \times 1.4 \times 400/32 - 745$

$= 578$ pounds per foot

$M_u = 0.75\ (\gamma W_S + 18\gamma W_B/32) - 1.5^2\ (829 + 2 \times 164)/6$

$= 558$ pounds feet per foot.

The shear capacity of the heel is given by ACI Equation (11-3) as:

$$\phi V_c = 2\phi b_w d\sqrt{f_c'}$$
$$= 2 \times 0.85 \times 12 \times 9.5 \times \sqrt{3000}$$
$$= 10{,}615 \text{ pounds}$$
$$> V_u \ldots \text{ satisfactory.}$$

The required reinforcement ratio is derived from ACI Section 10.2, with the aid of a calculator program[3], and is

$$\rho = 0.85 f_c'\left(1 - \sqrt{1 - 2K/0.765 f_c'}\right)/f_y$$

where $K = 12M_u/bd^2 = 0.0062$ kips per square inch

$\rho = 0.012$ percent

The minimum reinforcement ratio in a footing slab for Grade 60 bars is given by ACI Section 7.12 as:

$\rho_{\min} = 0.18$ percent of the gross area

$> \rho \ldots$ governs.

Hence, the required reinforcement area is:

$A_S = bh\rho_{\min}$

$= 12 \times 12 \times 0.18/100$

$= 0.26$ square inches per foot

Provide Number 5 bars at twelve inch spacing to give $A_s' = 0.31$ square inches per foot.

The critical section for shear and moment in the toe is at the front face of the stem and, due to factored loads, these values are:

$V_u = 0.5(1346 + 1124)/2 - 6\gamma W_B/32$

$= 513$ pounds per foot

$< \phi V_c \ldots$ satisfactory.

$M_u = 0.5^2\,(1124 + 2 \times 1346)/6 - 0.25 \times 6\gamma W_B/32$

$= 133$ pounds feet per foot.

The required reinforcement ratio is:

$r = 0.85\ f_c'(1 - \sqrt{1 - 2K/0.765 f_c'})/f_y$

$= 0.0034$ percent.

Providing Number 3 bars at 16 inch spacing gives a reinforcement ratio of

$\rho' = 0.083/(12 \times 8.75)$

$= 0.079$ percent

$> 1.33\rho \ldots$ satisfactory.

The required reinforcement layout is shown in Fig. 4-18(ii).

Foundations and Retaining Structures

Gravity Retaining Wall

In a gravity retaining wall, as shown in Fig. 4-19, stability and acceptable earth pressure are provided by the size and mass of the structure. The resultant of the self weight and applied loads must be adjusted over the full height of the structure to ensure that the allowable tensile stress of the material is not exceeded at any section. When specifications require zero tension at any section, the locus of the resultant thrust must lie within the middle third at any section, as shown in Fig. 4-19.

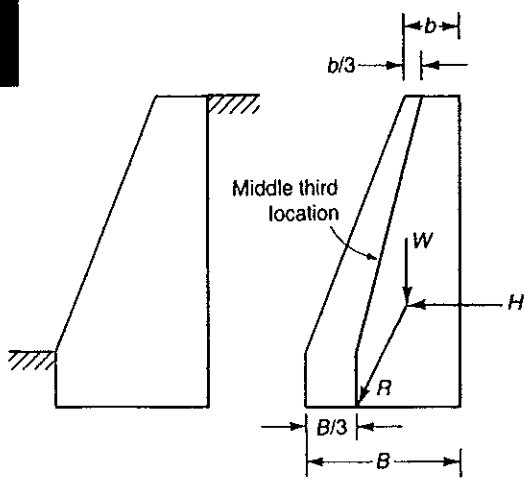


Fig. 4-19. Gravity retaining wall

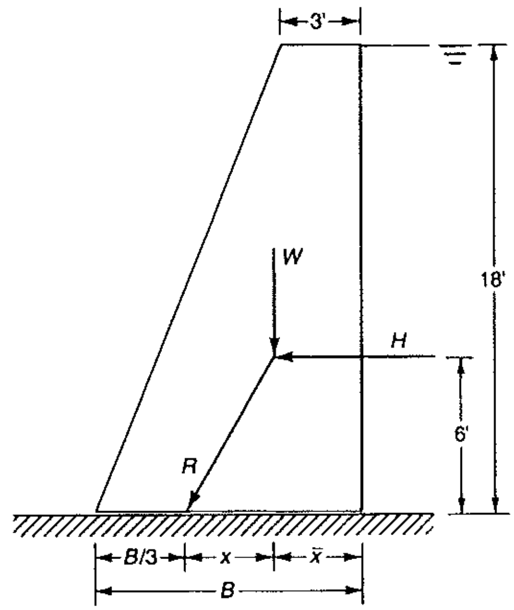


Fig. 4-20

Example 6

The mass concrete gravity dam shown in Fig. 4-20 is 18 feet high. Determine the minimum base width if no tensile stresses are allowed in the concrete and the water level is at the top of the dam wall.

Solution

The total weight of the wall is

$W = 144 \times 18\ (3 + B)/2$

$\quad = 1296\ (3 + B)$ pounds per foot.

The total water pressure acting on the wall at a height of 6 feet above the base is:

$H = 62.4 \times 18^2/2$

$\quad = 10{,}108$ pounds per foot

The centroid of the wall mass is located a distance from the heel which is given by:

$\bar{x} = (9 + 3B + B^2)/3(3 + B)$ feet.

The line of action of the resultant thrust cuts the base at a distance from the centroid which is given by:

$x = 6H/W$

$\quad = 6 \times 10{,}108/1296(3 + B)$

$\quad = 46.80/(3 + B)$

To ensure no tensile stress in the base:

$x + \bar{x} \leq 2B/3$

Hence, the minimum base width is given by:

$46.80/(3 + B) + (9 + 3B + B^2)/3(3 + B) - 2B/3 = 0$

$$B = 10.81 \text{ feet.}$$

The minimum required base width is 10.81 feet.

Cantilevered Sheetpile Wall

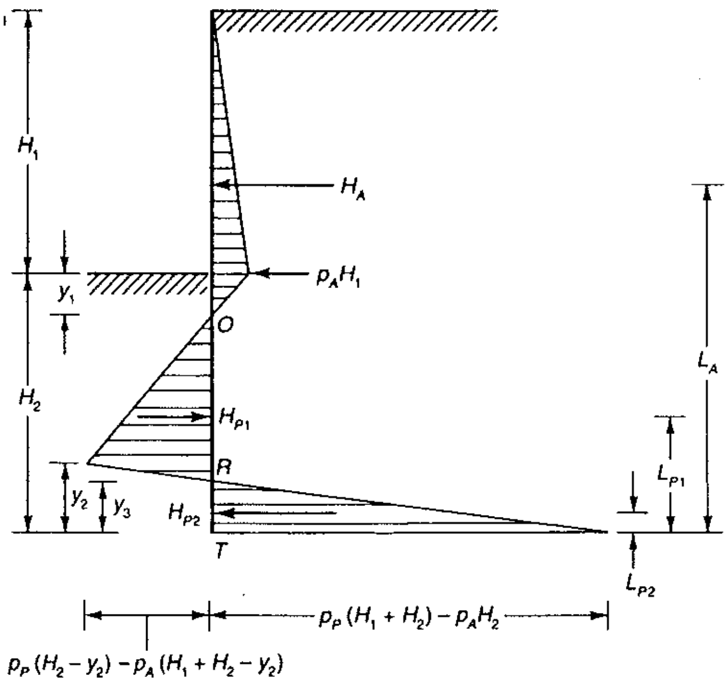


Fig. 4-21. Cantilevered sheetpile wall

The forces acting on a cantilevered sheetpile wall are shown in Fig. 4-21. An initial estimate[5] is required of the penetration of the sheetpiling and of the location of the Point R about which the sheetpiling rotates. Point O occurs where the passive pressure in front of the sheetpiling equals the active pressure behind the sheetpiling. The forces acting on the wall are:

$$H_A = p_A H_1(H_1 + y_1)/2$$

$$H_{P1} = [p_P(H_2 - y_2) - p_A(H_1 + H_2 - y_2)](H_2 - y_1 - y_3)/2$$

$$H_{P2} = [p_P(H_1 + H_2) - p_A H_2](H_2 - y_3)/2$$

where, $y_1 = p_A H_1/(p_P - p_A)$

$y_3 = y_2[p_P(H_1 + H_2) - p_A H_2]/(p_P - p_A)(H_1 + 2H_2 - y_2)$

Resolving horizontally gives

$$H_A - H_{P1} + H_{P2} = 0$$

Taking moments about the toe of the pile at T gives

$$H_A L_A - H_{P1} L_{P1} + H_{P2} L_{P2} = 0$$

Adjustments are made to the initial estimates until the above equations are satisfied. The shear and bending moment in the sheetpiling may then be obtained.

Anchored Sheetpile Retaining Wall

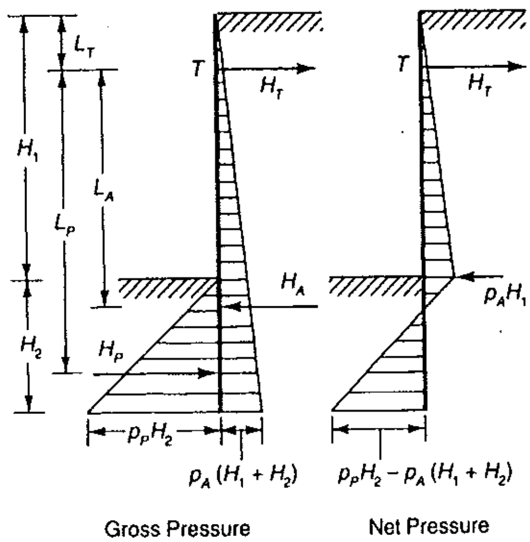


Fig. 4-22. Anchored sheetpile wall

The forces acting on an anchored sheetpile wall are shown in Fig. 4-22. An initial estimate[5,6] is required of the penetration of the sheetpiling. The forces acting on the wall are given by:

H_A = total active pressure behind wall

$= p_A(H_1 + H_2)^2/2$

H_P = total passive pressure in front of wall

$= p_P H_2^2/2$

H_T = force in tie

$= H_A - H_P$

Taking moments of all forces about the tie point T gives

$$H_A L_A - H_P L_P = 0$$

Adjustments are made to the initial estimate of the penetration until the above equation is satisfied.

The tie may be secured by means of a soil anchor, anchor piles or a dead-man. Anchor piles are shown in Fig. 4-23 and the forces in the piles are given by:

F_C = force in compression pile

$= H_T/(\sin\theta_1 + \cos\theta_1 \tan\theta_2)$

F_T = force in tension pile

$= H_T/(\sin\theta_2 + \cos\theta_2 \tan\theta_1)$

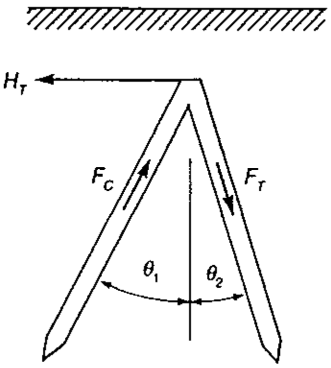


Fig. 4-23. Anchor piles

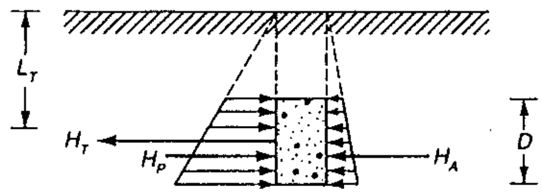


Fig. 4-24. Dead-man

A dead-man anchor is shown in Fig. 4-24 and may consist of a continuous beam or an isolated anchorage. The resistance of a continuous beam consists of the passive pressure in front of the beam less the active pressure behind the beam. The dead-man must be located a sufficient distance from the wall so that the passive wedge in front of the dead-man does not intersect the active wedge behind the wall. Resolving horizontally for a continuous anchor with a tie force per unit length of H_T gives:

$$H_P - H_A + H_T = 0$$

$$DL_T(p_P - p_A) - H_T = 0$$

Hence, the size of the anchor may be determined.

Example 7

Determine the required penetration of the sheetpile retaining wall shown in Fig. 4-25 if active earth pressure may be assumed equivalent to a fluid pressure of 30 pounds per square foot per foot, and passive pressure may be assumed equivalent to a fluid pressure of 400 pounds per square foot per foot. Calculate the force in the tie and the location and magnitude of the maximum shear and the maximum moment in the sheetpiling.

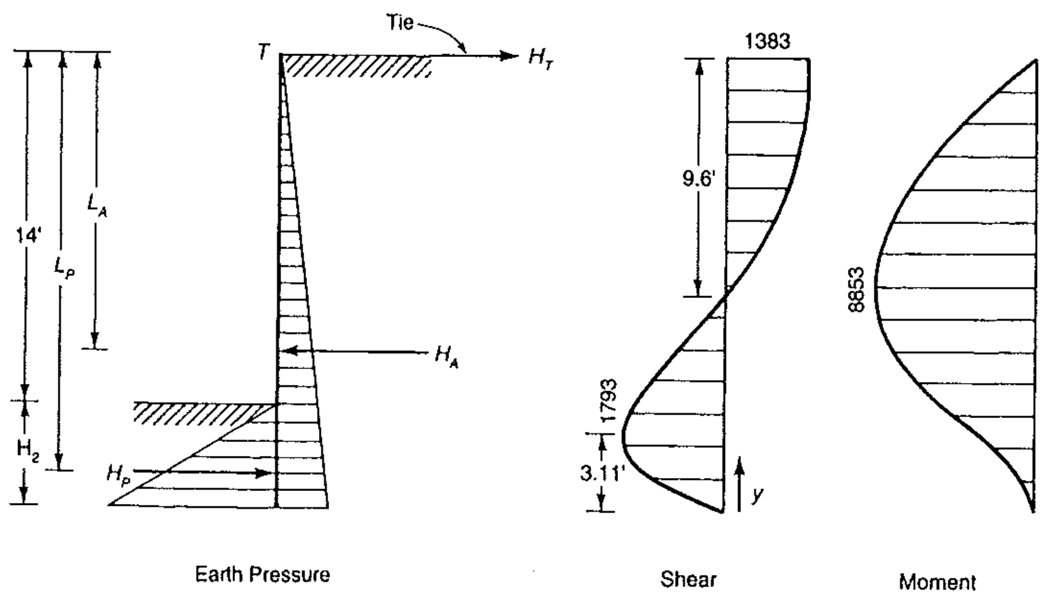


Fig. 4-25

Solution

The total active pressure on the back of the wall is:

$H_A = 30(14 + H_2)^2/2$

$= 2940 + 420H_2 + 15H_2^2$

The total passive pressure on the front of the wall is:

$H_P = 400H_2^2/2$

$= 200\ H_2^2$

The distance between the line of action of the active pressure and the tie is:

$L_A = 2(14 + H_2)/3$

$= 9.33 + 0.67H_2$

The distance between the line of action of the passive pressure and the tie is:

$L_P = 14 + 0.67H_2$

Taking moments about the tie point T gives:

$H_A L_A - H_P L_P = 0$

$(2940 + 420H_2 + 15H_2^2)(9.33 + 0.67H_2) - 200H_2^2\ (14 + 0.67H_2) = 0$

Solving for H_2 gives the required penetration as:

$H_2 = 4.25$ feet.

The force in the tie is:

$H_T = H_A - H_P$

$= 4996 - 3613$

$= 1383$ pounds per foot.

$p_P H_2$ = passive pressure at toe of wall:

$= 400 \times 4.25$

$= 1700$ pounds per square foot.

$p_A(14 + H_2)$ = active pressure at toe of wall:

$= 30(14 + 4.25)$

$= 548$ pounds per square foot.

The shear force at a distance y from the toe is:

$V = y(1700 - 200y) - y(548 - 15y)$

$= 1152y - 185y^2$

The maximum shear occurs when:

$dV/dy = 0$

$= 1152 - 370y$

Hence, $y = 3.11$ feet, and

$V_{max} = 1793$ pounds per foot.

The shear force at a distance x from the top of the wall is:

$V = H_T - p_A x^2/2$

$= 1383 - 15x^2$

The maximum moment occurs when:

$V = 0$

$x = \sqrt{1383/15}$

$= 9.6$ feet

The maximum moment is:

$M_{max} = H_T x - P_A x^3/6$

$= 1383 \times 9.6 - 30 \times 9.6^3/6$

$= 8853$ pounds feet per foot

PILE FOUNDATIONS

Pile Group with Vertical Piles

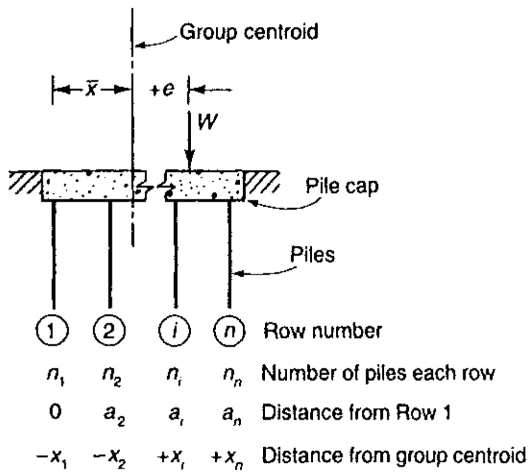


Fig. 4-26. Pile group with vertical piles and vertical load

Using the notation in Fig. 4-26, which shows a group of vertical piles with a rigid pile cap with an imposed vertical eccentric load, the location of the pile group centroid is

$$\bar{x} = \Sigma an/\Sigma n$$

The axial force on each pile in row i, neglecting the self weight of the pile and the pile cap, is given by

$$P_i = W/\Sigma n + Wex_i/\Sigma nx^2$$

For a symmetrical group, the location of the group centroid is

$$\bar{x} = a_n/2$$

When the applied load coincides with the pile group centroid, the axial force on each pile is given by

$$P = W/\Sigma n$$

When the pile cap is subjected to a bending moment, M, the eccentricity is given by

$$e = M/W$$

When each row in the group has an identical number of piles n_R and the number of piles in each line is n_L, the load applicable to each line is:

$$W' = W/n_R$$

The axial force on a pile in Row i is:

$$P_i = W'/n_L + W'ex_i/\Sigma x^2$$

Fig. 4-27 illustrates a group of vertical piles subjected to a horizontal load, as in the case of a pier or wharf. The piles are considered fixed ended at the pile cap and at a depth of penetration, H, below the bottom of the rigid pile cap, with a point of contraflexure occurring at a depth of $H/2$. The total moment acting on the pile group is

$$M_T = We + FH$$

The axial force on each pile in row i, neglecting the self weight of the pile and the pile cap, is given by

$$P_i = W/\Sigma n + M_T x_i/\Sigma n x^2$$

The shearing force on each pile is

$$V = F/\Sigma n$$

The maximum bending moment, at the point of fixity and at the pile cap, in each pile is

$$M_{max} = VH/2$$

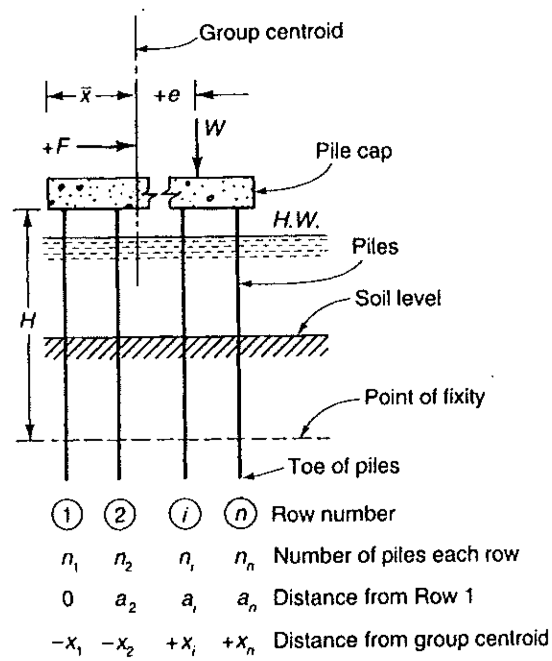


Fig. 4-27. Vertical piles with vertical and horizontal loads

Example 8

Fig. 4-28 shows the loading acting on one line of piles supporting a dock. The vertical load indicated includes the self weight of the rigid pile cap and the piles may be considered fixed at a depth of 20 feet below the bottom of the cap. Determine the shear force and maximum bending moment in each pile and the maximum axial force produced in a pile.

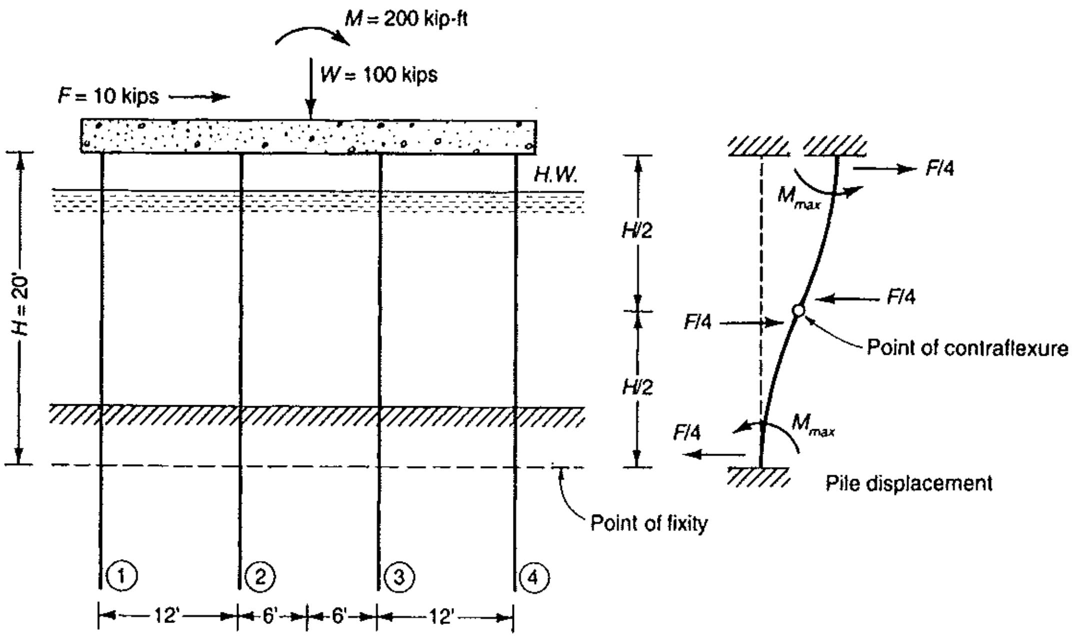


Fig. 4-28

Solution

The total moment acting on the pile group is

$M_T = M + FH$

$= 200 + 10 \times 20$

$= 400$ kip feet.

The pile group is symmetrical with an inertia of

$\Sigma x^2 = 2(6^2 + 18^2) = 720$

The maximum axial force occurs in Pile 4 and, neglecting the self weight, is given by

$P_4 = W'/n_L + M_T x_4 / \Sigma x^2$

$= 100/4 + 400 \times 18/720$

$= 35$ kips.

The shear force in each pile is

$V = F/\Sigma n$

$= 10/4$

$= 2.5$ kips.

The maximum bending moment in a pile is determined by assuming a point of contraflexure at a depth of $H/2$, as shown in Fig. 4-28. The maximum moment is:

$M_{max} = VH/2$

$= 2.5 \times 20/2$

$= 25$ kip feet.

Pile Group with Inclined Piles

The elastic center method[7,8] may be used to determine the forces in inclined piles. All piles are assumed to be hinged at each end and the rigid pile cap is assumed to rotate about the elastic center thus producing forces in the piles which balance the moment of the external load about the elastic center. For simple pile arrangements, the location of the elastic center may be determined by inspection, as shown in Fig. 4-29.

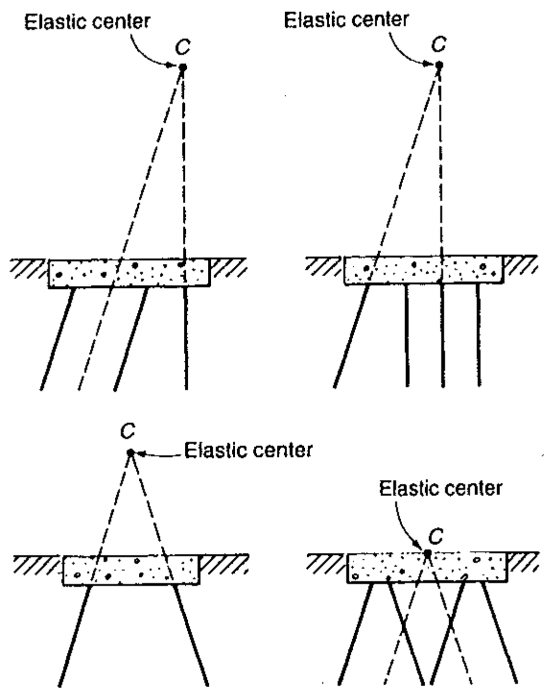


Fig. 4-29. Location of elastic center

When the line of action of the external load passes through the elastic center, no rotation of the pile cap occurs and the forces in the piles, due to the translation of the pile cap, may be obtained by resolution.

Example 9

Determine the forces, due to the indicated loads, in the raked piles shown in Fig. 4-30. The piles have a batter of 1:4 and the line of action of the external loads passes through the elastic center.

Solution

Resolving forces vertically, the vertical component of the axial force in pile 1 is given by:

$$V_1 = W/2 - FB/2$$
$$= 100/2 - 10 \times 4/2$$
$$= 30 \text{ kips}$$

The axial force in pile 1 is:

$$P_1 = V_1 \sqrt{1 + B^2} / B$$
$$= 30\sqrt{17}/4$$
$$= 30.92 \text{ kips}$$

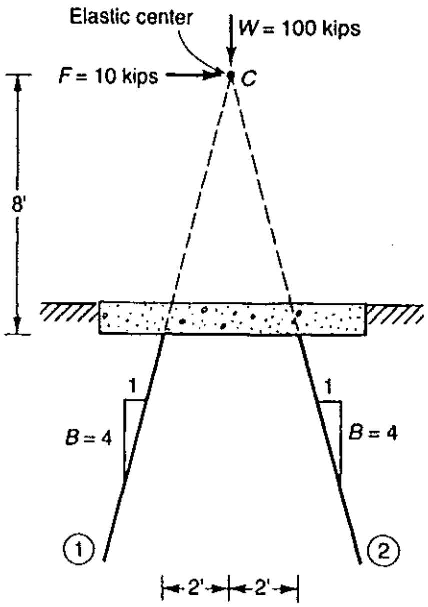


Fig. 4-30

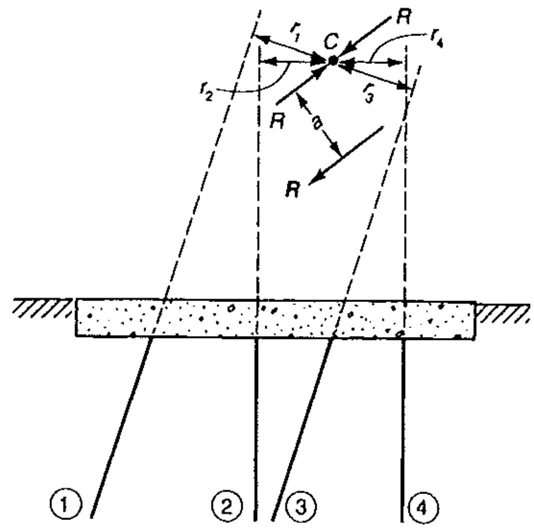


Fig. 4-31. Pile cap rotation

The vertical component of the axial force in Pile 2 is:

$V_2 = W/2 + FB/2 = 70.00$ kips.

The axial force in Pile 2 is:

$P_2 = V_2\sqrt{1+B^2}\,/\,B = 72.15$ kips.

When the line of action of the external loads does not pass through the elastic center, additional axial forces are produced in the piles. As shown in Fig. 4-31, the external load, R, may be replaced by a force, R, through the elastic center plus a couple of magnitude

$$M = Ra$$

where a is the perpendicular distance from the line of action of R to the elastic center at C. The force, R, through the elastic center produces a translation of the pile cap and the axial forces in the piles are determined by resolution as previously described.

The couple, M, causes a rotation, θ, of the pile cap which produces an axial deformation in each pile of magnitude:

$$\delta = r_i\theta$$

where r_i is the distance from the elastic center perpendicular to each pile.

The axial force produced in each pile is:

$$P_i = EA\delta/L = EAr_i\theta/L$$

This axial force produces a moment about the elastic center of:

$$M_P = P_ir_i = EA\,r_i^2\theta/L$$

For equilibrium, the sum of the moments of the pile forces about the elastic center must equal the external couple about the elastic center:

$$M = \theta \Sigma EAr^2/L$$

$$\theta = M/\Sigma EAr^2/L$$

$$P_i = (MEAr_i/L)/(\Sigma EAr^2/L) = Mr_i/\Sigma r^2 \ldots \text{ for identical piles.}$$

Example 10

Determine the forces, due to the indicated loads, in the raked piles shown in Fig. 4-32. The piles have a batter of 1:4 and have identical lengths and cross sectional areas.

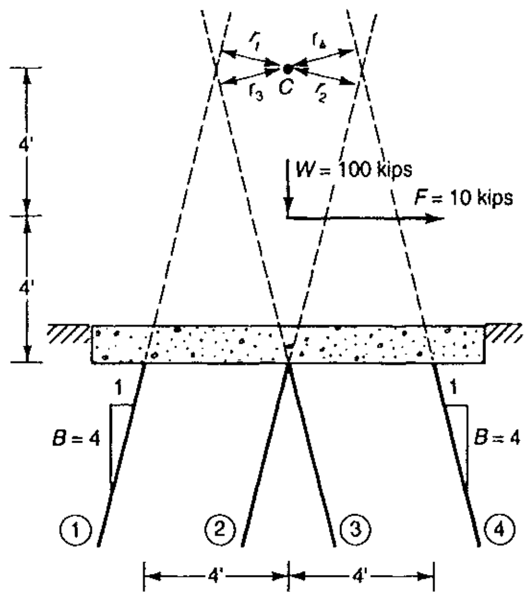


Fig. 4-32

Solution

The axial forces due to the translation are obtained from Example 4–9 as:

$P_i = P_2 = 30.92/2$

$= 15.46$ kips

$P_3 = P_4 = 72.15/2$

$= 36.08$ kips

The couple about the elastic center due to the applied load is:

$M = 4F$

$= 4 \times 10$

$= 40$ kip feet.

The distance from the elastic center perpendicular to each pile is:

$r_1 = r_2 = r_3 = r_4 = 2 \cos 14.04°$

$= 1.94$ feet.

$\Sigma r^2 = 4(1.94)^2$

$= 15.05$

The axial forces in the piles due to rotation of the pile cap are:

$P_1 = P_3 = + Mr/\Sigma r^2$

$= 40 \times 1.94/15.05$

$= 5.15$ kips . . . compression.

$P_2 = P_4 = 5.15$ kips . . . tension

The final pile forces are given by the sum of the forces due to translation and rotation and are:

$P_1 = 15.46 + 5.15 = 20.61$ kips

$P_2 = 15.46 - 5.15 = 10.31$ kips

$P_3 = 36.08 + 5.15 = 41.23$ kips

$P_4 = 36.08 - 5.15 = 30.93$ kips.

Pile Cap Design

To design the tension reinforcement in a pile cap, either a beam analogy method or a truss analogy method may be employed[9].

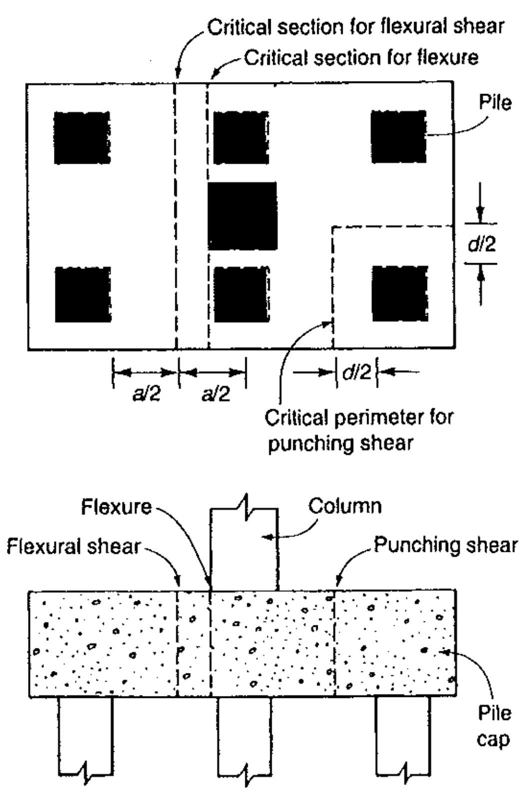


Fig. 4-33. Pile cap beam analogy

The pile cap depth is governed by shear requirements or by the necessity to provide adequate anchorage length for the reinforcement projecting from the piles or for the column starter bars.

The critical perimeter for punching shear is located a distance of $d/2$ from the face of the column or from the face of a pile. Punching shear at the column will generally not control since some piles are normally within the critical perimeter. As shown in Fig. 4-33, punching shear for a corner pile is most severe. The capacity of the pile cap for punching shear is the minimum value given by ACI Equations (11-36), (11-37) and (11-38) as

$$\phi V_c = (2 + 4/\beta_c)\phi b_o d\sqrt{f_c'}$$

$$\phi V_c = 4\phi b_o d\sqrt{f_c'}$$

$$\phi V_c = (\alpha_s d/b_o + 2)\phi b_o d\sqrt{f_c'}$$

where, β_c = ratio of long side to short side of column

ϕ = strength reduction factor = 0.85 from ACI Section 9.3

b_o = length of critical perimeter for punching shear

α_s = 40 ... interior piles

= 30 ... edge piles

= 20 ... corner piles.

In accordance with ACI Section 11.8.1, the pile cap may be classified as a deep flexural member when the ratio of the clear distance between piles to the effective depth is

$$l_n/d < 5$$

In this case, ACI, Section 11.8.5 specifies that the critical section for flexural shear is located a distance from the face of a pile given by

$$x = a/2$$
$$\leq \mathrm{d}$$

where, a = shear span = distance from face of nearest pile to center of column.
The capacity of the pile cap for flexural shear, across the full width of the cap, is given by ACI Equation (11-28) as

$$\phi V_c = 2\phi db\sqrt{f_c'}$$

Alternatively, advantage may be taken of the enhancement in shear strength due to the short shear span by utilizing ACI Equation (11-29) to give a capacity of

$$\phi V_c = \phi(3.5 - 2.5M_u/V_u d)(1.9\sqrt{f_c'} + 2500\rho V_u d/M_u)bd$$
$$\leq 6\phi db\sqrt{f_c'}$$

where, ρ = reinforcement ratio

V_u = factored shear at critical section

M_u = factored moment at the same section.

The location of the critical section for flexure is, in accordance with ACI Section 15.4.2, at the face of the column. The area of tension reinforcement required is obtained using the principles of ACI Section 10.3.

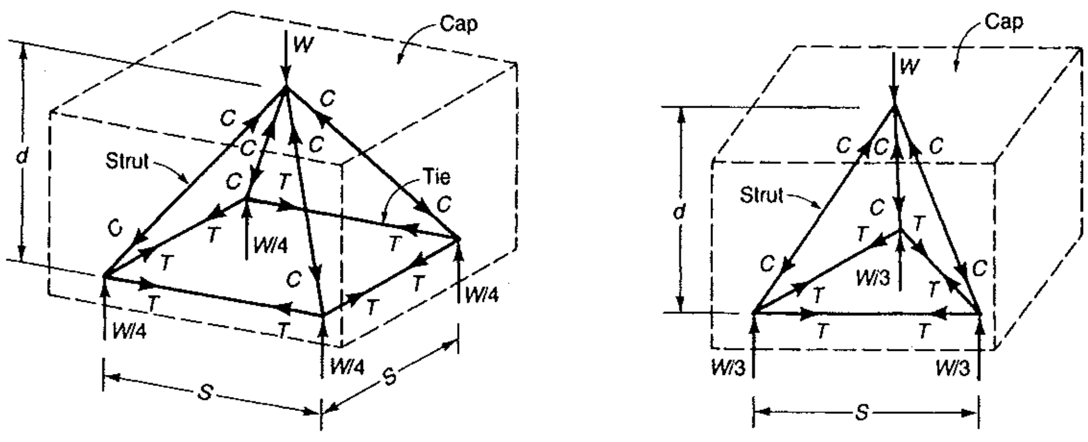


Fig. 4-34. Pile cap truss analogy

Alternatively, when the piles are symmetrically arranged about the column and do not exceed five in number, the truss analogy method may be used as shown in Fig. 4-34. The pile cap is assumed to act as a three dimensional triangulated space truss. The upper node of the truss is located at the center of the column with the lower nodes at the centers of the piles. The column load is transmitted to the piles by direct thrust on inclined planes through the concrete with the reinforcement between the piles forming horizontal ties. The reinforcement must be placed within a band width[10] not exceeding three times the width of the piles. Since the reinforcement is fully stressed at the center of the piles, a full anchorage length must be provided beyond this point. By resolving forces at the lower nodes, the tensile force in the reinforcement of a four pile group is given by:

$T = Ws/8d.$

In a three pile group the tensile force is

$T = Ws/9d.$

In a two pile group the tensile force is

$T = Ws/4d.$

In a five pile group the tensile force is

$T = Ws/10d.$

The area of reinforcement required is

$A_s = T_u/\phi f_y$

where, ϕ = strength reduction factor = 0.90 from ACI Section 9.3,

T_u = factored tie force,

f_y = reinforcement yield strength,

W = axial load in column, and

s = distance between pile centers.

Example 11

Determine the Grade 60 reinforcement required in the pile cap shown in Fig. 4-35 and check the shear capacity. The concrete strength is 3000 pounds per square inch.

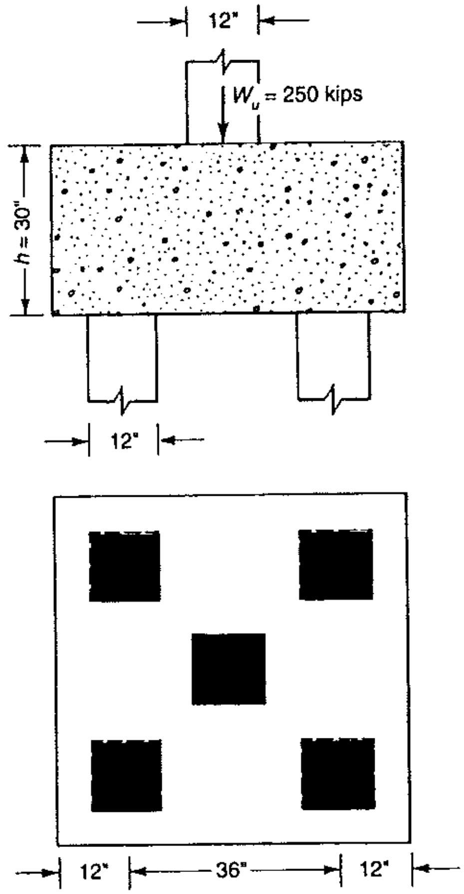


Fig. 4-35

Solution

The force in each of the reinforcement ties is:

$T_u = W_u s/8d$

$= 500 \times 3/(8 \times 2)$

$= 93.75$ kips

The reinforcement area required for each tie is:

$A_s = T_u/\phi f_y$

$= 93.75/(0.9 \times 60)$

$= 1.74$ square inches.

Since there are two ties in each direction, the total reinforcement area in each direction is:

$A_s = 2 \times 1.74$

$= 3.48$ square inches.

Providing eight Number 6 bars gives an area of

$A'_s = 3.52$ square inches

$> A_s$ . . . satisfactory.

This gives a reinforcement ratio, based on the gross cross sectional area of

$\rho' = A'_s/bh$

$= 3.52/(60 \times 30)$

$= 0.0020$

The minimum reinforcement area in a pile cap for Grade 60 bars, based on the gross cross sectional area, is given by ACI Section 7.12 as

$\rho'_{min} = 0.0018$

< 0.0020 . . . satisfactory.

The ratio of the clear distance between piles to the effective depth is

$l_n/d = 2/2$

$= 1$

$< 5.$

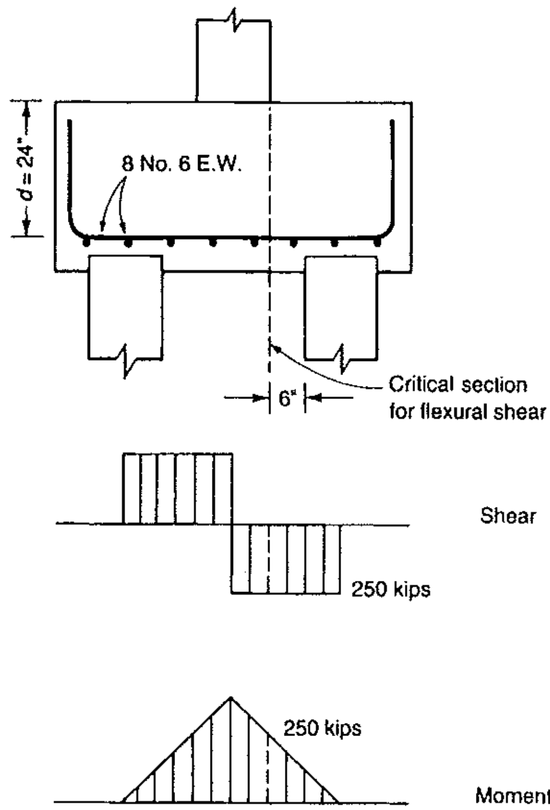


Fig. 4-35(a)

In accordance with ACI, Section 11.8.1 the pile cap may be classified as a deep flexural member and the critical section for flexural shear is given by ACI Section 11.8.5, as a distance from the face of the pile:

$$x = a/2$$
$$= (1.5 - 0.5)/2$$
$$= 0.5 \text{ feet.}$$

The factored shear at this location is:

$V_u = W_u/2$

$= 500/2$

$= 250$ kips.

The factored moment at the same section is:

$M_u = W_u (x + 0.5)/2$

$= 500 \times 1.0/2$

$= 250$ kip feet.

The reinforcement ratio provided is:

$\rho = A'_s/bd$

$= 3.52/(60 \times 24)$

$= 0.0024.$

The flexural shear capacity is obtained from ACI Equation (11-29) as:

$$\phi V_c = \phi(3.5 - 2.5M_u/V_u d)(1.9\sqrt{f_c'} + 2500\rho V_u d/M_u)bd$$

$$= 0.85[3.5 - 2.5 \times 250/(250 \times 2)](1.9\sqrt{3000} + 2500 \times 0.0024 \times 250 \times 2/250)60 \times 24$$

$$= 0.85 \times 2.25(104 + 12)1440$$

$$= 0.85 \times 261 \times 1440$$

$$= 319{,}464 \text{ pounds}$$

$$> V_u \ldots \text{satisfactory}$$

$$< 6\phi bd\sqrt{f_c'} \ldots \text{satisfactory.}$$

The flexural shear capacity is adequate.

Punching shear for the column is not applicable since the piles are located within the critical perimeter of the column. The length of the critical perimeter for a pile is:

$$b_o = 2(18 + d/2)$$

$$= 2(18 + 24/2)$$

$$= 60 \text{ inches}$$

The capacity of the pile cap for punching shear at a pile is given by ACI Equation (11-37) as:

$$\phi V_c = 4\phi b_o d\sqrt{f_c'}$$

$$= 4 \times 0.85 \times 60 \times 24\sqrt{3000}$$

$$= 268{,}165 \text{ pounds}$$

$$> W_u/4 \ldots \text{satisfactory.}$$

The punching shear capacity of the pile cap is adequate.

Selected Symbols and Abbreviations

Symbol or Abbreviation	Description
c_1	short side of column
c_2	long side of column
d	effective depth of footing
A_s	total required reinforcement area
$\beta = l_2/l_1$	ratio of long side to the short side of the footing
ϕ	strength reduction factor
b	width of footing
f'_c	concrete strength
$\beta_c = c_2/c_1$	ratio of the long side to the short side of the column
b_0	length of critical perimeter for punching shear
f_y	reinforcement yield strength
$A_{s(\min)}$	minimum area of reinforcement
f_p	bearing pressure
t_p	base plate thickness
P_u	factored applied load
μ	coefficient of friction
B	width of footing
L	length of footing
W	weight
K	Rankine coefficient
H	earth pressure
H_A	total active pressure behind wall
H_P	total passive pressure in front of wall
V_u	factored shear at critical section
M_u	factored moment at critical section

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Problems and Solutions

4-1. The stub column shown in Fig. 4-1 supports a steel column and base plate with the indicated loads. The stub column is 18 inches square and is reinforced with Grade 60 deformed bars. Concrete compressive strength is 2000 pounds per square inch. The stub column is subjected to axial load only and is effectively braced against sidesway by the floor slab.

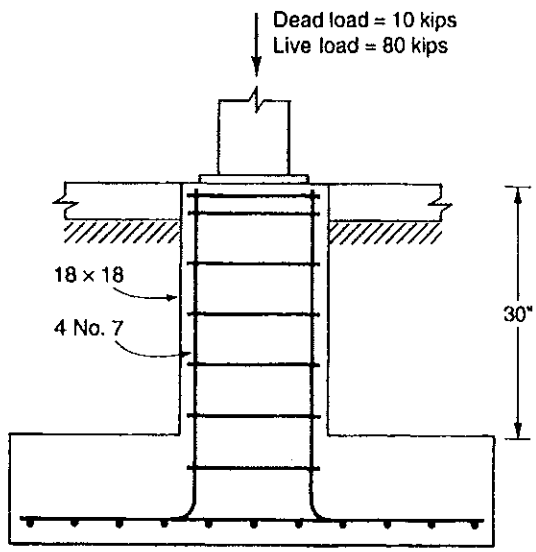

Fig. 4-1

a) What is the effective length factor of the stub column?

(a) 0.65

(b) 0.80

(c) 1.00

(d) 1.20

(e) 2.10

b) The slenderness ratio is most nearly

(a) 6

(b) 7

(c) 8

(d) 9

(e) 10

c) Do slenderness effects have to be considered?

(a) Yes

(b) No

d) The design axial load strength of the stub column is given most nearly by

(a) 330 kips
(b) 350 kips
(c) 370 kips
(d) 390 kips
(e) 410 kips

e) The allowable reduced effective area of the section is given most nearly by

(a) 160 square inches
(b) 170 square inches
(c) 180 square inches
(d) 190 square inches
(e) 200 square inches

f) The minimum allowable reinforcement area in the stub column is most nearly

(a) 1.6 square inches
(b) 1.7 square inches
(c) 1.8 square inches
(d) 1.9 square inches
(e) 2.0 square inches

g) The minimum allowable size of lateral ties is

(a) Number 3
(b) Number 4
(c) Number 5
(d) Number 6
(e) Number 7

h) The maximum allowable tie spacing is most nearly

(a) 10 inches
(b) 12 inches
(c) 14 inches
(d) 16 inches
(e) 18 inches

Solution

a) From ACI Section 10.12.1, the effective length factor for a column braced against side sway is

$k = 1.0$

The answer is (c).

b) The radius of gyration, in accordance with ACI Section 10.11.2 is

$$\begin{aligned} r &= 0.3c \\ &= 0.3 \times 18 \\ &= 5.4 \text{ inches.} \end{aligned}$$

The slenderness ratio, in accordance with ACI Section 10.11.4 is

$$\begin{aligned} kl_u/r &= 1.0 \times 30/5.4 \\ &= 5.6 \end{aligned}$$

The answer is (a).

c) From ACI Section 10.12.2, slenderness effects may be neglected when:

$kl_u/r < 34 - 12M_{1b}/M_{2b}$ where $M_{1b}/M_{2b} \geq -0.5$ and $5.6 < 40$.

The answer is (b).

d) For a column with lateral ties, the design axial load strength is given by Eq. (10-2) as:

$$\begin{aligned} \phi P_n &= 0.80\phi[0.85f_c'(A_g - A_{st}) + f_y A_{st}] \\ &= 0.8 \times 0.7[0.85 \times 2\,(324 - 2.4) + 60 \times 2.4] \\ &= 387 \text{ kips.} \end{aligned}$$

The answer is (d).

e) The ratio of design load strength to applied ultimate load is:

$$\begin{aligned} \phi P_n/P_u &= 387/(1.4 \times 10 + 1.7 \times 80) \\ &= 2.6 \\ &> 2. \end{aligned}$$

From ACI Section 10.8.4 the reduced effective area of the section is:

$$\begin{aligned} A_g' &= A_g/2 \\ &= 324/2 \\ &= 162 \text{ square inches} \end{aligned}$$

The answer is (a).

f) The minimum allowable reinforcement area, in accordance with ACI Section, 10.9.1 is:

$$\rho_{min} = 0.01A'_g$$
$$= 0.01 \times 162$$
$$= 1.62 \text{ square inches}$$

The answer is (a).

g) The minimum allowable size of lateral ties for enclosing bars smaller than Number 10 is given by ACI Section, 7.10.5.1 as Number 3.

The answer is (a).

h) The ACI, Section 7.10.5.2, specifies a tie spacing not greater than:

$16d_b = 16 \times 7/8 = 14$ inches

$48d_t = 48 \times 3/8 = 18$ inches

$c = 18$ inches

The answer is (c).

4-2. The fill behind the retaining wall in Fig. 4-2 has a unit weight of 110 pounds per cubic foot with an equivalent fluid pressure of 30 pounds per square foot per foot. Passive earth pressure may be assumed equivalent to a fluid pressure of 300 pounds per square foot per foot and the coefficient of friction at the underside of the base is 0.4. All concrete has a compressive strength of 3000 pounds per square inch and reinforcement consists of Grade 60 deformed bars. For Questions a to i, neglect the effects of the shear key.

a) The value of the overturning moment about the toe is most nearly

- (a) 52,000 pounds feet
- (b) 54,000 pounds feet
- (c) 56,000 pounds feet
- (d) 58,000 pounds feet
- (e) 60,000 pounds feet

b) The distance of the resultant vertical load from the toe is most nearly

- (a) 5.5 feet
- (b) 6.0 feet
- (c) 6.5 feet
- (d) 7.0 feet
- (e) 7.5 feet

c) The factor of safety against overturning is most nearly

(a) 2.3
(b) 2.5
(c) 2.7
(d) 2.9
(e) 3.1

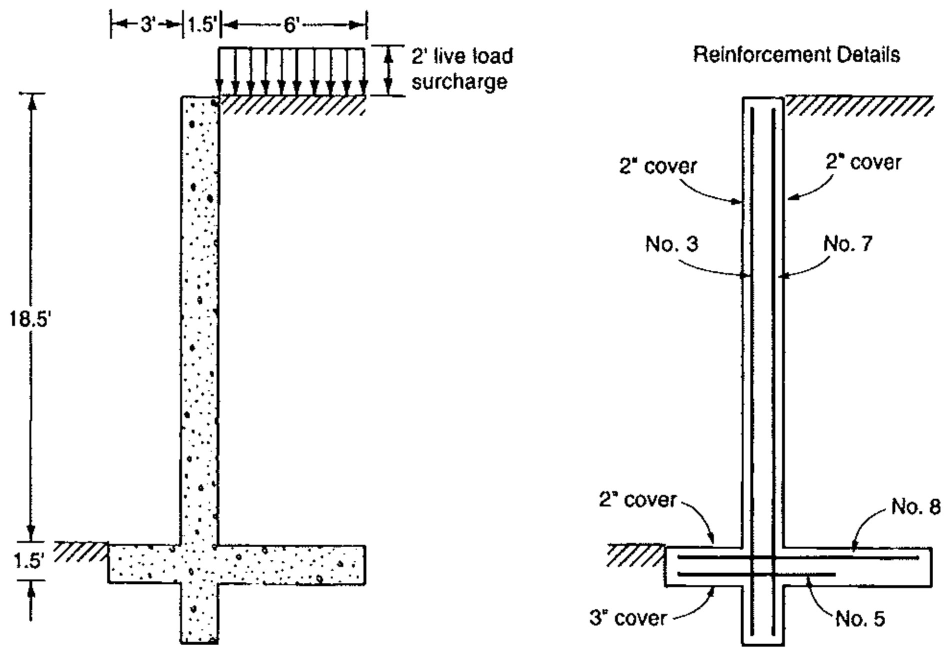

Fig. 4-2

d) The earth pressure at the toe is most nearly

(a) 3170 pounds per square foot
(b) 3220 pounds per square foot
(c) 3270 pounds per square foot
(d) 3320 pounds per square foot
(e) 3370 pounds per square foot

e) The earth pressure at the heel is most nearly

(a) 0 pounds per square foot
(b) 250 pounds per square foot
(c) 350 pounds per square foot
(d) 450 pounds per square foot
(e) 550 pounds per square foot

f) The maximum spacing of Number 8 bars in the heel is most nearly

(a) 11 inches
(b) 12 inches
(c) 13 inches
(d) 14 inches
(e) 15 inches

g) The maximum spacing of Number 5 bars in the toe is most nearly

(a) 11 inches
(b) 12 inches
(c) 13 inches
(d) 14 inches
(e) 15 inches

h) The maximum spacing of Number 7 bars in the stem is most nearly

(a) 5.5 inches
(b) 6.0 inches
(c) 6.5 inches
(d) 7.0 inches
(e) 7.5 inches

i) The maximum spacing of Number 3 bars in the front face of the stem is most nearly

(a) 14 inches
(b) 13 inches
(c) 12 inches
(d) 11 inches
(e) 10 inches

j) Combining the effects of friction and passive pressure, the minimum depth of the shear key to provide a factor of safety of 1.5 against sliding is most nearly:

(a) 2.5 feet
(b) 2.8 feet
(c) 3.1 feet
(d) 3.4 feet
(e) 3.7 feet

Solution

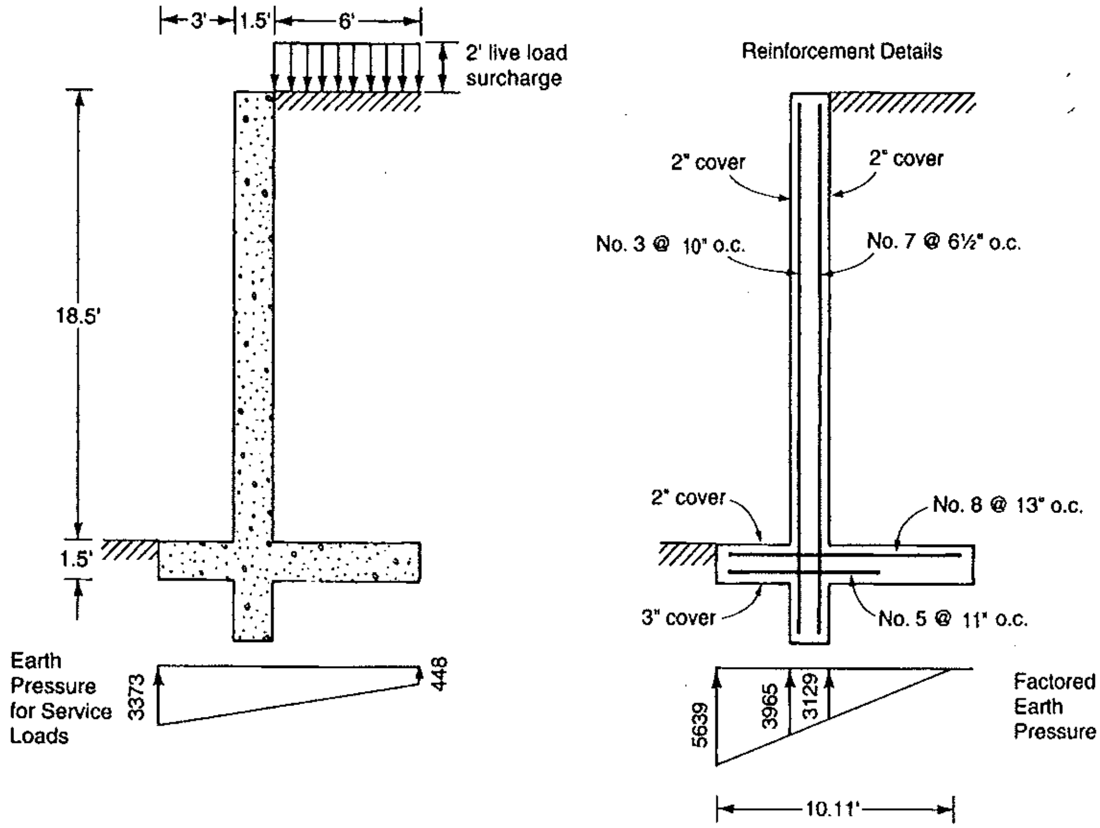

Fig. 4-2(a)

a) The horizontal service loads acting on the wall are given by:

H_A = lateral pressure from backfill

$= 30 \times 20^2/2$

$= 6000$ pounds.

H_L = lateral pressure from surcharge

$= 2 \times 30 \times 20$

$= 1200$ pounds.

Overturning moment about the toe is:

$M_O = H_A \times H/3 + H_L \times H/2$

$= 6000 \times 20/3 + 1200 \times 20/2$

$= 52{,}000$ pounds feet.

The answer is (a).

b) The vertical service loads acting on the wall are given by:

W_W = weight of stem wall

$= 150 \times 1.5 \times 18.5$

$= 4163$ pounds.

W_B = weight of base

$= 150 \times 1.5 \times 10.5$

$= 2363$ pounds.

W_S = weight of backfill

$= 110 \times 6 \times 18.5$

$= 12{,}210$ pounds.

W_L = weight of surcharge

$= 110 \times 6 \times 2$

$= 1320$ pounds.

The total vertical load is:

$\sum W = 20{,}056$

The distance of the resultant vertical load from the toe is:

$\bar{x} = (4163 \times 3.75 + 2363 \times 5.25 + 12{,}210 \times 7.5 + 1320 \times 7.5)/20{,}056$

$= 6.5$ feet.

The answer is (c).

c) The factor of safety against overturning is:

$\bar{x}\sum W/M_O = 6.5 \times 20{,}056/52{,}000$

$= 2.5$

The answer is (b).

d) The eccentricity of the applied loads about the toe is given by:

$e' = (\bar{x}\sum W - M_O)/\sum W$

$= (6.5 \times 20{,}056 - 52{,}000)/20{,}056$

$= 3.91$ feet.

The eccentricity about the base centroid is:

$e = 10.5/2 - 3.91$

$= 1.34$ feet

$< L_B/6$ . . . no tension under the base.

The pressure under the toe is:

$q_{(toe)} = \sum W(1 + 6e/L_B)/BL_B$

$= 20{,}056\ (1 + 6 \times 1.34/10.5)/(1 \times 10.5)$

$= 3373$ pounds per square foot.

The answer is (e).

e) The pressure under the heel is:

$q_{(\text{heel})} = \Sigma W(1 - 6e/L_B)/BL_B$

$= 448$ pounds per square foot.

The answer is (d).

f) The factored total overturning moment about the toe is:

$\gamma M_O = 1.7 \times 52{,}000$

$= 88{,}400$ pounds feet.

The factored total vertical load is:

$\Sigma\gamma W = 1.4(W_W + W_B + W_S) + 1.7W_L$

$= 5828 + 3308 + 17{,}094 + 2244$

$= 28{,}474$ pounds.

The factored restoring moment is:

$\gamma M_R = 5828 \times 3.75 + 3308 \times 5.25 + 17{,}094 \times 7.5 + 2244 \times 7.5$

$= 184{,}257$ pounds feet.

The eccentricity of the factored loads about the toe is given by:

$e' = (\gamma M_R - \gamma M_O)/\Sigma\gamma W$

$= 3.37$ feet

$< L_B/3$ . . . outside middle third.

The pressure under the toe is given by:

$q_{(\text{toe})} = 2\Sigma\gamma W/3e'$

$= 2 \times 28{,}474/(3 \times 3.37)$

$= 5639$ pounds per square foot.

The pressure distribution under the base is shown in Figure 4-2(a).
The maximum factored bending moment in the heel is:

$M_u = 3(\gamma W_S + \gamma W_L + 6\gamma W_B/10.5) - 3129 \times 5.61^2/6$

$= 3\,(17{,}094 + 2244 + 1890) - 16{,}413$

$= 47{,}271$ pounds feet.

The required reinforcement ratio is derived from ACI, Section 10.2, with the aid of a calculator program,[3] and is:

$\rho = 0.85f_c'(1 - \sqrt{1 - 2K/0.765f_c'})/f_y$

$= 0.38$ percent.

The maximum reinforcement ratio is given by ACI, Section 10.3 as:

$\rho_{\max} = 0.75 \times 0.85 \times 87\beta_1 f_c'/f_y(87 + f_y)$

$= 1.60$ percent

$> \rho$ satisfactory.

The minimum reinforcement ratio is given by ACI, Section 7.12 as:

$\rho_{min} = 0.18$ percent of the gross area

$< \rho$ . . satisfactory.

Thus, the required reinforcement area is:

$A_s = \rho bd$

$= 0.0038 \times 12 \times 15.5$

$= 0.71$ square inches per foot.

Shear is not critical and Number 8 bars at a spacing of 13 inches provides a reinforcement area of:

$A'_s = 0.72$ square inches per foot

$> A_s$ . . . satisfactory.

The answer is (c).

g) The maximum factored bending moment in the toe is:

$M_u = 3^2(3965 + 2 \times 5639)/6 - 1.5 \times 3\gamma W_B/10.5$

$= 22{,}865 - 1418$

$= 21{,}447$ pounds feet.

The required reinforcement ratio is:

$\rho = 0.188$ percent . . . satisfactory.

The required reinforcement area is:

$A_s = 0.00188 \times 12 \times 14.69$

$= 0.33$ square inches per foot.

Shear is not critical and Number 5 bars at a spacing of 11 inches provides a reinforcement area of:

$A'_s = 0.34$ square inches per foot

$> A_s$ . . . satisfactory.

The answer is (a).

h) The maximum factored bending moment in the stem is:

$M_u = 1.7(30 \times 18.5^3)/6 + 60 \times 18.5^2/2)$

$= 71{,}274$ pounds feet.

The required reinforcement ratio is

$\rho = 0.58$ percent . . . satisfactory.

The required reinforcement area is:

$A_s = 0.0058 \times 12 \times 15.5$

$= 1.08$ square inches per foot.

Shear is not critical and Number 7 bars at a spacing of 6.5 inches provides a reinforcement area of

$A'_s = 1.12$ square inches per foot

$> A_s$ . . . satisfactory.

The answer is (c).

i) Based on the gross concrete area, the reinforcement ratio required for the vertical reinforcement in the outer face of the wall is given by ACI, Section 14.3 as

$\rho = 0.0012/2$ for bars not larger than Number 5.

The required vertical reinforcement area is

$A_s = 0.0012 \times 12 \times 18/2$

$= 0.13$ square inches per foot.

Providing Number 3 bars at a spacing of 10 inches gives

$A'_s = 0.13$ square inches per foot

$= A_s$, satisfactory.

The answer is (e).

j) The applied horizontal force on the retaining wall is:

$H = H_A + H_L$

$= 6000 + 1200$

$= 7200$ pounds.

The frictional resistance available is:

$F = \mu \Sigma W$

$= 0.4 \times 20{,}056$

$= 8022$ pounds.

To provide a factor of safety of 1.5 against sliding requires a passive resistance of:

$H_P = 1.5H - F$

$= 1.5 \times 7200 - 8022$

$= 2777$ pounds.

To provide this passive force, the total required depth of the shear key plus footing is:

$H_K = \sqrt{2H_P / p_P}$

$= \sqrt{2 \times 2777 / 300}$

$= 4.3$ feet.

The depth of the shear key is:

$D = H_K - 1.5$

$= 2.8$ feet.

The answer is (b).

4-3. The ties for the anchored bulkhead shown in Fig. 4-3 are located four feet from the top of the sheetpiling and are spaced at fifteen foot centers. The end of the tie is secured to two anchor piles raked as indicated. The active earth pressure may be assumed equivalent to a fluid pressure of 30 pounds per square foot per foot and passive pressure may be assumed equivalent to a fluid pressure of 400 pounds per square foot per foot.

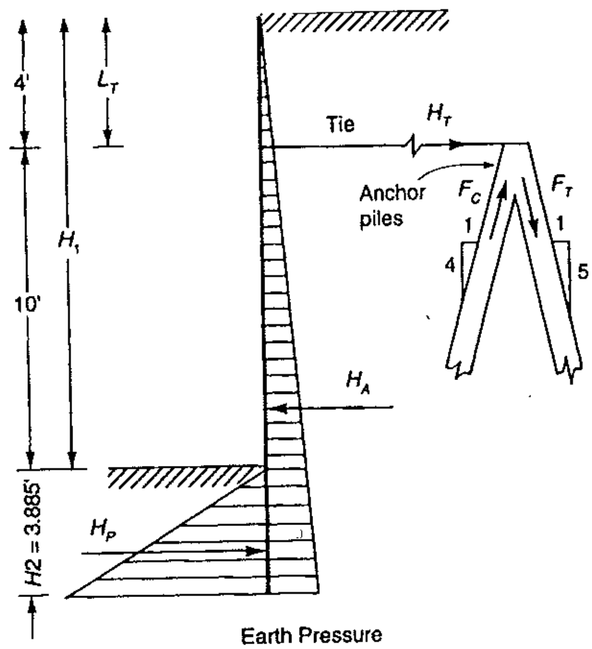

Fig. 4-3

a) The value of the total active pressure on the back of the wall is most nearly

(a) 1800 pounds per foot
(b) 2400 pounds per foot
(c) 3000 pounds per foot
(d) 3900 pounds per foot
(e) 4800 pounds per foot

b) The value of the total passive pressure on the front of the wall is most nearly

(a) 1800 pounds per foot
(b) 2400 pounds per foot
(c) 3000 pounds per foot
(d) 3900 pounds per foot
(e) 4800 pounds per foot

c) The moment of all forces about the tie point is most nearly

(a) 0 pounds feet per foot

(b) 200 pounds feet per foot

(c) 400 pounds feet per foot

(d) 600 pounds feet per foot

(e) 800 pounds feet per foot

d) The force in the tie is most nearly

(a) 1800 pounds per foot

(b) 2400 pounds per foot

(c) 3000 pounds per foot

(d) 3900 pounds per foot

(e) 4800 pounds per foot

e) The distance from the top of the wall to the position of maximum shear is most nearly

(a) 0 feet

(b) 4 feet

(c) 9 feet

(d) 11 feet

(e) 13 feet

f) The maximum shear in the wall is most nearly

(a) 1240 pounds

(b) 1340 pounds

(c) 1440 pounds

(d) 1540 pounds

(e) 1640 pounds

g) The distance from the top of the wall to the position of maximum moment is most nearly

(a) 0 feet

(b) 4 feet

(c) 9 feet

(d) 11 feet

(3) 13 feet

h) The maximum moment in the wall is most nearly

(a) 5600 pounds feet per foot
(b) 5700 pounds feet per foot
(c) 5800 pounds feet per foot
(d) 5900 pounds feet per foot
(e) 6000 pounds feet per foot

i) The force in the compression anchor pile is most nearly

(a) 60.5 kips
(b) 61.1 kips
(c) 61.7 kips
(d) 62.4 kips
(e) 63.1 kips

j) The force in the tension anchor pile is most nearly

(a) 60.5 kips
(b) 61.1 kips
(c) 61.7 kips
(d) 62.4 kips
(e) 63.1 kips

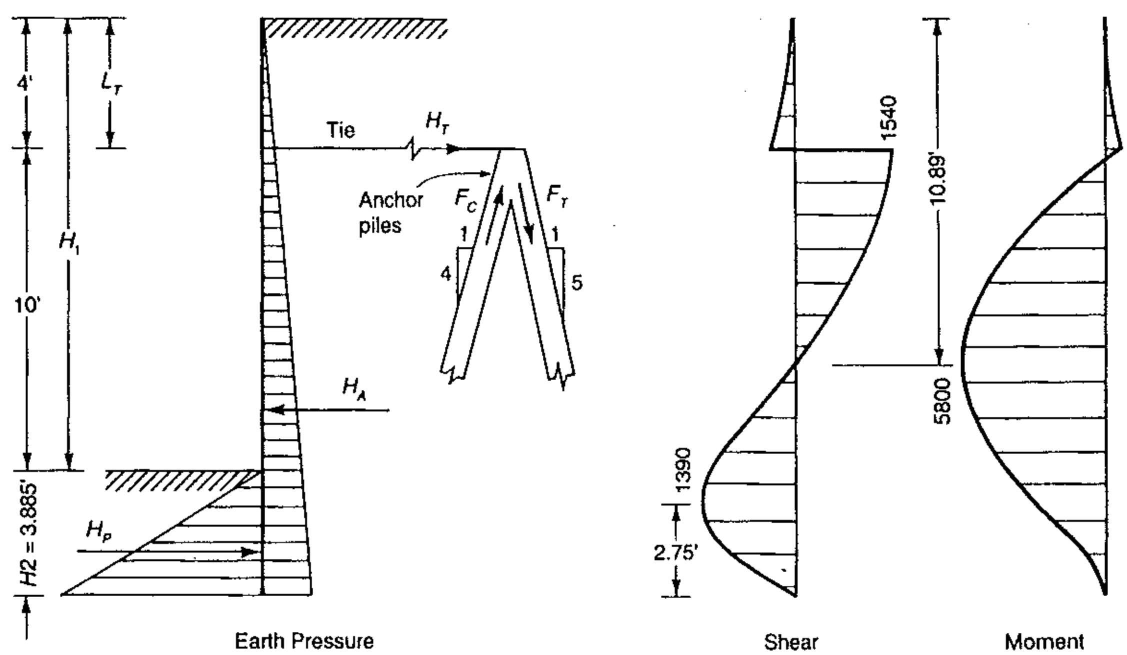

Fig. 4-3(a)

Solution

a) The total active pressure on the back of the wall is:

$H_A = p_A(H_1 + H_2)^2/2$

$= 30\ (14 + 3.885)^2/2$

$= 4798$ pounds per foot.

The answer is (e).

b) The total passive pressure on the front of the wall is:

$H_P = p_P H_2^2/2$

$= 400 \times 3.885^2/2$

$= 3018$ pounds per foot.

The answer is (c).

c) The bending moment at the tie point is:

$M_T = H_A[2(H_1 + H_2)/3 - L_T] - H_P(H_1 - L_T + 2H_2/3)$

$= 4798\ (2 \times 17.885)/3 - 4) - 3018(10 + 2 \times 3.885/3)$

$= 7$ pounds feet per foot.

The answer is (a).

d) The force in the tie is:

$H_T = H_A - H_P$

$= 4798 - 3018$

$= 1780$ pounds.

The answer is (a).

e) The maximum shear occurs at the tie point which is four feet from the top of the sheetpiling.

The answer is (b).

f) The maximum shear is given by:

$V_{\max} = H_T - p_A L_T^2/2$

$= 1780 - 240$

$= 1540$ pounds per foot.

The answer is (d).

g) The shear force at a distance x from the top of the sheetpiling, when x exceeds L_T, is:

$V = H_T - p_A x^2/2$

$= 1780 - 15x^2$

The maximum moment occurs when:

$V = 0$

Hence, $x = \sqrt{1780/15}$

$= 10.89$ feet.

The answer is (d).

h) The maximum moment is:

$M_{max} = H_T(x - L_T) - p_A x^3/6$

$= 1780\,(10.89 - 4) - 30 \times 10.89^3/6$

$= 5800$ pounds feet per foot.

The answer is (c).

i) The force in the compression pile is:

$F_C = 15H_T/(\sin\theta_1 + \cos\theta_1 \tan\theta_2)$

$= 15 \times 1780/(\sin 14° + \cos 14° \tan 11.3°)$

$= 15 \times 1780/(0.243 + 0.970 \times 0.200)$

$= 61{,}094$ pounds.

The answer is (b).

j) The force in the tension pile is:

$F_T = 15H_T/(\sin\theta_2 + \cos\theta_2 \tan\theta_1)$

$= 15 \times 1780/(\sin 11.3° + \cos 11.3° \tan 14°)$

$= 15 \times 1780/(0.196 + 0.981 \times 0.250)$

$= 60{,}524$ pounds.

The answer is (a).

4-4. The fill behind the retaining wall in Fig. 4-4 has a unit weight of 110 pounds per cubic foot with an equivalent fluid pressure of 30 pounds per square foot per foot. The live load surcharge behind the wall is equivalent to an additional two feet of fill. The 12-inch square piles are spaced at 5 feet on center longitudinally.

a) The value of the total factored vertical load is most nearly

(a) 138 kips

(b) 148 kips

(c) 158 kips

(d) 168 kips

(e) 178 kips

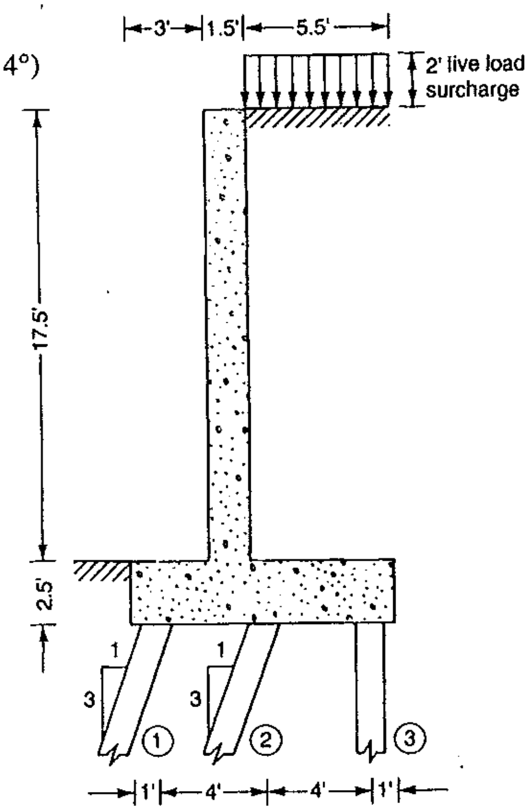

Fig. 4-4

b) The value of the total factored horizontal load is most nearly

(a) 51 kips

(b) 61 kips

(c) 71 kips

(d) 81 kips

(e) 91 kips

c) The height of the elastic center above the base is most nearly

(a) 12 feet

(b) 15 feet

(c) 18 feet

(d) 21 feet

(e) 24 feet

d) The horizontal distance of the elastic center from the toe is most nearly

(a) 5 feet

(b) 6 feet

(c) 7 feet

(d) 8 feet

(e) 9 feet

e) The moment of the factored loads about the elastic center is most nearly

(a) 202 kip feet

(b) 222 kip feet

(c) 242 kip feet

(d) 262 kip feet

(e) 282 kip feet

f) The force in Pile 1, due to translation of the pile cap, is most nearly

(a) 45 kips

(b) 69 kips

(c) 96 kips

(d) 135 kips

(e) 165 kips

g) The force in Pile 1, due to rotation of the pile cap, is most nearly

(a) 45 kips
(b) 69 kips
(c) 96 kips
(d) 135 kips
(e) 165 kips

h) The total force in Pile 2 is most nearly

(a) 45 kips
(b) 69 kips
(c) 96 kips
(d) 135 kips
(e) 165 kips

i) The total force in Pile 3 is most nearly

(a) 45 kips
(b) 69 kips
(c) 96 kips
(d) 135 kips
(e) 165 kips

j) The maximum shear in the base is most nearly

(a) 45 kips
(b) 69 kips
(c) 96 kips
(d) 135 kips
(e) 165 kips

Solution

a) The factored vertical loads for a 5 foot length of wall are given by:

W_W = factored weight of stem wall

$= 5 \times 1.4 \times 150 \times 1.5 \times 17.5/1000$

$= 27.56$ kips.

W_B = factored weight of base

$= 5 \times 1.4 \times 150 \times 2.5 \times 10/1000$

$= 26.25$ kips.

W_S = factored weight of backfill

$= 5 \times 1.4 \times 110 \times 5.5 \times 17.5/1000$

$= 74.11$ kips.

W_L = factored weight of surcharge

$= 5 \times 1.7 \times 110 \times 5.5 \times 2/1000$

$= 10.29$ kips.

W = the total factored vertical load

$= 27.56 + 26.25 + 74.11 + 10.29$

$= 138.21$ kips.

The answer is (a).

b) The factored horizontal loads for a 5 foot length of wall are given by:

H_A = factored lateral pressure from backfill

$= 5 \times 1.7 \times 30 \times 20^2/2000$

$= 51.00$ kips.

H_L = factored lateral pressure from backfill

$= 5 \times 1.7 \times 2 \times 30 \times 20/1000$

$= 10.20$ kips.

F = the total factored horizontal load

$= 51.00 + 10.20$

$= 61.20$ kips.

The answer is (b).

c) The height of the elastic center above the base is:

$L_y = 3 \times 6$

$= 18$ feet.

The answer is (c).

d) The horizontal distance of the elastic center from the toe is:

$L_x = 9$ feet.

The answer is (e).

e) The clockwise moment of the factored loads about the elastic center is:

$M = 11.33H_A + 8H_L - 5.25W_W - 4W_B - 1.75(W_S + W_L)$

$= 578.0 + 81.6 - 144.7 - 105.0 - 147.7$

$= 262.2$ kip feet.

The answer is (d).

f) Resolving forces, the axial force in Pile 1 due to translation of the pile cap is given by:

$$P_{T1} = F\sqrt{1+3^2}/2$$
$$= 61.2\sqrt{10}/2$$
$$= 96.77 \text{ kips} \ldots \text{compression;}$$

The answer is (c).

g) The distance from the elastic center perpendicular to each pile is:

$$r_1 = r_2 = 2 \cos 18.43°$$
$$= 1.90 \text{ feet.}$$
$$r_3 = 0$$
$$\Sigma r^2 = 2(1.90)^2$$
$$= 7.2.$$

The axial force in Pile 1 due to rotation of the pile cap is:

$$P_{R1} = Mr_1/\Sigma r^2$$
$$= 262.2 \times 1.90/7.2$$
$$= 69.20 \text{ kips} \ldots \text{tension.}$$

The answer is (b).

h) The axial force in Pile 2 due to translation, is:

$$P_{T2} = 96.77 \text{ kips} \ldots \text{compression}$$

The axial force in Pile 2 due to rotation is:

$$P_{R2} = 69.20 \text{ kips} \ldots \text{compression}$$

The total axial force in Pile 2 is:

$$P_2 = P_{T2} + P_{R2}$$
$$= 96.77 + 69.20$$
$$= 165.97 \text{ kips} \ldots \text{compression.}$$

The answer is (e).

i) The axial force in Pile 3, due to translation, is:

$$P_{T3} = W - 3F$$
$$= 138.21 - 3 \times 61.20$$
$$= -45.39 \text{ kips} \ldots \text{tension.}$$

The axial force in Pile 3 due to rotation is:

$$P_{R3} = Mr_3/\Sigma r^2$$
$$= 0$$

The total axial force in Pile 3 is:

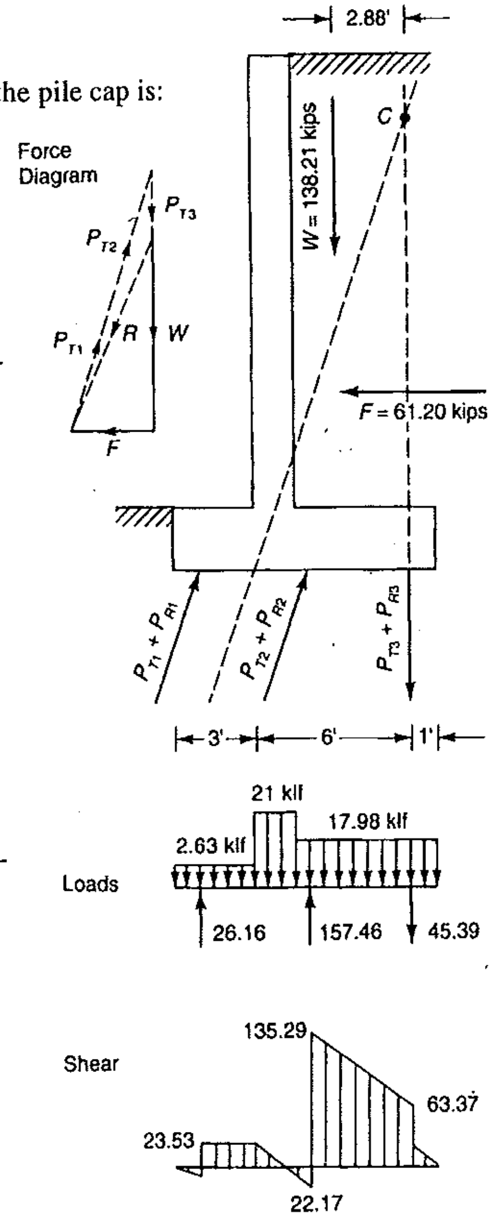

Fig. 4-4(a)

$P_3 = P_{T3} + P_{R3}$
$= -45.39$ kips . . . tension.

The answer is (a).

j) The loading diagram and corresponding shear force diagram are shown in Figure 4-4(a). The maximum shear force occurs at Pile 2 and has a value of 135.29 kips.
The answer is (d).

4-5. The tilt-up concrete wall panel shown in Fig. 4-5 is located six inches from the property line and supports an axial load of five kips per linear foot. The wall panel is supported on the eccentric footing indicated and the ground floor slab is tied to the wall panel with Grade 60 reinforcement to limit the soil bearing pressure to 3000 pounds per square foot.

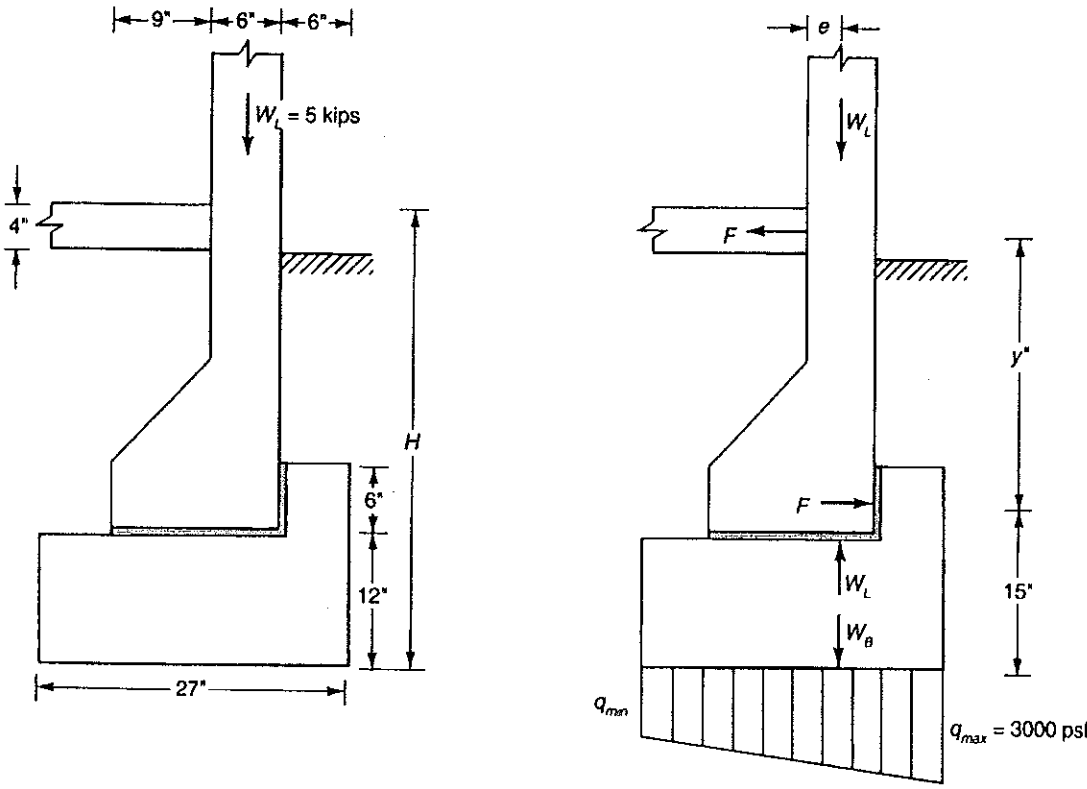

Fig. 4-5

a) Determine the required depth, H, of the footing below the level of the ground floor slab to produce a maximum soil bearing pressure of 3000 pounds per square foot.

b) Determine the area of the reinforcement required to tie the wall panel to the ground floor slab.

Solution

a) The forces acting on the footing are shown in Fig. 4-5 and assume a uniform distribution of pressure under the wall and between the side of the wall and the footing. The axial load in the wall panel is:

$W_L = 5$ kips per foot.

The weight of the base is:

$W_B = 0.15(2.25 \times 1 + 0.5 \times 0.5)$

$= 0.375$ kips per foot.

The total vertical load is:

$R = W_L + W_B$

$= 5.375$ kips per foot.

The eccentricity between the axial load in the wall panel and the center of pressure under the wall is:

$e = 15/2 - 6/2$

$= 4.5$ inches.

The clockwise couple produced by the axial load in the wall is

$M_W = eW_L$

This is balanced by the counterclockwise couple

$M_F = yF$

Equating the two couples gives:

$F = eW_L/y$

$= 4.5 \times 5/y$

$= 22.5/y$ kips.

The maximum pressure under the base is given by:

$q_{max} = R/A + Fa/S$

$= 5.375/2.25 + 22.5 \times 1.25/0.844y$

$= 3.0$ kips per square foot.

$y = 54.5$ inches.

The required depth of the footing below the level of the ground floor slab is:

$H = y + 15 + 4/2$

$= 71.5$ inches.

b) In accordance with ACI, Section A.3.2, the area of tensile reinforcement required in the floor slab is:

$A_S = F/f_s$

$= 22.5/(54.5 \times 24)$

$= 0.017$ square inches per foot.

4-6. An 18-inch-square column carrying a factored axial load of 900 kips is supported, as shown in Fig. 4-6, on a pile cap and a group of six 12-inch-square piles. The concrete strength is 3000 pounds per square inch and reinforcement consists of Grade 60 bars.

a) Determine the minimum allowable effective depth of the pile cap.

b) Determine the required number of Number 7 reinforcing bars required in the longitudinal direction.

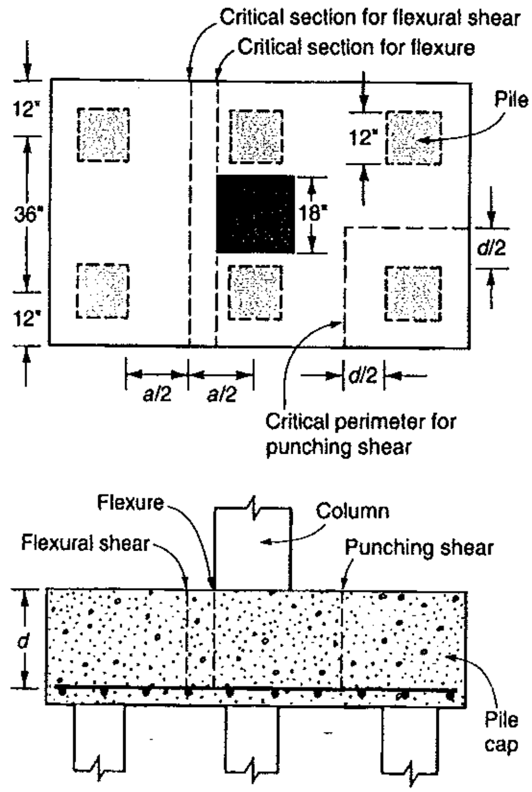

Fig. 4-6

Solution

a) Assuming that the pile cap may be classified as a deep flexural member, as specified in ACI Section 11.8.1, the critical section for flexural shear is given by ACI, Section 11.8.5 as a distance from the face of the outer piles of

$x = a/2$

$= (36 - 6)/2$

$= 15$ inches.

The factored shear at this location is:

$V_u = W_u/3$

$= 900/3$

$= 300$ kips.

The factored moment at the same location is:

$M_u = W_u(x + 6)/3$

$= 900(15 + 6)/3$

$= 6300$ kip inches.

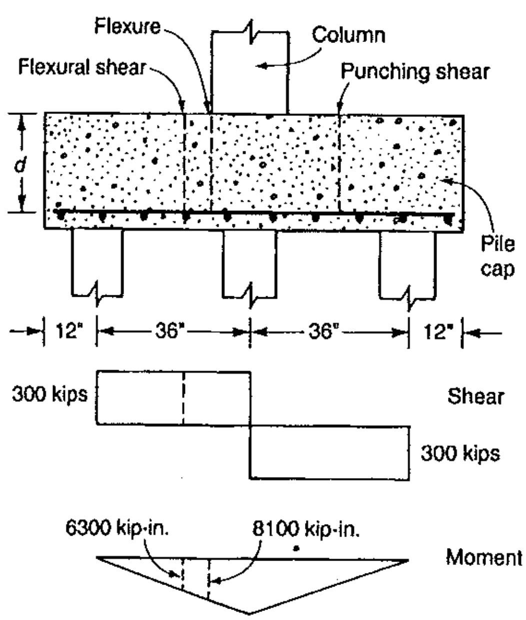

Fig. 4-6(a)

The reinforcement ratio may be assumed to be:

$\rho = A_S/bd$

$= 0.0020$

The flexural shear capacity of the section is obtained from ACI Equation (11-29) as:

$\phi V_c = \phi(3.5 - 2.5M_u/V_u d)(1.9\sqrt{f'_c} + 2500\rho V_u d/M_u)bd$

$= 0.85(3.5 - 2.5 \times 6300/300\text{d})(1.9\sqrt{3000} + 2500 \times 0.002 \times 300d/6300)60d$

$= 51d(3.5 - 52.5/d)(104.06 + 0.238d)$

$= V_u$

$= 300{,}000$ pounds.

Solving for the effective depth gives:

$d = 30$ inches.

And $V_u < 6\phi db\sqrt{f'_c}$ . . . satisfactory.

Punching shear for the column is not applicable since the central piles are located within the critical perimeter of the column. The critical perimeter for a corner pile is:

$b_o = 2(18 + d/2)$

$= 2(18 + 30/2)$

$= 66$ inches.

The capacity of the pile cap for punching shear at a pile is given by ACI Equation (11-37) as:

$\phi V_c = 4\phi b_o d\sqrt{f'_c}$

$= 4 \times 0.85 \times 66 \times 30\sqrt{3000}$

$= 368{,}730$ pounds

$> W_u/6$ . . . satisfactory.

The ratio of the clear distance between piles to the effective depth is:

$l_n/d = 24/30$

$= 0.8$

< 5

In accordance with ACI Section 11.8.1 the pile cap may be classified as a deep flexural member, and the minimum effective depth of 30 inches is satisfactory.

b) The critical section for flexure is located at the face of the column and the factored moment at this section is:

$M_u = W_u(36 - 9)/(3 \times 12)$

$= 675$ kip feet.

The required reinforcement ratio is derived from ACI, Section 10.3, with the aid of a calculator program,[3] and is:

$\rho = 0.85f'_c(1 - \sqrt{1 - 2K/0.765f'_c})/f_y$

$= 0.0029$

This is greater than the initial assumed value which is, therefore, satisfactory.

The maximum reinforcement ratio is given by ACI Section 10.3 as:

$$\rho_{max} = 0.75 \times 0.85 \times 87\beta_1 f'_c/f_y(87 + f_y)$$
$$= 0.0160$$
$$> \rho \ldots \text{satisfactory.}$$

The minimum reinforcement ratio is given by ACI Section 7.12 as:

$$\rho_{min} = 0.18 \text{ percent of the gross area}$$
$$< \rho \ldots \text{satisfactory}$$

Thus, the required reinforcement area is:

$$A_s = \rho bd$$
$$= 0.0029 \times 60 \times 30$$
$$= 5.22 \text{ square inches.}$$

Provide nine Number 7 bars to give an area of

$$A'_s = 5.40 \text{ square inches}$$
$$> A_s \ldots \text{satisfactory.}$$

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