CHAPTER 11.1 ABUTMENTS

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11.1.1 INTRODUCTION

Abutments are the substructure components at the ends of a bridge used to transfer the loads from the superstructure to foundations, support approach slabs, and retain the approach embankment. In the first part of this chapter, common types of abutments and basic aspects of abutment design according to *AASHTO LRFD Bridge Design Specifications,* 8th Edition with *California Amendments,* referred to herein as AASHTO-CA BDS-8 (AASHTO, 2017; Caltrans, 2019a) are discussed. Subsequently, a design example of the short seat (non-integral) abutment is presented to illustrate the typical design procedure.

11.1.2 TYPES OF ABUTMENTS

The most common types of abutments used for highway bridges are shown in Figure 11.1.2-1. In general, abutments can be classified as open-end and closed-end. The selection of an abutment type depends on the requirements for structural connections to the superstructure, the superstructure's horizontal movements, drainage, roadway approach, and earthquake effects.

11.1.2.1 Open-End Abutments

As shown in Figure 11.1.2-1 (a), open-end abutments are constructed with a front slope that allows an easier inspection and provides the room for future widening of the roadway or waterway. They include diaphragm and short-seat abutments, also commonly referred to as "integral" and "non-integral" abutments, respectively. Open-end abutments are the most frequently used abutments for girder bridges and are usually the most economical, adaptable, and attractive. The basic structural difference between the two types is that seat abutments permit the superstructure to move independently from the abutment, while the diaphragm abutments do not. Figure 11.1.2-2 shows the structural components of a short-seat abutment.

11.1.2.2 Closed-End Abutments

As shown in Figure 11.1.2-1 (b) and (c), closed-end abutments are constructed close to the edge of the roadway or waterway without a front embankment. They include high cantilever, strutted, rigid frame, bin, and closure abutments. These are less commonly used but suitable for bridge widening of the same kind, unusual sites, or tightly constrained urban locations. Rigid frame abutments are generally utilized with tunnel-type single-span connectors and overhead structures which permit passage through a roadway embankment. Because the structural supports are adjacent to the traffic or the waterway, these have a high initial cost and present a closed appearance to the approaching traffic.

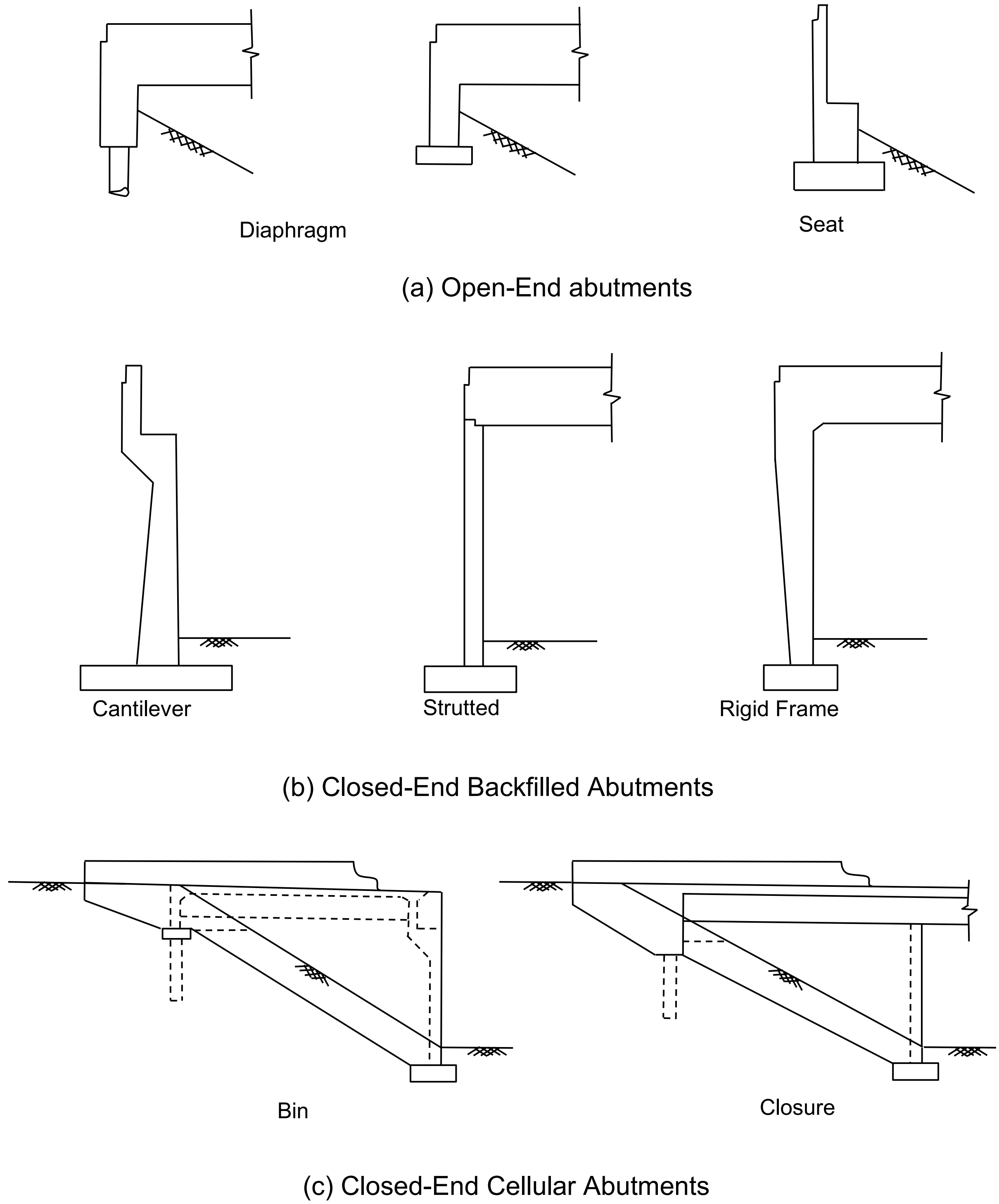

Figure 11.1.2-1 Typical Types of Abutments

The general detailing of abutments is covered in Bridge Design Details (BDD) Chapter 6.

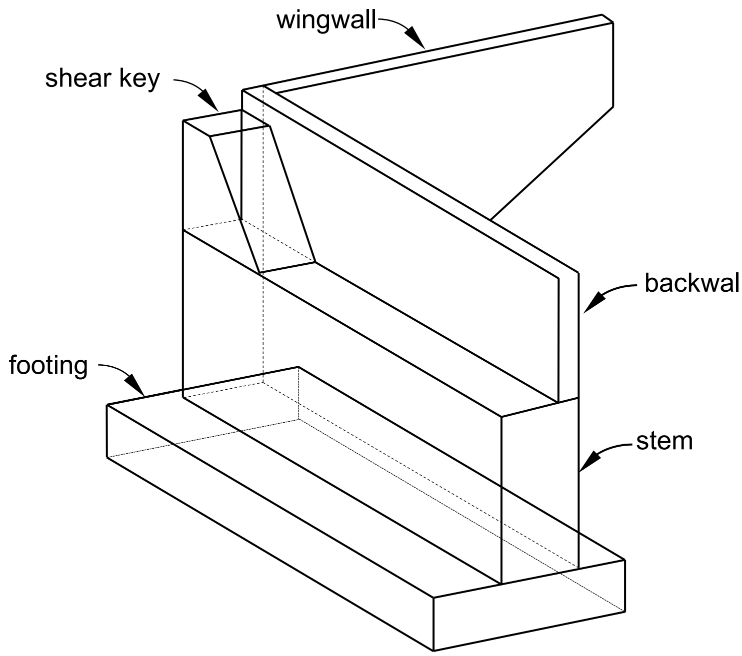

Figure 11.1.2-2 Structural Components of Short-Seat (Non-integral) Abutment

11.1.3 LOADS AND LOAD COMBINATIONS

The load factors and load combinations applicable to the abutment design are given in Tables 11.1.3-1 and 11.1.3-2 (Tables 3.4.1-1 and 3.4.5.1-1 of AASHTO-CA BDS-8). The construction load combinations have been added to carry Caltrans traditional working stress design practice, where abutments are also checked during temporary construction conditions. Construction load combinations, including Construction-I and Construction-II cases according to Article 3.4.1 (AASHTO-CA BDS-8), are shown in Table 11.1.3-2.

Furthermore, sacrificial components of abutments such as backwalls and shear keys shall be designed in accordance with SDC and AASHTO-CA BDS-8 requirements. Refer to Article 3.4.5.2 for height limitations associated with this provision. The dynamic allowance (*IM*) of the live load is disregarded for non-integral abutments refer to Article 3.6.2.1 (AASHTO-CA BDS-8).

Table 11.1.3-1 Service Limit State Load Factors for Abutments

Combination	$DC_{Sup.}$	$DC_{Sub.}$	*DD, DW*	*EH, ES_H*	*EV, ES_V*	LL_{HL93}, *IM, CE, BR, PL, LS*	LL_{Permit}, *IM, CE*	*WA*	*WS*	*WL*	*TU*	*PS, CR, SH*
Service-I	1.0	1.0	1.0	1.0	1.0	1.0	0	1.0	0.3	1.0	1.0	1.0

Table 11.1.3-2 Strength and Construction Load Factors for Abutments

Combination	$DC_{Sup.}$	$DC_{Sub.}$	DD	DW	EH ES_H EV ES_V	LL_{HL93} IM CE BR PL LS	LL_{Permit} IM CE	WA	WS	WL	TU	PS CR SH
Strength I	γ_p	γ_p	γ_p	γ_p	γ_p	1.75	0	1.00	0	0	1.00	1.00
Strength II	γ_p	γ_p	γ_p	γ_p	γ_p	0	1.35	1.00	0	0	1.00	1.00
Strength III	γ_p	γ_p	γ_p	γ_p	γ_p	0	0	1.00	1.00	0	1.00	1.00
Strength V	γ_p	γ_p	γ_p	γ_p	γ_p	1.35	0	1.00	1.00	1.00	1.00	1.00
Construction I	0	γ_p	0	0	γ_p	0	0	0	0	0	0	0
Construction II	1.25	1.25	0	1.50	0	0	0	0	0	0	1.00	1.00

Note: For γ_p values of abutments, refer to Table 11.1.3-2a.

Table 11.1.3-2a Load Factors for Permanent Loads, γ_p

Type of Load and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC_{Sub}: Dead Load of Structure Components and Nonstructural Attachments of Substructure		1.25	0.90
DC_{Sup}: Dead Load of Structure Components and Nonstructural Attachments of Superstructure		1.25	0.90
DD: Downdrag	Pile, α Tomlinson Method	1.40	0.25
	Pile, λ Method	1.05	0.30
	Drilled Shaft, O'Neill and Reese (2010) Method	1.25	0.35
DW: Dead load of Wearing Surface and Utilities		1.50	0.65
EH: Active Horizontal Earth Pressure		1.50	0.75
ES_H: Earth Surcharge Horizontal Load		1.50	0.75
ES_V: Earth Surcharge Vertical Load		1.35	1.00
EV: Vertical Earth Pressure		1.35	1.00

11.1.3.1 Dead Load (DC_{sup}, DC_{sub}, DW)

The component and wearing surface dead loads transferred from the superstructure (DC_{sup} and DW) to the abutment are support reactions commonly obtained from superstructure analysis software such as CTBridge or CSiBridge. The weight of the abutment components (backwalls, shear keys, stems, and footings), DC_{sub}, can be easily calculated from the geometry of the abutment. The weight of the abutment components, including shear keys, is assumed to be uniformly distributed along the abutment width. In the case of non-integral abutments, only reaction forces (vertical and horizontal) are applied to the abutments. However, moments are also transferred from the superstructure to the abutment for integral abutments. Figure 11.1.3-1 shows typical forces acting on a seat-type abutment.

11.1.3.2 Live Load (LL_{HL93}, LL_{Permit}, LS)

Live load (LL_{HL93} and LL_{Permit}) forces are typically obtained from superstructure analysis software and are calculated as support reactions for a single lane of a truck without any dynamic allowance (impact) for substructure analysis. There are several methods to calculate the number of live load lanes needed for abutment design. Section 11.1.6 Design Example illustrates more details on the calculation of the number of live load lanes.

The equivalent height of the soil (surcharge) used for the traffic load on the embankment (Live Load Surcharge, *LS*) varies from 2 to 4 feet depending on the height of the abutment, as discussed in Article 3.11.6.4. Caltrans practice is to apply the live-load surcharge on the approach fill regardless of the presence of an approach slab.

The traffic live load acting on the backwall is only used to calculate the maximum force effects in the backwall. This force is not considered in the analysis of other abutment components (stem, foundations) since it is already included in the live load reaction forces transferred from the superstructure.

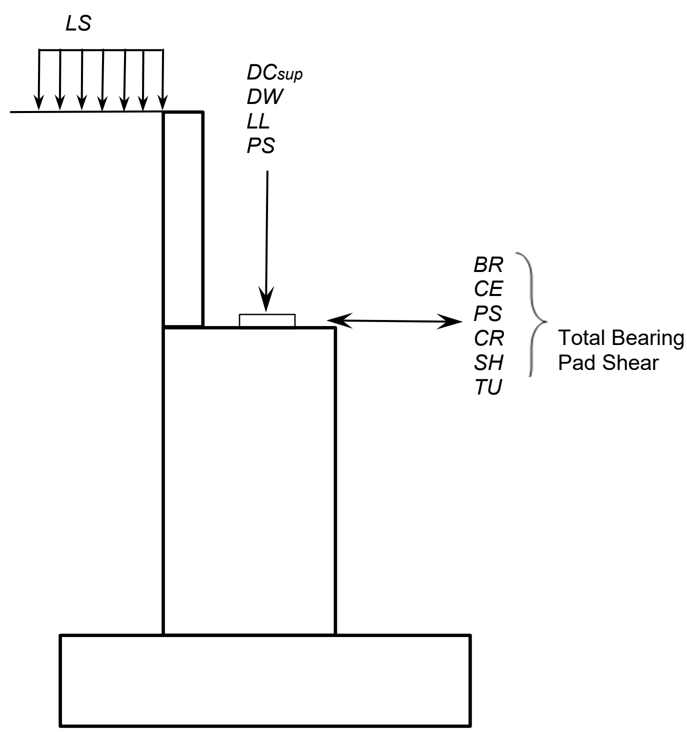

Figure 11.1.3-1 Typical Loads Acting on A Seat Type Abutment

11.1.3.3 Horizontal Loads from Superstructure (*BR*, *CE*, *WS*, *WL*, *TU*, *PS*, *CR*, *SH*)

For the seat type abutments with elastomeric bearing pads, the total amount of horizontal load that may be transmitted through the bearings before slippage occurs is limited to 0.2 (DC_{Sup} + DW) according to Article 3.4.5 (AASHTO-CA BDS-8). This force should be applied in both directions (toward and away from the backwall) horizontally.

11.1.3.4 Earth Pressure Components (*EH, EV,* ES_H*,* ES_V)

For a cantilever abutment, the overturning forces should be balanced by the vertical earth load on the abutment heel and the self-weight of the abutment. The AASHTO-CA BDS-8 Commentary C3.11.1 provides guidance on the selection of appropriate earth pressure coefficients based on the relative movement of the abutment and the retained soil.

The passive earth pressure resistance exerted by the fill in front of the abutment is usually neglected in the design due to the potential for erosion, scour, or future excavation in front of the abutment. Furthermore, a larger relative movement is needed to activate the passive pressure. The vertical load from the toe backfill should be included in the analysis for overturning if it increases overturning. Figure 11.1.3-2 shows earth pressure components acting on a typical abutment.

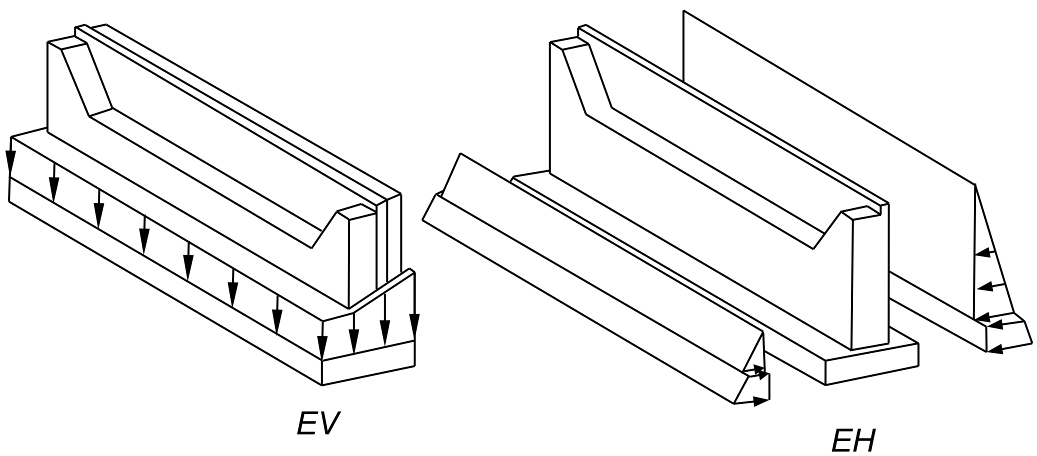

Figure 11.1.3-2 Earth Pressure Components Acting on the Abutment

11.1.3.5 Seismic Effects

Extreme Event-I (Earthquake load combination) is not considered in the design of short seat-type abutments founded in S1 (competent) soil according to AASHTO-CA BDS 3.4.5.2. However, the global stability analysis of abutments built in the S1 (competent) soil is not exempt from Extreme Event-I (Earthquake) load combination. Design requirements for individual components, such as shear keys and the seat width, are still governed by the applicable sections of the SDC 6.3, and Section 20 of the Structure

Technical Policies. According to SDC 6.3.4, the shear key is limited by shear capacity of the foundation. Therefore, the shear capacity of piles under the Extreme (Earthquake) Event load combination needs to be calculated.

For abutments in the S2 (non-competent) soil, special design provisions are required. Such provisions should be discussed in the Type Selection meeting.

11.1.3.6 Construction Load Cases

Construction load combinations for bridge abutment design are adopted from past practices of Caltrans working stress design and are shown in Figure 11.1.3-3. The Construction I load combination includes the dead load of the substructure and earth load with the surcharge calculated per AASHTO Table 3.11.6.4-1, but no superstructure loads. Construction II load combination includes the dead load of substructure and superstructure loads without wind, live load, and earth pressure components.

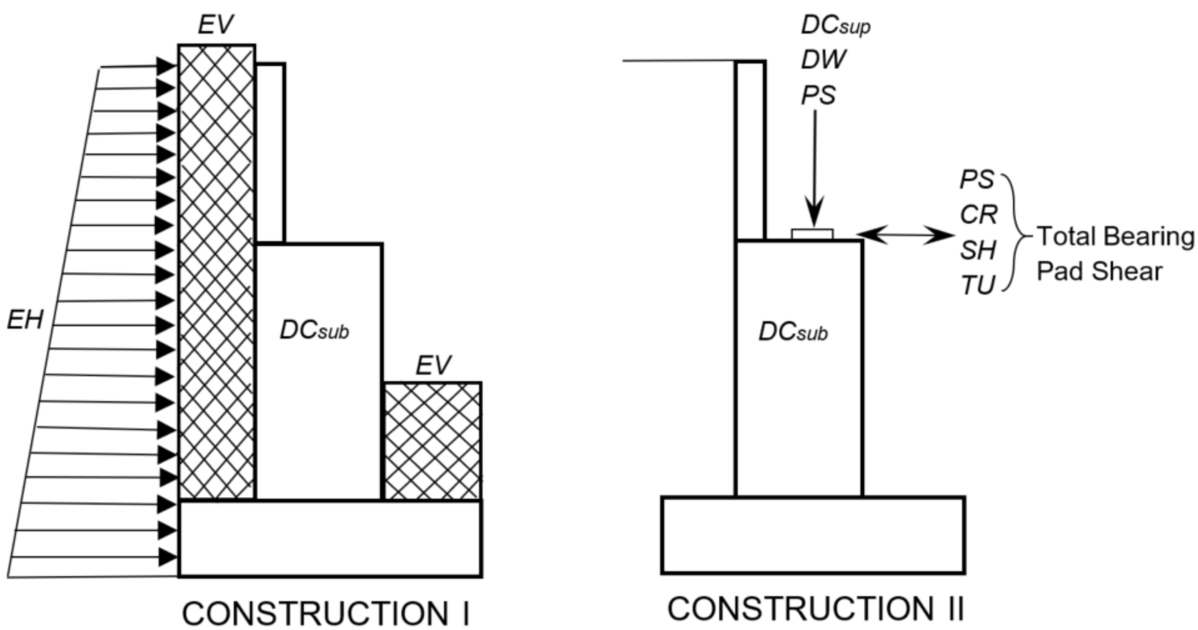

Figure 11.1.3-3 Construction Load Combinations

11.1.4 STRUCTURAL COMPONENT DESIGN CONSIDERATION

The structural design of abutment components shall be in accordance with the requirements of Sections 5 and 6 of AASHTO-CA BDS-8. The shear keys should be designed following SDC 6.3.4 requirements. The moment caused by eccentricity of the loads in the transverse direction may be neglected in the analysis and design of seat-type abutments; however, the bending moment in the transverse direction may need attention for narrow bridges. For bridges on straight alignments with an abutment skew exceeding

60 degrees and for horizontally curved bridges with an abutment skew exceeding 45 degrees, A refined analysis should be performed to capture live load distributions more accurately.

11.1.5 FOUNDATION DESIGN CONSIDERATION

The foundation design process includes sizing the spread footing and tipping the pile foundations for tension, compression, settlement, and lateral.

11.1.5.1 Size Spread Footing

Sizing a spread footing is an iterative process, as nominal bearing resistance and permissible net contact stress depend on the effective size of the footing. Therefore, the geotechnical designer (GD) needs to present the permissible net contact stresses as a group of curves for different ratios of (effective length)/ (effective width). For each load combination, the permissible stress is extracted from the curves knowing the calculated effective width and the effective length.

11.1.5.2 Determine Pile Tip Elevations

GD provides design tip elevations for compression, tension, settlement, and lateral. The first two tip elevations are commonly calculated for Strength/Construction Limit States and the settlement tip for the Service-I Limit State. For abutments in class S1 soil supported on a single row of piles, a lateral tip is calculated by the Structural Designer (SD) considering stability and the critical depth of the pile. The lowest design tip is selected as the specified tip elevation and is used in the design.

11.1.5.3 Design Piles for Shear

Shear forces in the piles need to be checked for Service and Strength/Construction Limit States. Under Service-I loads, displacement of the pile at a cut-off elevation is usually limited to 0.25 inches. The permissible (allowable) horizontal load for a single standard plan pile is the shear force at the cut-off elevation at 0.25 in. of the horizontal displacement of the pile cap. Shear force developed in the pile foundation under Service-I loads shall be less than the permissible horizontal load of the foundation. Group reduction effects shall be considered in the calculation of the permissible horizontal load. When non-standard piles are used in abutments, the permissible horizontal load is determined by geotechnical analysis and considering the structural adequacy of the pile under factored Strength/Construction loads. The horizontal component of a battered pile's axial force may be subtracted from the lateral load to determine the applied horizontal load on the pile foundations.

The factored shear resistance of the pile foundation shall be compared to the factored shear force (Strength and Construction) applied to the foundation. The shear resistance of the foundation system is developed by the resistance of all the piles considering group and batter effects. The shear resistance of a single pile is the smaller of the structural shear resistance of the pile and the shear force applied at the cut-off elevation when the maximum moment in a pile reaches its factored nominal flexural resistance.

11.1.5.4 Design Piles for Tension

Piles should not undergo a sustained tension under permanent loads for the Service-I Limit State combination. If this condition occurs, the GD should be contacted to determine if the proposed foundation type is appropriate for this condition. To ensure the structure capacity, the SD always needs to check the structure capacity per AASHTO-CA BDS-8 Sections 5 and 6 requirements.

11.1.6 DESIGN EXAMPLE

The design process, including abutment live load analysis, load combinations, backwall design, stem wall design, and footing design, for both spread footing and pile foundation, are illustrated in the following example.

11.1.6.1 Abutment Data

The elevation and typical section of the three-span continuous post-tensioned concrete box girder bridge are shown in Figures 11.1.6-1 and 11.1.6-2, respectively. Abutment 1 (first abutment) is considered in this example. Soil is class S1 soil.

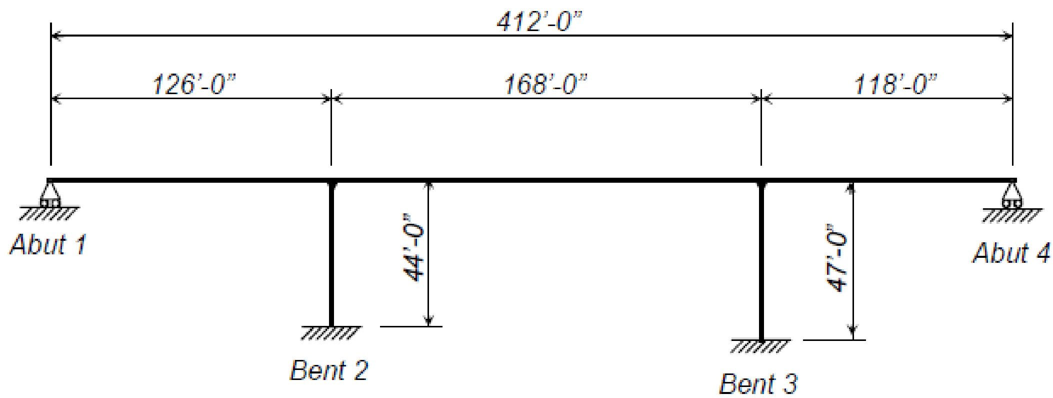

Figure 11.1.6-1 Elevation of the Example Bridge

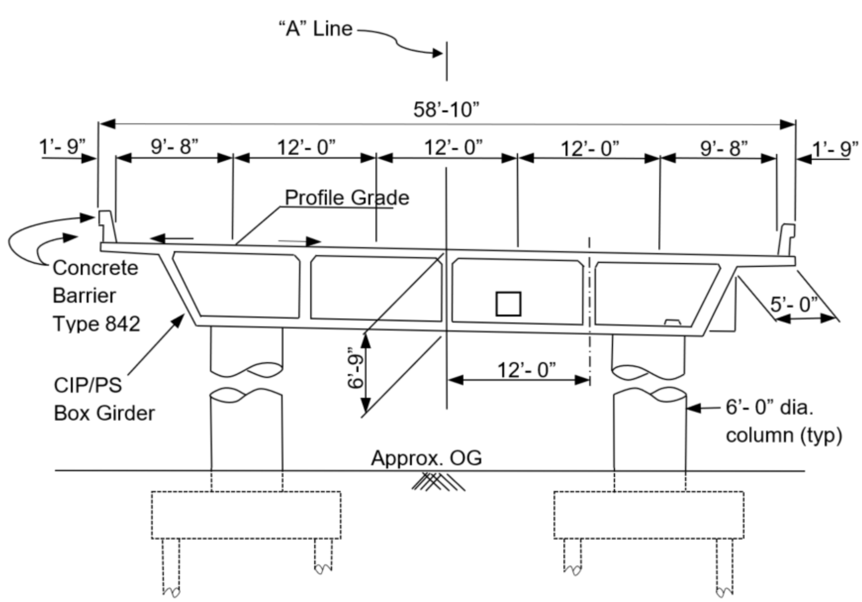

Figure 11.1.6-2 Typical Section of the Example Bridge

Plan view and typical section of the abutment are shown in Figure 11.1.6-3.

Finish Grade (FG) elevation at beginning of bridge (BB) = 16.75 ft

Average Original Grade (OG) Elevation at berm = 6.5 ft

Bottom of the footing elevation = 0 ft

Abutment height: h = 14.25 ft

Abutment length along the skew: W = 62.6 ft

Backwall height: h_{bw} = 6.75 ft

Footing thickness: d_{ftg} = 2.5 ft

Footing length: L_{ftg} = 64 ft

Footing width: B_{ftg} = 10 ft

Footing toe width (footing face to face of abutment) = 3.5 ft

Depth of soil on the toe of the footing = 4.0 ft

Depth of live load surcharge on the heel is assumed d_{LS} = 2.0 ft

Assumed edge distance (from the edge of the deck) for live load analysis = 1.0 ft (use 0 if need to consider future widening or use a width of the barrier if there is no room for future widening.)

Total shear keys weight = 28.38 kip

Barrier weight = 0.656 kip/ ft (using the Barrier Type 842 weight)

Soil slope $H:V$ = 2:1

End of wing wall depth = 3.0 ft

Wingwall and backwall thickness = 1.0 ft

Skew angle: θ_{sk} = 20°

Backfill soil unit weight: γ_s = 120 pcf

Angle of internal friction of drained backfill soil = 34°, Ka = 0.3

Pile Type – Steel Pipe 14 in. (Class 140 Alt W)

Per Standard plan B 2-5

- Compression: Nominal axial structural resistance = 280 kip
  Service state = 140 kip
- Tension: Nominal axial structural resistance = 140 kip
  Service state = 56 kip

Permissible horizontal load for vertical pile for this example = 27 kip (pile should have a minimum length of 35 ft to achieve this permissible horizontal load)

Assume permissible horizontal load for battered pile = 0.6(27 kip) = 16.2 kip

The shear resistance of the pile group under the Extreme (Earthquake Event) = 64 kip

Concrete: $f_c^{'} = 3.6\,\text{ksi}$, $\gamma_c = 150$ pcf

Reinforcing steel: f_y = 60 ksi

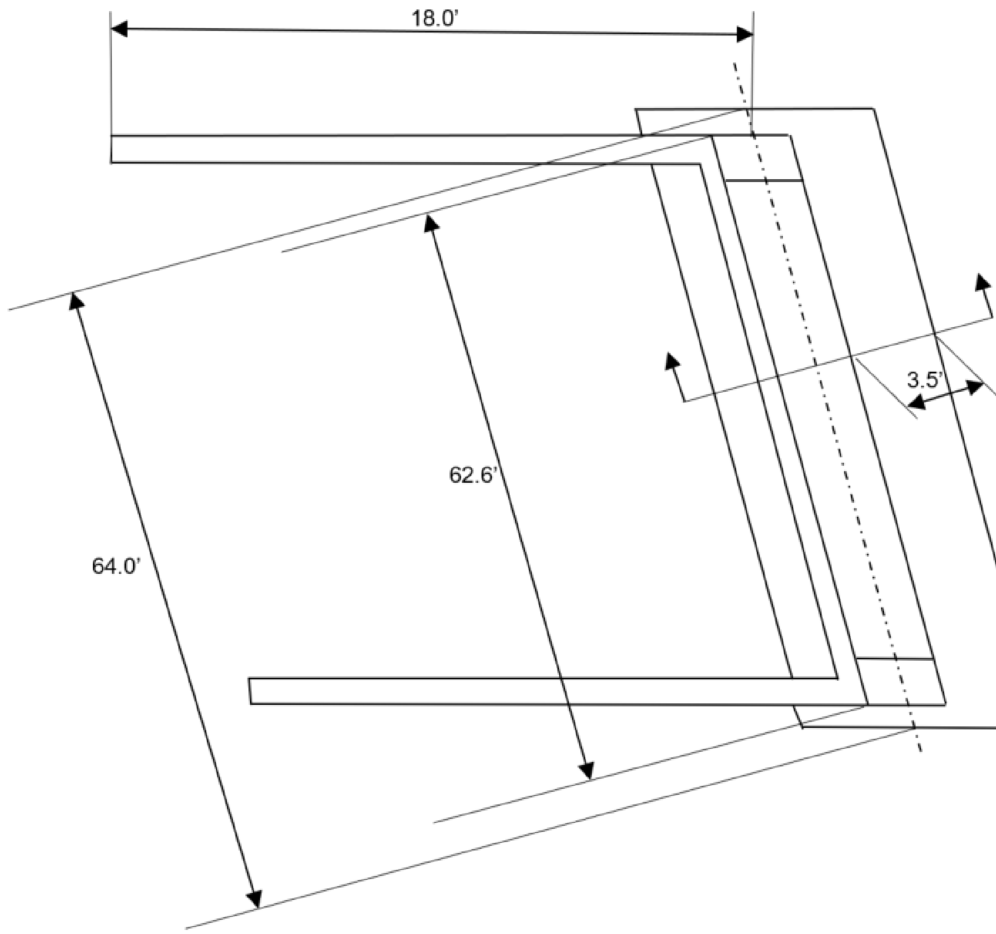

Figure 11.1-6-3a Plan View of Abutment

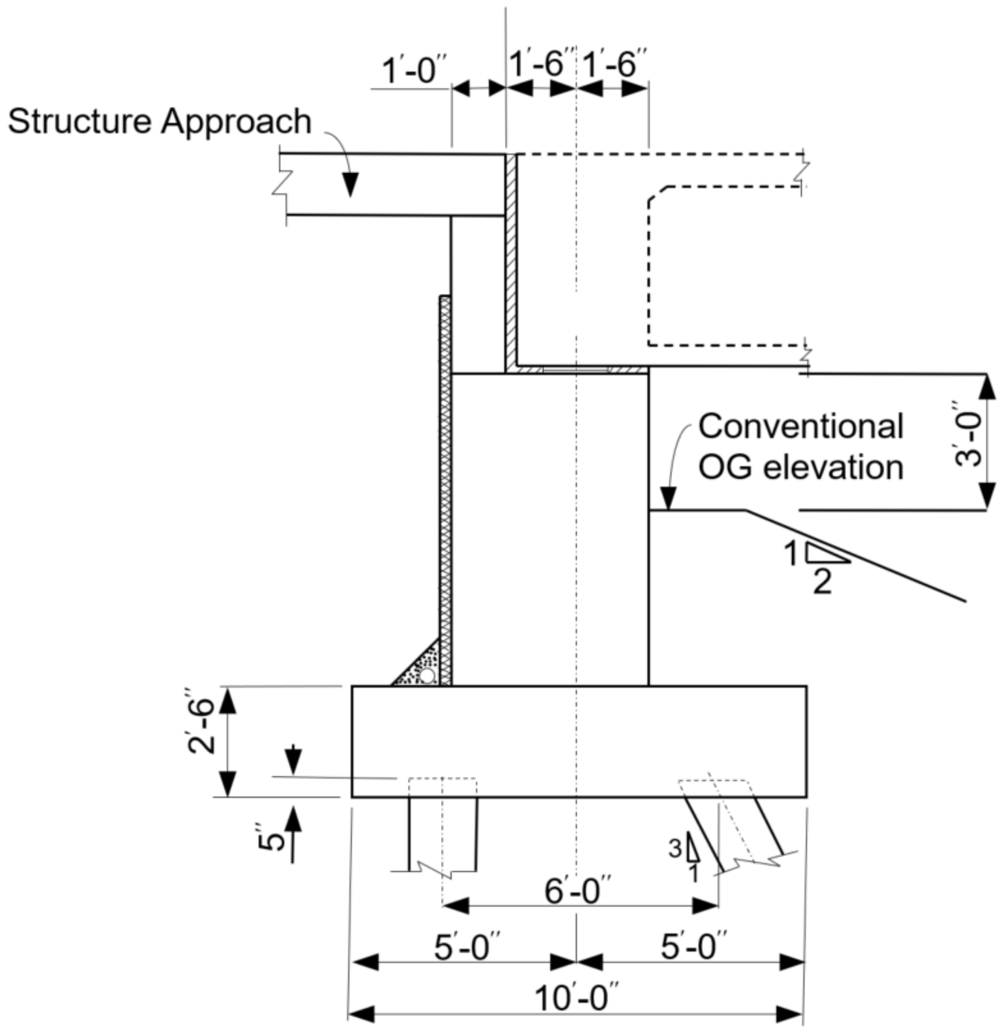

Figure 11.1-6-3b Typical Section of Abutment

11.1.6.2 Design Requirements

Perform the following design portions in accordance with AASHTO-CA BDS-8:

Calculate loads transferred from the superstructure to Abutment 1

Perform live load analysis

Calculate load combinations

Design backwall

Design stem wall

Design pile foundation

Design spread footing

11.1.6.3 Calculate Loads Transferred from Superstructure to Abutment 1

The output results of the superstructure analysis using the CTBridge program (Caltrans, 2019c) for dead load (DC_{sup}), additional dead load (DW), live load (LL_{HL93} and LL_{Permit}), and prestressing (PS) reaction forces at Abutment 1 are listed in Tables 11.1.6-1 to 11.1.6-4. The effects of other loads are neglected to simplify analysis and design procedure.

Table 11.1.6-1 – Output of CTBridge -Dead Load Reactions

Dead Load - Unfactored Abutment Reactions - Final

Abutment	AX	VY	VZ	TX	MY	MZ
	kip	kip	kip	kip·ft	kip·ft	kip·ft
First Abutment	-0.6	772.2	0.0	0.0	-0.0	0.0
Last Abutment	-0.6	-705.4	-0.0	0.0	0.0	0.0

Additional Dead Load - Unfactored Abutment Reactions

Abutment	AX	VY	VZ	TX	MY	MZ
	kip	kip	kip	kip·ft	kip·ft	kip·ft
First Abutment	-0.1	93.9	-0.0	0.0	-0.0	0.0
Last Abutment	-0.1	-85.7	0.0	0.0	0.0	0.0

Table 11.1.6-3 – Output of CTBridge -Live Load Reactions

Live Load - Controlling Unfactored Abutment Reactions

LRFD Design Vehicle

No Dynamic Load Allowance - Single Lane

Abutment	# Lanes	MZ+ kip·ft	assoc VY kip	# Lanes	MZ- kip·ft	assoc VY kip
First Abutment	1.000	0.00	98.23	1.000	-0.00	-12.48
Last Abutment	1.000	0.00	-95.73	1.000	-0.00	13.66

Abutment	# Lanes	VY+ kip	assoc MZ kip·ft	assoc TX kip·ft	# Lanes	VY- kip	assoc MZ kip·ft	assoc TX kip·ft
First Abutment	1.000	98.23	0.00	-0.00	1.000	-12.48	-0.00	0.00
Last Abutment	1.000	13.66	-0.00	-0.00	1.000	-95.73	0.00	0.00

Abutment	# Lanes	TX+ kip·ft	# Lanes	AX- kip	# Lanes	TX- kip·ft
First Abutment	1.000	0.00	1.000	-0.08	1.000	0.00
Last Abutment	1.000	0.00	1.000	-0.08	1.000	0.00

LRFD Permit Vehicle

No Dynamic Load Allowance - Single Lane

Abutment	# Lanes	MZ+ kip·ft	assoc VY kip	# Lanes	MZ- kip·ft	assoc VY kip
First Abutment	1.000	0.00	184.01	1.000	-0.00	-26.72
Last Abutment	1.000	0.00	-174.47	1.000	-0.00	29.28

Abutment	# Lanes	VY+ kip	assoc MZ kip·ft	assoc TX kip·ft	# Lanes	VY- kip	assoc MZ kip·ft	assoc TX kip·ft
First Abutment	1.000	184.01	0.00	-0.00	1.000	-26.72	-0.00	0.00
Last Abutment	1.000	29.28	-0.00	-0.00	1.000	-174.47	0.00	0.00

Abutment	# Lanes	TX+ kip·ft	# Lanes	AX- kip	# Lanes	TX- kip·ft
First Abutment	1.000	0.00	1.000	-0.15	1.000	0.00
Last Abutment	1.000	0.00	1.000	-0.15	1.000	0.00

Table 11.1.6-4 – Output of CTBridge -Prestressing Reactions

Prestress - Unfactored Abutment Reactions

Abutment	AX kip	VY kip	VZ kip	TX kip·ft	MY kip·ft	MZ kip·ft
First Abutment	-0.3	58.5	0.0	0.0	-0.0	0.0
Last Abutment	-0.2	-61.6	0.0	0.0	0.0	0.0

Unfactored loads at Abutment 1 from CTBridge output are summarized as follows:

Dead Load of Superstructure (DC_{Sup}) = 772.2 kip

Addition Dead Load (DW) = 93.9 kip

Design Vehicle (LL_{HL93}) per lane = 98.23 kip

Permit Vehicle (LL_{Permit}) per lane = 184.01 kip

Prestress Load (PS) = 58.5 kip

11.1.6.4 Perform Live Load Analysis

The CTAbut program (Caltrans, 2022) assumes a uniform distribution of live load reaction forces along the abutment in the transverse direction. However, localized effects associated with the concentration of live load reaction forces at different locations of the abutment need to be considered in the analysis. An equivalent number of live load lanes can be calculated based on 45-degree distribution from the height of the abutment wall at the deck to the top of the footing, as shown in Figure 11.1.6-4 (T. Zokaie et al., 2015). When calculating an equivalent number of live load lanes, a multiple presence factor (*MPF*) is multiplied by number of whole lanes that can be accommodated on the bridge. The equivalent number of lanes, *N,* is calculated as:

$$N = \frac{W(n_l)(MPF)}{b_n} \quad (11.1.6\text{-}1)$$

where:

W = abutment length along the skew (62.6 ft in this example)

n_l = number of whole lanes that are under consideration

b_n = effective live load distribution width at the top of the footing

MPF = multiple presence factor

Per Article 3.6.1.1.1, the maximum number of design lanes that can be placed on the bridge is determined by taking the integer part of the ratio of the clear roadway width in feet between curbs and/or barriers then divided by 12. Furthermore, roadway widths from 20 to 24 ft should have two design lanes, each equal to one-half the roadway width.

Therefore

$$n_{max} = \frac{W\cos\theta_{sk} - 2(\text{edge distance})}{12} \quad (11.1.6\text{-}2)$$

The edge distance is the width of the barrier; however, it can be assumed to be zero as the designer may need to consider future widening per Article 2.3.2.1 and use the edge of the deck (EOD) to EOD.

The effective live load distribution width at the top of the footing can be written as:

$$b_n = a_n + h\ (tan\,\theta) \leq W \tag{11.1.6-3}$$

where, a_n is the effective live load distribution width at the deck elevation (ft):

$$a_n = \frac{8 + (\text{edge distance})\ +\ 12(n-1)}{\cos\,\theta_{sk}} \tag{11.1.6-4}$$

h = abutment height (deck to top of the footing) (ft)

θ = angle of the load distribution (can be assumed 45 degrees)

The equivalent number of lanes, *N,* is calculated for different values of *n* varying from 1 to n_{max}. The maximum value of *N* is used in CTAbut to calculate design vehicular live loads.

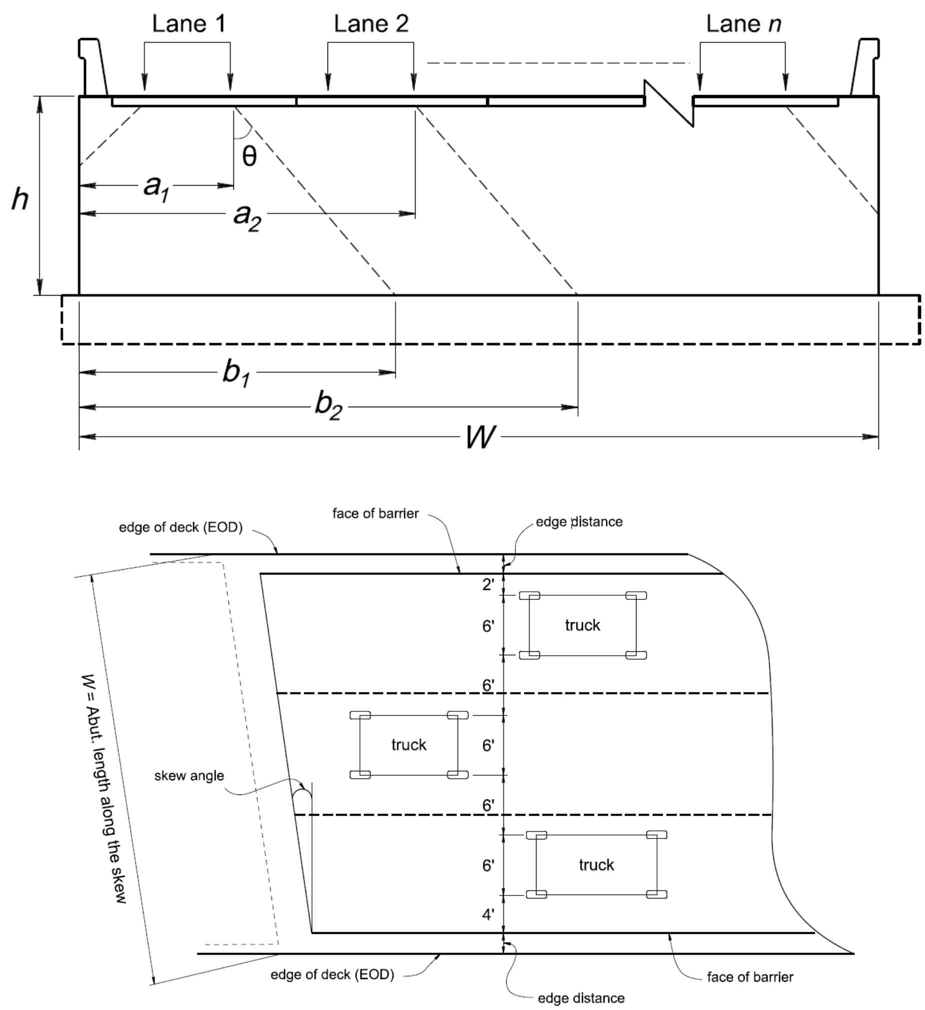

Figure 11.1.6-4 Schematics of Live Load

The maximum number of whole lanes:

$$n_{max} = \frac{W\cos\theta_{sk} - 2(\text{edge distance})}{12} = \frac{62.6\cos(20^{o}) - 2(1)}{12} = 4.74 \text{ lanes}$$

Using only the integer part: $n_{max} = 4$ lanes

Since *MPF* depends on the number of lanes, the designer needs to calculate the equivalent number of lanes for each case (one, two, three,…*n* lanes) from Equation 11.1.6-4
For example, for two live load lanes (n = 2)

$$a_2 = \frac{8 + (1)\ +\ 12(2-1)}{\cos\left(20^{\circ}\right)} = 22.35 \text{ ft}$$

Height of the abutment, h = 14.25 ft

The effective width at the top of the footing is calculated by Equation 11.1.6-3 as follows:

$$b_2 = a_2 + h\,tan\,\theta = 22.35 + (14.25)\,tan\left(45^{\circ}\right) = 36.6 \text{ ft}$$

The equivalent number of *HL* live load lanes is calculated by Equation 11.1.6-1

$$N_2 = \frac{W(n_l)(MPF)}{b_n} = \frac{(62.6)(2)(1.0)}{36.6} = 3.42 \text{ lanes}$$

Table 11.1.6-5 summarizes calculated values of a_n and b_n for different values of n, as well as the equivalent number of lanes, *N,* for HL-93 live load:

Table 11.1.6-5 Equivalent Number of HL-93 Live Load Lanes

n_l	a_n (ft)	b_n (ft)	*MPF*	*N* (lanes)
1	9.58	23.83	1.2	3.15
2	22.35	36.60	1.0	3.42
3	35.12	49.37	0.85	3.23
4	47.89	62.14	0.65	2.62

In this example, two lanes result in the largest number of equivalent lanes. Therefore, an equivalent number of live load lanes for HL-93 truck, *N,* is taken as 3.42 lanes.

The same method is used for permit trucks; however, the designer should consider only one or two lanes of P truck with *MPF* of one used for both cases.

Table 11.1.6-6 provides a summary of calculations for the number of Permit Truck live load lanes.

Table 11.1.6-6 Equivalent Number of Permit Truck Live Load Lanes

n_l	a_n (ft)	b_n (ft)	*MPF*	*N*(lanes)
1	9.58	23.83	1.0	2.63
2	22.35	36.60	1.0	3.42

The equivalent number of live load lanes for permit truck is 3.42 lanes which is calculated by placing two lanes side by side.

In summary, design live loads are calculated as:

- Design truck (LL_{HL93}) load for the abutment design:

$$LL_{HL93} = (\text{equivalent number of lanes})(\text{HL93 design vehicular load per lane})$$

$$= (3.42 \text{ lanes})(98.23 \text{ kip/lane}) = 335.95 \text{ kip}$$

- Permit truck load for abutment design:

$$LL_{Permit} = (\text{equivalent number of lanes})(\text{Permit truck load per lane})$$

$$= (3.42 \text{ lanes})(184.01 \text{ kips/lane}) = 629.31 \text{ kip}$$

11.1.6.5 Calculate Load Combinations

The permanent loads, including the weight of the different components and the soil pressure exerted on the abutment are calculated as follows. Figure 11.1.6-5 shows how the abutment is broken down into several components and also shows forces acting on those components.

Sample calculations of unfactored forces are shown as follows:

Weight of the footing, W_1:

$$W_1 = L_{ftg} B_{ftg} d_{ftg} \gamma_c = (64)(10)(2.5)(0.15) = 240.0 \text{ kip}$$

Weight of the soil behind the stem wall, V_1

$$V_1 = (16.75 - 2.5)(10 - 3.5 - 4.0)(62.6\text{-}2(1))(0.12) = 259.1 \text{ kip}$$

Active soil pressure behind the abutment, H_1:

$$H_1 = \frac{(h + d_{ftg})^2 WK_a\gamma_s}{2} = \frac{(14.25 + 2.5)^2 (62.6)(0.3)(0.12)}{2} = 316.1 \text{ kip}$$

Horizontal active live load surcharge on abutment, LS_1:

$$LS_1 = d_{LS}(h + d_{ftg})WK_a\gamma_s = (2)(14.25 + 2.5)(62.6)(0.3)(0.12) = 75.5 \text{ kip}$$

Vertical live load surcharge on the heel, $LS_{vertical}$:

$$LS_{vertical} = d_{LS}(\text{heel side width})L\gamma_s = (2)(10 - 4 - 3.5)(64)(0.12)$$

$$= 38.4 \text{ kip}$$

The equivalent bearing pad shear, V_{pad}, is assumed as 20% of (DC_{Sup} + DW).

$$V_{pad} = 0.2(DC_{sup} + DW) = 0.2(772.2 + 93.9) = 173.22 \text{ kip}$$

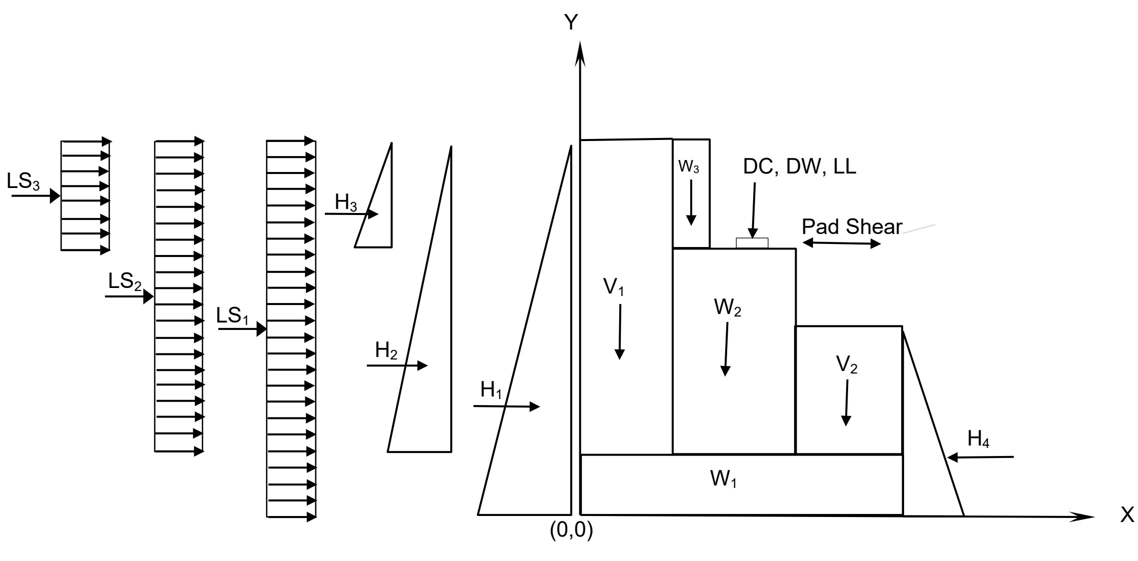

Figure 11.1.6-5a Schematics of Forces Acting to the Abutment

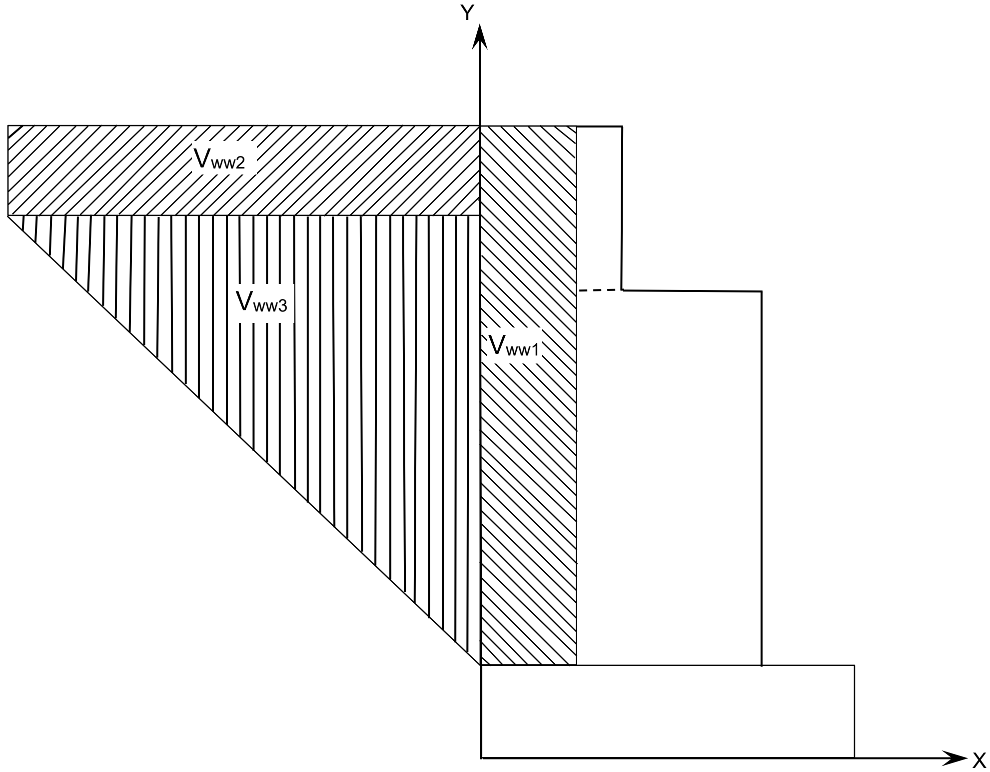

Figure 11.1.6-5b Schematics of Forces Acting to the Abutment

Table 11.1.6-7 shows the summary of unfactored forces and coordinates of their application points in the system of coordinates shown in Figure 11.1.6.5a. The moment arm is calculated with respect to the point to be used in design calculations. For example, for backwall design, the moment should be calculated about the center of the base of the backwall. Therefore, the reference point will be the centerline of the backwall at the base, and moment arms are calculated by subtracting the coordinates of the reference point from the coordinates of the point of the application. To simplify dead load calculations of the wingwall, it is divided into three parts, as shown in Figure 11.1.6-5b.

Table 11.1.6-7 Summary of Unfactored Loads and Points of Application

Unfactored Loads (kip)		Coordinates (ft)		Moment Arm (clockwise positive) (ft)		
		x	*y*	Backwall Check	Stem Check	Footing CL
W_1	240.0	5.00				0.00
W_2	281.7	4.50			0.00	-0.50
W_3	63.4	3.00		0.00	-1.50	-2.00
DC	772.2	5.00			0.50	0.00
DW	93.9	5.00			0.50	0.00
V_{pad}	173.22		10.00	0.00	7.50	10.00
PS	58.50	5.00			0.50	0.00
V_1	259.1	1.25				-3.75
V_2	107.5	8.25				3.25
V_3	6.1	2.937				-2.063
$LS_{Vertical}$	38.4	1.25				-3.75
V_{keys}	28.4	5.00			0.50	0.00
V_{ww1}	10.7	1.25			-3.25	-3.75
V_{ww2}	11.7	-6.50			-11.00	-11.50
V_{ww3}	21.9	-4.33			-8.83	-9.33
$V_{barrier}$	21.65	-4.75			-9.25	-9.75
H_1	316.1		5.58			5.58
H_2	228.8		7.25		4.75	
H_3	51.3		12.25	2.25		
H_4	0.0		2.167		1.33	2.167
LS_3	30.4		13.375	3.375		
LS_2	64.2		9.625		7.125	
LS_1	75.5		8.375			8.375

Note: $LS_{Vertical}$ is the vertical force due to the live load surcharge acting on the heel side of the footing. $V_{barrier}$ is the weight of the barrier acting at the top of the wingwall. H_4 is equal to zero because the passive pressure coefficient, K_p, is assumed to be zero in this example. Historically, the passive pressure for a short seat abutment design is ignored in Caltrans practice for several reasons. The first reason is that the quality of the backfill in front of the abutment is unknown, and the soil in this area would be eroded during the life of the bridge. The second reason is that a large movement is needed in order for passive soil to engage. The designer should consult with GD for the value of K_p if passive soil pressure needs to be considered.

The load factor for the bearing pad shear is either 0 or 1.25. The bearing pad shear should be applied in both directions horizontally to capture the worst effect for each component.

Tables 11.1.3-1 and 11.1.3-2 summarize load factors to be used for abutment design. Considering the large number of loads to be considered use of engineering judgement to identify governing cases for the design of each component is not practical. CTAbut examines all possible combinations and reports the governing cases that produce the largest Demand-to-Capacity (D/C) ratio for each check. In this example, the governing load cases have been extracted from CTAbut to show the design process.

11.1.6.6 Design Backwall

For the backwall design, the design loads are the weight of the backwall, the horizontal surcharge live load, and the horizontal soil pressure acting on the backwall. The wheel loads on the backwall are not considered in this example.

11.1.6.6.1 Calculate Factored Load Effects

The backwall can be easily analyzed as a cantilever beam to calculate factored load effects for different load combinations:

<u>Strength Limit State</u>

There are two load factors for the substructure dead load, 0.9 and 1.25.

The vertical axial load at the backwall can either be:

$$Pb_w = 0.9\ W_3 = 0.9(63.4) = 57.1 \text{ kip, or}$$

$$Pb_w = 1.25\ W_3 = 1.25(63.4) = 79.3 \text{ kip}$$

However, the effect of axial load is negligible in the backwall design.

The horizontal earth pressure, *EH* (shown as H_3), has two load factors, 0.75 and 1.5. The factor for the horizontal live load surcharge, LS_3, is 1.75. Design shear force for the backwall is calculated based on the greatest effect, which is:

$$Vb_w = 1.5\ H_3 + 1.75\ LS_3 = 1.5(51.3) + 1.75(30.4) = 130.2 \text{ kip}$$

Similarly, the governing factored moment for the backwall design is calculated using the upper limits of contributing loads (W_3, H_3, and LS_3). However, W_3 does not produce a bending moment, therefore:

$$M_{bw} = 1.5H_3\left(\frac{h_{bw}}{3}\right) + 1.75LS_3\left(\frac{h_{bw}}{2}\right)$$

$$= (1.5)(51.3)\left(\frac{6.75}{3}\right) + (1.75)(30.4)\left(\frac{6.75}{2}\right) = 352.7 \text{ kip-ft}$$

Service Limit State

The load factors are one for Service Limit State, and only a service moment is needed for the crack control check:

$$M_{service} = (1.00))H_3\left(\frac{h_{bw}}{3}\right) + (1.00)LS_3\left(\frac{h_{bw}}{2}\right)$$

$$= (1.0)(51.3)\left(\frac{6.75}{3}\right) + (1.0)(30.4)\left(\frac{6.75}{2}\right) = 218.0 \text{ kip-ft}$$

11.1.6.6.2 Design for Flexure

The backwall thickness of d_{bw} = 12 in. is used to check the following factored moments acting per unit length of the backwall

$$M_u = \frac{352.7}{62.6} = 5.63 \text{ kip-ft/ft}$$

$$M_{service} = \frac{218.0}{62.6} = 3.48 \text{ kip-ft/ft}$$

Assume:

- Vertical rebar #7 spacing at 12 in.
- Concrete cover = 2 in.
- Bar diameter, d_{bd} = 1.0 in.
- Area of the bar A_s= 0.60 in.^{2}

Factored Flexural Resistance

The factored flexural resistance is calculated in accordance with Articles 5.6.3.2 and 5.6.3.3 as follows:

The area of steel contributing to unit width of the backwall

$$A_s = (0.60)\frac{(12)}{(12)} = 0.60 \text{in.}^2$$

Per article 5.6.2.2, the coefficient, β_1, is taken as 0.85 for f'_c = 3.6 ksi and α_1 is 0.85. Neglecting compression steel, the distance between the neutral axis and the depth of the concrete stress block is obtained:

$$c = \frac{A_s f_y}{0.85 f'_c \beta_1 b} = \frac{(0.60)(60)}{(0.85)(3.6)(0.85)(12)} = 1.153 \text{ in.}$$

$$a = c\beta_1 = (1.153)(0.85) = 0.98 \text{ in.}$$

Distance from the extreme compression fiber to the centroid of nonprestressed tensile reinforcement:

$$d_e = d_{bw} - (\text{concrete cover}) - \frac{d_{bd}}{2} = 12 - 2 - \frac{1.0}{2} = 9.5 \text{ in.}$$

The net tensile strain in the extreme tension steel reinforcement is calculated as follows:

$$\varepsilon_s = \frac{0.003(d_e - c)}{c} = \frac{(0.003)(9.5 - 1.153)}{1.153} = 0.0217$$

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

$$M_r = \phi M_n = \phi(A_s f_y)\left(d_e - \frac{a}{2}\right) = (0.9)(0.60)(60)\left(9.5 - \frac{0.98}{2}\right)$$

$$= 291.92 \text{ kip-in.} = 24.33 \text{ kip-ft} > M_u = 5.63 \text{ kip-ft} \qquad \text{OK}$$

Therefore, the selected number of bars is adequate for strength

Minimum Reinforcement

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

Gross section modulus: $S_c = S_{nc} = \dfrac{(12)(12)^2}{6} = 288 \text{ in.}^3$

Flexural cracking variability factor: γ_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: γ_3 = 0.75 for A706, Grade 60 reinforcement per Article 5.6.3.3. The calculations are as follows:

$$M_{cr} = \gamma_3 \gamma_1 f_r S_c$$
$$= (0.75)(1.6)(0.455)(288) = 157.25 \text{ kip-in} = 13.10 \text{ kip-ft} \qquad \text{(AASHTO 5.6.3.3-1)}$$

$$1.33M_u = 1.33(5.63) = 7.49 \text{ kip-ft}$$

$$M_r = \phi M_n = 24.38 \text{ kip-ft} > \text{smaller of} \begin{Bmatrix} M_{cr} = 13.10 \\ 1.33M_u = 7.49 \end{Bmatrix} = 7.49 \text{ kip-ft} \quad \text{(AASHTO 5.6.3.3)}$$

<u>Crack Control</u>

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

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

Assuming exposure factor γ_e is equal to 1 (class-I exposure), and d_c is equal to 2.5 in.

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)} = 1 + \frac{2.5}{(0.7)(12 - 2.5)} = 1.376$$

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:

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

$$= (33{,}000)(1.0)(0.15)^{1.5}\sqrt{3.6} = 3637 \text{ ksi}$$

$$n = \frac{E_s}{E_c} = \frac{29{,}000}{3{,}637} = 7.97$$

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

$$\rho = \frac{A_s}{bd_e} = \frac{0.60}{(12)(9.5)} = 0.0053$$

$$k = \sqrt{(\rho n)^2 + 2(\rho n)} - \rho n$$
$$= \sqrt{((0.0053)(8))^2 + 2(0.0053)(8)} - (0.0053)(8) = 0.252$$

$$k_{de} = kd_e = (0.252)(9.5) = 2.394 \text{ in.}$$

$$I_{tr} = \frac{bk_{de}^3}{3} + nA_s\left(d_e - k_{de}\right)^2$$

$$= \frac{(12)(2.394)^3}{3} + (8)(0.60)(9.5 - 2.394)^2 = 297.26 \text{ in.}$$

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

$$f_{ss} = n\frac{M_s\left(d_e - k_{de}\right)}{I_{tr}} = (8)\frac{(3.48)(12)(9.5 - 2.394)}{297.26} = 7.99 \text{ ksi -use 8 ksi}$$

The maximum spacing is checked as (Article 5.6.7-1):

$$s = 12 \text{ in.} \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c = \frac{700(1)}{(1.376)(8)} - 2(2.5) = 58.59 \text{ in.} \qquad \text{OK}$$

Therefore, the #7 bar at a spacing of 12 in. is acceptable.

For backwall, b = 12 in. (unit width) and backwall height h = 6.75 ft = 81 inches, the horizontal shrinkage and temperature reinforcement per unit foot width shall satisfy the following equations (Article 5.10.6):

$$A_s \geq \frac{1.3bh}{2(b+h)f_y} = \frac{1.3(12)(81)}{2(12+81)(60)} = 0.113 \text{ in.}^2 \qquad \text{(AASHTO 5.10.6-1)}$$

$$0.11 \text{ in.}^2 \leq A_s \leq 0.6 \text{ in.}^2 \qquad \text{(AASHTO 5.10.6-2)}$$

Assume horizontal temperature bars of #4 @ 12 in. A_s = 0.1963 in.^{2};

$$A_s = 0.1963 \text{ in.}^2 > 0.113 \text{ in.}^2 \qquad \text{OK}$$

$$0.11 \text{ in.}^2 \leq A_s = 0.1963 \text{ in.}^2 \leq 0.6 \text{ in.}^2 \qquad \text{OK}$$

Using temperature bars of #4 @ 12” is acceptable.

11.1.6.6.3 Design for Shear

The shear design for the abutment backwall is usually based on the shear resistance of the concrete alone. The backwall is not heavily reinforced since it is designed to break during a seismic event. The design procedure is the same as the steam wall shown in Section 11.1.6.7.3 and is not repeated herein.

11.1.6.7 Design Stem Wall

11.1.6.7.1 Calculate Factored Load Effects

The weights of backwall, stem wall, wing walls, shear keys, superstructure (DC_{sup}), wearing surface, and utilities load (*DW*), as well as the design truck (*HL*93) and the permit truck (*Permit*), are vertical gravity loads applied to the stem wall. The bearing pad shear load, prestressing load (*PS*), horizontal pressure by live load surcharge, horizontal active soil pressure from the back of stem wall and backwall, and the horizontal passive soil pressure from the fill in front of the stem wall (this passive pressure may be conservatively ignored for short seat abutments however it is considered for tall cantilever abutments) are horizontal loads that are considered in stem wall design.

Similar to the backwall, thirteen permanent loads are considered in the stem wall design, which result in a very high number of possible load combinations. CTAbut is used to identify controlling load combinations in this design example. CTAbut prints out the load factors for controlling combinations to be used for the design of each component of the abutment at the end of the full report. Tables 11.1.6-8 and 11.1.6-9 show the values of load factors for governing cases for stem wall design.

Table 11.1.6-8 Load Factors for Strength and Construction Limit States

Stem wall*	$DC_{Sup.}$	$DC_{Sub.}$	*DW*	*PS*	*Pad shear*[+]	LL_{HL93}	LL_{Permit}	EH_a	LS_h	EH_p	*Comb.*
P_{max}	1.25	1.25	1.50	1.00	1.25	0.00	1.35	1.50	0.00	-	STR2
$V_{max(B)}$	1.25	0.90	1.50	1.00	1.25	1.75	0.00	1.50	1.75	-	STR1
$M_{max(B)}$	1.25	0.90	1.50	1.00	1.25	1.75	0.00	1.50	1.75	-	STR1
$V_{max(F)}$	0.90	1.25	0.65	1.00	-1.25	0.00	1.35	0.75	0.00	-	STR2
$M_{max(F)}$	0.90	1.25	0.65	1.00	-1.25	0.00	1.35	0.75	0.00	-	STR2

*B = Back face of the stem wall in tension; F = Front face of the stem wall in tension

[+]Negative gamma factor for Pad shear has been applied to capture bi-directional force effects

Table 11.1.6-9 Load Factors for Service Limit State

Stem wall	$DC_{Sup.}$	$DC_{Sub.}$	*DW*	*PS*	*Pad shear*	LL_{HL93}	LL_{Permit}	EH_a	LS_h	EH_p	*Comb.*
$M_{max(B)}$	1.0	1.0	1.0	1.0	1.0	1.0	0	1.0	1.0	-	SER1
$M_{max(F)}$	1.0	1.0	1.0	1.0	-1.0	1.0	0	1.0	1.0	-	SER1

All the controlling factored design loads could be calculated by using the tables above. For example, the factored design shear for the stem wall is calculated as:

$$V_{max(B)} = 1.25V_{pad} + 1.5H_2 + 1.75LS_2$$

$$= 1.25(173.2) + 1.5(228.8) + 1.75(64.2) = 672.1\text{kip}$$

Summary of factored loads effects for the stem wall design are given in Table 11.1.6-10.

Table 11.1.6-10 Factored Load Effects for Stem Wall

Strength Limit State	$M_{max(B)}$ (kip-ft)	4356.80
	$M_{max(F)}$ (kip-ft)	1199.47
	$V_{max(B)}$ (kip)	672.1
Service Limit State	$M_{max(B)}$ (kip-ft)	2835.55
	$M_{max(F)}$ (kip-ft)	0.00

Irrespective of the direction, the maximum calculated shear is used for the design of the shear reinforcement in the stem wall. The designer should check both faces (the heel side and the toe side) when designing the shear reinforcement if the concrete cover is different. CTAbut reports some cases as 0.00 if the compression or tension does not affect the section.

11.1.6.7.2 Design for Flexure

An axial force is not usually considered in the flexural design of the stem wall since the effect is often minimal. The effect of the moment acting in the transverse direction (biaxial bending) is also negligible. However, in the case of narrow abutments (i.e., single lane on or off-ramp), Article 5.6.4.5 requirements shall be satisfied.

The design procedure for flexural, crack control, and shrinkage and temperature reinforcement is the same as the backwall and is not repeated here. However, the detailed steps for the shear design are shown in the next section.

11.1.6.7.3 Design for Shear

The shear design for the abutment stem wall and footing follows the General Procedure per Article 5.7.3.4.2 and its CA amendment. The flowchart of the process in CTAbut is shown in figure 11.1.6-6, and the shear design for the stem wall follows.

The following variable would be needed to calculate the shear:

d = 48 in.

β_1 = 0.85

b_v = 62.6(12) = 751.2 in.

#7 @ 12 vertical bars total 64 bars is used.

Hence, A_s = (64)(0.6) = 38.4 in.^{2}

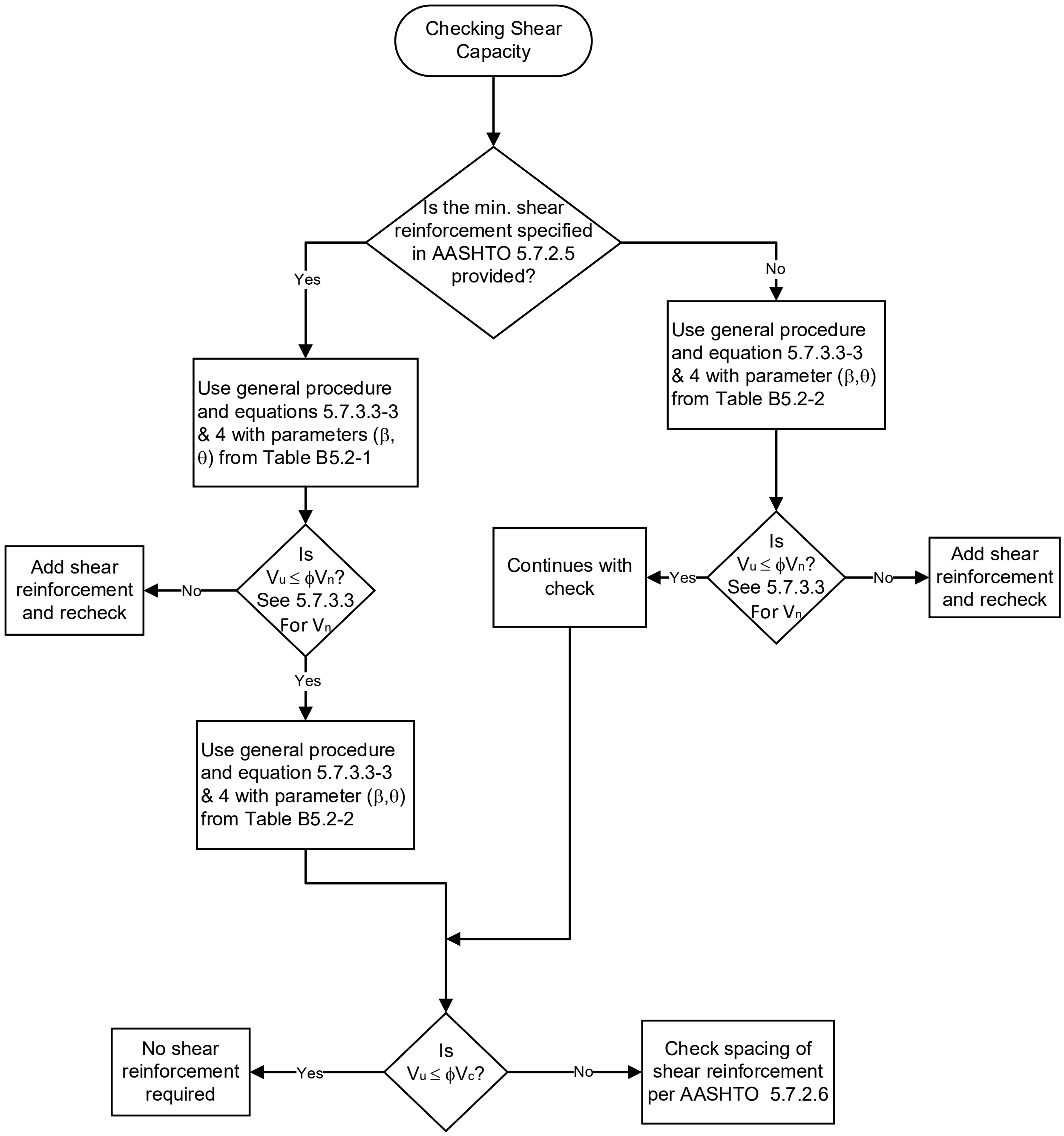

Figure 11.1.6-6 Shear Design Flowchart

$$c = \frac{A_s f_y}{\alpha_1 f_c^{'} \beta_1 b} = \frac{(38.4)(60)}{(0.85)(3.6)(0.85)(751.2)} = 1.18$$

$a = c\beta_1 = (1.18)(0.85) = 1.00$

$$d_e = 48 - \text{concrete cover} - \frac{\text{bar diameter}}{2} = 48 - 2 - \frac{1}{2} = 45.5 \text{ in.}$$

Assume there is no shear reinforcement, $A_v = 0$.

The minimum transverse reinforcement requirement from Article 5.7.2.5-1 is not satisfied.

The effective shear depth:

$$d_v = d_e - \frac{a}{2} = 45.5 - \frac{1}{2} = 45.0$$

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

$0.9\ d_e = 0.9\ (45.5) = 40.95$ 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, $d_v = 45.0$ in.

$$V_c = 0.0316 \beta \lambda \sqrt{f_c^{'}} b_v d_v \qquad \text{(AASHTO 5.7.3.3-3)}$$

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \qquad \text{(AASHTO 5.7.3.3-4)}$$

$\alpha = 90°$ and above equation reduces to

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} \qquad \text{(AASHTO C5.7.3.3-1)}$$

Using Table B5.2-2, θ and β could be obtained.

Since the section contains less than the minimum transverse reinforcement as specified in Article 5.7.2.5, equation B5.2-4 would be used to obtain the ε_x.

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5\left|V_u - V_p\right|\cot\theta - A_{ps} f_{po}\right)}{E_s A_s + E_p A_{ps}} \qquad \text{(AASHTO B5.2-4)}$$

Since there is no prestressing reinforcement, the equation reduces to

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u|\cot\theta\right)}{E_s A_s} \tag{11.1.6.1-1}$$

Per AASHTO CB5.2, 0.5 cot θ could be assumed 1 for the first trial to limit a trial and error process.

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u|\right)}{E_s A_s} \tag{11.1.6.1-2}$$

CTAbut checks both V_{umax} with its associated moment and M_{umax} with its associated shear for each section of the component.

Table 11.1.6.1-2 Stem Wall Back Section Loads to Calculate the Shear Reinforcement

V_{umax} = 672.1 kip	V_{uM} = -4356.80 kip-ft	V_{uN} = - 2148.00 kip
M_{umax} = -4356.80 kip-ft	M_{uV} = 672.1 kip	M_{uN} = - 2148.00 kip

In this example, the associated moment or shear is exceptionally equal to the controlling values.

$$\varepsilon_x = \frac{\left(\frac{|-4356.80|}{3.75} + 0.5(-2148.00) + |672.1|\right)}{(29000)(38.4)} = 0.6824 * 10^{-3}$$

$$s_{xe} = s_x\left(\frac{1.38}{a_g + 0.63}\right) = (3.75)(12)\left(\frac{1.38}{1 + 0.63}\right) = 38.10 \text{ in.} \le 80 \text{ in.} \qquad \text{OK}$$

θ = 53.7 and β =1.66 using AASHTO Table B5.2-2 for $\varepsilon_x \le 1.5$ and $s_{xe} \le 40$.

$$V_c = 0.0316\beta\lambda\sqrt{f_c'}\, b_v d_v = 0.0316(1.66)(1.0)\sqrt{3.6}(751.2)(45.0) = 3364.45 \text{ kip}$$

V_s = 0 since initially, they are no shear reinforcement

$$\text{Lesser of } \begin{cases} \phi V_n = \phi(V_c + V_s) \\ \phi(0.25) f_c' b_v d_v \end{cases}$$

$$\phi V_n = \phi(0.25)f_c^{'} b_v d_v = (0.9)(0.25)(3.6)(751.2)(45.0) = 27381.24 \text{ kip}$$

$$\phi V_n = \phi(V_c + V_s) = 0.9(3364.45 + 0) = 3028.00 \text{ kip (controlled)}$$

$$V_{u\max} = 672.1 \text{ kip} < \phi V_n = 3028.00 \text{ kip}$$

No shear reinforcement is required for the stem wall.

11.1.6.8 Design Pile Foundation

11.1.6.8.1 Select Piles

The standard Class 140 piles are selected in this example, where the diameter of the pile is 14 in. with a batter of 1 to 3. There are two rows of piles with 13 piles in each row which brings the total number of piles to 26. One row of piles is battered. The distances to the center of the pile from footing heel are 2 ft and 8 ft for Rows 1 and 2, respectively.

11.1.6.8.2 Calculate Factored Load Effects

Using controlling load combinations from CTAbut output, shown in Tables 11.1.6-11 and 11.1.6-12, the factored loads for pile group design are calculated. The summary of factored loads is shown in Table 11.1.6-13.

Table 11.1.6-11 Load Factor for Strength Limit State and Construction Combination

Pile Group*	$DC_{Sup.}$	$DC_{Sub.}$	*DW*	*PS*	*Pad shear*	LL_{HL93}	LL_{Permit}	EH_a	LS_h	EH_p	EV_a	EV_p	LS_v	Comb.
Pmax(C)	1.25	1.25	1.50	1.00	1.25	0	1.35	1.50	0	-	1.35	1.35	0	STR2
Pmax(T)	0	0.90	0	0	0	0	0	1.50	0	-	1.00	1.00	0	CON1
FRow(T)	0.90	0.90	0.65	1.00	1.25	1.75	0	1.50	1.75	-	1.00	1.35	0	STR1
FRow(C)	1.25	1.25	1.50	1.00	-1.25	0	1.35	0.75	0	-	1.35	1.00	0	STR2
LRow(T)	-	-	-	-	-	-	-	-	-	-	-	-	-	-
LRow(C)	1.25	1.25	1.50	1.00	1.25	1.75	0	1.50	1.75	-	1.00	1.35	0	STR1

*FRow: First row from Heel, LRow: Last row from Heel, T: Tension, C: Compression

Table 11.1.6-12 Load Factors for Service Limit State

Pile Group	$DC_{Sup.}$	$DC_{Sub.}$	*DW*	*PS*	*Pad shear*	LL_{HL93}	LL_{Permit}	EH_a	LS_h	EH_p	EV_a	EV_p	LS_v	Comb.
Pmax(C)	1.0	1.0	1.0	1.0	1.0	1.0	0	1.0	1.0	-	1.0	1.0	1.0	SER1
Pmax(T)	1.0	1.0	1.0	1.0	1.0	1.0	0	1.0	1.0	-	1.0	1.0	1.0	SER1
LatDC+	1.0	1.0	1.0	1.0	1.0	1.0	0	1.0	1.0	-	1.0	1.0	1.0	SER1
FRow(T)	-	-	-	-	-	-	-	-	-	-	-	-	-	-
FRow(C)	1.0	1.0	1.0	1.0	-1.0	1.0	0	1.0	1.0	-	1.0	1.0	1.0	SER1
LRow(T)	-	-	-	-	-	-	-	-	-	-	-	-	-	-
LRow(C)	1.0	1.0	1.0	1.0	1.0	1.0	0	1.0	1.0	-	1.0	1.0	1.0	SER1

+LatDC: DC ratio of entire pile group lateral resistance capacity

Table 11.1.6-13 – Summary of Factored Load Effects for Pile Group Design

Pile Group		*P* (axial load) kip	*V* (shear) kip	*M* (moment) kip-ft
Strength Limit State	Pmax(C)	3366.65	690.73	2883.45
	Pmax(T)	984.22	474.21	1240.67
	FRow(T)	1836.37	822.85	4634.71
	FRow(C)	3329.01	20.58	-2893.18
	LRow(C)	3012.17	822.85	4334.39
Service Limit State	Pmax(C)	2351.11	564.85	2492.79
	Pmax(T)	2015.16	564.85	2492.79
	LatDC	2015.16	564.85	2492.79
	FRow(C)	2351.11	218.41	-971.61
	LRow(C)	2351.11	564.85	2492.79

11.1.6.8.3 Check Strength Limit State

The resistance of Class 140 standard piles is given in Standard Plan B2-5 as the nominal axial structure resistance of 280 kip for the compression and the nominal axial structure resistance of 140 kip for the tension.

The geotechnical resistance factor (ϕ) is reported by Geotechnical Services on the Foundation Design Recommendations table and is assumed 0.7 for the Strength Limit State. The designer needs to compare the factored load on the pile with the factored nominal axial resistance, which is the geotechnical capacity of the pile under the Strength Limit State. The factored nominal axial resistance (geotechnical) for standard plan piles can be assumed as 0.7 (nominal axial structural resistance), that is 0.7(280) = 196 kip for the compression and 0.7(140) = 98 kip for the tension.

The calculation of the moment of inertia of pile group, I, is shown in table 11.1.6-14.

Where n_p is the number of piles in each row; d is the distance from the face of footing (toe side), and equivalent C_{gpile} is the center of gravity for the pile group from the footing toe.

Table 11.1.6-14 Calculation of Moment of Inertia

	$n_p d$ (ft)	$I = n_p(d - C_{gpile})^2$ (ft^2)
Row 1	13(2) = 26	$13(2 -5)^2$=117
Row 2	13(8) = 104	$13(8-5)^2$ = 117
∑ =	130	234

The center of the gravity of the pile group from the toe of the footing (C_{gpile}) = 130/26 = 5.0 ft which is the center line of footing since the number of piles in each row are the same.

The moment of inertia is evaluated as I = 234 ft^{2,} as shown in Table 11.1.6-14.

The pile reaction force is calculated as follows.

The axial force of any vertical pile is calculated from $P_{pile} = P/n_p \pm Mc/I$, where c is the distance between the centerline of the pile and the center of gravity of the pile group.

As an example, the last row pile reaction (maximum that is used in the design) is calculated as:

$$P_{LRow(C)} = \frac{3012.17}{26} + \frac{4334.39(5-2)}{234} = 171.4 \text{ kip}$$

Since the pile reaction force is less than factored nominal resistance in the compression (186.0 kip), it is acceptable. This check is needed for each load combination and for each row of the piles.

The summary of pile reactions for the strength limit state is shown in Table 11.1.6-15

Table 11.1.6-15 – Summary of Pile Axial Design

	Factored Load (kip)	Factored Resistance (kip)	Factored Load/Factored Resistance ratio	Check
Row 1	165.1	196.0	165.1/196.0= 0.84	Less than 1- OK
Row 2	171.4	186.0	171.4/186.0 =0.92	Less than 1- OK

where: the factored vertical resistance of battered pile = $\frac{(196)(3)}{\sqrt{1^2+3^2}} = 186.0 \text{ kip}$.

The permissible horizontal (lateral) load of the pile group under the service limit state shall also be checked. The permissible horizontal load for a single pile assuming 5 feet of the embedment and zero axial force is 27 kip, and the reduction factor for battered piles is taken as 0.6 for this example.

Permissible horizontal resistance of all piles, L_r

$$= 27(\text{number of vertical piles}) + 27(\text{batter factor})(\text{number of battered piles})$$

$$= 27(13) + 27(0.6)(13) = 561.6 \text{ kip}$$

Controlling Service Limit State (LatDC) pile group:

P = 2015.16 kip and M = 2492.79 kip-ft

$$P_{FRow(C)} = \frac{2015.16}{26} + \frac{2492.79(5-2)}{234} = 109.47 \text{ kip}$$

Horizontal reaction force of a batter pile $= \frac{109.47}{3} = 36.49 \text{ kip}$

Horizontal reaction force of all battered piles, F_{pile} = (36.49)(13) = 474.30 kip

Total Maximum Lateral load under Service Limit State, F_x = 564.85 kip

Required horizontal load = Total maximum lateral load – horizontal reaction force of all batter piles = 564.85 – 474.30 = 90.55 kip

The permissible horizontal load (561.6 kip) is greater than required horizontal load (90.55 kip). Therefore, it is acceptable. The designer also needs to check the Construction II combination; however, that combination usually does not govern.

Note - This example is to show the designer the use of batter piles. The demand/capacity for this example is extremally low, perhaps, the pile may not need to be battered for this example.

11.1.6.8.4 Communicate with Geotechnical Designer

Under the Service I Limit State, both total and the permanent support loads (calculated as net) are reported to GD:

Total Load (net) = Maximum load under Service-I – the weight of the overburden soil

$$\begin{aligned}\text{Total Load}(\text{net}) &= 2351.11 - (OG - BOF)L_{ftg}B_{ftg}\gamma_s \\ &= 2351.11 - (6.5-0)(64)(10)(0.12) = 1851.91\text{ kip}\end{aligned}$$

The permanent load (net) = Total Load – Live Load – Live Load Surcharge (if any)

The permanent load (net) = 1851.91 - 336.0 - 38.4 = 1477.5 kip. Under the Strength Limit State, the maximum force per support, the minimum force per support, also the maximum compression, and the tension force per pile are reported:

The maximum force per support = 3366.65 kip

The maximum compression load per pile = 171.4 kip

There is no tension in a pile for this example. The General Foundation information to be sent from SD to GD is shown in Table 11.1.6-16.

Table 11.1.6-16a Information to be Provided to GD

Foundation Design Data Sheet							
Support No.	Pile Type	Finished Grade Elevation (ft)	Cut-off Elevation (ft)	Pile Cap Size (ft)		Permissible Settlement Under Service Load (in)	Number of Piles per Support
				B	*L*		
Abut 1	Class 140	16.75	0.5	10	64	1”	26
Bent 2							
Bent 3							
Abut 4	Class 140	16.75	0.5	10	64	1”	26

Table 11.1.6-16b Loads to be Provided to GD

Foundation Factored Design Loads											
Support No.	Service –I Limit State		Strength Limit State (Controlling Group, kip)				Extreme Event Limit State (Controlling Group, kip)				
	Total Load per Support	Permanent Loads per Support	Compression		Tension		Compression		Tension		
			Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile	
Abut 1	1852	1478	3367	171	0	0	N/A	N/A	N/A	N/A	
Bent 2											
Bent 3											
Abut 4	1852	1478	3367	171	0	0	N/A	N/A	N/A	N/A	

Note -Since this design example is for abutment design, information on bents is not shown.

11.1.6.8.5 Design Pile Cap

Tables 11.1.6-17 and 11.1.6-18 summarize the load factors for the controlling load combinations for the design of the pile cap.

Table 11.1.6-17 Load Factors for Strength Limit State

Pile Cap Sections	$DC_{Sup.}$	$DC_{Sub.}$	*DW*	*PS*	*Pad shear*	LL_{HL93}	LL_{Permit}	EH_a	LS_h	EH_p	EV_a	EV_p	LS_v	*Comb.*
MTopHel	0.90	0.90	0.65	1.00	1.25	1.75	0	1.50	1.75	-	1.35	1.35	1.75	STR1
VTopHel	0.90	0.90	0.65	1.00	1.25	1.75	0	1.50	1.75	-	1.00	1.35	0	STR1
MBotHel	1.25	1.25	1.50	1.00	-1.25	0	1.35	0.75	0	-	1.00	1.00	0	STR2
VBotHel	1.25	1.25	1.50	1.00	-1.25	0	1.35	0.75	0	-	1.35	1.00	0	STR2
MTopToe	0	0.90	0	0	0	0	0	0.75	0	-	1.35	1.35	0	CON1
VTopToe	0	0.90	0	0	0	0	0	0.75	0	-	1.35	1.00	0	CON1
MBotToe	1.25	1.25	1.50	1.00	1.25	1.75	0	1.50	1.75	-	1.00	1.00	0	STR1
VBotToe	1.25	1.25	1.50	1.00	1.25	1.75	0	1.50	1.75	-	1.00	1.35	0	STR1

Table 11.1.6-18 Load Factors for Service Limit State

Pile Cap Sections	$DC_{Sup.}$	$DC_{Sub.}$	*DW*	*PS*	*Pad shear*	LL_{HL93}	LL_{Permit}	EH_a	LS_h	EH_p	EV_a	EV_p	LS_v	*Comb.*
MTopHel	1.00	1.00	1.00	1.00	1.00	1.00	0	1.00	1.00	-	1.00	1.00	1.00	SER1
MBotHel	1.00	1.00	1.00	1.00	-1.00	1.00	0	1.00	1.00	-	1.00	1.00	1.00	SER1
MTopToe	1.00	1.00	1.00	1.00	-1.00	1.00	0	1.00	1.00	-	1.00	1.00	1.00	SER1
MBotToe	1.00	1.00	1.00	1.00	1.00	1.00	0	1.00	1.00	-	1.00	1.00	1.00	SER1

Using the information shown in Tables 11.1.6-17 and 11.1.6-18, the factored loads for the pile cap are calculated. The summary of factored loads shown in Table 11.1.6-19 is used for the top and bottom reinforcement design as well as the shear design.

Table 11.1.6-19 Summary of Factored Loads

Pile Cap Sections		*P* (axial load) kip	*V* (shear) kip	*M* (moment) kip-ft
Strength Limit State	MTopHel	1996.39	822.85	4038.25
	VTopHel	1836.37	822.85	4634.71
	MBotHel	3236.19	20.58	-2548.72
	VBotHel	3329.01	20.58	-2893.18
	MTopToe	1114.67	237.10	-305.31
	VTopToe	1077.04	237.10	-427.61
	MBotToe	2974.53	822.85	4212.08
	VBotToe	3012.17	822.85	4334.39
Service Limit State	MTopHel	2015.16	564.85	2492.79
	MBotHel	2351.11	218.41	-971.61
	MTopToe	2015.16	218.41	-971.61
	MBotToe	2351.11	564.85	2492.79

The pile cap is designed for shear forces and bending moments calculated on the toe and heel sides of the stem. The shear force is conservatively calculated at the face of the stem rather than at a distance equal to the depth of the footing. For example, the shear force at the heel side is calculated by reducing forces of the piles that are partially located in the free body diagram, as well as considering other factored forces:

$$
\begin{aligned}
V_{heel} &= \frac{(\text{pile reaction})(\text{number of piles})(\text{effective fraction of pile reaction})}{\text{footing length}} \\
&\quad - \frac{(\text{load factor})V_1}{\text{footing length}} - \frac{(\text{load factor})V_3}{\text{footing length}} - \frac{(\text{load factor})LS_{Vertical}}{\text{footing length}} \\
&\quad - \frac{(\text{load factor})W_1(\text{heel width})/(\text{footing width})}{\text{footing length}} \\
&= \frac{(165.13)(13)(0.928)}{64} - \frac{(1.35)(259.1)}{64} - \frac{(1.35)(6.1)}{64} - \frac{(0)(38.4)}{64} - \frac{(1.25)(240)(2.5)/(10)}{64} \\
&= 24.36 \text{ kip/ft}
\end{aligned}
$$

Pile reaction forces may be fractional depending on the pile's location with respect to the heel's face. In this case, the center line of the second row of piles is located 8 ft from the toe edge of the footing. The fraction of the pile reaction that contributes to the shear at the heel is approximated as:

$$
\begin{aligned}
\text{fraction} &\approx \frac{(\text{heel width}) - (\text{footing width-location of pile - pile diameter/2})}{\text{pile diameter}} \\
&\approx \frac{2.5 - (10 - 8 - 1.167/2)}{1.167} = 0.928
\end{aligned}
$$

The summary of the cap design forces per linear foot at the face of the stem (the toe side and the heel side) are shown in Table 11.1.6-20.

Table 11.1.6-20 Summary of Cap Design Forces per Liner Foot at the Face of Stem (Toe Side and Heel Side)

Limit State	Forces	Top of Toe	Bottom of Toe	Top of Heel	Bottom of Heel
Strength	Shear at face (kip/ft)	4.46	30.91	2.87	24.36
	Moment at face (kip-ft/ft)	0.00	45.50	6.60	9.48
Service	Moment at face (kip-ft/ft)	0.00	32.05	2.31	3.51

The steps for flexural design, shear design, crack control, and horizontal temperature reinforcement of the pile cap are similar to the backwall or the stem wall.

11.1.6.9 Design Spread Footing

The backwall and stem wall design for the spread footing is the same as for the pile foundation. The next portion of this example concentrates on the design of the same abutment supported on a shallow foundation.

Table 11.1.6-21 provides the nominal bearing resistance (q_n) and permissible net contact stress (q_{pn}) provided by GD based on the effective size of the footing. The contact stress under the Service-I load combination is compared to q_{pn}, and the bearing stresses under Strength and Construction factored loads are compared to q_R to meet design requirements, where $q_R = \phi_b \, q_n$.

Table 11.1.6-21 Nominal Bearing Resistance and Permissible Net Contact Stress

B' (ft)	Nominal bearing Resistance-q_n (ksf)	Permissible net Stress-q_{pn} (ksf)
4.0	26.96	17.8
5.0	29.12	16.3
6.0	31.28	14.8
7.0	33.44	13.3
8.0	35.57	12.2
9.0	37.67	11.3
10.0	39.72	10.5
11.0	41.73	9.9

Using governing load combinations and load factors reported by the CTAbut program (not shown here), Table 11.1.6-22a and 11.1.6-22b summarize governing factored loads for soil and structural checks, respectively of the abutment design.

Table 11.1.6-22a Summary of Governing Factored Loads for soil checks

Limit State	Check	P_u (kip)	V_u (kip)	M_u (kip-ft)
Strength/Construction	Bearing	1836.37	822.85	4634.71
	Sliding	984.22	474.21	1240.67
Service	Settlement	2351.11	564.85	2492.79
	Eccentricity	2015.16	564.85	2492.79

Table 11.1.6-22b Summary of Factored Loads for structural check

Footing Sections		*P* (axial load) kip	*V* (shear) kip	*M* (moment) kip-ft
Strength Limit State	MTopHel	1996.39	822.85	4038.25
	VTopHel	1996.39	822.85	4038.25
	MBotHel	3236.19	20.58	2548.73
	VBotHel	3236.19	20.58	2548.73
	MTopToe	-	-	-
	VTopToe	-	-	-
	MBotToe	2974.53	822.85	4212.08
	VBotToe	2974.53	822.85	4212.08
Service Limit State	MTopHel	2015.16	564.85	2492.79
	MBotHel	2351.11	218.41	971.61
	MTopToe	-	-	-
	MBotToe	2351.11	564.85	2492.79

11.1.6.9.1 Check Bearing Stresses

The first check for Strength/Construction load combinations is a bearing stress check. The governing load combination is used to check the soil's bearing resistance and the footing's size. Using absolute values of the moment and the axial force, the eccentricity, effective footing width, and effective area are calculated as:

$$e = \frac{|M_u|}{|P_g|} = \frac{4634.71}{1836.37} = 2.52\,\text{ft}$$

$$B' = 10 - 2(2.52) = 4.95\,\text{ft}$$

$$A_e = (64)(4.95) = 316.95\,\text{ft}^2$$

The ultimate bearing stress is calculated based on a uniform stress distribution as the footing is on soil:

$$q_{g,u} = \frac{1836.37}{316.95} = 5.79\,\text{ksf}$$

The nominal bearing resistance q_n is calculated from Table 11.1.6-21 using

B' = 4.95 ft and interpolation between 26.96 ksf and 29.12 ksf as:

q_n = 29.02 ksf

$q_R = \phi_b\, q_n$ = (0.45)(29.02) =13.06 ksf (AASHTO 10.6.3.1.1-1)

As $q_R > q_{g,u}$, the bearing stress is acceptable.

11.1.6.9.2 Check Sliding

The second check for Strength/Construction load combinations is the sliding check. Ignoring the backfill passive resistance, the factored sliding resistance is obtained by Article 10.6.3.4 as:

$$R_R = \phi R_n = \phi_\tau R_\tau = 0.8P\mu$$

where P is the total vertical force, ϕ_τ is the resistance factor for sliding between soil and foundation (AASHTO Table 10.5.5.2.2-1), and μ = tan (ϕ_f) where ϕ_f is the internal friction angle of drained soil.

$$R_R = 0.8(984.22)\tan(34^o) = 531.1 \text{ kip}$$

Comparing the shear force effect of the footing to the factored shear resistance:

$$V_u = 474.21 \text{ kip} < R_R = 531.1 \text{ kip}$$

Shear keys are not required herein to resist the sliding. In case shear keys are required, the advantages and disadvantages of using shear keys should be considered in the design. CTAbut provides the additional required shear force to design the shear key under the footing in the full report only.

11.1.6.9.3 Check Settlements

The first check under the Service-I load combination is to compare the net uniform bearing stress ($q_{n,u}$) to the permissible net contact stress (q_{pn}) to limit the settlement to the permissible level. The axial load should be used as the net when calculating the bearing stress for the settlement check. Using absolute values of the moment and the axial force, the eccentricity, effective footing width, and effective area are calculated as:

$$e = \frac{|M_n|}{|P_g|} = \frac{2492.79}{2351.11} = 1.06 \text{ ft}$$

$$B' = 10 - 2(1.06) = 7.88 \text{ ft}$$

$$A_e = (64)(7.88) = 504.32 \text{ ft}^2$$

$$q_{g,u} = \frac{2351.11}{504.32} = 4.66 \text{ ksf}$$

$$q_{n,u} = \left(q_{g,u} - \left(Average\,OG - bottom\,of\,footing\right)\right)\gamma_{SE}$$

$$q_{n,u} = \left(4.66 - (6.5 - 0) * \frac{120}{1000}\right)1.00 = 3.88 \text{ ksf}$$

The permissible net contact stress q_{pn} is calculated from Table 11.1.6-21 using B' = 7.88 ft and interpolation between13.3 ksf and 12.2 ksf as:

q_{pn} = 12.33 ksf

As $q_{pn} > q_{n,u}$, the contact bearing stress is acceptable.

11.1.6.9.4 Check Eccentricity

The second check under the Service-I load combination is the eccentricity check. The gross axial force is used for this check. Therefore, the eccentricity is calculated as:

$$e = \frac{|M_n|}{|P_g|} = \frac{2492.79}{2015.16} = 1.24\,\text{ft}$$

According to Article 10.5.2.2 (AASHTO-CA BDS-8), the maximum acceptable eccentricity limit for footing on soil is:

$B_{ftg}/6$ = 10/6 = 1.67 ft

The calculated eccentricity is less than the specified limit and is acceptable.

Note: If the footing is on the rock, the maximum eccentricity limit is $B_{ftg}/4$.

11.1.6.9.5 Communicate with GD

Table 11.1.6-23 summarizes design loads to be provided by SD to GD during design.

Table 11.1.6-23a Design Loads to be provided by SD to GD

Service Limit State		M_x (kip-ft)		V_y (kip)		*P (gross)* (kip)	
		M_{x_total}	M_{x_perm}	V_{y_total}	V_{y_perm}	P_{total}	P_{perm}
Eccentricity	M_{x_max}	971.61	272.32	218.41	316.14	2351.11	1976.76
	M_{x_min}	2492.79	272.32	564.85	316.14	2351.11	1976.76
	P_{grs_min}	2492.79	272.32	564.85	316.14	2015.16	1976.76
	Controlling Load	2492.79	272.32	564.85	316.14	2015.16	1976.76
Settlement	M_{x_max}	971.61	272.32	218.41	316.14	2351.11	1976.76
	M_{x_min}	2492.79	272.32	564.85	316.14	2351.11	1976.76
	P_{net_max}	2492.79	272.32	564.85	316.14	2351.11	1976.76
	Controlling Load	2492.79	272.32	564.85	316.14	2351.11	1976.76

Note – CTAbut reports P_{gross} in the soil check table. However, the settlement check calculation must use P_{net}.

Table 11.1.6-23b Design Loads to be provided by SD to GD

Strength/Construction Limit States		M_x (kip-ft)	V_y (kip)	P_{total}*(gross)* (kip)
Bearing	M_{x_max}	2893.18	20.58	3329.01
	M_{x_min}	4634.71	822.85	2774.36
	P_{total_max}	2883.45	690.73	3366.65
	Controlling Load	4634.71	822.85	1836.37
Sliding	V_{y_max}	4634.71	822.85	2774.36
	V_{y_min}	1072.58	0.00	2013.89
	P_{total_min}	1240.67	474.21	984.22
	Controlling Load	1240.67	474.21	984.22

11.1.6.9.6 Design Strength

In order to calculate the internal forces of the footing, as shown in Figure 11.1.6-7, the soil pressures, q_{left} (heel edge), q_{right} (toe edge), q_1 (at the face of the heel), and q_2 (at the face of the toe) are calculated. The following symbols are used:

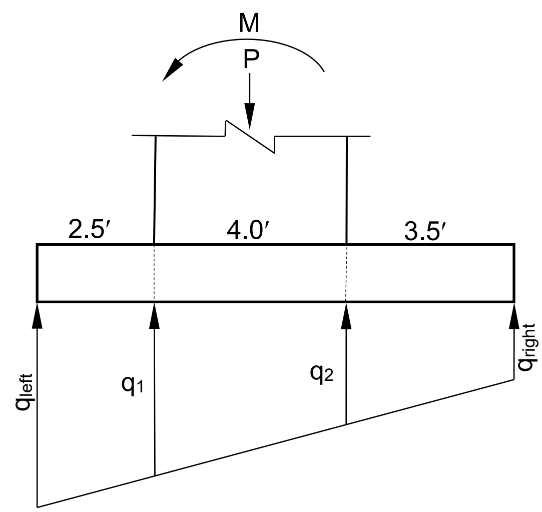

Figure 11.1.6-7 Soil Pressures

L_{ftg} = footing length (ft)

M_{heel} = moment at the face of the heel (kip-ft/ft)

M_{heel_soil} = moment due to the soil pressure at the face of the heel (kip-ft/ft)

M_{toe} = moment at the face of the toe (kip-ft/ft)

M_{toe_soil} = moment due to the soil pressure at the face of the toe (kip-ft/ft)

V_{heel} = shear at the face of the heel (kip/ft)

V_{heel_soil} = shear due to the soil pressure at the face of the heel (kip/ft)

V_{toe} = shear at the face of the toe (kip/ft)

V_{toe_soil} = shear due to the soil pressure at the face of the toe (kip/ft)

B_{ftg} = footing width (ft)

W_{heel} = heel width (ft)

W_{toe} = toe length (ft)

The q_{left} and q_{right} are calculated by the following equation

$$\begin{Bmatrix} q_{left} \\ q_{right} \end{Bmatrix} = \frac{P}{B_{ftg}L_{ftg}} \pm \frac{M}{L_{ftg}B_{ftg}^2 / 6}$$

For example, for the case shown in Table 11.1.6-22b, the “MBotHel” forces are given as: P = 3236.19 kip, and M = 2548.73 kip-ft.

Following is a summary of soil pressures calculations:

$$q_{left} = \frac{3236.19}{(10)(64)} + \frac{2548.73}{\frac{(64)(10)^2}{6}} = 7.45\,\text{ksf}$$

$$q_{right} = \frac{3236.19}{(10)(64)} - \frac{2548.73}{\frac{(64)(10)^2}{6}} = 2.67\,\text{ksf}$$

$$q_1 = q_{left} - \frac{\left(q_{left} - q_{right}\right)W_{heel}}{W_{ftg}} = 7.45 - \frac{(7.45 - 2.67)(2.5)}{10} = 6.25\,\text{ksf}$$

$$q_2 = q_{right} + \frac{\left(q_{left} - q_{right}\right)W_{toe}}{W_{ftg}} = 2.67 + \frac{(7.45 - 2.67)(3.5)}{10} = 4.34\,\text{ksf}$$

Therefore, forces caused by the soil pressure are calculated as follows:

$$V_{heel_{soil}} = \frac{\left(q_{left} + q_1\right)W_{heel}}{2} = \frac{(7.45 + 6.25)(2.5)}{2} = 17.13 \text{ kip/ft}$$

$$M_{heel_{soil}} = \frac{q_1 W_{heel}^2}{2} + \frac{\left(q_{left} - q_1\right)W_{heel}^2}{3}$$

$$= \frac{(6.25)(2.5)^2}{2} + \frac{(7.45 - 6.25)(2.5)^2}{3} = 22.03 \text{ kip-ft/ft}$$

For the structure design of the footing, the forces from the overburden soil and footing need to be subtracted from the forces caused by the soil pressure, calculated above.

$$V_{heel} = V_{heel\_soil} - \frac{(LF)V_1 + (LF)V_3 + (LF)LS_{Vertical} + (LF)W_1 W_{heel} / B_{ftg}}{L_{ftg}}$$

$$= 17.13 - \frac{(1.0)(259.1) + (1.0)(6.1) + (0)(38.4) + (1.25)(240)\left(\frac{2.5}{10}\right)}{64} = 11.81 \text{ kip/ft}$$

$$M_{heel} = M_{heel\_soil} - \frac{(LF)V_1(MA) + (LF)V_3(MA) + (LF)LS_{Vertical}(MA) + (LF)W_1(MA)W_{heel} / B_{ftg}}{L_{ftg}}$$

$$= 22.03 - \frac{(1.0)(259.1)(1.25) + (1.0)(6.1)(1.25) + (0)(38.4)(1.25) + (1.25)(240)(1.25)\left(\frac{2.5}{10}\right)}{64} = 15.38 \text{ kip/ft}$$

LF: Load Factor; *MA*: Moment Arm

After repeating the calculation for other sections of the footing a summary of the shear and flexural footing design load effects was generated and shown in Table 11.1.6-24.

Table 11.1.6-24 Summary of the shear and flexural footing design loads

Limit State	Forces	Top Toe	Bottom Toe	Top Heel	Bottom Heel
Strength	Shear at face (kip/ft)	0.00	21.93	6.69	11.81
	Moment at face (kip-ft/ft)	0.00	41.20	8.98	15.38
Service	Moment at face (kip-ft/ft)	0.00	28.24	3.35	6.75

The flexural and shear design steps for the footing are the same as for the pile foundation or stem wall design. Therefore, it is not shown here.

NOTATION

A_e	=	effective shear area of a cross-section (ft^2)
A_s	=	total area of non-prestressed tension reinforcement ($in.^2$)
a	=	depth of equivalent rectangular stress block (in.)
an	=	effective live load distribution width at the deck elevation (ft)
B'	=	effective footing width (ft)
BOF	=	bottom of footing elevation (ft)
B_{ftg}	=	tooting width (ft)
b_n	=	effective live load distribution width at the top of the footing (ft)
b_v	=	effective width of a member for shear stress calculations (in.)
C	=	correction factor for concrete-soil interference
C_{gpile}	=	center of gravity for pile group (ft)
c	=	distance from the extreme compression fiber to the neutral axis (in)
DC_{sup}	=	dead load from superstructure (kip)
DC_{sub}	=	dead load from substructure (kip)
DW	=	additional dead load from superstructure (kip)
d	=	distance from the face of footing to center of the pile (ft)
d_{bd}	=	deformed bar diameter (in.)
d_{bw}	=	backwall thickness (in. or ft)
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_{ftg}	=	footing thickness (ft)
d_{LS}	=	depth of live load surcharge on heel (ft)
d_v	=	effective shear depth (in.)
EH	=	horizontal earth pressure (kip)
LS_H	=	horizontal live load surcharge (kip)
E_c	=	modulus of elasticity of concrete (ksi)
E_s	=	modulus of elasticity of reinforcing steel (ksi)
e	=	eccentricity (ft)
f'_c	=	specified 28-day compressive strength of unconfined concrete (ksi)
f_r	=	modulus of rupture of concrete (ksi)

F_{pile} = horizontal reaction force of all batter piles (kip)

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

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

F_x = total maximum lateral load (kip)

h = abutment height (ft)

h_{bw} = backwall height (ft)

I = moment of inertia

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

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

K_a = Coulomb's active earth-pressure coefficient

K_p = Coulomb's passive earth-pressure coefficient

k = ratio for transformed section

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

L_{ftg} = footing length (ft)

LL_{HL93} = design vehicular live load- HL-93 load (kip)

LL_{permit} = permit vehicular live load (kip)

L_r = permissible horizontal resistance of all piles (kip)

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)

MPF = multiple presence factor

N = equivalent number of lanes

n = modular ratio

n_l = number of whole lanes that can be accommodated on the bridge

n_p = number of pile in each row

n_{max} = maximum number of design lanes that can be placed on the bridge

OG = original ground elevation (ft)

P = total vertical force (kip)

PS = prestressing force at abutment (kip)

P_{gross} = factored axial force (kip)

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

P_{Pile} = axial force of vertical pile (kip)

P_u = factored axial force (kip)

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

q_n = nominal bearing resistance (ksf)

$q_{n,u}$ = net uniform bearing stress (ksf)

q_{pn} = permissible net stress (ksf)

q_R = factored bearing resistance (ksf)

R_τ = nominal sliding resistance against failure by sliding (kip)

R_R = factored resistance force against failure by sliding (kip)

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

s = spacing of reinforcing bars (in.)

V_c = nominal shear strength provided by concrete (kip)

V_n = nominal shear strength of a section (kip)

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

V_{pad} = bearing pad shear (kip)

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

W = abutment length along the skew (ft)

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

β_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

β_s = ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face

ε_s = strain in the centroid of the tension reinforcement (in/in)

ϕ = angle of internal friction; strength reduction factor

ϕ_b = resistance factor for bearing of shallow foundation

ϕ_f = internal friction angle

$\phi\tau$ = resistance factor for shear resistance between soil and foundation specified in Table 10.5.5.2.2-1

γ_1 = flexural cracking variability factor (AASHTO 5.6.3.3)

γ_3	=	ratio of specified minimum yield strength to ultimate tensile strength of the non-prestressed reinforcement (AASHTO 5.6.3.3)
γ_c	=	weight of the concrete per unit volume (pcf)
g_s	=	weight of the soil per unit volume (pcf)
γ_e	=	crack control exposure factor (AASHTO 5.6.7)
θ	=	angle of the load distribution (degree)
θ_{sk}	=	skew angle
ρ	=	ratio of 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,
3. CA.Caltrans. (2019b). *Caltrans Seismic Design Criteria,* Version 2.0, California Department of Transportation, Sacramento, CA.
4. Caltrans. (2019c). *CTBridge Program,* Version 1.8.3 California Department of Transportation, Sacramento, CA.
5. Caltrans. (2020). *Bridge Design Details, Chapter 6,* California Department of Transportation, Sacramento, CA.
6. Caltrans. (2022). CT- Abut program, Version 3.1.0, California Department of Transportation, Sacramento, CA.
7. Zokaie, T., Malek, A., and Rahbari, A. (2015). "Live Load Distribution on Bridge Abutments," *Western Bridge Engineers Seminar*, September 9-11, Reno, NV.