# CHAPTER 10.1
# SHALLOW FOUNDATIONS

This page is intentionally left blank.

## 10.1.1 INTRODUCTION

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

In general, the size of the spread footing is determined based on the bearing resistance of the supporting soil or rock and the permissible level of settlement. Design of spread footings requires constant communication between the Structural Designer (SD) and the Geoprofessional throughout the design process. Factored loads are provided by the SD, and factored resistance for the supporting soil and rock, that is a permissible net contact stress $q_{p,net}$ and factored gross nominal bearing resistance $q_R$ are calculated and reported by the Geoprofessional. The SD performs the structural design. Consistency between the SD and the Geoprofessional in the use of required gross or net stresses is important. In the first part of this chapter, common types of spread footings and fundamental aspects of shallow foundation design according to *AASHTO LRFD Bridge Design Specifications,* 8$^{th}$ Edition with *California Amendments,* referred to herein as AASHTO-CA BDS-8 (AASHTO, 2017; Caltrans, 2019a), and Seismic Design Criteria (SDC) Version 2.0 (Caltrans, 2019b) are discussed. Subsequently, a design example of bridge bent spread footing is presented to illustrate the typical design procedure.

## 10.1.2 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, whereas moments act only in a short direction.

## 10.1.3 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 the required depth of the footing, when selecting the size and location of the footing. Although square footings are more common for footings supporting pinned columns, rectangular shapes may be more efficient when the 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-CA BDS-8, variation of geotechnical resistances with eccentricities of loads acting on the footing and any type of optimization can be rigorous.

### 10.1.3.1 Sizing of Spread Footings

The trial minimum size of the spread footing can be selected based on footings of similar conditions and past experience. The size of a spread footing is usually governed by the column size, intensity of loads acting on the footing, and resistances of the substrate. The effective length to effective width ratio, ($L'/B'$), is commonly between 1.0 ~ 2.0. Geoprofessionals should be consulted to select the ratios. The allowable settlement will be assumed as 1 in. or 2 in. based on the 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, 2019a).

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 the strength and extreme event limit states.

The factored nominal bearing resistance and permissible net stress are functions of the effective width as well as the effective length to effective width ratio ($L'/B'$); therefore, they are presented by a family of curves and 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 Geoprofessional may be contacted to revise the information and provide a new set of curves and tables to avoid extrapolation.

## 10.1.3.2 Embedment and Depth of Footings

The footing embedment shall be carefully determined for degradation and contraction scour for the design (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 10.1.3-1. If the footing is not in water and freezing is not a concern, a minimum cover of 2 ft is recommended.

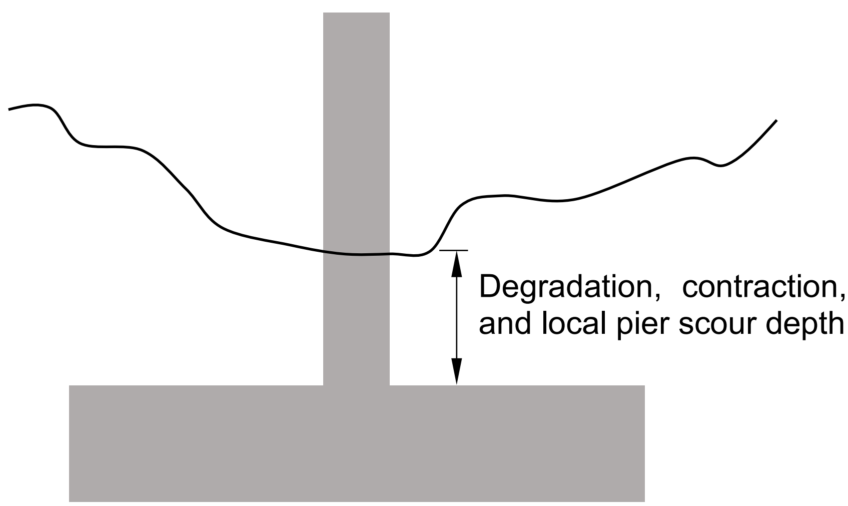



**Figure 10.1.3-1 Minimum Embedment for Scour Protection**

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.

## 10.1.4 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 thickness.

## 10.1.5 BEARING STRESS DISTRIBUTION

The sign convention shown in Figure 10.1.5-1 is to avoid mistakes in communications between the SD and the Geoprofessional. 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 10.1.5-2. 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 with the longitudinal and transverse directions of the bridge.

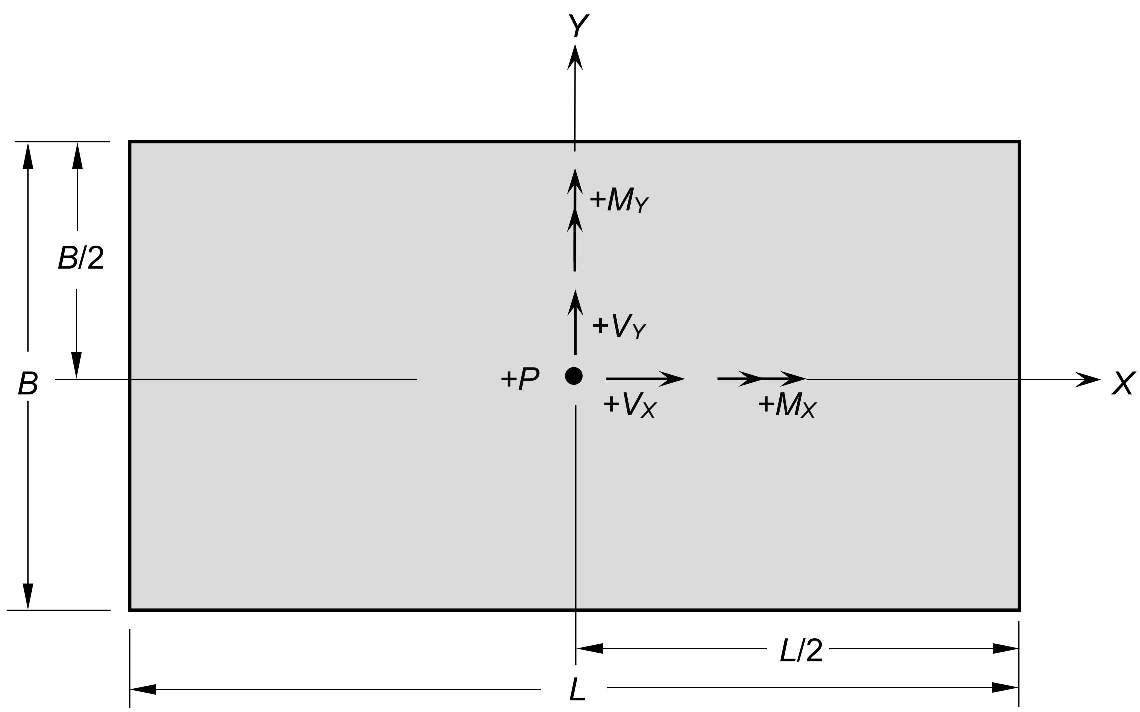



**Figure 10.1.5-1 Components of Moment and Shear in Local Coordinates of the Spread Footing**

Bearing stress distribution depends on the 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 per AASHTO-CA BDS-8 Article 10.6.1.4 (Caltrans, 2019a). For the structural design of the footing, bearing stress is assumed to be linearly distributed per AASHTO Article 10.6.5 (AASHTO, 2017).

For eccentrically loaded footings on soil, the effective footing dimensions ($B'$ and $L'$) specified in AASHTO Article 10.6.1.3 (AASHTO, 2017) shall be used for the 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 10.1.5.2 and shall be taken as follows:

$$B' = B - 2e_y$$

$$L' = L - 2e_x \qquad (10.1.5\text{-}1)$$

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)

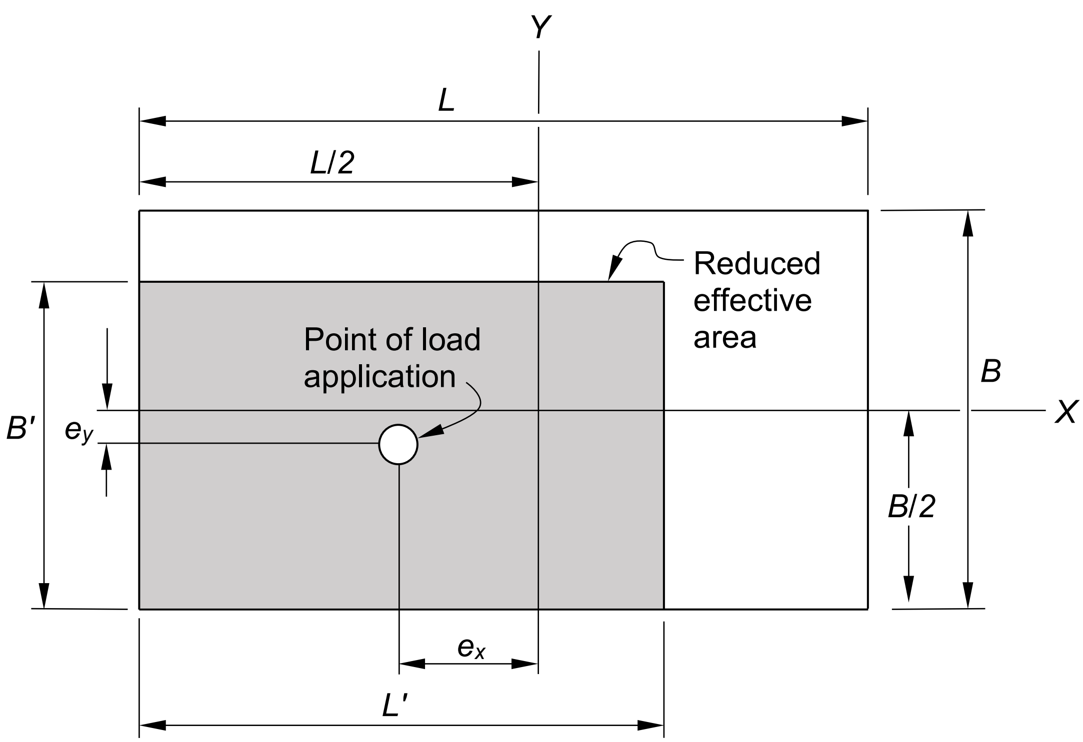



**Figure 10.1.5-2 Effective Footing Area**

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 controlling bearing stresses are calculated as:

$$q_{\max} = \frac{P}{A} \pm \left|\frac{M_y}{S_y}\right| \pm \left|\frac{M_x}{S_x}\right| \tag{10.1.5-2}$$

where:

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

$P$ = vertical force acting at the centroid of the bottom of the footing area (kip)

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

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

Equation (10.1.5-2) is valid only if stresses calculated at corners of the footing are all positive (compression). Otherwise, the reduced contact area of the footing must be determined. In this case, the orientation of the natural axis will not be parallel to the *X* or *Y* axes, and a complex analysis may be needed.

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

## 10.1.6 GENERAL DESIGN REQUIREMENTS

The bearing stresses calculated under various AASHTO-CA BDS-8 limit states must be checked against acceptable stresses provided by the Geoprofessional. After receiving foundation information and scour data (if applicable), the Geoprofessional will provide “permissible net contact stress” used for Service Limit State 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 B′ for a given *L′/ B′* ratio*′*. 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 Geoprofessional may be contacted to revise the information and provide a new set of curves and tables to avoid extrapolation.

### 10.1.6.1 Settlement Check

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

$$q_{net,u} \leq q_{p,net} \text{ for footing on soil} \quad (10.1.6\text{-}1)$$

$$q_{net,max} \leq q_{p,net} \text{ for footing on rock} \quad (10.1.6\text{-}2)$$

where:

$q_{p,net}$ = permissible net contact stress provided by the Geoprofessional and calculated based on a specified allowable settlement (ksf)

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

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

### 10.1.6.2 Bearing Check

For Strength and Extreme Event Limit States, the design requirements are written as:

$$q_{g,u} \leq q_R \text{ for footing on soil} \quad (10.1.6\text{-}3)$$

$$q_{g,max} \leq q_R \text{ for footing on rock} \quad (10.1.6\text{-}4)$$

where:

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

$q_{g,max}$ = applied 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 Geoprofessional = $\varphi_b \, q_n$ (ksf)

$q_n$ = gross nominal bearing resistance (ksf)

$\varphi_b$ = resistance factor (AASHTO-CA BDS-8 Table 10.5.5.2.2-1)

### 10.1.6.3 Eccentricity Limits

The eccentricity limits for Service, Strength, and Extreme Event Limit States specified in AASHTO-CA BDS-8 are summarized in Table 10.1.6-1.

**Table 10.1.6-1 Eccentricity Limits**

| Limit State | Footing on Soil | Footing on Rock | Article Number |
|---|---|---|---|
| Service | $B/6$ or $L/6$ | $B/4$ or $L/4$ | AASHTO-CA BDS-8 10.5.2.2 |
| Strength | - | - | AASHTO-CA BDS-8 10.6.3.3 |
| Extreme Event – I (Seismic) $\gamma_{EQ} = 0.0$ | $B/3$ or $L/3$ | $B/3$ or $L/3$ | AASHTO 10.6.4.2 and 11.6.5.1 |
| Extreme Event – I (Seismic) $\gamma_{EQ} = 1.0$ | $2B/5$ or $2L/5$ | $2B/5$ or $2L/5$ | AASHTO 10.6.4.2 and 11.6.5.1 |

Note: Seismic forces should be applied in all directions with 15-degree increments per STP 10.6. It is not necessary to include live load (design or permit truck) in Extreme Event Limit State load combinations for ordinary standard bridges; therefore $\gamma_{EQ} = 0.0$.

## 10.1.6.4 Sliding Check

The 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 the equation is summarized as $R_R = \phi R_n = \phi_\tau R_\tau$. For cohesionless soil, $R_\tau$ is written as:

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

where:

$R_n$ = nominal sliding resistance against failure by sliding (kip)

$\varphi_\tau$ = resistance factor against sliding = 0.80 for cast-in-place concrete on cohesionless soil (AASHTO-CA BDS-8 Table 10.5.5.2.2-1)

$R_\tau$ = nominal sliding resistance between soil and concrete (kip)

$\varphi_{ep}$ = resistance factor for passive resistance specified in AASHTO-CA BDS-8 Table 10.5.5.2.2-1

$R_{ep}$ = nominal passive resistance of soil available throughout the design life of the structure (kip)

$C$ = 1.0 for concrete cast against soil

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

$\phi_f$ = internal friction angle of the drained soil for concrete cast against soil (degree)

## 10.1.7 STRUCTURAL DESIGN OF FOOTINGS

The structural design of the footing includes the following steps:

- Select footing thickness based on the required development length of the column reinforcement
- Design flexural reinforcement in both directions with a consideration of the minimum reinforcement requirement for shrinkage and temperature
- Check the thickness of the footing for one-way and two-way shears and design shear reinforcement if required
- Check seismic details per Caltrans SDC (Caltrans, 2019b) and other practice manuals

Table 10.1.7-1 summarizes the requirements for the structural design of the footings specified in AASHTO-CA BDS-8. The application of these requirements will be illustrated in the design example.

**Table 10.1.7-1 Requirements for Structural Design of Footings**

| Topic | Articles | Application |
|---|---|---|
| Strut & tie Applicability | 5.8.2 | Requirement check |
| Flexural design | 5.6.3.2 | Reinforcement design |
| Direct shear design | 5.7.3.3 | Footing depth and reinforcement design |
| Shear friction | 5.7.4 | Shear key design |
| Reinforcement spacing | 5.6.3.3, 5.6.7 , 5.10.3 , 5.10.6 | Design and detailing |
| Reinforcement development | 5.10.8.2 | Structural design of footings |
| Concrete cover | 5.14.3 | Footing depth and detailing |
| Footings | 5.12.8 | General provisions |

# 10.1.8 DESIGN EXAMPLE

## 10.1.8.1 Bridge Footing Data

The design process for an overcrossing 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. The footing shown in Figure 10.1.8-1 rests on cohesionless soil with an 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 10.1.8-1.
- Unfactored dead load and seismic forces at the base of the column are listed in Table 10.1.8-2.
- Plastic moment and shear applied at the column base are:

  $M_p$ = 15,573 kip-ft; $V_p$ = 716 kip

  Overturning column axial force in transverse push is 992 kip.

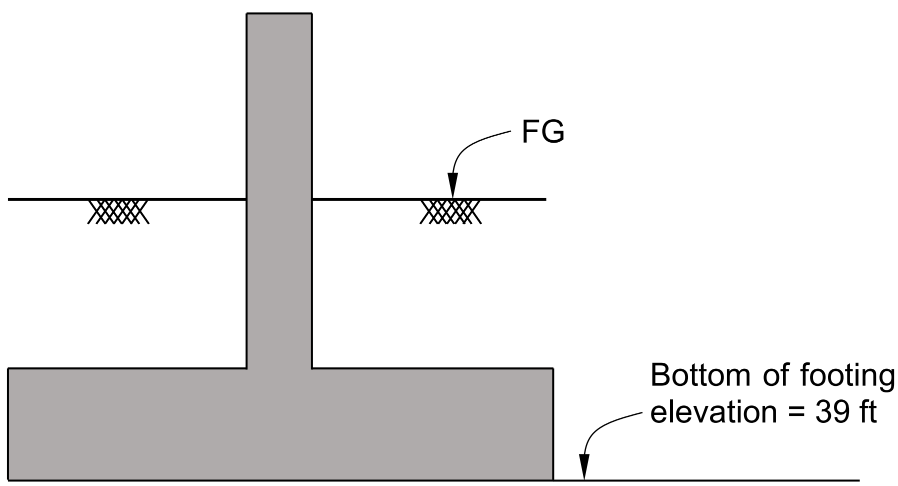



**Figure 10.1.8-1 Elevation of the Spread Footing**

### Table 10.1.8-1 Unfactored Live Load Moments and Forces at Column Base

| Load Case | HL-93 Truck | | | Permit Truck | | |
|---|---|---|---|---|---|---|
| | Case 1 | Case 2 | Case 3 | Case 1 | Case 2 | Case 3 |
| $M_T$ (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 |

Case 1 - Maximum Transverse Moment ($M_T$) and associated effects
Case 2 - Maximum Longitudinal Moment ($M_L$) and associated effects
Case 3 - Maximum Axial Force ($P$) and associated effects

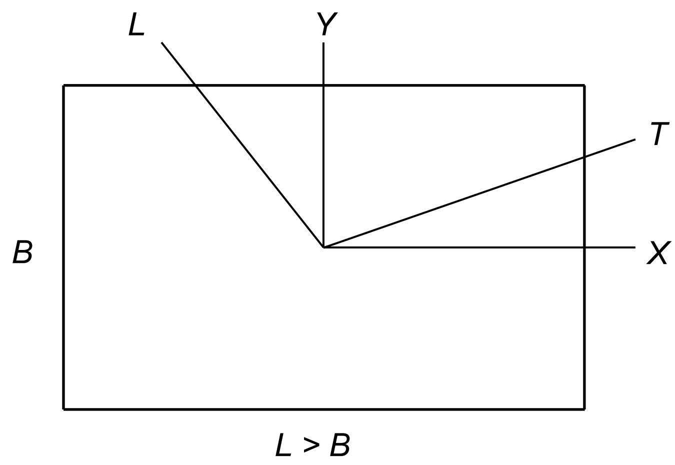
### Table 10.1.8-2 Unfactored Dead Load and Seismic Forces Applied at Column Base

| Load Case | *DC* | *DW* | *PS* | Seismic-I* ($M_P$ applied) | Seismic-II# ($M_P$ applied) |
|---|---|---|---|---|---|
| $M_T$ (kip-ft) | 62 | 9 | 0 | 15,573 | 0 |
| $M_L$ (kip-ft) | 833 | 139 | –14 | 0 | 15,573 |
| $P$ (kip) | 1,503 | 227 | –21 | 992 | 0 |
| $V_T$ (kip) | 4 | 1 | 0 | 716 | 0 |
| $V_L$ (kip) | 44 | 7 | –16 | 0 | 716 |

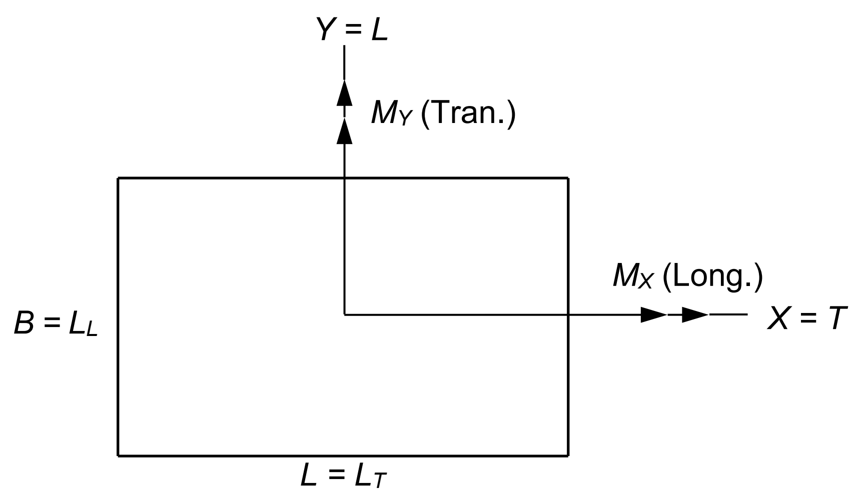
*Forces and moments resulting from seismic analysis in the transverse direction
#Forces and moments resulting from seismic analysis in the longitudinal direction

Notes:

1) To facilitate communications between the SD and the Geoprofessional, the local coordinates of the foundation have been defined as *X* and *Y*. As shown in Figure 10.1.8-2a, the local *X* axis is parallel to the 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 Geoprofessional shall be based on forces/moments calculated in the local coordinates of the footing. In this example, local and global coordinates coincide $X = T$ and $Y = L$. Therefore, local and global coordinates may have been used interchangeably, as shown in Figure 10.1.8-2b.
2) In this example, overstrength moment and shear have been applied only in transverse and longitudinal directions; however, software such as CTFoot use 15-degree increments required by STP 10.6.




**(a) General Case**




**(b) Example Problem**

**Figure 10.1.8-2 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 Geoprofessional.

### 10.1.8.2 Design Requirements

Perform the following design steps for the footing in accordance with the *AASHTO LRFD Bridge Design Specifications*, 8th Edition (AASHTO, 2017) with the California Amendments (Caltrans, 2019a), and design peak ground acceleration (PGA) = 0.6*g*.

- Determine the minimum footing thickness required to develop the column reinforcement. (Assume #9 bars for footing bottom reinforcement)
- Calculate LRFD factored loads 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

### 10.1.8.3 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 nominal diameters of the bars used for the bottom of footing reinforcement, plus the required development length of the main column reinforcement.

$$d_{min} = clr. + 2(d_b) + l'_d \quad (10.1.8\text{-}1)$$

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 the reinforcement bar (in.)

$l'_d$ = required development length of the main column reinforcement (in.)

From AASHTO-CA BDS-8 Table 5.10.1-1, *clr.* = 3 in., and for #9 bars, $d_b$ = 1.128 in.. The development length is calculated in accordance with AASHTO Articles 5.10.8.2.2 and 5.10.8.2.4.

The column's main reinforcement bars are #14 with $d_b$ = 1.693 in.

**Development of Deformed Bars in Compression:**

The basic development length ($l_{db}$) shall be larger than the greater of the following:

$$l_{db} \geq 0.63(1.693)(60)/\sqrt{3.6} = 33.7 \text{ in.} \qquad \text{(AASHTO 5.10.8.2.2a-2)}$$

$$l_{db} \geq 0.3\ (1.693)(60) = 30.5 \text{ in.} \qquad \text{(AASHTO 5.10.8.2.2a-3)}$$

AASHTO Article 5.10.8.2.2b states that the basic development length ($l_{db}$) 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 the main column hoops are spaced at 5 inches.

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

**Development of Standard Hooks in Tension**

$$l_{hb} = 38.0(1.693)(60)/(60)\sqrt{3.6} = 33.9 \text{ in.} \qquad \text{(AASHTO 5.10.8.2.4a-2)}$$

Although Article 5.10.8.2.4 states, “the provision herein may be used for No. 11 bars or smaller in normal-weight concrete….”, the ACI 318-19 method of 10 $d_b$ will provide a much lower hook basic development length ($l_{hb}$) and compression development length ($l_{db}$) controls the thickness of the footing. This example uses the AASHTO equation 5.10.8.2.4a-2 to calculate the development length in tension. Per AASHTO Article 5.10.8.2.4a, the $l_{hb}$ shall be multiplied by applicable modification factors specified in AASHTO Article 5.10.8.2.4b.

By inspection, none of the modification factors are applied since #14 bars have been used for columns; therefore, the development length of standard hooks in tension = 33.9 in., say 34 in. (It is also greater than 8 × 1.693 in. and 6 in. limits per AASHTO Article 5.10.8.2.4a).

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

$$d_{min.} = clr. + 2(d_b) + l_d' = 3 + 2(1.128) + 34 = 39.26 \text{ in.} = 3.27 \text{ ft.}$$

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

## 10.1.8.4 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 1) Maximum Transverse Moment ($M_T$) and associated effects

Case 2) Maximum Longitudinal Moment ($M_L$) and associated effects

Case 3) 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 1,

Forces applied at the column base are:

$$M_T = -206 \text{ kip-ft}$$

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

For the footing thickness $d_{footing}$ = 4 ft, forces applied at the bottom of the 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 10.1.8-3.

**Table 10.1.8-3 Unfactored Live Load Forces at Bottom of Footing**

| Load Case | HL-93 Truck | | | Permit Truck | | |
|---|---|---|---|---|---|---|
| | Case 1 | Case 2 | Case 3 | Case 1 | Case 2 | Case 3 |
| $M_T$ (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 |

In this example, only Case 3 for live loads (both HL-93 and Permit Trucks) has been illustrated; however, all three cases are 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. Per Caltrans SDC (Caltrans, 2019b), Article 5.4.1, the footing is classified as a capacity protected member (CPM); thus, it will be designed for column overstrength shear and moment. For the footing thickness $d_{footing}$ = 4 ft, overstrength moment and shear applied at the bottom of the footing are calculated as:

$$M_o = 1.2\,[15{,}573 + (716)(4)] = 22{,}124 \text{ kip-ft}$$

$V_o$ = 1.2 (716) = 859 kip

The unfactored dead load forces and seismic forces at the bottom of the footing are shown in Table 10.1.8-4.

**Table 10.1.8-4 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 | 22,124 | 0 |
| $M_L$ (kip-ft) | 1,009 | 167 | -78 | 0 | 22,124 |
| $P$ (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, 2017) used in foundation design and corresponding load factors (AASHTO Tables 3.4.1-1, 3.4.1-2, and 3.4.1-3) are summarized in Table 10.1.8-5. The upper and lower limits of permanent load factors ($\gamma_p$) are shown as "*U*" and "*L*", respectively.

**Table 10.1.8-5 Load Factors for Footing Design**

| Load | *DC* | *DW* | *PS*[1] | *EV*[2] | *HL-93* | *P-15* | *EQ* |
|---|---|---|---|---|---|---|---|
| Strength I-U | 1.25 | 1.50 | 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 | 0.90 | 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 | -[3] | - | 1.00 |

[1]$I_{effective}$ used for Column Moment of Inertia
[2]Rigid Buried Structure
[3]Based on AASHTO-CA BDS-8 Article 3.4.1

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 bottom of the footing, as summarized in Table 10.1.8-6. Only the Seismic-I case has been used in the Extreme Event-I load combination in this example. In Caltrans' practice, the column's seismic over-strength moment and associated shear force at the top of the shallow foundation are applied in 15-degree increments to determine the maximum effects. The reader may refer to STP 10.6 (Caltrans 2022).

**Table 10.1.8-6 Factored Forces at Bottom of Footing 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 | 22,124 | 0 | 2,701 | 859 | 0 | 859 |

* at the column base

For example, calculations of the factored axial force ($P$) and resultant shear ($V_{Total}$) for Strength II-U limit state:

$$P = 1.25(1{,}503) + 1.5(227) + 1(-21) + 1.35(760) = 3{,}224 \text{ kip}$$

$$V_{Total} = \sqrt{(16)^2 + (72)^2} = 74 \text{ kip}$$

## 10.1.8.5 Footing Size Determination

In order to design a spread footing, all live load combinations (Cases 1, 2, and 3) should be considered for both design and permit trucks. It is recommended to consider the maximum axial case (Case 3) for the initial sizing of the footing and check footing size and stresses for the other two cases (1 and 2); however, this example only considers Case 3. Based on the preliminary analysis of the footing, reasonable estimates for "width of the footing" and "length to width ratios" are provided to the Geoprofessional to be used in the design.

The Geoprofessional 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 10.1.8-3 to 10.1.8-5, and Table 10.1.8-7 for given ranges of footing widths and also effective length to effective width ratios.

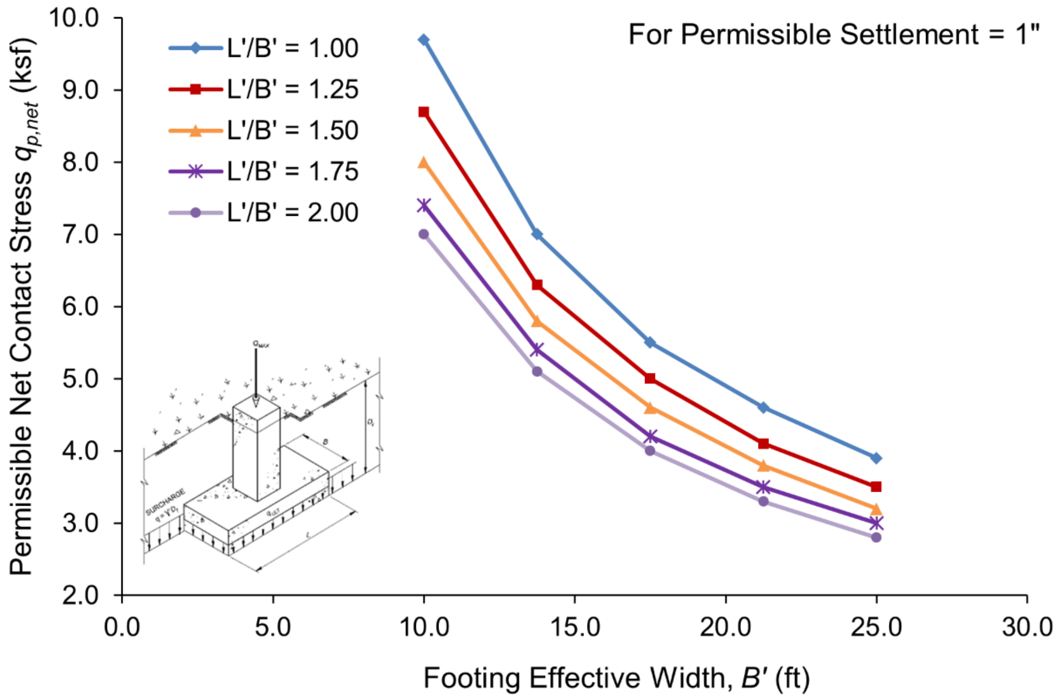



**Figure 10.1.8-3 Variations of Permissible Net Contact Stress**

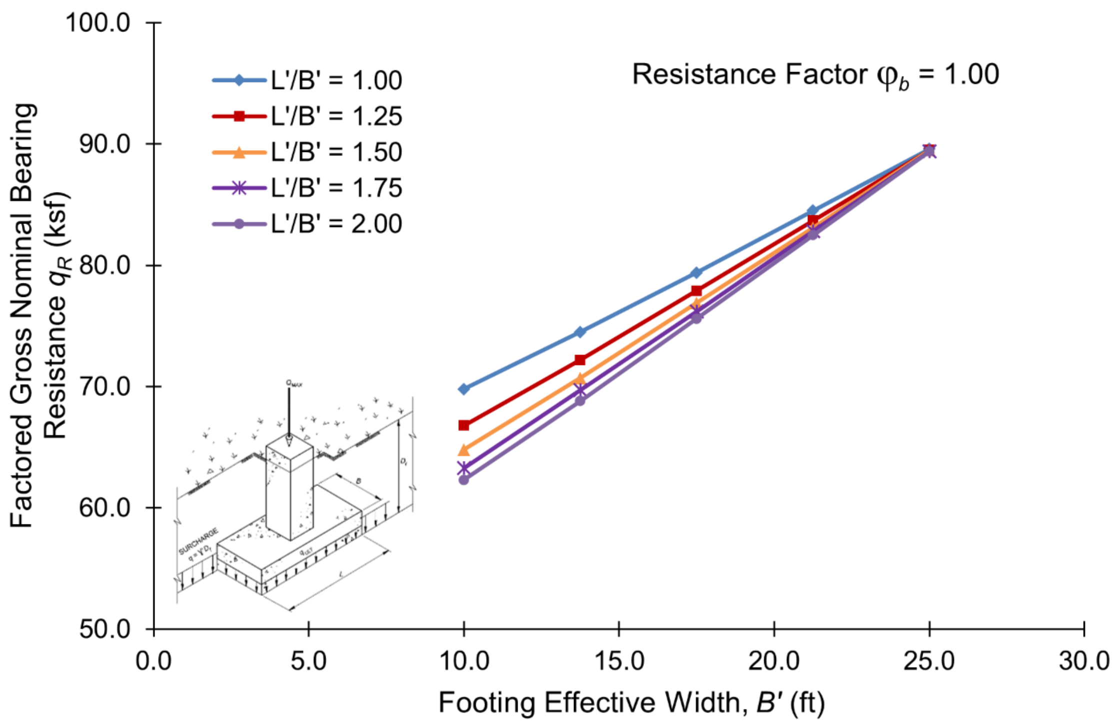



**Figure 10.1.8-4 Variations of Factored Gross Nominal Bearing Resistance (Strength Limit State)**

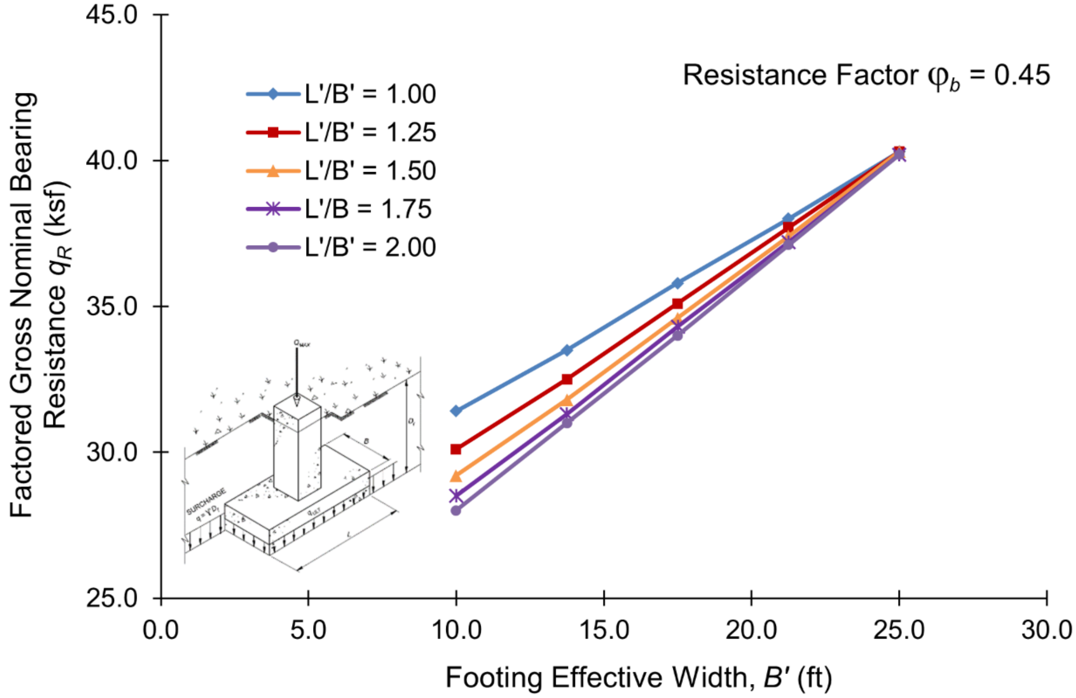



**Figure 10.1.8-5 Variations of Factored Gross Nominal Bearing Resistance (Extreme Event Limit State)**

**Table 10.1.8-7 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) | $q_{p,net}$ (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 | 17.5 | 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 |

In the first trial, a square footing of 20 ft × 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). Since the footing rests on soil, the contact stress distribution is assumed uniform over the effective area of the footing. The bearing stresses should be calculated as a net for the Service-I limit state and gross for all strength and extreme event limit states, as shown in Figure 10.1.8-6. Therefore, the weight of overburden soil and footing with corresponding load factors have been considered in the axial forces shown in Table 10.1.8-8.

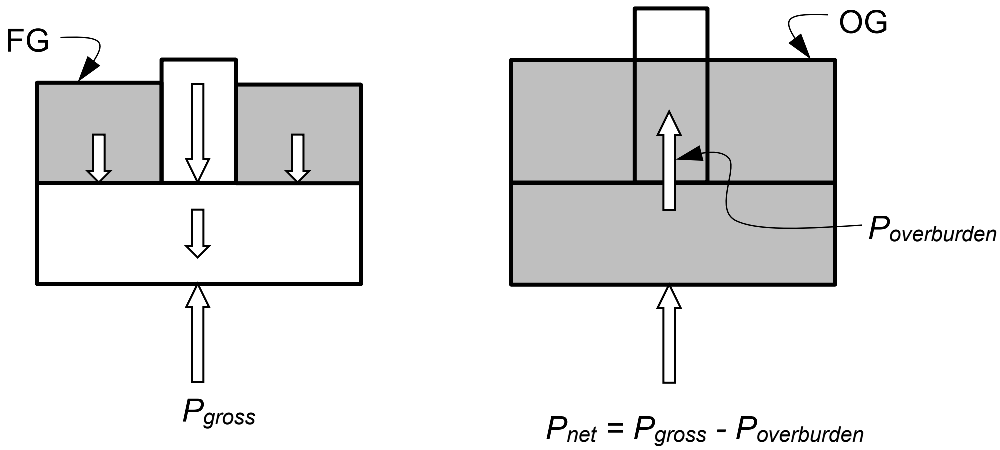



**Figure 10.1.8-6 Definition of Gross and Net Bearing Stresses**

For example:

Strength I-U

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

$P_{gross}$ = 3,307 + [(20)(20) – 28.27](48-39-4)(120/1,000)(1.35) + (20)(20)(4)(150/1,000)(1.25) = 3,638 kip

Service-I

$P_{net}$ = $P_{at\ column\ base}$ + weight of backfill soil on footing + weight of footing – excavated soil (which is overburden soil)

$P_{net}$ = 2,188 + [(20)(20) - 28.26](48-39-4)(120/1,000) + (20)(20)(4)(150/1,000) – (48-39)(20)(20)(120/1,000) = 2,219 kip

$P_{net}$ will be utilized for settlement checks only.

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

$M_T$ = –37 kip-ft; $P$ = 3,638 kip

$e_T$ = 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 = 0.71 ft

$L_L'$ = 20–2(0.71) = 18.58 ft

$A_e$ = 19.98(18.58) = 371.23 ft$^2$; $q_{g,u}$ = 3,638/371.23 = 9.80 ksf

$L'/B'$ = 19.98/18.58 = 1.08, therefore $q_R$ = 36.26 ksf (From Figure 10.1.8-4)

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

A similar calculation is required for every load combination, as shown in Table 10.1.8-8.

### Table 10.1.8-8 Detailed Check for Footing Size (First Trial)

| Load Combination | $M_T$ (kip-ft) | $M_L$ (kip-ft) | $P_{gross}$ (kip) | $e_T$ (ft) | $e_L$ (ft) | $L'_T$ (ft) | $L'_L$ (ft) | $A_e$ (ft$^2$) | $L'/B'$ | $q_0$ (ksf) | $q_{p,net}$ or $q_R$ (ksf) | $q_0/q_{p,net}$ or $q_0/q_R$ Ratio | Check |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Strength I-U | –37 | 2,582 | 3,638 | 0.01 | 0.71 | 19.98 | 18.58 | 371.23 | 1.08 | 9.80 | 36.26 | 0.27 | OK |
| Strength I-L | –75 | 2,087 | 2,734 | 0.03 | 0.76 | 19.94 | 18.47 | 368.46 | 1.08 | 7.42 | 36.18 | 0.21 | OK |
| Strength II-U | 201 | 2,003 | 3,825 | 0.05 | 0.52 | 19.90 | 18.95 | 377.06 | 1.05 | 10.15 | 36.54 | 0.28 | OK |
| Strength II-L | 162 | 1,508 | 2,922 | 0.06 | 0.52 | 19.89 | 18.97 | 377.24 | 1.05 | 7.75 | 36.56 | 0.21 | OK |
| Strength III-U | 117 | 1,434 | 2,799 | 0.04 | 0.51 | 19.92 | 18.98 | 377.93 | 1.05 | 7.41 | 36.56 | 0.20 | OK |
| Strength III-L | 79 | 939 | 1,896 | 0.04 | 0.50 | 19.92 | 19.01 | 378.62 | 1.05 | 5.01 | 36.58 | 0.14 | OK |
| Strength V-U | –2 | 2,319 | 3,446 | 0.00 | 0.67 | 20.00 | 18.65 | 373.06 | 1.07 | 9.24 | 36.31 | 0.25 | OK |
| Strength V-L | –40 | 1,824 | 2,543 | 0.02 | 0.72 | 19.97 | 18.57 | 370.72 | 1.08 | 6.86 | 36.25 | 0.19 | OK |
| Service I | –3 | 1,754 | 2,219* | 0.00 | 0.79 | 20.00 | 18.42 | 368.33 | 1.09 | 6.02 | 5.11 | 1.18 | NG |
| Extreme Event I | 22,126 | 0 | 3,164 | 6.99 | 0.00 | 6.01 | 20.00 | 120.29 | 3.32 | 26.30 | 40.43** | 0.65 | OK |

* $P_{net}$ (kip)
** *L'/B'* is out of range; therefore, factored nominal bearing resistance was calculated by extrapolation. For practical design purposes, the SD needs to ask the Geoprofessional to provide adequate (revised) data to cover all applicable cases without extrapolating.

In Table 10.1.8-8:

$L'_L$, $L'_T$ = 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 a net for service ($q_{net,u}$) and a gross for Strength and Extreme Event limits ($q_{g,u}$) (ksf)

$q_{p,net}$ = 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 (0.66 ft) is acceptable. Under Extreme Event, the calculated eccentricity of 6.99 ft is larger than the permissible eccentricity of $B/3 = L/3 = 20/3 = 6.67$ 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 Geoprofessional. Therefore, the size of the footing is increased to 24 ft × 24 ft and stresses are recalculated, as shown in Table 10.1.8-9.

### Table 10.1.8-9 Detailed Check for Footing Size (Second Trial)

| Load Combination | $M_T$ (kip-ft) | $M_L$ (kip-ft) | $P_{gross}$ (kip) | $e_T$ (ft) | $e_L$ (ft) | $L'_T$ (ft) | $L'_L$ (ft) | $A_e$ (ft$^2$) | $L'/B'$ | $q_0$ (ksf) | $q_{p,net}$ or $q_R$ (ksf) | $q_0/q_{p,net}$ or $q_0/q_R$ Ratio | Check |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Strength I-U | –37 | 2,582 | 3,912 | 0.01 | 0.66 | 23.98 | 22.68 | 543.89 | 1.06 | 7.19 | 38.83 | 0.19 | OK |
| Strength I-L | –75 | 2,087 | 2,924 | 0.03 | 0.71 | 23.95 | 22.57 | 540.59 | 1.06 | 5.41 | 38.76 | 0.14 | OK |
| Strength II-U | 201 | 2,003 | 4,100 | 0.05 | 0.49 | 23.90 | 23.02 | 550.29 | 1.04 | 7.45 | 39.06 | 0.19 | OK |
| Strength II-L | 162 | 1,508 | 3,112 | 0.05 | 0.48 | 23.90 | 23.03 | 550.33 | 1.04 | 5.65 | 39.07 | 0.14 | OK |
| Strength III-U | 117 | 1,434 | 3,074 | 0.04 | 0.47 | 23.92 | 23.07 | 551.86 | 1.04 | 5.57 | 39.09 | 0.14 | OK |
| Strength III-L | 79 | 939 | 2,086 | 0.04 | 0.45 | 23.92 | 23.10 | 552.66 | 1.04 | 3.77 | 39.11 | 0.10 | OK |
| Strength V-U | –2 | 2,319 | 3,721 | 0.00 | 0.62 | 24.00 | 22.75 | 546.06 | 1.05 | 6.81 | 38.88 | 0.18 | OK |
| Strength V-L | –40 | 1,824 | 2,733 | 0.01 | 0.67 | 23.97 | 22.66 | 543.29 | 1.06 | 5.03 | 38.82 | 0.13 | OK |
| Service I | 3 | 1,754 | 2,240* | 0.00 | 0.78 | 24.00 | 22.43 | 538.36 | 1.07 | 4.16 | 4.25 | 0.98 | OK |
| Extreme Event I | 22,124 | 0 | 3,375 | 6.56 | 0.00 | 10.89 | 24.00 | 261.35 | 2.20 | 12.91 | 63.1** | 0.20 | OK |

* $P_{net}$ (kip)
** *L'/B'* is out of range; therefore, factored nominal bearing resistance was calculated by extrapolation. For practical design purposes, the SD needs to ask the Geoprofessional to provide adequate (revised) data to cover all applicable cases without extrapolating.

Table 10.1.8-9 shows that a 24 ft × 24 ft footing size satisfies stress requirements. Furthermore, the calculated eccentricities under service and extreme event limit states (0.61 ft and 6.55 ft, respectively) are smaller than the limits (4 ft and 8 ft, respectively).

The ratio of the length of footing from column's face to face of the footing to the thickness of footing, $L_{ftg}$ / $D_{ftg}$ = (0.5)(24-6) / 4 = 2.25, which is slightly over the limit of 2.2 required by SDC 6.2.2.4 (Caltrans 2019b) for rigidity of the footing. In addition, the ratio of the thickness of the footing to the diameter of the circular column, $D_{ftg}$ / $D_c$ = 4/6 = 0.67, is slightly lower than the limit of 0.7 required by SDC 7.6.2 (Caltrans 2019b). In both cases, the SDC limitations are mostly applicable to pile caps, and they are 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:

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

Note: terms and abbreviations provided under section 10.1.6.4

The factored resistance against sliding failure for cast-in-place concrete on sand is calculated using $\varphi_\tau$ = 0.8 for the strength limit state (AASHTO-CA BDS-8 Table 10.5.5.2.2-1) and $\varphi_\tau$ = 1.0 for the extreme event limit state.

Table 10.1.8-10 shows that the requirements of Article 10.6.3.4 for sliding failure are met. Therefore, a footing size of 24 ft × 24 ft is acceptable and will be used throughout this example.

**Table 10.1.8-10 Sliding Check for Footing**

| Load Combination | Factored Resultant Shear, $V_{Total}$ (kip) | Factored Vertical Load (kip) | $R_R$ (kip) | Check |
|---|---|---|---|---|
| Strength I-U | 95 | 3,912 | 2,445 | OK |
| Strength I-L | 74 | 2,957 | 1,848 | OK |
| Strength II-U | 74 | 4,100 | 2,563 | OK |
| Strength II-L | 53 | 3,145 | 1,966 | OK |
| Strength III-U | 50 | 3,074 | 1,921 | OK |
| Strength III-L | 28 | 2,119 | 1,324 | OK |
| Strength V-U | 85 | 3,721 | 2,325 | OK |
| Strength V-L | 63 | 2,766 | 1,729 | OK |
| Extreme Event I | 859 | 3,375 | 2,637 | OK |

### 10.1.8.6 Communicate with Geotechnical Designer

Upon finalizing the footing size, the “Foundation Design Data Sheets” below are completed and forwarded to the Geotechnical Services (GS) to be used for the preparation of “Foundation Design Recommendations”.

**Preliminary Foundation Design Data Sheet (Trial Footing Size)**

**Table 10.1.8-11 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 | 20 | 20 | 1 | 0.78 |
| Abut 3 | | | | | 1 or 2 | - |

**Foundation Design Data Sheet (Final Footing Size)**

**Table 10.1.8.12 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 10.1.8.13 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 | | | | | | | | | | |
| Bent 2 | 2,240 | 1,754 | 3 | - | - | 1,761 | 1,098 | 91 | - | - |
| Abut 3 | | | | | | | | | | |

**Table 10.1.8.14 LRFD Strength/Construction and Extreme Event Limit States Load Data**

| Support No. | Strength Limit State (Controlling Group) | | | | | Extreme Event Limit State (Controlling Group) | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | $P_{Total}$ (kip) Gross | $M_x$ (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 | | | | | | | | | | |
| Bent 2 | 4,100 | 2,003 | 201 | - | - | 3,375 | 0 | 22,124 | 859 | 0 |
| Abut 3 | | | | | | | | | | |

Note: Since this design example is for bent design, information on abutments is not shown.

## 10.1.8.7 Flexural Design

For the structural design of the footing, the distribution of contact stresses is assumed to be linear (trapezoidal or triangular) irrespective of the substrate stiffness (resting on soil or rock) per AASHTO Article 10.6.5 (AASHTO, 2017). 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 per AASHTO Article 11.6.3.2 (AASHTO, 2017).

Forces acting at the bottom of the footing of selected service and strength limit states for Case 3 used to demonstrate the design process:

Service I: $P_{gross}$ = 2,862 kip; $M_L$ = 1,754 kip-ft; $M_T$ = 3 kip-ft

Strength I-U: $P_{gross}$ = 3,912 kip; $M_L$ = 2,582 kip-ft; $M_T$ = –37 kip-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: $q_1$ = 7.91 ksf; $q_2$ = 5.67 ksf

  $T$ Direction: $q_1$ = 6.78 ksf; $q_2$ = 6.81 ksf

- Service Limit State:

  $L$ Direction: $q_1$ = 5.73 ksf; $q_2$ = 4.21 ksf

  $T$ Direction: $q_1$ = 4.97 ksf; $q_2$ = 4.97 ksf

Since the column has a circular cross section, it is transformed into an effective square section for footing analysis with an equivalent column width of $\sqrt{28.27} = 5.32$ ft .

Factored Flexural Resistance

Assuming #5 ($d_b$ = 0.625 in.) and #9 ($d_b$ = 1.128 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 44.06 in. and 43.31 in., respectively.

Critical sections for moment and shear calculations:

- Bending moment at the face of the column (AASHTO 5.12.8.4)
- One-way shear at distance "$d_v$" from the face of the column (AASHTO 5.7.3.2)
- Two-way (punching) shear on the perimeter of a surface located at a distance "$d_{v,avg}$" from the face of the column (AASHTO 5.12.8.6)

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: $M_L$ = 264.50 kip-ft; $M_T$ = 228.80 kip-ft
- Service Limit State: $M_L$ = 189.03 kip-ft; $M_T$ = 164.50 kip-ft

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

$$s = [24(12) - 2(3) - 1.128] / (42-1) = 6.85 \text{ in.}$$

Per AASHTO Article 5.10.3.2, the maximum spacing of reinforcement bars shall not be greater than the lesser of the following:

- 1.5 times the thickness of the member = 1.5(4 × 12) = 72 in.; or
- 18.0 in.

The calculated spacing is less than the specified maximum spacing of 18 in.

In addition, according to AASHTO Article 5.10.3.1, the clear distance between parallel bars in a layer shall not be less than the largest of the following:

- 1.5 times the nominal diameter of the bars = 1.5(1.128) = 1.69 in.;

- 1.5 times the maximum size of the coarse aggregate (1 in. is used here) = 1.5(1.0) = 1.5 in.; or
- 1.5 in.

The computed spacing of 6.85 in. is larger than the specified minimum spacing of 2.82 in..

The area of steel contributing to the unit width of the footing is:

$$A_s = (1.0)\left(\frac{12}{6.85}\right) = 1.75 \text{ in.}^2$$

Per article 5.6.2.2, the coefficient, $\beta_1$, is taken as 0.85 for $f'_c$ = 3.6 ksi and $\alpha_1$ is 0.85. Neglecting compression steel, the depth of the concrete stress block and resisting moment are calculated as:

$$a = \frac{A_s f_y}{\alpha_1 f'_c b} = \frac{(1.75)(60)}{(0.85)(3.6)(12)} = 2.86 \text{ in.}$$

The corresponding depth of the neutral axis will be $c = a/\beta_1$ = (2.86) / (0.85) = 3.36 in., and the net tensile strain in extreme tension steel reinforcement is calculated as:

$$\varepsilon_s = \frac{0.003(d_e - c)}{c} = \frac{(0.003)(43.31 - 3.36)}{3.36} = 0.0356$$

Since the calculated strain $\varepsilon_s$ is larger than 0.005, the section is considered tension-controlled, and a resistance factor $\phi$ is 0.9 (AASHTO-CA BDS 5.5.4.2 & 5.6.2.1). The factored flexural resistance is calculated as:

$$M_r = \phi M_n = \phi\left(A_s f_y\right)\left(d_e - \frac{a}{2}\right) = (0.9)(1.75)(60)\left(43.31 - \frac{2.86}{2}\right)$$

= 3957.66 kip-in = 329.81 kip-ft > $M_u$ = 264.5 kip-ft

Therefore, a selected number of bars is adequate for strength in both directions.

Minimum Reinforcement

AASHTO Article 5.6.3.3 requires a minimum amount of reinforcement to be provided for crack control. The factored flexural resistance $M_r$ is required to be at least equal to the smaller of $M_{cr}$ and 1.33 $M_u$ as follows (gross section properties are used instead of transformed sections):

Modulus of rupture: $f_r = 0.24\sqrt{f'_c} = 0.24\sqrt{3.6} = 0.455$ ksi (AASHTO 5.4.2.6)

Gross section modulus: $S_c = S_{nc} = \dfrac{(12)(48)^2}{6} = 4608 \text{ in.}^3$

Flexural cracking variability factor: $\gamma_1$ = 1.6 for all concrete structures except precast segmental structures per Article 5.6.3.3.

The ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement: $\gamma_3$ = 0.75 for A706, Grade 60 reinforcement per Article 5.6.3.3. The calculations are as follows:

$M_{cr} = \gamma_3 \gamma_1 f_r S_c$ (AASHTO 5.6.3.3-1)

= (0.75)(1.6)(0.455)(4608) = 2518 kip-in. = 209.83 kip-ft

1.33 $M_u$ = 1.33 (264.5) = 351.79 kip-ft

$$M_r = \phi M_n = 328.34 \text{ kip-ft} > \text{smaller of} \begin{Bmatrix} M_{cr} = 209.83 \\ 1.33 M_u = 351.79 \end{Bmatrix} = 209.83 \text{ kip-ft} \quad \text{OK}$$

(AASHTO 5.6.3.3)

<u>Crack Control</u>

AASHTO Article 5.6.7 requires maximum limits of rebar spacing for crack control.

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$$ (AASHTO 5.6.7-1)

Assuming exposure factor $\gamma_e$ is equal to 1 (class-I exposure), and

$d_c$ = 3+(1.128 / 2) = 3.564 in.

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)} = 1 + \frac{3.564}{(0.7)(48 - 3.564)} = 1.115$$ (AASHTO 5.6.7-2)

The cracked concrete section is used to calculate tensile stress in steel reinforcement under service loads, and the moment of inertia for unit width (12 in.) of the transformed section (based on concrete), $I_{tr}$, is calculated as follows:

$$E_c = 33{,}000 K_1 \gamma_c^{1.5} \sqrt{f_c'}$$ (AASHTO C5.4.2.4-2)

$$= (33{,}000)(1.0)(0.15)^{1.5}\sqrt{3.6} = 3637.50 \text{ ksi}$$

$$n = \frac{E_s}{E_c} = \frac{29{,}000}{3{,}637.50} = 7.97 \qquad \text{(AASHTO 5.6.1)}$$

Usually, $n$ is rounded to the nearest integer number. Therefore, $n$ = 8 will be used.

$$\rho = \frac{A_s}{bd_e} = \frac{1.75}{(12)(43.31)} = 0.0034$$

$$k = \sqrt{(\rho n)^2 + 2(\rho n)} - \rho n$$
$$= \sqrt{((0.0034)(8))^2 + 2(0.0034)(8)} - (0.0034)(8) \;\; = 0.207$$

$k_{de} = k\, d_e = (0.207)(43.31) = 8.96$ in.

$$I_{tr} = \frac{bk_{de}^3}{3} + n A_s \left(d_e - k_{de}\right)^2$$
$$= \frac{(12)(8.96)^3}{3} + (8)(1.75)(43.31 - 8.96)^2 = 19{,}396.21 \text{ in.}^4$$

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

$$f_{ss} = n\frac{M_s\left(d_e - k_{de}\right)}{I_{tr}} = (8)\frac{(189.03)(12)(43.31 - 8.96)}{19{,}396.21} = 32.14 \text{ ksi}$$

The calculated tensile stress is less than 0.6 $f_y$ = 36 ksi OK

Then, the maximum spacing is checked as (Article 5.6.7-1):

$$s = 6.85 \text{ in.} \;\le\; \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c = \frac{700(1.0)}{(1.115)(32.14)} - 2(3.564) = 12.40 \text{ in.} \qquad \text{OK}$$

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

<u>Shrinkage and Temperature Reinforcement</u>

The shrinkage and temperature reinforcement for the top mat per unit foot width shall satisfy (AASHTO Article 5.10.6):

$$A_s > \frac{1.3bh}{2(b+h)f_y} = \frac{1.3(24\times 12)(4\times 12)}{2(24\times 12 + 4\times 12)(60)} = 0.446 \text{ in.}^2 \qquad \text{(AASHTO 5.10.6.1)}$$

$$0.11 \le A_s \le 0.6 \qquad \text{(AASHTO 5.10.6-2)}$$

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

$s$ = [ 24 (12) – 2 (3) – 0.625] / (42-1) = 6.86 in. < 12.0 in. OK

$A_s$ = (0.31) (12) / (6.86) = 0.542 in.$^2$ >0.446 in.$^2$ / ft OK

$0.11 \leq 0.542 \leq 0.6$ OK

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

## 10.1.8.8 Shear Design

According to AASHTO Article 5.12.8.6.1, 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 is located at a distance as specified in AASHTO 5.7.3.2, which 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 the effective shear depth ($d_v$):

$$d_v = d_e - \frac{a}{2} = (43.31) - \frac{(2.86)}{2} = 41.88 \text{ in.} \approx 3.5 \text{ ft}$$

Per AASHTO Article 5.7.2.8: $d_v$ need not be taken to be less than the greater of

$0.9\ d_e = 0.9\ (43.31) = 38.98$ in.

or $0.72\ h = 0.72\ (48) = 34.56$ in.

since 0.9 $d_e$ and 0.72 $h$ is smaller than $d_v$, $d_v$ = 3.5 ft will be used in calculations.

### 10.1.8.8.1 Direct (One-Way) Shear

The extreme contact stresses for the most critical strength limit state case (L direction) are 5.67 ksf and 7.91 ksf. As shown in Figure 10.1.8-7, assuming a linear stress distribution, the contact stress at a distance $d_v$ from the face of the column is calculated:

$$q_3 = 7.91 - (7.91 - 5.67)\left(12 - \frac{5.32}{2} - 3.5\right) / (24) = 7.36 \text{ ksf}$$

Shear force at the critical section for unit width conservatively (the factored weight of backfill soil on the footing and the weight of footing omitted for simplification) calculated as:

$$V_u = (7.91 + 7.36)\left(12 - \frac{5.32}{2} - 3.5\right) / (2) = 44.60 \text{ kip}$$

The maximum shear resistance of the section (considering shear reinforcement contribution) is limited to 0.25 $f'_c b_v d_v$ (AASHTO 5.7.3.3-2):

$$V_{n,max} = 0.25\ (3.6)(12)(42) = 453.60 \text{ kip} \qquad \text{(AASHTO 5.7.3.3-2)}$$

This maximum shear resistance is much higher than the factored shear force of 44.60 kip and is not governing.

The shear resistance of concrete ($V_c$) is $0.0316\beta\sqrt{f'_c}b_v d_v$, where $\beta$ = 2.0 per AASHTO Article 5.7.3.4.1:

$$V_c = 0.0316(2.0)\sqrt{3.6}(12)(42) = 60.44 \text{ kips} \qquad \text{(AASHTO 5.7.3.3-3)}$$

Assuming that no shear reinforcement will be used, $V_s$ = 0, and the resistance factor $\phi$ is 0.9 (AASHTO Article 5.5.4.2). The factored shear resistance is calculated as:

$\phi$Vn = 0.9 (60.26) = 54.23 kip > 44.60 kip OK

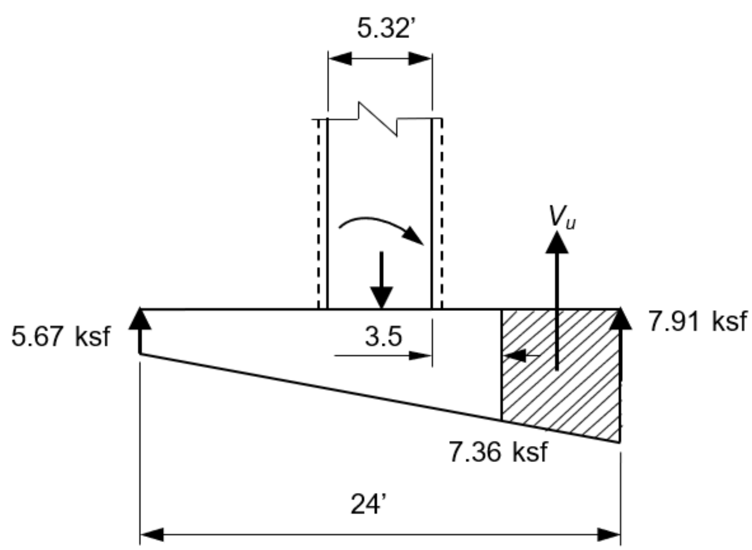



**Figure 10.1.8-7 Direct Shear Force Calculation**

### 10.1.8.8.2 Punching (Two-Way) Shear

The critical section is located at a distance of 0.5 $d_{v,avg}$. from the face of the column, as shown in Figure 10.1.8-8. Using a conservative assumption of $d_{v,avg}$ = 3.5 ft results in $b_0$ = 4(5.32 + 3.5) = 35.28 ft = 423.36 in.

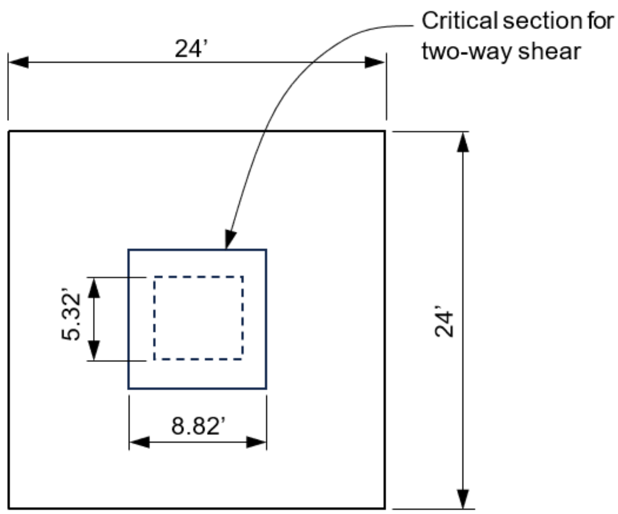



**Figure 10.1.8-8 Critical Section for Two-Way Shear**

For square footing $\beta_c$ = 1 and assuming that no shear reinforcement will be used, $V_s$ = 0. The nominal shear resistance can be calculated per AASHTO Article 5.12.8.6.3 as:

$$V_n = \left(0.063 + \frac{0.126}{\beta_c}\right)\sqrt{f'_c}\, b_0 d_v = \left(0.063 + \frac{0.126}{1.0}\right)\sqrt{3.6}(423.36)(42) = 6{,}376 \text{ kip}$$

$$\because V_n = 6{,}376 \text{kips} > 0.126\sqrt{f'_c}\, b_0 d_v = 0.126\sqrt{3.6}(423.36)(42) = 4{,}250 \text{ kip}$$

(AASHTO 5.12.8.6.3-1)

$\therefore$ Use $V_n$ = 4,250 kip

$\phi V_n$ = 0.9 (4,250) = 3825 kip

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:

$$P_{2-way} = 3{,}912 - \frac{\pi(4.75)^2}{(24)(24)} 3{,}912 = 3{,}431 \text{ kips} < \phi V_n = 3{,}825 \text{ kip} \qquad \text{OK}$$

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

Note: Although seismic loads were considered when sizing the footing, the structural design was only based on service and strength I-U (Case-3) limit states. Refer to Caltrans SDC (Caltrans, 2019b) for other design and detailing (e.g., Section 6.2.2.2) requirements.

## NOTATION

| | | |
|---|---|---|
| $A$ | = | actual footing area (ft$^2$) |
| $A'$ | = | reduced effective area of the footing (ft$^2$) |
| $A_e$ | = | effective area of the footing (ft$^2$) |
| $A_s$ | = | total area of non prestressed tension reinforcement (in.$^2$) |
| $A_v$ | = | area of transverse reinforcement within a distance s (in.$^2$) |
| $a$ | = | depth of equivalent rectangular stress block (in.) |
| $B, L$ | = | actual dimensions of the footing (ft) |
| $B', L'$ | = | effective dimensions of the footing (ft) |
| BOF | = | bottom of footing elevation (ft) |
| $b$ | = | design width (ft) |
| $b_0$ | = | the perimeter of the critical section for shear (in.) |
| $b_v$ | = | effective width of a member for shear stress calculations (in.) |
| $c$ | = | distance from the extreme compression fiber to the neutral axis (in.) |
| *clr.* | = | minimum clearance from the bottom of footing to the bottom mat of footing reinforcement (in.) |
| $D_{ftg}$ | = | depth of footing (ft) |
| $D_c$ | = | column cross-sectional dimension parallel to the direction of bending (ft) |
| $d_b$ | = | nominal diameter of bar (in.) |
| $d_c$ | = | thickness of concrete cover measured from extreme tension fiber to center of closest bar (in.) |
| $d_e$ | = | effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (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) |
| $E_c$ | = | modulus of elasticity of concrete (ksi) |

$E_s$ = modulus of elasticity of reinforcing steel (ksi)

$e$ = eccentricity (ft)

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

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

FG = finish ground elevation (ft)

$f'_c$ = specified 28-day compressive strength of unconfined concrete (ksi)

$f_r$ = modulus of rupture of concrete (ksi)

$f_{ss}$ = tensile stress in mild steel at the service limit state (ksi)

$f_y$ = nominal yield stress for A706 reinforcing steel (ksi)

$h$ = section thickness (ft)

$I$ = moment of inertia (in.$^4$)

$I_{tr}$ = moment of inertia of the transformed cross-section of a member about its centroidal axis (in.$^4$)

$k$ = ratio for a transformed section

$k_{de}$ = effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement in the transformed section (in.)

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

$L_{ftg}$ = cantilever length of the footing or pile cap measured from the face of the column to the edge of the footing (ft)

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

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

$l'_d$ = Required development length of the main column reinforcement (in.)

$M$ = moment (kip-ft)

$M_{cr}$ = cracking moment of a member's cross-section (kip-ft)

$M_n$ = nominal flexural resistance of a member's cross-section (kip-ft)

$M_r$ = factored flexural resistance of a section in bending (kip-ft)

$M_u$ = factored moment at a section (kip-ft)

$M_s$ = factored moment at a section for service limit state (kip-ft/ft)

$M_L$, $M_T$ = 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)

$M_0$ = overstrength moment of a seismic critical member (kip-ft)

$n$ = modular ratio

OG = original ground elevation (ft)

PGA = design peak ground acceleration (ft/s$^2$)

$P$ = vertical force acting at the centroid of the bottom of the footing area (kip)

$P_{gross}$ = factored axial force (kip)

$P_{net}$ = net effective load acting on the bottom of the footing (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)

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

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

$q_{p,net}$ = permissible net contact stress provided by the Geoprofessional and calculated based on a specified allowable settlement (ksf)

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

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

$R_\tau$ = nominal sliding resistance between soil and concrete (kip)

$S$ = section modulus (ft$^3$)

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

$s$ = spacing of reinforcing bars (in.)

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

$V_0$ = overstrength shear force (kip)

$V_c$ = nominal shear strength provided by concrete (kip)

$V_n$ = nominal shear strength of a section (kip)

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

$V_{Total}$ = resultant shear force (kip)

$V_p$ = plastic shear at column base (kip)

$V_u$ = component of the prestressing force in the direction of applied shear (kip)

$V_s$ = nominal shear strength provided by shear reinforcement (kip)

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

$\beta$ = factor indicating ability to diagonally cracked concrete to transmit tension and shear (AASHTO 5.7.3.4.1)

$\beta_1$ = stress block factor taken as the ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone

$\beta_c$ = ratio of the long side to the short side of the rectangle through which the concentrated load or reaction force is transmitted

$\beta_s$ = ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face

$\varepsilon_s$ = strain in the centroid of the tension reinforcement (in./in.)

$\phi$ = strength reduction factor

$\phi_f$ = internal friction angle of drained soil (degree)

$\varphi_b$ = resistance factor for bearing

$\varphi_\tau$ = resistance factor against sliding

$\gamma_1$ = flexural cracking variability factor (AASHTO 5.6.3.3)

$\gamma_3$ = ratio of specified minimum yield strength to ultimate tensile strength of the nonpretressed reinforcement (AASHTO 5.6.3.3)

$\gamma_c$ = weight of the concrete per unit volume (pcf)

$\gamma_e$ = crack control exposure factor (AASHTO 5.6.7)

$\theta$ = angle of inclination of diagonal compressive stresses as determined in Article 5.7.3.4 (degrees)

$\rho$ = ratio of the volume of reinforcement to the concrete volume confined by the reinforcement

# REFERENCES


1. AASHTO. (2017). *AASHTO LRFD Bridge Design Specifications*, 8th Edition, American Association of State Highway and Transportation Officials, Washington DC.
2. Caltrans. (2019a). *California Amendments to AASHTO LRFD Bridge Design Specifications, 8th Edition*, California Department of Transportation, Sacramento, CA.
3. Caltrans. (2019b). *Caltrans Seismic Design Criteria*, Version 2.0, California Department of Transportation, Sacramento, CA.
4. Caltrans (2022). Structure Technical Policy (STP): 10.6 *Shallow Foundations*, Transportation, Sacramento, CA.