Bridge Design - Shallow Foundations

Course Number: CE-02-411

PDH: 3

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Division of Engineering Services

BRIDGE DESIGN PRACTICE

4th Edition

2015

State of California
Department of Transportation

CHAPTER 15
SHALLOW FOUNDATIONS

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CHAPTER 15
SHALLOW FOUNDATIONS

15.1 INTRODUCTION

Shallow foundations (spread footings) are advantageous to pile foundations considering lower cost, easier construction, and fewer environmental constraints. However, weak soil and seismic considerations may limit use of spread footings and impact the foundation type selection.

In general, size of the spread footing is determined based on bearing resistance of the supporting soil or rock and also permissible level of settlement. Design of spread footings requires constant communication between the Structural Designer (SD) and the Geotechnical Designer (GD) throughout the design process. Factored loads are provided by the SD and factored resistance for the supporting soil and rock, that is permissible net contact stress q_{pn} and factored gross nominal bearing resistance q_R are calculated and reported by the GD. The structural design is performed by the SD. Consistency between the SD and the GD in the use of required gross or net stresses is important. Caltrans *Memo to Designers* (*MTD*) 4-1 (Caltrans, 2014b) provides general guidance on design process and also the minimum level of required communications between the SD and the GD. The analysis and design of a spread footing based on the 6th Edition of the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2012) and the *California Amendments* (Caltrans, 2014a), and *Seismic Design Criteria* (*SDC*) Version 1.7 (Caltrans, 2013), will be illustrated through an example.

15.2 COMMON TYPES OF SPREAD FOOTINGS FOR BRIDGES

Spread footings can be used as isolated footings to support single columns or as combined footings to support multi-columns when columns are closely spaced. Elongated spread footings under abutments and pier walls act as strip footings where moments act only in the short direction. Strip footings under abutments or piers can be analyzed and designed similar to column footings, with moments acting in one direction only.

15.3 PROPORTIONING AND EMBEDMENT OF FOOTINGS

The designer should consider several parameters such as axial force and biaxial moment acting on the footing, right of way, existing structures, and also depth of footing when selecting size and location of the footing. Although square footings are more common for footings supporting pinned columns, rectangular shapes may be more efficient when column is fixed at the base, since moments acting on the footing in two directions may be very different. Considering various load combinations specified in *AASHTO* (2012) and *Caltrans* (2014a), and variation of geotechnical resistances with eccentricities of loads acting on the footing any type of optimization can be rigorous.

15.3.1 Sizing of Spread Footings

The trial minimum size of the spread footing can be selected based on footings of similar conditions and past experience. Size of a spread footing is usually governed by the column size, magnitude of loads acting on the footing, and resistances of the substrate. The effective length to effective width (L'/B') ratio is commonly 1.0 ~ 2.0. The GD should be consulted for selection of the ratios. The allowable settlement will be assumed as 1 in. or 2 in. according to *MTD* 4-1 (Caltrans, 2014b) and based on continuity of the superstructure. Larger limits can be used if structural analysis shows that the superstructure can tolerate such settlement without adverse serviceability impacts (Caltrans, 2014a).

The footing size is usually proportioned based on "***permissible net contact stress***" at the service limit state and checked for "***factored gross nominal bearing resistance***" at strength and extreme event limit states.

These stresses are functions of the effective width as well as the effective length to effective width ratio, therefore they are presented by a family of curves and also a table as shown in the design example. The SD needs to use double interpolation to extract the information required for design under different load combinations using corresponding effective dimensions. If necessary, the GD may be contacted to revise the information and provide a new set of curves and tables to avoid extrapolation.

15.3.2 Embedment and Depth of Footings

The footing embedment shall be carefully determined for degradation and contraction scour for the base (100 year) flood, as well as short term scour depth. The embedment depth of the footing should be adequate to ensure the top of the footing is not exposed when total scour has occurred, as shown in Figure 15.3-1. If the footing is not in water and freezing is not of concern, a minimum cover of 2 to 3 ft is recommended.

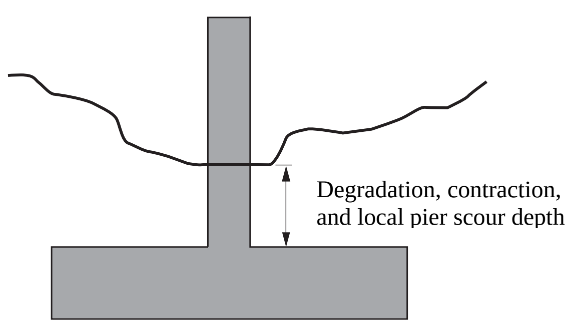


Figure 15.3-1 – Minimum Embedment for Scour Protection

The depth (thickness) of the footing is preliminary selected based on the required development length of the column reinforcement and then designed for flexural and shear strength.

15.4 DESIGN LOADS

The factored shear forces (V_x and V_y), column axial force (P) and bending moments (M_x and M_y) resulting from structural analysis are usually reported at the base of the column and must be transferred to the bottom of the footing in order to calculate contact bearing stresses. Therefore, the resultant moment at the base of the columns must be modified to include the additional moment caused by shear force transfer. The modified moment in a generic format can be written as $M + (V \times d_{footing})$, where $d_{footing}$ is the actual footing depth.

15.5 BEARING STRESS DISTRIBUTION

The sign convention shown in *MTD* 4-1 (Caltrans, 2014b) is to avoid mistakes in communications between the SD and the GD. The footing local *X* axis is defined along the longer dimension of the footing (L), and the *Y* axis along the short dimensions (B) as shown in Figure 15.5-1. Forces and moments resulting from superstructure analysis acting at the column base are resolved in the directions of local axes if local axes do not coincide longitudinal and transverse directions of the bridge.

Bearing stress distribution depends on relative stiffness of the footing and supporting soil and rock. For determination of the footing size based on the bearing resistance and settlement requirements, the bearing stress is assumed to be uniformly distributed for footings on soil and linearly distributed for footings on rock. For structural design of the footing, bearing stress is assumed to be linearly distributed.

For eccentrically loaded footings on soil, the effective footing dimensions (B' and L') specified in *AASHTO* Article 10.6.1.3 (AASHTO, 2012) shall be used for design of settlement and bearing resistance. Bearing stress distribution over effective footing area is assumed to be uniform. The effective dimensions for a rectangular footing are shown in Figure 15.5.1 and shall be taken as follows:

$$B' = B - 2e_y$$
$$L' = L - 2e_x \qquad \text{(AASHTO 10.6.1.3-1)}$$

where:

$B,\ L$ = actual dimensions of the footing (ft)
e_y, e_x = eccentricities parallel to dimensions B and L, respectively (ft)
A' = reduced effective area of the footing $= B' \times L'$ (ft^2)
q = uniform bearing stress = P/A' (ksf)

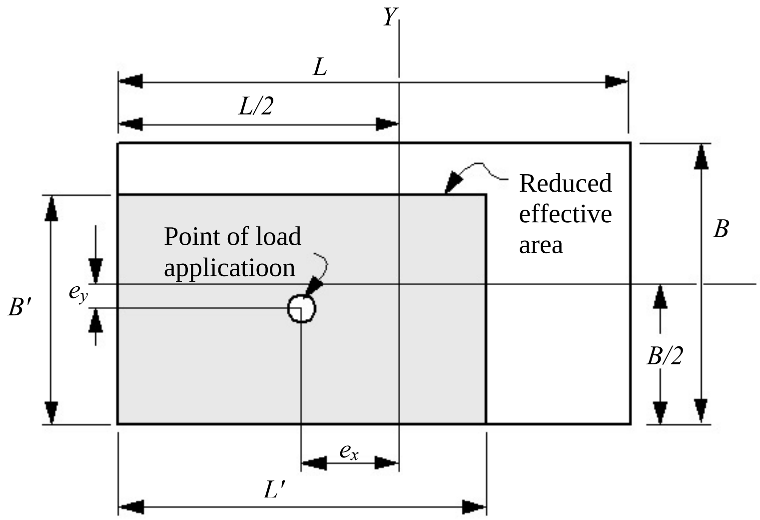


Figure 15.5-1 Effective Footing Area

For footings on rock and for structural design of footings, the bearing stress is assumed to be linearly distributed. If the eccentricity is less than $B/6$ (or $L/6$) the maximum bearing stress is calculated as:

$$q_{\max} = \frac{P}{A} \pm \left|\frac{M_y}{S_y}\right| + \left|\frac{M_x}{S_x}\right| \tag{15.5-1}$$

where:

P = vertical force acting at the center of gravity of the bottom of the footing area (kip)

M_x, M_y = moments acting at the bottom of the footing about X and Y directions, respectively (kip-ft)

S_x, S_y = section modulus of the footing area about X and Y directions, respectively (ft^3)

A = actual footing area = $B \times L$ (ft^2)

Equation (15.5-1) is valid only if stresses calculated at corners of the footing are all positive (compression), otherwise the reduced contact area of footing must be determined and rocking must be considered in analysis.

Bearing stresses can be calculated as “net” or ‘gross”. The weight of the footing and all overburden soil from top of the footing to finished grade must be included when calculating “gross bearing stress”. The weight of overburden soil between bottom of footing and original grade at excavation time is subtracted from gross bearing stress to calculate “net bearing stress.” Net bearing stress under *AASHTO* Service I Load Combination is used to evaluate footing settlement.

15.6 GENERAL DESIGN REQUIREMENTS

The bearing stresses calculated under various *AASHTO* LRFD limit states must be checked against acceptable stresses provided by the GD. After receiving foundation information and scour data (if applicable), the GD will provide "*permissible net contact stress*" used for Service Limit States checks, and "*factored gross nominal bearing resistance*" used for Strength and Extreme Event Limit States checks, respectively. The stresses are functions of the effective width as well as effective length to effective width ratio, therefore information will be provided as a family of data points for different values of *L/B* ratios for a given "*B*". The SD needs to use double interpolation to extract the information required for design under different load combinations using corresponding effective dimensions. If necessary, the GD may be contacted to revise the information and provide a new set of curves and table to avoid extrapolation.

15.6.1 Settlement Check

For Service Limit State, the following requirement must be met:

$$q_{n,u} \leq q_{pn} \quad \text{for footing on soil} \tag{15.6-1}$$

$$q_{n,amx} \leq q_{pn} \quad \text{for footing on rock} \tag{15.6-2}$$

where

q_{pn} = permissible net contact stress provided by the GD and calculated based on a specified allowable settlement (ksf)

$q_{n,u}$ = net uniform bearing stress calculated using Service-I Limit State loads assuming uniform stress distribution for footings on soil (ksf)

$q_{n,max}$ = net maximum bearing stress calculated using Service-I Limit State loads assuming linear stress distribution for footings on rock (ksf)

15.6.2 Bearing Check

For Strength and Extreme Event Limit States, the design requirement is written as:

$$q_{g,u} \leq q_R \quad \text{for footing on soil} \tag{15.6-3}$$

$$q_{g,\max} \leq q_R \quad \text{for footing on rock} \tag{15.6-4}$$

where:

$q_{g,u}$ = gross bearing stress calculated based on uniform stress distribution for footings on soil (ksf)

$q_{g,max}$ = gross maximum bearing stress calculated based on linear stress distribution for footings on rock (ksf)

q_R = factored gross nominal bearing resistance provided by the GD = $\varphi_b q_n$ (ksf)

q_n = gross nominal bearing resistance (ksf)

φ_b = resistance factor

15.6.3 Eccentricity Limits

The eccentricity limits for Service and Extreme Event Limit States specified in *AASHTO* (2012) and *Caltrans* (2014a) are summarized as:

Table 15.6-1 AASHTO (2012) Eccentricity Limits

Limit State	Footing on Soil	Footing on Rock	AASHTO Article Number
Service	$B/6$ or $L/6$	$B/4$ or $L/4$	10.5.2.2
Extreme Event (Seismic) $\gamma_{EQ}=0$	$B/3$ or $L/3$	$B/3$ or $L/3$	10.6.4.2 and 11.6.5.1
Extreme Event (Seismic) $\gamma_{EQ}=1.0$	$2B/5$ or $2L/5$	$2B/5$ or $2L/5$	10.6.4.2 and 11.6.5.1

Note: Seismic forces should be applied in all directions per *SDC* (Caltrans 2013). It is not necessary to include live load (design or permit truck) in Extreme Event Limit State load combinations therefore $\gamma_{EQ} = 0$.

15.6.4 Sliding Check

Shear force acting at the interface of footing and substrate should be calculated and compared to the factored nominal sliding resistance specified as:

$$R_R = \varphi R_n = \varphi_\tau R_\tau + \varphi_{ep} R_{ep} \qquad \text{(AASHTO 10.6.3.4-1)}$$

The contribution of soil passive pressure (second term) is generally negligible and equation is summarized to $R_R = \varphi R_n = \varphi_\tau R_\tau$. For cohesionless soil R_τ is written as:

$$R_\tau = V \tan(\delta) \qquad \text{(AASHTO 10.6.3.4-2)}$$

where:

R_τ = nominal sliding resistance between soil and concrete (kip)
V = total force acting perpendicular to the interface (kip)
δ = friction angle at interface of footing and soil = ϕ_f, internal friction angle of the drained soil for concrete cast against soil (degree)
φ_τ = resistance factor against sliding = 0.8 for cast-in-place concrete against sand (AASHTO Table 10.5.5.2.2-1).

15.7 STRUCTURAL DESIGN OF FOOTINGS

Structural design of the footing includes the following steps:

- Select footing thickness based on required development length of the column reinforcement
- Design flexural reinforcement in both directions considering minimum reinforcement required for shrinkage and temperature
- Check thickness of the footing for one-way and two-way shears and design shear reinforcement if required
- Check seismic details per Caltrans *SDC* (Caltrans, 2013a) and other practice manuals

Table 15.7-1 provides highlights of requirements for structural design of the footings specified in *AASHTO* (2012) and *Caltrans* (2014a). Application of these requirements will be illustrated in the design example.

Table 15.7-1 – *AASHTO* (2012) and *Caltrans* (2014a) Requirements for Structural Design of Footings

Topic	AASHTO Articles	Application
Strut & tie Applicability	5.6.3	Requirement check
Flexural design	5.7.3.2	Reinforcement design
Direct shear design	5.8.3.3	Footing depth and reinforcement design
Shear friction	5.8.4	Shear key design
Reinforcement spacing	5.7.3.3, 5.7.3.4 5.10.3, 5.10.8	Design and detailing
Reinforcement development	5.11.2	Structural design of footings
Concrete cover	5.12.3	Footing depth and detailing
Footings	5.13.3	Footing depth

15.8 DESIGN EXAMPLE

15.8.1 Bridge Footing Data

Design process for a bridge bent spread footing is illustrated through the following example. A circular column of 6 ft diameter with 26#14 main rebars, and #8 hoops spaced at 5 in. is used for a two-span post-tensioned box girder bridge. Footing as shown in Figure 15.8-1 rests in cohesionless soil with internal friction angle of 38°. Original ground (OG) elevation is 48 ft, finished grade (FG) elevation is 48 ft, and bottom footing elevation (BOF) is 39 ft.

- Concrete material f'_c = 3,600 psi
- Reinforcement f_y = 60,000 psi (A706 steel).
- Governing unfactored live load forces at the base of the column are listed in Table 15.8-1.
- Unfactored dead load and seismic forces at the base of the column are listed in Table 15.8-2.
- Plastic moment and shear applied at the column base are:
  M_p = 15,573 kip-ft; V_p = 716 kips
  Overturning column axial force in transverse push is 992 kips.

Note: To facilitate communications of the SD and the GD, local coordinate of foundation have been defined as X and Y. As shown in Figure 15.8-2a. Local X axis is parallel to long dimension plan of footing (L) and the local Y axis is perpendicular to X. The global coordinates L (Longitudinal) and T (Transverse) are commonly used for bridge analysis. The structural designer needs to transfer forces and moments acting on the footing to shear forces and moments acting in local coordinates. All communications between the SD and the GD shall be based on forces/moments calculated in local coordinates of the footing. In this example local and global coordinates coincide, that is $X = T$ and $Y = L$. Therefore, local and global coordinates may have been used interchangeable, as shown in Figure 15.8-2b.

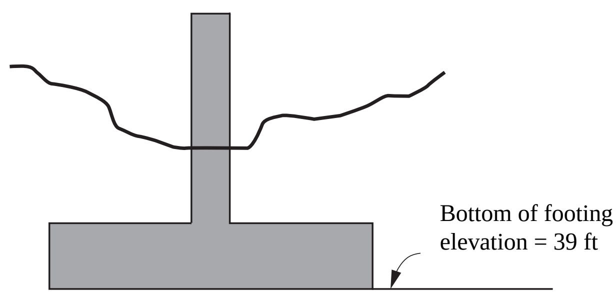


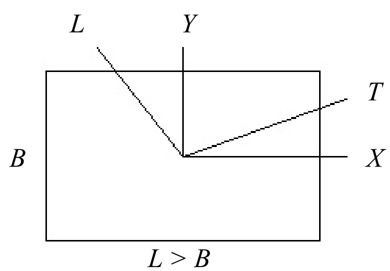
Figure 15.8-1 Elevation of the Spread Footing

Table 15.8-1 – Unfactored Live Load Forces at Column Base

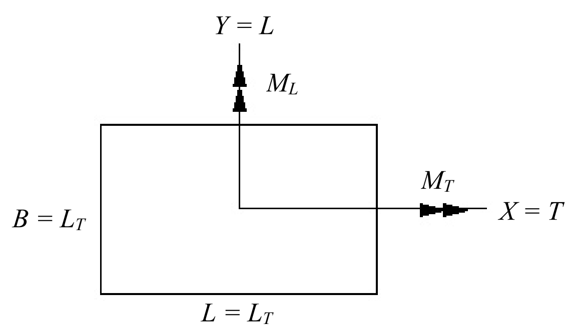
Load Case	HL-93 Truck			Permit Truck		
	I	II	**III**	I	II	**III**
M_T (kip-ft)	-206	-40	**-80**	-348	19	**34**
M_L (kip-ft)	250	1,456	**552**	171	2,562	**354**
P (kip)	217	238	**479**	367	439	**760**
V_T (kip)	-12	-1	**-2**	-16	4	**7**
V_L (kip)	12	81	**26**	8	144	**17**

Table 15.8-2 Unfactored Dead Load and Seismic Forces Applied at Column Base

Load Case	*DC*	*DW*	*PS*	Seismic-I (M_p applied)	Seismic-II (M_p applied)
M_T (kip-ft)	62	9	0	15,574	0
M_L (kip-ft)	833	139	-14	0	15,574
P (kip)	1,503	227	-21	992	0
V_T (kip)	4	1	0	716	0
V_L (kip)	44	7	-16	0	716



(a) General Case



(b) Example Problem

Figure 15.8-2 Local Footing Coordinates vs. Global Structure Coordinates

Upon calculation of effective dimensions under any load combination, the larger effective dimension is designated as "*L*" and smaller as "*B*" to calculate q_{pn} and q_R from information provided by the GD.

15.8.2 Design Requirements

Perform the following design portion for the footing in accordance with the *AASHTO LRFD Bridge Design Specifications,* 6th Edition (AASHTO, 2012) with the *California Amendments* (Caltrans, 2014a), and design peak ground acceleration (PGA) = 0.6g .

- Determine the minimum footing thickness required to develop the column reinforcement. (Assume #9 bars for footing bottom reinforcement)
- Calculate LRFD factored forces for Service, Strength, and Extreme Event limit states applicable to footing design
- Determine the minimum size of the square footing adequate for applicable LRFD limit states
- Calculate required rebar spacing if #5 and #9 bars are used for top and bottom mats, respectively
- Check footing thickness for one-way and two-way shears

15.8.3 Footing Thickness Determination

Minimum footing thickness is equal to the minimum clearance from the bottom of footing to the bottom mat of footing reinforcement, plus the deformed diameters of the bars used for the bottom of footing reinforcement, plus the required development length of the main column reinforcement.

$$d_{min.} = clr. + 2(d_b) + l'_d \qquad (15.8\text{-}1)$$

where:

$d_{min.}$ = minimum footing thickness (ft)
$clr.$ = minimum clearance from the bottom of footing to the bottom mat of footing reinforcement (in.)
d_b = nominal diameter of bar used for the bottom of footing reinforcement (in.)
l'_d = required development length of the main column reinforcement (in.)

From *AASHTO* Table 5.12.3-1, $clr. = 3$ in., and for #9 bars, $d_b = 1.25$ in. The development length is calculated in accordance with *AASHTO* Articles 5.11.2.2, and 5.11.2.4.

Development of Deformed Bars in Compression:

$$l_{db} \geq 0.63\,(1.693)(60) / (3.6)^{0.5} = 33.7 \text{ in.} \qquad \text{(AASHTO 5.11.2.2.1-1)}$$

$$l_{db} \geq 0.3(1.693)(60) = 30.5 \text{ in.} \qquad \text{(AASHTO 5.11.2.2.1-2)}$$

AASHTO Article 5.11.2.2.2 states that the basic development length may be multiplied by applicable modification factors, and requires that reinforcement is enclosed within a spiral of not less than 0.25 in. in diameter and spaced at not more than a 4 in. pitch, in order to use modification factor of 0.75. This reduction does not apply because we have the main column hoops spaced at 5 inches.

Hooks shall not be considered effective in developing bars in compression, therefore, development length required for compression is equal to 33.7 inches.

Development of Standard Hooks in Tension

$l_{hb} = 38.0\ (1.693) / (3.6)^{0.5} = 33.9$ in. (AASHTO 5.11.2.4.1-1)

Basic development length shall be multiplied by applicable modification factors (AASHTO 5.11.2.4.2).

Concrete Cover – For #11 bar and smaller, side cover (normal to plane of hook) not less than 2.5 in., and for a 90 degree hook cover on bar extension beyond the hook not less than 2 in., then modification factor = 0.70.

Note - For determining modification factors, the specifications refer to the portion of the bar from the critical section to the bend as the "hook", and the portion of the bar from the bend to the end of the bar as the "extension beyond the hook".

Ties or Stirrups – Hooks for #11 bar and smaller, enclosed vertically or horizontally within ties or stirrup-ties spaced along the full development, l_{dh}, at a spacing not greater than $3d_b$, where d_b is diameter of hooked bar, then modification factor = 0.80.

None of the modification factors are applied, since #14 bars have been used for columns, therefore, development length of standard hooks in tension = 33.9 in. say 34 in. (Also greater than 8×1.693 in. and 6 in.).

Development length for tension (34 in.), controls over the development length for compression (33.7 in.). The required footing thickness is calculated as:

$$d_{min.} = clr. + 2(d_b) + l'_d = 3 + 2(1.25) + 34 = 39.5 \text{ in.} = 3.29 \text{ ft}$$

Try footing thickness $d_{footing}$ = 4.0 ft

15.8.4 Calculation of Factored Loads

Considering live load movements in the longitudinal and transverse directions, the following three cases of live load forces have been considered in this example:

Case I) Maximum Transverse Moment (M_T) and associated effects

Case II) Maximum Longitudinal Moment (M_L) and associated effects

Case III) Maximum Axial Force (P) and associated effects

Moments and shears at the column base must be transferred to the bottom of the footing for the footing design. The following unfactored forces are obtained to include the additional moment ($V \times d_{footing}$) caused by shear force transfer.

For example, HL-93 Truck – Load Case I,
Forces applied at the column base are:

M_T = - 206 kip-ft

V_T = -12 kip

For the footing thickness $d_{footing}$ = 4 ft, forces applied at the bottom of footing are obtained as follows:

$$M_T = -206 + (-12)(4) = -254 \text{ kip-ft}$$

$$V_T = -12 \text{ kip}$$

The unfactored live load forces (without impact) at the bottom of the footing are calculated in Table 15.8-3.

Table 15.8-3 – Unfactored Live Load Forces at Bottom of Footing

Load	HL-93 Truck			Permit Truck		
Case	I	II	**III**	I	II	**III**
M_T (kip-ft)	- 254	- 44	**- 88**	- 412	35	**62**
M_L (kip-ft)	298	1,780	**656**	203	3,138	**422**
P (kip)	217	238	**479**	367	439	**760**
V_T (kip)	-12	- 1	**- 2**	- 16	4	**7**
V_L (kip)	12	81	**26**	8	144	**17**

The design for live loads for Case-III (both HL-93 and Permit Trucks) is only illustrated in this example, however all three cases need to be considered in practice. Forces and moments resulting from seismic analysis in transverse and longitudinal directions are also shown as Seismic-I and Seismic-II, respectively.

As PGA > 0.5g shallow foundation will be designed for column plastic hinging, (rocking is not allowed). For the footing thickness $d_{footing}$ = 4 ft, overstrength moment and shear applied at the bottom of the footing are calculated as:

$$M_o = 1.2 [15{,}574 + (716)(4)] = 22{,}126 \text{ kip-ft}$$

$$V_{To} = 1.2(716) = 859 \text{ kip}$$

The unfactored dead load forces and seismic forces at the bottom of the footing are shown in Table 15.8-4.

Table 15.8-4 Unfactored Forces Applied at Bottom of Footing

Load Case	*DC*	*DW*	*PS*	Seismic-I (M_o applied)	Seismic-II (M_o applied)
M_T (kip-ft)	78	13	0	22,126	0
M_L (kip-ft)	1,009	167	-78	0	21,126
P (kip)	1,503	227	-21	992	0
V_T (kip)	4	1	0	859	0
V_L (kip)	44	7	-16	0	859

The LRFD load combinations (AASHTO, 2012) used in foundation design and corresponding load factors (AASHTO Tables 3.4.1-1 and 3.4.1-2) are summarized in

Table 15.8-5. The upper and lower limits of permanent load factors (γ_p) are shown as "*U*" and "*L*", respectively.

Table 15.8-5 Load Factors for Footing Design

Load	*DC*	*DW*	*PS*	*EV*	*HL-93*	*P-15*	*EQ*
Strength I-U	1.25	1.5	1.00	1.35	1.75	-	-
Strength I-L	0.90	0.65	1.00	0.90	1.75	-	-
Strength II-U	1.25	1.50	1.00	1.35	-	1.35	-
Strength II-L	0.90	0.65	1.00	0.90	-	1.35	-
Strength III-U	1.25	1.50	1.00	1.35	-	-	-
Strength III-L	0.90	0.65	1.00	0.90	-	-	-
Strength V-U	1.25	1.5	1.00	1.35	1.35	-	-
Strength V-L	0.9	0.65	1.00	0.90	1.35	-	-
Service I	1.00	1.00	1.00	1.00	1.00	-	-
Extreme Event I	1.00	1.00	1.00	1.00	-	-	1.00

The LRFD load factors are applied to axial force, shear forces, and moments in longitudinal and transverse directions to calculate factored loads for Strength, Service and Extreme Event limit states at the base of the column, as summarized in Table 15.8-6. Only the governing seismic case that is Seismic-I is used in Extreme Event-I load combination.

Table 15.8-6 Factored Forces at Column Base for Footing Design

Factored Loads	M_T (kip-ft)	M_L (kip-ft)	P (kip)	V_T (kip)	V_L (kip)	V_{Total} (kip)
Strength I-U	-37	2,582	3,037	3	95	95
Strength I-L	-75	2,087	2,318	1	74	74
Strength II-U	201	2,003	**3,224**	16	72	74
Strength II-L	162	1,508	2,505	14	51	53
Strength III-U	117	1,434	2,198	7	50	50
Strength III-L	79	939	1,479	4	28	28
Strength V-U	-2	2,319	2,845	4	85	85
Strength V-L	-40	1,824	2,126	2	63	63
Service I	3	1,754	2,188	3	61	61
Extreme Event I	22,126	0	2,701	864	35	865

Example: Calculation of gross axial force at bottom of footing for Strength-II-U limit state:

$$P_g = 1.25(1503) + 1.5(227) + 1(-21) + 1.35(760) = 3{,}224 \text{ kips}$$

15.8.5 Footing Size Determination

In order to design a spread footing all live load combinations (Cases I, II and III) should be considered for both design and permit trucks. It is recommended to consider maximum axial case (Case III) for initial sizing of the footing and check footing size and stresses for the other two cases (I and II), however this example only considers Case-III.

Based on preliminary analysis of the footing, reasonable estimates for "width of the footing" as well as "length to width ratios" are provided to the GD to be used in design (Appendix A). Refer to *MTD* 4-1 (Caltrans, 2014b) for other information to be submitted to the GD.

The GD will provide graphs and also a table of "permissible net contact stress" (used for Service-I limit state check), and "factored gross nominal bearing resistance" (used for strength and extreme event limit states) for numerous "*B'* " "and "*L'/B'* " ratios, as shown in Figures 15.8-3 to 15.8-5, and Table 15.8-7 for given ranges of footing widths and also effective length to effective width ratios.

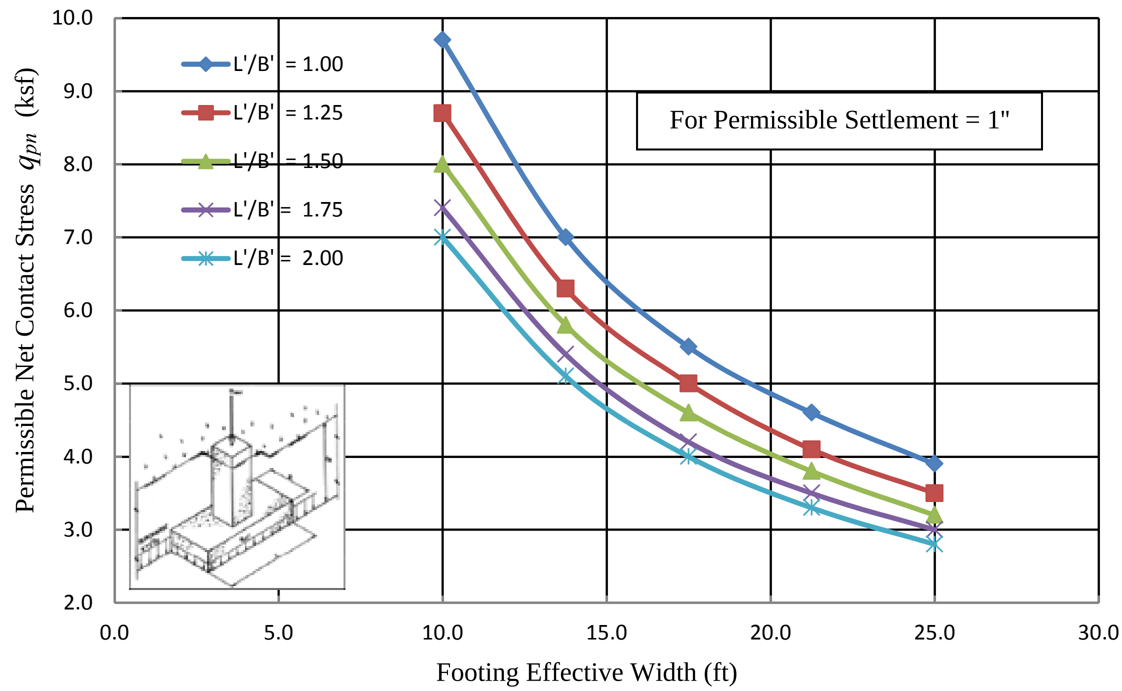


Figure 15.8-3 Variations of Permissible Net Contact Stress

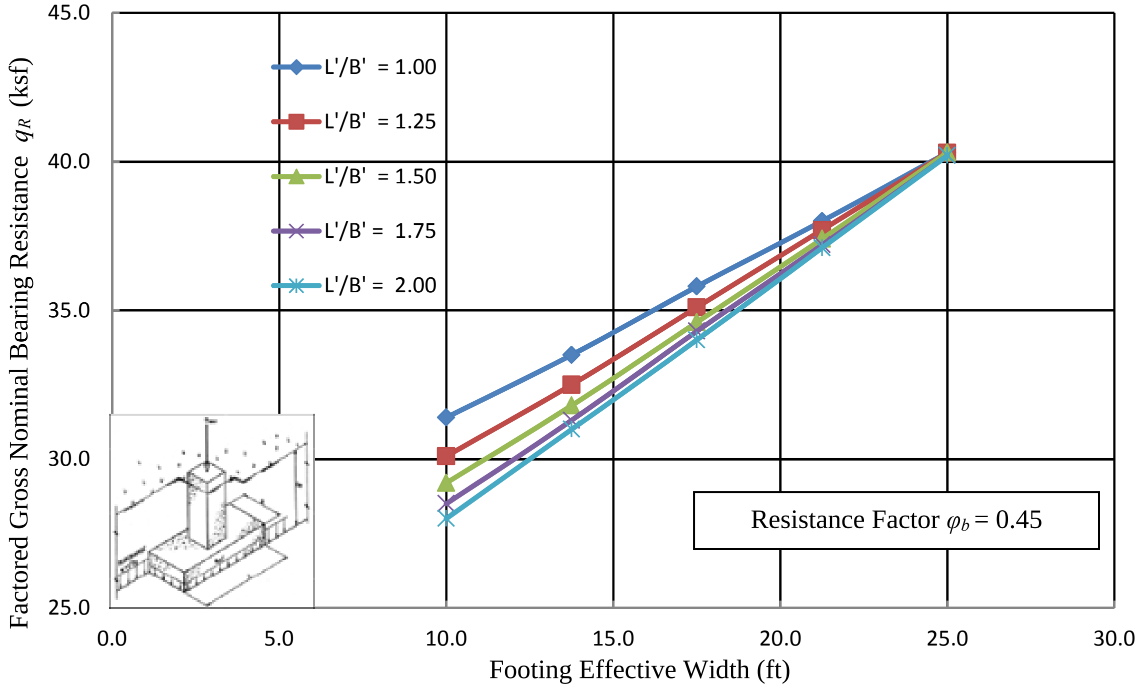


Figure 15.8-4 Variations of Factored Gross Nominal Bearing Resistance (Strength Limit State)

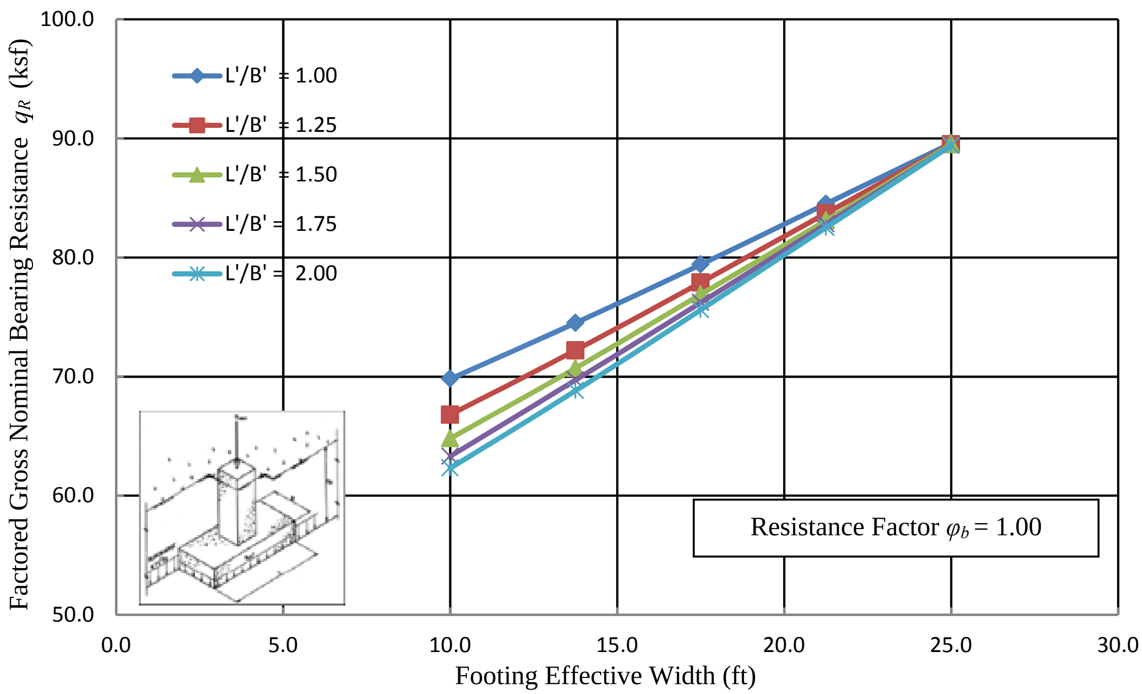


Figure 15.8-5 Variations of Factored Gross Nominal Bearing Resistance (Extreme Event Limit State)

Table 15.8-7 Variations of Bearing Resistance for Different Limit States

No.	Effective Footing Size		Effective Footing Size Ratio	Factored Gross Nominal Bearing Resistance (Extreme Event Limit)	Factored Gross Nominal Bearing Resistance (Strength Limit)	Permissible Net Contact Stress (Service Limit)
	B' (ft)	L' (ft)	L'/B'	q_R (ksf)	q_R (ksf)	q_{pn} (ksf)
1	10.00	10.00	1.00	69.8	31.4	9.7
2	13.75	13.75	1.00	74.5	33.5	7.0
3	17.50	17.50	1.00	79.4	35.8	5.5
4	21.25	21.25	1.00	84.5	38.0	4.6
5	25.00	25.00	1.00	89.6	40.3	3.9
1	10.00	12.50	1.25	66.8	30.1	8.7
2	13.75	17.19	1.25	72.2	32.5	6.3
3	17.50	21.88	1.25	77.9	35.1	5.0
4	21.25	26.56	1.25	83.7	37.7	4.1
5	25.00	31.25	1.25	89.5	40.3	3.5
1	10.00	15.00	1.50	64.8	29.2	8.0
2	13.75	20.63	1.50	70.7	31.8	5.8
3	17.50	26.25	1.50	76.9	34.6	4.6
4	21.25	31.88	1.50	83.1	37.4	3.8
5	25.00	37.50	1.50	89.5	40.3	3.2
1	10.00	17.50	1.75	63.3	28.5	7.4
2	13.75	24.06	1.75	69.7	31.3	5.4
3	17.50	30.63	1.75	76.2	34.3	4.2
4	21.25	37.19	1.75	82.8	37.2	3.5
5	25.00	43.75	1.75	89.4	40.2	3.0
1	10.00	20.00	2.00	62.3	28.0	7.0
2	13.75	27.50	2.00	68.8	31.0	5.1
3	17.50	35.00	2.00	75.6	34.0	4.0
4	21.25	42.50	2.00	82.5	37.1	3.3
5	25.00	50.00	2.00	89.4	40.2	2.8

As the first trial, a square footing of 20×20 ft is selected and contact stresses under service, strength, and extreme event factored loads are calculated as summarized in the following tables. Stresses are compared to “permissible net contact stress” (Service-I), and “factored gross nominal bearing resistance” (Strength and Extreme Event), as explained in *MTD* 4-1. Since the footing rests on soil, contact stress distribution is assumed uniform over the effective area of the footing.

The bearing stresses should be calculated as net for Service-I limit state and gross for all strength and extreme event limit states as shown in Figure 15.8-6, therefore, weight of overburden soil and footing with corresponding load factors have been considered in the axial forces shown in Table 15.8-8.

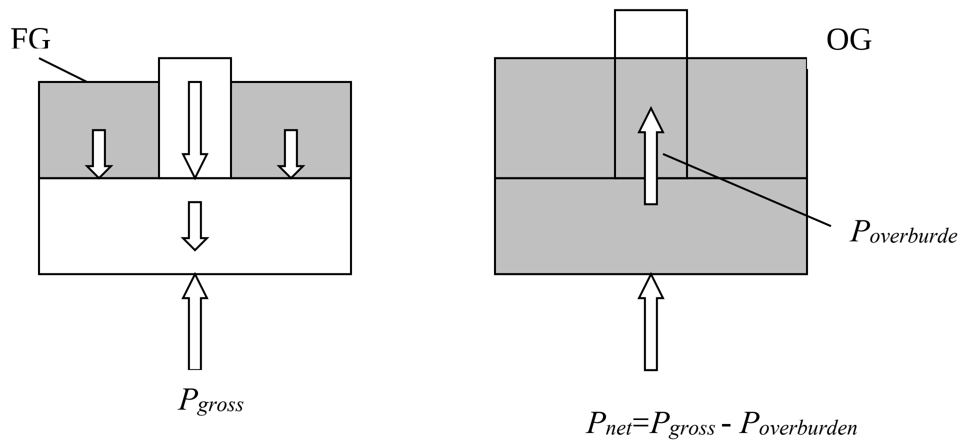


Figure 15.8-6 Definition of Gross and Net Bearing Stresses

For example:

Strength I-U

$P_{gross} = P_{gross\ at\ column\ base}$ + factored weight at soil on footing + factored weight of footing

P_{gross} = 3,037 + (20× 20-28.26) (48-39-4) (120/1,000) (1.35) + (20× 20× 4) (150/,000) (1.25) = 3,638 kips

Service-I

$P_{net} = P_{net\ at\ column\ base}$ + weight of soil on footing + weight of footing – excavated soil (over burden)

P_{net} = 2,188 + (20×20-28.26) (48-39-4) (120/1,000) + (20×20×4)(150/1,000)-(48-39) (20x20) (120/1,000) = 2,220 kips

Detailed calculations for Strength I-U limit state can be summarized as:

M_T = -37 kip-ft; P = 3,638 kips

e_T = 37/3,638 = 0.01 ft; L'_T = 20-2(0.01) = 19.98 ft

M_L= 2,582 kip-ft, e_L =2,582/3,638 = 0.71ft

L'_L =20-2(0.71) = 18.58 ft

A_e= 19.98(18.58) = 371 ft^2 ; $q_{g,u}$ = 3,638/371 = 9.80 ksf

L'/B' =19.98/18.58 =1.08, therefore q_R = 36.23 (From Figure 15.8-3)

Since q_R is greater than $q_{g,u}$, bearing resistance is adequate.

Similar calculation is required for every load combination as shown in Table 15.8-8

Table 15.8-8 Detailed Check for Footing Size (First Trial)

Load Combination	M_T (kip-ft)	M_L (kip-ft)	P (kip)	e_T (ft)	e_L (ft)	L'_T (ft)	L'_L (ft)
Strength I-U	-37	2,582	**3,638**	0.01	0.71	19.98	18.58
Strength I-L	-75	2,087	2,735	0.03	0.76	19.94	18.47
Strength II-U	201	2,003	3,826	0.05	0.52	19.90	18.95
Strength II-L	162	1,508	2,923	0.06	0.52	19.89	18.97
Strength III-U	117	1,434	2,800	0.04	0.51	19.92	18.97
Strength III-L	79	939	1,896	0.04	0.50	19.92	19.01
Strength V-U	-2	2,319	3,447	0.00	0.67	20.19	18.65
Strength V-L	-40	1,824	2,543	0.02	0.72	19.97	18.56
Service I	-3	1,754	**2,220**	0.00	0.79	20.00	18.42
Extreme Event I	22126	0	3,164	6.99	0.00	6.01	20.00

Table 15.8-8 Detailed Check for Footing Size (First Trial) (Continued)

Load Combination	A_e (ft^2)	L'/B'	q_o (ksf)	q_{pn} *or* q_R (ksf)	q_o/q_{pn} *or* q_o/q_R *Ratio*	Check
Strength I-U	371.2	1.08	9.80	**36.23**	**0.27**	OK
Strength I-L	368.5	1.08	7.42	**36.15**	**0.21**	OK
Strength II-U	377.1	1.05	10.15	**36.52**	**0.28**	OK
Strength II-L	377.2	1.05	7.75	**36.53**	**0.21**	OK
Strength III-U	377.9	1.05	7.41	**36.53**	**0.20**	OK
Strength III-L	378.6	1.05	5.01	**36.56**	**0.14**	OK
Strength V-U	373.1	1.07	9.24	**36.28**	**0.25**	OK
Strength V-L	370.7	1.08	6.86	**36.22**	**0.15**	OK
Service I	368.3	1.09	6.02	**5.11**	**1.18**	NG
Extreme Event I	119.1	3.32	26.30	**40.43***	**0.65**	OK

* L'/B' is out of range, therefore, factored nominal bearing resistance was calculated by extrapolation. For design purposes, the SD needs to ask the GD to provide adequate data to cover all applicable cases, without any need to extrapolation.

In Table 15.8-8:

L'_L, L'_T = effective dimensions of the footing in the directions of L and T, respectively (ft). $L'_T = L_T - 2e_T$ and $L'_L = L_L - 2e_L$

e_L, e_T = eccentricities calculated from M_L and M_T, respectively (ft)

A_e = effective area of the footing (ft^2)

B' = shorter effective dimension (ft)

L' = longer effective dimension (ft)

q_0 = uniform bearing stress calculated as net for service ($q_{n,u}$) and gross for Strength and Extreme Event limits ($q_{g,u}$) (ksf)

q_{pn} = permissible net contact stress (ksf)

q_R = factored gross nominal bearing resistance (ksf)

The permissible eccentricity under Service-I Load is calculated as:

$B/6 = L/6 = 20/6 = 3.33$ ft. Therefore the eccentricity calculated under Service-I loads (0.79 ft) is acceptable. Under Extreme Event, the calculated eccentricity of 6.99 ft is larger than the permissible eccentricity of $B/3 = L/3 = 20/6 = 6.66$ ft and is not acceptable.

Examination of stresses shows that contact stress calculated under Service-I limit state is higher than permissible net stress calculated from information (chart or table) provided by the GD. Therefore, size of the footing is increased to 24 ft × 24 ft and stresses are recalculated as shown in Table 15.8-9.

Table 15.8-9 Detailed Check for Footing Size (Second Trial)

Load Combination	M_T (kip-ft)	M_L (kip-ft)	P (kip)	e_T (ft)	e_L (ft)	L'_T (ft)	L'_L (ft)
Strength I-U	-37	2,582	3,913	0.01	0.66	23.98	22.68
Strength I-L	-75	2,087	2,925	0.03	0.71	23.95	22.57
Strength II-U	201	2,003	4,101	0.05	0.49	23.90	23.02
Strength II-L	162	1,508	3,113	0.05	0.48	23.90	23.03
Strength III-U	117	1,434	3,074	0.04	0.47	23.92	23.07
Strength III-L	79	939	2,086	0.04	0.45	23.92	23.10
Strength V-U	-2	2,319	3,721	0.00	0.62	24.00	22.75
Strength V-L	-40	1,824	2,733	0.01	0.67	23.97	22.66
Service I	3	1,754	2,241	0.00	0.78	24.00	22.43
Extreme Event I	22,126	0	3,376	6.55	0.00	10.89	24.00

Table 15.8-9 Detailed Check for Footing Size (Second Trial) (Continued)

Load Combination	A_e (ft^2)	L'/B'	q_o (ksf)	q_{pn} *or* q_R (ksf)	q_o/q_{pn} *or* q_o/q_R *Ratio*	Check
Strength I-U	543.9	1.06	7.20	**38.85**	0.15	OK
Strength I-L	540.6	1.06	5.41	**38.77**	0.14	OK
Strength II-U	550.3	1.04	7.45	**39.08**	0.15	OK
Strength II-L	550.3	1.04	5.66	**39.09**	0.14	OK
Strength III-U	551.9	1.04	5.57	**39.11**	0.14	OK
Strength III-L	552.7	1.04	3.77	**39.13**	0.10	OK
Strength V-U	546.1	1.05	6.82	**38.90**	0.18	OK
Strength V-L	543.3	1.06	5.03	**38.84**	0.13	OK
Service I	538.4	1.07	4.16	**4.22**	0.99	OK
Extreme Event I	261.3	2.20	12.91	**63.1***	0.23	OK

* L'/B' is out of range, therefore, factored nominal bearing resistance was calculated by extrapolation. For design purposes, the SD needs to ask the GD to provide adequate data to cover all applicable cases, without any need to extrapolation.

As shown in above tables, a 24 ft × 24 ft footing size satisfies stress requirements. Furthermore, the calculated eccentricities under service and extreme event limit states (0.78 ft and 6.55 ft, respectively) are smaller than the limits (4 ft and 8 ft, respectively). The ratio of length of footing from column face to face of footing, to thickness of footing, L_{ftg}/D_{ftg} = (0.5)(24-6)/4 = 2.25 that is slightly over the limit of 2.2 required by *SDC* 7.7.1.3 (Caltrans 2013) for rigidity of the footing. However, the *SDC* limitation is mostly applicable to pile cap and it is less critical for spread footings.

The factored nominal sliding resistance between footing and soil is calculated as:

$$R_R = \varphi R_n = \varphi_\tau R_\tau + \varphi_{ep} R_{ep} \qquad \text{(AASHTO 10.6.3.4-1)}$$

Assuming that soil passive pressure is negligible, $R_R = \varphi_\tau R_\tau$, and for cohesionless soil:

$$R_\tau = V \tan(\delta) \qquad \text{(AASHTO 10.6.3.4-2)}$$

where:

R_τ = nominal sliding resistance between soil and foundation (kip)
V = total vertical force (kip)

For concrete cast against soil : $\delta = \phi_f$ = internal friction angle of drained soil

The factored resistance against sliding failure for cast-in-place concrete on sand is calculated using φ_τ = 0.8 for strength limit states and φ_τ = 1.0 for extreme event limit state (AASHTO Table 10.5.5.2.2-1).

Similar to axial force, LRFD load factors are applied to unfactored resultant shear forces ($V_{Res.}$) to calculate factored shear forces for each load combination.

Table 15.8-10 shows that requirements of *AASHTO* Article 10.6.3.4 for sliding failure is met, therefore footing size of 24 ft × 24 ft is acceptable and will be used throughout this example.

Table 15.8-10 Sliding Check for Footing

Load Combination	Factored Resultant Shear (kip)	Factored Vertical Load (kip)	R_R (kip)	Check
Strength I-U	95	3,913	2,444	OK
Strength I-L	74	2,925	1,827	OK
Strength II-U	74	4,101	2,561	OK
Strength II-L	53	3,113	1,544	OK
Strength III-U	50	3,074	1,520	OK
Strength III-L	28	2,086	1,303	OK
Strength V-U	83	3,721	2,324	OK
Strength V-L	63	2,733	1,707	OK
Extreme Event I	865	3,376	2,635	OK

Upon finalizing the footing size, "Foundation Design Data Sheets" shown in Appendix are completed and forwarded to the GS to be used for preparation of "Foundation Design Recommendations".

15.8.6 Flexural Design

For structural design of the footing, distribution of contact stresses is assumed linear (trapezoidal or triangular) irrespective of the substrate stiffness (resting on soil or rock). If the eccentricity ($e=M/P$) is less than $L/6$ then the soil under the entire area of the footing is in compression and contact stresses can be determined based on trapezoidal distribution.

Maximum forces acting at the bottom of column for case-III can be summarized as:

Service: P = 2,188 kips; M_L = 1,754 kip-ft; M_T = 3 kip-ft

Strength I-U: P = 3638 kips; M_L = 2,582 kip-ft; M_T = -37 k-ft

The area and section modulus of the footing contact surface are: 576 ft^2 and 2,304 ft^3, respectively. Maximum and minimum contact stresses acting along the edges of the footing (q_1 and q_2) are calculated using the generic equation of $(P/A) \pm (M/S)$:

- *Strength Limit State:*

  L Direction: q_1=7.44 ksf; q_2=5.20 ksf

  T Direction: q_1=6.33 ksf; q_2=6.30 ksf

- *Service Limit State:*

  L Direction: q_1= 4.56 ksf; q_2= 3.04 ksf

  T Direction: q_1= 3.80 ksf; q_2= 3.80 ksf

Since the column has a circular cross section, it is transformed into an effective square section for footing analysis with equivalent column width of: $(28.26)^{0.5}$ = 5.32 ft.

Assuming #5 (d_b=0.69 in.) and #9 (d_b=1.25 in.) bars are used for top and bottom mat reinforcement, the minimum effective depths (d_e) of the footing for the top and bottom mats are calculated as 43.96 in. and 43.1 in., respectively.

Critical sections for moment and shear calculations:

- Bending moment at the face of the column (AASHTO 5.13.3.4)
- One-way shear at distance "d_v" from the face of the column (AASHTO 5.8.3.2)
- Two-way (punching) shear on the perimeter of a surface located at distance "$d_{v,avg}$" from the face of the column (AASHTO 5.13.3.6)

where:

d_v = effective shear depth of the section (ft)

$d_{v,avg}$ = average of effective shear depths for both directions (ft)

Using critical contact stresses (q_1 and q_2), maximum moments at the face of the column for unit foot width of the footing are calculated as:

- *Strength Limit State*: M_L = 311.8 kip-ft; M_T = 276.1 kip-ft
- *Service Limit State*: M_L =190.4 kip-ft; M_T = 165.8 kip-ft

Assuming 3 in. concrete cover, and using 42#9 bars for bottom mat, the spacing of rebars is calculated as:

$$s = [24(12)\text{-}2(3)\text{-}1.25]/(42\text{-}1) = 6.85 \text{ in.}$$

The calculated spacing is less than maximum spacing of 12 in. specified in *AASHTO* Article 5.10.8, and it is acceptable.

The area of steel contributing to unit width of the footing is: (1.0)(12)/6.85=1.75 in.^{2}, therefore the depth of the concrete stress block and resisting moment are calculated as:

$$a = \frac{(1.75)(60)}{(0.85)(3.6)(12)} = 2.86 \text{ in.}$$

Corresponding depth of the neutral axis will be c = (2.86)/(0.85) = 3.36 in. and the net tensile strain in extreme tension steel reinforcement is calculated as:

$$\varepsilon_s = (0.003)(43.1\text{-}3.36)/(3.36) = 0.035$$

Since calculated strain is larger than 0.005, the section is considered as tension-controlled and resistance factor ϕ is taken as 0.9 (AASHTO 5.5.4.2). The factored moment is calculated as:

$$M_r = \phi M_n = (0.9)(1.75)(60)(43.1\text{-}0.5\times 2.86)(1/12) = 328.1 \text{ kip-ft}$$

$$> M_L = 311.8 \text{ kip-ft} \qquad \text{OK}$$

Therefore, selected number of bars is adequate for strength in both directions. However *AASHTO* Article 5.7.3.3.2 requires minimum amount of reinforcement to be provided for crack control. The factored flexural resistance M_r is required at least equal to as the smaller of M_{cr} and 1.33 M_u as follows (gross section properties are used instead of transformed sections):

Modulus of rupture: $f_r = (0.24)(3.6)^{0.5} = 0.455$ ksi

Gross section modulus: $S_c = S_{nc} = (12)(48)^2/6 = 4{,}608$ in.^{3}

$$M_{cr} = \gamma_3\, \gamma_1\, f_r\, S_c = 1.6(0.75)(0.455)(4{,}608) = 2{,}516 \text{ kip-in.} = 209.7 \text{ kip-ft}$$

(AASHTO 5.7.3.3.2-1)

$$1.33M_u = 1.33(311.8) = 414.7 \text{ kip-ft}$$

$$M_r = \phi M_n = 328.1 \text{ kip-ft}$$

$$> \text{ smaller of } \begin{Bmatrix} M_{cr} = 209.7 \text{ kip-ft} \\ 1.33 M_u = 414.7 \text{ kip-ft} \end{Bmatrix} = 209.7 \text{ kip-ft}$$

OK (AASHTO 5.7.3.3.2)

AASHTO Article 5.7.3.4 requires maximum limits of rebar spacing for crack control.

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \qquad \text{(AASHTO 5.7.3.4-1)}$$

Assuming exposure factor $\gamma_e = 1$ (class-I exposure) and $d_c = 3 + (1.25/2) = 3.625$ in.

$$\beta_s = 1 + \frac{3.625}{0.7(48 - 3.625)} = 1.117$$

Cracked concrete section is used to calculate tensile stress in steel reinforcement under service loads:

$E_c = 57(3{,}600)^{0.5} = 3{,}420$ ksi

$n = 2{,}9000/3{,}420 = 8.48.$

Per *AASHTO* Article 5.7.1, n is rounded to the nearest integer number, therefore $n = 8$ will be used.

The distance of the neutral axis to the top of the footing is calculated as:

$y_b = 8.93$ *in.*

The moment of inertia for unit width (12 in.) of the transformed section (based on concrete) is calculated as:

$I_{trans} = (12)(8.93^3)/3 + (1.75)(8)(43.1 - 8.93)^2 = 19{,}194 \text{ in.}^4$

Tensile stress in steel reinforcement at the service limit state is calculated as:

$$f_{ss} = n\frac{M(d_e - y_b)}{I} = (8)\frac{(190.4)(12)(43.1 - 8.93)}{19{,}194} = 32.54 \text{ ksi}$$

The maximum spacing is checked as:

$$s = 6.85 \text{ in.} < \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c = \frac{700(1)}{(1.117)(32.54)} - 2(3.625) = 12 \text{ in.} \qquad \text{OK}$$

Therefore, 42#9 bars are acceptable for the bottom mat.

The shrinkage and temperature reinforcement for the top mat per unit foot width shall satisfy (AASHTO Article 5.10.8):

$$A_s > \frac{1.3bh}{2(b+h)f_y} = \frac{1.3(12 \times 24)(48)}{2(12 \times 24 + 48)(60)} = 0.446 \text{ in.}^2 \qquad \text{(AASHTO 5.10.8.1)}$$

$$0.11 \le A_s \le 0.6 \qquad \text{(AASHTO 5.10.8-2)}$$

Since thickness of the footing is greater than 18 in., spacing of the rebar shall not exceed 12 in. If 42#5 bars are considered:

s = [24(12)-2(3)-0.69]/(42-1) = 6.85 in. < 2 in. OK

A_s = (0.31)(12)/(6.85) = 0.543 in.^{2} > 0.446 in.^{2}/ft OK (AASHTO 5.10.8-1)

0.11 < A_s = 0.543 < 0.6 OK (AASHTO 5.10.8-2)

Therefore, 42#5 bars in each direction will be used for the top mat.

Note: For square footings the reinforcement shall be distributed uniformly across the entire width of the footing. (AASHTO Article 5.13.3.5)

15.8.7 Shear Design

According to *AASHTO* Article 5.13.3.6.1, both one-way and two-way shears shall be considered in footing design:

- The critical section for one-way action extends in a plane across the entire width and located at a distance as specified in *AASHTO* 5.8.3.2 (that is mostly at distance d_v from the face of the column).
- The critical section for two-way action is perpendicular to the plane of the footing and located so that its perimeter b_0, is a minimum but not closer than $0.5d_v$ to the perimeter of the concentrated load or reaction area.

where: $d_v = d - 0.5a$=43.1-0.5(2.86)=41.67 in. ≈ 3.5 ft. , however, d_v should be greater than both $0.9d_e$=0.9(43.1) = 38.79 in. and 0.72h= 0.72(48) = 34.56 inch. Therefore, d_v=3.5 ft will be used in calculations.

15.8.7.1 Direct (One-Way) Shear

The extreme contact stresses for most critical strength limit state case (*L* direction) are 5.20 ksf and 7.44 ksf. As shown in Figure 15.8-7, assuming a linear stress distribution the contact stress at distance d_v from face of the column is calculated:

q_3 = 7.44 - (7.44 – 5.20)(12-2.66-3.5)/(24) = 6.89 ksf

Shear force at critical section for unit width:

V_u = (7.44+6.89)(12-2.66-3.5)/2 = 41.84 kips

The maximum shear resistance of the section (considering shear reinforcement contribution) is limited to $0.25 f'_c b_v d_v$ (AASHTO 5.8.3.3-3):

$V_{n,\,max}$ = 0.25(3.6)(12)(41.67) = 450 kips

This maximum shear resistance is much higher than factored shear force of 41.84 kips, and is not governing.

Shear resistance of concrete (V_c) is $0.0316\beta\sqrt{f'_c}b_v d_v$, where β = 2.0.

$$V_c = 0.0316(2)\sqrt{3.6}(12)(41.67) = 59.96\ \text{kips}$$

Assuming that no shear reinforcement will be used, $V_s=0$ and

ϕV_n=0.9(60) = 54 kips > 39.15 kips OK

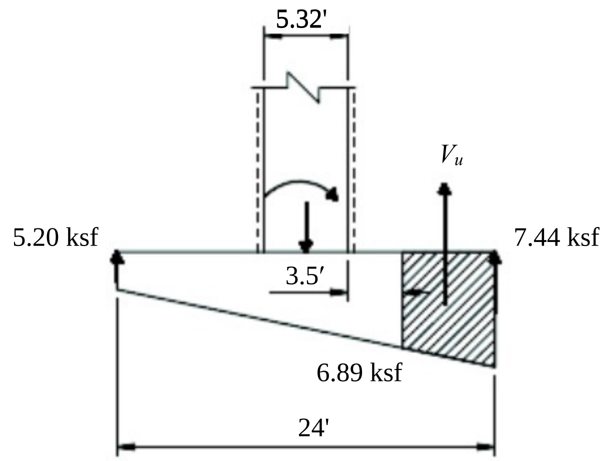


Fig 15.8-7 Direct Shear Force Calculation

15.8.7.2 Punching (Two-Way) Shear

The critical section is located at the distance of 0.5 $d_{v,avg.}$ from face of the column as shown in Figure 15.8-8.

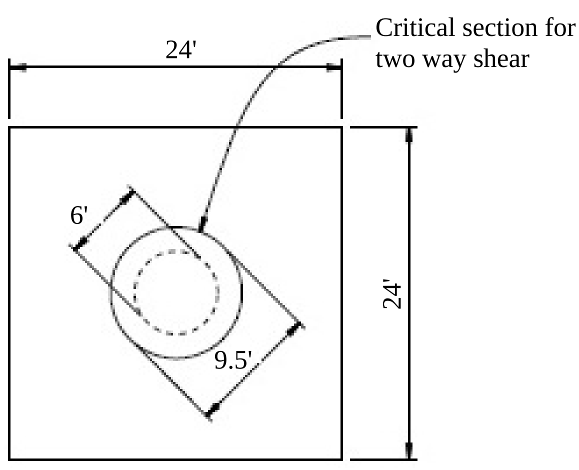


Fig 15.8-8 Critical Section for Two-Way Shear

Using conservative assumption of $d_{v,avg}$=3.5 ft results in $b_0 = \pi\,(6+3.5) = 29.83$ ft = 358 in. For square footing $\beta_c = 1$, and nominal shear resistance is as:

$$V_n = \left(0.063 + \frac{0.126}{\beta_c}\right)\sqrt{f'_c}\,b_o d_v = \left(0.063 + \frac{0.126}{1.0}\right)\sqrt{3.6}\,(358)(41.67) = 5{,}350 \text{ kips}$$

$$\because\ V_n = 5{,}350 \text{ kips} > 0.126\sqrt{f'_c}\ b_o d_v = 0.126\sqrt{3.6}(358)(41.67) = 3{,}566 \text{ kips}$$

$$\therefore \text{Use } V_n = 3{,}566 \text{ kips}$$

$$\phi V_n = 0.9(3{,}566) = 3{,}210 \text{ kips}$$

The punching shear force acting on the critical surface is calculated by subtracting the force resulting from soil contact stress acting on the critical surface from the axial force of the column:

$$P_{2-way} = 3{,}638 - \frac{3{,}638}{(24)(24)}\pi(4.75)^2 = 3{,}190 \text{ kips} < \phi V_n = 3{,}210 \text{ kips} \qquad \text{OK}$$

Shear reinforcement is not required and the footing depth d = 4.0 ft is acceptable.

Note: Although seismic loads were considered in sizing of the footing, structural design of the footing was only based on service and strength I-U (Case-III) limit states. Refer to Caltrans *SDC* (Caltrans, 2013) for other design and detailing requirements.

APPENDIX

COMMUNICATIONS OF STRUCTURAL AND GEOTECHNICAL DESIGNERS (REFER TO MTD 4-1 AND MTD 1-35)

Preliminary Foundation Design Data Sheet (Trial Footing Size)

Table 1 Preliminary Foundation Data

Support No.	Finished Grade Elevation (ft)	BOF Elevation (ft)	Estimated Footing Dimensions (ft)		Permissible Settlement under Service-I Load (in.)	Approximate Ratio of $\left(\frac{\text{Permanent}}{\text{Total}}\right)$ Service –I Load
			B	*L*		
Abut 1					1 or 2	-
Bent 2	48	39	10	10	1	0.75
Abut 3					1 or 2	-

Table 2 – Scour Data

Support No.	Long-term (Degradation and Contraction) Scour Elevation (ft)	Short-term (Local) Scour Depth (ft)
Abut 1	N/A	N/A
Bent 2	48	0.00
Abut 3	N/A	N/A

Foundation Design Data Sheet (Final Footing Size)

Table 3 Foundation Data

Support No.	Finished Grade Elevation (ft)	BOF Elevation (ft)	Footing Dimensions (ft)		Permissible Settlement under Service Load (in.)
			B	*L*	
Abut 1					1 or 2
Bent 2	48	39	24	24	1
Abut 3					1 or 2

Table 4 LRFD Service-I Limit State Loads for Controlling Load Combination

Support No.	Total Load					Permanent Load				
	P_{Total} (kip) Net	M_x (kip-ft)	M_y (kip-ft)	V_x (kip)	V_y (kip)	P_{Perm} (kip) Gross	M_x (kip-ft)	M_y (kip-ft)	V_x (kip)	V_y (kip)
Abut 1			N/A	N/A				N/A	N/A	
Bent 2	2,241	1,754	3	N/A	N/A	1,762	1,098	85	N/A	N/A
Abut 3			N/A	N/A				N/A	N/A	

Table 5 LRFD Strength/Construction and Extreme Event Limit States Load Data

Support No.	Strength/Construction Limit State (Controlling Group)					Extreme Event Limit State (Controlling Group)				
	P_{Total} (kips) Gross	M_x (kip-ft)	M_y (kip-ft)	V_x (kip)	V_y (kip)	P_{Total} (kip) Gross	M_x (kip-ft)	M_y (kip-ft)	V_x (kip)	V_y (kip)
Abut 1	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A
Bent 2	4101	2003	201	N/A		3,376	0	22,126	864	35
Abut 3	N/A	N/A	N/A	N/A		N/A	N/A	N/A	N/A	N/A

Note: Load tables may be modified to submit multiple lines of critical load combinations for each limit state, if necessary.

NOTATION

A = actual footing area (ft^2)

A' = reduced effective area of the footing (ft^2)

B, L = actual dimensions of the footing (ft)

B', L' = effective dimensions of the footing (ft)

clr. = minimum clearance from the bottom of footing to the bottom mat of footing reinforcement (in.)

d_b = diameter of bar (in.)

$d_{footing}$ = footing depth (ft)

$d_{min.}$ = minimum footing depth (ft)

d_v = effective shear depth of the section (ft)

$d_{v,avg}$ = average of effective shear depths for both directions (ft)

e_y, e_x = eccentricities parallel to dimensions B and L, respectively (ft)

f_c' = 28-day compressive strength of concrete (psi)

f_y = specified minimum yield strength of steel (ksi)

l_{db} = development length for deformed bars (in.)

l_{hb} = development length for deformed bars (in.)

l_d' = required development length of the main column reinforcement (in.)

M_L, M_T = moments acting about L and T directions, respectively (kip-ft)

M_p = plastic moment at column base (kip-ft)

M_x, M_y = moments acting X and Y directions, respectively (kip-ft)

P = vertical force acting at the center of gravity of the bottom of the footing area (kip)

q = uniform bearing stress (ksf)

$q_{g,u}$ = gross uniform bearing stress (ksf)

$q_{g,max}$ = gross maximum bearing stress (ksf)

q_n = gross nominal bearing resistance (ksf)

$q_{n,max}$ = net maximum bearing stress calculated using Service-I Limit State loads assuming linear stress distribution for footings on rock (ksf)

$q_{n,u}$ = net uniform bearing stress calculated using Service-I Limit State loads assuming uniform stress distribution for footings on soil (ksf)

q_{pn} = permissible net contact stress provided by the GD and calculated based on a specified allowable settlement (ksf)

q_R = factored gross nominal bearing resistance provided by the GD = $\varphi_b q_n$ (ksf)

R_τ = nominal sliding resistance between soil and concrete (kip)

S_x, S_y = section modulus of the footing area about X and Y directions, respectively (ft^3)

V = total force acting perpendicular to the interface (kip)

V_L, V_T = shears acting along L and T directions, respectively (kip)

V_p = plastic shear at column base (kip)

V_x, V_y = shears acting along X and Y directions, respectively (kip)

δ = friction angle at interface of concrete and soil (degree)

ϕ_f = internal friction angle of drained soil (degree)

φ_b = resistance factor for bearing

φ_τ = resistance factor against sliding

REFERENCES

1. AASHTO. (2012). *AASHTO LRFD Bridge Design Specifications,* Customary U.S. Units, 2012 (6th Edition), American Association of State Highway and Transportation Officials, Washington, D.C.

2. Caltrans. (2013). *Caltrans Seismic Design Criteria,* Version 1.7, California Department of Transportation, Sacramento, CA.

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5. Caltrans. (2008). *Bridge Memo to Designers 1-35: Foundation Recommendation and Reports,* California Department of Transportation, Sacramento, CA.